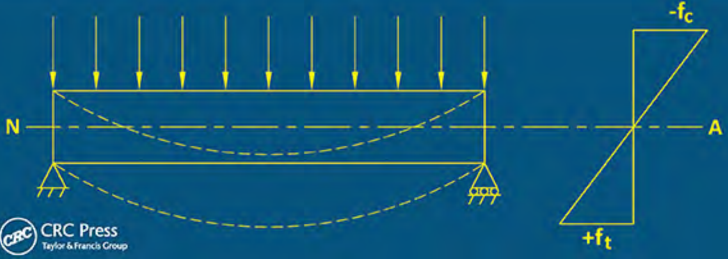
Syed Mehdi Ashraf, P.E.

Practical Design of Reinforced Concrete Buildings



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Thank you, Mom and Dad

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Preface

The goal of this book is to provide a practical, code-based approach to the structural design of reinforced concrete building elements. The book is based upon the ACI concrete code (ACI 318-14—Building Code Requirements for Structural Concrete and Commentary) published by the American Concrete Institute. The ACI 318-14 is consistently referred to in this book as "code." The book is divided into three sections. The first section (Chapters 1 through 5) deals with understanding the basic concepts of forces that could act on structural elements—bending, shear, axial forces, axial forces combined with bending, diagonal tension, shear friction, and torsion. The code requirements are explained, and discussions of materials used in reinforced concrete are provided. Procedures to calculate design loads and determine the load path of buildings are explained. The second section (Chapters 6 through 12) deals with the design, detailing, inspection, construction, and testing of various building structural concrete elements—slabs, beams, columns, footings, and walls. The third section (Chapters 13 through 15) provides discussions on various types of buildings in the United States, a detailed explanation of the calculations of wind loads acting on buildings, and an introduction to the philosophy of engineering and critical thinking.

Floor plans of two types of buildings are provided in Chapter 1, and the calculations of various elements of these buildings are manually performed to illustrate the design of the buildings. The design examples are included in the appendices of the book.

Chapter 1—"Main Design Concepts"—deals with a variety of concepts beginning with a brief overview of the history of concrete. The basic mechanism of reinforced concrete, the types of loads that can act on reinforced concrete elements, and the strength and serviceability requirements are discussed. Models of two types of multistoried buildings in the form of drawings are provided. Both buildings are ten storied—one with beams and columns and the other with flat plates supported on columns and shear walls. The concepts of different structural elements of these two buildings, such as one-way slabs, two-way slabs supported on beams, flat slabs, flat plates, columns, shear walls, and various types of footings, are introduced. What types of forces can act on these elements? Theories of calculations for each type of force is explained—bending, shear, shear friction, compression, tension, torsion, and axial forces combined with bending are examined to prepare the student to understand the code requirements and design methodology. The concept of load path of a building structure is explained. A general arrangement of the code is discussed. There was a major change in the arrangement of the ACI 318 during its 2014 edition as compared to the previous editions. It is advised that professors and students spend a great deal of time on this chapter to understand the subsequent chapters on the code and the design.

Chapter 2—"Properties of Materials Used in Reinforced Concrete"—provides discussions on the standards and properties of various materials used in reinforced concrete—cement, aggregates, steel, water, and admixtures. Concrete mix design procedures are demonstrated.

Chapter 3—"Design Loads"—is based on ASCE 7-10 "Minimum Design Loads for Buildings and Other Structures" published by the American Society of Civil Engineers and IBC (2015)—International Building Code—published by the International Code Council. The seven load combinations and procedures to calculate the dead load, floor live load, roof live load, snow load, and rain load are illustrated. Earthquake loads are not addressed in this edition of the book. Wind loads are dealt with separately in Chapter 14. Load calculations for various structural elements of buildings are demonstrated.

Chapter 4—"ACI Strength Requirements"—deals with strength requirements of the code for the structural elements to resist bending, shear, axial forces, torsion, shear friction, and axial forces combined with bending.

Chapter 5—"Other ACI Code Requirements"—discusses the structural systems and the structural analysis methodologies permitted by the code along with strength and serviceability requirements. Topics include deflection, cracking, durability, sustainability, integrity, fire resistance, embedments, connection design, and strength reduction factors. The design properties of concrete and steel are also discussed. This book uses normal weight concrete and steel with a yield strength of 60,000 psi.

Chapter 6—"Slabs"—includes the design of one-way slabs with beams, simply supported and continuous slabs, two-way slabs with beams, two-way flat plates, and flat slabs. The calculations of the required strength, design strength, flexure, and shear reinforcement along with their detailing are demonstrated. Two design methods for two-way slabs—direct design method and equivalent frame method—are used in the design calculations of the two-way slabs.

Chapter 7—"Beams"—includes the design for flexure, shear, and torsion of rectangular beams, T beams, L beams, singly and doubly reinforced beams, and deep beams. The required strength, design strength, and reinforcement requirements of the codes are discussed.

Chapter 8—"Columns"—deals with the design of short and slender columns for axial loads, and axial loads combined with bending. The first-order analysis and P–Δ effects on columns are illustrated.

Chapter 9—"Walls"—introduces the student to gravity and lateral load distribution on reinforced concrete walls along with the code requirements for strength and reinforcement.

Chapter 10—“Foundations”—deals with various types of foundation designs and the situations where each type of foundation is adopted. Foundations include pad (spread) footings, continuous wall footings, combined footings, strapped footings, pile caps, and mat foundations.

Chapter 11—“Reinforcement Details”—discusses the code requirements for minimum spacing of reinforcement, hooks, splices, and development length of steel bars in reinforced concrete design.

Chapter 12—“Drawings, Inspections, and Testing”—provides students an opportunity to understand what to expect when they assume working for a design office or at a construction site. It includes the author’s hands-on experience in the preparation of structural drawings, inspection, testing, and construction of concrete elements. It includes how an engineer should prepare reports while performing inspection-like duties.

Chapter 13—“Various Types of Buildings”—introduces students to various types of buildings constructed in the United States—wood, steel, concrete, precast concrete, and masonry.

Chapter 14—“Wind Load Analysis of Buildings”—is a bonus chapter that discusses in detail the concept of wind load in accordance with ASCE 7-10 (Minimum Design Loads for Buildings and Other Structures).

Chapter 15—“Engineering in Popper’s Three Worlds”—introduces engineers to the philosophy of engineering and how to think critically. After the author attended the graduate school in humanities and social sciences at the University of Middlesex, he experienced a vast difference in his approach to structural engineering. Philosophers have not paid sufficient attention to engineering and vice versa. Exposure to philosophy would enable engineers to understand the criticism to engineering and ethics of the profession and to better their understanding of engineering.

Problems for Chapters 1 through 10 are included in the appendices. The structural calculations are performed on engineering graph sheets, simulating the actual way structural design is performed. Manual calculations are included to provide a human touch of engineering practice.

When I joined Gopman Consulting Engineers in 2006, there was a note stuck to the board behind Herb’s desk, which read:

> What a good engineer can do on the back of an envelope, tons and tons of software print out cannot.

Herb Gopman is one of most accomplished structural engineers of South Florida and actually started designing high-rise buildings with a slide rule.

It is my humble request to professors and students to build the knowledge of how structures behave rather than getting entangled into computer software, which takes away from the thinking as a structural engineer. Definitely computer software is a boon, but only after an engineer masters the physical understanding of structures and the code requirements.

My father, Dr. S.M. Ashraf, a chemical engineer, was the first person who influenced me during an interaction with him after my grandfather’s death, giving me insight of grasping mathematical concepts. That changed me. At high school (Little Flower High School, Hyderabad), I was mentored by my math teacher, Mr. C.R. Murthy, in learning theorems and solving riders, which made me very fast in solving mathematical problems. Then at the National Institute of Technology, Warangal, Professors N.K. Adimurthy and Kamasundar Rao provided me a solid foundation for structural engineering. I started training with Wali Quadri & Associates in the design of multistoried buildings when I was in my sophomore year at the National Institute of Technology. My cousin, Siraj Hasan, who is one of the leading architects in India, provided me with opportunities to work on some challenging design projects in India. Professor Ali Paya, my humanities professor at the University of Middlesex, taught me how to think critically, which changed my approach toward engineering.

Suhail Siddiqui, a very close friend of mine and a South Carolina resident, kept insisting that I come to the United States to pursue graduate studies, which, in hindsight, was the best professional decision I ever made. He was the man who introduced me to this great country, full of opportunities. Professor Joseph Bradburn, then the chairman of the Civil Engineering Department, University of South Carolina, awarded me an assistantship on the first day I attended the college and closely monitored my progress. After I graduated in 1994, when this country was going through an economic recession and there were not many construction projects in Columbia, South Carolina, Professor Richard Pool helped me obtain my first job in structural engineering at Kyzer & Timmerman.

I moved to Florida in the last quarter of 1994. Khush Daruwalla, Sam Gilmore, Phil Azan, and Hamid Dolokhani helped me in my jobs and immigration, and I do not have enough words to thank them. While I was working for the City of Miami Beach, Herb Gopman (fondly called Hi-Rise Herbie in South Florida) walked into my office and offered me a partnership in his structural engineering firm. Gopman Consulting Engineers provided me with opportunities to work on some of the landmark projects in South Florida, inclusive of the three Trump Towers in Sunny Isles Beach. I also thank Andrew Barnett for preparing the drawings and sketches included in this book.

I am indebted to my employers, Siraj & Renu, Bechamps & Associates, Gopman Consulting Engineers, and the cities of Miami Beach, North Miami Beach, and North Miami for providing me with opportunities to grow in my profession. Finally, my thanks to this great nation—God bless America.

Author

Syed Mehdi Ashraf, P.E. is a licensed engineer under the structural disciplines I and II, general contractor, building official, building plans examiner, and special inspector in the state of Florida. He is certified by the Miami-Dade County as a building plans examiner, building inspector, and structural plans examiner. He has trade certifications from several agencies, such as the International Code Council and the Building Security Council. He holds a master's degree in engineering from the University of South Carolina and a master's degree in humanities and social sciences from the University of Middlesex. He is the recipient of numerous awards, including Florida ASCE Government Engineer of the Year, Miami-Dade ASCE Engineer of the Year, Broward ASCE Engineer of the Year, South Florida Plans Examiner of the Year, and Distinguished Alumni of the National Institute of Technology, Warangal. He has also been honored at the Florida International University, University of Miami, City of Miami Beach, and Temple Emanu-El. He is a fellow of the American Society of Civil Engineers.

Mehdi Ashraf has worked on the design of a variety of building structures, such as hospitals; airport terminals; cargo buildings; residential and commercial buildings; hotels; educational facilities; sports stadiums; holiday resorts; industrial structures; reverse osmosis desalination plants; high-capacity water tanks; high-rise, mid-rise, and low-rise buildings; and single-family housing. The list of major clients includes Florida Board of Professional Engineers, Trump Dezer, United States Postal Service, Miami-Dade Building Code Compliance Office, Miami International Airport, Williams-Brice Stadium, University of South Carolina, Sharjah Cricket Stadium, World Bank, University of North Carolina, Miami-Dade Department of Environmental Resources Management, Holiday Inn, Montenay Garbage Treatment Plant, Related Group, and several renowned architects, attorneys, and developers. His areas of expertise include structural engineering, threshold inspections, construction administration, building code review and implementation, concrete restoration, expert witness, condition assessments of buildings, project management, and historic preservation of buildings.

Mehdi Ashraf has served as an adjunct professor in the Department of Civil Engineering, Florida International University, teaching graduate and undergraduate courses in structural steel, prestressed concrete, and timber design. He has also conducted several seminars on engineering laws and rules and many other engineering topics.

Section I

Concepts and Codes

1 Main Design Concepts

1.1 INTRODUCTION

In this chapter, the basic concepts of reinforced concrete are introduced. Two building models are provided to illustrate the actions of building structural elements such as slabs, beams, columns, footings, and walls. The actions of forces such as bending, shear, axial force, axial force plus bending, torsion, diagonal tension, and shear friction are conceptually explained. This chapter is concept based and the later chapters are code based. In order to understand the code requirements and the design methodology, understanding of the concepts of structural elements and the actions of the forces on them due to the applied load is very important. Hence, the reader should give prime importance to this chapter.

1.2 BRIEF HISTORY OF CONCRETE

Steel and concrete seem the most modern of materials, yet they can be traced back through thousands of years of human civilization. Although steel itself has been used only since the middle of the nineteenth century, iron, the major component of steel, has been found in tools dating back to 3000 BCE. Concrete, the quintessential component of the modern urban landscape, was discovered in the grounds of a Stone Age Yugoslavian village built around 5600 BCE. Although a type of concrete called pozzolanic cement was widely used in bridges and aqueducts built by the Romans from 200 BCE onward, the technique of making concrete was largely forgotten until reinvented in the eighteenth century. Today's modern cityscape is built using reinforced concrete, concrete strengthened by steel bars and stronger than either material alone.

In 1848, Jean-Louis Lambot, a French agriculturalist, used reinforced concrete to manufacture a rowboat, which sank. Later in the 1870s, Jean Monier, a French gardener, successfully used reinforced concrete to produce tree and flower pots and later pipes and reservoirs.[1] In 1885, Englishman W.B. Wilkinson developed reinforced floors with wire ropes to produce fire-resistant floors.[2] In 1865, Francois Coignet constructed concrete floors with longitudinal and transverse rods embedded in them.[3] William Ward, a New York mechanical engineer, engaged architect Robert Mook to construct the first reinforced concrete house in the United States in 1875.[3] All beams, floors, walls, cornices, and towers were constructed using reinforced concrete. In 1903, the American Society of Testing and Materials (ASTM) and the American Society of Civil Engineers appointed committees to investigate the problems pertaining to reinforced concrete. The American Concrete Institute (ACI) (formerly National Association of Cement Users) was established in 1906. The ACI codes of practice was first published in 1940, with revisions in 1941, 1947, and 1951.[4]

Although steel is mostly iron, the other materials present in it, mainly carbon, give it its unique strength and other properties. Over 90% of all steels are general-purpose carbon steels, which contain up to 2% carbon. Steel was first produced around 1860 by William Kelly (1811–1888) in the United States and Henry Bessemer (1813–1898) in England.[5] The Bessemer process, by which most steel has been produced ever since, involves blasting drafts of air through a furnace containing impure molten iron. The oxygen in the air bonds chemically with the impurities and removes them, leaving behind steel. Modern variations include the basic oxygen process, in which pure oxygen is used instead of air, and the electric furnace, in which massive electrodes create high temperatures by passing enormous electric sparks through iron.

1.3 HOW DOES REINFORCED CONCRETE WORK?

Concrete is a type of artificial stone made by mixing dry aggregates (sand and gravel) and cement, then adding water. This makes a soft mix that can be molded easily or transported in a rotating concrete mixer. Different types of concrete can be made by varying the basic ingredients. Stronger concrete can be made by increasing the density, which involves increasing the cement content and reducing the aggregate and the water content. Concrete typically gets stronger as it gets older. It takes several days for wet concrete to set properly, but it will continue to gain strength. Washa and Wendt tested concrete specimens and concluded that the compressive strength of concrete increases as a logarithm of age for approximately 10 years. It could then remain the same or decrease depending upon the atmospheric condition it is exposed to.[6] The properties of concrete can be varied in other ways. It can be made either waterproof, to resist rain, or porous. It can be smoothed off or textured to look like wooden paneling.

Reinforced concrete is a composite material primarily consisting of concrete and reinforcing steel (termed as "rebar" in the American construction industry). Concrete has relatively low tensile strength and ductility; on the other hand, steel has high tensile strength and ductility. Typically, the reinforcement in concrete is steel, which is embedded in concrete during its casting. Reinforcement patterns are designed to resist tensile stresses in particular regions of the concrete that might cause unacceptable cracking and/or structural failure. Modern reinforced concrete can contain varied reinforcing materials made of steel, polymers, or alternate composite material in conjunction with steel.

The advantages of reinforced concrete are many: relatively low cost, good weather resistance, good fire resistance, and excellent formability of concrete, which makes it usable in various structures, including buildings, bridges, culverts, dams, reservoirs, silos, tanks, and many others.

Dealing with concrete involves the mechanics of elements consisting of two materials—steel and concrete. Concrete behaves differently in tension than in compression and may be elastic or inelastic. In order to overcome this problem, certain assumptions are made, to simplify the issue and come to an acceptable solution. The assumptions made are discussed in Section 4.2 of the book.

The main goal of this chapter is to establish a firm understanding of the behavior of reinforced concrete elements, and in the subsequent chapters, we will get familiarized with the requirements of the codes, specifications, design, and construction methods. A design engineer aims at safe, economical, and efficient structures, and reinforced concrete supports the cause because of its low cost, weathering, fire resistance, high compressive strength, and placeability.

1.4 TYPES OF LOADS

During the lifetime of a building, several types of loads may act, but the most permanent load is the self-weight of the building because it would be acting till the time the building is demolished or it collapses due to other external loads. Other dead loads could be the floor finishes and wall partitions permanently fixed. The probability of the full design dead load acting throughout the lifetime of the building is the highest. The next load with the highest probability of action is the live load, which consists of furniture and people living and moving in the building. Then there are other loads such as snow load and rain load, which are seasonal natural loads. Loads such as dead load, live load, snow load, and rain load are attracted by the earth's gravity and hence are termed as "gravity loads." There is also a probability of natural loads such as wind load and earthquake load acting on the building. These loads can act laterally, in the direction of the earth's gravity or against the earth's gravity. Typically, the wind load and the earthquake load are called "lateral loads."

The load that can act independently (without combination with any other load) is the dead load if the building is abandoned; otherwise, loads act in combinations. A factor is provided to the load, according to the probability of its occurrence in the modern "strength design" method. In the past, for the "allowable stress design" method, no factor was provided to the loads and the loads were called "service loads." The strength design and allowable stress design methods are explained in the subsequent sections of this chapter. Loads, load combinations, and load factors are discussed in detail in Chapter 3 of the book.

1.5 STRENGTH AND SERVICEABILITY

In order to resist the intended loads, the building structure and each element of the structure shall be strong enough to resist a collapse of the structure upon the application of the load. The structure or its part should be stiff enough to resist cracks, sagging, hogging, tilt, and vibrations to provide a level of comfort to the occupants of the building, which is the serviceability aspect of the structure.

During the process of design, the engineer estimates the load according to the occupation of the building, specifies the strength of materials to be used, sizes the members, and details the reinforcement in the members. However, during construction, the dimensions of the members may not be the same as specified on the drawing, the reinforcement of elements may not be placed accurately, and the material may not have the strength as specified on the drawings due to poor workmanship and construction administration. During the occupancy of the building, the magnitude and location of the loads acting may be different than the assumed loads and their locations, loads may be distributed in a different manner than assumed, assumptions of the analysis may not be correct, and actual behavior of the material might be different. Due to these issues, a safety factor is used during the design:

$$\mathrm{SF} = \mathrm{SM} - \mathrm{LA} \tag{1.1}$$

where

SM is the strength of materials
LA is the loads acting
SF is the safety factor

As discussed above, the loads may differ from the assumed loads, and the strength of materials may differ from the specified strength. Hence, since SM and LA are variables, safety factor (SF) is also a variable in the practical world of design, construction, and the life of the structure.

The strength design method can be simply explained in the following terms:

Strength provided ≥ strength required to carry factored load.

Strength provided is obtained from the code, and strength required is obtained from structural analysis, and then appropriate safety factors are applied.

The structure and its elements must be strong enough to resist loads, which may be applied during its lifetime, with an extra capacity. Hence, structure must be sized and provided with reinforcement such that each element is capable of resisting loads resulting from some overloading cases, which are more than the assumed or intended load. This concept is called the "strength design."

Concrete is elastic if it is stressed till approximately half its strength and steel remains elastic practically till a point before it reaches its yield stress. However, the strength design method uses the nonlinear inelastic range of both materials to calculate the nominal strength of the members. The nominal strength is reduced by a strength factor. The applied load should be less than the reduced nominal strength. Using the strength design method, focus should also be given to the service loads to control deflections and cracks. The serviceability limit state is also an important part of the design.

In the past, the "allowable stress design" method was used, wherein members are designed such that the stresses in steel and concrete due to the service loads are less than the specified limits, which is approximately half the strength of concrete and steel. This method is outdated and is not specified in the current concrete code.

The allowable stress in steel and concrete is only a fraction of the yield stress of the steel and concrete. In the allowable stress design, the stress in steel and concrete is not allowed to exceed the allowable. The stresses are calculated based on the service loads, which are not factored.

1.6 BUILDINGS

This book is catered to building designs. In order to understand how reinforced concrete buildings or structures are designed and constructed, it is very important to ascertain how each structural element of the building works, how they are integrated, and how the assembly works after the integration of the elements. In order to explain the functions and actions of individual structural elements of a building, I have selected two types of buildings:

1. Buildings with beams and column frames
2. Building with flat plates supported on columns and shear walls

1.6.1 Buildings with Beams and Column Frames

A ten-storied office building (Figures 1.1 through 1.4) is considered to explain the building type (A). The building has five bays and six frames in each direction. Grid lines 'A' through 'F' are used in one direction and grid lines '1' through '6' are used in the other direction to establish the locations of the structural elements such as columns, beams, slabs, and footings. The horizontal shallow plates are called slabs. Vertical gravity loads act on the slabs. They resist the loads and transfer them to the beams. Beams are stiffer horizontal elements. Beams are stiffer than the slabs because they have more depth than the slabs. The transfer of the loads from the slabs to the beams is based on the tributary area of the slab, which is explained later in this chapter. Each span of the beam transfers the load to the columns according to the action of the beam. Columns are vertical elements capable of resisting compressive and bending forces. In this example, only uniform loads of the slabs are considered, and hence, the beams transfer half of their loads to the two supporting columns. The columns and walls transfer the loads to the footing and then the load is transferred to the ground. Footings can behave such as slabs or beams or a combination of both depending on their type.

1.6.2 Building with Flat Plates Supported on Columns and Shearwalls

A ten-storied office building (Figures 1.5 through 1.7) is considered to explain the building type (B). The building has five bays and six frames in each direction. Grid lines "A" through "F" are used in one direction and grid lines "1" through "6" are used in the other direction to establish the locations of the structural elements such as columns, slabs, and footings. The horizontal shallow plates are called flat plates. Vertical gravity loads act on the slabs. They resist the loads and transfer them directly to the columns and walls according to their tributary area. Walls are vertical members capable of resisting gravity loads and also a large magnitude of lateral loads. The columns and walls transfer the loads to the footings, and then the load is transferred to the ground. In flat-plate construction, beams are not used. The slabs are directly supported on the columns or walls.

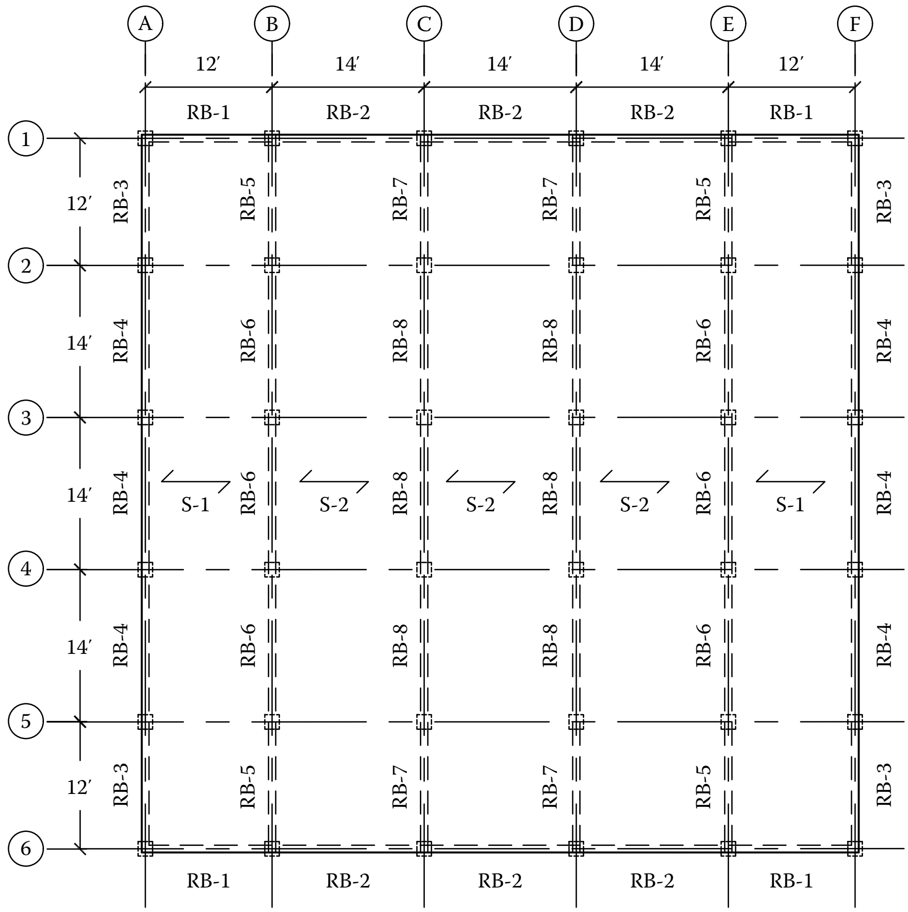


FIGURE 1.1 This is the roof framing plan of building type (A). There are five panels of one-way continuous slabs. Slab type S-1 has one edge continuous and the other edge discontinuous. Slab type S-2 has both edges continuous. The five panels are supported on six continuous beams. The beams supporting the slabs are located along grid lines 'A' through 'F.' Beams on grid lines 'A' and 'F' are exterior beams. These are five-span continuous beams (RB-3, RB-4, RB-4, RB-4, and RB-3). Beams on grid lines 'B,' 'C,' 'D,' and 'E' are interior beams. These are also five-span continuous beams (RB-5, RB-6, RB-6, RB-6, and RB-5). Beams are also provided along grid lines '1' and '6.' These beams do not support the slab. The term RB is used because these are roof beams. Examples in the design chapters on slabs and beams illustrate that the design of the two exterior slab panels (S-1) and the three interior slab panels (S-2) is identical, and hence, the two identical exterior slab panels are labeled 'S-1' and the three identical interior slab panels are labeled 'S-2.' Similarly, the labeling of the typical beams is unique.

1.6.3 Structural Elements

In Sections 1.6.1 and 1.6.2 of the book, the student was introduced to the most used structural elements in modern buildings—slabs, flat plates, beams, columns, walls, and footings. Each of these elements is discussed in detail in the following sections. The student is encouraged to review the plans provided in Figures 1.1 through 1.7.

1.7 SLABS

Concrete slabs are horizontal members supporting the gravity load—dead, live, snow, rain, etc., as discussed above. Slabs are designed to adequately support these gravity loads, fulfilling the strength and serviceability requirements of the code and transferring them to their supports. Slabs also act as diaphragms to collect the lateral loads and transfer them to the supports such as columns and shearwalls. They also provide bracing to the columns and walls. Slabs can be supported on beams, columns, walls, joists, and ribs. Sometimes, ribs and slabs are integrated and act together and are called ribbed slabs (if there are ribs only in one direction) and waffle slabs (if there are ribs in both directions). The choice of the slab depends on many factors such as economy, column-free spans, the number of floors in the structure, types of loads, and strength and serviceability requirements.

Slabs can be classified under several categories according to their support conditions and spans. They can be supported on beams, columns, and walls (Figures 1.1 through 1.7). If the slabs are supported on beams and walls, the edges of the slabs have a continuous support. If the slabs are supported on columns, only four points of the slab have a column support and are called flat plates. Sometimes, the slabs supported on the columns directly may not have a rectilinear shape due to architectural

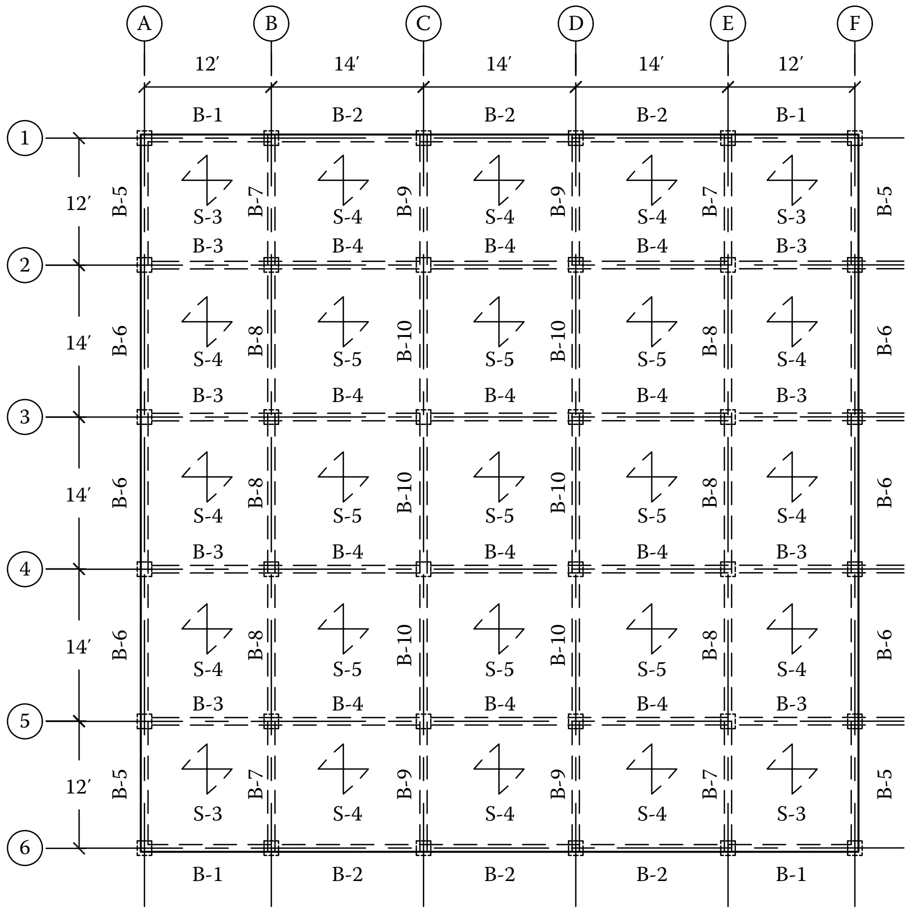


FIGURE 1.2 This is the typical floor plan of building type (A). All slabs are two-way slabs. There are four typical corner panels (S-3), twelve typical edge panels (S-4), and nine typical interior panels (S-5). The corner panels have two edges continuous, the edge panels have three edges continuous, and the interior panels have all four edges continuous. In this slab system, all beams support slab loads.

requirements, and the analysis of the slab becomes complicated, requiring more sophisticated methods of analysis like the finite-element method.

Slabs can also be classified based on their length-to-breadth ratio called the "aspect ratio." If the aspect ratio of a slab is greater than or equal to 2, they are classified as "one-way" slabs (Figure 1.8), and if the aspect ratio is less than 2, they are classified as "two-way" slabs (Figure 1.9). Either with the beams and walls support or the columns support, for a one-way slab, two parallel long edges of the slabs are supported. The shorter edges do not require a support. For a two-way slab, all the four edges (in the case of a beam- or wall-supported slab) and four corners (in case of a column-supported slab) are supported.

Slabs supported on columns are called "flat plates" if they do not have a capital at the supporting columns (Figure 1.10). If there is a capital present at the supporting columns, they are called "flat slabs" (Figure 1.11). According to the code definition (section 2.3), "column capital" is the enlargement of the top of the concrete column located directly below the slab or drop panel that is cast monolithically with the column. Column capitals or drop panels help in resisting the punching shear. Since there are no beams in flat plates and flat slabs, they are also required to act as horizontal diaphragms supporting the lateral load (wind) during the interaction with framed columns or shearwalls.

Based on the structural action and continuity of support, slabs can also be classified as continuous (having two or more spans), simply supported, or cantilever (supported only on one edge). Continuous or simply supported slabs can be one way or two way.

If the spans and/or loads are very large, then ribbed or waffle slabs can be used. Ribbed slabs are one-way slabs having thin concrete beams called "ribs" running parallel at a close spacing (3–5 feet) across the shorter span to increase the stiffness of the slab (Figure 1.12). Waffle slabs are two-way slabs having thin concrete beams called "ribs" running parallel in both directions at a close spacing (3–5 feet) to increase the stiffness of the slab (Figure 1.13).

While selecting the type of slab during the preliminary design, the effect of the cost of material and labor and strength is examined. In some countries, the cost of labor is lower compared to the cost of material, while in other countries, it is the opposite. Cost sensitivities of the building code performance requirements are compared with those of the basic design parameters,

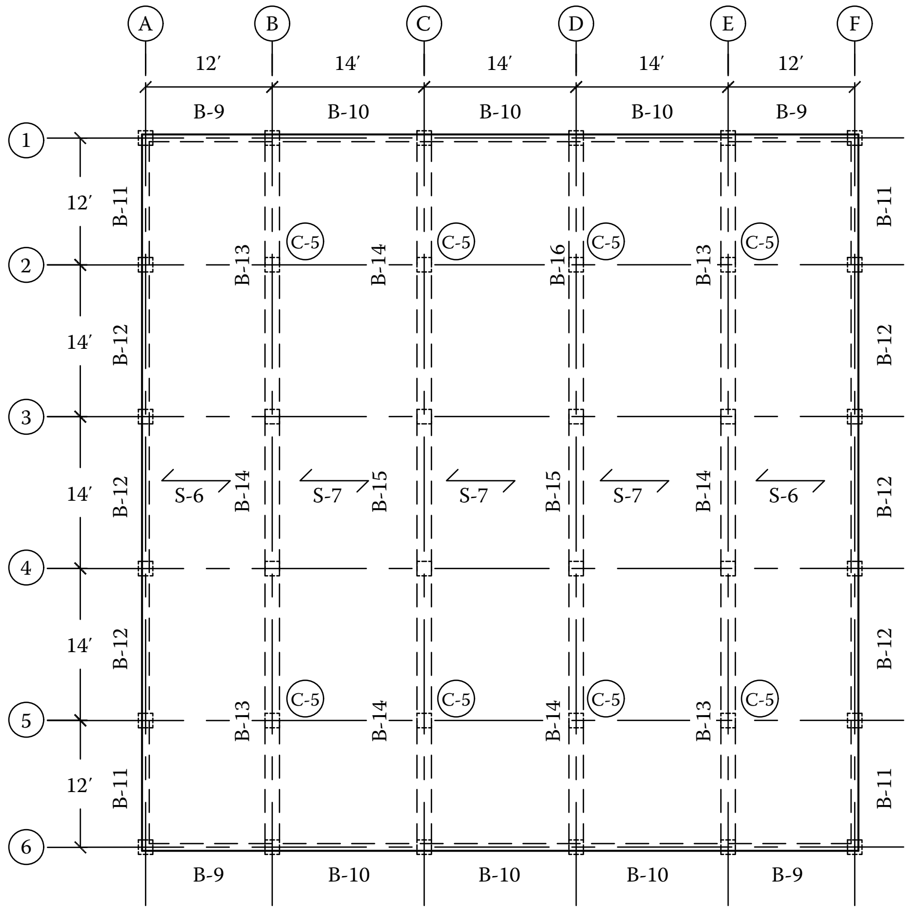


FIGURE 1.3 This is the second floor plan of building type (A). The slab system consists of five one-way panels supported on beams along grid lines 'A' through 'F.' It is similar to the roof plan, but columns on the intersection of grid lines '2' and '5' and grid lines 'B,' 'C,' 'D,' and 'E' are discontinued on the second floor. The columns do not continue to the ground floor (See Figure 1.4). The beam type B-13 on grid lines 'B,' 'C,' 'D,' and 'E' supports these columns and is called the transfer beam.

and several trends are recognized. This function is then minimized subject to several constraints, the most important being the strength requirement. This helps in the selection of the type of slab. If large column-free spans are required, then posttensioned, ribbed or waffle slabs are preferred over regular reinforced concrete slabs with beams or flat plates. For a multistoried building with residential or office load (40 or 50 psf), flat plates with or without posttensioning would be an ideal choice because the columns are regularly placed and the spans are limited to about 30 feet. Flat plates help in reducing the story height. For every 10 stories, an additional story can be obtained using flat plates. Typically, slabs in multistoried buildings are two-way slabs. For one- or two-storied buildings, one-way or two-way slabs with beams are a good choice. In one- or two-storied residential buildings, slabs can also be supported on concrete tie beams and load-bearing masonry walls. If loads are high, then the deflection of the slab would require beams to support the slab.

Slabs with beams are more expensive because of formwork and the labor involved, while flat plates have no formwork and would require less labor. They are easy to construct. Flat plates are thicker and require more reinforcement than slabs supported on beams. However, flat slabs allow easy passage of the air conditioning and electrical ducts between the underside of the slab and the ceiling.

1.8 BEAMS

Beams are stiff horizontal members that support the slab and brace the columns that support them. In modern buildings, beams support the exterior masonry infill walls, which are heavier than the internal partition walls are usually made of wood or metal studs. These beams are called "spandrel beams." Beams can be simply supported or continuous. They span between columns or walls or a combination of both. Beams are a part of the framing system. They transfer loads to the columns in the form of an axial force and moment. In shorter buildings, they support the slab for the gravity loads, but in taller buildings, they are also part of the lateral load-resisting system.

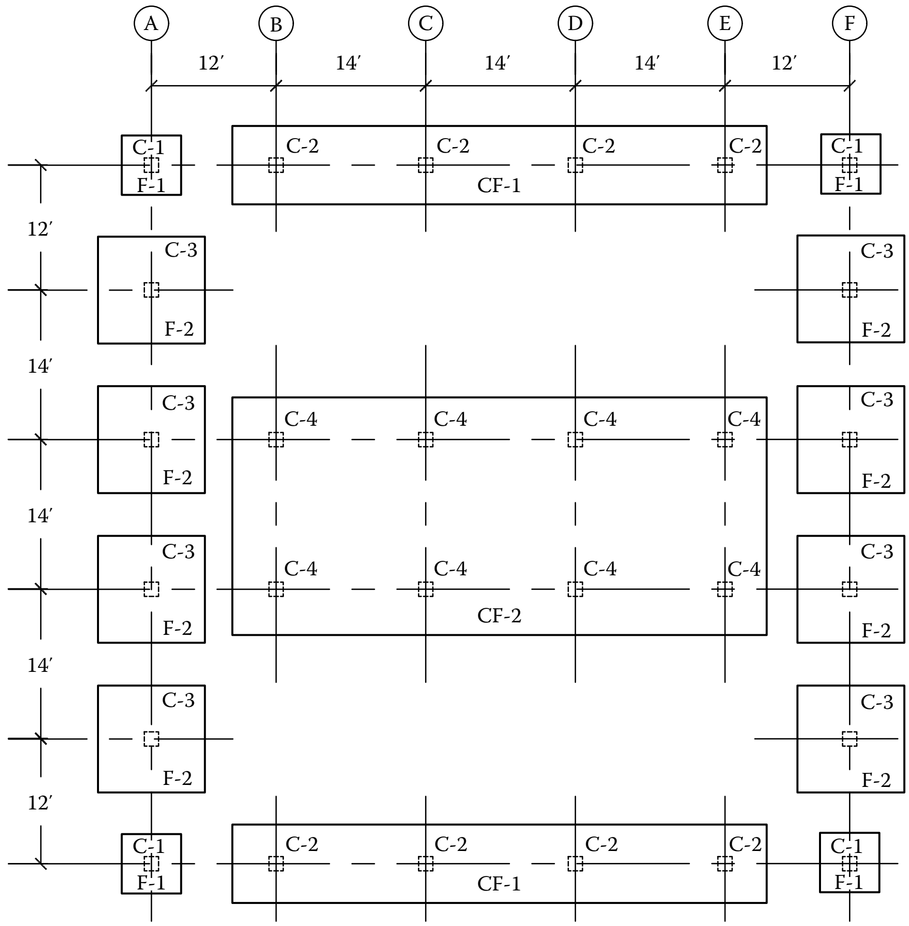


FIGURE 1.4 This is the foundation plan of building type (A). In this plan, individual pad footings (type F-1 and F-2) and combined footings (type CF-1 and CF-2) are illustrated.

Beams are used in one- or two-storied load-bearing masonry buildings where they serve the purpose of tying the concrete slab or metal or wood trusses to the load-bearing masonry walls. In such cases, they are designed to resist the lateral loads. They do not resist gravity loads except over door and window openings.

The structural action of the beams is similar to the structural action of one-way slabs in many ways, as would be illustrated in the design chapters.

1.9 COLUMNS

Columns are vertical members that support gravity loads from beams or from slabs directly. They transfer the loads to the foundations. Columns also support lateral loads in framed construction along with beams. Columns are typically continuous from the foundation to the roof, braced by the beams or floor slabs at each level, unless they are not required from a floor upward. From top to bottom, the loads acting on the columns increase, and hence, the design of the columns at the lower floors is heavier than the design of the columns at upper floors in terms of the dimensions of the column, amount of reinforcement, and strength of the concrete. From the lower floors to the upper floors, the dimensions of the columns, amount of reinforcement, and/or grade of the concrete of the columns is reduced.

1.10 SHEARWALLS

Shearwalls are vertical members that support gravity loads from beams or from slabs directly. They transfer the loads to the foundations. The main purpose of the shearwalls is to resist the lateral forces that are collected at each floor and transferred to them. Shearwalls are typically continuous from the foundation to the roof, braced by floor slabs at each level, unless they are not required from a floor upward. From top to bottom, the loads acting on the shearwall increase, and hence, the design of the shearwalls at the lower floors is heavier than the design of the columns at the upper floors in terms of the wall

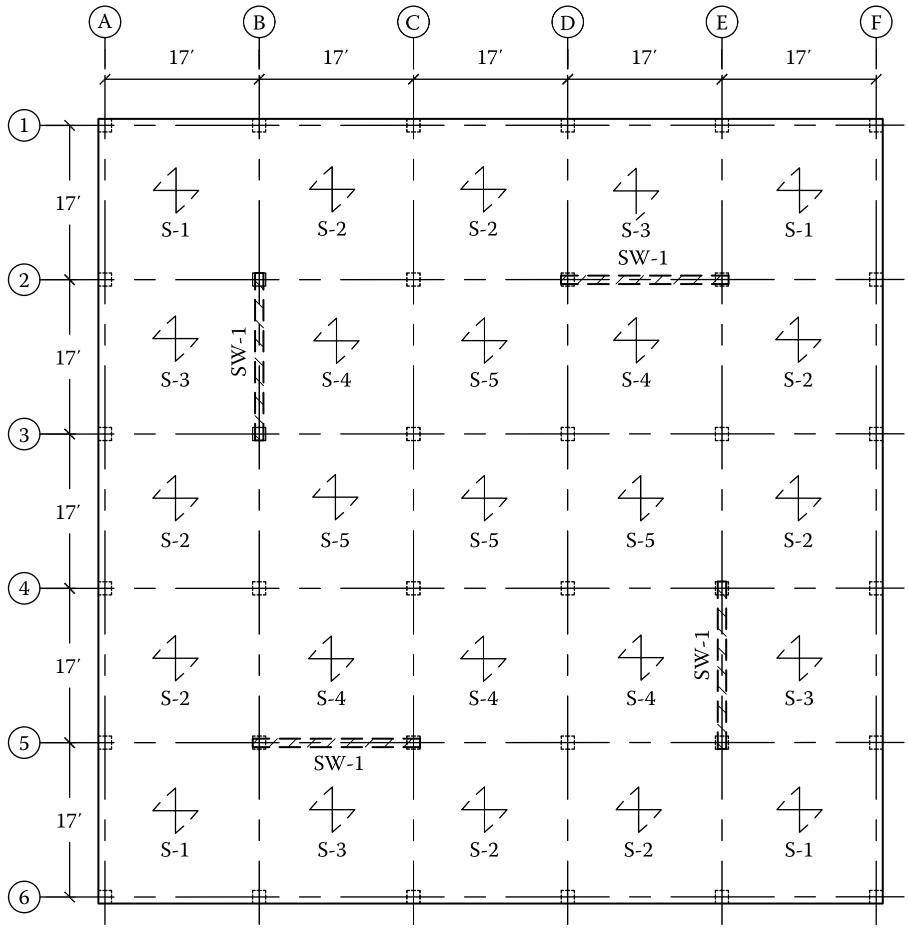


FIGURE 1.5 This is the roof plan of building type (B). All slab panels are square (17 feet × 17 feet) flat plates. Slab S-1 is a corner panel with two edges continuous, supported directly on four columns. Slab S-2 is an edge panel with three edges continuous, supported directly on four columns. Slab S-3 is an edge panel with three edges continuous, supported directly on two columns and continuously supported on a wall along one edge. Slab S-4 is an interior panel with all four edges continuous, supported directly on two columns and continuously supported on a wall along one edge. Slab S-5 is an interior panel with all four edges continuous, supported directly on four columns.

thickness, amount of reinforcement, and strength of the concrete. From the lower floors to the upper floors, the dimensions of the shearwall, amount of reinforcement, and/or grade of the concrete of the walls is reduced.

1.11 FOUNDATIONS

Foundations are the members of a structure that collect the loads from the superstructure (all structural elements above grade) and transfer them to the ground. In the case of basements, they collect the load from the underground structure too. The types of foundations a designer can select is based on the magnitude of the load and the type of soil supporting those foundations. The main criteria in the design of the foundation are that the total settlement of the structure is small and the partial settlements be avoided.

The bearing capacity of the soil depends on the type of the soil and the underground water table. While sizing the foundation, it is ascertained that the load applied on the soil does not exceed the safe bearing capacity of the soil. The simplest of the foundations are individual pads supporting columns (Figure 1.14) and continuous wall footings supporting load-bearing walls (Figure 1.15). If columns are closely spaced and on sizing the footings based on the safe bearing capacity, if it is determined that the foundations would overlap, then footings of two or more columns are combined and the footings are called "combined footings" (Figure 1.16). When it is not possible to provide a footing concentric with the line of action of the column due to site restrictions, footings are strapped to the adjacent footing with a strap beam to distribute the moment created due to the eccentricity to the adjacent footing. These footings are called "strapped footings" (Figure 1.17). In case of large column loads and weak soil, if there are several overlaps of footings of different columns, then a footing is provided over the entire footprint of the building, which is called a "mat footing" (Figure 1.18). For tall buildings, due to very large loads being transmitted to soil, even mat footings may not be sufficient. A column or a group of columns or shearwalls are placed on deep footings, with the

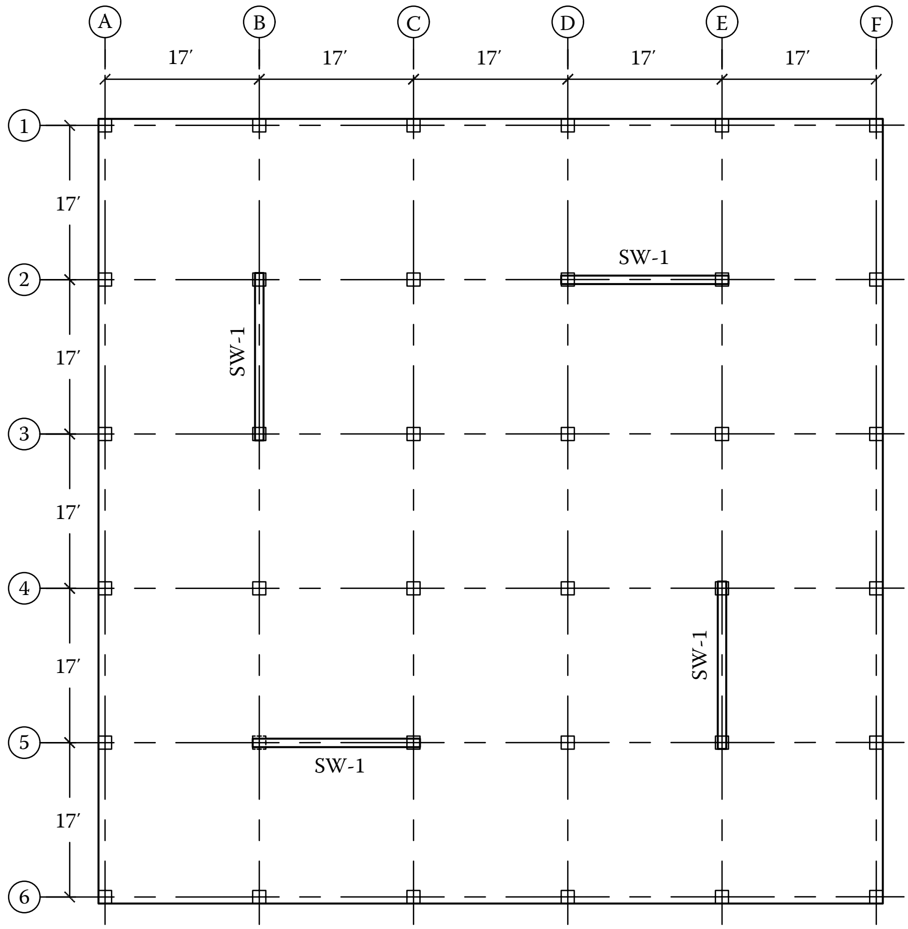


FIGURE 1.6 The typical floor plan of building type (B) follows the same pattern as that of the roof plan.

footprint resembling an individual pad or combined footings. These footings are called "pile caps." The pile caps are supported on piles, which are very deep vertical concrete members. The piles use adhesion with the surrounding soil and the end bearing support of the soil to transfer the load to the soil (Figure 1.19).

1.12 FORCES FOR EACH TYPE OF STRUCTURAL ELEMENT

On the application of gravity loads directly to the slabs and also to beams sometimes and lateral loads to the walls or claddings, various forces start acting on the structural elements as discussed above. These forces are used to determine the sectional strength of the elements and are discussed in detail in Chapter 4 of the book. In this section, we will not discuss the serviceability requirements, which are discussed in the design chapters.

Slabs are subject to gravity loads that cause bending and shear in the slab. Slabs are also used as diaphragms to transmit wind loads to shearwalls in tall buildings. Beams are subject to gravity loads transferred from the slabs. They may also be directly subject to gravity loads like in case of a spandrel beam, which supports exterior masonry walls. These loads apply bending and shear and sometimes torsion on the beams. Beams may also be part of lateral load resisting frames and are subject to bending, axial force and shear due to lateral loads.

Columns carry gravity loads and lateral loads if they are part of the lateral load resisting system. Gravity loads acting on the columns are transferred from beams and slabs in the form of axial load and moment. Shearwalls are designed to resist shear and moment due to lateral loads. Footings support gravity loads from the columns. They may also support moments due to lateral loads and eccentrically placed columns or walls. Footings are designed for bending and shear.

From the above discussion, it is imperative that we learn how concrete members resist bending, shear, axial loads, axial loads combined with bending, shear friction, and torsion. In the following sections, we will also learn the concepts of diagonal tension, an important aspect of the design of beams. The concept of shear friction is applied to cracks in concrete or when new concrete is placed adjacent to existing concrete.

In this chapter, only the concepts of the forces are discussed; the ACI code requirements are discussed in the subsequent chapters.

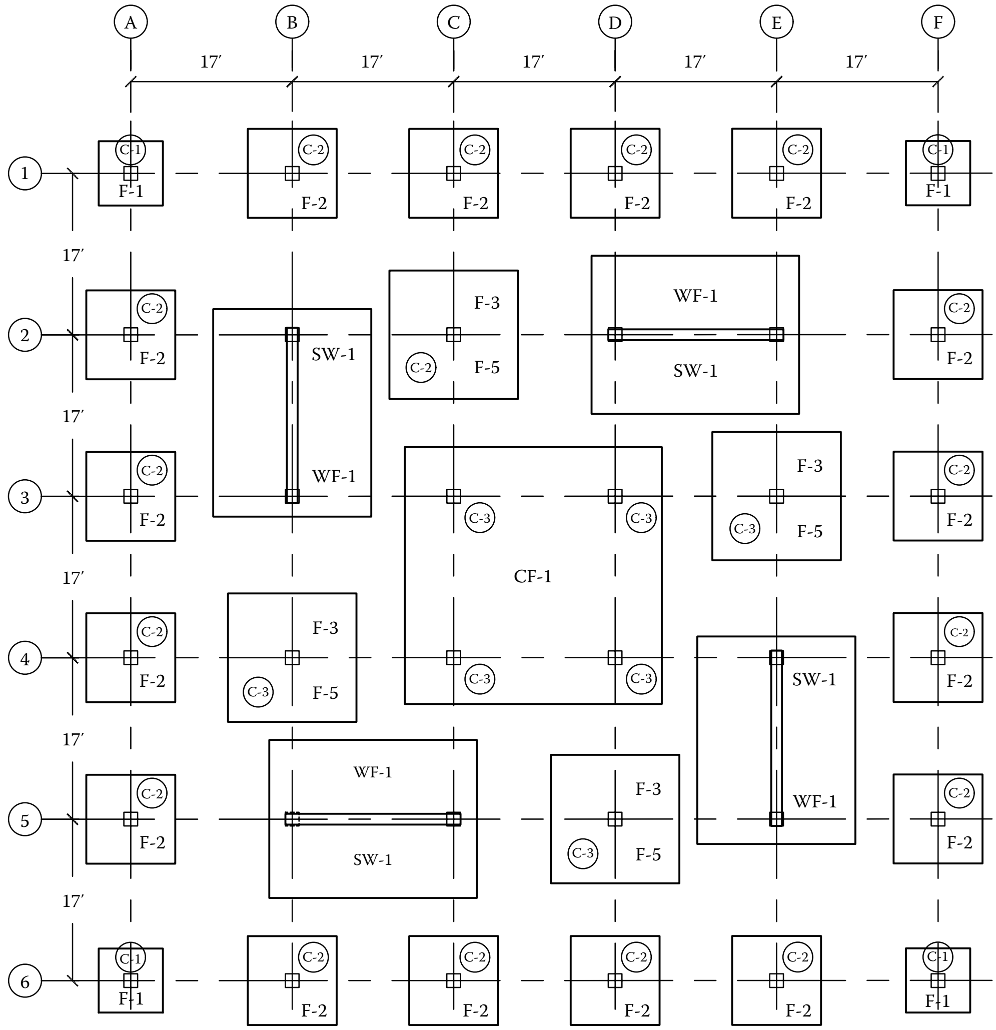


FIGURE 1.7 This is the foundation plan of building type (B), which illustrates individual pad footings (types F-1, F-2, and F-3), combined footing (type CF-1), and wall footings (type WF-1).

1.13 BENDING THEORY

In this section, the bending theory for a flexural member such as a slab or a beam is discussed. Throughout this section, a beam or a slab is referred to as "flexural member" or "member." The bending theory was first developed by Galileo and later modified by Leonhard Euler.[7] Along the span of a flexural member, bending causes normal stresses, which are maximum tensile stresses at one surface and maximum compressive stresses at the opposite surface. The location of the maximum stresses (tensile or compressive) depends on the span of the flexural member and the loading conditions. A flexural member or its portions subject to positive moments develop a concave shape during bending and, when subject to negative moments, develop a convex shape during bending.

A simply supported flexural member with a uniform load experiences a positive bending moment throughout its span. It sags and develops a concave shape. The top fibers of the member compress, and the bottom fibers expand. Hence, the maximum compressive stress is developed at the top and decreases toward the neutral axis. At the neutral axis, the stress is zero. Then, below the neutral axis, the stresses become tensile and the maximum tensile stress occurs at the bottom fiber of the member. Hence, in a flexural member, the main longitudinal reinforcement is provided at the bottom because steel is good at resisting tensile stresses.

A cantilever flexural member subjected to uniform load experiences a negative moment. It hogs and develops a convex shape. The top fibers expand and the bottom fibers get compressed. Hence, the maximum tensile stress is developed at the top and reduces toward the neutral axis. As with the simply supported flexural member, stress at neutral axis is zero. Then below the neutral axis, stresses become compressive, and at the bottom fiber, the compressive stress is maximum. Hence, in a cantilever flexural member, the main longitudinal reinforcement is provided at the top to resist tension.

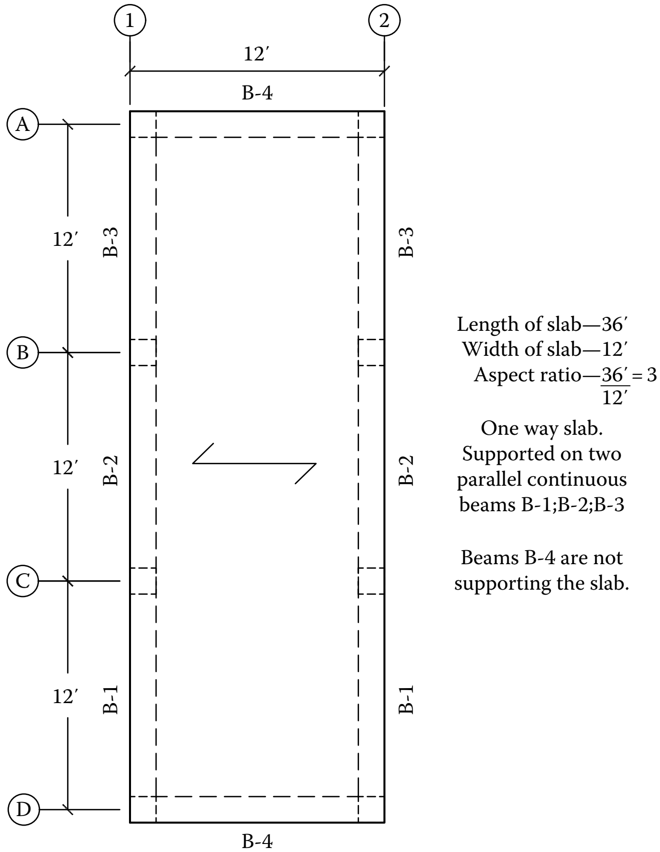


FIGURE 1.8 One-way slab.

The distribution of positive and negative moments in a continuous multispan flexural member varies with spans and loads, but definitely near the supports, the member is subject to a negative moment.

According to Hooke's law, stress is proportional to strain within the elastic limit of the body. When a flexural member behaves elastically, the maximum stress (f_{max}) and strain (ε_{max}) to which the member is subject are less than or equal to the stress (f_e) and strain (ε_e) at the elastic limit.

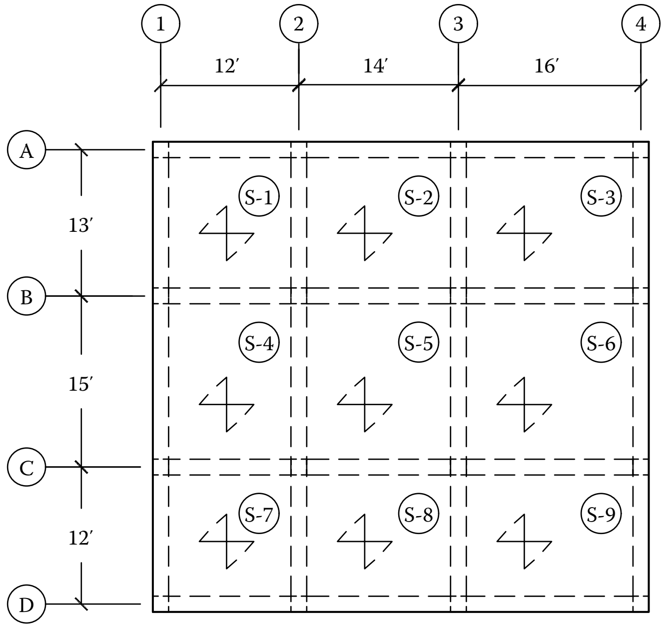
The maximum bending stress occurs at the top and bottom fibers:

$$f = \frac{Mc}{I} \quad (1.2)$$

where

f is the bending stress at a distance y from the neutral axis (psi)
M is the bending moment at the section (lb-in.)
c is the depth of the neutral axis (in.)
I is the moment of inertia of the section about the neutral axis (in^4)

If a flexural member is not reinforced, then it cannot work because it is weak in tension. The tensile strength of concrete is very small compared to its compressive strength. Please refer to Section 5.9 of the book for the explanation of modulus of rupture. Such a flexural member fails before the maximum strength of the concrete is utilized. Hence, reinforcing bars are placed near the surface of the member where the tensile stresses occur. As explained before, in a simply supported flexural member, reinforcing bars are placed near the bottom surface, and in a cantilever flexural member, they are placed near the top surface with a proper cover. Bonding between the concrete and steel is essential for the concrete and steel to work together. In the modern times, twisted bars or deformed bars are used to provide the bonding. Furthermore, the bars are anchored into the support, as discussed in Chapter 11 of the book. Prior to the invention of twisted bars or deformed bars, plain bars were used, and checking the bonding with the reinforcement bars and concrete was part of the design.



Slab	L/B	Continuous Edges	Discontinuous Edges
S-1	13′/12′ = 1.803	2	2
S-2	14′/13′ = 1.077	3	1
S-3	16′/13′ = 1.231	2	2
S-4	15′/12′ = 1.250	3	1
S-5	15′/14′ = 1.071	4	0
S-6	16′/15′ = 1.067	3	1
S-7	12′/12′ = 1.000	2	2
S-8	14′/12′ = 1.167	3	1
S-9	16′/12′ = 1.333	2	2

FIGURE 1.9 Two-way slabs.

Consider a simply supported flexural member reinforced at the bottom. Start applying the load gradually on the slab or the beam. As explained before, the top fibers of the simply supported member are subject to compression, and the bottom fibers are subject to tension. When the loads are small, the tensile stresses on the concrete are smaller than the modulus of rupture. The concrete can resist the tensile stresses, and the reinforcing bars placed at the bottom also resist the tensile stresses. When load is increased to such a magnitude that the tensile stresses in the concrete exceed the modulus of rupture, the concrete begins to crack. The complete tensile stresses are resisted by the reinforcing bars, and the concrete does not resist any tensile stresses. The section would still be within the elastic limit. When more load is applied and the load reaches the ultimate value, the capacity of the member is reached. If the member is under-reinforced, then the reinforcing steel yields first and gives alarm to the occupant of the structure. Cracks are formed, and they widen. If the member is over-reinforced, then the concrete reaches its capacity first, and the structure collapses all of a sudden with no alarm to the occupant. From this discussion, we can conclude that during the design of a flexural member, it is better that we provide lesser steel and a bigger concrete section so that if the member is overloaded during its lifespan, it alarms the occupants instead of collapsing all of a sudden (brittle collapse).

Stresses in steel and concrete can be calculated based on the uncracked and cracked sections (Figure 1.20). Assume the concrete section under discussion has compression at the top and tension at the bottom. The reinforcement is provided at the bottom. When the concrete is not cracked, it is taking the tensile stresses. Stresses in concrete at the top and the bottom are calculated by converting the steel area into an equivalent concrete area. This is done by multiplying the total cross-sectional area of steel provided by a factor equal to ($n - 1$), where n is the modular ratio ($n = E_s/E_c$) (Figure 1.21). The modular ratio is ratio of the moduli of elasticity of steel and concrete. This is called a transformed section, which converts the cross-sectional area of the reinforcing steel into an equivalent area of concrete.

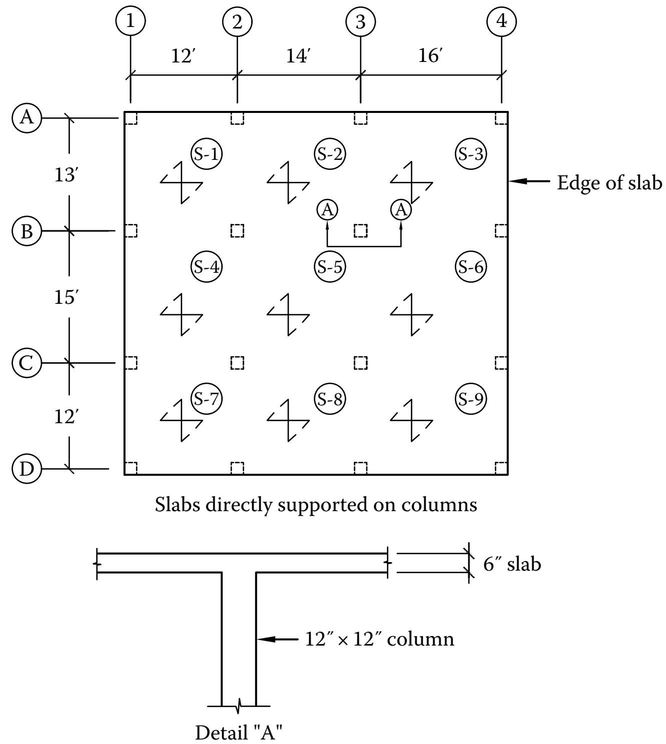


FIGURE 1.10 Flat Plate.

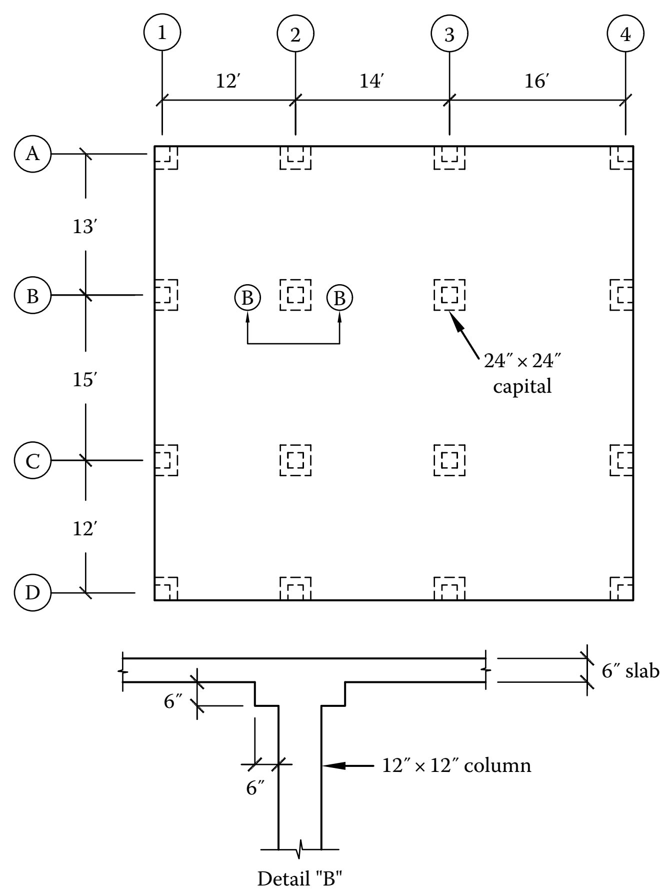


FIGURE 1.11 Two-way flat slab.

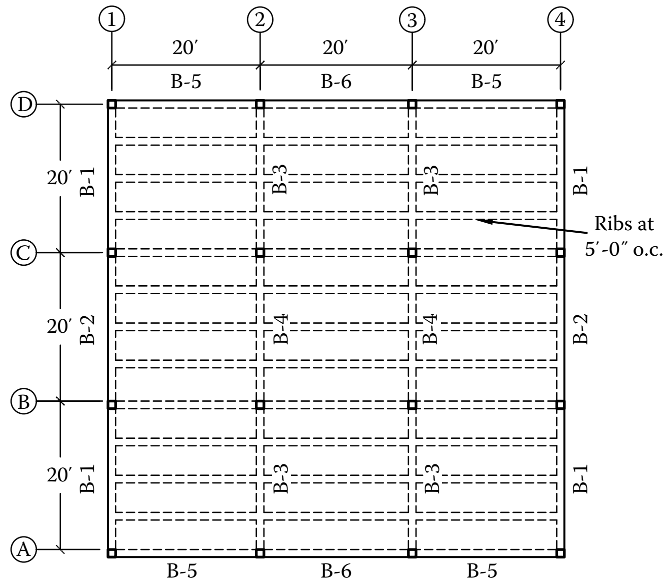


FIGURE 1.12 One-way ribbed slab.

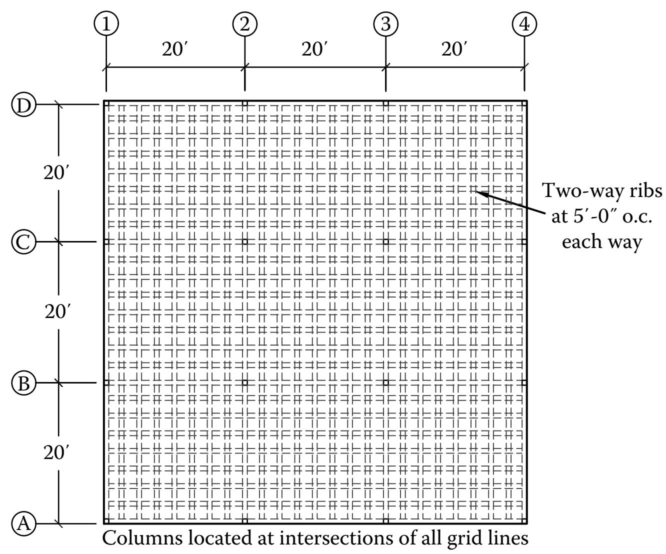


FIGURE 1.13 Two-way ribbed slab.

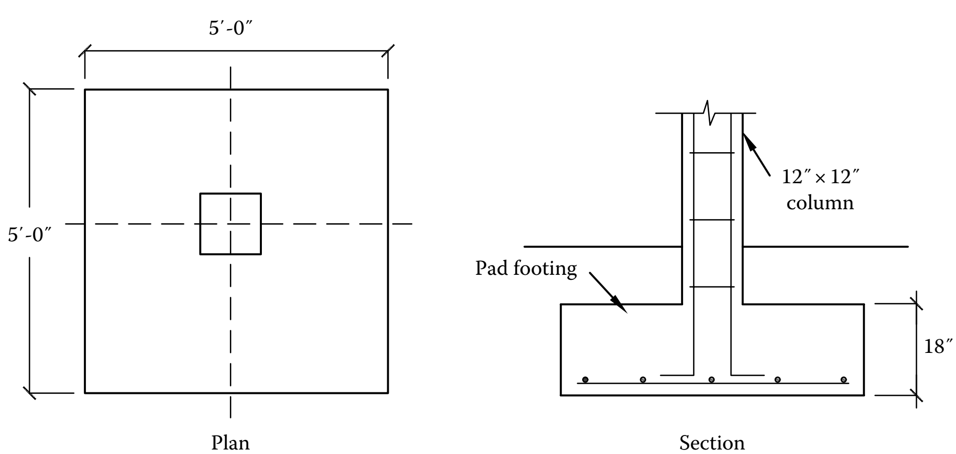


FIGURE 1.14 Pad footings.

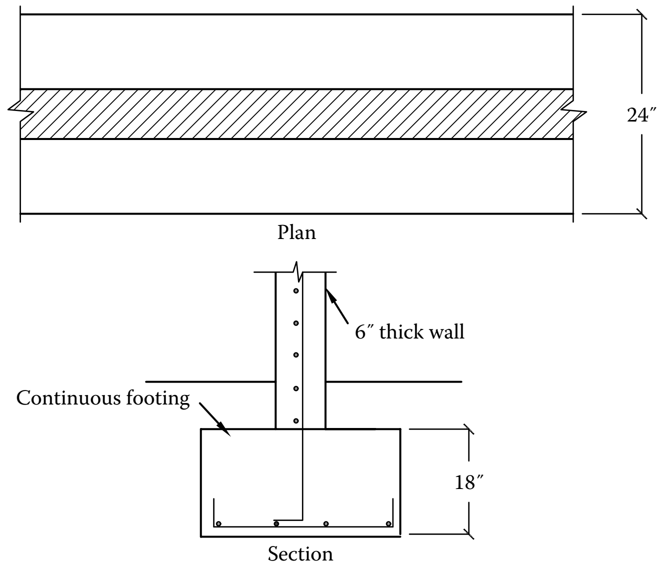


FIGURE 1.15 Wall footings.

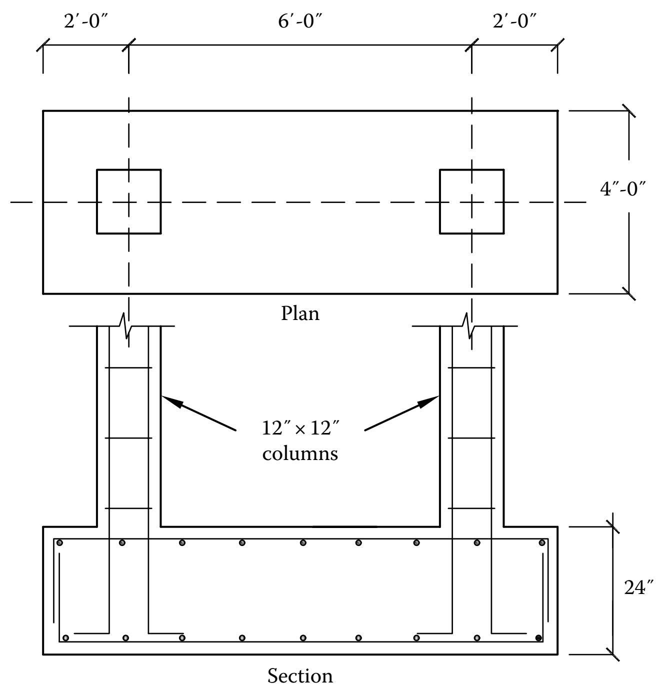


FIGURE 1.16 Combined footings.

Taking moment about the neutral axis,

$$\frac{bc^2}{2} = \frac{b(h-c)^2}{2} + (n-1)A_s(d-c) \tag{1.3}$$

where

- b is the width of the slab or beam
- h is the total depth of the slab or beam
- d is the effective depth of slab or beam (effective depth of the member is measured from the top surface of the member till the centroid of the reinforcing steel)
- c is the depth of neutral axis
- A_s is the area of the reinforcing steel

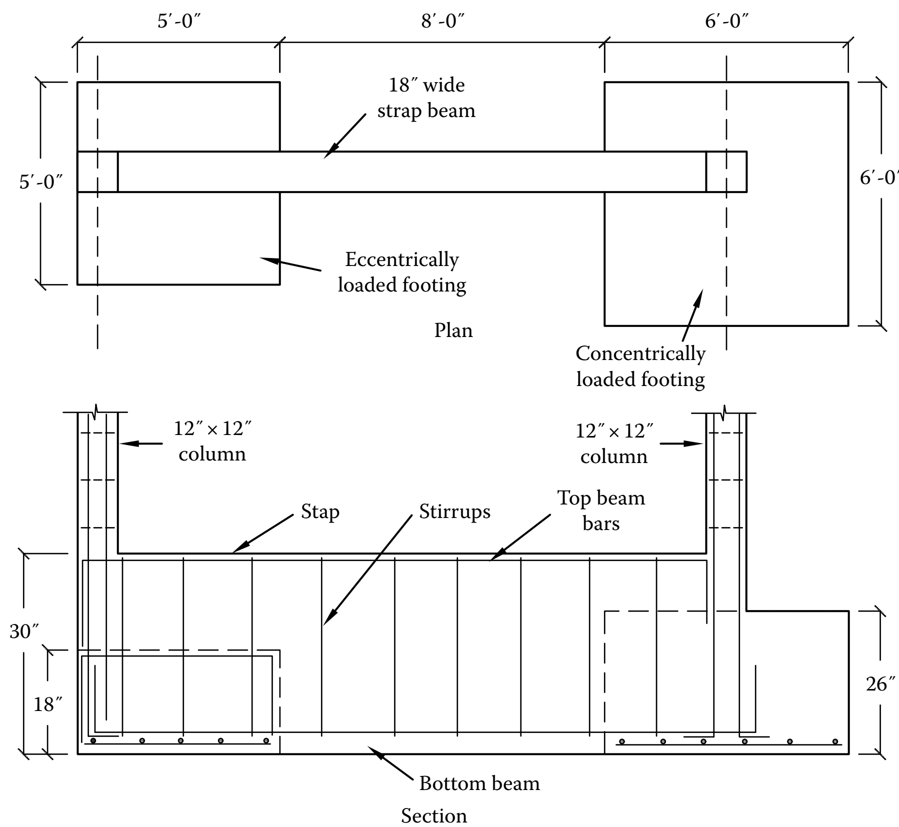


FIGURE 1.17 Strapped footing.

Solve for "c," to obtain the depth of the neutral axis from the top

$$\text{Stress in concrete at the top fiber} = \frac{Mc}{I}(\text{compressive}) \tag{1.4}$$

$$\text{Stress in concrete at the bottom fiber} = \frac{M(h-c)}{I}(\text{tensile}) < \text{modulus of rupture} \tag{1.5}$$

$$\text{Stress in the reinforcing steel} = n\frac{M(d-c)}{I} \tag{1.6}$$

The second condition is that the tensile stresses exceed the modulus of rupture of the concrete, and the section cracks (Figure 1.21). However, if the compressive stresses in the concrete are less than half the compressive strength of the concrete (f_c') and the steel stresses do not reach the yield strength, the section is still in the elastic state (Figure 1.22). In this discussion, the model of a flexural member with compression at the top and tension at the bottom is used. It is assumed that the concrete below the neutral axis is cracked and only the transformed section of steel is used (nA_s). The distance from the top fiber of the concrete to the neutral axis is assumed as "kd," where k is a fraction to be determined.

Taking moments about the neutral axis,

$$b\left(\frac{kd}{2}\right)^2 = nA_s(d-kd) \tag{1.7}$$

Hence,

$$b\left(\frac{kd}{2}\right)^2 - nA_s(d-kd) = 0 \tag{1.8}$$

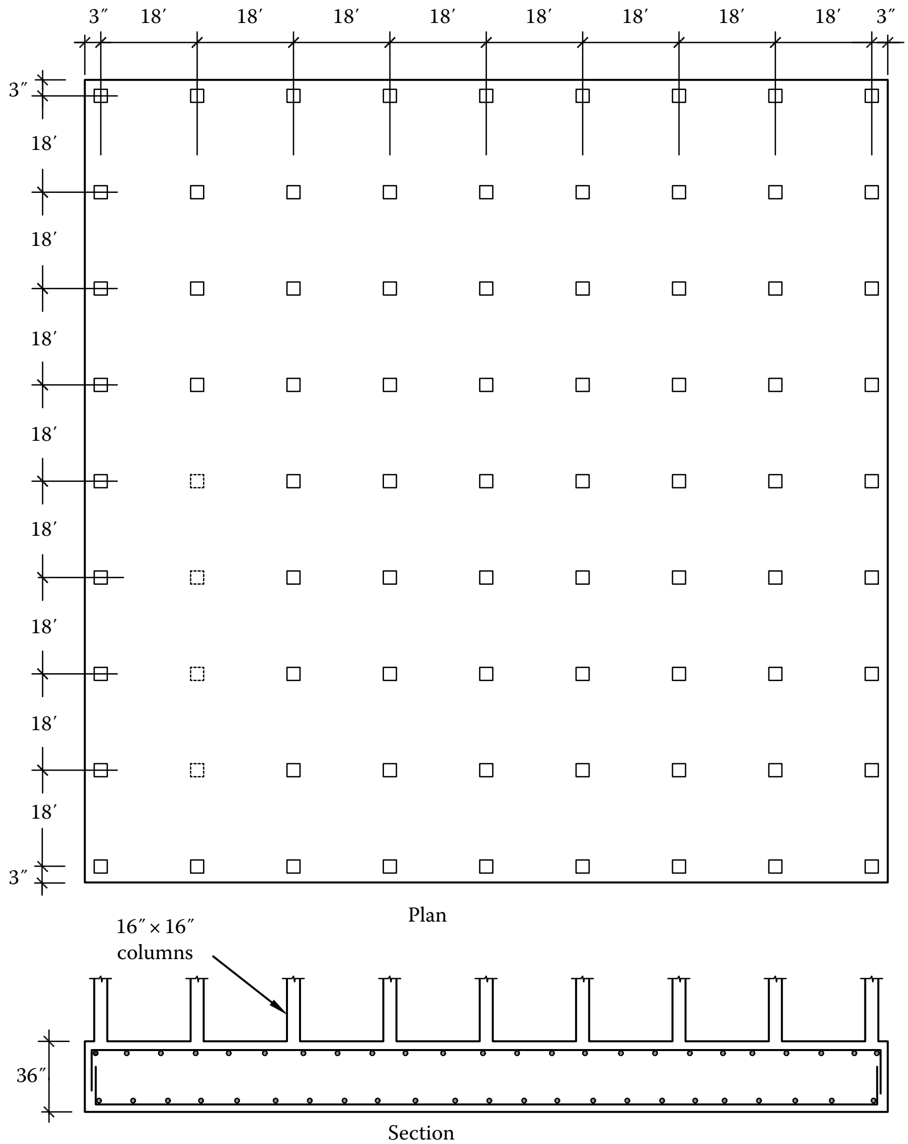


FIGURE 1.18 Mat foundations.

Knowing b, d, n, and A_s, the value of k can be obtained. Usually, the designer can predefine the percentage of the steel (ρ):

$$A_s = \rho bd \tag{1.9}$$

Substituting the value of A_s in the above quadratic equation, the value of "k" can be obtained:

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n \tag{1.10}$$

Defining,

$$jd = d - \frac{kd}{3} \tag{1.11}$$

$$j = 1 - \frac{k}{3} \tag{1.12}$$

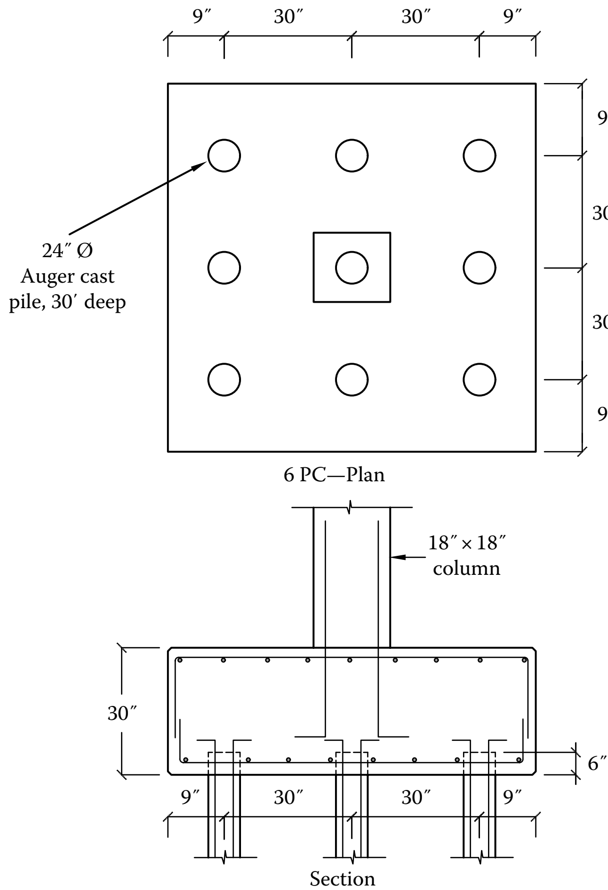


FIGURE 1.19 Pile cap.

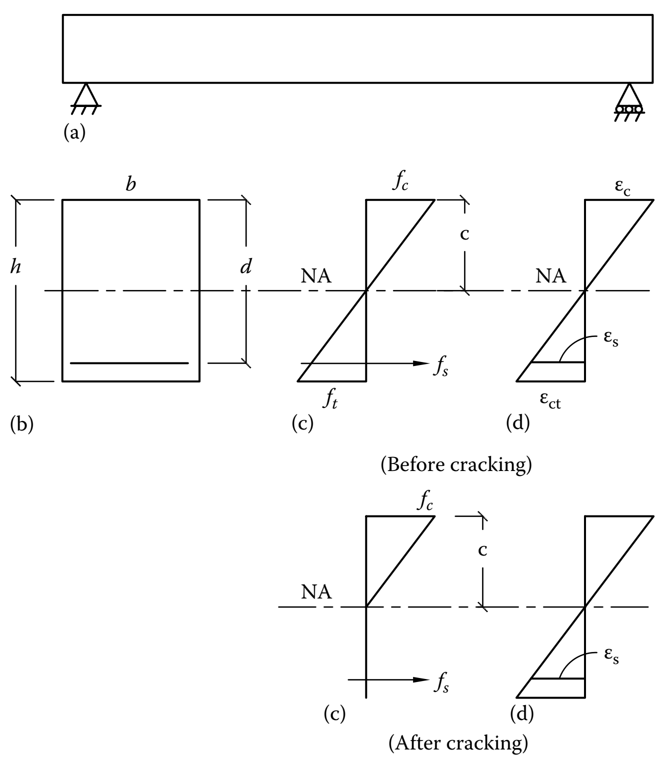


FIGURE 1.20 Bending theory. (a) Span, (b) section, (c) shears, and (d) strain.

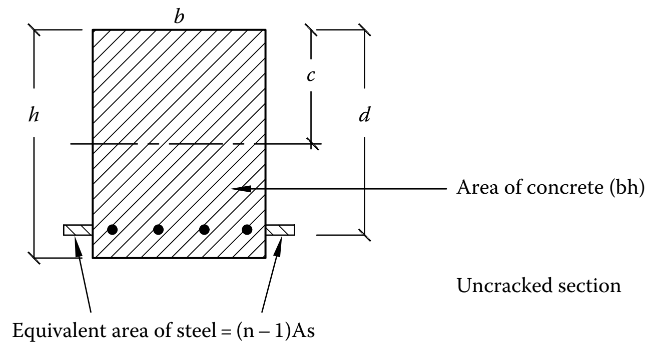


FIGURE 1.21 Transformed section.

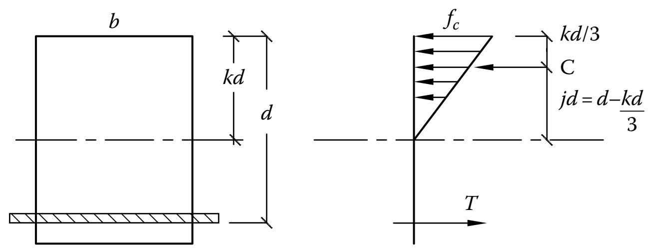


FIGURE 1.22 Elastic state.

$$\text{Compression}(\text{C}) = \frac{f_c'}{2} bkd \tag{1.13}$$

$$\text{Tension}(\text{T}) = A_s f_s \tag{1.14}$$

Since the compressive force acts at the one-third the distance from the top of the triangle of height kd, the lever arm between the tensile and compressive force is $\left(d - \frac{kd}{3}\right)$.

$$\text{Moment}(\text{M}) = Tjd = A_s f_s jd \tag{1.15}$$

Hence,

$$f_s = \frac{M}{A_s jd} \tag{1.16}$$

Also,

$$\text{Moment}(\text{M}) = \text{Cjd} = \frac{f_c}{2} bkdjd = \frac{f_c}{2} kjbd^2 \tag{1.17}$$

Hence,

$$f_c = \frac{2M}{kjbd^2} \tag{1.18}$$

where
- f_s is the stress in steel
- f_c is the stress in concrete

These are not the strengths of steel and concrete.

The methods presented above for the calculation of stresses of uncracked and cracked sections are based on the elastic behavior of the concrete. The current practice for the design of concrete is based on the ultimate strength. Beyond elasticity, stress is not proportional to strain. Ultimate strength is the method specified in the ACI 318-14 code. The method specified in the code is discussed in Chapter 4 of the book.

1.14 SHEAR

Shearing forces are unaligned forces pushing one part of a body in one direction and the other part of the body in the opposite direction. Typically, for a slab or a beam carrying uniform load, the maximum shear force occurs near the support. For an applied concentrated load on a slab or a beam, the maximum shear force can occur under the load, provided the influence of other loads is small.

Shear stress (τ) is the component of stress coplanar with a material cross section. It arises from the force vector component parallel to the cross section. Shear stresses act parallel to shear forces:

$$\tau = \frac{VQ}{Ib} \tag{1.19}$$

where

- τ is the shear stress (psi)
- V is the shear force (lb)
- Q is the first moment of inertia (in^3)
- b is the width of the section (in.)
- I is the moment of inertia (in^4)

For a rectangular section,

$$Q = \left(\frac{bh}{2}\right)\left(\frac{h}{4}\right) = \frac{bh^2}{8} \tag{1.20}$$

$$I = \frac{bh^3}{12} \tag{1.21}$$

Hence,

$$\tau = \frac{V\left(bh^2/8\right)}{\left(\frac{bh^3}{12}\right)\left(b\right)} = \frac{3V}{2bh} = \frac{3V}{2A} \tag{1.22}$$

where

- h is the height of the section (in.)
- A is the cross-sectional area (in^2)

The stress calculated above (τ) is the maximum shear stress. The average shear stress is V/A. The maximum shear stress is 1.5 times the average shear stress for a rectangular section (Figure 1.23).

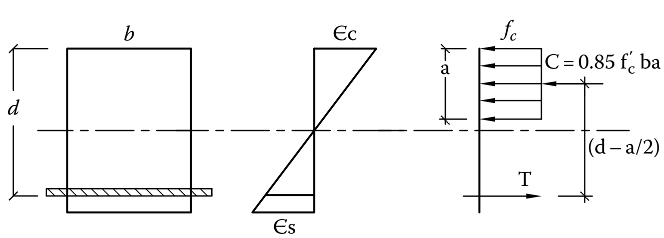


FIGURE 1.23 Rectangular stress block.

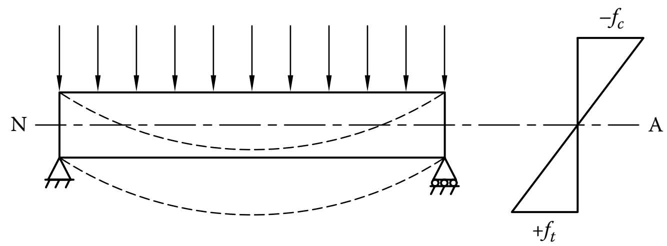


FIGURE 1.24 Simply supported with uniform load.

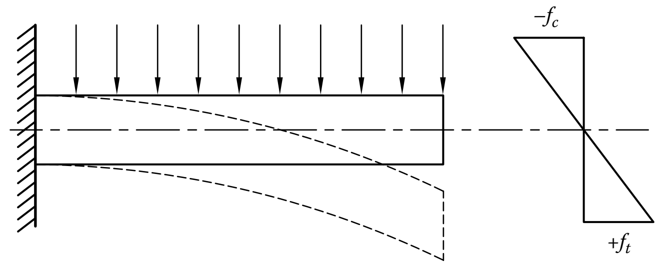


FIGURE 1.25 Cantilever beam with uniform load.

Tensile stresses are developed because of horizontal shear stresses that occur along with the bending stresses, where the concrete cracks. Vertical reinforcement is required to bridge the cracks. This reinforcement is called shear stirrups. Shear forces in a concrete element are resisted by the concrete and the stirrups. In a slab, the shear may be completely resisted by the concrete with no special reinforcement for the shear, but stirrups or headed studs may have to be added in some cases of very high shear forces in the slab. In beams, even if the concrete resists all the shear forces, minimum stirrups are typically added.

Failure due to shear forces in the structural elements is also called diagonal tension failure. This failure is sudden and does not give an alarm to the occupant of the building. Diagonal tension results from the combination of shear stresses and flexural stresses.

In order to understand diagonal tension, consider a simply supported beam subject to a uniform load (Figures 1.24 and 1.25). The compressive stresses are above the neutral axis, and the tensile stresses are below the neutral axis. An element located at the neutral axis does not experience tensile or compressive stresses (Figures 1.26 and 1.27). It is only subjected to shear stresses. As shown in Figure 1.28, the element is subjected to horizontal and vertical shear stresses to maintain equilibrium.

An element located below the neutral axis will be subjected to the horizontal and vertical shear stresses and also tensile stresses due to bending. If all these stresses are combined, they become a pair of compressive and tensile stresses acting perpendicular to each other and are called "principal stresses." The concept of principal stresses was introduced in the strength of material class. The principal stresses are determined by combining the horizontal and vertical shear stresses with the tensile stresses due to bending below the neutral axis and by combining the horizontal and vertical shear stresses with the compressive stresses due to bending above the neutral axis:

$$p = \frac{f}{2} + \sqrt{\frac{f^2}{4} + \tau^2} \tag{1.23}$$

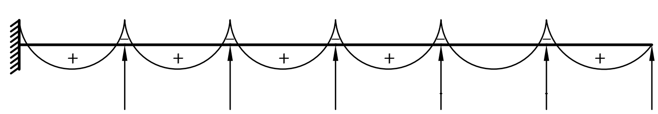


FIGURE 1.26 Continuous beam moment distribution.

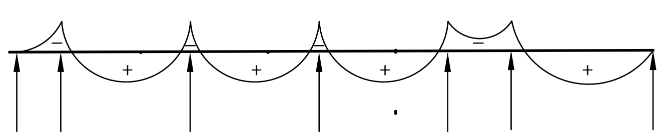


FIGURE 1.27 Continuous beam moment distribution.

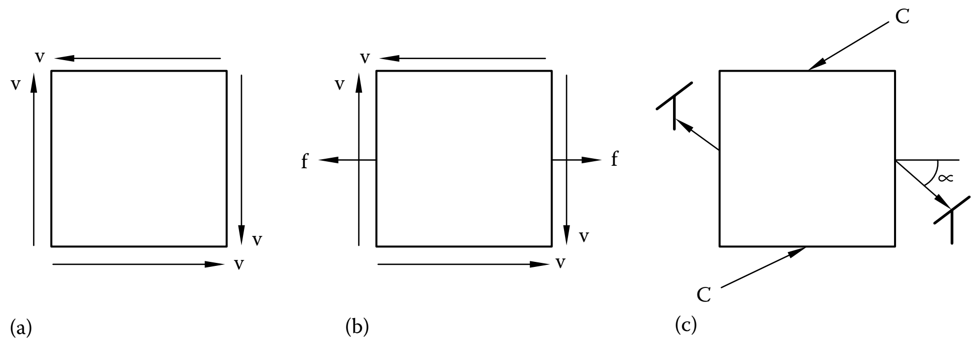


FIGURE 1.28 Shear. (a) Shear stresses at the neutral axis, (b) shear and remote stresses below the neutral axis, (c) combined stresses below the neutral axis.

$$\alpha = \frac{tan^{-1}\left(2\tau/f\right)}{2} \tag{1.24}$$

where

f is the flexural stress (tensile or compressive, determined in Section 1.13 of the book)
τ is the shear stress (determined above in this section)
p is the principal stress
α is the angle of inclination of principal stresses

Since '*f*' and 'τ' vary along the depth of the section, 'p' and 'α' also vary.

The principal stresses are horizontal near the top and bottom surfaces of the beam and are inclined at approximately 45° at the neutral axis. The tensile stresses are not confined to the horizontal stresses at the bottom surface caused due to bending alone, but they are inclined because they are formed due to either shear alone (near the neutral axis) or combined shear and bending (below the neutral axis). These inclined tensile stresses are critical and are called "diagonal tension." They form diagonal cracks.

Consider a simply supported beam with a uniform load and no shear reinforcement. The beam has sufficient tensile reinforcement just above the bottom face of the beam. Tensile stresses are at maximum at the bottom surface of the beam. The compressive stresses are at maximum at the top surface of the beam. Shear stresses are at maximum at the neutral axis. The maximum tensile and compressive stresses are at maximum at the midspan, and shear stresses are at maximum near the support. If we move from the midspan toward the left or right support, principal stresses start acting. As we keep moving from the midspan to the support, the influence of the shear stresses on the principal stresses increases. The bottom reinforcement is designed to resist the tensile stresses due to bending. The principal tensile stresses are inclined at approximately 45°, and the reinforcement does not resist those inclined stresses. When the inclined tensile principal stresses exceed the modulus of rupture of the concrete, cracks are formed perpendicular to the inclined forces. If the simply supported beam is loaded with a concentrated load at the midspan, cracks are seen at the midspan because it becomes a region of high shear stresses and bending stresses. Various types of shear–flexural stress interactions are shown in Figure 1.29.

At the location of high principal stress, the diagonal cracks that start at the bottom go all the way to the top, widen, and split the beam into two and cause a sudden collapse. At the location of low principal stress, diagonal cracks cross the neutral axis and stop well below the compression face. In such a case, the beam does not experience a sudden failure.

Due to the diagonal tension, stirrups are provided even if a beam does not require shear reinforcement. Stirrups perform the function of resisting the shear stress if it exceeds the concrete shear-resisting capacity; they restrict the widening of diagonal tension cracks and tie the longitudinal reinforcement at the top and bottom faces together.

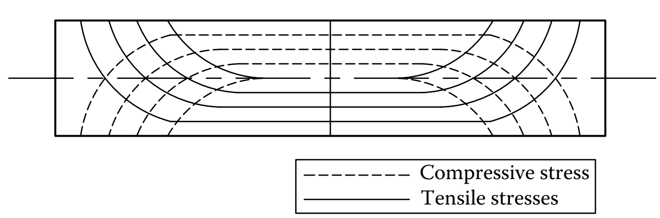


FIGURE 1.29 Diagonal tension.

1.15 TORSION

Torsion occurs in beams when the line of action of the load does not coincide with the shear center of the beam. The shear center is a point on a line parallel to the axis of the beam through which any transverse load must be applied to avoid the twisting of the section. A beam section will rotate when the resultant of the internal shearing force is not collinear with the externally applied force. For a rectangular beam, the shear center will coincide with the centroid of the section. Torsion is a moment that acts upon the beam around its axis in contrast to the bending moment, which acts perpendicular to the axis of the beam.

In actual structures, torsion is combined with shear. Shear and torsion acting on the beam along with combined shear and torsion are demonstrated in Figure 1.30. As can be seen, stresses due to torsion add to the shear stresses on the inside face of the beam, while torsional stresses counteract the shear stresses on the outside face. As explained in Section 4.4.2 of the book, shear reinforcement is provided to make up the difference between the factored shear force and the allowable concrete contribution to shear. For a typical beam, the reinforcement required for the shear and the reinforcement required for the skew bending component of torsion are additive on the inside face. Conversely, the compression due to skew bending on the outside face counteracts the shear force (Figure 1.31).

Torsion may be statically determinate or indeterminate. The statically determinate torsion is also called "primary torsion" or "equilibrium torsion." This type of torsion exists when the external load is supported only by the torsion, as it is required to maintain the equilibrium of the beam. The cantilever in Figure 1.32 demonstrates this concept. The line of action of the load of the slab and the centroid of the beam do not coincide, and due to this eccentricity, a twist acts on the beam. This twist is resisted by the resisting torque provided at the column.

The statically indeterminate torsion is also called "secondary torsion" or "compatibility torsion." Here, the torsional moment can be reduced by redistribution of internal forces after cracking. It is illustrated in Figure 1.33. The external frames with the spandrel beams are supported by the beams perpendicular to the spandrel beams and are spanning between the frames. The fixed end moments of these beams act as torsion on the spandrel beams. Since the spandrel beams have multiple continuous spans, the torsional moment can be redistributed.

Shear stresses due to torsion create diagonal tension and produce diagonal cracking. If not properly designed for torsion, a structural member can suddenly collapse. The ideal design should incorporate moment, shear, and torsion in an interaction equation.

Figure 1.34 shows a simply supported beam subject to torsion. Shear stresses are developed at the four faces of the beam. The principal tensile stress is equal to the principal compressive stress, and both are equal to shearing stress. If the principal tensile stress exceeds the modulus of rupture of the concrete, cracking in the concrete of the beam occurs. If the beam is not adequately reinforced to resist torsion, then it could experience a brittle failure (in the form of a sudden collapse).

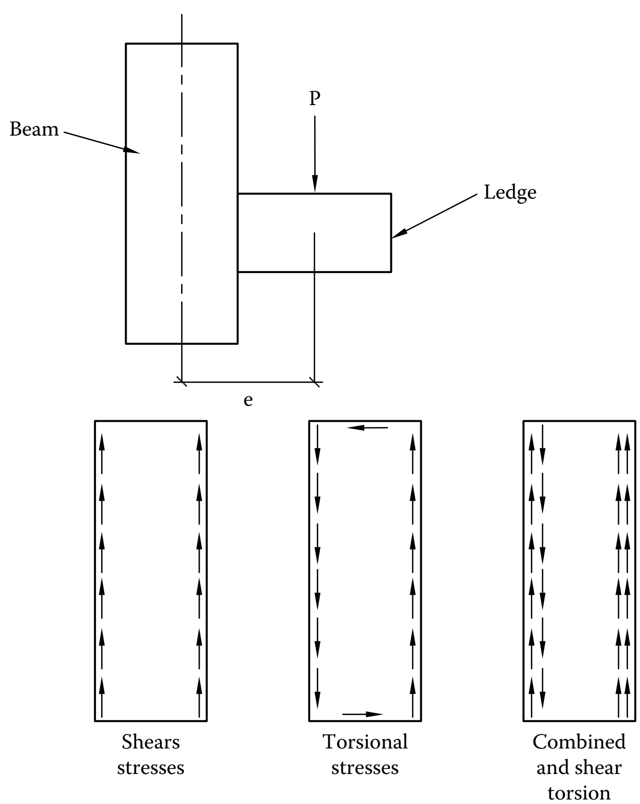


FIGURE 1.30 Combined shear and torsion.

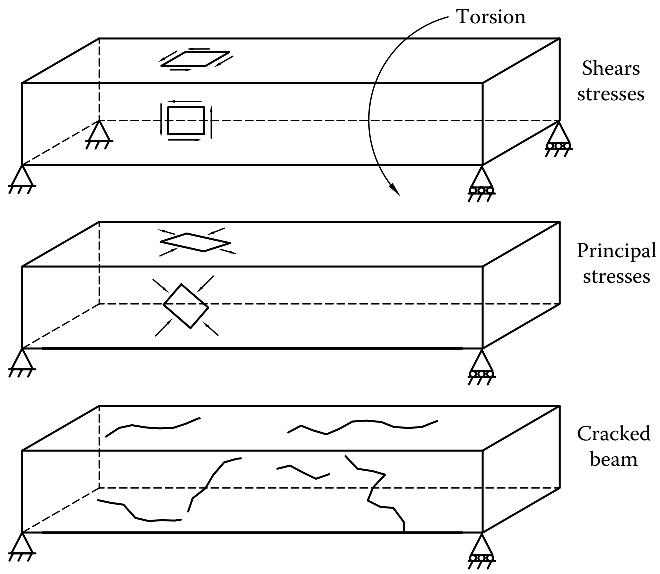


FIGURE 1.31 Shear and torsion.

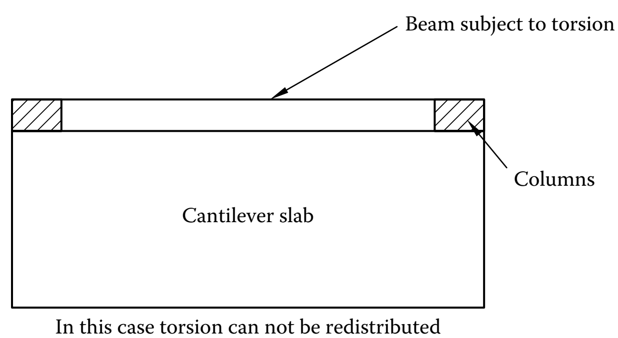


FIGURE 1.32 Primary torsion.

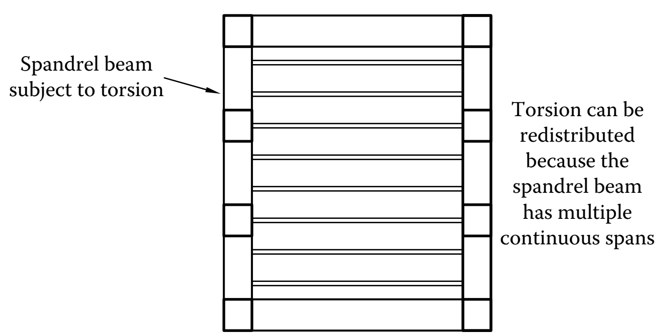


FIGURE 1.33 Secondary torsion.

The behavior of uncracked concrete members is neither completely elastic nor plastic, but the elasticity equations can be used to determine the torsion. In torsional analysis, both solid and hollow sections are considered "tubes." The ACI has adopted the thin-walled tube truss analogy for the design of the torsion.

A closed thin wall section is one in which uninterrupted circuits for a shear flow (q) can be established (Figure 1.35). Shear flow is an internal force per unit length tangential to the wall and resists the applied torsion. If the wall thickness is much smaller than the other dimensions, the shear flow (q) can be assumed to be uniform across the wall thickness (t), provided the thickness does not exceed approximately 20% of the smallest cross-sectional dimension (Figure 1.36).

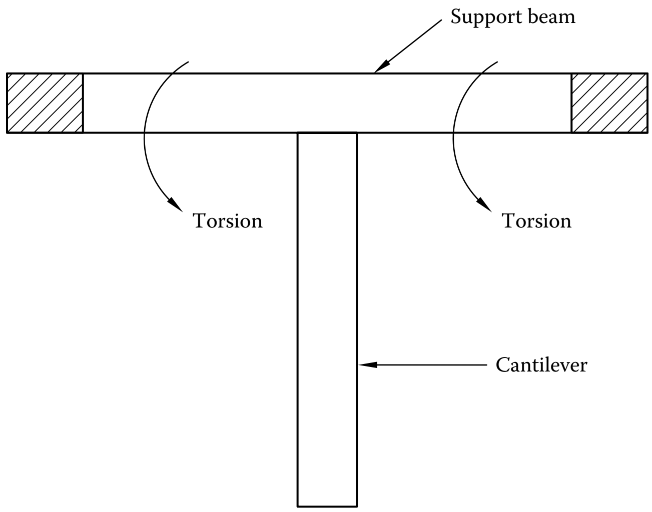


FIGURE 1.34 Beam torsion.

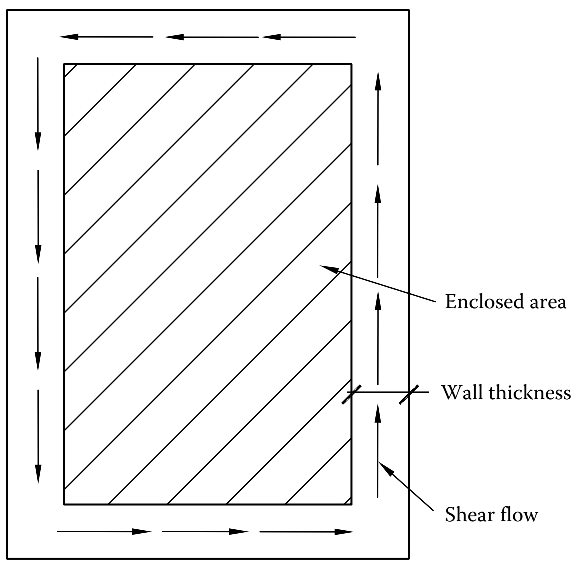


FIGURE 1.35 Shear flow in beams subject to torsion.

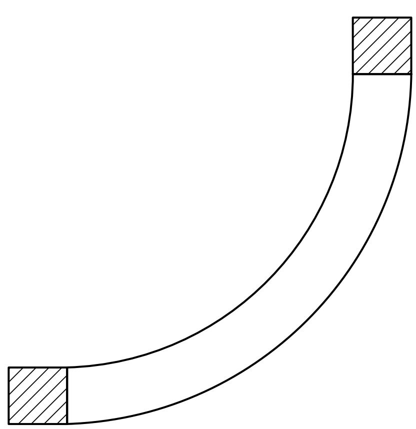


FIGURE 1.36 Curved beam.

$$\text{Wall shear stress}\left(\tau\right)=\frac{q}{t} \tag{1.25}$$

$$\text{Torsion}\left(T\right)=2qA_o \tag{1.26}$$

where A_o is the enclosed area by the shear flow path.[8]

$$q=\frac{T}{2A_o} \tag{1.27}$$

Hence,

$$\left(\tau\right)=\frac{q}{t}=\frac{T}{2A_o t} \tag{1.28}$$

Since q is constant along the contour, plainly the maximum shear stress occurs when the thickness (t) is at its minimum.

Torsion is resisted by the closely spaced stirrups and longitudinal bars. The stirrups are designed for torsion and shear.

1.16 SHEAR FRICTION

As illustrated in Section 1.14, shear is used as a measure of diagonal tension in the design of reinforced concrete members. However, there may be occasions when pure shear acts and causes the failure of the member. The locations of the potential failure due to direct shear can be identified, and the necessary reinforcements are provided for shear friction. Shear friction is a mechanism that takes place during shear transfer across an interface between two concrete members that can slip relative to one another. It arises from the roughness of the interface and the clamping force created by the steel reinforcement across it. The shear friction mechanism occurs in stages, and the concrete component contributes to the majority of the shear friction capacity; the steel component develops only after significant cracking. In all practicality, the concrete and steel components of the shear friction mechanism do not act simultaneously, as depicted in the structural calculations (design example) of shear friction. The interface on which shear acts is called the "shear plane" or "slip plane."

If a concrete structural element is cracked or, in the case of a concrete structural element, cast against an existing concrete structural element and shear is applied at the point of separation, relative slip of the layers causes a separation of the surfaces. The reinforcement placed at the point of separation and developed on both sides experiences tensile stresses, and it resists the slip at the crack or at the surface of the two concrete elements (existing and new). The reinforcement provides a clamping force across the crack or at the surface of the two concrete elements (existing and new).

For the equilibrium of the structure, a compressive stress is required. Shear is transmitted across the crack by friction resulting from the compressive stresses, the interlocking of aggregate roughness on the cracked surfaces, and the dowel action of the reinforcement crossing the surface, as demonstrated in Figure 1.37.

Let us consider an example of a shear friction design. In low-rise buildings, such as single-family homes, there sometimes arises a need to expand the width of the continuous wall footings due to the addition of loads on the walls. The footings can be expanded on one side or both sides. In this example, the footing is expanded on the exterior side of the load-bearing concrete masonry unit wall (Figure 1.38). We will not involve the eccentricity of load acting on the footing due to the expansion of

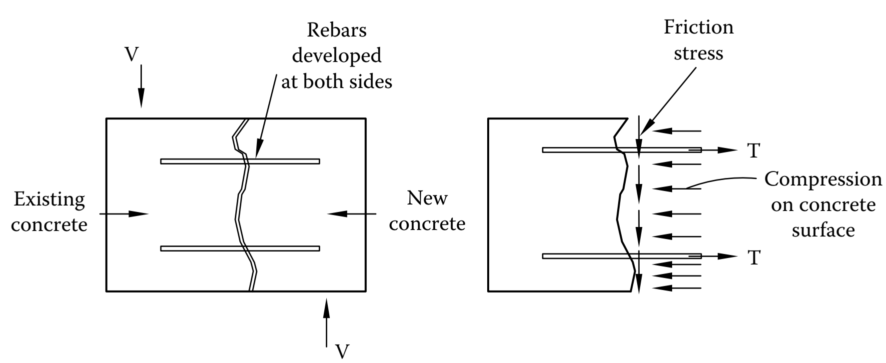


FIGURE 1.37 Shear friction model.

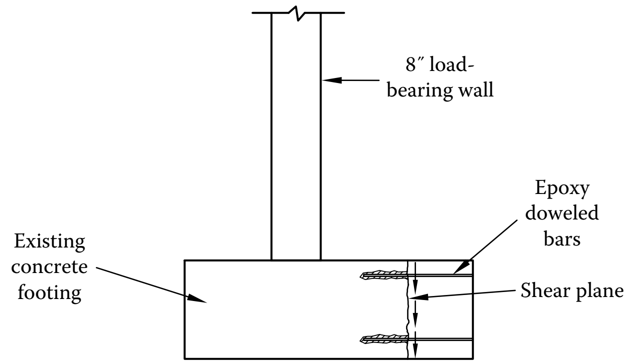


FIGURE 1.38 Expansion of wall footings.

the footing and the moment acting on the base of the footing because of the net upward pressure of the soil in our discussion. During the construction of such footings, the surface of the existing concrete is roughened. Steel bars are dowelled into the existing footing using epoxy grout, and the concrete of the addition to the footing is cast against the existing surface of the footing. The roughening of the existing surface of the footing helps the interlocking of the new concrete cast against the existing concrete. A shear friction is developed at the interface of the new and existing concrete, and dowels experience tensile forces and resist the slip between the two surfaces.

1.17 AXIAL FORCES

Columns are structural elements mainly capable of resisting compression. However, in some cases, moments act on the columns, which require columns to resist tension on one face. Columns can be designed in many shapes, but the most used shapes in the design of columns are square, rectangle, circular, and L-shaped. Columns consist of vertical reinforcement called the longitudinal reinforcement and hoops around the longitudinal reinforcement called lateral ties. Both the longitudinal reinforcement and the lateral ties are enclosed in concrete, which resist the compressive forces. Broadly, columns can be classified into two categories: short columns and slender columns. In building types (A) and (B) introduced in Section 1.5 of the book, the columns are short columns because the height of the floor does not exceed a certain limit based on the lateral dimensions of the columns, and the columns are tied by beams at each level. The design of slender columns is discussed in Chapter 8 of the book. If short columns are subject to only compressive forces and no moment acts on them, then the compressive capacity of the column is

$$P = f_c'\left[A_g + (n-1)A_{st}\right] \tag{1.29}$$

where
A_g is gross area of concrete (in^2)
A_{st} is area of longitudinal steel (in^2)
n is modular ratio

The compressive strength of the short column subject to only axial compression is determined by adding the compressive strength provided by the concrete and the transformed area of the longitudinal reinforcement. The concept of the transformed area was introduced in Section 1.13 of the book.

The lateral reinforcements serve many purposes. They hold the longitudinal bars together. Before placing the column reinforcement at the desired location at the construction site, the longitudinal reinforcement and the lateral ties are wired together. The lateral reinforcement in slender columns helps the highly stressed longitudinal reinforcement from bursting through the concrete.

1.18 AXIAL FORCE PLUS BENDING

We discussed axial forces on short columns in the previous section. This condition rarely exists. Columns that are part of a frame are subject to moments from the beam, even if there is no lateral load acting on the structure. Columns that are a part of the Main Wind Force Resisting System are subject to moment. Columns supporting the eccentric loads are subject to moment. These eccentricities can be deliberately applied on the columns during the design phase or can be introduced due to construction defects.

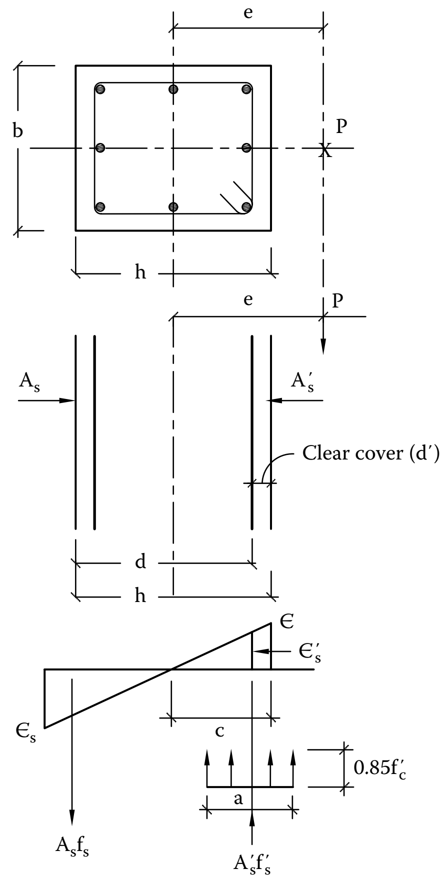


FIGURE 1.39 Eccentrically loaded columns.

If a moment 'M' and an axial load 'P' are applied on the column, then the moment can be converted to axial load 'P' with an eccentricity e (=M/P). If the eccentricity is small, then the column will generally be subjected to compression over the entire cross section, but if the eccentricity is large, then the opposite face of the column will be subjected to tension. The longitudinal reinforcement provided at that face should be capable of resisting this tension.

Consider a rectangular column with dimensions b × h. Consider a compressive force P acting at an eccentricity 'e' from the centroid of the column along the side 'h.' Consider the column reinforced with A_s' on the face near the load 'P' and A_s on the face away from the load 'P' (Figure 1.39):

$$P = f_c' x b + A_s' f_s' - A_s f_s \tag{1.30}$$

where x is an arbitrary value based on the triangular stress block. Replacing x with an equivalent rectangular stress block value,

$$P = 0.85 f_c' a b + A_s' f_s' - A_s f_s \tag{1.31}$$

Taking moment about the centerline of the section,

$$M = Pe = 0.85 f_c' a b \left[\frac{h}{2} - \frac{a}{2}\right] + A_s' f_s' \left[\frac{h}{2} - d\right] + A_s f_s \left[d - \frac{h}{2}\right] \tag{1.32}$$

A column strength interaction diagram is a graph plotted with applied compressive load on the Y-axis and the moment on the X-axis. This diagram is based on the zero-to-infinity range of the eccentricities. For any eccentricity, P and M values are unique at failure. The radial lines in the strength interaction diagram represent the eccentricities. The vertical axis represents zero eccentricity, where the capacity of the column is P_0, and the horizontal axis represents infinite eccentricity, where the

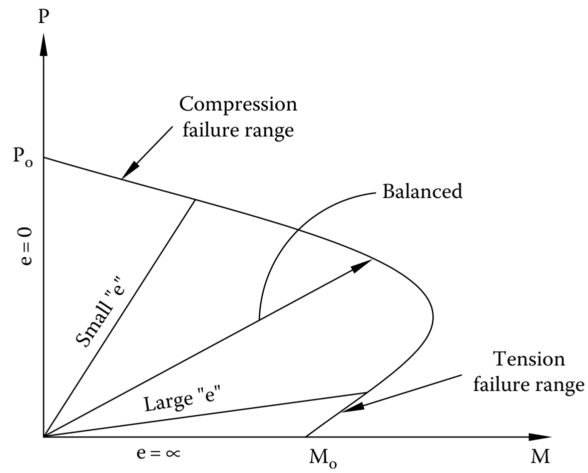


FIGURE 1.40 Column interaction diagram.

capacity of the columns is M_0. In order to construct the strength interaction diagram for a column, different values of 'c' are selected. Using the strain diagram in Figure 1.40,

$$\text{Strain in steel on the far end } (\varepsilon_s) = \frac{\varepsilon_0 (d - c)}{c} \tag{1.33}$$

$$\text{Stress in steel on the far end } (f_s) = \frac{E_s \varepsilon_0 (d - c)}{c} \leq f_y \tag{1.34}$$

$$\text{Strain in steel near the load } (\varepsilon_s') = \frac{\varepsilon_0 (c - d')}{c} \tag{1.35}$$

$$\text{Stress in steel near the load } (f_s') = \frac{E_s \varepsilon_0 (c - d')}{c} \tag{1.36}$$

$$\text{Compressive force } C = 0.85 f_c' ab \tag{1.37}$$

Using these values of f_s and f_s' in the above equations for P and M, the values P_0 and M_0 are calculated.

A balanced failure mode is defined as when failure occurs at a point when the concrete reaches its ultimate strain and steel reaches the yield strain. The eccentricity is e_b, load is P_b, and the corresponding moment is M_b. In the interaction diagram, it is a dividing point between a compressive failure and a tensile failure. Compressive failure occurs at small eccentricities, and tensile failure occurs at large eccentricities. For a balanced failure,

$$c = c_b = \frac{d(\varepsilon_u)}{\varepsilon_u + \varepsilon_y} \tag{1.38}$$

$$a = a_b = \beta_1 c_b \tag{1.39}$$

A balanced failure mode provides a useful point on the interaction diagram to assess the safety measures while designing a column. In the interaction diagram, the curve above the balanced failure mode represents compression failure and the curve below represents tension failure. In the region of the compression failure, if the axial force is high, the section will be able to resist small moments before it fails because failure occurs due to the overstraining of the concrete. If the axial compressive force is large, then it leaves little room for the compressive stresses caused due to bending. In the region of tension failure, if the axial force is high, then the section will resist larger moments before it fails because the yielding of steel causes failure. The logic is that the yielding of the steel occurs due to bending, and compressive forces only help in negating the tensile stresses in the steel. Hence, in the region of tension failure, a reduction of the compressive force may result in failure. The designer needs to be careful and check the column for high moments and low compressive force also from the load combinations because this may be the critical combination.

In several cases, columns are not subjected to moments about one axis only. They are subjected to moments about both the axes. Such cases arise in the corner columns or interior columns of a building where the main force-resisting frames run in both directions. The design of columns subject to biaxial bending is discussed in Chapter 8 of the book.

1.19 LOAD PATHS OF STRUCTURE

The structural system and the load path are defined in section 4.4 of the code. Before learning how to design structures, it is very important for an engineer to learn how structures behave as a whole and how various components contribute to resist the intended loads and transfer them to the foundations and the ground. The structure is an essential part of any building, and its conceptual choice is a part of the architectural design. The structure is a part of the building and should not be conceived in isolation but as a part of the whole design. However, the role of the structure in a building is restricted to strength and serviceability. Strength provides safety to the occupants, and serviceability provides comfort. The structure should be strong enough to support the intended loads without failing or collapsing, and it should not have excessive deflections, vibrations, and local deformation to be serviceable to the occupants (by providing comfort and a sense of safety). A skyscraper can sway severely and cause the occupants to be sick (much like seasickness) yet be perfectly sound structurally. This building is in no danger of collapsing, yet since it is obviously no longer fit for human occupation, it is considered to have exceeded its serviceability limit state. There is no unique solution to the structural problem in a building. The best solution to the structural problem is based on the economy and the least hindrance to the architectural, mechanical, electrical, and plumbing design without compromising the strength, serviceability, and durability of structure.

Load path is the term used to describe the path by which loads are transmitted to the foundations. Different structures have different load paths. Some structures have only one path, while others have several (redundancy). The load path is simply the direction in which each consecutive load will pass through connected members. The sequence commences at the highest point of the structure, working all the way down to the footing system, and ultimately transferring the total load of the structure to the foundation. Finally, the lowest structural member must be strong enough to support all the members above it. This is why engineers often design the uppermost members first and progressively work their way down the structure following the load path.

In order to demonstrate the concept of load path, let us consider a multistoried concrete building in the simplest form. Primarily, two types of loads act:

- Gravity loads consisting of the weight of the concrete elements, superimposed dead and live loads, and snow and rain loads on the roof
- Lateral loads consisting of the wind and earthquake forces

The load path is evaluated from top to bottom because, ultimately, the loads have to be transferred to the ground. The roof consists of shallow plates in the horizontal plane supported directly either on the vertical structural elements called "columns" or on the stiffer horizontal structural elements called "beams." The loads that can act on the roof are the self-weight of the roof, superimposed dead load such as roof finishes, live load (which in most cases is smaller than the live load on floor slabs unless the roof is used as a recreation deck or a pool deck), or any of the following loads present: snow load, loads of rooftop mechanical equipment, rain load, etc. The roof slab is designed for bending, shear, and punching shear if it is directly placed on the columns. If the roof slab is directly placed on the columns, then the load is transferred to the columns according to the geometric tributary area of the slab. If the roof slab is supported on the beams, then the load is transferred to the beams depending on the type of slab. The slabs can be one way, two way, or cantilever. In the case of a one-way slab, half the load is transferred to each of the two parallel beams supporting the slab. For a two-way slab, the trapezoidal portion of the load is transferred to the two parallel beams along the long side of the trapezoid and the triangular portion of the load is transferred to the two parallel beams along the short side of the slab. A cantilever slab transfers the entire load to the beam supporting it. If the slab is not rectangular or square in shape, then the load is transferred according to the geometry of the beam, and the engineer has to use experience and judgment while calculating the loads transferred to the supporting beams. The beams transfer the load to the columns supporting them. There may be special cases of load transfer due to architectural requirements. Columns from the upper level may not be taken till the ground. The columns get supported on slabs or beams, which are termed "transfer slab" or "transfer beam," and have to be designed to resist these concentrated loads (Figures 1.6 and 1.7).

In the reinforced concrete buildings, two systems are popularly used to resist and transfer the lateral loads to the ground—moment-resisting frames and shearwalls or a combination of the two. The moment-resisting frames consist of beams and columns, which resist both the gravity and lateral loads. The beams are designed for bending and shear, and the columns are designed for axial forces and bending. Other forces such as torsion and axial loads may also act on the beams. Typically, the columns are braced in both directions at the joints with beams. If the lateral loads are very high and the column sizes are restrictive, then cross bracings may be added to the columns. In the shear wall system, the beams and columns resist the gravity load, and the shearwalls resist the lateral loads and some gravity loads depending on their location and configuration. In the case of wind loads, the horizontal shears to be resisted by the shearwalls are cumulative from the top floor to the ground.

The load is finally transferred to the ground through the footings, which support the columns in the walls.

1.20 GENERAL ARRANGEMENT OF THE CODE

There is a major revision in the concrete code "American Concrete Institute Building Code Requirements for Structural Concrete—ACI 318-14." The code has been completely rearranged as compared to the previous versions of the code. The code is divided into 10 parts with 27 chapters.

Part 1—"General"—has four chapters. Chapter 1 of the code is the general chapter defining the applicability of the code, methods of interpretations of the code, the role of the building official and the design professional, construction documents and design records, testing, inspections, and approval of the special system. The design professional, usually called the "Engineer of Record," is ultimately responsible for the safety of the structure. The building official implements the building code and verifies the compliance of the design documents and the construction at site with the prevailing codes. When the drawings are submitted to the building department, the building official or her/his authorized representative ensures that the various concrete structural elements used in the building are in compliance with the relevant requirements of the code. In Miami-Dade County, the structural plans examiner reviewing the structural drawings needs to be a licensed engineer having taken her/his engineering license examination under the structural discipline.[1] In most parts of the nation, this is not a requirement and the building official relies on the licensed design professional for the adequacy of the design. Typically, the building department personnel have sovereign immunity for minor incompetency, and hence, the liability of the design lies on the Engineer of Record. Ultimately, the safety of the building is the responsibility of the licensed design professional. The structural design documents prepared for the construction have to be approved by the building department of the local jurisdiction. Any revisions made to the drawings after the permit is issued have to be submitted for the revision permit from the building department. The inspections performed at the site usually include checking the size and spacing of the reinforcement, cover to the reinforcement, orientation of the reinforcement, and the thickness of the member. Testing includes compressive strength and slump tests. Inspection and testing are discussed in Chapter 12 of the book. Chapter 2 of the code deals with the notation and terminology of the code, and chapter 3 of the code includes the other standards (such as the ASTM standards) that are used in the preparation of the code. The reader is advised to review these chapters carefully because the code depends on these chapters, and they should not be taken lightly. Chapter 4 of the code discusses the materials (Chapter 2 of the book); structural systems and load path (Chapter 1 of the book); seismic force–resisting system (beyond the scope of this book); strength, serviceability and durability (discussed in Chapters 4 and 5 of the book); sustainability; structural integrity (discussed in design chapters of the book); fire resistance (a building code requirement in accordance with the code adopted by the local jurisdiction, e.g., the International Building Code or the Florida Building Code); requirements for precast concrete systems, prestressed concrete systems, composite concrete flexural members, composite steel and concrete construction, and structural plain concrete systems (beyond the scope of this book); and construction and inspection (Chapter 12 of the book), and strength evaluation.

Part 2—"Loads and Analysis"—has two chapters. Chapter 5 of the code specifies the load combinations to be used (Chapter 3 of the book), and chapter 6 of the code specifies the structural analysis methodology (discussed in the various design chapters of the book).

Part 3—"Members"—has eight chapters. Chapters 7 and 8 of the code specify the design requirements of one-way and two-way slabs, respectively (Chapter 6 of the book). Chapter 9 of the code specifies the design requirements of beams (Chapter 7 of the book). Chapter 10 of the code specifies the design requirements of columns (Chapter 8 of the book). Chapter 11 of the code specifies the design requirements of walls (Chapter 9 of the book). Chapter 12 of the code specifies the design requirements of diaphragm (beyond the scope of the book). Chapter 13 of the code specifies the design requirements of foundations (Chapter 10 of the book). Chapter 14 of the code specifies the design requirements of plain concrete (not used in the book).

Part 4—"Joints/Connections/Anchors"—has three chapters. Chapter 15 of the code specifies the design requirements of beam–column and slab–column joints. Chapter 16 of the code specifies the design requirements of the connections between members. Chapter 17 of the code specifies the design requirements of anchoring to the concrete. These chapters of the code are addressed in the various design chapters of the book.

Part 5—"Earthquake Resistance"—has only chapter 18 that deals with the design of concrete structures in seismic zones. Seismic design is not dealt with in this book.

Part 6—"Materials and Durability"—has two chapters. Chapter 19 of the code specifies concrete properties and durability requirements. Chapter 20 of the code specifies the requirements of non-prestressed bars/wires; prestressing strands; structural steel, pipes, and tubes; and headed shear stud reinforcement. The code requirements for concrete and reinforcement steel are discussed later in Chapters 4 and 5 of the book.

Part (7)—"Strength and Serviceability"—has four chapters. Chapter 21 of the code specifies strength reduction factors to be used in design (explained in various design chapters of the book). Chapter 22 of the code deals with flexural strength, axial strength, combined flexural and axial strength, one-way shear strength, two-way shear strength, torsional strength, and the bearing and shear friction of the concrete structural elements. These are dealt with in various design chapters of the book. Chapter 23 of the code specifies the strut-and-tie models of the concrete structural elements (not used in this book). Chapter 24 of the code specifies the serviceability requirements of the concrete structural elements, including deflection, distribution of steel in one-way slabs and beams, shrinkage and temperature reinforcement, and permissible stresses in prestressed concrete. Apart from the requirements of the prestressed concrete, the rest of the requirements are discussed in the various design chapters of the book.

Part (8)—"Reinforcement"—has only chapter 25, which specifies the reinforcement details. This is explained and demonstrated in Chapter 11 of the book.

Part (9)—"Construction Documents and Inspections"—has only chapter 26. Construction documents are discussed in Chapter 12 of the book, and inspections are also discussed in Chapter 12 of the book. For a design engineer, construction documents are very important for a good, economical, safe structure to be constructed according to the schedule of construction. An engineer may perform a very good analysis and design, but if the documentation is improper and inadequate, then problems arise at the site. This may lead to delay and changes of orders by the contractor, which in many cases goes to litigations.

Part 10—"Strength Evaluation of Existing Structures"—has only chapter 27.

1.21 ASSIGNMENTS

1. A simply supported beam of span 15' carries a factored load of 3,000 lbs-feet^{-1}. The beam is reinforced with 3#5 at the bottom with a 1.5" clear cover. Size of the beam—12 in^2 × 24 in^2; $f_c' = 4{,}000$ psi; $f_y = 60{,}000$ psi.
   a. Calculate the maximum bending moment and shear force.
   b. Draw the bending moment and shear force diagram.
   c. Show the location of the maximum bending moment and shear force.
   d. Calculate the maximum and average shear stresses.
   e. Calculate the stresses in the concrete at the top fiber and the stresses at the location of the reinforcement.
2. A beam with a rectangular section has dimensions 8 in^2 × 24 in^2. The effective depth of the beam is 22 in^2. The beam is reinforced with 3#6 bars of $f_y = 60$ ksi. The strength of concrete (f_c') is 4 ksi. Determine the ultimate moment at which the beam will fail.

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2 Properties of Materials Used in Reinforced Concrete

2.1 MATERIALS OF REINFORCED CONCRETE

The main materials used in reinforced concrete are cement, aggregates, water, steel, and admixtures. In its simplest form, concrete is a mixture of adhesive paste and aggregates. The paste consists of cement and water. It coats the surface of the fine and coarse aggregates. Hydration, a chemical reaction, hardens the paste, which then gains strength to form the rock-like mass known as concrete. Concrete is plastic and malleable when freshly mixed, and it is strong and durable when it hardens. Concrete is said to be malleable in the fresh state because it is pliable and ductile and can be molded into any shape.

2.2 CEMENT

Cement is a binder, a substance that sets and hardens, and can bind other materials together. The various cements specified in section 3.2.1 of the code are Portland cement (ASTM C150), blended hydraulic cement (ASTM C595), expansive hydraulic cement (ASTM C845), hydraulic cement (ASTM C1157), slag cement (ASTM C989), and silica fume (ASTM C1240). The most used cement in the construction of buildings is Portland cement, specified by ASTM C150.

ASTM C150 covers eight types of Portland cement, which consists of Portland cement clinker; water or calcium sulfate, or both; limestone; processing additions; and air-entraining addition for air-entraining Portland cement; aluminum oxide, ferric oxide, magnesium oxide, sulfur trioxide, tricalcium silicate, dicalcium silicate, tricalcium aluminate, and tetracalcium aluminofernite. The eight types of Portland cement specified in ASTM C150 are as follows:

Type I For use when the special properties specified for any other type are not required.
Type IA Air-entraining cement for the same use as Type I, where air entrainment is desired.
Type II For general use, especially when moderate sulfate resistance is desired.
Type IIA Air-entraining cement for the same use as Type II, where air entrainment is desired.
Type II(MH) For general use, especially when moderate heat of hydration and moderate sulfate resistance are desired.
Type II(MH)A Air-entraining cement for the same use as Type II(MH), where air entrainment is desired.
Type III For use when high early strength is desired.
Type IIIA Air-entraining cement for the same use as Type III, where air entrainment is desired.
Type IV For use when a low heat of hydration is desired.
Type V For use when high sulfate resistance is desired.

Type I Portland cement is suitable for all uses where special properties are not required, like when concrete is exposed to sulfate ions in soil or water. It is used in the construction of pavements, sidewalks, buildings, bridges, railway structures, tanks, reservoirs, culverts, sewers, water pipes, and masonry units. Type II cement is used in structures such as large piers, heavy abutments, and heavy retaining walls in warm places as it reduces the temperature rise. Type III cement is used when it is necessary to provide early compressive strength to concrete; like in fast-paced construction where forms are needed to be removed within a week. In cold weather, it permits a reduction in controlled curing period. Type IV cement is used where the rate and amount of heat generated must be minimized, but it develops strength at a lower rate. It is used in structures such as large gravity dams. Type V cement is used in concrete that needs to have high sulfate resistance, in locations where soil or groundwater has a high sulfate content. Type IA, IIA, and IIIA are air-entraining cements where small quantities of air-entraining materials are added during the manufacturing process to provide improved resistance to freezing and thawing processes in cold areas. Refer to Section 2.7 of the book for discussion on air entrainment.[1]

Quality control of Portland cement is performed according to ASTM specifications, which covers both chemical and physical requirements of cement. The chemical requirements are not very stringent because cements with different chemical compounds can have similar physical behavior. However, the physical requirements have more significance. The final step in the production of cement is the grinding of the clinker, and the degree to which the clinker is ground is called "fineness." The rate of hydration increases with fineness, and it leads to high strength and heat generation. Hydration takes place on the cement particle surface. Finer particles will be more completely hydrated. Increasing the fineness decreases the amount of bleeding but also requires more water for workability, which can result in an increase in dry shrinkage. Increased fineness requires more gypsum to control setting. Fineness is measured by the specific surface area of the particles. The two ASTM tests for fineness are as follows:

- Wagner turbidimeter is used to measure the specific surface area from a suspension of the cement in a tall glass container. The test is based on Stokes' law that states a sphere will obtain a constant velocity under the action of gravity (ASTM C115 / C115M-10e1—"Standard Test Method for Fineness of Portland Cement by the Turbidimeter").
- Blaine air permeability apparatus test is based on the relationship between the surface area in a porous bed and the rate of fluid flow (air) through the bed. The test is compared to a standard sample determined by the U.S. Bureau of Standards. The Blaine method is used more often in practice because of the ease of maintenance of the apparatus and the simplicity of the procedure. However, in cases of dispute, the Wagner method governs (ASTM C204-16—"Standard Test Methods for Fineness of Hydraulic Cement by Air-Permeability Apparatus").

Tests are also performed on cement paste. Time of setting and soundness are the two important properties of cement paste. Soundness depends on the water content of the cement, which is measured in terms of "normal consistency." A cement paste is said to be of normal consistency when a 300 g, 10 mm diameter Vicat needle penetrates 10 + 1 mm below the surface in 30 seconds (ASTM C191-13—"Test Methods for Time of Setting of Hydraulic Cement by Vicat Needle").

Two arbitrary points of no real significance are used to develop general relationships between addition of water and strength gain for quality control purpose. The initial setting point is when the paste begins to stiffen usually in 2–4 hours. Initial setting occurs when a 1 mm needle penetrates 25 mm into the cement paste. The final setting point is when the paste has the ability to withstand the needle load, usually in 5–8 hours. Final set occurs when there is no visible penetration. In the Vicat needle apparatus, the cement paste is mixed such that Vicat needle penetrates 32 ± 4 mm after 20 seconds. The final penetration is measured at 5 minutes. The result is a percentage of (final penetration/initial penetration) × 100%.

Unsoundness is the characteristic of excessive volume change of the cement paste after setting. It may appear many months or years after setting. Therefore, any test for unsoundness must detect the potential for this type of failure. Two standard tests are the Le Chatelier test and the autoclave expansion test.

Le Chatelier test is designed to test for expansion due to excessive lime. The device is filled with cement of normal consistency, covered with glass plates, and immersed in water at 20°C + 1°C for 24 hours. The distance between the indicator points is measured, and the device is returned to the water and brought to boiling point in 25–30 minutes and boiled for 1 hour. The device is cooled and the indicator points are measured again. The difference in the readings cannot exceed 10 mm.

In the autoclave expansion test, cement paste of normal consistency is molded and cured for 24 hours. It is then measured and placed in an autoclave, and the temperature is increased for 45–75 minutes until a pressure of 295 psi is achieved. It remains

FIGURE 2.1 Vicat needle apparatus to measure consistency of cement.

for 3 hours and then is cooled in the autoclave for 1–½ hours, then 15 minutes in the water, and 15 minutes in the air. Thereafter, its length is measured. The change in length must be less than 0.80% to be acceptable. Autoclave is a strong, heated container used for chemical reactions and other processes using high pressure and temperature.

Heat of hydration of the cement paste is determined by the heat of solution method. The heat of solution of dry cement is compared to partially hydrated cement at 7 and 28 days. Heat of hydration is the difference between the dry and the partially hydrated cements.

Other tests used to determine the properties of Portland cement include: air content if mortar, chemical analysis, strength, false set, fineness by air permeability, time of setting by Gillmore needles, sulfate resistance, calcium sulfate, and compressive strength.

2.3 AGGREGATES

Aggregates are inert granular materials such as sand, gravel, or crushed stone. Aggregates need to be clean, hard, strong particles free of absorbed chemicals or coatings of clay and other fine materials, which could cause the deterioration of concrete. In terms of its volume, concrete usually consists of 60%–75% of aggregates (fine and coarse). Section 26.4.1.2.1(a)(4) of the code requires that the normal weight aggregates comply with ASTM C33 and the lightweight aggregates comply with ASTM C330. In accordance with the ASTM C29, lightweight aggregates have a bulk density of 70 pcf.

Fine aggregates (usually natural sand) shall have particles passing through a 3/8-in. sieve. Coarse aggregates (usually produced by crushing quarry rock, boulders, cobbles, or large-sized gravel) shall not be larger than 1/5 the narrowest dimension between sides of form, 1/3 the depth of the slab, and 3/4 the clear spacing between the individual reinforcing bars or wires, bundles of bars, individual tendons, bundled tendons, or ducts (section 26.4.2.1(4) of the code).

ASTM C33 defines the requirements for grading and quality of fine and coarse aggregates for use in normal weight concrete. It defines the sampling and test methods of grading and fineness modulus test, organic impurities test, effect of organic impurities on strength test, soundness test, clay lumps and friable particles test, coal and lignite test, bulk density of slag test, abrasion of coarse aggregate test, reactive aggregate test, freezing and thawing test, and chert test method.

ASTM C330 covers lightweight aggregates intended for use in structural concrete in which the prime consideration is reducing the density while maintaining the compressive strength of the concrete. The aggregates shall be composed predominantly of lightweight cellular and granular inorganic material. Lightweight aggregates should not contain excessive amounts of deleterious substances; they should conform to the specified values of organic impurities, aggregate staining, aggregate loss of ignition, clay lumps and friable particles, loose bulk density, compressive strength, drying shrinkage, popouts, and resistance to freezing and thawing.

The main characteristics of concrete aggregates are grading (fine aggregate and coarse aggregate), fineness modulus, combined aggregate grading, particle shape and surface texture, bulk density and voids, relative density (specific gravity), density, absorption and surface moisture, bulking, resistance to freezing and thawing, wetting and drying properties, abrasion and skid resistance, strength and shrinkage, resistance to acid and other corrosive substances, fire resistance and thermal properties, potentially harmful materials, alkali–aggregate reactivity, alkali–silicon reaction, and alkali–carbonate reaction.

The main ASTM tests required for the aggregates are listed in the following table.

ASTM Standard	Definition and Significance
C131	"Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine" Used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions.
C535	"Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine" Used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions.
C779	"Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces" Used to evaluate the effects on abrasion resistance of concrete, concrete materials, and curing or finishing procedures and for quality acceptance of products and surfaces exposed to wear.
C666	"Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing" Determines the effects of variations in both properties and the conditioning of concrete in resistance to freezing and thawing cycles specified in the particular procedure.
C682	"Standard Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures" Evaluation of frost resistance of coarse aggregates in air-entrained concrete.
C88	"Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate" Estimation of the soundness of aggregates for use in concrete and other purposes.

(*Continued*)

ASTM Standard	Definition and Significance
C295	"Standard Guide for Petrographic Examination of Aggregates for Concrete" Determination of the physical and chemical characteristics of the material that may be observed by petrographic methods and that have a bearing on the performance of the material in its intended use; description and classification of the constituents of the sample; determination of the relative amounts of the constituents of the sample that are essential for proper evaluation of the sample when the constituents differ significantly in properties that have a bearing on the performance of the material in its intended use; and the comparison of samples of the aggregate from new sources with samples of the aggregate from one or more sources, for which test data or performance records are available.
D3398	"Standard Test Method for Index of Aggregate Particle Shape and Texture" Determination of the particle index of aggregate as an overall measure of particle shape and texture characteristics.
C117	"Standard Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing" The results of this test method are included in the calculation in Test Method C136.
C136	"Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates" Used to determine the grading of materials proposed for use as aggregates or being used as aggregates.
C1137	"Standard Test Method for Degradation of Fine Aggregate Due to Attrition" Provides a procedure for indicating the degree to which a fine aggregate may be subject to degradation due to the mixing and agitation of Portland cement concrete.
C1252	"Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete"
C29	"Standard Test Method for Bulk Density ('Unit Weight') and Voids in Aggregate" Determination of bulk density values that are necessary for use in many methods of selecting proportions for concrete mixtures.
C127	"Standard Test Method for Relative Density (Specific Gravity) and Absorption of Coarse Aggregate" Absorption values are used to calculate the change in the mass of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it is deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential.
C128	"Standard Test Method for Relative Density (Specific Gravity) and Absorption of Fine Aggregate"
C70	"Standard Test Method for Surface Moisture in Fine Aggregate" Convenient procedure for field or plant determination of moisture content of fine aggregate if specific gravity values are known and if drying facilities are not available.
C566	"Standard Test Method for Total Evaporable Moisture Content of Aggregate by Drying" Determination of the percentage of evaporable moisture in a sample of aggregate by drying both surface moisture and moisture in the pores of the aggregate.
C39	"Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens" Determination of compressive strength of cylindrical specimens prepared and cured.
C78	"Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)" Determination of the flexural strength of specimens prepared and cured.
C125	"Standard Terminology Relating to Concrete and Concrete Aggregates" Compilation of the definitions of terms as they are used in the ASTM standards related to subject matter.
C294	"Standard Descriptive Nomenclature for Constituents of Concrete Aggregates" Descriptive nomenclature that provides information on terms commonly applied to concrete aggregates.
C40	"Standard Test Method for Organic Impurities in Fine Aggregates for Concrete" Determination of the acceptability of fine aggregates with respect to the requirements of Specification C33, which relate to organic impurities.
C87	"Standard Test Method for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar" Determination of the acceptability of fine aggregates with respect to the requirements of Specification C33 concerning organic impurities.
C123	"Standard Test Method for Lightweight Particles in Aggregate" Determination of conformance with provisions of Specification C33 pertaining to the amount of lightweight material in fine and coarse aggregates.
C142	"Standard Test Method for Clay Lumps and Friable Particles in Aggregates" Determination of the acceptability of aggregate with respect to the requirements of Specification C33.
C227	"Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)"
C289	"Standard Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)" Covers chemical determination of the potential reactivity of an aggregate with alkalis in Portland cement concrete as indicated by the amount of reaction during 24 hours at 80°C between 1 *N* sodium hydroxide solution and an aggregate that has been crushed and sieved to pass a 300 μm sieve and retained on a 150 μm sieve.
C342	"Standard Test Method for Potential Volume Change of Cement-Aggregate Combinations" Determination of the potential expansion of cement–aggregate combinations by measuring the linear expansion developed by the combinations in mortar bars subjected to variations in temperature and water saturation during storage under prescribed conditions of test.
C586	"Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)" Gives a relatively rapid indication of the potential expansive reactivity of certain carbonate rocks that may be used as concrete aggregates.

(*Continued*)

ASTM Standard	Definition and Significance
C1260	"Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)" Provides a means of detecting the potential of an aggregate intended for use in concrete for undergoing alkali–silica reaction resulting in potentially deleterious internal expansion.
C1293	"Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction" Evaluates the potential of an aggregate or combination of an aggregate with pozzolan or slag to expand deleteriously due to any form of alkali–silica reactivity.

FIGURE 2.2 Fine aggregates of concrete.

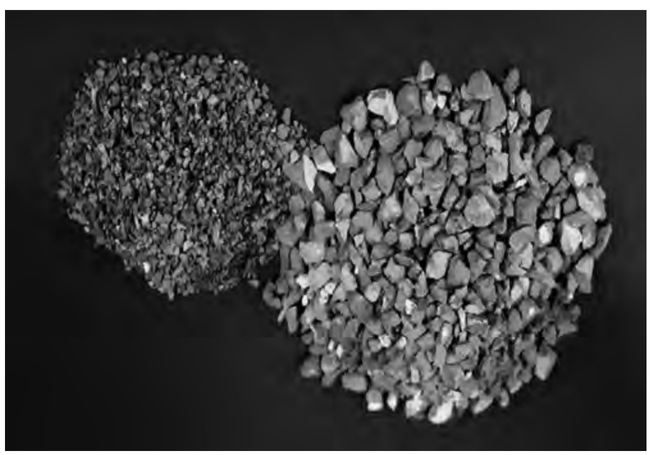


FIGURE 2.3 Coarse aggregates of concrete.

2.4 WATER

In accordance with section 26.4.1.3.1(a) of the code, water used in the mixing of concrete shall comply with ASTM C1602. The standard covers the mixing water that is used in the production of concrete. Water used in concrete shall be in the form of batch water, ice, water added by truck operator, free moisture on the aggregates, and water introduced in the form of admixtures. Potable and non-potable water is permitted to be used as mixing water in concrete. Chemical limits for combined mixing water (chloride, sulfate, alkalis, and total solids) are provided in the standard.

ASTM C1602 is the standard specification for mixing water used in the production of hydraulic cement concrete. Batch water discharged into the mixer from municipal water supply, reclaimed municipal water, or water resulting from concrete production operations is the main source of water in concrete. During hot weather concreting, ice may be used as part of the mixing water. The ice should be completely melted by the time mixing is completed. ASTM C94 (standard specification for ready-mixed concrete) allows the addition of water on site if the slump is less than specified, provided the maximum allowable water–cement ratio is not exceeded.

2.5 STEEL REINFORCEMENT

The design strength of steel reinforcement shall not exceed 80,000 psi in accordance with table 20.2.2.4a of the code, except for prestressing steel and transverse reinforcement. Table 20.2.2.4a of the code allows up to 100,000 psi strength for volumetric spiral reinforcement and confinement reinforcement for special frames and special walls of earthquake-resistant structures. The deformed steel shall comply with ASTM A615 (Standard Specification for Deformed and Plain Carbon Steel Bars for Concrete Reinforcement), ASTM A706 (Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement), ASTM A955 (Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement), and ASTM A996 (Standard Practice for Magnetic Particle Examination of Steel Forgings Using Alternating Current) in accordance with table 20.2.2.4a of the code.

In highly corrosive environments, zinc-coated (ASTM A767—"Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement") and epoxy-coated (ASTM A775—"Standard Specification for Epoxy-Coated Steel Reinforcing Bars" or ASTM A934—"Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars") reinforcements are permitted. Parking structures, bridge structures, and oceanfront balconies are good examples where these reinforcements are used.

FIGURE 2.4 Reinforcing steel.

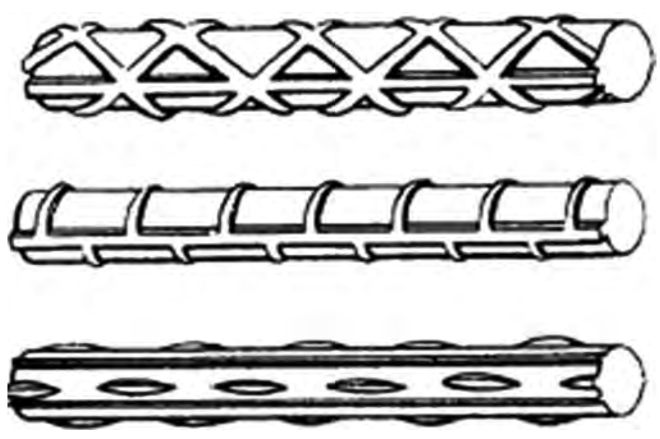
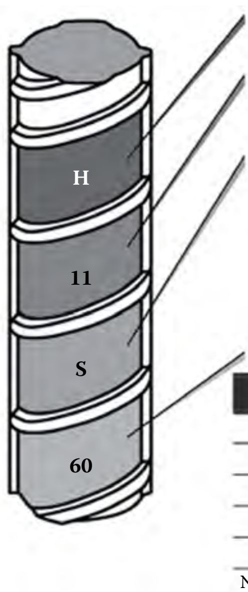


FIGURE 2.5 Types of deformed reinforcing bars.



Point of origin
Letter(s) or symbol(s) of producing mill

Size designation
3, 4, 5, 6, 7, 8, 9, 10, 11, 14, 18
[10, 13, 16, 19, 22, 25, 29, 32, 36, 42, 57]

Type of steel
S = Carbon-steel (A615/A615M)
W = Low-alloy steel (A706/A706M)
SS = Stainless-steel (A955/A955M)
R = Rail-steel (A996/A996M)
⌶ = Rail-steel (A996/A996M)
A = Axle-steel (A996/A996M)
CS = Low-carbon chromium (A1035/A1035M)

Minimum Yield Designation (Grade Mark or Grade Line)

in.-Pound Markings		Metric Markings	
60	Grade 60*	4	Grade 420*
75	Grade 75**	5	Grade 520**
80	Grade 80***	6	Grade 550***
100	Grade 100***	6	Grade 690***
120	Grade 120****	8	Grade 830****
None	Grade 40 or 50	None	Grade 280 or 350

*Or one grade line/**Or two grade lines/***Or three grade lines/****Or four grade lines

For stainless-steel (ASTM A955/A955M) reinforcing bars:
"•" for Grade 60 [420], "••" for Grade 75 [520]

FIGURE 2.6 Concrete reinforcing steel institute (CRSI) bar identification notation.

TABLE 2.1
Reinforcing Bars Data

Bar Size	Weight (lbs/ft)	Dia. (in.)	Area (in²)
3	0.376	0.375	0.110
4	0.668	0.500	0.200
5	1.043	0.625	0.310
6	1.502	0.750	0.440
7	2.044	0.875	0.600
8	2.670	1.000	0.790
9	3.400	1.128	1.000
10	4.303	1.270	1.270
11	5.313	1.410	1.560
14	7.650	1.693	2.250
18	13.60	2.257	4.000

2.6 ADMIXTURES

Admixtures are added to concrete before or during the mixing or batching of concrete, and broadly they serve the following general purposes:

- Reducing the construction cost
- Modifying the properties of concrete
- Maintaining the required properties of concrete during mixing, transporting, placing, and curing

Admixtures are classified as water-reducing admixtures, retarding admixtures, accelerating admixtures, superplasticizers, and corrosion-inhibiting admixtures. The water-reducing admixture helps the concrete to have a lower water–cement ratio. Retarding admixture slows down the setting rate of concrete, which is required during periods of high temperatures. Accelerating admixture increases the setting rate of concrete during cold weather. Superplasticizers are added to concrete at the jobsite to make up for the slump loss. Corrosion-inhibiting admixtures are used to resist corrosion of reinforcing steel in concrete.

Section 26.4.1.4.1 of the code allows the use of water-reducing and time-setting admixtures (ASTM C494—"Standard Specification for Chemical Admixtures for Concrete"); flowing concrete (ASTM C1017—"Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete"); air-entraining (ASTM C260—"Standard Specification for Air-Entraining Admixtures for Concrete"), and concrete with expansive cement (ASTM C845—"Standard Specification for Expansive Hydraulic Cement"). Any admixture containing calcium chloride is not permitted to be used as an admixture.

The Portland Cement Association defines the major advantages of concrete admixtures as follows[2]:

1. Water-reducing admixtures usually reduce the required water content for a concrete mixture by about 5%–10%. Consequently, concrete containing a water-reducing admixture needs less water to reach a required slump than untreated concrete. The treated concrete can have a lower water–cement ratio. This usually indicates that a higher strength concrete can be produced without increasing the amount of cement. Recent advancements in admixture technology have led to the development of midrange water reducers. These admixtures reduce water content by at least 8% and tend to be more stable over a wider range of temperatures. Midrange water reducers provide more consistent setting times than standard water reducers.
2. Retarding admixtures, which slow the setting rate of concrete, are used to counteract the accelerating effect of hot weather on concrete setting. High temperatures often cause an increased rate of hardening, which makes placing and finishing difficult. Retarders keep concrete workable during placement and delay the initial set of concrete. Most retarders also function as water reducers and may entrain some air in concrete.
3. Accelerating admixtures increase the rate of early strength development, reduce the time required for proper curing and protection, and speed up the start of finishing operations. Accelerating admixtures are especially useful for modifying the properties of concrete in cold weather.
4. Superplasticizers, also known as plasticizers or high-range water reducers, reduce water content by 12%–30% and can be added to concrete with a low-to-normal slump and water–cement ratio to make high-slump flowing concrete. Flowing concrete is a highly fluid but workable concrete that can be placed with little or no vibration or compaction. The effect of superplasticizers lasts only 30–60 minutes, depending on the brand and dosage rate, and is followed by a rapid loss in workability. As a result of the slump loss, superplasticizers are usually added to concrete at the jobsite.
5. Corrosion-inhibiting admixtures fall into the specialty admixture category and are used to slow down corrosion of reinforcing steel in concrete. Corrosion inhibitors can be used as a defensive strategy for concrete structures, such as marine facilities, highway bridges, and parking garages, that are exposed to high concentrations of chloride. Other specialty admixtures include shrinkage-reducing admixtures and alkali–silica reactivity (ASR) inhibitors. The shrinkage reducers are used to control drying shrinkage and minimize cracking, while ASR inhibitors control durability problems associated with alkali–silica reactivity.

2.7 CONCRETE MIX DESIGN

After studying the properties of cement, fine and coarse aggregates, water and admixtures, it is time to put them together in proper proportions and according to established procedures to obtain the concrete of the required strength, workability, and other properties. This procedure is called "concrete mix design." The ACI Committee Report 211.1 (Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete) provides guidelines for concrete mix design.[3] The three main phases of concrete mix design are specifications, design, and proportioning. In general, the concrete mix design procedure provides a first approximation of the proportions and is checked by trial batches. Due consideration is given to the locally available materials and their cost and properties, labor and equipment costs, and optimum use of cement to reduce cost without compromising the strength. The ACI Report 211.1 also considers placeability, consistency, strength, water–cement ratio, durability, density, and generation of heat as the prime criteria for concrete mix design.

Placeability is the quality of wet concrete that allows it to be placed and finished. It prevents concrete from being segregated while pumping and placement. Workability is closely related to placeability and the two terms are sometimes used synonymously. The factors that influence placeability are grading, particle shape, proportioning of aggregates, amount and quality of cement, the presence of entrained air and admixtures, and the consistency of the mixture. Consistency of concrete is also related to placeability. Slump of concrete determines its consistency.

Strength is always specified for a concrete mix. In structural concrete applications, strength is critical because the designer uses the strength parameter to size the structural elements adequate for the spans and the intended loads. Water–cement ratio is an important factor in determining the strength of concrete. Maximum size, grading, surface texture, shape, strength, and stiffness of aggregates; cement types; air content; and use of admixtures also influence the strength of concrete.

Durability of concrete is important for its strength and serviceability and is given due consideration in the concrete mix design. It is discussed in detail in Section 5.5.4. The density of concrete is a property used for its weight consideration. A reinforced concrete member is also designed to be adequate to resist its own weight.

Generation of heat during the mixing of concrete is an important parameter to be considered during concrete mix design. The temperature rise of concrete is required to be kept to minimal to control cracking. This is done by lowering the temperature during concrete placement, reducing quantities of cement, circulating chilled water through the concrete, and insulating concrete surfaces to reduce dissipation of heat.

It is ideal to introduce the definition of air entrainment of concrete at this juncture. According to the definition provided by the Portland Cement Association, air-entrained concrete contains billions of microscopic air cells per cubic foot. These air pockets relieve internal pressure on the concrete by providing tiny chambers for water to expand into, when it freezes. Air-entrained concrete is produced using air-entraining Portland cement, or by the introduction of air-entraining agents, under careful engineering supervision, as the concrete is mixed on the job. The amount of entrained air is usually between 4% and 7% of the volume of the concrete but may be varied as required by special conditions.[4] The primary purpose of air entrainment is to increase the durability of the hardened concrete, especially in climates subject to freeze–thaw cycles; the secondary purpose is to increase workability of the concrete while in a plastic state.

2.8 CONCRETE MIX DESIGN PROCEDURE

Chapter (6) of ACI 211.1 discusses the procedure of concrete mix design for normal weight concrete. The Engineer of Record may specify maximum water-cement ratio, minimum cement content, air content, slump, maximum size of aggregate and strength, etc. In general, the following steps are required in the concrete mix design:

1. The selection of slump of concrete is based upon the type of concrete member (slab, beam, wall, column, footing, etc.) for which the concrete is being used. In accordance with table 6.3.1 of the ACI 211.1, the minimum slump shall be 1 in. and the maximum slump varies between 3 in^2 and 4 in^2. A slump of 4 in^2 can be used for beams, walls, and columns.
2. The size of the coarse aggregate is dictated by the economy and the dimensions of the structural elements. As discussed in Section 2.3 of this chapter, section 26.4.2.1 (a)(4) of the code restricts the size of the coarse aggregates. Well-graded coarse aggregates have less voids and hence require less cement mortar per unit volume of concrete. For higher strength concrete or concrete with congested area of reinforcing steel, coarse aggregate sizes are reduced to accommodate lower water–cement ratios.
3. Size, shape, and grading of aggregates; temperature of concrete; amount of entrained air; and the use of chemical admixtures determine the quantity of water required to be mixed with concrete. Table 6.3.3 of ACI 211.1 provides an estimate of the water to be mixed with concrete in lb per cubic yard based on the maximum size of the aggregate for the required slump of air-entrained and non-air-entrained concrete. Air entrainment depends on the atmospheric exposure given to the concrete—mild, moderate, and severe. Exposure is explained further in Section 5.5.4.
4. An estimate of the water–cement ratio is provided in table 6.3.4(a) of ACI 211.1 for the required compressive strength for air-entrained and non-air-entrained concrete. Severe exposures require lower water–cement ratio.
5. The quantity of cement is calculated based on steps (3) and (4).
6. The volume of coarse aggregates is determined based on nominal maximum size of the coarse aggregates and the fineness moduli of fine aggregates. The fineness moduli of fine aggregates is an empirical factor obtained by adding the cumulative percentages of aggregates retained on each of the standard sieves ranging from 80 mm to 150 μm and dividing this sum by 100. Table 6.3.6 of ACI 211.1 provides an estimate of the volume of coarse aggregate in a unit volume of concrete based on nominal maximum size of the coarse aggregates and the fineness moduli of fine aggregates. For workable concrete, like in areas of congested reinforcement, this volume is reduced by 10%.
7. Two methods—weight method and absolute volume method—are used to determine the volume of fine aggregates. In the weight method, weight of fine aggregates is derived by subtracting the weights of coarse aggregates, cement, and water from the total weight of concrete. Table 6.3.7.1 of ACI 211.1 provides a first estimate of the weight of fine aggregates. In the absolute volume method, the volume of water, air, cement, additives, and coarse aggregates is subtracted from the total required volume of concrete.
8. The aggregates used are generally moist. Hence, the water estimated to be mixed with concrete needs to be reduced. Laboratory trial batch procedures are performed in accordance with ASTM C192.[5]
9. The final step is checking the mixture proportions by preparing trial batches, testing in accordance with ASTM C192 and adjusting the concrete mix design.

These steps are demonstrated in the following example.

2.9 CONCRETE MIX DESIGN EXAMPLE

Prepare a mix design of concrete to be used for a parking garage floor in an oceanfront (extreme) condition of a tropical zone. The 28 days compressive strength of concrete shall be 5000 psi.

Assume that the concrete production facility does not have field strength data available.

$$\text{Required average field strength } \left(f'_{cr}\right) = 1.10f'_c + 700 \quad \text{(ACI 318 Table 5.3.2.2)}$$

$$= (1.10)(5000) + 700 = 6200 \text{ psi}$$

Step (1): Maximum slump of concrete = 3 in^2 (ACI 211.1 Table 6.3.1)
Step (2): Choose a nominal aggregate size of ¾ in. It shall be ascertained that it is locally available.
Step (3): Since the building is located in a tropical zone, air-entrained concrete is not required.

$$\text{Water required} = 340 \text{ lbs/yd}^3 \quad \text{(ACI 211.1 Table 6.3.3)}$$

$$\text{Volume of water} = 340/62.4 = 5.45 \text{ feet}^3$$

Step (4): For non-air-entrained 6000 psi concrete, water–cement ratio = 0.41 (ACI 211.1 Table 6.3.4a)

Step (5): $\text{Cement content} = \dfrac{\textit{Water content}}{\textit{Water} - \textit{cement ratio}} = \dfrac{340}{0.41} = 829.3 \text{ lbs/yd}^3$

$$\text{Volume of cement} = 829.3/196.5 = 4.22 \text{ feet}^3$$

Step (6): $\text{Volume of coarse aggregate} = \dfrac{1728}{2.68 \times 62.4} = 10.33 \text{ feet}^3$
(To calculate the solid volume of coarse aggregate, the specific gravity of 2.68 is used.)
Step (7): Volume of fine aggregate for 1 yd^3 of concrete = 27 − 5.45 − 4.22 − 10.33 = 7 $feet^3$
Hence, weight of fine aggregate = 7 × 2.64 × 62.4 = 1153 lbs
(Specific gravity of fine aggregate = 2.64)
Step (8): If the tests indicate that the coarse aggregates contain 3% moisture and fine aggregates contain 5% moisture, then adjusted aggregate weights are as follows:

$$\text{Coarse aggregate}(\text{wet}) = 1728 \times 1.03 = 1780 \text{ lbs}$$

$$\text{Fine aggregate}(\text{wet}) = 1153 \times 1.05 = 1211 \text{ lbs}$$

The weight of water needs to be adjusted:

$$340 - (1780)(0.03) - (1040)(0.05) = 234.6 \text{ lbs}$$

2.10 CONCLUSION

In this chapter, the reader was introduced to a brief history of the use of concrete and reinforced concrete in building construction. Properties and standards of cement, aggregates, water, steel, and admixtures were discussed. The procedure for concrete mix design along with an example was demonstrated. Before learning the design of reinforced concrete structural elements, it is very important for the reader to understand the properties of the materials used in reinforced concrete.

ASSIGNMENTS

1. Perform online and library research to review all the ASTM standards that relate to concrete materials and testing. Chapter 3 of the code is a good starting point. Prepare a record of the standards with a summary of its intent.
2. Study the purpose and procedures for the following experiments related to concrete technology.
   a. Fineness of cement
   b. Normal consistency of cement
   c. Initial and final setting times of cement
   d. Specific gravity of cement

   e. Soundness of cement
   f. Fineness modulus of fine and coarse aggregates
   g. Specific gravity, void ratio, porosity, and bulk density of coarse and fine aggregates
   h. Bulking of sand
   i. Workability tests on fresh concrete
   j. Compaction factor test
   k. Test for compressive strength of cement concrete
3. Prepare a concrete mix design using the following parameters:
   a. Required 28 days compressive strength of concrete—4000 psi
   b. Concrete is required for columns in Chicago
   c. Available nominal maximum size of coarse aggregate—1 in.
   d. Fineness modulus of sand—2.6
   e. Specific gravity of Portland cement—3.15
   f. Specific gravity of coarse aggregate—2.68
   g. Specific gravity of fine aggregate—2.64
   h. Moisture in coarse aggregate—2%
   i. Moisture in fine aggregate—6%

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1. Rinker Info Series, Types of Portland cement, www.rinkerpipe.com/TechnicalInfo/files/InfoBriefs/IS102TypesPortlandCement (Accessed June 23, 2016).
2. Portland Cement Association, Chemical admixtures, http://www.cement.org/cement-concrete-basics/concrete-materials/chemical-admixtures (Accessed April 27, 2016).
3. 211.1-91: Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete, American Concrete Institute, Farmington Hills, MI.
4. Portland Cement Association, Air entrained concrete, http://www.cement.org/cement-concrete-basics/working-with-concrete/air-entrained-concrete (Accessed December 24, 2015).
5. ASTM C192 provides standardized requirements for preparation of materials, mixing concrete, and making and curing concrete test specimens under laboratory conditions. It is the standard for mixture proportioning, evaluation of different mixtures and materials, correlation with nondestructive tests, and providing concrete specimens for research purposes.

3 Design Loads

3.1 INTRODUCTION

Design loads in the United States are specified primarily in the American Society of Civil Engineers (ASCE) standard ASCE 7 "Minimum Design Loads for Buildings and Other Structures" and the International Building Code (IBC). This book is based upon ASCE 7-10 and IBC (2015). The two standards specify the minimum load requirements for the design of buildings and other structures that are subject to building code requirements. This book follows the strength design and the loads, and the appropriate load combinations specified in these codes are for the strength design method.

According to the ASCE 7-10, strength design is a method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the member design strength. Ultimate strength design is a method of structural design based on the ultimate strength due to the inelastic action of reinforced structural concrete cross sections subject to simple bending, axial load, shear, bond, or combinations thereof. Ultimate strength design does not necessarily involve an inelastic theory of structures. An evaluation of external moments and forces that act in indeterminate structural frameworks by virtue of dead, live, wind, earthquake, and other loads may be carried out either by the theory of elastic displacements or by limit state design. Limit state design indicates a design method involving an inelastic theory of structures in which readjustments in the relative magnitude of bending moments at various sections due to nonlinear relationships between loads and moments at high loads are recognized. Two design methods (working stress method and strength method) are discussed in Chapter 1 of the book.

3.2 LOAD COMBINATIONS

The load factors used in the strength design in accordance with section 2.3.1 of ASCE 7-10 are as follows:

$$1.4\mathrm{D} \tag{1}$$

$$1.2\mathrm{D} + 1.6\mathrm{L} + 0.5\left(\mathrm{L_r \text{ or } S \text{ or } R}\right) \tag{2}$$

$$1.2\mathrm{D} + 1.6\left(\mathrm{L_r \text{ or } S \text{ or } R}\right) + \left(\mathrm{L \text{ or } 0.5W}\right) \tag{3}$$

$$1.2\mathrm{D} + 1.0\mathrm{W} + \mathrm{L} + 0.5\left(\mathrm{L_r \text{ or } S \text{ or } R}\right) \tag{4}$$

$$1.2\mathrm{D} + 1.0\mathrm{E} + \mathrm{L} + 0.2\mathrm{S} \tag{5}$$

where
- D is the dead load
- L is the live load
- E is the earthquake load
- L_r is the roof live load
- R is the rain load
- S is the snow load
- W is the wind load

Since flood loads, atmospheric ice loads, self-restraining loads, and other loads are not used in this book, they are not defined in these load combinations. For the complete list of load combinations, refer to chapter 2 of ASCE 7-10.

3.3 DEAD LOAD

In accordance with chapter 2 of IBC (2015) and section 3.1.1 of ASCE 7-10, dead load is the weight of all materials of construction incorporated into the building including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment such as cranes, plumbing stacks and risers, electric feeders, heating, ventilating and air-conditioning systems, and automatic sprinkler systems. The actual weight of materials and equipment shall be used in the design. However, in this book, the primary dead load used is the weight of the concrete elements and the finished floor weight. The density of reinforced concrete is universally taken as 150 psf.

Generally, anything that is a fixed part of the structure is a dead load. To be considered dead load, an item must be physically attached to the structure. If an object can be moved, then it is not dead load. Objects such as movable shelving, desks, chairs, beds, chests, books, copiers, stored items, or anything else that can or may be moved around during the life of the structure are considered as "live loads," explained in the following section of this chapter.

Dead loads are the weights of the final structure. In the design process, initially the sizes of members are assumed and weights are calculated. A preliminary design is performed to check if the assumed sizes are appropriate. If the sizes are changed after the preliminary design, then the weights are recalculated. Sometimes, more than a single iteration may be required.

Floors and roofs have uniform density, and their weights are expressed in terms of weight per unit area (psf). They are designed for this dead weight. Loads of finishes are also applied in the design of floors and roofs. The loads are distributed to their supports using the geometric concept of tributary area. The supports for floors and roofs could be walls, beams, or columns. Walls are used to support floors and roofs in the load-bearing walls structural system. In the framed systems, loads of floors and roofs are transferred to beams and from the beams to the columns. In the flat-plate system, loads of floors and roofs are directly transferred to the columns. A combination of these three systems can also be used to design buildings.

The weights of walls are also calculated in terms of weight per unit area (psf). Then, according to the height of the wall, the weight of the wall is converted in terms of weight per linear distance (plf). The weight of the floors and the roof supported are applied on the walls according to their tributary area.

When the floors and the roof are supported on beams, their weight is applied on beams according to their tributary area. The weights of beams are calculated in accordance with their cross-sectional area and expressed in terms of weight per linear distance.

Items that are not spread over the surface and not uniformly spaced over the entire area are generally not included in the unit weight calculation but are treated as individual loads in addition to the unit load. An example of these loads is columns carrying the loads of upper floors supported on transfer slabs. This occurs when the floor below the transfer slab is accommodating column-free spaces such as parking garages below the living floors. Then there are miscellaneous loads for electrical and mechanical items such as wiring and plumbing whose exact location is unknown during design, as well as added density at the connections to supporting structures. To account for these items, an additional "miscellaneous" load may be applied. The magnitude of the miscellaneous load generally varies depending on what is expected of these items. For typical cases, values between 1 and 2 psf are typical. Additional loads are applied to roofs for the slopes given for drainage after the roof slab is cast. Materials such as lightweight concrete or cement slurries are used to provide the required slope for storm water drainage. The weight of these materials is calculated based on an average thickness spread over the complete area of the slab.

Section 4.3.2 of ASCE 7-10 specifies a minimum load of 15 psf for partitions, which are not classified as dead load but instead are classified as live load.

Dead loads can be estimated with the help of tables C3.1-1a, C3.1-1b and C3.1-2 of the ASCE 7-10, chapter 17 of the American Institute of Steel Construction Manual (AISC) 360 or from manufacturer catalogs:

Weight of a 6 in. thick slab = (6 in./12 in.) × 150 pcf = 75 psf
Weight of 8 in. × 18 in. beam = (8 in./12 in.) × (18 in./12 in.) × 150 pcf = 150 plf
Weight of 8 in. × 18 in. × 10 feet high column = (8 in./12 in.) × (18 in./12 in.) × (10 feet) × 150 pcf = 1500 lbs
Weight of 5 feet × 5 feet × 18 in. footing = (5 feet) × (5 feet) × (18 in./12 in.) × 150 pcf = 5625 lbs

3.4 LIVE LOAD

Live load is a load that is created by the use and occupancy of the building that does not include other loads such as the wind load, snow load, rain load, earthquake load, flood load, or dead load. The required live load for the design of buildings and other structures is defined in table 4.3-1 of ASCE 7-10, which is also adopted in the IBC (2015). Live loads are applied on structural elements (especially the slabs) as uniform loads, partial loads, and concentrated loads. Partial loads are applied to produce the maximum effect on structural members.

Floor live loads are reduced using the following formula:

$$L = L_0\left(\frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad \text{(ASCE 7-10 Equation 4.7-1)}$$

Where L is the reduced live load, L_0 is the unreduced live load, K_{LL} is the live load element factor (table 4.7-1 of ASCE 7-10), and A_T is tributary area. The equation is applied when $K_{LL}A_T$ exceeds 400 sft. L shall not be less than $0.50L_0$ for members supporting one floor, and L shall not be less than $0.40L_0$ for members supporting two or more floors.

3.5 SNOW LOADS

Snow loads on the roof slabs are determined in accordance with chapter 7 of ASCE 7-10. The snow load on a flat roof is given by

$$p_f = 0.7C_eC_tI_sp_g \qquad \text{(ASCE 7-10 Equation 7.3-1)}$$

where
C_e is the exposure factor (ASCE table 7.3-1)
C_t is the thermal factor (ASCE table 7.3-2)
I_s is the importance factor (ASCE section 7.3.3)
p_g is the ground snow (ASCE figure 7.2-1 and ASCE table 7.2-1) or a site-specific analysis

Chapter 7 of ASCE 7-10 also specifies determination of snow loads in other conditions such as sloped and curved roofs. It also deals with partial and unbalanced snow loading on the roofs, snow drifts on low roofs, roof projections, parapets, sliding snow, rain on snow surcharge, and ponding instability.

3.6 EARTHQUAKE LOADS

Chapters 11 through 23 of the ASCE 7-10 deal with earthquake-related loads and design. This is a specialized subject and beyond the purview of this book.

3.7 FLOOD LOADS

Chapter 5 of ASCE 7-10 and ASCE 24-14 ("Flood Resistant Design and Construction") provide the basis of calculating the flood loads for the design of structures. The structural elements are designed to resist flotation, collapse, and permanent lateral displacement due to design flood loads. The Federal Emergency Management Agency (FEMA) prepares the Flood Insurance Rate Maps (FIRM), which provides the base flood elevation (BFE) in flood hazard areas in the United States. FEMA has divided the country primarily in three zones.

***V-Zone*:** This is the Special Flood Hazard Area extending from off shore to the inland limit of the primary frontal dune along an open coast, and any other area that is subject to high-velocity wave action from storms or seismic sources.
***A-Zone*:** This is an area within a Special Flood Hazard Area landward of a V-Zone. The primary source of flooding must be astronomical tides, storm surges, seiches, or tsunamis. During the base flood, a potential for breaking wave height greater than or equal to 1.5 feet exists.
***X-Zone*:** This is not a flood zone and does not have a flood potential.

According to section 5.2 of ASCE 7-10, design flood is defined as the greater of the following two flood events:

1. Base flood affecting those areas identified as Special Flood Hazard Areas on the community's FIRM.
2. Flood corresponding to the area designated as a flood hazard area on a community's Flood Hazard Map or otherwise legally designated.

Flood loads affect ground slabs and retaining walls. Breakaway walls are specially designed walls in flood zones. They are subject to flooding that is not required to provide structural support to a building or other structure and are designed such that, under base flood or lesser flood conditions, they will collapse and allow the free passage of floodwaters but not damage the structure or supporting foundation system.

During flood, the three primary types of loads are hydrostatic load, hydrodynamic load, and wave load. Hydrostatic loads are caused by a depth of water to the level of the design flood elevation and are applied over all surfaces involved, both above and below ground level. Hydrodynamic loads are dynamic effects of moving water and are determined by a detailed analysis utilizing the basic concepts of fluid mechanics. Wave loads result from water waves propagating over the water surface and striking a building or other structure.

The local still water depth (d_s) shall be calculated. It is an important measure used in the calculations of flood loads:

$$d_s = 0.65(BFE - G) \qquad \text{(ASCE 7-10 Equation 5.4-3)}$$

where
BFE is the base flood elevation
G is the ground elevation

3.8 RAIN LOADS

Chapter 8 of the ASCE 7-10 addresses the rain loads:

$$R = 5.2(d_s + d_h) \quad \text{(ASCE 7-10 Equation 8.3-1)}$$

where

R is the rain load on the undeflected roof (psf)

d_s is the depth of water on the undeflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (i.e., the static head) (in.)

d_h is the additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow (i.e., the hydraulic head) (in.)

In accordance with section 1611.3 of the IBC (2015), roofs shall be designed to sustain the rainwater that would accumulate on them to the elevation of the secondary drainage system plus the uniform load of the water that rises above the secondary drainage system. Typically, in flat roofs with parapets, overflow scuppers are provided in the parapets as secondary drainage. The area of the overflow scuppers is proportional to the area of the roof. Slopes in flat roofs towards the secondary drainage should not be less than ¼ in. to a foot.

3.9 SOIL LOAD

The design of the soil-retaining walls shall account for lateral pressure due to the surrounding soil. If the soil loads are not given in a soil investigation report, then the soil loads specified in table 3.2-1 of ASCE 7-10 can be used for the lateral soil pressures. An allowance for a surcharge from fixed or moving loads (if any) shall be made. If a portion or the whole of the adjacent soil is below a free-water surface, computations shall be based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure.

3.10 WIND LOADS

Wind loads are dealt with in detail in Chapter 14 of the book.

3.11 CONCLUSIONS

This chapter discusses the most common types of load that could act on a building during its lifetime. The methods to calculate the dead loads, live loads and its reductions, snow loads, flood loads, rain loads, and soil loads using the ASCE 7-10 are introduced. The FEMA flood zones are explained. Wind loads are discussed in detail in Chapter 14 of the book. The estimation of earthquake loads is beyond the scope of this book.

3.12 ASSIGNMENTS

1. The reactions on a column due to various types of loads are mentioned below. Determine the factored loads using the various load combinations of chapter 2 of ASCE 7-10 (simplified in Section 3.2 of the book). Identify the load combination that governs the design of the column.

Dead load of slabs	20 K
Floor finishes	3 K
Floor live load	12 K
Partition load	3 K
Roof live load	2 K
Snow load	4 K
Wind load	8 K
Rain load	1.5 K

2. Review section 4.7 of the ASCE 7-10 and learn all factors of the standard influencing the reduction of live loads.
3. What is the tributary area of a one-way slab on the beam that is supporting it? The slab has a span of 12 feet and a length of 30 feet.

4. The interior beam of an office floor (live load—50 psf) has a tributary area of 500 feet^2 and a span of 20 feet. Calculate the total reduced live load acting on the beam.
5. Review chapter 7 of ASCE 7-10 and prepare notes on snow load calculations for sloped and curved roofs, partial and unbalanced snow loading on the roofs, snowdrifts on low roofs, roof projections, parapets, sliding snow, rain on snow surcharge, and ponding instability.
6. Review chapter 5 of ASCE 7-10 and prepare notes on hydrostatic, hydrodynamic, and wave flood loads on walls.
7. Visit the FEMA website noted in Reference 3 and determine the flood zone in which your home is located. If it is located in any zone other than the X-Zone, then determine the BFE.
8. Review chapter 5 of ACI 318-14, chapter 2 of ASCE 7-10, and chapter 16 of IBC (2015) and prepare a report for the various load combinations for working stress method and strength method.

REFERENCES

1. International Building Code (2015), International Code Council, Birmingham, AL.
2. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-10, American Society of Civil Engineers, Reston, VA.
3. FEMA Maps accessible at http://fema.maps.arcgis.com/home/webmap/viewer.html?webmap=cbe088e7c8704464aa0fc34eb99e7f30.

4 ACI Strength Requirements

4.1 INTRODUCTION

A concrete section should have adequate strength to resist flexure, axial loads, axial and flexural combined loads, shear, torsion, bearing, and shear friction. This chapter deals with such sectional strengths required for the design of structural elements. Though bending of concrete beams and slabs and also of columns and footings is very important, as we discussed in the previous chapters, beams and slabs must also have adequate safety against shear, torsion, and shear friction. In my experience of reviewing plans for several years, I have seen some engineers who perform manual structural calculations tend to ignore shear, torsion, and shear friction. In this chapter, we shall review the ACI requirements to provide sectional strength to the structural element during the design for strength and stability of structure.

4.2 BENDING

Section 22.2 of the code makes design assumptions for "Equilibrium and Strain Compatibility," "Concrete," and "Reinforcing Steel." The flexural and axial strengths of a member calculated by the strength design method of the code require the equilibrium and compatibility of the strains to be satisfied.

Reinforced concrete design takes advantage of the high compressive strength of concrete and the high tensile strength of steel. Since reinforced concrete is a homogeneous material, the following assumptions are made during the design:

1. The forces acting in each cross section must be balanced at nominal strength, which is the strength of a member or cross section calculated in accordance with the provisions of the code and these assumptions, before the application of any strength reduction factors (discussed in Section 5.14 of the book).
2. Strain in concrete is the same as in reinforcing bars at the same level, provided that the bond between the concrete and steel is adequate.
3. Strain in concrete is linearly proportional to the distance from the neutral axis.
4. The stress in the elastic range is equal to the strain multiplied by the modulus of elasticity.
5. Plane cross sections before bending remain plane after bending.
6. Tensile strength of concrete is neglected because it is very low.
7. Cracked concrete is assumed to be not effective. Before cracking, the entire cross section is effective in resisting the external moments.
8. 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.
9. At ultimate strength, the maximum strain at the extreme compression fibers is assumed to be equal to 0.003 (section 22.2.2.1 of the code). At the ultimate strength, the shape of the compressive stress distribution may be assumed to be rectangular, parabolic, or trapezoidal.

Section 22.2.2.3 of the code allows the relationship between the concrete compressive stress and strain to be represented by a rectangular, trapezoidal, or parabolic shape, when in actuality it is complex. In this book, a rectangular stress distribution is used (Whitney Rectangular Stress Block). A compressive stress of concrete of $0.85f_c'$ is assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a line parallel to the neutral axis located at a distance "a" from the fiber of maximum compressive strain (Figure 4.1).

$$a = \beta_1 c \qquad \text{(ACI Equation 22.2.2.4.1)}$$

where

c is the distance from the fiber of maximum compressive strain to the neutral axis

β_1 is according to table 22.2.4.3 of the code

$\beta_1 = 0.85$ for 2500 psi $\le f_c' \le$ 4000 psi

$\beta_1 = 0.85 - (0.05\,(f_c' - 4000)/1000)$ for 4000 psi $< f_c' <$ 8000 psi

$\beta_1 = 0.65$ for $f_c' \ge$ 8000 psi

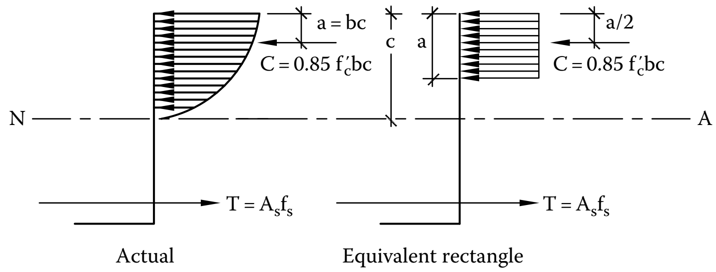


FIGURE 4.1 Stress distribution.

For reinforcing steel, the stress below f_y shall be E_s times the steel strain, and for strains greater than that corresponding to f_y, the stress shall be independent of the strain. The value of yield strength of steel for various available grades of steel is provided in table 20.2.2.4a of the code, but as stated in Section 5.10 of the book, in this book, we shall consistently use steel with a maximum yield strength (f_y) of 60,000 psi.

$E_s = 29{,}000{,}000$ psi
For $\varepsilon_s \le \varepsilon_y$, $A_sf_s = A_sE_s\varepsilon_s$
For $\varepsilon_s > \varepsilon_y$, $A_sf_s = A_sf_y$

In the calculation of the flexural strength, as mentioned earlier in this section, a rectangular stress block is used.

Compression $(C) = 0.85f_c'ba$
Tension $(T) = A_sf_y$

It is assumed that the steel yields before the concrete crushes and hence f_y is used.

Equating compression with tension

$$C = T$$

$$0.85f_c'ba = A_sf_y$$

$$a = \frac{A_sf_y}{0.85f_c'b}$$

$$M_n = A_sf_y\left(d - a/2\right)$$

Substituting the value of "a" in the above equation

$$M_n = A_sf_y\left(d - 0.59\frac{A_sf_y}{f_c'b}\right)$$

4.3 AXIAL STRENGTH AND COMBINED AXIAL–FLEXURAL STRENGTHS

Section 22.4.2 of the code provides specifications for the strength of columns to resist axial forces and combined axial and flexural forces.

$$P_0 = 0.85f_c'\left(A_g - A_{st}\right) + f_yA_{st} \qquad \text{(ACI Equation 22.4.2.2)}$$

Maximum axial compressive strength according to table 22.4.2.1 of the code is

$$\begin{aligned} P_{n,max} &= 0.80P_0 \quad \text{for ties} \\ &= 0.85P_0 \quad \text{for spirals} \end{aligned}$$

Tie reinforcements for lateral support of longitudinal reinforcements are specified in sections 10.7.6.2 and 25.7.2 of the code. The bottom tie shall not be located more than half the tie spacing above the top of the footing or the slab, and the top tie shall not be located at a distance more than half the tie spacing from the lowest reinforcement of slab, drop panel, shear cap, or beam.

The diameter of the tie shall be at least #3 for #10 or smaller longitudinal bars and at least #4 for #11 or larger longitudinal bars.

The clear spacing of the ties shall be at least 4/3 times the maximum size of the aggregate, and the center-to-center spacing shall not exceed the least of the following: 16 times the diameter of the longitudinal bars, 48 times the diameter of the tie bars, and smallest dimension of the column.

The rectilinear ties shall be designed in accordance with section 25.7.2.3 of the code, and the anchorage of the tie shall be designed in accordance with section 25.3.2 of the code.

4.4 SHEAR

The code provides shear strength requirements for members subject to one-way shear (such as one-way slabs and beams) in section 22.5 and two-way shear (for two-way slabs) in section 22.6.

4.4.1 One-Way Shear Strength of Concrete

The one-way shear strength of concrete is addressed in section 22.5 of the code.

$$\text{Nominal one-way shear strength is } \left(V_n\right) = V_c + V_s \qquad \text{(ACI Equation 22.5.1.1)}$$

where

V_c is the nominal shear strength provided by concrete (lb)

V_s is the nominal shear strength provided by steel (lb)

$$\text{Design shear force is } \left(V_u\right) \le \Phi\left(V_c + 8\sqrt{f_c'}b_w d\right) \qquad \text{(ACI Equation 22.5.1.2)}$$

The design shear force (V_u) is calculated at d/2 from the face of the support (Figure 4.2).

Where $\sqrt{f_c'} \le 100\, psi$.

$$V_c = 2\sqrt{f_c'}b_w\, d \quad \left(\text{with no axial compression}\right) \qquad \text{(ACI Equation 22.5.5.1)}$$

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}b_w d \quad \left(\text{with axial compression}\right) \qquad \text{(ACI Equation 22.5.6.1)}$$

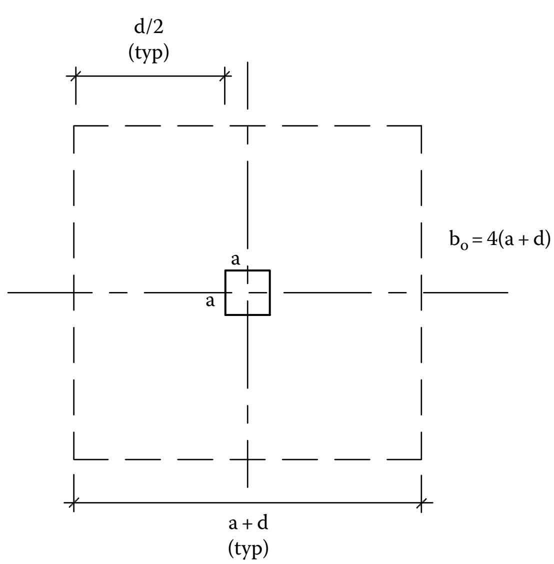


FIGURE 4.2 Square interior column critical section.

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}\,b_w d \quad (\text{with axial tension}) \qquad \text{(ACI Equation 22.5.7.1)}$$

where
- b_w is the width of the slab or beam (in.)
- d is the effective depth of slab or beam (in.)
- A_g is the gross cross-section area under consideration (in^2)
- N_u is the axial compression or tension (lb) (+ve for compression and –ve for tension)

Detailed methods for calculating (V_c) are provided in tables 22.5.5.1 and 22.5.6.1 of the code and are illustrated in the examples.

4.4.2 Shear Reinforcement for Beams and One-Way Slabs

In general, shear reinforcements are designed to account for the deficit between the shear force acting on a reinforced concrete structural element and the shear capacity of the concrete of the element (Figure 4.3). In beams, shear reinforcements are provided perpendicular to the main longitudinal reinforcement for bending. They are in the form of stirrups in beams. In one-way slabs, shear reinforcements are typically not required unless the loading is heavy.

$$V_s \geq \frac{V_u}{\Phi} - V_c \qquad \text{(ACI Equation 22.5.10.1)}$$

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(ACI Equation 22.5.10.5.3)}$$

where
- A_v is the total cross-sectional area of shear reinforcement (in^2)
- s is the spacing of shear reinforcement (in.)

$$\frac{A_v}{s} = \frac{(V_u - \Phi V_c)}{\Phi f_y d}$$

For inclined shear reinforcement,

$$V_s = \frac{A_v f_y (Sin \propto + Cos \propto)}{s} d \qquad \text{(ACI Equation 22.5.10.5.4)}$$

where $\propto$ is the angle between the inclined stirrups and the longitudinal axis of the member.

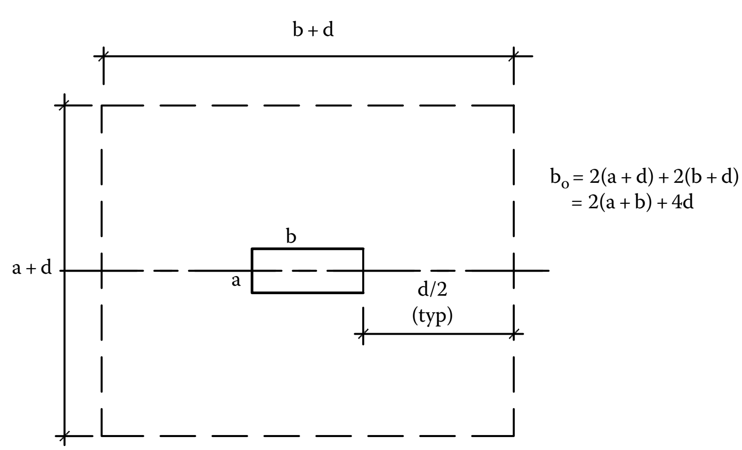


FIGURE 4.3 Rectangular column critical section.

Maximum spacing of shear reinforcement shall be in accordance with section 9.7.6.2.2 of the code. If the shear resistance provided by stirrups (V_s) is less than or equal to $4\sqrt{f_c'}b_w d$, then the maximum spacing of the stirrups' spacing shall be lesser of d/2 or 24 in², and if the shear resistance provided by stirrups (V_s) is greater than $4\sqrt{f_c'}b_w d$, then the maximum spacing of the stirrups' spacing shall be lesser of d/4 or 12 in².

Sometimes the bottom reinforcements of slabs and beams are bent and taken to the top of the section when they are no longer required at the bottom. These bent-up bars provide shear resistance. This type of construction has become obsolete and hence not discussed in this book.

4.4.3 Two-Way Slab Shear Design

The two-way shear strength is discussed in section 22.6 of the code. Shear forces in two-way slabs and footings are calculated at critical sections. Slabs and footings are evaluated for shear at the edges or corners of columns, concentrated loads, reaction areas, and changes in slab or footing thickness such as edges of capitals and drop panels (Figure 4.4). If shear reinforcement is needed in two-way slabs, it is provided in the form of stirrups, headed shear studs, and shearheads. The shearheads can be structural steel W section.

Section 8.4.4 of the code provides the method to calculate the two-way shear. When the factored slab moment is resisted by the columns, then

$$v_u = v_{ug} + \frac{\gamma_v M_{sc} c}{J} \qquad \text{(ACI Section 8.4.4.2.2)}$$

$$\gamma_v = 1 - \gamma_f \qquad \text{(ACI Equation 8.4.4.2.2)}$$

$$\gamma_v = \frac{1}{1 + \left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \qquad \text{(ACI Equation 8.4.2.3.2)}$$

where

v_u is the factored shear stress (psi)

v_{ug} is the factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer (psi)

M_{sc} is the factored slab moment that is resisted by the column at a joint (lb-in)

γ_f is the factor used to determine the fraction of M_{sc} transferred by slab flexure at slab–column connections

γ_v is the factor used to determine the fraction of M_{sc} transferred by eccentricity of shear at slab–column connections

b_1 is the dimensions of the critical section (b_0) measured in the direction of the span for which moments are determined (in.)

b_2 is the dimensions of the critical section (b_0) measured in the direction perpendicular to b_1 (in.)

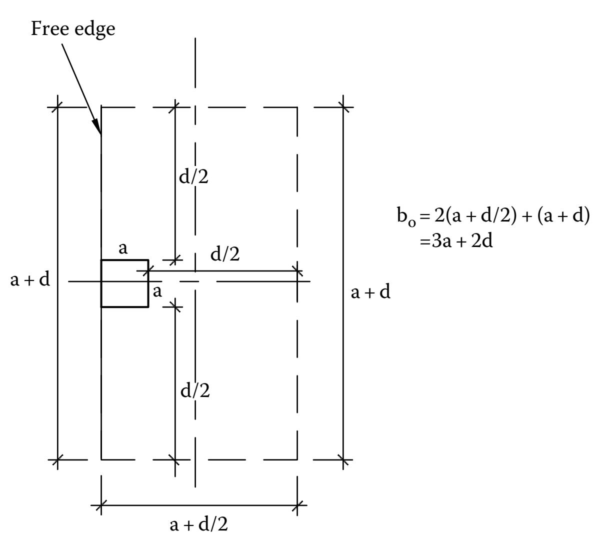


FIGURE 4.4 Edge square column critical section.

Stress distribution is explained in the slab design example. In accordance with section 22.6.4.1 of the code, critical section for shear is located such that the perimeter (b_0) is minimum, but the critical section need not be closer than d/2 from the structural elements. The shear perimeter (critical section) can be constructed with straight lines at a distance "d/2" from the edges of square or rectangular columns, concentrated loads, or reaction areas. For circular or polygon-shaped columns, the shear perimeter is calculated using an equivalent square area of the columns.

If there is an opening located in the column strip of the slab or at a distance closer than 10 times the thickness of the slab from the concentrated load or reaction area, then the shear perimeter is reduced as illustrated in figure R2.6.4.3 of the code.

Where shear reinforcement is provided in a slab, a critical section located outside the shear reinforcement is also considered, with a polygon shape as illustrated in figure R22.6.4.2 of the code.

The shear strength (v_c) provided by concrete (in accordance with section 22.6.5.1 of the code) is the least of

a. $4\sqrt{f_c'}$

b. $\left(2+\dfrac{4}{\beta}\right)\sqrt{f_c'}$

c. $\left(2+\dfrac{\alpha_s d}{b_0}\right)\sqrt{f_c'}$

where

β is the long-side-to-short-side ratio of the column or loaded area

α_s = 40 for interior columns, 30 for edge columns, and 20 for corner columns

b_0 is the critical section perimeter (in.)

The maximum value of the shear capacity of concrete with shear reinforcement and the maximum shear that can act on a two-way slab are defined in tables 22.6.6.1 and 22.6.6.2 of the code.

Stirrups are allowed to be used to resist shear if the effective depth of the slab is at least 6 in^2 and 16 times the diameter of the stirrups used. Section 8.7.6 of the code permits single-leg, simple-U, multiple-U, and closed stirrups for shear reinforcement of two-way slabs. If the stirrups are placed perpendicular to the column face, then the first stirrup shall be placed at a distance d/2 from the face of the column and the spacing between the stirrups shall not exceed d/2. If the stirrups are placed parallel to the column face, then the spacing between the stirrups shall not exceed 2d. In accordance with section 22.6.7 of the code, the shear strength provided by the stirrups is

$$v_s = \frac{A_v f_{yt}}{b_0 s} \qquad \text{(ACI Equation 22.6.7.2)}$$

where A_v is the sum of the area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the column section and s is the spacing of the peripheral lines of shear reinforcement in the direction perpendicular to the column face.

Headed studs are also permitted to be used to resist shear forces in two-way slabs. They shall be placed perpendicular to the plane of the slab. In order to obtain the maximum height of the stud assembly, section 8.7.7 of the code specifies that the following be subtracted from the overall thickness of the slab:

a. Concrete cover on the top flexural reinforcement
b. Concrete cover on the base rail
c. One-half the bar diameter of the flexural tension reinforcement

The headed shear stud reinforcement location and spacing shall be in accordance with table 8.7.7.1.2 of the code and is illustrated in the design example. Section 22.6.8 provides the specification of the headed shear stud reinforcement design. The strength provided by the headed shear stud reinforcement is

$$v_s = \frac{A_v f_{yt}}{b_0 s} \qquad \text{(ACI Equation 22.6.8.2)}$$

where A_v is the sum of the areas of all shear studs on one peripheral line that is geometrically similar to the perimeter of the column section and s is the spacing of the peripheral lines of headed shear stud reinforcement in the direction perpendicular to the column face.

$$\frac{A_v}{s} \geq 2\sqrt{f_c'}\,\frac{b_0}{f_{yt}} \qquad \text{(ACI Equation 22.6.8.3)}$$

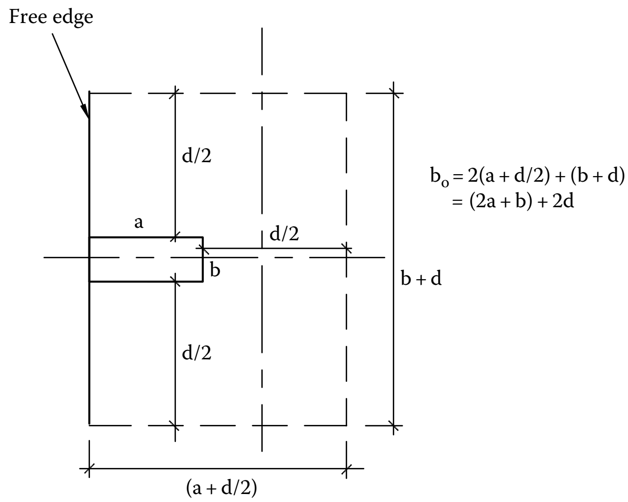


FIGURE 4.5 Edge rectangular column.

Section 22.6.9 of the code allows the use of shearhead, which shall not be deeper than 70 times the web thickness of the steel shape. As long as the remaining portion of the shearhead has the capacity to resist the shear attributed to it, the end of the shearhead is allowed to be cut at angles of at least 30° with the horizontal. The compression flange of the shearhead shall be within 0.3d of the compression surface of the slab (Figure 4.5). The required plastic moment strength of each arm of the shearhead shall be a minimum:

$$M_p \geq \frac{V_u}{2Øn}\left[h_v + \alpha_v\left(l_v - \frac{c_1}{2}\right)\right] \qquad \text{(ACI Equation 22.6.9.6)}$$

where
- M_p is the required plastic moment strength of each arm of the shearhead (lb-in)
- V_u is the applied shear force (lb)
- Ø is the strength reduction factor corresponding to tension-controlled members
- h_v is the depth of shearhead cross section (in.)
- n is the number of shearhead arms
- l_v is the minimum length of shearhead arm (in.)
- c_1 is the dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of span for which moments are being determined (in.)
- α_v is the ratio of the flexural stiffness of shearhead arm to that of the surrounding slab section

The nominal flexural strength contributed to each slab column strip by a shearhead (M_v) shall be as follows, and shall not exceed 30% of M_u in each slab column strip nor change in M_v in column strip over length (l_v):

$$M_v \leq \frac{Ø\alpha_v V_u}{2n}\left(l_v - \frac{c_1}{2}\right) \qquad \text{(ACI Equation 22.6.9.7)}$$

The critical section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at a distance of $(3/4)\left[l_v - (c_1/2)\right]$ from the column face and shall be located so that b_0 is a minimum but need not be closer than d/2 to the edge of the supporting column. The factored shear force (v_u) in this critical section shall not be greater than $Ø7\sqrt{f_c'}$.

4.5 TORSION

The Code ignores the contribution of concrete to the torsional strength, and in case of combined torsion and shear design, it does not consider the shear strength of concrete. Section 22.7 of the code addresses torsional strength of concrete members, which defines the following terms—cracking torsion and threshold torsion.

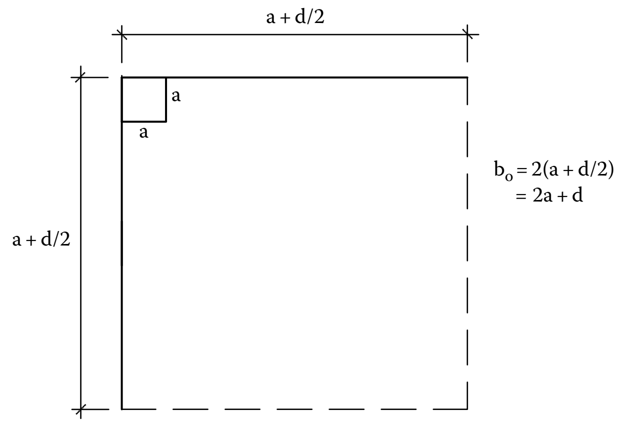


FIGURE 4.6 Corner square column.

Cracking torsion (T_{cr}) is derived by replacing the actual section with an equivalent thin-walled tube of thickness (t) equal to $0.75A_{cp}/P_{cp}$ prior to cracking. The area (A_o) enclosed by the wall is $2A_{cp}/3$. Cracking is assumed to occur when principal tensile stress reaches $4\sqrt{f_c'}$. A_{cp} is the area enclosed by an outside perimeter of concrete cross section.

$$T_{cr} = 4\sqrt{f_c'}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

where

A_{cp} is the area enclosed by outside perimeter of concrete cross section (in^2)
P_{cp} is the outside perimeter of concrete cross section (in.)
A_o is the gross area enclosed by torsional shear flow path (in^2)

Threshold torsion (T_{th}) is defined as ¼ of the cracking torsional moment (T_{cr}). The values of threshold torsion are specified in tables 22.7.4.1 (a) and (b) of the code.

If $T_u < øT_{th}$, then torsional effects are neglected. The value of $\sqrt{f_c'}$ used in the torsional calculations shall be less than or equal to 100 psi. If $T_u \geq øT_{th}$, then the member shall be designed for T_u.

Torsional strength is lesser of the following in accordance with section 22.7.6.1 of the code:

$$T_n = \frac{2A_oA_tf_{yt}}{s}Cot(\theta) \qquad \text{(ACI Equation 22.7.6.1a)}$$

$$T_n = \frac{2A_oA_tf_{yt}}{P_h}Tan(\theta) \qquad \text{(ACI Equation 22.7.6.1b)}$$

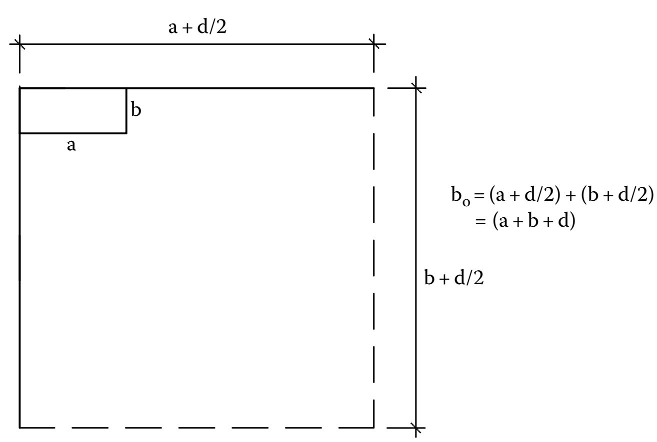


FIGURE 4.7 Corner rectangular column.

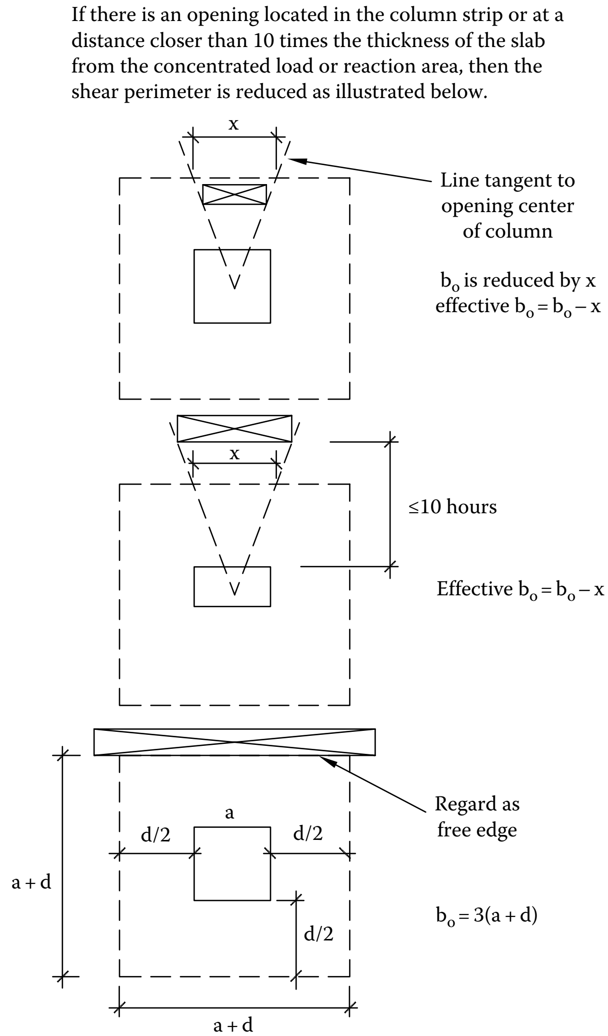


FIGURE 4.8 Shear perimeter reduction.

A_o is defined above. It is also permissible to take A_o as 0.85 times area enclosed by the centerline of the outmost closed transverse torsional reinforcement, A_{oh} (in^2). A_t is the area of longitudinal torsional reinforcement. P_h is the perimeter of the centerline of the outermost closed stirrup (in.). f_{yt} is the tensile strength of the longitudinal torsional reinforcement (psi). θ shall be such that ($30^\circ \leq \theta \leq 60^\circ$). It can be taken 45° for non-prestressed concrete elements.

For a solid section, the cross-sectional dimension shall be selected to satisfy the following inequality to reduce excessive cracking and to minimize the potential for crushing of the surface concrete due to inclined compressive stress due to shear and torsion:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \leq \text{Ø}\left(\frac{V_c}{b_w d} + 8\sqrt{f_c'}\right) \quad \text{(ACI Equation 22.7.7.1a)}$$

For hollow sections, please refer to ACI Equation 22.7.7.1b and section 22.7.7.1.2 of the code. Hollow sections are not dealt with in this book.

4.6 SHEAR FRICTION

Shear friction is addressed in section 22.9 of the code. The ACI procedure for the design of shear friction is explained as follows:

a. The forces (V_u) across the shear place are calculated using the load combinations discussed in Chapter 3 of the book.
b. If the shear friction is perpendicular to the shear plane, nominal shear strength is calculated as

$$V_n = \mu A_{vf} f_y \quad \text{(ACI Equation 22.9.4.2)}$$

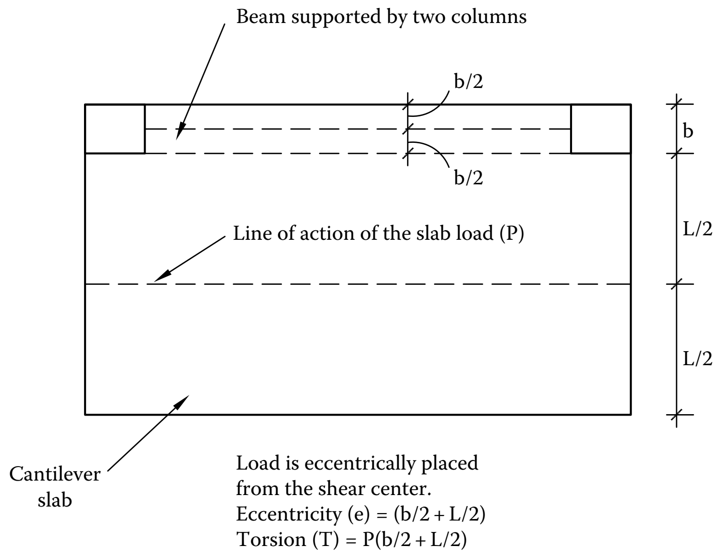


FIGURE 4.9 Torsion.

c. If the shear friction is inclined to the shear plane and the shear force induces tension in the shear friction reinforcement, nominal shear strength is calculated as

$$V_n = A_{vf} f_y \left(\mu \, Sin\,\alpha + Cos\,\alpha\right) \quad \text{(ACI Equation 22.9.4.3)}$$

where
μ is the coefficient of friction obtained from table 22.9.4.2 of the code for the condition under consideration
A_{vf} is the area of reinforcement crossing the shear plane to resist shear (in^2)
α is the angle between shear friction reinforcement and the assumed shear plane in case of inclined reinforcement

d. The maximum value of the nominal shear strength (V_n) across the shear strength for normal weight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately ¼ in. shall be least $0.2f'_cA_c$, $\left(480 + 0.08f'_c\right)A_c$, and $1600A_c$ (Figure 4.9).
e. If there is permanent compressive force across the shear plane, the code allows it to be added to $A_{vf}f_y$ to calculate A_{vf}.
f. If there is a tension across the shear plane due to restraint of deformations caused by temperature change, creep, and shrinkage, the area of reinforcement required to resist it shall be added to the area of reinforcement required by the shear friction crossing the shear plane.
g. $\Phi V_n \geq V_u$, where Φ is the strength reduction factor as discussed in Section 5.14 of the book.
h. Details of shear friction reinforcement: Proper details of the shear friction reinforcement shall be provided on the construction drawings. If there is no moment acting across the shear plane, then the shear friction reinforcement shall be distributed uniformly along the shear plane to minimize crack width; otherwise, the shear friction reinforcement shall be placed in the flexural tension zone. Anchorage of the reinforcement is developed by bond or a mechanical device or threaded rods. The bond is usually provided by epoxy grout in the holes in which the reinforcement is inserted.

4.7 ASSIGNMENTS

1. A rectangular beam with an over-dimension of 8 in. × 24 in. has an effective depth of 22 in. It is reinforced with three #6 bars at the bottom. The strength of concrete is 5,000 psi and the strength of steel is 60,000 psi. Calculate the nominal flexural strength (M_n).
2. In problem (1), if the bending moment acting is 100 kfeet, calculate the compressive and tensile stresses caused by it on the concrete and also the tensile stress on the steel.
3. If the shear reduction factor (ø) = 0.75, calculate the shear capacity of the beam,
   a. If there is no axial force
   b. If there is an axial compressive force of 100 kips
4. If the shear force acting on the beam of problem (1) is 100 kips and shear reduction factor (ø) = 0.75, then what should be the nominal strength (V_s) provided by the shear stirrups.

5. Calculate the shear strength (v_c) provided by a 10 in. thick concrete slab (with an effective depth of 9 in.) supported by a square interior column. The compressive strength of the concrete of the slab is 5000 psi.
6. What is the maximum shear strength of concrete $\left(f_c' = 5000\,\text{psi}\right)$ of two-way slabs for the following cases:
   a. Slab reinforced with stirrups
   b. Slab reinforced with headed shear stud
7. An 8 in. thick slab has 10 legs of #3 stirrups in a peripheral line perpendicular to a 12 in. × 12 in. column. The peripheral lines of stirrups are spaced at 12 in. on center. The strength of steel used for the stirrups is f_{yt} = 60,000 psi. Calculate the strength provided by the stirrups (v_s).
8. Solve problem (2) of Chapter 1 using the equivalent rectangular stress diagram.

5 Other ACI Code Requirements

5.1 INTRODUCTION

In Chapter 4, we dealt with the code requirements of strength of the reinforced concrete elements. In this chapter, we will be dealing with other important code requirements in the design of reinforced concrete buildings. This version of the code specifies the structural system and methods of analysis, which was not detailed in the previous versions of the code. We will also deal with serviceability (which includes deflection, shrinkage, cracking, and durability), sustainability, structural integrity, and fire resistance. The design properties of concrete and steel, the two design methods (working stress and ultimate load), design of embedments and connections, strength reduction factors, and lightweight concrete are also discussed. This book does not deal with lightweight concrete, and hence in all equations, the lightweight concrete factor (λ) is taken as 1.0 and not shown in the equations.

5.2 STRUCTURAL SYSTEM

The latest version of the code has defined the structural system in section 4.4.1 but has permitted an exception to this system in section 4.4.3 provided it is approved by the building official. The structural system shall consist of the following:

1. Floor and roof construction including one-way and two-way slabs
2. Beams and joists
3. Columns
4. Walls
5. Diaphragms
6. Foundations
7. Joints, connections, and anchors to transmit force between members

Diaphragms will not be dealt with in this book because lateral load analysis is beyond the scope of this book. However, Chapter 14 on wind load analysis is included for the benefit of the students. Slabs, beams, columns, walls, and footings are discussed in separate chapters. Joints, connections, and anchors are included in these chapters, as required.

The goal in the design of a concrete structural element is to ensure that the factored loads acting on any element are adequately resisted. Section 4.4.5 of the code specifies that the structural element should also accommodate volume change and differential settlement.

5.3 STRUCTURAL ANALYSIS

Chapter 6 of the code lists the analytical methods permitted for the determination of forces on various structural elements (sections 6.2.3 and 6.2.4) as follows:

1. Simplified method for the analysis of continuous beams and one-way slabs for gravity loads
2. First-order analysis
3. Elastic second-order analysis
4. Inelastic second-order analysis
5. Finite element analysis
6. Direct design method and equivalent frame method for two-way slabs
7. Alternate method for slender walls for out-of-plane effects

The methods 1, 2, 6, and 7 are discussed in Chapters 6 through 9 of the book. The methods 3, 4, and 5 are beyond the scope of this book. Chapter 6 of the code also provides modeling assumptions and arrangement of live loads in sections 6.3 and 6.4, respectively, which are discussed in the design chapters.

5.4 STRENGTH

The strength of a structural element depends on its material properties and its capacity to withstand axial, flexural, shear, and torsional stresses. A structural element fails the strength criterion when the stress induced by the intended loading exceeds the capacity of the structural material to resist the load, or when the strain is so great that the element no longer fulfills its function and the element yields.

The design strength of structural elements shall be greater than the required strength established by the code using the required loads and load combinations, which are discussed in Chapter 3 of the book. The structural elements and their connections shall be designed to resist the applied axial, shear, flexural, and torsional stresses. The design strength is determined in accordance with the requirement of the code with a strength reduction factor specified in table 21.2.1 of the code. The strength of the element determined using the code is called the nominal strength. The concept of the determination of design strength, nominal strength, and strength reduction factor in accordance with the code is illustrated in the design examples.

Design strength = Φ times the nominal strength (S)
Design strength $\geq$ required strength (U)
Φ = strength reduction factor
$\Phi S \geq U$

The code requirements for the strength of reinforced concrete building structural elements to resist bending, axial forces, combination of axial and bending loads, shear, torsion, and shear friction are discussed in Chapter 4 of the book.

5.5 SERVICEABILITY

Concrete structures and elements are required to be serviceable and provide the required strength to resist the intended loads during their life-span. Control of cracking and avoidance of excessive deflection and vibration are required for the concrete to be serviceable. The serviceability requirements address issues such as deflection, cracking, shrinkage, and durability.

5.5.1 Deflection

Deflection in a structural element is the degree to which it is displaced under the applied loads. It is expressed as a differential change in length between its upper and lower portions. A flexural member such as a slab or a beam undergoes compression and tension. The compression fiber of the elements shortens and the tension fiber elongates. The extent of the deflection of a structural element depends on the slope of the deflected shape of the element. Other factors influencing the deflection of a structural element are elastic strains, creep strains, shrinkage and moisture changes, temperature differentials, and the form of structure itself. From the serviceability point of view, deflections should be recognized during the design of the structural elements.

The control of deflection in reinforced concrete elements is addressed in section 24.2 of the code. The element should be sized such that it is stiff enough to limit deflections, thereby not adversely affecting the serviceability and the strength of the building. The code specifies the requirements to control deflection in beams, one-way slabs, and two-way slabs. The minimum thickness of one-way slabs is specified in table 7.3.1.1 of the code. The minimum thickness of two-way slabs is specified in tables 8.3.1.1 and 8.3.1.2 of the code. The minimum thickness of beams is specified in table 9.3.1.1 of the code. If the depth of the beams and the thickness of the slabs do not comply with the above requirements, then the deflections need to be calculated based on the effective moment of inertia in accordance with chapter 24 of the code. Additional long-term deflections caused by shrinkage and creep shall also be determined. Examples of the calculations of deflections of beams and slabs are provided in design chapters.

5.5.2 Cracking of Concrete

Cracks can be either active or dormant. Active cracks exhibit movement in direction, width, or depth over a period of time. Dormant cracks remain unchanged. They may not be dangerous themselves but they allow penetration of moisture. Cracking could be shrinkage cracking, settlement cracking, plastic cracking, structural cracking, tension cracking, rust cracking, and thermally induced cracking. Shrinkage cracking occurs only in unhardened concrete, and it often consists of series of cracks parallel to the span of the member. Plastic cracking is also a type of shrinkage cracking occurring only in unhardened concrete as diagonal lines. It is caused by rapid drying of the surface of the concrete element due to delays in applying the curing membrane. Settlement cracking is caused by local restraining of unhardened concrete around the reinforcement or some other obstruction. Structural cracking is caused by the corrosion or overstressing of concrete reinforcement. Tension cracking is caused by elongation of the reinforcement in tension zones, usually seen around columns in flat slabs and on beam soffits near the midspan. Rust cracking is caused by inadequate reinforcement cover. Thermally induced cracking is caused by changes in temperature. Cracks are defined by pattern, width, and depth. Patterns include the direction of the crack such as longitudinal, diagonal, and transverse cracks. Cracks can be classified based on their width as fine (1 mm), medium (1–2 mm), and wide (>2 mm).

5.5.3 Shrinkage

Shrinkage of concrete is a principal factor in controlling deflection and cracking. During the design of concrete elements, an accurate assessment of the properties of the actual concrete is difficult because it depends on the weather conditions, materials used, and workmanship. The service load behavior depends primarily on the properties of concrete, which behaves in a nonlinear and inelastic manner at the service loads, and it complicates the design due to cracking, shrinkage, creep, and tension stiffening.

Concrete shrinks because of a loss in concrete volume and leads to cracking when base friction or other restraints occur. It also causes curling and warping, which can lead to decreased load-carrying capacity of the structural element. After hardening, concrete begins to shrink, as water not consumed during cement hydration leaves the system. Shrinkage of concrete is a time-dependent strain. Three types of shrinkage are important: plastic shrinkage, chemical shrinkage, and drying shrinkage.

Plastic shrinkage occurs in wet concrete, causing significant reduction in the volume of concrete during the settling process due to capillary tension in water present in the pores. While the concrete is still wet, the reinforcing steel is still not bonded to the concrete and hence is ineffective in controlling the cracks at that stage. Chemical shrinkage is caused due to various chemical reactions within the cementitious material. Drying shrinkage is the reduction in the volume of concrete caused due to the loss of water during the drying process. If the water–cement ratio of the concrete is less, drying shrinkage is less.

Section 5.3.6 of the code addresses volume change of concrete. The code defines "T" in chapter 2 as the cumulative effect of temperature creep, shrinkage, differential settlement, and shrinkage compensating concrete. Section 5.3.6 of the code recommends a load factor not less than 1.0 in combining the effects of "T" with other loads. Expansion joints and control joints (Figures 5.1 and 5.4) help limit the strains caused by volume changes. Section 24.4 of the code specifies reinforcement for shrinkage and temperature in structural slabs, which is discussed in Chapter 6 of the book.

Usually, in large concrete slabs on grade, saw-cut control joints are provided at an approximate spacing of 20 feet in each direction after the initial setting of the concrete to prevent cracks on the surface of the concrete. These cracks give an ugly appearance to the top surface of the concrete, and if finishes such as tiled flooring are present on top of the concrete, they get cracked too. Construction joints are provided when continuity in the casting of concrete slabs or mats needs to be broken (Figure 5.2). Expansion joints are provided in buildings that are longer than 300 feet to account for the expansion of structural elements during the period of high temperatures (Figure 5.4).

5.5.4 Durability

The durability of concrete is addressed in sections 19.3 and 26.4 of the code. Protection to reinforcement from corrosion is addressed in section 20.6 of the code. Durability is the capability of concrete to withstand the external effects (weathering

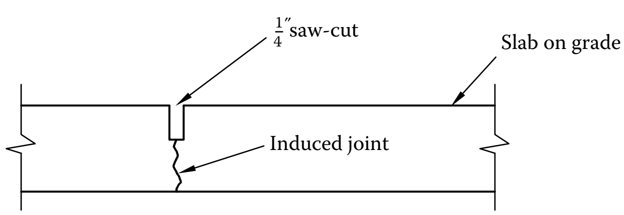


FIGURE 5.1 Control joint.

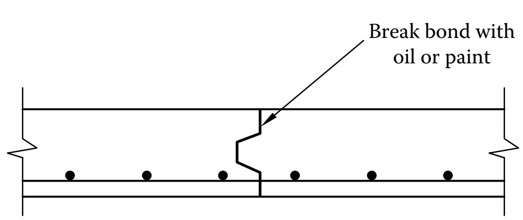


FIGURE 5.2 Construction joint.

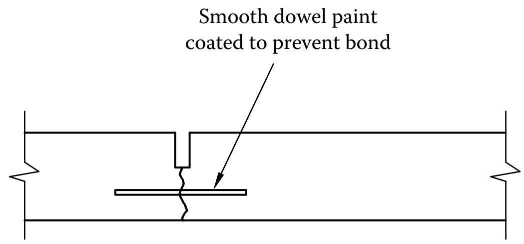


FIGURE 5.3 Contraction joint.

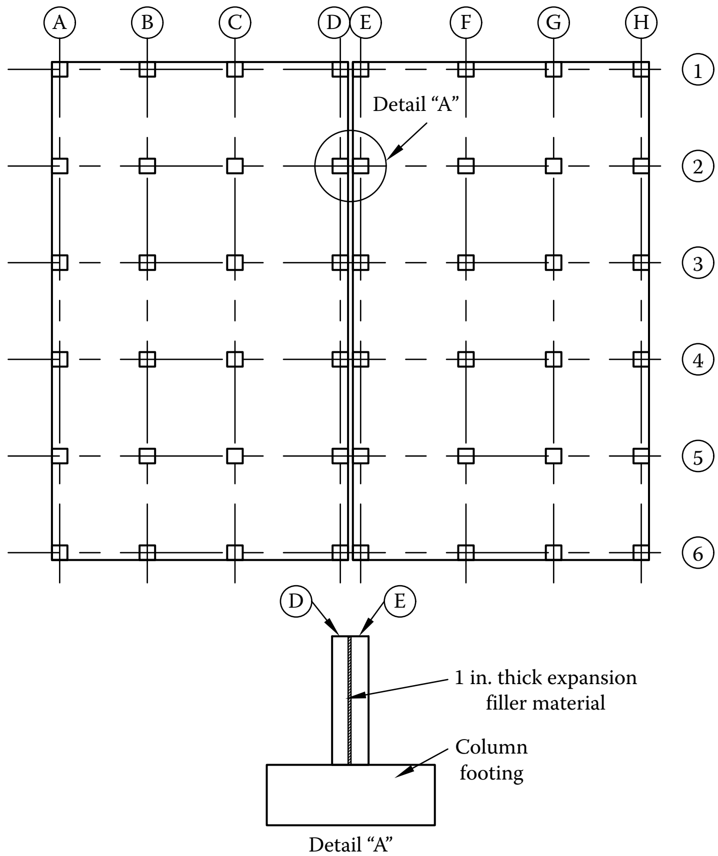


FIGURE 5.4 Expansion joint.

action, chemical attack, and abrasion) on the concrete elements. The constituent parts of the concrete (cement, fine aggregates, coarse aggregates, and admixtures) and the interaction between them, the way concrete is placed and cures, the environment to which it is exposed to, and the way it is maintained, determines the durability of concrete. Apart from the sustaining gravity loads and more ephemeral loads such as wind and earthquakes, concrete has to sustain the environmental conditions. Requirements for durability of concrete depend on the environmental condition it is exposed to. For example, balconies in an oceanfront condominium building would be exposed to chloride ion attacks. The damage caused to the reinforced concrete due to chloride ions is discussed later in this section.

If moisture penetrates inside the concrete due to humidity or rain, it can result in rusting of the steel reinforcement, which causes spalling of concrete. Spalling is the broken, flaked, or pitted concrete on the surface of a concrete element. Concrete is resistant to ultraviolet rays of solar radiation and is not harmed by organic attacks. Unlike wood, bugs and pests cannot harm concrete. If concrete is properly designed and mixed, it can resist severe environmental conditions such as extremely high temperature, freezing, and thawing. Care should also be taken to make concrete resistant to sulfate and chloride ion attacks. Table 19.3.1.1 of the code classifies exposure into four categories—F, S, W, and C. The requirements of the concrete mixtures in accordance with exposures are specified in table 19.3.2.1 of the code. The code restricts the water–cement ratio of the concrete and specifies a minimum strength for the exposure. There are also other requirements such as limitations on air content, cementitious materials, admixtures containing calcium chloride, and water-soluble chloride ion content. For example, the concrete exposed to severe weather near the ocean in South Florida shall not have a water–cement ratio exceeding 0.40 and a minimum compressive strength of 5000 psi. The four exposure categories are discussed below.

5.5.4.1 Freezing and Thawing

Exposure category "F" deals with severity of freezing and thawing, which is most destructive to concrete when it is wet and has presence of deicing chemicals. Deterioration is caused by the freezing of water and subsequent expansion in the paste, the aggregate particles, or both. Where concrete is exposed to freezing and thawing, air-entraining admixtures are added during the batching and mixing of concrete and a low water–cement ratio (less than 0.40) is maintained. The air bubbles within the concrete accommodate the expansion of water into ice and thus relieve the internal pressure generated and help the concrete to withstand freezing and thawing without distress. The air entrainment requirement for exposure category "F" is provided in table 19.3.3.1 of the code.

5.5.4.2 Sulfate Attacks

Exposure category "S" deals with the sulfate chemical attack on concrete. Chemicals such as calcium sulfate, sodium sulfate, and magnesium sulfate undergo a chemical reaction with the hydrated compounds in the hardened cement paste and induce pressure to deteriorate concrete. Sulfate attacks are more severe when concrete is exposed to wet and dry cycles. Cement complying with the ASTM C150, ASTM C595, ASTM C485, ASTM C1157, ASTM C618, ASTM C989, and ASTM C1240 is specified in table 26.4.1.1.1(a) of the code. Low water–cement ratios and high compressive strength are also required to resist sulfate attacks on concrete.

5.5.4.3 Permeability

Permeability of concrete is the state of concrete that allows liquid or gas to pass through it and possibly affect its durability. The permeation of water into concrete reduces the durability of concrete by becoming the cause of deterioration of concrete due to chemical and physical processes that it may impose on the concrete. Exposure category "W" addresses the requirements of low permeability of concrete, which restrains the infiltration of water with chemicals containing sulfate and chloride ions. This category deals with concrete in contact with water but not exposed to freezing and thawing, chlorides, and sulfates.

5.5.4.4 Chloride Ions

Reaction of chloride ions is the primary cause of the corrosion of reinforcing steel. When chloride ions intrude the porous concrete, the presence of oxygen and moisture encourages a chemical reaction, which causes corrosion of the reinforcing steel, which in turn spalls or cracks the concrete and reduces the strength of the structural element. Chloride-containing admixtures can also cause corrosion. The exposure category "C" of the code addresses the requirement of limitation of chloride ion exposure to the concrete, maximum water–cement ratio, and minimum compressive strength of concrete.

There are four classes (F0, F1, F2, and F3) in F category, four classes (S0, S1, S2, and S3) in S category, two classes (W0 and W1) in W category, and three classes (C0, C1, and C2) in C category. The severity is defined by the class, and it increases with the number assigned to the class.

5.5.4.5 Alkalinity and Silica

Further, concrete can be subject to reaction between silica present in the aggregates and the alkaline nature of the cement (which has the presence of potassium and sodium). The issue can be resolved by selecting proper aggregates and the use of cementitious materials such as fly ash and slag cement. The ASTM C1778 provides a guidance for the reduction of this reaction.

5.5.4.6 Abrasion

Concrete is subject to abrasion in the form of weather factors and use of the concrete. Conditions such as rapidly moving water and floating ice on concrete and vehicular movement on concrete cause abrasion to the concrete surface. The strength of concrete provides resistance to abrasion of the concrete.

5.5.4.7 Corrosion of Reinforcement

Corrosion of reinforcing steel and other embedded metals is the leading cause of deterioration in concrete. When steel corrodes, the resulting rust occupies a greater volume than the steel. This expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling. The protection to the reinforcement is covered in table 20.6.1.3.1 of the code.

5.5.4.8 Some Practical Considerations

Some of the oceanfront buildings constructed in South Florida, constructed before the plant batching of concrete, had used beach sand to make in situ concrete. After 30–40 years, severe damages to exposed concrete elements have occurred. Cracking and spalling of concrete in balconies and parking garages have been common, and buildings require a major retrofit. The 40-year recertification process in South Florida requires buildings to be inspected by a licensed architect or engineer and certified by them for continued occupancy. The Florida Building Code (2004) had mandated stringent requirements for exterior balcony slabs. Section 1926.5.5 of the Florida Building Code (2004) required a slope of 1/8 unit in 12 units and either a concrete cover of 1.5 in. to the reinforcement resisting negative moment or placement of the slab reinforcement under the supervision of a licensed architect and engineer, water–cement ratio of concrete not exceeding 0.40, minimum compressive strength of concrete 4750 psi, zinc-coated reinforcement, and a prepared surface of the slab (with alkyl alkoxy silane penetrant).

5.6 SUSTAINABILITY

The code specifies the minimum requirements for strength, serviceability, and durability of concrete structures. Section 4.9.1 of the code allows the Engineer of Record to specify the sustainability requirements in addition to the other mandatory requirements of the code. A sustainable concrete structure is constructed to ensure that the total environmental impact during its life cycle, including its use, will be minimal. Sustainable concrete should have a very low inherent energy requirement, be produced with little waste, be made from some of the most plentiful resources on earth, produce durable structures, have a very high thermal mass, and be made with recycled materials. Sustainable constructions have a small impact on the environment. They use "green" materials, which have low energy costs, high durability, and low maintenance requirements and contain a large proportion of recycled or recyclable materials. Green materials also use less energy and resources and can lead to high-performance cements and concrete. Concrete must keep evolving to satisfy the increasing demands of all its users. Designing for sustainability means accounting for the short-term and long-term environmental consequences in the design.

5.7 STRUCTURAL INTEGRITY

Typically, structures are designed for normal loads as discussed in Chapter 3 of the book, but in reality, a possibility of abnormal loads to damage the structure locally or the complete structure always exists. Blasts, accidents, and tornadoes are common examples of abnormal loads that may act on the structure and may not be considered during the design phase. Local failures of major elements could lead to the collapse of the structure. On May 16, 1968, the corner column in the 22-storied East London tower Ronan Point collapsed due to a cooking gas cylinder explosion at the 19th floor, which led to the collapse of the building. This is called the "progressive collapse," and it changed the building regulations in the United Kingdom. Unless, a specialized design is performed for buildings, it is impractical to design every building to resist these types of loads. However, minor reinforcement details can help the structure to partially resist these abnormal loads. For two-way slabs (section 8.7.4.2) and beams (section 9.7.7), the code addresses such reinforcement details, which are discussed in the design chapters.

With a little redundancy in the detailing of the reinforcement, the integrity of the structure can be enhanced to account for unpredicted loads or faulty construction. Even though local damages may occur, the overall safety of the structure is enhanced. Like discussed earlier, the positive reinforcement shall be spliced at the support and the negative reinforcement shall be spliced near the midspan because of the intensity of moments. The longitudinal reinforcement of a beam shall pass through the region bounded by the vertical reinforcement of the column. If there is damage to the column, then the possibility of the beam reinforcement to burst is less if it is bounded by the column reinforcement. One-fourth of the longitudinal reinforcement (a minimum of two bars) shall be continuous along all spans of the continuous beams with splicing only as discussed above. For details on bar placement, refer to section 9.7.7 of the code.

5.8 FIRE RESISTANCE

The ACI 216 report "Standard Method for Determining Fire Resistance of Concrete and Masonry Construction Assemblies" and the International Building Code (IBC) (2015) address the fire resistance requirements of the concrete elements.

Fire resistance is the property of materials or their assemblies that prevents or retards the passage of excessive heat, hot gases, or flames under conditions of use. Fire resistance rating is the period of time, a building element, component, or assembly that maintains the ability to confine a fire and continues to perform a given structural function. It is determined by performing tests on the building elements.

The requirements of fire rating are based on the construction classification defined in the IBC (2015). IBC (2015) classifies the construction of all types of buildings into five types (I, II, III, IV, and V), and each type has two subtypes (A and B).

Table 601 of the IBC (2015) defines the fire resistance requirement in terms of hours for primary structural frame, exterior and interior load-bearing walls, and floor and roof construction. For example, type IA construction is used for tall buildings, and the primary structural frame shall resist fire for 3 hours and the roof shall resist fire for 1½ hours before they collapse. Table 721.1(2) of the IBC (2015) provides the minimum thickness of the material to resist the fire for that duration as specified in table 601 of the IBC (2015). In order for a siliceous aggregate concrete to resist the fire for 3 hours, the minimum thickness of material should be 6.2 in^2, and for 1½ hours, it should be $\frac{(5+3.5)}{2} = 4.25$ in^2 in accordance with table 721.1(2) of the IBC (2015). Typically, the thickness requirement for structural elements for strength and serviceability is higher than the fire resistance requirements.

However, structural steel buildings may require treatments by wrapping the structural elements by fire-rated components to provide the required fire rating. Though ACI 318-14 does not specify the minimum reinforcement cover requirements specifically for fire rating, IBC (2015) tables 722.2.3(1) through 722.2.3(5) provide cover requirements for slabs and beams. Section 722.2.2.3 of the IBC (2015) deals with the cover requirements of reinforced concrete elements. For higher fire resistance, IBC (2015) may require more concrete cover to the reinforcement than required by ACI 318-14 for various exposures discussed in Section 5.5 of the book.

5.9 DESIGN PROPERTIES OF CONCRETE

The design properties of concrete are selected based on section 19.2 of the code. For general-purpose applications, the minimum compressive strength of normal weight and lightweight concrete shall be 2500 psi, and there is no limit for the maximum compressive strength. For special moment frames and special structural walls, the minimum compressive strength of normal weight and lightweight concrete shall be 3000 psi, and there is no limit for the maximum compressive strength for normal weight concrete. However, for special moment frames and special structural walls, the maximum compressive strength of lightweight concrete shall not exceed 5000 psi. These strengths shall be attained in 28 days.

The modulus of elasticity of concrete is the ratio of normal stress to the corresponding strain for tensile or compressive stresses below the proportional limit of concrete. The modulus of elasticity of concrete is a function of the modulus of elasticity of the aggregates and the cement matrix and their relative proportions. The modulus of elasticity of concrete is relatively constant at low stress levels but starts decreasing at higher stress levels as matrix cracking develops. The modulus of elasticity of concrete is calculated based on the weight and 28 days compressive strength of the concrete:

$$E_c = w_c^{1.5} 33\sqrt{f_c'}\ (\text{in psi})\ \text{for } w_c \text{ between 90 and 160 pcf} \qquad \text{(ACI Equation 19.2.2.1.a)}$$

$$E_c = 57{,}000\sqrt{f_c'}\ (\text{in psi})\ \text{for } w_c \text{ for normal weight concrete} \qquad \text{(ACI Equation 19.2.2.1.b)}$$

where

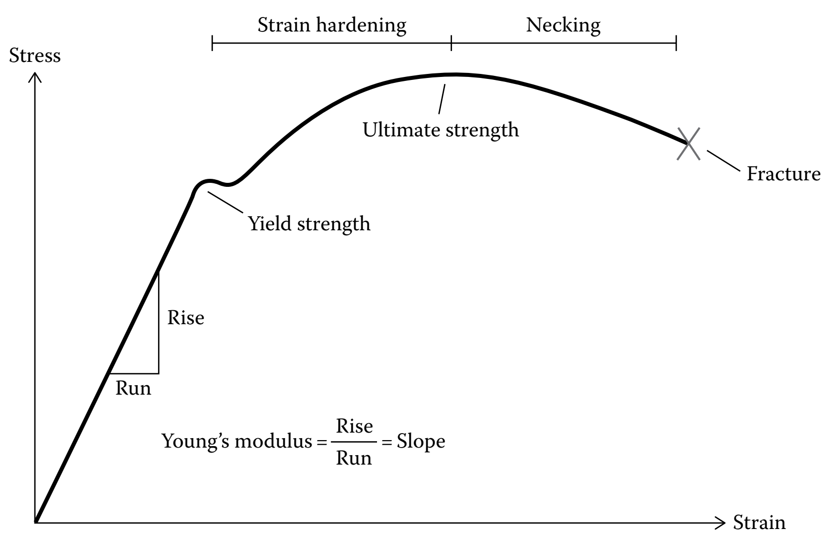
E_c is the modulus of elasticity of concrete (psi)
w_c is the weight of concrete (pcf)
$\sqrt{f_c'}$ is the 28 days compressive strength of concrete (psi)

The modulus of rupture of concrete is the stress in concrete just before it yields in a flexure test. It is the tensile strength of the concrete and it varies between 10% and 15% of the compressive strength. It is neglected during the design process. However, it is important during the evaluation of cracking and deflections at the service load. The transverse bending test is most frequently employed, in which a specimen having either a circular or rectangular cross section is bent until fracture or yielding using a three-point flexural test technique. The flexural strength represents the highest stress experienced within the material at its modulus of rupture. For normal weight concrete, modulus of rupture (f_r) is based on its 28 days compressive strength:

$$f_r = 7.5\sqrt{f_c'}\ (\text{psi}) \qquad \text{(ACI Equation 19.2.3.1)}$$

5.10 DESIGN PROPERTIES OF STEEL

Elasticity is the property of a material by virtue of which deformation caused by applied load disappears upon removal of the load. In other words, the elasticity of a material is its power of coming back to its original position after deformation, when the stress or load is removed. The plasticity of a material is its ability to undergo some degree of permanent deformation without rupture or failure. Generally, plasticity increases with increase in temperature. Plasticity is the deformation of a (solid) material undergoing nonreversible changes of shape in response to applied forces. For example, a solid piece of metal being bent or pounded into a new shape displays plasticity as permanent changes occur within the material itself. In engineering, the transition from elastic behavior to plastic behavior is called "yield." A yield strength or yield point is the stress at which a material begins to deform plastically. Prior to the yield point, the material will deform elastically and will return to its original shape when the applied stress is removed. Once the yield point is passed, some fraction of the deformation will be permanent and nonreversible. Ultimate tensile strength is the capacity of a material or structure to withstand loads tending to elongate. Ultimate tensile strength is measured by the maximum stress that a material can withstand while being stretched or pulled before breaking. Fracture of steel is the separation of steel into two or more pieces under the action of stress. The fracture of steel usually occurs due to the development of certain displacement discontinuity surfaces within the solid. If a displacement develops perpendicular to the surface of displacement, it is called a normal tensile crack or simply a crack; if a displacement develops tangentially to the surface of displacement, it is called a shear crack, slip band, or dislocation. Fracture strength or breaking strength is the stress when a specimen fails or fractures. In the plastic region of steel, as the steel is continuously stressed and goes beyond the yield point, more and more stress is required to produce additional plastic deformation and the metal seems to have become stronger and more difficult to deform. This implies that the metal is becoming stronger as the strain increases. This phenomenon is called strain hardening. After the ultimate stress, the cross-sectional area begins to decrease in a localized region of the specimen, instead of over its entire length. So, a neck is formed as the specimen elongates further. This phenomenon is called necking.



In this book, only the ASTM A615 steel with a specified yield strength (f_y) of 60,000 psi will be used. Properties and the relevant ASTM standards are provided in tables 20.2.2.4a and b of the code. The modulus of elasticity (E_s) of the reinforcing steel is 29,000,000 psi.

The provisions of durability of steel reinforcement are provided in section 20.6 of the code. Concrete cover to reinforcement is provided to protect the steel reinforcement from environment and fire. It is measured from the outer surface of the concrete to the outer surface of the main reinforcement to which it is applied. For example, in a beam, the cover is measured to the outer surface of the main longitudinal reinforcement and not the shear stirrups. The reinforcement cover requirements in accordance with table 20.6.1.3.1 of the code are as follows:

Concrete cast permanently against the ground	3 in^2
Exposed to weather or in contact with the ground	2 in^2 (for #6 through #18 bars)
	1½ in^2 (#5 or smaller bars)
Slabs, joists, and walls not exposed to weather	1½ in^2 (#14 and #18 bars)
	¾ in^2 (#11 or smaller bars)
Beams, columns, pedestals, and tensions ties (not exposed to weather or in contact with the ground)	1½ in^2

Zinc-coated (ASTM A767), epoxy-coated (ASTM A775 or A934), and zinc- and epoxy-coated (ASTM 1055) reinforcing bars are used in a highly corrosive atmosphere.

5.11 DESIGN METHODS

As discussed in Chapter 4, two design methods have been used in the design of reinforced concrete elements—working stress design and strength design. Working stress design was also called the alternate design method till ACI 318-11.

In the working stress design, stresses resulting from the action of service loads calculated using structural mechanics should not exceed certain allowable stresses. The method is also called allowable stress design method (ASD):

$$f \leq f_{allow}$$

where

stress (f) = Mc/I

M is the bending moment at the section (lb-in.)

c is the depth of the centroidal axis (in.)

I is the moment of inertia (in^4)

f_{allow} is the allowable stress by the prevailing code (no more defined in ACI 318-14) (psi)

Working stress method is based on the elastic theory, assuming a straight line stress distribution along the depth of the concrete. The actual loads or working loads acting on the structure are estimated, and 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 (f_c') and the yield strength (f_y). Because of the differences in realism and reliability over the past several decades, the strength design method has displaced the older allowable stress design method.

Strength design method is based on the ultimate strength of the structural members assuming a failure condition, whether due to the crushing of concrete or due to the yield of reinforced steel bars. Although there is additional strength in the bar after yielding (due to strain hardening), this additional strength in the bar is not considered in the analysis or design of the reinforced concrete members. In the strength design method, actual loads or working loads are multiplied by a load factor to obtain the ultimate design loads. The load factor represents a high percentage of factor for safety required in the design. The code emphasizes this method of design.

Strength provided to members (Ultimate Strength) ≥ Strength required to resist factored loads.

The strength provided to the members is defined in the requirement of the code for bending, shear, axial, torsion, etc.

5.12 EMBEDMENTS

Conduits, pipes, and sleeves may be embedded in concrete, but care should be taken that embedments do not damage the concrete. Embedments do not replace the strength lost in displacing the concrete for their volume. The use of bare aluminum in structural concrete is prohibited because it reacts with concrete and, in the presence of chloride ions, may produce an electrolytic reaction with steel reinforcement causing corrosion of the steel that may result in cracking or spalling of the concrete or both. Aluminum embedments in concrete must be suitably coated or covered to prevent direct contact with the concrete. Conduits, pipes, or sleeves passing through these structural elements shall not be placed in locations that would create planes of weakness or distress and compromise the strength of the structural element. Reinforcement with an area at least 0.002 times the area of concrete section shall be provided perpendicular to pipe embedments (section 20.7.4 of the code). The cover to the pipe embedments shall be at least 1½ in^2 for concrete exposed to weather and ¾ in. for concrete not exposed to weather or in contact with the ground (section 20.7.4 of the code). The design and construction documents shall show the embedment's type, size, and location; reinforcement to be placed perpendicular to embedment; concrete cover; and corrosion protection details (Figure 5.5).

Conduits and pipes are permitted to be embedded within a column, provided that such installations do not displace more than 4% of the column cross-sectional area on which the strength is calculated or that is required for fire protection purposes. This allowance is based on the premise that a column is a compression member and that a nominal displacement of concrete with steel will not seriously affect structural integrity.

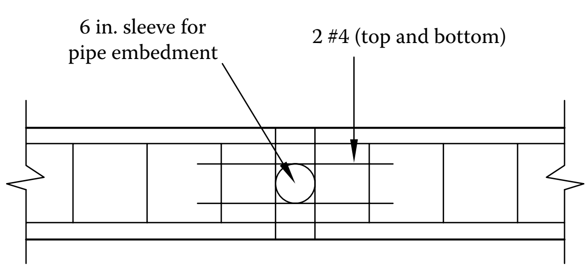


FIGURE 5.5 Embedment detail.

5.13 CONNECTIONS DESIGN

The design of connections between beams and columns and slabs and columns is very important. The connections shall be designed for the transfer of axial forces through the floor. If the gravity and lateral loads transfer of moments at the joints, then the shear resulting from the moment transfer must be considered in the design of the joint.

In high-rise buildings, the concrete compressive strength varies from the columns at the lower floor to the columns at the upper floors. High compressive strengths of 10,000 psi can be used at the lower floor columns, and at the upper floor columns, the strength is reduced to 5,000 psi. However, the compressive strength of the concrete for the slabs at all floors typically remains the same. Typically, in high-rise buildings, a concrete compressive strength of 5000 psi is used for the slabs, unless the slab is a transfer slab. Transfer slabs support columns from upper levels so that the space beneath the slab has less columns for use like parking. The strength of the concrete used in transfer slabs is very high, in order of 10,000 psi. Sections 15.3 and 15.4 of the code specify the following if the compressive strength of concrete of the columns is 1.4 times higher than the compressive strength of concrete of the slabs:

1. The strength of concrete placed at the joint and in the slab for a distance of 2 feet around the columns shall be the same as the strength of concrete of the column below the slabs (Figure 5.6).
2. The strength of concrete used in the design of the joints shall be the strength of the concrete used for the slabs and not for the columns.
3. If the beam–column joint is restrained by laterally supporting it on all four sides by beams or if the slab–column joint is restrained by providing slab on all four sides, then the compressive strength of concrete used in the design of the joint shall be equal to 75% of the compressive strength of concrete used for columns plus 35% of the compressive strength of concrete used for slabs.
4. If the beam–column and slab–column joints are not restrained as described above and are not part of seismic force resisting system, then the area of all legs of transverse reinforcement in each principal direction of the joint shall be the greater of
   a. $0.75\sqrt{f_c'}\,\dfrac{bs}{f_{yt}}$
   b. $50\dfrac{bs}{f_{yt}}$

   where
   b are the dimensions of the column section perpendicular to the direction under consideration (in.)
   s is the center-to-center spacing of legs (in.)
   f_{yt} is the yield strength of reinforcement (psi)

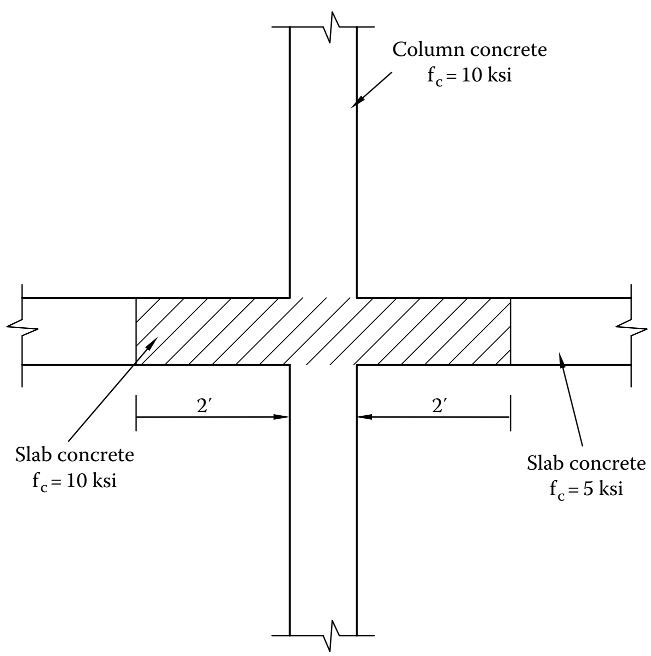


FIGURE 5.6 Joint detail.

5. The above reinforcement shall be distributed in the columns not greater than the height of the deepest beam in case of a beam–column joint and not greater than the slab thickness in case of slab–column joint
6. For a beam–column joint, the spacing of the transverse reinforcement (s) shall not exceed one-half the depth of the shallowest beam.

5.14 STRENGTH REDUCTION FACTORS

Structural elements are designed to carry some more load than the load that is expected during their normal life-span, and there is also a possibility of them being understrength. Overloading can be caused by change in the normal occupancy of the building. For example, a room in a house may be changed to a storeroom with heavy cabinets and goods, thereby increasing the live load tremendously in that localized area. Understrength to a structure is caused due to substandard materials used, poor workmanship, inaccurate dimensioning, and lack of supervision during construction. Overloading is considered in the design load factors (discussed in Section 3.2 of the book). Chapter 21 of the code specifies strength reduction factors for the understrength of the concrete structural elements in table 21.2.1.

Moment, axial or combined moment, and axial: Ø = 0.65–0.090
(For compression-controlled section, Ø = 0.65 and increases from 0.65 to 0.90 as ε_t is increased from ε_{ty} to 0.005, where ε_{ty} is the strain at yield.)
Shear: Ø = 0.75
Torsion: Ø = 0.75
Bearing: Ø = 0.65

5.15 LIGHTWEIGHT CONCRETE

Structural lightweight concrete has a density of 90–115 pcf as compared to 140–150 pcf density of normal weight concrete. It is made of expanded shale, clay, or slate material, fired in a rotary kiln to develop a porous structure. Reducing the weight of the slab helps in a more economical design of columns and footings. Other benefits of lightweight concrete include higher fire rating, energy conservation (because lightweight concrete provides higher insulation values), and better curing due to its porosity. It can be used for bridges, piers, beams, slabs, walls, topping slabs, and composite slabs on metal deck. Air content of lightweight structural concrete is monitored using the ASTM volumetric method (C173) and gravimetric method (C138), and the concrete is air entrained. Excessive slump of lightweight concrete must be avoided to prevent the aggregates segregating from the mortar. To account for the properties of lightweight concrete, a modification factor (λ) is used as a multiplier of $\sqrt{f_c'}$ in all applicable provisions of the code in accordance with section 19.2.4.1 of the code. The value of the modification factor (λ) is based on the composition of aggregates in the concrete and is defined in table 19.2.4.2 of the code for different types of aggregate. For normal weight concrete, λ = 1.0. This book deals with normal weight concrete; hence, λ is eliminated from all ACI equations used.

When lightweight concrete is used, the values for shear, friction, splitting resistance, and bond between concrete and reinforcement and the development length are reduced by the modification factor (λ). The tensile-to-compressive-strength ratio of lightweight concrete is lower as compared to the normal weight concrete. Apart from table 19.2.4.2 of the code, mentioned in the previous paragraph, ACI Equation 19.2.4.3 with the values of average splitting tensile strength (f_{ct}) and the average compressive strength (f_{cm}) is used to determine the value of the modification factor (λ). The values of f_{ct} and f_{cm} are determined using the ASTM C330 procedure:

$$\lambda = \frac{f_{ct}}{6.7\sqrt{f_{cm}}} \leq 1.0 \qquad \text{(ACI Equation 19.2.4.3)}$$

5.16 ASSIGNMENTS

1. Review the table of contents of ACI 318-14 and familiarize yourself with the chapters and how the code is arranged.
2. Review section 722.2.3 of the IBC (2015) along with the relevant tables. Prepare a list of cases where the minimum cover requirement for fire resistance by the IBC (2015) is higher than that required for exposure category by ACI 318-14.

Section II

Design

6 Slabs

6.1 INTRODUCTION

Introduction to the concepts of slabs, their behavior, and bending theory and shear force theory is provided in Chapter 1. Sectional strength of the slabs is discussed in Chapter 4. This chapter deals with the code specifications and the design methodology for one-way and two-way slabs. Several design examples are provided that demonstrate the code specifications for design and sectional strength of the slabs. In the examples, the serviceability requirements of the code are also checked.

6.2 ONE-WAY SLABS WITH BEAMS

Slabs are classified as one-way slabs when their aspect ratio is 2 or greater. The length of the slab is at least twice the width.

6.2.1 Simply Supported Slab

Simply supported one-way slabs would bend (sag) in the shorter direction due to its geometry. Hence, the main reinforcement to resist the bending moment would be placed in the shorter direction. Since the slab is simply supported, it has only one span and is required to have two beams along the long edges to support the slab (Figure 6.1). Unless the two short edges of the slab are loaded (e.g., with a masonry wall above the slab along its short edges), beams are not required along the short edges. If the columns are short columns, do not require bracing and are not resisting any lateral loads, then also beams are not required along short edges.

6.2.2 Continuous Slab

Continuous one-way slabs have two or more spans (Figure 6.2). If the slabs have uniform spans and loads, then the spans would be subject to sagging moment (assigned positive sign throughout this book) at the span and hogging moment (assigned negative sign throughout this book) at the support. However, depending upon the spans and loading, hogging moment could occur at the span too.

6.2.3 Materials

The properties of concrete and steel, detailing of the embedments, and connections to other members are discussed in Sections 5.9, 5.10, 5.12, and 5.13 of this book, respectively.

6.2.4 Minimum Thickness of Slab

Though the thickness of the slab is selected at the beginning of the design, deflections are calculated after the flexural and shear reinforcement of the slab are designed. The concept of deflection is discussed in this chapter along with the selection of the thickness of the slab. The code specifies the minimum thickness of slab in table 7.3.1.1, unless the slab is supporting concentrated loads. The values provided are for steel grade f_y = 60,000 psi. If any other grade of steel is used, the values need to be multiplied by $(0.4 + f_y/100{,}000)$. For lightweight concrete (weight w_c), the value needs to be multiplied by the greater of (1) $1.65–0.005w_c$ and (2) 1.09.

Simply supported	L/20
One end continuous	L/24
Two ends continuous	L/28
Cantilever	L/10

In case the above requirements of table 7.3.1.1 of the code are not met, the immediate and time-dependent deflections shall be calculated based upon section 24.2 of the code.

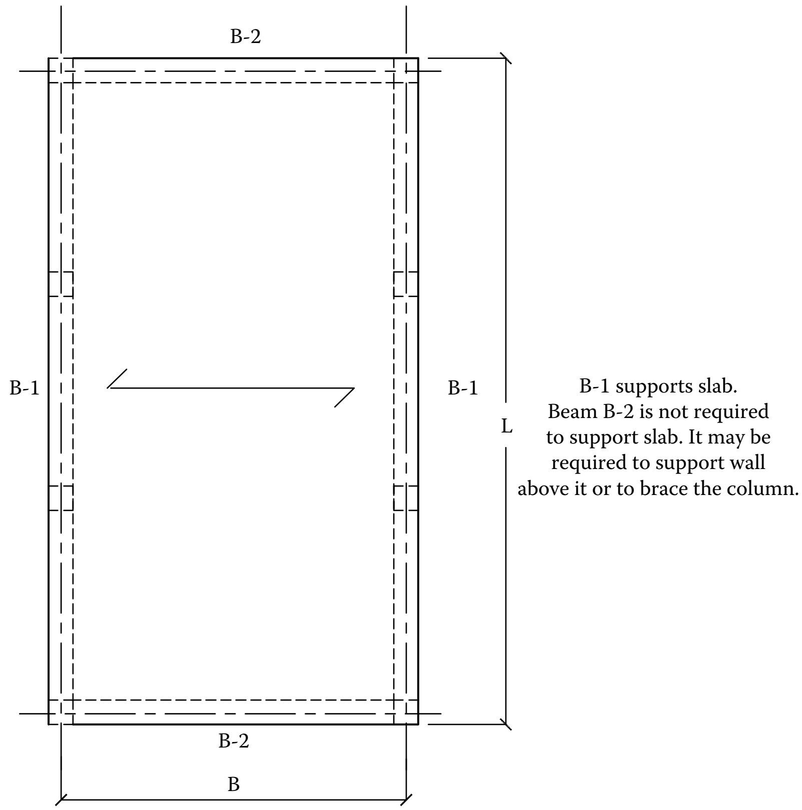


FIGURE 6.1 One-way slab (simply supported slab).

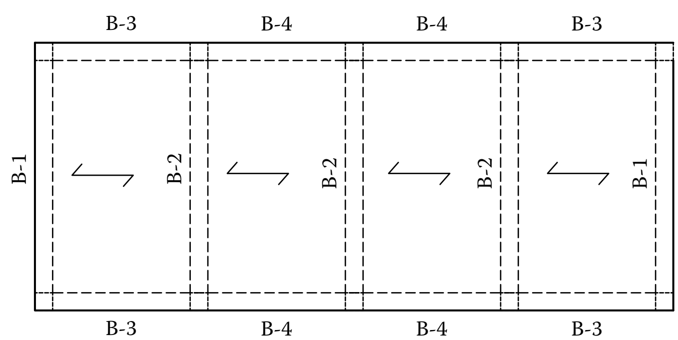


FIGURE 6.2 One-way continuous slab.

For immediate deflections, section 24.2.3 allows the use of gross sectional area of the member. The immediate deflection of a simply supported slab is $5wL^4/384E_cI_g$. I_g is the gross moment of inertia.

For time-dependent deflections, the following method specified by the code shall be used:

1. Calculate the modulus of elasticity of concrete.

$$E_c = w_c^{1.5} 33\sqrt{f_c'}\ (\text{in psi}) \quad \text{for } w_c \text{ between 90 and 160 pcf} \qquad \text{(ACI Equation 19.2.2.1.a)}$$

$$E_c = 57{,}000\sqrt{f_c'}\ (\text{in psi}) \quad \text{for } w_c \text{ for normal weight concrete} \qquad \text{(ACI Equation 19.2.2.1.b)}$$

where
E_c is the modulus of elasticity of concrete
w_c is the weight of concrete (pcf)
f_c' is the 28-day compressive strength of concrete (psi)

2. Calculate the modulus of rupture (psi).

$$f_r = 7.5\sqrt{f_c'} \qquad \text{(ACI Equation 19.2.3.1)}$$

3. Calculate the cracking moment (lb-in.).

$$M_{cr} = f_r \cdot I_g / y_t \qquad \text{(ACI Equation 24.2.35b)}$$

where y_t is the centroidal distance from the top surface of the slab, based on the gross section

4. Calculate modular ratio.

$$n = \frac{E_s}{E_c}$$

5. $B = \dfrac{b}{nA_s}$

where

b is the width of the section (in case of a slab, a 1 foot width is considered)

A_s is the area of steel in the width b

6. Calculate gross moment of inertia (I_g).

$$I_g = \frac{bh^3}{12}$$

7. Calculate kd.

$$kd = \frac{2\left(\sqrt{Bd+1}-1\right)}{B}$$

where d is the effective depth of the member.

8. Calculate moment of inertia of cracked section.

$$I_{cr} = b(kd)^3/3 + nA_s(d-kd)^2$$

9. Calculate the effective moment of inertia (I_e).

$$I_e = \left[\frac{M_{cr}}{M_a}\right]^3 I_g + \left[1 - \left[\frac{M_{cr}}{M_a}\right]^3\right] I_{cr} \qquad \text{(ACI Equation 24.2.3.5a)}$$

where M_a is the moment at service load. In a continuous slab, I_e can be taken as the average of the values of the critical positive and negative moment sections (section 24.2.3.6 of the code).

10. Deflection is calculated using the effective moment of inertia (I_e).

Additional time-dependent deflections due to creep and shrinkage are calculated by multiplying the immediate deflections by the factor λ_Δ.

$$\lambda_\Delta = \frac{\xi}{1+50\rho'} \qquad \text{(ACI Equation 24.2.4.1.1)}$$

where

$\rho' = A_s'/bd$

A_s' is compression steel

ξ is obtained from table 24.2.4.1.3 of the code for the sustained time

6.2.5 Strength Requirements

Section 7.4 of the code addresses the required strength of the one-way slabs. Use the load combinations discussed in Chapter 3 of this book. The live load can be applied to the level of the slab under consideration. Maximum positive moment near mid-span occurs with the live load on the span and the alternate span. Maximum negative moment at the support occurs with live load on adjacent spans only. The most demanding sets of design forces should be established by investigating the effects of live loads placed in various critical pattern. Section 6.5 of the code specifies a simplified method for determining design moments provided the following conditions are satisfied:

1. Members are prismatic.
2. Loads are uniformly distributed.
3. Live load/dead load ≤3.
4. There are at least two spans.
5. The longer of the two adjacent spans does not exceed the shorter by 20%.

Moment	Location	Condition	M_u
Positive	End span	Discontinuous end integral with support	$\frac{w_u l_n^2}{14}$
		Discontinuous end unrestrained	$\frac{w_u l_n^2}{11}$
	Interior span	All	$\frac{w_u l_n^2}{16}$
Negative	Interior face of exterior support	Member built integrally with supporting spandrel beam	$\frac{w_u l_n^2}{24}$
		Member built integrally with supporting column	$\frac{w_u l_n^2}{16}$
	Exterior face of first interior support	Two spans	$\frac{w_u l_n^2}{9}$
		More than two spans	$\frac{w_u l_n^2}{10}$
	Face of other supports	All	$\frac{w_u l_n^2}{11}$
	Face of all supports (Conditions a and b)	a. Slab with spans not exceeding 10 feet b. Beams where ratio of sum of column stiffness to beam stiffness exceeds 8 at each end of span	$\frac{w_u l_n^2}{12}$

In accordance with sections 7.4.2.1 and 7.4.3.1 of the code, for slabs built integrally with the support, factored moment (M_u) and factored shear (V_u) shall be calculated at the face of the support.

The approximate shear for continuous one-way slabs and beams shall be calculated in accordance with table 6.5.4 of the code.

For the exterior face of the first interior support, $V_u = 1.5w_u l_n/2$.

For face of all other supports, $V_u = w_u l_n/2$.

In accordance with section 7.4.3.2 of the code, if (a) support reaction of applied shear introduces compression into the end region of the slab, (b) loads are applied at or near the end region of the slab, and (c) no concentrated load occurs between the face of the support and critical section, then the shear is calculated at the section located at a distance 'd' from the face of the support.

6.2.6 Design Strength

The slab shall be designed such that the design strength $ØM_n \geq M_u$ and $ØV_n \geq V_u$. The value of the strength deduction factor (Ø) is discussed in Section 5.14 of this book. The method to calculate M_n for a rectangular section is discussed in Section 4.2 of this book. Design for shear is dealt with in Section 4.4 of this book.

6.2.7 Reinforcement

Concrete cover to reinforcement (Section 5.10), development length of reinforcement (Section 11.4), splice length (Section 11.5), and minimum spacing (Section 11.2) have been discussed in this book.

The main reinforcement in a one-way slab resists flexure (also called flexural reinforcement), and is placed along the short span. The temperature and shrinkage reinforcement is required in the direction perpendicular to the main reinforcement. They are used to minimize cracking and to tie the structure together to ensure that it is acting as assumed in the design.

The minimum area of main reinforcement in a one-slab is specified in table 7.6.1.1 of the code. As mentioned before, in this book we are using grade 60 steel and the minimum area of reinforcement shall be greater of

1. $\dfrac{0.0018 \times 60{,}000}{f_y} A_g$, where Ag is the gross cross-sectional area of concrete
2. $0.0014\ A_g$

The minimum area of temperature and shrinkage reinforcement in a one-slab is specified in table 24.4.3.2 of the code. For the grade 60 steel, the minimum area of reinforcement shall be greater of

1. $\dfrac{0.0018 \times 60{,}000}{f_y} A_g$, where Ag is the gross cross-sectional area of concrete
2. $0.0014\ A_g$

The following specifications of the code shall be followed during the detailing of the reinforcement:

1. The spacing of the shrinkage and temperature reinforcement shall not exceed five times the thickness of the slab or 18 in^2 (section 24.3.3.3 of the code).
2. Cover to the reinforcement is discussed in Section 5.10 of this book.
3. Development length of the reinforcement is discussed in Section 11.4 of this book.
4. The minimum spacing of the reinforcing bars shall be the greater than 1 in. or the diameter of the bar (d_b) or 1.5 times the aggregate size ($1.5d_{agg}$) (section 25.2.1 of the code).
5. The maximum spacing between the reinforcing bars shall be the lesser of three times the thickness of the slab (3h) and 18 in^2 (section 7.7.2.3 of the code).
6. At simple supports, at least one-third of the maximum positive reinforcement shall be extended along the slab bottom into the support, and at other supports, at least one-fourth of the maximum positive reinforcement shall be extended along the slab bottom into the support at least 6 in^2 (sections 7.7.3.8.1 and 7.7.3.8.2 of the code).

Section 7.7.3 of the code further addresses the reinforcement detailing. The reinforcement of slabs and beams are required to be developed for the calculated forces. The locations for the development of reinforcement are determined at maximum stress points where the reinforcement is no longer required.

Other than the simple supports or free ends of cantilevers, reinforcement are required to be developed for a distance equal to the greater of the effective depth (d) of the slab and 12 times the diameter of the bar (d_b). Continuing flexural tension reinforcement shall have an embedment length of at least l_d beyond the point where the bent or terminated bar is not required to resist flexure, where l_d is the development length discussed in Section 11.4 of this book.

The flexural tension reinforcement shall not terminate in the tension zone unless one of the following three conditions are satisfied:

a. $V_u \le (2/3)ØV_n$ at the cutoff point.
b. For #11 and smaller bars, continuing reinforcement provides double the area required for flexure at the cutoff point and $V_u \le (3/4)ØV_n$.
c. Stirrup area in excess of that required for shear is provided along each terminated bar over a distance 3/4d from the termination point. Each stirrup area shall be not less than $60b_w s/f_{yt}$. Spacing (s) shall not exceed $d/8\beta_b$, where β_b is the ratio of area of reinforcement cutoff to total area of tension reinforcement at section.

At simple supports, at least one-third of maximum positive moment reinforcement shall extend along the slab bottom into the support, and at other supports, at least one-fourth of maximum positive reinforcement shall extend along the slab bottom for at least 6 in^2.

At least one-third of the negative reinforcement at the support shall have an embedment length beyond the point of inflection of at least the greatest of d, $12d_b$, and $l_n/16$. l_n is the clear span measured face-to-face of the support.

Point of inflection or contraflexure is the point where the bending changes sign. A positive moment (sagging) could change to a negative moment (hogging) and vice versa. In this book, the sagging moment is taken as positive moment and the hogging moment is taken as negative. At simple support or point of inflection, d_b for positive moment tension reinforcement shall be limited such that l_d for that reinforcement is less than or equal to $(1.3M_n/V_n + l_a)$ if the end of the reinforcement is confined by a compressive reaction, or it is less than or equal to $(M_n/V_n + l_a)$ if the end of the reinforcement is not confined by a compressive reaction. At the support, l_a is the embedment depth beyond the center of the support. l_a is the embedment depth beyond the point of inflection limited to the greater of d and $12d_b$. M_n is calculated assuming all reinforcement at the section is stressed to f_y and V_u is calculated at the section.

The main reinforcement of the slabs shall be distributed to prevent flexural cracking, and the spacing of the bar shall not exceed $15\left(\frac{40,000}{f_s}\right) - 2.5c_c$ in accordance with table 24.3.2 of the code. The stress in the reinforcing bars (f_s) is calculated based upon service load moments (unfactored) and can also be taken as $(2/3)f_y$. c_c is the least distance from the surface of the reinforcing bar to the tension face of the slab.

Temperature and shrinkage reinforcement is provided perpendicular to the main reinforcement of a one-way slab to minimize cracking in the slab. Discussion of the shrinkage is provided in Section 5.5.3 of this book. In accordance with table 24.4.3.2 of the code, for grade 60 steel, a temperature reinforcement of a cross-sectional area of 0.0018bh shall be provided, where 'h' is the thickness of the slab and 'b' is width under consideration. Usually, the temperature reinforcement area is calculated for a foot and hence 'b' is taken as 12 in^2. The spacing of the temperature reinforcement shall be lesser of '5h' and 18 in^2.

6.3 TWO-WAY SLABS

Two-way slabs are slabs whose aspect ratio (l/b) is than 2. The two-way slabs can be supported on beams or directly on the columns (Figure 6.3). When the two-way slabs are directly supported on the columns, they can be flat plates or flat slabs. Flat plates are slabs with uniform thickness (Figure 6.5). Flat slabs are thickened around the supporting columns (Figure 6.6). The thickened portion of the slabs are called column capitals. Unlike the one-way slabs, two-way slabs disperse loads to all their four edges. As demonstrated in Figure 6.4, the long edge support of the two-way slabs is subject to trapezoidal load and the short edge support is subject to triangular load.

6.3.1 Strength Requirements

The structural engineer begins designing the two-ways by determining the required strength of the slab based upon the spans and the loads. The load combinations in chapter 5 of the code and chapter 2 of the ASCE 7-16 are discussed in Chapter 3 of

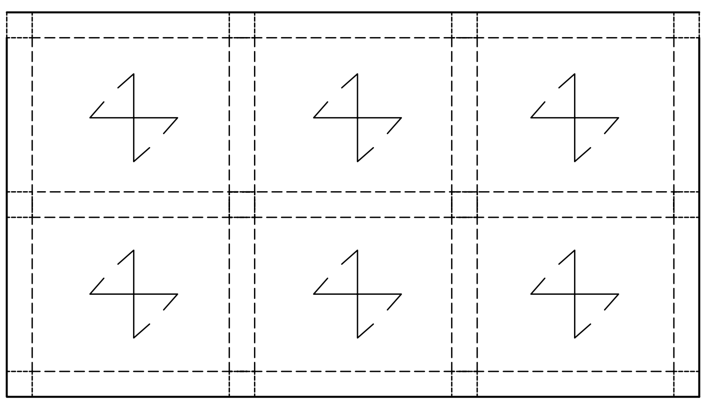


FIGURE 6.3 Two-way slab (supported by beams).

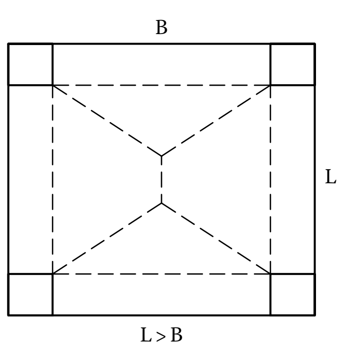


FIGURE 6.4 Load dispersal two-way slab.

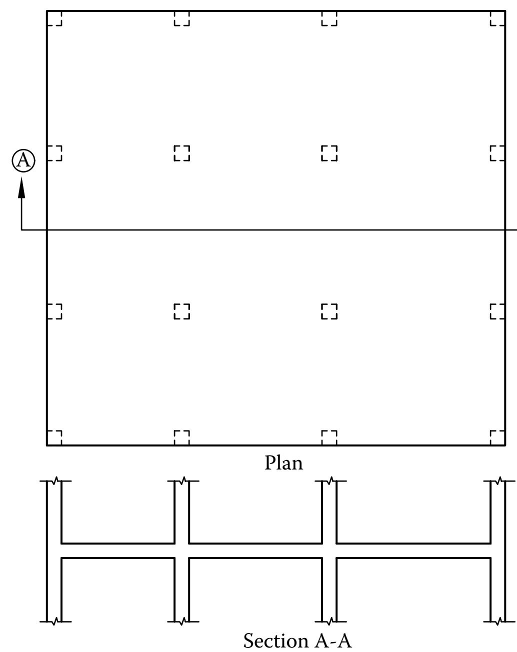


FIGURE 6.5 Flat plates.

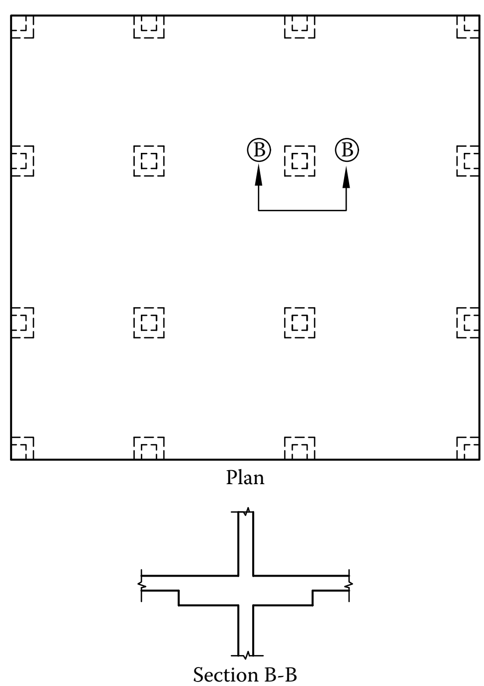


FIGURE 6.6 Flat slab.

this book. The procedures to calculate the required strength for the two-way slabs, as specified in chapter 8 of the code, are as follows:

a. Finite element method (FEM)
b. Direct design method (DDM)
c. Equivalent frame method (EFM)

While FEM is beyond the scope of this book, DDM and EFM are discussed later in this section.

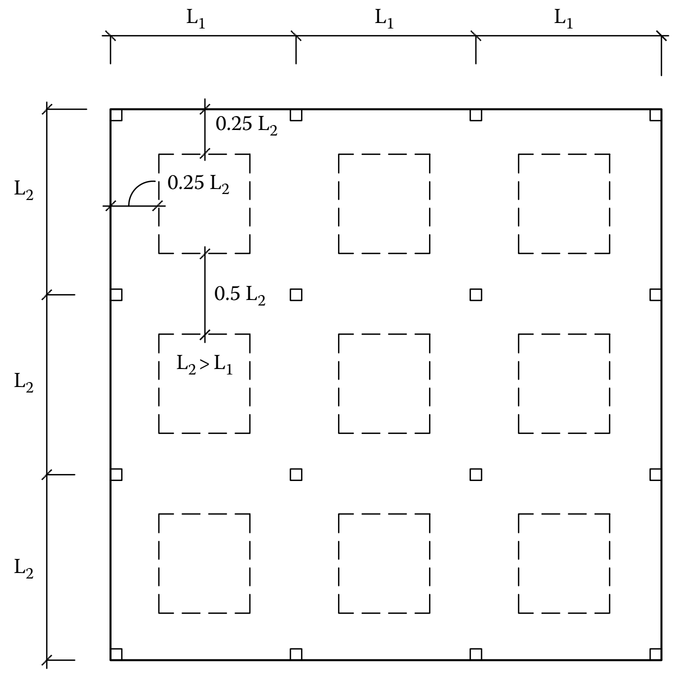


FIGURE 6.7 Two-way slab—strips.

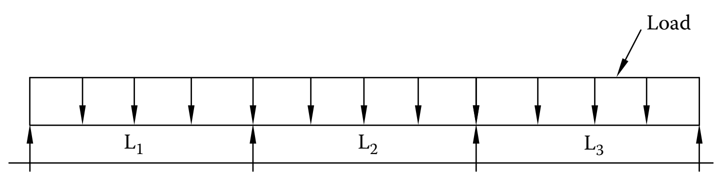


FIGURE 6.8 Two-way slabs (LL < 75% DL).

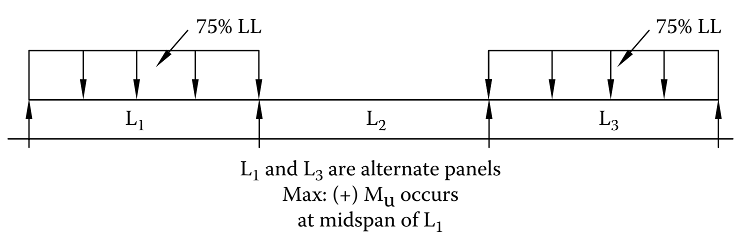


FIGURE 6.9 Two-way slabs.

If the live load arrangement on the slab is known, the slab system is analyzed for that arrangement. If the live load is less than 75% of the dead load, all spans of the slab are loaded with the live load for the analysis. Otherwise, the live load is arranged such that maximum positive moment (M_u) occurs at midspan of the panel with 75% factored live load on the panel and alternate panel and maximum negative moment (M_u) occurs at the support with 75% factored live load on adjacent panels only (Figures 6.8 through 6.10).

6.3.1.1 Slab Strips

Two-way flat slabs and plates are divided into column strips and middle strips with a width on each side of the column equal to 25% of the longer span of the slab. A middle strip is bounded by two column strips. A panel is bounded by column, beam, or wall centerlines on all four sides (Figure 6.7).

When the concrete of the beams are cast monolithically with the concrete of the slab, then the edge beams are designed as L beams and the interior beams are designed as T beams, as shown in Figure 6.11.

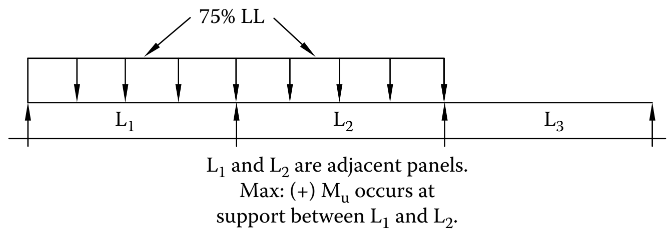


FIGURE 6.10 Two-way slabs.

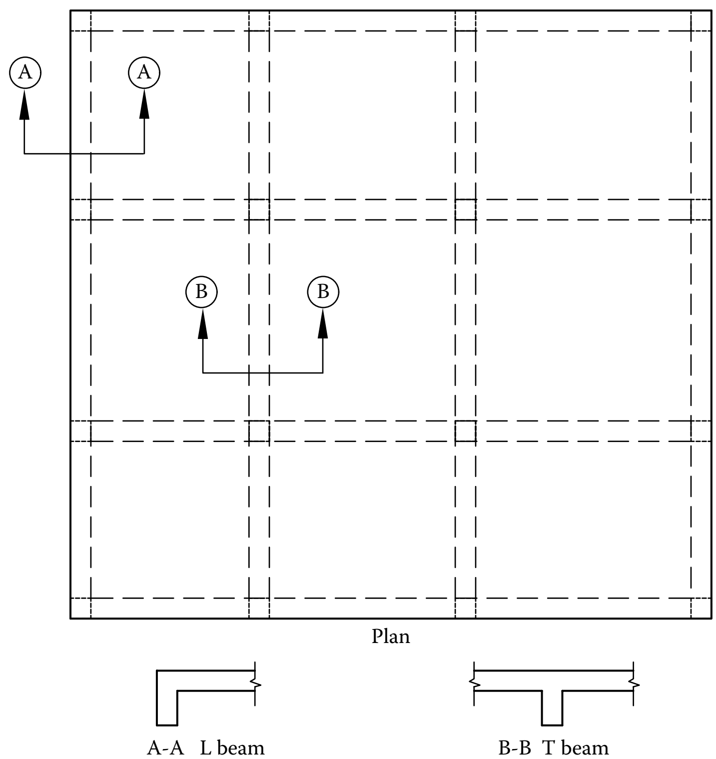


FIGURE 6.11 Two-way slab with beams.

6.3.1.2 Slab Moment Resisted by Column

A portion of the slab moment is transferred to the column ($\Upsilon_f M_{sc}$) by flexure, where M_{sc} is the slab moment that is resisted by column and Υ_f is the portion of the slab moment resisted by the column, transferred by slab flexure.

$$\Upsilon_f = \frac{1}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \qquad \text{(ACI Equation 8.4.2.3.2)}$$

where b_1 is the dimension of the critical section (b_0) measured in the direction of the span for which moments are being determined (in.) and b_2 is the dimension of the critical section (b_0) measured in the direction perpendicular to b_1 (in.) (Figure 6.12).

The effective slab width (b_{slab}) for resisting $\Upsilon_f M_{sc}$ shall be width of the column or capital plus 1.5 times the thickness of the slab or drop panel on either side of column or capital. The reinforcement of the slab shall be designed for this effective slab width for M_{sc}.

The factor (γ_f) can be modified in accordance with table 8.4.2.3.4 of the code based upon the location of the column, value of factored shear stress on the slab critical section, and the strain in steel (ε_t) that shall be at least 0.004.

One-way shear: For flat slabs or flat plates built integrally with supports, shear (V_u) is calculated at the face of the support. Sections between the face of the support and at a critical section at a distance equal to the effective depth of the slab (d) shall be designed for shear (V_u) calculated at the critical section provided: support reaction in the direction of the applied shear produces

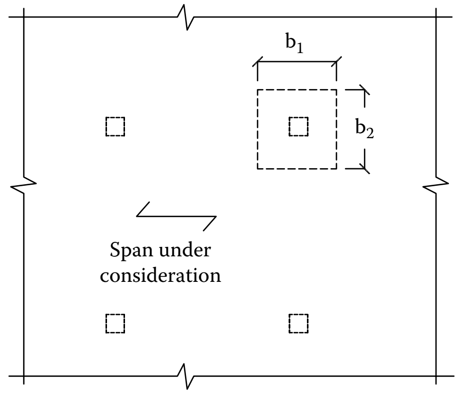


FIGURE 6.12 Slab moment resisted by column.

compression into the end regions of the slab, loads are applied at or near the top surface of the slab, and there is no concentrated load between the face of the support and at the critical section.

Two-way shear: The critical section for two-way shear is discussed in Section 4.4.3 of this book. The column shall resist a factored shear of $v_{ug} + \Upsilon_v M_{sc}$, where v_{ug} is the factored shear stress of the slab critical section for two-way action due to gravity loads without moment transfer (psi) and $\Upsilon_v = 1 - \Upsilon_f$. This fraction of the slab moment ($\Upsilon_v M_{sc}$) is resisted by the column by the eccentricity of shear, and the shear is assumed to be varying linearly about the centroid of the critical section.

6.3.2 Design Strength

Two-way slabs are designed for the thickness and grade of the concrete and amount of reinforcing bars and their locations such that their design strength multiplied by a strength reduction (Φ) shall be greater than or equal to the required strength discussed above. Strength reduction (Φ) is discussed in Section 5.14 of this book. Design strength is discussed in section 8.5 of the code.

a. $\Phi M_n \geq M_u$ at all sections along the span in each direction.
b. $\Phi M_n \geq M_{sc}$ within b_{slab}, which is defined above.
c. $\Phi V_n \geq V_u$ at all sections along the span in each direction for one-way shear.
d. $\Phi v_n \geq v_u$ at critical sections for two-way shear.

Calculations for the nominal strength of the slab bending are discussed in Section 4.2 of this book. If drop panels are provided (flat slabs), the thickness of the drop panel below the slab shall not be assumed to be greater than one-fourth the distance between the edge of the drop panel to the face of the column or the column capital.

Calculations for the nominal shear strength of the slabs are discussed in Section 4.4.3 of this book. If openings are provided in slabs, then the strength and serviceability requirements of the code must be satisfied. Section 8.5.4.2 of the code provides the specifications for the openings in two-way slabs.

a. Openings of any size are allowed in the area common to the two intersecting middle strips of the slab but the total area of the reinforcement for the strips (without openings) shall be provided. This reinforcement is provided in the strip where there is no opening.
b. Openings in two intersecting column strips cannot exceed one-eighth the width of either of the two column strips. Reinforcement at least equal to the amount lost due to the opening must be provided on the sides of the openings.
c. Openings in intersecting column and middle strips cannot exceed one-fourth the width of either of the strips. Reinforcement at least equal to the amount lost due to the opening must be provided on the sides of the openings.
d. For an opening in the column strip or close to a concentrated load, refer to the discussion in Section 4.4.3 of this book.

6.3.3 Minimum Reinforcement

The minimum reinforcement for a two-way slab for a steel grade f_y = 60,000 psi, in accordance with table 8.6.1.1 of the code, shall be greater of

a. $\dfrac{0.0018 \times 60{,}000}{f_y} A_g$
b. $0.0014\, A_g$

where A_g is the gross cross-sectional area of the slab (in^2).

6.3.4 Reinforcement Detailing

Concrete cover to reinforcement (Section 5.10 of this book), development length of reinforcement (Section 11.4 of this book), splice length (Section 11.5 of this book), and minimum spacing (Section 11.2 of this book) have been discussed. The maximum spacing of reinforcement of two-way slabs shall be lesser of twice the thickness of the slab and 18 in^2 at critical sections and lesser of thrice the thickness of the slab and 18 in^2 at other sections. Corners in two-way slab tend to lift and hence need to be restrained. Reinforcing bars are provided at the corners of the slab for a length equal to the fifth of the larger span. These bars are designed for a moment equal to the maximum positive moment (M_u) per unit width in the slab panel. Either a diagonal or an orthogonal pattern for placing the reinforcement can be selected.

6.3.5 Shear Reinforcement

Shear design in form of stirrups, shear heads, or studs is discussed in Section 4.4.3 of this book.

6.3.6 Direct Design Method (DDM)

This method consists of determining total factored static moment and distributing it to negative and positive sections. Then the negative and positive moments are distributed to the middle and column strips of the slab. The limitations in the use of DDM are as follows:

a. There shall be at least three continuous spans in each direction.
b. Successive span lengths (center to center between supports) shall not differ by more than one-third of the longer span. For example, if four successive spans in one direction are 25, 16, 18, and 16 feet, then $\left(\frac{25'-16'}{25'}\right)=\left(\frac{9'}{25'}\right)>\frac{1}{3}$. Hence, DDM cannot be used.
c. DDM is applicable only to rectangular panels.
d. The long/short span ratio for each panel shall be less than 2.0.
e. Column offsets shall not exceed 10% of the span in the direction of the offset.
f. All loads shall be gravity loads and uniformly distributed over an entire panel.
g. The unfactored live load shall not exceed twice the unfactored dead load.
h. If there are panels with beams on all four sides, then

$$0.2 \le \frac{\alpha_{f1} l_2^2}{\alpha_{f2} l_1^2} \le 5.0 \qquad \text{(ACI Equation 8.10.2.7a)}$$

$$\text{where} \quad \alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s} \qquad \text{(ACI Equation 8.10.2.7b)}$$

The DDM methodology is as follows:

1. Calculate the factored static moment in each direction.

$$M_0 = \frac{q_u l_2 l_n^2}{8} \qquad \text{(ACI Equation 8.10.3.2)}$$

   where l_n is the clear span and is greater than 0.65 times the length of the longer span (l_1) and (l_2) is the width of the panel (the center-to-center distance between two adjacent panels).
   If two adjacent traverse spans (l_2) have different lengths, then an average of the two is taken.
   For the exterior spans, l_2 shall be the distance from the slab edge to the panel centerline.
2. If the cross section of the columns is not square or rectangle, then a square section with an equivalent area of the given section is considered to calculate the dimensions of the square column.
3. The static moment (M_0) for the interior spans are distributed—65% negative moment and 35% positive moment. For the distribution of moments of end spans, refer to table 8.10.4.2 of the code. A modification up to 10% is permitted for the positive and negative moments, which means the negative moment can range between $0.55M_0$ and $0.75M_0$ and the positive moment can range between $0.25M_0$ and $0.45M_0$. Critical section for the negative moment shall be at the face of the support. Unless moment distribution is performed over the support, for column strip, the design negative

moment shall be the larger of the two negative moments of the two adjacent spans. Edge beams or edges of the slab shall be designed for the torsional stresses caused by the slab negative moments.

4. The factored negative moment is distributed at the interior span to the column strips in accordance with table 8.10.5.1 of the code. The factored negative moment is distributed at the exterior span to the column strips in accordance with table 8.10.5.2 of the code based upon the torsional stiffness parameter (β_t):

$$\beta_t = \frac{E_{cb} C}{2 E_{cs} I_s} \quad \text{(ACI Equation 8.10.5.2a)}$$

where C is the cross-sectional constant to define torsional properties of the slab and the beam. For T and L sections, the sections can be divided into parts to calculate C:

$$C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3} \quad \text{(ACI Equation 8.10.3.2b)}$$

where x is the shorter overall dimension of the rectangular part of the cross section (in^2) and y is the longer overall dimension of the rectangular part of the cross section (in^2).

If the slab is supported on a wall, then $\propto_{f1} l_2/l_1$ is taken as greater than 1.0.

The positive factored moment is distributed to the column strip in accordance with table 8.10.5.5 of the code.

If slabs are supported by beams, the factored moment of the column strip calculated above is distributed to the beams in accordance with table 8.10.5.7.1 of the code. Additionally, the beams shall resist any load applied directly on them such as the CMU walls.

5. The portions of the moment not assigned to the column strip are resisted by the middle strip. A middle strip adjacent and parallel to a wall-supported edge shall resist twice the moment assigned to half the middle strip corresponding to the first row interior panel.
6. An unbalanced moment between two adjacent slab panels is transferred to the column or wall supporting the panels directly. The calculated moment (M_{sc}) is distributed to the column or wall below and above the slab according to their stiffness.

$$M_{sc} = 0.07\left[\left(q_{DU} + 0.5 q_{LU}\right) l_c l_n^2 - q_{DU}' l_2' \left(l_n'\right)^2\right] \quad \text{(ACI Equation 8.10.7.2)}$$

M_{sc} is not less than 0.3 times M_0

where q_{DU} (factored dead load), q_{LU} (factored live load), l_2, and l_n are for longer of the two adjacent spans and q_{DU}' (factored dead load), l_2', and l_n' are for shorter of the two adjacent spans

7. In two-way slabs, shear is distributed to the adjacent beams using trapezoidal load path for long beams and triangular load paths for short beams supporting the rectangular two-way panels, as shown in Figure 6.4. If $\propto_{f1} l_2/l_1$ is greater than or equal to 1.0, then the complete load is transferred to the beam. However, if $\propto_{f1} l_2/l_1$ is less than 1.0, then the values of the distribution coefficient provided in table 8.10.8.1 of the code are less than 1.0, using linear interpolation of the values. These beams will not account for the total shear force of the slab. The remaining shear force will produce shear stresses in the slab around the columns, and the slab needs to be adequate to resist these stresses.

6.3.7 Equivalent Frame Method (EFM)

Equivalent frame method (EFM) is used when lateral loads are also applied to a floor system. The method can also address cases for which the DDM is not applicable, such as irregular slab layout, slab where partial loading patterns are significant, and for slabs with high live/dead load ratios. EFM involves breaking a 3-dimensional slab system into parallel 2-dimensional plane frames in the two orthogonal directions. These frames are analyzed for the gravity and lateral loads acting on them. Each frame consists of the slab (with beams, if any), columns/walls (above and below the slab), and any structural elements that provide moment transfer between the slabs and the columns/walls. The width of the slab in an interior frame shall be the sum of half the widths of the adjacent panels. The width of the slab in an exterior frame shall be the distance between the edge of the slab and the centerline of the end panel.

The frames can be analyzed for the entire building with all floor levels included or for each floor with columns/walls above and below the floor, treated as fixed at their far ends.

6.4 ASSIGNMENTS

1. The span of a one-way slab with normal concrete is 16 feet. What is the minimum thickness required by table 7.3.1.1 of the code?
2. What is the minimum thickness of a 12 feet span cantilever slab with normal concrete required by table 7.3.1.1 of the code?

3. Calculate the modulus of elasticity and modulus of rupture for 3,500 psi grade concrete. If the yield strength of the steel f_y = 60,000 psi, then what is the modular ratio?
4. If the service moment of a slab section (8 in. thick) is 5120 lb-feet/feet, the slab is reinforced with #5 bars at 10 in. o.c., the grade of concrete is 5,000 psi, and the grade of steel is 60,000 psi, then what is the effective moment of inertia of a 1 foot wide section?
5. Design and detail the one-way slab shown in Figure 6.13 using the simplified method of section 6.5 of the code (superimposed dead load—20 psf; live load—40 psf; grade of concrete—4,000 psi; grade of steel—60,000 psi).
6. Design and detail the one-way slab shown in Figure 6.14, using moment distribution method to analyze the slab (superimposed dead load—25 psf; live load—50 psf; grade of concrete—5,000 psi; grade of steel—60,000 psi).

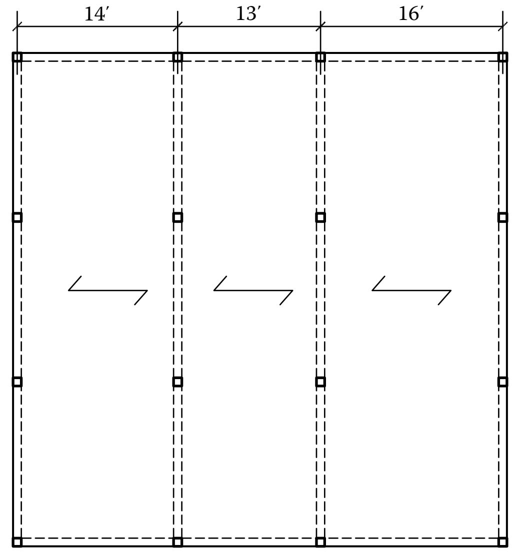


FIGURE 6.13 One-way slab.

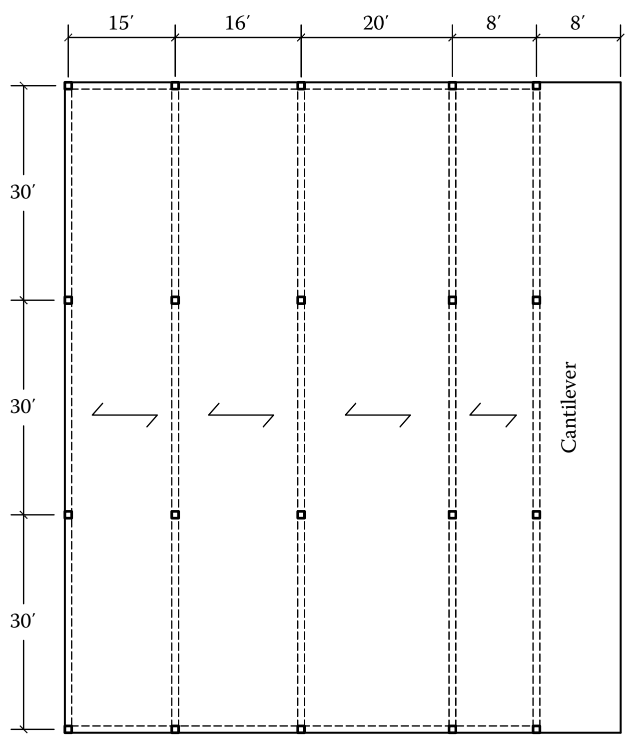


FIGURE 6.14 One-way slab (all beams 8 in^2 × 24 in^2).

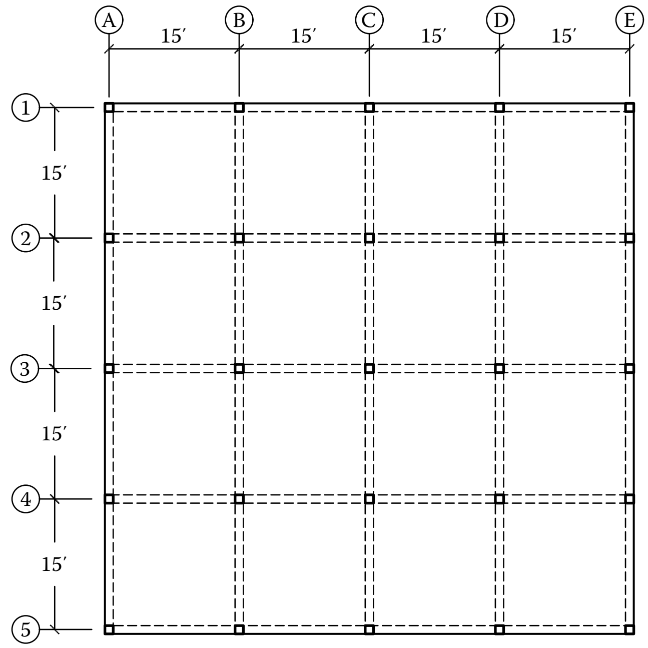


FIGURE 6.15 Two-way slabs with beams (all beams and columns are 12 in^2 × 12 in^2).

7. Design and detail the two-way slab with beams shown in Figure 6.15, using direct design method (superimposed dead load—20 psf; live load—40 psf; grade of concrete—4,000 psi; grade of steel—60,000 psi).
8. Design and detail the two-way flat plate slab shown in Figure 6.16, using direct design method (superimposed dead load—20 psf; live load—40 psf; grade of concrete—4,000 psi; grade of steel—60,000 psi).
9. Design and detail the two-way flat slab shown in Figure 6.17, using direct design method (superimposed dead load—20 psf; live load—40 psf; grade of concrete—4,000 psi; grade of steel—60,000 psi).

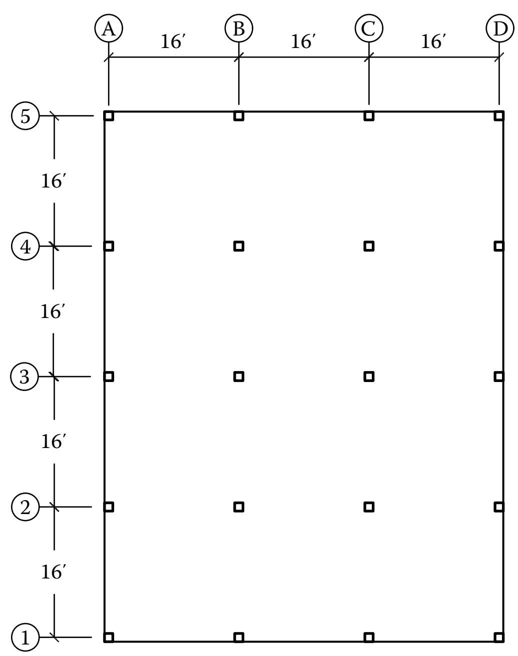


FIGURE 6.16 Two-way flat plate (all columns are 12 in^2 × 12 in^2).

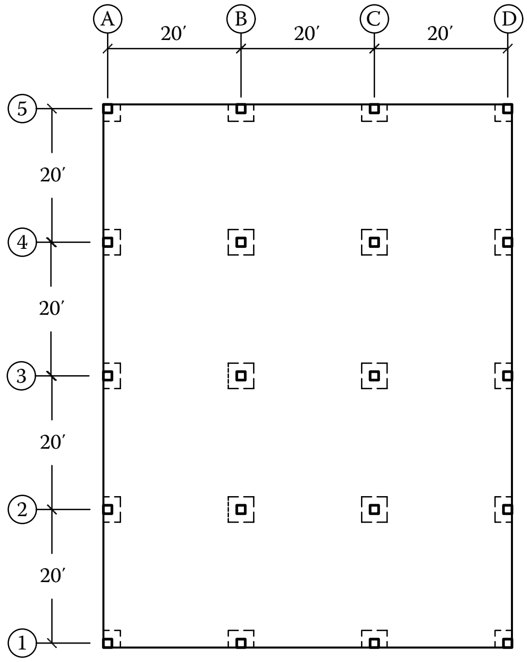


FIGURE 6.17 Two-way flat slab.

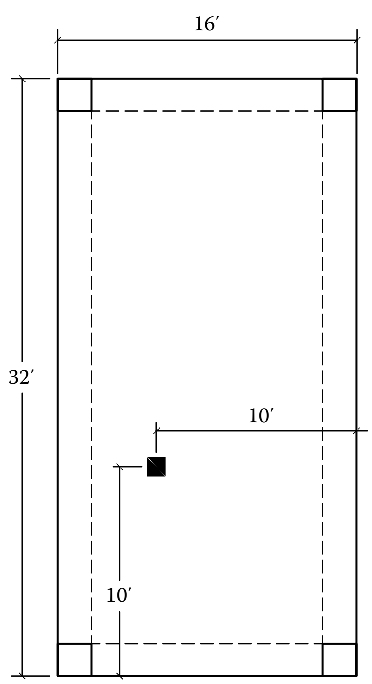


FIGURE 6.18 One-way slab with concentrated load.

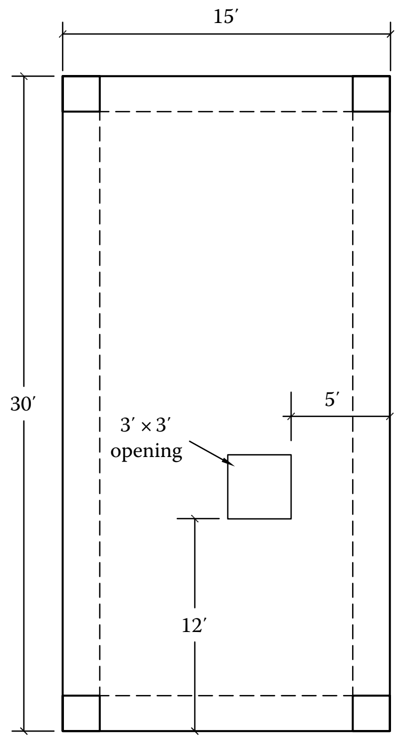


FIGURE 6.19 One-way slab with opening.

10. Design and detail the two-way slab with beams shown in Figure 6.15, using equivalent frame method (superimposed dead load—20 psf; live load—40 psf; grade of concrete—4,000 psi; grade of steel—60,000 psi).
11. Design and detail the one-way simply supported slab shown in Figure 6.18, supporting a column with a dead load of 200,000 lbs and a live load of 150,000 lbs (superimposed dead load—25 psf; live load—30 psf; grade of steel—60,000 psi). Select grade of concrete.
12. Design and detail the one-way simply supported slab with an opening of 3 feet × 3 feet as shown in Figure 6.19 (superimposed dead load—25 psf; live load—30 psf; grade of concrete—5,000 psi; grade of steel—60,000 psi).

7 Beams

7.1 INTRODUCTION

Beams are horizontal members that span columns. In the case of a cantilever, one end of the beam is supported and the other end is free. Beams support the loads from slabs, other beams, walls, and columns. They transfer the loads to the columns supporting them. Beams supporting columns are called "transfer beams." The columns terminate on top of the beams because they might be hindering free space in the floor below. Beams can be simply supported, continuous, or cantilevered. Beams can be designed as rectangular, square, T-shaped, and L-shaped sections. Beams can be singly reinforced or doubly reinforced. Doubly reinforced beams are used if the depth of the beam is restricted.

Typically, loads acting on the beams can be

- Rectangular loads from one-way and cantilever slabs (Figure 7.1)
- Trapezoidal or triangular loads from two-way slabs (Figure 7.2)
- Irregularly shaped load from cantilever and two-way slabs (Figure 7.3)
- Self-weight of the beam
- Weight of walls supported on the beams
- Loads from columns supported on beams

Load from slabs supported on beams are converted into a uniform load along the length of the beam using the tributary width of the slab if the loads are rectangular or square shaped. If loads are trapezoidal or triangular in shape, then formulas of basic strength of materials are used to calculate the moment and shear. If the loads are irregular, then either a computer program or approximations are used to distribute them as uniform linear loads on the beams. The self-weight of the beam and wall loads are uniform linear loads, and loads of columns are concentrated or point loads.

Chapter 9 of the code provides specifications for beams. Beams shall be laterally braced. The spacing of the lateral bracing shall not exceed 50 times the least width of the compression flange or face. Slabs can be considered as laterally bracing the beams.

7.2 T- AND L-SHAPED BEAMS

Beams are designed as T-shaped or L-shaped sections because they provide additional compression area to the beam other than the top portion of a rectangular or square section above the neutral axis. When designed as T-shaped or L-shaped, the concrete of the beams must be cast monolithically with the concrete of the slab. The effective flange width of the T-shaped or L-shaped sections, in accordance with table 6.3.2.1 of the code, shall be as follows:

T-shaped: width of the web plus the least of 8h; $s_w/2$ and $l_n/8$ (Figure 7.4)
L-shaped: width of the web plus the least of 6h; $s_w/2$ and $l_n/12$ (Figure 7.4)

where h is the slab thickness; s_w is the clear distance to the adjacent web; l_n is the clear span of the beam.

In the case of an isolated T section (without a continuous slab panel), the flange thickness shall be greater than or equal to half the width of the web and the effective flange width shall be less than or equal to four times the width of the web.

In cases of T beams in one-way slabs, where primary flexural reinforcement is parallel to the axis of the beam, slab reinforcement perpendicular to the longitudinal axis of the beam must be designed assuming that the flanges (calculated in accordance with table 6.3.2.1 of the code) are cantilevers.

The concept of torsion is discussed in Section 1.15 of this book. The three main geometric parameters discussed in torsion design are:

A_{cp}—area enclosed by the outside perimeter of the concrete cross section
A_g—gross area of the concrete section. For a hollow section, the area of the voids is not considered in A_g
P_{cp}—outside perimeter of the concrete cross section

During the calculations of A_{cp}, A_g, and P_{cp} for a T- or L-shaped beam, the overhang on each side of the web shall be taken more than four times the thickness of the flange (or the slab thickness). If A_{cp}^2/P_{cp} for solid section or A_g^2/P_{cp} for hollow section calculated for T- or L-section is less than that calculated without the flanges, then the flanges shall be ignored.

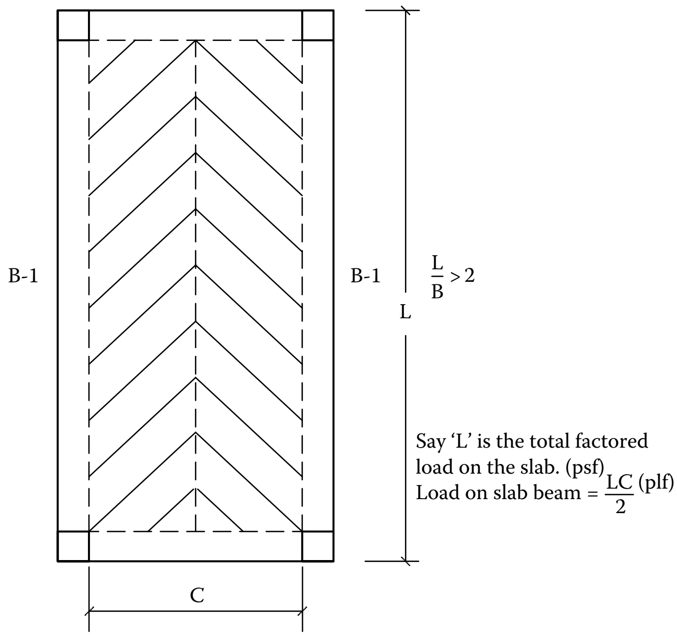


FIGURE 7.1 Loads on beams (one way slabs).

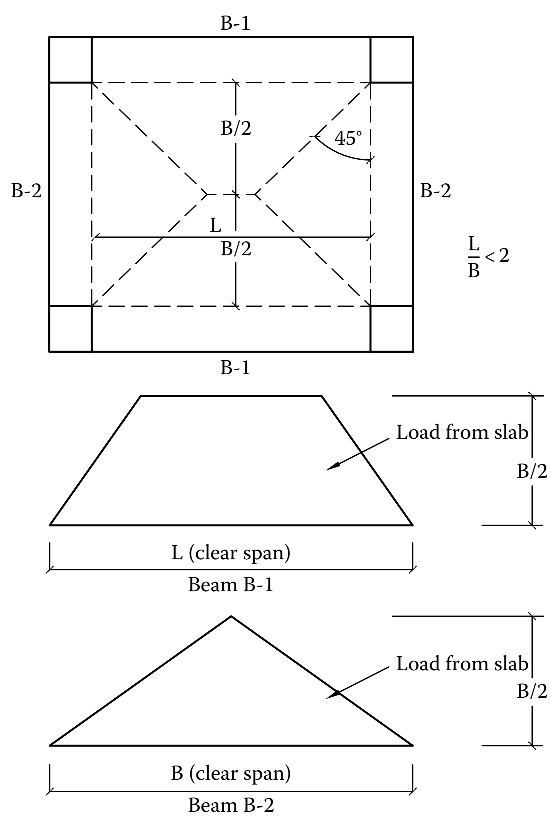


FIGURE 7.2 Loads on beams (two-way slabs).

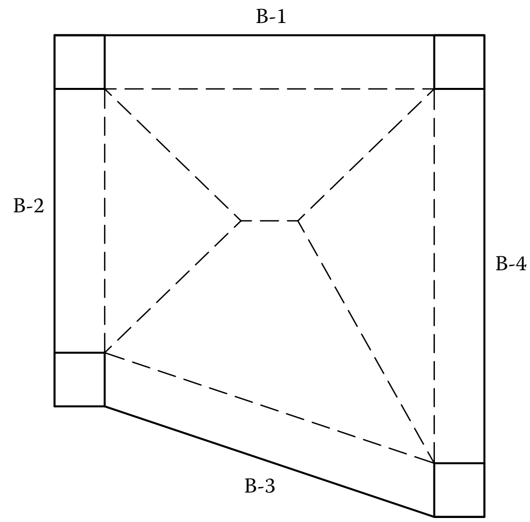


FIGURE 7.3 Loads on beams (irregular slabs).

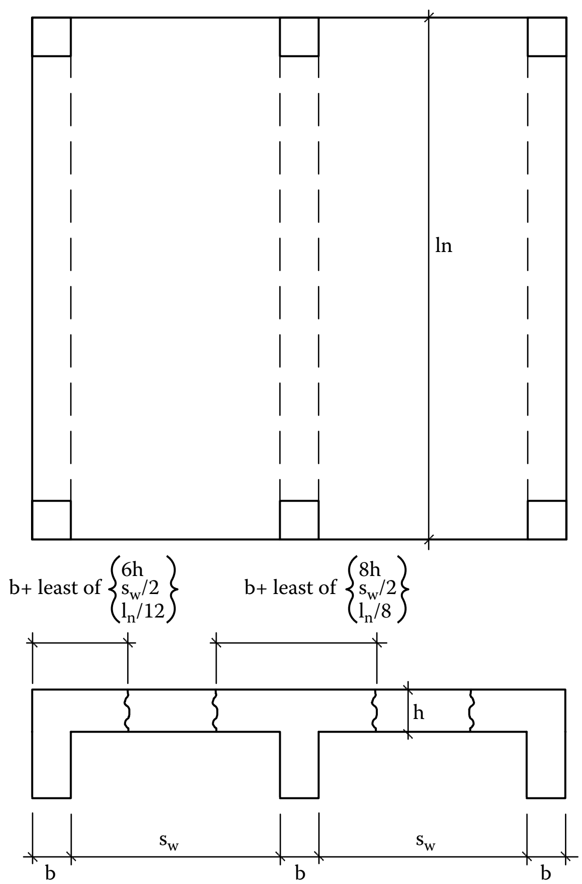


FIGURE 7.4 T and L beams.

7.3 MINIMUM BEAM DEPTH

The minimum beam overall depth for normal-weight concrete with reinforcing steel (f_y = 60,000 psi) is specified in table 9.3.1.1 of the code, unless immediate and time-dependent deflections are calculated, as explained in Section 5.5.1 of this book.

Simply supported	l/16
One end continuous	l/18.5
Both ends continuous	l/21
Cantilever	l/8

Where l is the span of the beam (in^2).

7.4 STRENGTH REQUIREMENTS

Section 9.4 of the code addresses the required strength of beams. Use the load combinations discussed in Chapter 3 of this book. After transferring the load on the beams—a uniform load, uniformly varying load, or concentrated load—moments are calculated on critical locations to obtain the maximum positive and negative moment on the beam. As mentioned above, for irregular loads, either an approximation of loads is made or a computer program is used to calculate the moments. For beams built integral with the support, the factor moment (M_u) and factored shear (V_u) at the support are calculated at the face of the support.

Sections located between the face and a critical section located at a distance “d” from the face of the support are permitted to be designed for a shear (V_u) at the critical section if the following conditions are met:

- Support reaction applied in the direction of applied shear introduces compression into end region of the beam.
- Loads are applied at or near the top surface of the beam.
- There is no concentrated load between the face of the beam and the critical section.

Beams shall be designed for a uniform torsion from the slab. For beams built integral with the support, the factor torsion (T_u) is calculated at the face of the support. Sections located between the face and a critical section located at a distance ‘d’ from the face of the support are permitted to be designed for torsion (T_u) at the critical section.

7.5 DESIGN STRENGTH

The slab shall be designed such that the design strength $ØM_n \geq M_u$, $ØV_n \geq V_u$, $ØT_n \geq T_u$, and $ØP_n \geq P_u$. The value of the strength deduction factor (Ø) is discussed in Section 5.14 of this book. The design strength for moment (section 4.2); shear (section 4.4); torsion (section 4.5) and axial load (section 4.3) have been discussed in the book.

If P_u is greater than or equal to $0.10f_c'A_g$, then the section is dealt for combined axial and flexural strength as discussed in section 4.3 of this book. If T_u is less than $øT_{th}$ (threshold torsion), then torsional effects are neglected.

The requirements for torsional and shear reinforcement are added and stirrups are provided for the total.

Total $\left(\frac{A_{v+t}}{s}\right) = \frac{A_v}{s} + 2\frac{A_t}{s}$; since A_t is only for one leg in the torsion calculations, the term $\frac{A_t}{s}$ is multiplied by 2. If there are more than two shear legs, then the inner legs provided for shear become ineffective for torsion. The longitudinal bars for torsion are added to the bar requirements for bending. Typically, torsion bars are provided at the face of the beam above the bottom reinforcement. If one torsion bar is required at each face, then they are typically provided at mid-depth of the beam. If multiple bars are required for torsion at each face, then they are placed such that equal spaces are formed between layers of bars at each face.

7.6 REINFORCEMENT LIMITS

Section 9.6 of the code provides specification for minimum reinforcement for flexure, shear, and torsion.

7.6.1 Flexural Reinforcement

If the reinforcing steel provided for flexure is greater than four-thirds of the reinforcing steel required by the analysis and design, then a minimum steel for flexure is not required; otherwise, it shall be the greater of $\left(3\sqrt{f_c'}\,b_w d/f_y\right)$ and $(200 b_w d/f_y)$.

7.6.2 Shear Reinforcement

Where $V_u > 0.5øV_c$, a minimum area of shear reinforcement ($A_{v,\,min}$) shall be provided. However, this minimum area of shear reinforcement is not required if the depth of the beam is less than or equal to 10 in^2 or when the concrete of a 24-in. or shallower beam is cast monolithically with the concrete of the slab and the overall depth of the beam is less than the greater of 2.5 times the thickness of the flange or 0.5 times the width of the web.

The minimum area of shear reinforcement ($A_{v,\,min}$) shall be the greater of

a. $0.75\sqrt{f_c'}\dfrac{b_w}{f_{yt}}$

b. $50\dfrac{b_w}{f_{yt}}$

7.6.3 Torsional Reinforcement

A minimum area of torsional reinforcement shall be provided where $T_u \geq øT_{th}$.

The transverse reinforcement for torsion and shear $\left(\dfrac{A_{v+t}}{s}\right)$ shall be the greater of

a. $0.75\sqrt{f_c'}\left(\dfrac{b_w}{f_{yt}}\right)$

b. $50\left(\dfrac{b_w}{f_{yt}}\right)$

The longitudinal reinforcement for torsion shall be the lesser of

a. $\dfrac{5\sqrt{f_c'}\,A_{cp}}{f_y}-\left(\dfrac{A_t}{s}\right)p_h\dfrac{f_{yt}}{f_y}$

b. $\dfrac{5\sqrt{f_c'}\,A_{cp}}{f_y}-\left(\dfrac{25b_w}{f_{yt}}\right)p_h\dfrac{f_{yt}}{f_y}$

where

f_{yt} is the specified yield strength of the transverse reinforcement (psi)

A_{cp} is the area enclosed by outside perimeter of concrete cross section (in^2)

A_t is the total area of longitudinal reinforcement to resist torsion (in^2)

p_h is perimeter of the centerline of the outermost torsional stirrup (in.)

7.6.4 Reinforcement Details

Concrete cover to reinforcement (Section 5.10 of this book); development length of reinforcement (Section 11.4 of this book); splice length (Section 11.5 of this book); and minimum spacing (Section 11.2 of this book) have been discussed in this book.

If the overall depth of the beam exceeds 36 in^2, then skin reinforcement (also called face bars) need to be provided. In accordance with table 24.3.2 of the code, the spacing of the skin reinforcement shall be the lesser of

a. $15\left(\dfrac{40{,}000}{f_y}\right)-2.5c_c$

b. $12\left(\dfrac{40{,}000}{f_y}\right)$

where c_c is the least distance from the surface of the bar to the tension face.

The following specification of section 9.7.3 of the code must be followed for the development of reinforcement. Development lengths shall be provided for both positive and negative reinforcement on each side of the section from the point where the reinforcement is no longer required to resist flexure.

1. During the calculations of moments, approximations are made and there may be a shift of maximum moment from a calculated point. A diagonal tension crack may also shift the location of the maximum tensile stresses. Hence, except at simple supports or the free ends of the cantilever, reinforcement shall extend for a distance equal to the greater of the effective depth of the beam (d) and 12 times the diameter of the bar (d_b) beyond the point it is no longer required to resist tension (Figure 7.5).
2. Continuing flexural reinforcement shall have an embedment length of at least (l_d) beyond the point where the reinforcement is no longer required to resist flexure. This length l_d is the development length discussed in Section 11.4 of this book.

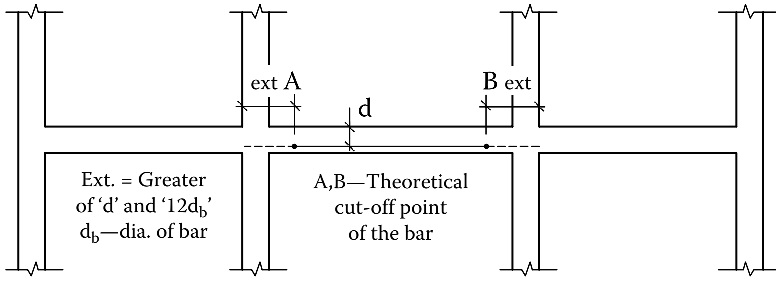


FIGURE 7.5 Extension of the flexure bar.

3. There is a reduced shear strength and loss of ductility when bars are cut off in the tension zone. Hence, the code does not allow termination of flexural tensile reinforcement in the tension zone unless the following are satisfied:
   a. $V_u \leq (2/3)øV_n$ at the cutoff point.
   b. For a #11 bar or smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and $V_u \leq (3/4)øV_n$.
   c. Stirrups in excess of that required for shear and torsion shall be provided along each terminated bar over a distance 3/4d from the termination point. Excess stirrup area shall be at least $60b_ws/f_{yt}$. The spacing of the stirrups shall be less than $d/8\beta_b$, where β_b is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.
4. At simple supports, at least one-third of the maximum positive reinforcement shall extend at least 6 in^2 into the support (Figure 7.6)
5. At other supports, at least one-fourth of the maximum positive reinforcement shall extend at least 6 in^2 into the support.
6. If a beam is part of the lateral load-resisting system, then at other supports, the maximum positive reinforcement shall be anchored to develop f_y at the face of the support to provide ductility in the event of the reversal of moment. For the concept of development length, please refer to Section 11.4 of this book.
7. It is better to provide a standard hook at the end of the positive moment tension reinforcement at the support. Or the development length of the bar shall be limited such that one of the following conditions is satisfied. These conditions also need to be satisfied at the point of inflection.
   a. $l_d \leq (1.3M_n/V_u + l_a)$ if end of reinforcement is confined by a compressive reaction
   b. $l_d \leq (M_n/V_u + l_a)$ if end of reinforcement is not confined by a compressive reaction
   where M_n is calculated assuming all reinforcement at the section is stressed to f_y and V_u is calculated at that section. l_a is the additional embedment length beyond the centerline of the support or point of inflection (in.). l_a at point of inflection is limited to the greater of the effective depth of the beam (d) or 12 times the diameter of the bar (d_b).
8. At least one-third of the negative moment reinforcement at support shall have an embedment length beyond the point of inflection at least the greatest of the effective depth of the beam (d); 12 times the diameter of the bar (d_b); and clear span (l_n) divided by 16.
9. Longitudinal torsional reinforcement shall be distributed along the inner surface of the stirrups with spacing not greater than 12 in^2. They shall have a diameter of at least 0.042 times the stirrups spacing but not less than 3/8 in. And they shall extend at least $(b_t + t)$ beyond the point required by analysis, where b_t is the width of that part of the cross section containing the closed stirrup resisting the torsion (in.). The longitudinal torsional reinforcement shall be developed at the face of the support at both ends of the beam.
10. The maximum spacing for the shear reinforcement according to table 9.7.6.2.2 shall be
    a. If $V_s \leq 4\sqrt{f_c'}\ b_wd$, then s is lesser of d/2 or 24 in^2.
    b. If $V_s > 4\sqrt{f_c'}\ b_wd$, then s is lesser of d/4 or 24 in^2.

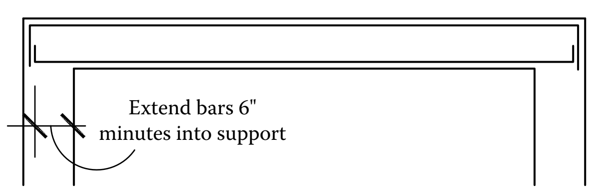


FIGURE 7.6 Simply supported beam.

11. Stirrups resisting torsion shall extend at least (b_t + d) beyond the point required by the analysis to resist any cracking due to diagonal tension. The spacing of these stirrups shall not exceed the lesser of $p_h/2$ and 12 in^2. For a hollow section, the distance between the centerline of the torsional stirrup to the inside face of the wall of the hollow section shall be at least $0.5A_{oh}/p_h$, where A_{oh} is the area enclosed by the centerline of the outermost torsional stirrup (in^2) and p_h is the perimeter of centerline of outermost closed transverse torsional reinforcement (in.).
12. Even if shear reinforcement is not required, lateral support to the compressive reinforcement shall be provided, whose diameter shall be 3/8 in. for compressive reinforcement #10 or smaller and ½ in. for compression reinforcement #11 or larger. The spacing of these stirrups shall be the least of 16 times the bar diameter of the longitudinal reinforcement; 48 times the bar diameter of the stirrups; and least dimension of the beam.
13. Longitudinal compression reinforcement shall be arranged such that every corner and alternate compression shall be enclosed by a corner of the stirrup and no bar shall be farther than 6 in^2 on each side along the stirrup from such an enclosed bar.

7.7 DEEP BEAMS

Deep beams are beams in which a significant amount of the load is carried to the supports by a compression force combining the load and the reaction. As a result, the strain distribution is no longer considered linear, and the shear deformations become significant when compared to pure flexure.

Some examples of deep beams are floor slabs subject to horizontal load; short span beams carrying heavy loads; and transfer girders. Deep beams play a very significant role in the design of mega as well as small structures. Sometimes, for architectural purposes, buildings are designed without using any column for a very large span. In such case, if ordinary beams are provided, they can cause failure such as flexural failure.

A deep beam is a beam that has a large depth/thickness ratio and a shear span-to-depth ratio less than 2.5 for concentrated load and less than 5.0 for distributed load. Because of the geometry of deep beams, their behavior is different from that of slender or intermediate beams. Shear span is the zone where the shear force is constant. It is the distance between the reaction and the nearest load point.

The following are the major differences between a deep beam and other beams:

- Because of their dimensions, deep beams behave in two dimensions rather than one dimension.
- The assumption in the design of regular beams that "plane section remains plane" cannot be used in deep beam design. The strain distribution is not linear.
- Shear deformation cannot be neglected as in the other simply supported beams. The stress distribution is not linear even in the elastic stage. At the ultimate limit state, the shape of a concrete compressive stress block is not parabolic.

The design of deep beams is specified in section 9.9 of the code and applies to those flexural members having a clear span-to-depth ratio of less than 4.0. In a deep beam, a concentrated load is allowed within a distance of twice the overall depth of the beam from the face of the support. The flexural reinforcement is designed taking into account the reduced lever arm due to the nonlinearity of the strain distribution.

The dimensions of the beams are so selected that they satisfy:

$$V_u \leq \varphi 10\sqrt{f_c'}\, b_w d \qquad \text{(ACI Equation 9.9.2.1)}$$

According to section 9.9.3 of the code, distributed reinforcement along the two faces of the deep beam shall satisfy the two conditions below. The spacing of these reinforcements shall not exceed the lesser of a fifth of the effective depth of the beam and 12 in^2:

a. Area of stirrups shall be at least $0.0025b_ws$.
b. Area of face bars shall be at least $0.0025b_ws_2$, where s_2 is the spacing of the face bars.

The minimum area of the flexural steel shall be as discussed above for other beams.

The concrete cover (Section 5.10 of this book) and the minimum spacing of the longitudinal reinforcement (Section 11.2 of this book) have been discussed before.

In deep beams, the stress in the longitudinal reinforcement is more uniform than in beams that are not deep. Hence, the high stresses can extend from the midspan to the support region. The stresses in the reinforcement are not directly proportional to the bending moment. Hence, the ends of deep beams require positive anchorage such as standard hooks, bar heads, or other mechanical devices to develop a stress of f_y.

At interior supports, negative moment tensile reinforcement shall be continuous with that of the adjacent span and the positive moment tensile reinforcement shall be continuous or spliced with the reinforcement of the adjacent span.

7.8 OTHER CONCEPTS

In this section, we will discuss the design of singly reinforced beams, doubly reinforced beams, and T beams. Designers take advantage of the flange provided to the beam by the slab to economize the beam.

7.8.1 Rectangular Section: Singly Reinforced Beam

Singly reinforced beams are designed for tension reinforcement only. An easy and popular method to design singly reinforced beams is to use the equivalent rectangular stress block, discussed in Section 1.13 of this book (Figure 7.7).

A balanced steel ratio is calculated when both the reinforcing steel and concrete reach failure:

$$\mathrm{T} = \mathrm{C}$$

$$\rho_b f_y bd = 0.85 f_c' ab$$

$$a = \beta_1 c$$

$$c = \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

$$\rho_b = 0.85\beta_1 \frac{f_c'}{f_y}\left(\frac{\epsilon_u}{\epsilon_u + \epsilon_y}\right)$$

At failure,

$$\epsilon_u = 0.003 \quad \text{and} \quad \epsilon_y = \frac{f_y}{E_s}$$

where $E_s = 29{,}000{,}000$ psi.

Hence,

$$\rho_b = 0.85\beta_1 \frac{f_c'}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right)$$

A failure in tension of a flexural member gives warning to an occupant. The failure is gradual. However, a failure in a compression is sudden with no warning to the occupants. Hence, an under-reinforced beam is preferred to an over-reinforced beam. A balanced section is economical where both steel and concrete reach failure at the same time, but in the practical world it does not happen because properties of materials may vary from the assumed values and the workmanship may not be accurate.

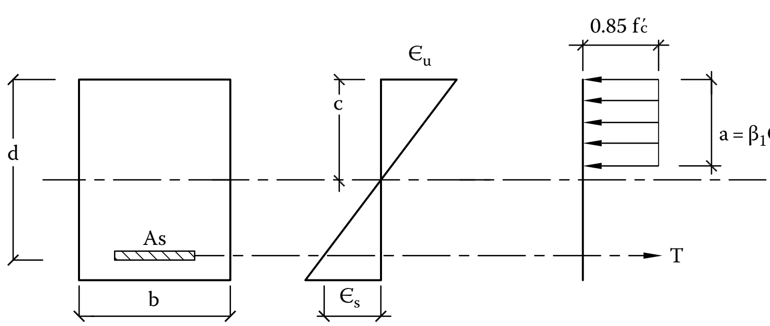


FIGURE 7.7 Singly reinforced beam.

It is a good practice to design beams with a maximum steel ratio being 75% of the balanced steel ratio:

$$\rho_{max} = 0.75\rho_b$$

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

From Figure 7.7, $T = A_s f_y$ and $C = 0.85 f_c' \beta_1 cb$

Equating T = C

$$A_s f_y = 0.85 f_c' \beta_1 cb$$

where $\beta_1 c = a$.

Hence, $A_s f_y = 0.85 f_c' ab$.

Hence, $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{\rho f_y d}{0.85 f_c'}$ because $A_s = \rho bd$.

$$\text{So, } M_n = \rho bdf_y\left(d - \frac{0.59\rho f_y d}{f_c'}\right) = \rho bd^2 f_y\left(1 - \frac{0.59\rho f_y}{f_c'}\right)$$

If $\rho f_y\left(1 - \frac{0.59\rho f_y}{f_c'}\right)$ is replaced by R, then

$$M_n = Rbd^2$$

Over-reinforced beams fail in compression before a failure in steel occurs. The actual stress in steel (f_y) is less than the yield strength (f_y). In Figure 7.7, $\varepsilon_s < \varepsilon_y$.

7.8.2 Rectangular Sections: Doubly Reinforced Beams

Doubly reinforced beams have both tension and compression reinforcement. Beams are doubly reinforced when the actual site conditions restrict the size of the beam. Concrete may not develop the compression force to resist the acting bending moment. Reinforcement is added at the compression face of the beam (Figure 7.8).

b is the width of beam (in.)
d is the effective depth of beam (in.)
d′ is the cover to compressive steel
A_s is the area of tension steel
A_s' is the area of compression steel
c is the depth of neutral axis
a is the depth of stress block

(these are illustrated in Figure 7.8).

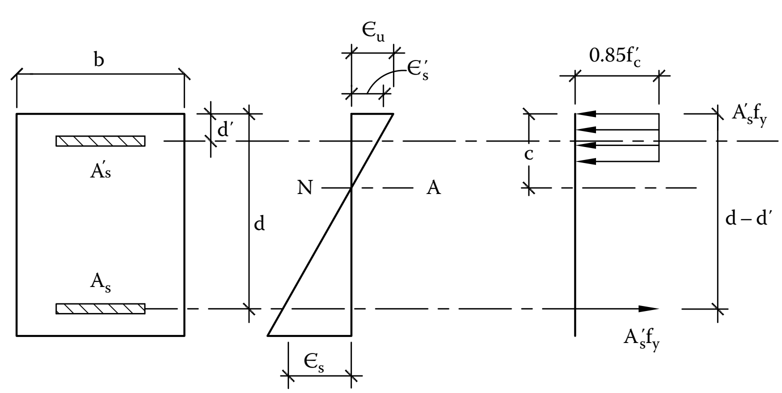


FIGURE 7.8 Doubly reinforced beam.

If the compression steel yields, then from Figure 7.8,
Moment resisted by tensile steel equivalent to the force in compression steel:

$$M_{n1} = A_s' f_y (d - d')$$

Moment contributed by remaining steel:

$$M_{n2} = (A_s - A_s') f_y (d - a/2)$$

Depth of stress block $(a) = \dfrac{(A_s - A_s') f_y}{0.85 f_c' b}$

$$A_s = \rho bd; \quad A_s' = \rho' bd$$

Hence, $a = \dfrac{(\rho - \rho') f_y d}{0.85 f_c'}$

$$M_n = M_{n1} + M_{n2} = A_s' f_y (d - d') + (A_s - A_s') f_y (d - a/2)$$

If the compression steel is below yield stress, then the limiting case would $\varepsilon_s' = \varepsilon_s$.

$$\frac{c}{d'} = \frac{\epsilon_u}{\epsilon_u - \epsilon_y}$$

from Figure 7.8.
Hence, $c = \left(\dfrac{\epsilon_u}{\epsilon_u - \epsilon_y}\right) d'$

$$A_s f_y = 0.85 f_c' \beta_1 cb + A_s' f_s'$$

$$f_s' = \epsilon_u E_s \left(\frac{c - d'}{c}\right)$$

Hence, $A_s f_y = 0.85 f_c' \beta_1 cb + A_s' \epsilon_u E_s \left(\dfrac{c - d'}{c}\right)$

But $a = \beta_1 c$.

Hence, $A_s f_y = 0.85 f_c' ab + A_s' \epsilon_u E_s \left(\dfrac{c - d'}{c}\right)$

$$M_n = 0.85 f_c' ab (d - a/2) + A_s' f_s' (d - d')$$

7.8.3 T Beams

The requirements of the code for the design of T beams are discussed in Section 7.2. For the strength analysis of the T beams, two cases as shown in Figure 7.9 arise:

a. Neutral axis in the flange
b. Neutral axis in the web

If the neutral axis is within the flange or if the depth of the stress block (a) is less than the thickness of the flange (t_f), then the beam is designed as rectangular section with a width (b), where ‘b’ is the effective width of the flange, as discussed in Section 7.2 (Figure 7.10).

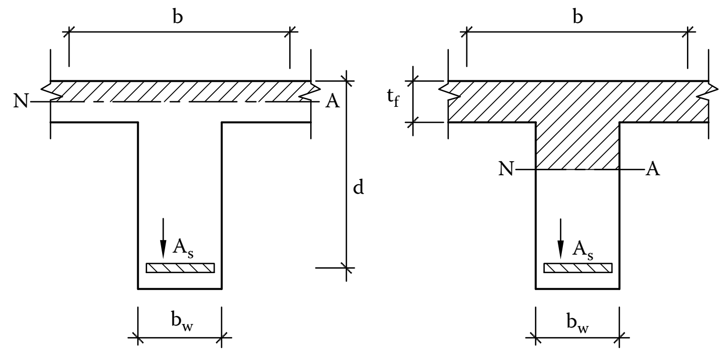


FIGURE 7.9 T beam.

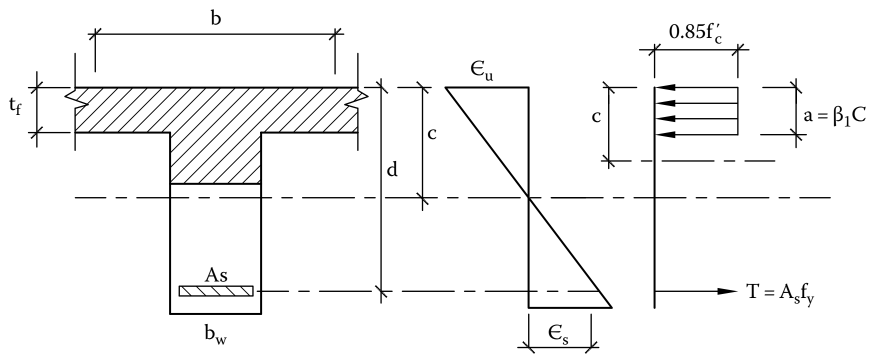


FIGURE 7.10 Mechanics of T beam.

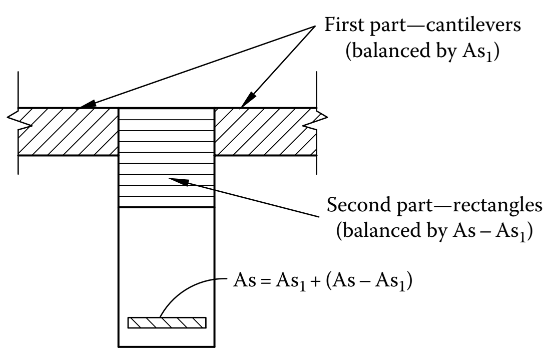


FIGURE 7.11 Mechanics of T beam.

When the neutral axis lies in the web and the depth of the stress block (a) is greater than the thickness of the flange (t_f), the tensile steel of the beam is divided into two parts. The compression-resisting concrete is also divided into two parts, as demonstrated in Figure 7.11.

The first part of the reinforcing steel (A_{s1}) balances the longitudinal compression of the first part of the T (the two cantilevers), and the second part of the reinforcing steel ($As - A_{s1}$) balances the compression of the second part of the T (rectangle):

$$A_{s1} = \frac{0.85f_c'(b - b_w)t_f}{f_y}$$

$$M_n = A_{s1}f_y\left(d - \frac{t_f}{2}\right)$$

The depth of equivalent stress block (a) of the rectangular portion is given by

$$a = \frac{\left(A_s - A_{s1}\right) f_y}{0.85\, f_c' b_w}$$

and

$$M_{n2} = \left(A_s - A_{s1}\right) f_y \left(d - \frac{a}{2}\right)$$

$$M_n = M_{n1} + M_{n2} = A_{s1} f_y \left(d - \frac{t_f}{2}\right) + \left(A_s - A_{sf}\right) f_y \left(d - \frac{a}{2}\right)$$

7.9 ASSIGNMENTS

1. Design and detail the single span simply supported beam shown in Figure 7.12 with the following loads:

Superimposed dead load from slab	400 lb/feet
Live load	1200 lb/feet
Masonry supported by beam	600 lb/feet

   Use an 8 in. wide beam with $f_c' = 5{,}000$ psi and $f_y = 60{,}000$ psi.
2. Design and details the single-span simply supported beam shown in Figure 7.13. Use the uniform load and material information from problem (1). Use the following concentrated loads:

Concentrated dead load	30,000 lb
Concentrated live load	40,000 lb

3. Design and detail the five-span continuous beam on grid line ‘B’ of Figure 1.1, supporting the two panels of the one-way slabs, S-1 and S-2. The slab is 6 in. thick supporting a live load of 100 psf and a superimposed dead load of 25 psf. Use an 8 in. wide web of the T beam. Use $f_c' = 4{,}000$ psi and $f_y = 60{,}000$ psi.

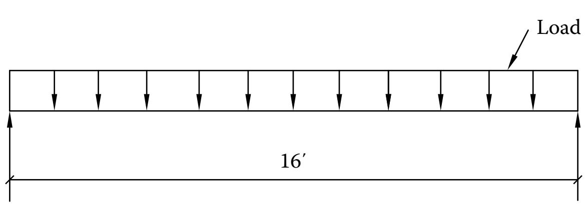


FIGURE 7.12

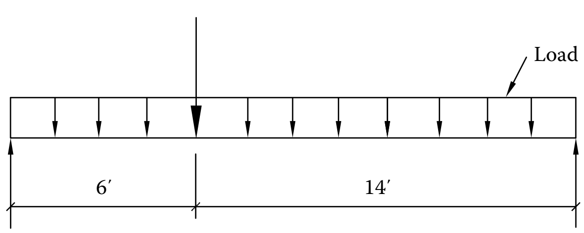


FIGURE 7.13

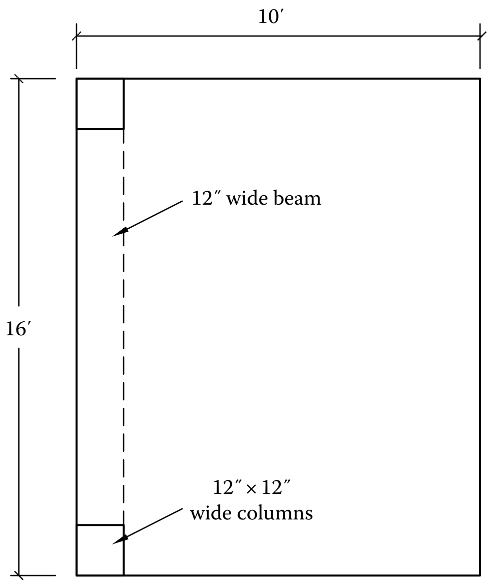


FIGURE 7.14

4. Design and detail the five-span continuous beam on grid line ‘A’ of Figure 1.1, as a rectangular beam. The beam supports the 6 in. thick slab S-1 carrying a live load of 30 psf and a superimposed dead load of 40 psf. Additionally, the beam supports a masonry parapet weighing 150 lb/feet. Use f'_c = 4,000 psi and f_y = 60,000 psi.
5. Design a 12 feet span simply supported deep beam supporting a factored concentrated load of 800,000 lbs at its midspan. The beam is 8 in. wide. Use f'_c = 8,000 psi and f_y = 60,000 psi.
6. Design a 12 in. wide simply supported beam, as shown in Figure 7.14. The slab is 10 in. thick, carrying a live load of 50 psf. Use f'_c = 4,000 psi and f_y = 60,000 si. The beam is subject to torsion.

8 Columns

8.1 INTRODUCTION

A reinforced concrete column is a structural element designed to support compressive loads or a combination of axial loads and bending. The primary function of a column in a building is to support beams or slabs and transfer the load to the foundations. Columns have two axes, and bending can occur on one or both the axes. The bending action may produce tensile stresses on part of the cross section of the column. Despite the tensile forces acting on columns, they are generalized as compressive members because compressive forces dominate in the column design.

Columns can be classified into three categories (Figure 8.1):

a. Members reinforced with longitudinal bars and lateral ties
b. Members reinforced with longitudinal bars and continuous spirals
c. Composite columns longitudinally encasing structural steel members such as tubes, channels, or W sections and may further be reinforced with longitudinal bars with transverse ties

Columns are also divided into two categories: short columns and slender columns. The strength of short columns is controlled by the strength of the material and the geometry of the cross section. Reinforcing bars are placed axially in the column to provide additional axial stiffness. In short columns, the height is such that lateral buckling is not considered. Slender columns are long columns as related to their cross-sectional dimensions, which necessitates the consideration of lateral buckling. The strength of the column reduces, as the height of the column increases for same the cross-sectional dimensions of the column due to lateral buckling. Buckling is a mode of failure resulting from structural instability due to compressive action on columns. Buckling is a sudden large deformation of the column due to a slight increase in the existing load on the column that had exhibited little, if any, deformation before the load was increased.

8.2 SHORT COLUMNS

Columns that are subject to pure compression have concrete taking all the stresses because it is good in compression. However, reinforcing bars are provided in columns because there might be eccentricity of load acting on the column, which creates moment and thereby tensile stresses. The presence of reinforcing bars in columns reduces shrinkage cracks. Finally, reinforcing bars have some compressive capacity that helps in reducing the size of the columns.

Concrete is elastic up to half its strength ($f_c'/2$), wherein the stress is proportional to strain. Steel is elastic till its yield strength. Hence, it is assumed that as long as concrete is elastic, the compressive strain in concrete is equal to the compressive strain in steel.

$$\epsilon_c = \frac{f_c}{E_c} \quad \text{and} \quad \epsilon_s = \frac{f_s}{E_s} \; \epsilon_c = \epsilon_s$$

Hence,

$$f_s = \frac{E_s}{E_c} f_c = nf_c$$

where n – modular ratio $= \dfrac{E_s}{E_c}$

Load capacity of column $P = f_cA_c + f_sA_s = f_cA_c + nf_cA_s = f_c(A_c + nA_s)$

Since $A_c = A_g - A_s$

$$P = f_c\left(A_g - A_s + nA_s\right) = f_c\left(A_g + \left(n-1\right)A_s\right)$$

The term $(n - 1)A_s$ is called the transformed section.

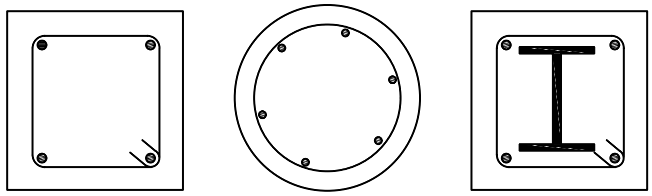


FIGURE 8.1 Types of columns. (a) Lateral tie; (b) continuous spiral; (c) composite section.

For nonlinear response of reinforced concrete, the above equation is transformed to

$$P_n = 0.85f_c'\left(A_g - A_{st}\right) + f_y A_{st}$$

where
- P_n is the capacity of the column (lb)
- f_c' is the strength of concrete (psi)
- f_y is the yield strength of steel (psi)
- A_g is the gross cross-sectional area of column (in^2)
- A_{st} is the cross-sectional area of reinforcing steel (in^2)
- A_c is the cross-sectional area of concrete ($A_g - A_{st}$) (in^2)
- f_c is the stress in concrete (psi)
- f_y is the stress in steel (psi)

Columns are rarely concentrically loaded with compressive loads. Usually, columns are members of frames carrying loads from continuous beams or slabs. There is an eccentricity of loads applied on columns that converts into a bending moment. Or there may be bending moments acting on columns from the frame action. The action of columns under direct compression and bending is discussed in Section 1.18 of this book.

8.3 SLENDER COLUMNS

Due to high-strength concrete and steel available in the market, the dimensions of columns are reduced to accommodate more utility space and architectural features. Columns become slender when their dimensions are small compared to their height. Slenderness of a column is defined with the term slenderness ratio (l/r) of the column, which is the ratio of the height of the column (l) to its radius of gyration (r). The radius of gyration is the square root of the moment of inertia (I) of the column divided by the cross-sectional area (A) of the column:

$$r = \sqrt{\left(\frac{I}{A}\right)}$$

For a square or a circular column, r is the same for any axis, but for a rectangular column, it will be different for the two axes.

If the two cross-sectional dimensions of a rectangular column are b and d, then in one direction, I = (bd^3/12); I/A = bd^3/(12bd) = (d^2/12); $r = \sqrt{(d^2/12)} = d/\sqrt{12}$.

Similarly, in the other direction, $r = \dfrac{b}{\sqrt{12}}$.

When r increases, the slenderness ratio (l/r) decreases. Typically, the axis along which the moment acts is made stronger than the other axis in a rectangular column by orienting the column with its longer side along the axis of the application of the moment.

Swiss mathematician, Leonard Euler stated that slender columns fail by buckling at critical load (P_c).

$$P_c = \frac{\pi^2 E_t I}{\left(kl\right)^2}$$

where k is the effective length factor for the column, which depends upon the end conditions of the column. ‘kl’ is the effective length of the column. As can be seen from the equation, as the height of the column increases, the critical load (P_c) decreases.

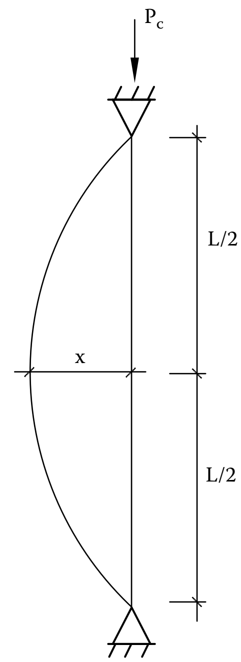


FIGURE 8.2

In other words, it means that the capacity of the column to resist buckling decreases. For a column hinged at both ends, k = 1. For columns fixed at both ends, k = ½. Hence, a column with both ends fixed will carry four times the load as a column with the same dimensions with both ends hinged. The values of 'k' for various end conditions of columns are provided in the AISC Steel Design Manual (AISC 360). Columns fixed at both ends are stronger than columns hinged at both ends.

Let us examine how buckling affects the strength of the column. Consider a square column of dimension 'b' for each side of the square, hinged at each support (Figure 8.2). A critical load (P_c) is applied on the column that initiates buckling. When the column buckles, it deflects by a distance 'x' at the mid-height, thereby creating a moment of Px. Now, there are two forces acting on the column: P and Px. The deflection 'x' increases, thereby increasing the moment Px. The column continues to be overstressed till it fails.

In actual structures, columns support concrete beams, either from one axis or both axes. There are no special fixed or hinged joints designed for the concrete columns. These joints are between fixed and hinged joints, and hence the value of 'k' can be taken between 0.5 (for fixed) and 1.0 (for hinged). Concrete frames are analyzed in such a way that the sidesway of the column is eliminated or at least reduced to make it insignificant. If this is not done, then columns would become unstable and slide and topple. The load capacity of braced columns is significantly higher than the load capacity of unbraced columns. A designer should make sure that a column is braced at both axes.

8.4 AXIAL LOAD AND BENDING

Axial loads on columns can be in the form of loads from beams and slabs and from the column above, supported on the subject column. Bending moments can act on columns due to eccentric loads, unbalanced moments from continuous beams supported on the columns, and lateral loads on frames.

When columns support beams, the loads from the beams are transferred to the columns as the reactions from the beam. When columns support slabs directly, tributary areas are used to calculate the loads transferred from the slab to the beams.

Let us assume a column with an axial load (P) and end moments M_A and M_B at the top and bottom of the column. For the sake of simplicity, let us assume that M_A and M_B are equal. The column is deflected due to the action axial load and the moments applied at the end. Assume that the maximum deflection in the column occurs at the mid-height of the column (X). Let $M_A = M_B = M$. Moment caused due to the deflection at the mid-height = PX. The total bending moment at the mid-height is equal to M + PX. This second-order force is called the P-Δ effect.

Now let us assume that there is no moment acting on the top and bottom of the column. Assume that a lateral load of magnitude L/2 is acting at the top and bottom of the column. Let the height of the column be 'h.' The lateral load creates a moment Lh/2 at the mid-height. Now if we apply the axial load, the column is deflected due to the action of the axial load and the moments applied due to the lateral load. Moment caused due to the deflection at the mid-height = PX. If there was no axial load, the column would have been subjected to a triangular moment with a maximum of Lh/2 at mid-height. The total bending moment at the mid-height is equal to PX + Lh/2. This is the second-order force, the P-Δ effect.

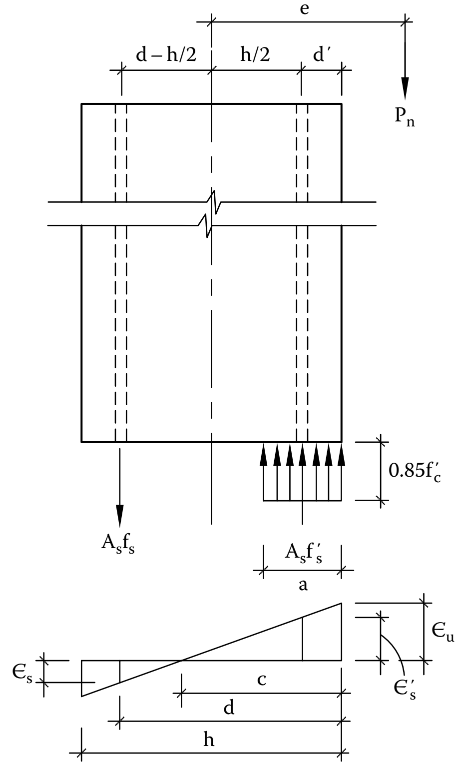


FIGURE 8.3 Column—Eccentric Load.

When concrete reaches its limiting strain (ε_u) at the same time as the steel reaches its yield strain (ε_y) for a given column, the load (P_b) and the moment (M_b) are said to be in balanced failure mode. The corresponding eccentricity (e_b) = M_b/P_b. Please refer to Figure 8.4 and Section 1.18 of this book.

Depth of neutral axis (c_b) = $d\left(\dfrac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right)$

Depth of compressive stress block (a_b) = $\beta_1 c_1$

$$P_b = 0.85 f_c' a_b b + A_s' f_s' - A_s f_s$$

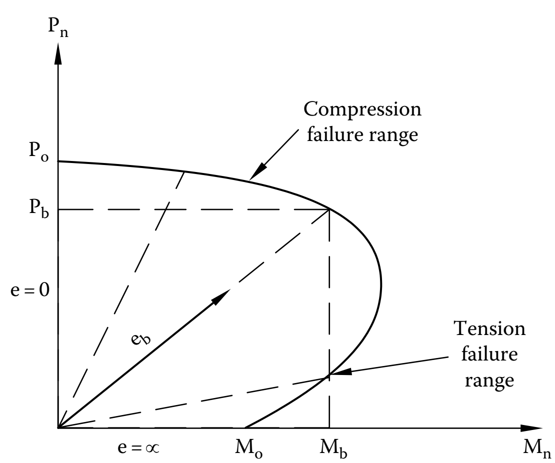


FIGURE 8.4 Interaction diagram.

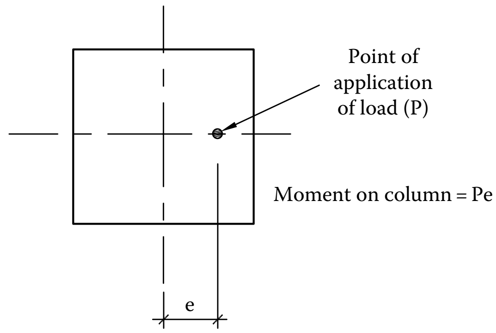


FIGURE 8.5 Eccentrically loaded columns.

and

$$M_b = P_b e_b = 0.85 f_c' a_b b\left(\frac{h}{2} - \frac{a_b}{2}\right) + A_s' f_s'\left(\frac{h}{2} - d'\right) + A_s f_s\left(d - \frac{h}{2}\right).$$

8.5 STRUCTURAL MODELING

A frame analysis consists of distributing the internal forces and the corresponding deformation created by a loading. When a load is applied, a structure responds. Once the limit of linearity in the P-Δ (load deflection) curve is reached, three kinds of nonlinearities appear:

- Geometric
- Joint
- Material

Joint nonlinearity appears at low levels of loads. Then the geometric nonlinearity appears, which is caused by the influence of the structure's actual deformed shape on the internal force distribution. Finally, the material nonlinearity manifests and at the peak load, the structure is in an imminent danger of collapse.

Structural frames are modeled as two-dimensional (2D) or three-dimensional (3D) frames. 2D frames are also called plane frames and 3D frames are also called space frames. Before the modeling of the frames, the main structural elements such as the main frames, joints, and foundations are identified. The frames are classified as braced or unbraced frames and sway or nonsway frames.

Braced frames are adequately stiff frames, and bracing such as trusses or shear walls are used to prevent sidesway. The function of the columns is to resist gravity loads, and the function of the bracing is to support lateral loads and any gravity loads applied on them. Unbraced frames do not have a strong bracing system and are not very stiff. In unbraced frames, columns resist both gravity and lateral loads.

Nonsway frames are adequately stiff frames and do not allow any additional force (P-Δ) arising from node displacement while resisting the in-plane horizontal forces. The second-order effects (P-Δ) are negligible in nonsway frames. Normally, a braced frame is a nonsway frame. Sway frames are generally unbraced frames, and the second-order effects (P-Δ) are not negligible in them. The ratio of the design value of the total vertical load (P) and the critical load (P_{cr}) determines the sway stability. When the (P/P_{cr}) ratio increases, the risk of instability increases and the second-order (P-Δ) effects come into play.

8.6 FIRST-ORDER ANALYSIS

Section 6.6 of the code provides the specifications for the first-order analysis of frames. The slenderness effects of the columns are taken into account using the moment magnifier approach. During the first-order analysis, the gross sectional area of the elements (A_g) is considered. However, the moment of inertia is reduced in accordance with table 6.6.3.1.1(a) or (b) of the code after applying the stiffness reduction factors. If the analysis shows that the columns have cracked, then the value of the cracked moment of inertia in the table is considered. If sustained lateral loads are considered, then the moment of inertia of the columns is divided by $1 + \beta_d$, where β_d is the ratio of the maximum sustained shear in a story to the maximum factored shear in that story associated with the same load combination.

Moments for slender columns are magnified using a moment magnification factor that is based upon the axial load (P_u) acting on the column and the critical buckling load (P_{cr}) for a nonsway column. However, for a sway column, the moment magnification factor is based upon the sum of axial load (P_u) acting on the column and the sum of critical buckling load (P_{cr}) at that level.

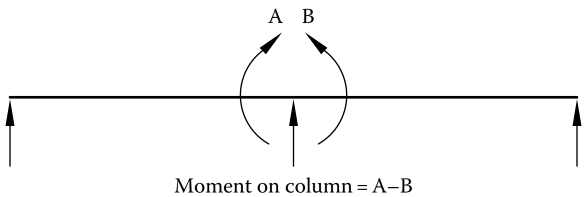


FIGURE 8.6 Unbalanced moment on beams.

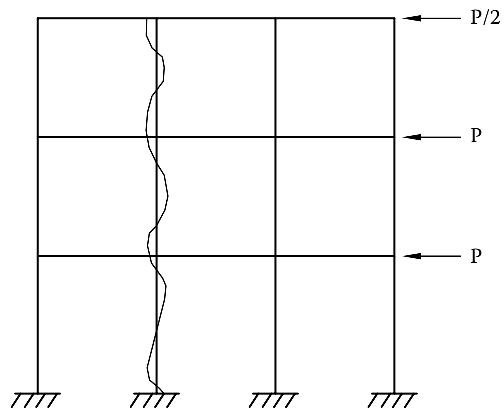


FIGURE 8.7 Moment on columns due to lateral load.

In the first-order analysis, which is an elastic analysis, the effects of internal forces due to deflections are not considered. If the analysis shows that the assumed sizes of the members are undersized by 10%, then the dimensions of the members need to be revised and the analysis performed again. A frame can be analyzed as a nonsway frame if the increase in column end moments due to the second-order effects does not exceed the column moments obtained from the first-order analysis by more than 5% and the value of the stability index (Q) does not exceed 0.05.

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} l_c} \qquad \text{(ACI Equation 6.6.4.4.1)}$$

where ΣP_u and V_{us} are the total factored vertical load and the horizontal shear and Δ_o is the first-order relative lateral deflection between the top and bottom of the story due to V_{us}. l_c is the length of the column measured center to center of the joint.

According to section 6.6.4.4.2 of the code the critical buckling load shall be

$$P_c = \frac{\pi^2 (EI)_{eff}}{(kl_u)^2} \qquad \text{(ACI Equation 6.6.4.4.2)}$$

where $(EI)_{eff}$ is calculated by any of the three equations below:

$$(EI)_{eff} = \frac{0.4 E_c I_g}{1+\beta_{dns}} \qquad \text{(ACI Equation 6.6.4.4.4a)}$$

$$(EI)_{eff} = \frac{(0.2 E_c I_g + E_s I_{se})}{1+\beta_{dns}} \qquad \text{(ACI Equation 6.6.4.4.4b)}$$

$$(EI)_{eff} = \frac{E_c I}{1+\beta_{dns}} \qquad \text{(ACI Equation 6.6.4.4.4c)}$$

where β_{dns} is the ratio of the maximum sustained factored axial load to maximum factored load with the same load combination and alternate moment of inertia (I) is calculated in accordance with table 6.6.3.1.1(b) of the code.

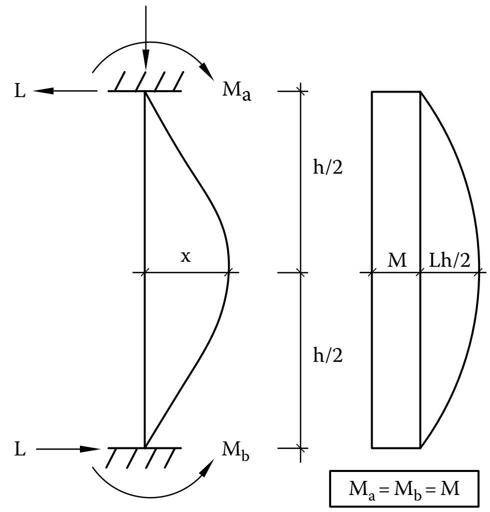


FIGURE 8.8 P-Δ effect.

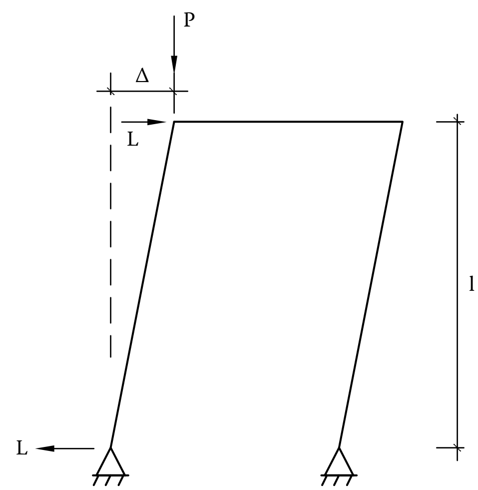


FIGURE 8.9 P-Δ effect in columns.

8.6.1 Nonsway Frames: Moment Magnification

For a nonsway frame, the magnified moment (M_c) is

$$M_c = \delta M_2 \qquad \text{(ACI Equation 6.6.4.5.1)}$$

where δ is the moment magnification factor and M_2 is the moment obtained by the first-order method.

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1.0 \qquad \text{(ACI Equation 6.6.4.5.2)}$$

where C_m is the factor relating the actual moment diagram to an equivalent uniform moment diagram. For transverse loads applied between supports, $C_m = 1.0$, else

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} \qquad \text{(ACI Equation 6.6.4.5.3a)}$$

where M_1/M_2 is negative if the column is bent in a single curvature and positive if bent in a double curvature. M_1 has the lesser absolute value. The minimum value of M_2 is

$$M_{2\,min} = P_u(0.6 + 0.03h) \qquad \text{(ACI Equation 6.6.4.5.4)}$$

and if $M_{2\,min}$ exceeds M_2 then C_m can be taken as 1.0.

8.6.2 Sway Frames: Moment Magnification

For a sway frame, moment magnification is performed to calculate moments of the columns at both ends:

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad \text{(ACI Equation 6.6.4.6.1a)}$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \qquad \text{(ACI Equation 6.6.4.6.1b)}$$

where δ_s is the moment magnification factor used for frames not braced against side sway, to reflect lateral drift resulting from lateral and gravity loads.

$$\delta_s = \frac{1}{1-Q} \geq 1.0 \qquad \text{(ACI Equation 6.6.4.6.2a)}$$

(This equation can be used till a value of 1.5 for δ_s.)

$$\delta_s = \frac{1}{1-\dfrac{\Sigma P_u}{0.75\Sigma P_c}} \geq 1.0 \qquad \text{(ACI Equation 6.6.4.6.2b)}$$

Section 6.7 of the code deals with the requirements of the second-order analysis of frames. In the second-order elastic analysis of frames, the influence of axial loads, presence of cracked regions along the length of the member, and effects of load duration along with slenderness effects along the length of the column are also considered. In the inelastic second-order analysis, material nonlinearity, member curvature, lateral drift duration of loads, shrinkage, creep, and the interaction with the supporting foundations are considered. The cross-sectional dimensions and the other sectional properties of elements are calculated as explained in the previous sections. The reader is encouraged to review theory of structures books on second-order analysis to understand the various procedures and to develop and use the available software to model the structures.

8.7 DESIGN OF COLUMNS

The reader is encouraged to refer to other chapters of this book for the following information:

Design properties of concrete
Design properties of steel reinforcement
Materials, design, and detailing requirements for embedment
Required strength

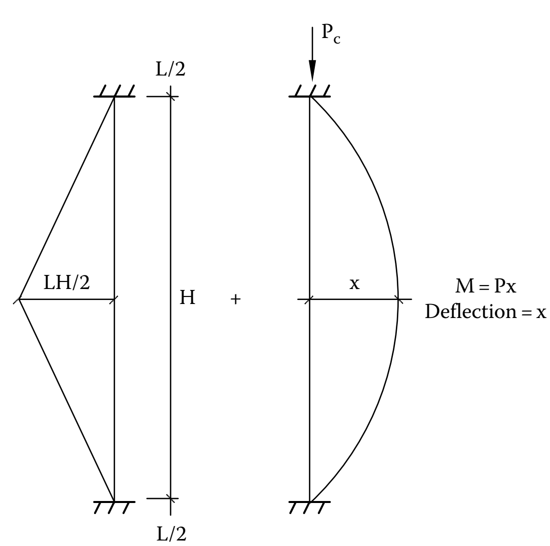


FIGURE 8.10 Critical Load.

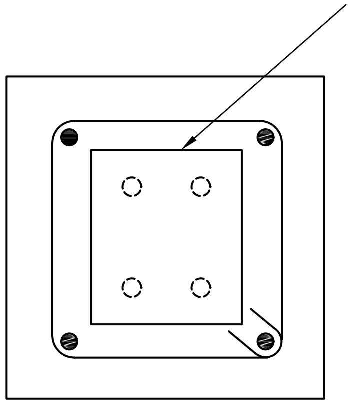


FIGURE 8.11 Encased Steel Plate.

Concrete cover requirements
Development lengths
Minimum spacing of reinforcement

If structural steel is embedded as longitudinal reinforcement in columns, then the columns are called "composite columns." Concrete can also be encased in structural steel to form a composite column. The minimum thickness of the steel encasement shall be $b\sqrt{\frac{f_y}{3E_s}}$ for width 'b' of the rectilinear columns and $h\sqrt{\frac{f_y}{8E_s}}$ for diameter 'h' of the circular column. In composite columns, forces are transferred between the steel and concrete sections by means of direct bearing, shear connectors, or bond, in accordance with the axial strength assigned to each element. For a concrete core encased in a structural steel plate, longitudinal reinforcement is not required. For structural steel encased in concrete, a minimum area of longitudinal steel equal to $0.01(A_g - A_{sx})$ is required but it should not exceed $0.08(A_g - A_{sx})$. A_{sx} is the area of the structural steel. In a composite column with structural steel cores, a longitudinal reinforcing bar shall be provided at every corner of a rectangular cross section with other longitudinal bars spaced not further than half the least-side dimension of the column. The transverse reinforcement shall have a minimum diameter equal to 0.02 times the greatest dimension of the column, but a minimum of 3/8 in. diameter bar. The transverse reinforcement bar size need not exceed 5/8 in. diameter. The clear spacing of the transverse bars shall be 4/3 times the size of the aggregate, and the center-to-center spacing shall not exceed 16 times the diameter of the longitudinal reinforcing bar, 48 times the diameter of the transverse bar, and 0.5 times the least lateral dimension of the column.

8.7.1 Design Strength

The design strength of the columns shall be verified as follows:

$$\Phi P_n \geq P_u$$
$$\Phi M_n \geq M_u$$
$$\Phi V_n \geq V_u$$
$$\Phi T_n \geq T_u$$

where
P_n is the nominal axial strength (Section 4.3 of this book)
M_n is the nominal flexural strength (Section 4.2 of this book)
V_n is the nominal shear strength (Section 4.4 of this book)
T_n is the nominal torsional strength (Section 4.5 of this book)
Φ is the strength reduction factor (Section 5.14 of this book)

8.7.2 Longitudinal Reinforcement

Reinforcement limits and detailing of columns is specified in sections 10.6 and 10.7 of the code. A minimum reinforcement steel of area $0.01A_g$ but not exceeding $0.08A_g$ shall be provided in the columns. If the design requires an area of reinforcement

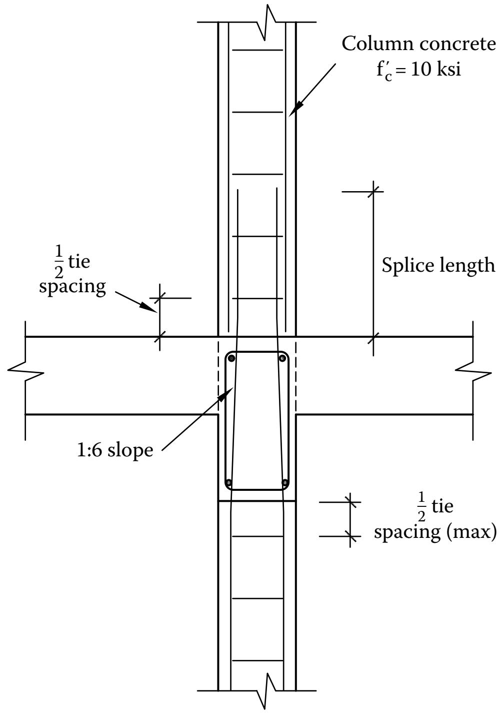


FIGURE 8.12 Column detail.

more than $0.08A_g$, then the column dimensions need to be increased. There shall be at least three longitudinal reinforcing bars within triangular ties and four within rectangular or circular ties. Transverse reinforcement or ties are discussed in Section 4.3 of this book.

At the transition of the columns at floors, an offset bent is provided for the longitudinal reinforcement bars being continued to the floor above. Typically, these bars serve as dowels for the column above, and their lengths are equal to splice length calculated. The slope of the inclined portion of the offset bent longitudinal reinforcing bar in relation with axis of the column cannot exceed 1 in 6 (Figure 8.12). The portions of the longitudinal reinforcing bar above and below the bent shall be parallel to the axis of the column. Where longitudinal bars are offset, horizontal support must be provided by ties, hoops, spirals, or parts of floor construction. These elements are designed to resist 1.5 times the horizontal component of the calculated force in the inclined portion of the offset bars and shall be placed not more than 6 in^2 from the points of bends. Now, if the column is offset by 3 in. or more at the transition of floors, then the offset bents are not provided. Instead, separate dowels are provided that are lap spliced with the longitudinal reinforcing bars adjacent to the offset column faces.

If anchor bolts are placed on the top of columns, they shall be enclosed by the longitudinal and transverse reinforcing bars for better confinement and transfer of loads (Figure 8.11). This region has a potential of cracks due to temperature and shrinkage. The transverse reinforcement shall be within 5 in^2 from the top of the column and shall consist of at least two #4 or three #3 bars.

The bottom most transverse reinforcement (tie) of the column starting at the foundation or at a slab must be located at not more than half the spacing of the ties above the footing or the slab (Figure 8.12). The top most tie of a column below a floor shall be located at not more than half the spacing of the tie below the lower most horizontal reinforcement of the slab, drop panel, or shear cap above that column. In case beams or brackets are framing into the column, then top most tie of a column below a floor shall be located at not more than 3 in^2 below the lowest horizontal reinforcement of the beam or bracket.

When spirals are used to provide lateral support to the longitudinal reinforcement of the column, then the bottom of the spiral shall be placed at the top of the footing or slab. The top of the spiral of a column with beams or brackets framing into all sides of the columns shall extend to the level of the lowest horizontal reinforcement of the beam or bracket. The top of the spiral of a column with beams or brackets not framing into all sides of the columns shall extend to the level of the lowest horizontal reinforcement of the beam or bracket. Additional ties must be provided above the termination of the spiral at the bottom of slab, drop panel, or shear cap. If columns have capitals, then the top of the spiral of the columns must extend to the level at which the diameter or width of the capital is twice that of the column.

Lap splices are of two types: tension and compression. If the tensile stress in the lap splice is less than or equal to 0.5 times the yield strength of the splice (f_y), then a class 'A' splice is provided; otherwise, a class 'B' splice is provided. The compression lap splice shall have 0.25 times the yield strength of the tensile lap splice. The length of the bar can be reduced but not less than 12 in^2. For discussions on splice, please refer to Chapter 11 of this book. A class ‘A’ lap splice is equal to the development length, and a class ‘B’ lap splice is equal to 1.3 times the development length.

8.7.3 Shear Reinforcement

Shear reinforcement in columns is required when $V_u > 0.5\phi V_c$ and the minimum area of shear reinforcement shall be greater of: $0.75\sqrt{f_c'}\frac{b_w s}{f_{yt}}$ and $50\frac{b_w s}{f_{yt}}$ (section 10.6.2.2 of the code), where b_w is the width of a rectilinear column or diameter of a circular column and f_{yt} is the yield strength of the shear reinforcement. The maximum spacing of the shear reinforcement shall be (according to table 10.7.6.5.2 of the code)

a. The lesser of d/2 and 24 in^2 if $V_s \le 4\sqrt{f_c'}b_w d$
b. The lesser of d/4 and 12 in^2 if $V_s > 4\sqrt{f_c'}b_w d$

8.7.4 Ties in Columns

In columns subject to pure compression or compression plus a small magnitude of bending moment, the longitudinal reinforcements are uniformly placed. When columns are subject to a large magnitude of bending moment, reinforcement is placed on the faces of the column in the direction where a bending moment is acting. The column acts like a vertical beam. The longitudinal reinforcements in columns are tied together in a cage by using lateral tie in a rectilinear form or in the form of spirals. The ties also serve the purpose of inhibiting the highly stressed longitudinal reinforcement from bursting out of the concrete.

8.8 TENSION COLUMNS

In reinforced concrete columns subject to tension, initially the concrete takes the tension. Since the tensile capacity of concrete is low, cracks are developed, forming air gaps in the column and not allowing the tensile stresses to follow a path. Then, the reinforcing steel comes into action and resists all tensile forces.

$$P = f_s A_s$$

When the tensile load is increased further, the reinforcing steel reaches its yield point and

$$P = f_y A_s$$

Typically, columns subject to tensile loads are designed at twice their capacity in order to prevent cracks in the concrete. Concrete does not contribute to the load-carrying capacity of the tension columns but provides cover to the reinforcement for fire and weather protection.

8.9 ASSIGNMENTS

1. A rectangular 8 in. × 18 in. column, protected from sidesway has four #8 longitudinal bars. Determine the balanced failure mode parameters (M_b, P_b, and e_b) and draw the interaction diagram. What would be the capacity of the column if it was carrying only axial load? Use $f_c' = 4{,}000$ psi, $f_y = 60{,}000$ psi, and a clear cover of 1.5 in^2 for the reinforcement.
2. Design a 12 in. × 20 in. interior column of a multistoried building. The column is braced with 12 in. × 20 in. beams in both directions at the top and bottom and is protected from sidesway. Use $f_c' = 5{,}000$ psi, $f_y = 60{,}000$ psi, and a clear cover of 1.5 in^2 for the reinforcement.

Type of Load	Dead Load	Live Load
Axial	160 K	120 K
Moment at top	12 K-feet	55 K-feet
Moment at bottom	10 K-feet	40 K-feet

3. Design the column in problem (2) if it is not protected from sidesway and additionally, an axial load of 80 K, a moment of 30 K-feet at the top, and a moment of 20 K-feet act on the column due to wind. The sum of factored loads at the bottom of all the columns at that level in that frame is 1600 K.

4. A 12 in. × 12 in. reinforced concrete column has an unsupported height of 20 feet. It is reinforced with six #9 bars. The factored loads are P_u = 300 K, M_u (top) = 60 K-feet and M_u (bottom) = 75 K-feet. The column is assumed bent at the top and bottom and undergoes a bent in double curvature. Use f_c' = 5,000 psi, f_y = 60,000 psi, and a clear cover of 1.5 in^2 for the reinforcement. Check the adequacy of the column.
5. A 16 in. × 16 in. short concrete column is reinforced with six #8 bars. If the axial load on the column is 200 K, what applied moment would fail it? Use f_c' = 5,000 psi, f_y = 60,000 psi, and a clear cover of 1.5 in^2 for the reinforcement.
6. Construct the interaction diagram of a symmetric 12 in. × 12 in. column reinforced with four #8 bars, assuming bending in one direction. Use f_c' = 4,000 psi, f_y = 60,000 psi, and a clear cover of 1.5 in^2 for the reinforcement.
7. Solve problem (6) assuming bending in both directions.

9 Walls

9.1 INTRODUCTION

Several types of walls are used in building construction. Walls can be classified based upon the material used and their functionality. Materials that are mainly used for walls are reinforced concrete, precast concrete, concrete masonry units (CMUs), and wood and metal stud. Based on their functionality, walls can be gravity load–bearing walls, lateral load–bearing walls, retaining walls, partition walls, infill walls, and parapet walls (Figures 9.1 and 9.2).

In low-rise buildings, CMU and wood and metal stud walls can be used to resist the combination of gravity and lateral loads. These walls carry the gravity loads of the floors and roof and also act as shear walls to resist lateral loads such as wind and earthquake. In high-rise buildings, CMU walls are used as infill exterior walls between exterior columns. They accommodate the doors and windows and resist their self-weight plus any lateral loads such as wind acting perpendicular to their surface. They transfer the wind loads to the slabs that are horizontal diaphragms. These infill walls span vertically between two floors. They are connected to the reinforced concrete slabs using reinforcement in their grouted cells. Metal and wood stud walls are also used for interior partitions in low-rise and high-rise buildings.

Reinforced concrete walls have a variety of uses. They can be used to resist gravity loads and act as shear walls in high-rise buildings to resist lateral loads. They brace the concrete frames in the building and prevent sidesway of the structure. Concrete walls are also used as retaining walls to retain earth and water.

Other varieties of walls include precast concrete walls and tilt-up concrete walls. The concrete of precast concrete walls is cast at the factory, and they are shipped to the construction site, installed in place, and connected to the concrete frames or the diaphragms. The concrete of the tilt-up walls is cast at ground of the construction site and the walls are tilted up to install them at place and connect them to the concrete frames or diaphragms.

Chapter 11 of the code provides specifications for reinforced concrete walls. In this chapter, we will focus on the design of reinforced concrete walls to carry gravity loads, shear walls (both in-plane and out-of-plane shear), retaining walls, and parapet walls. We will not discuss precast concrete or tilt-up walls. Special walls for earthquake design related to chapter 18 of the code are also beyond the scope of this book.

9.2 LOADS ACTING ON WALLS

For a wall located inside a building, which may or may not brace the structure, the following loads act:

1. Axial loads
2. In-plane shear
3. Out-of-plane shear
4. In-plane moment
5. Out-of-plane moment
6. Self-weight

The axial loads are gravity loads that come from floors, roofs, and any beams supported on the walls. The self-weight of the walls above the subject wall can also be considered as axial load. In-plane shear acts on walls due to the lateral forces acting parallel to the wall. Out-of-plane shear acts on walls due to the lateral forces acting perpendicular to the wall. In-plane moments on the walls are created due to the lateral forces acting parallel to the wall. Out-of-plane moments on the walls are created due to the lateral forces acting perpendicular to the wall. Walls can also experience in-plane moment due to beams supported on the walls and spanning in the direction of the walls and out-of-plane moment due to beams supported on the walls and spanning perpendicular to the walls (Figure 9.3).

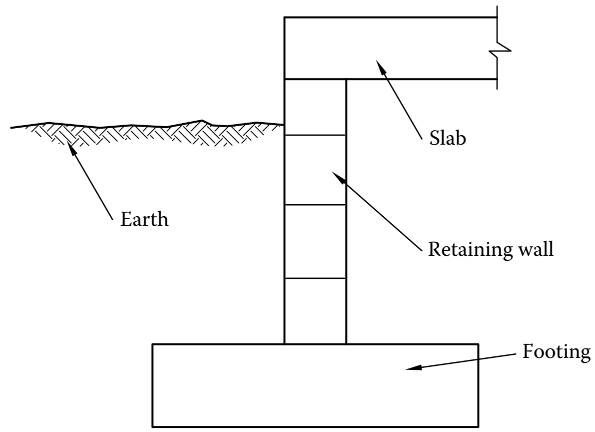


FIGURE 9.1 Retaining wall.

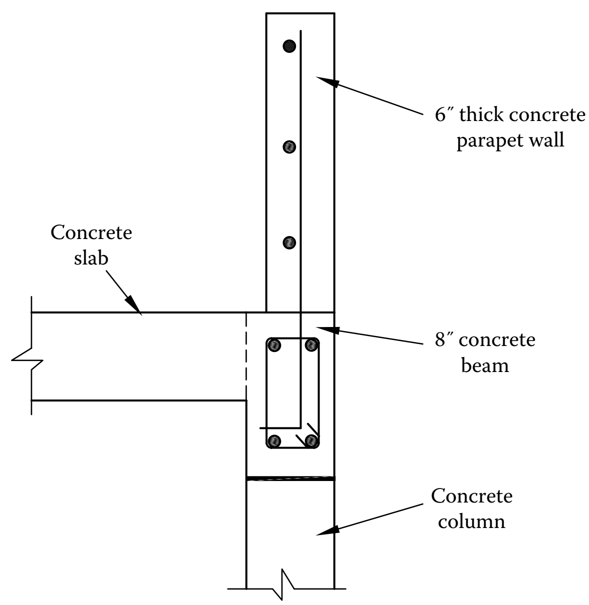


FIGURE 9.2 Parapet wall.

9.3 GRAVITY LOAD DISTRIBUTION ON WALLS

When concentrated loads act on walls (beams or columns), then the effective width of the wall considered to resist that load shall be lesser of center-to-center distance between consecutive loads and bearing width plus four times the wall thickness (Section 11.2.3 of the code) (Figure 9.4).

9.4 MINIMUM THICKNESS

The minimum thickness of the bearing walls shall be greater of 4 in^2 and 1/25 of the lesser of unsupported height or length. The minimum thickness of the nonbearing walls shall be greater of 4 in^2 and 1/30 of the lesser of unsupported height or length. The minimum thickness of exterior basement and foundation wall shall be 7.5 in^2 (Section 11.3.1.1 of the code).

9.5 STRENGTH REQUIREMENT

The design load combinations for walls are discussed in Chapter 3 of this book. The required strength and slenderness effects for walls is same as that for columns as discussed in Chapter 8 of this book, and the concept of moment magnification factor for columns also applies on walls.

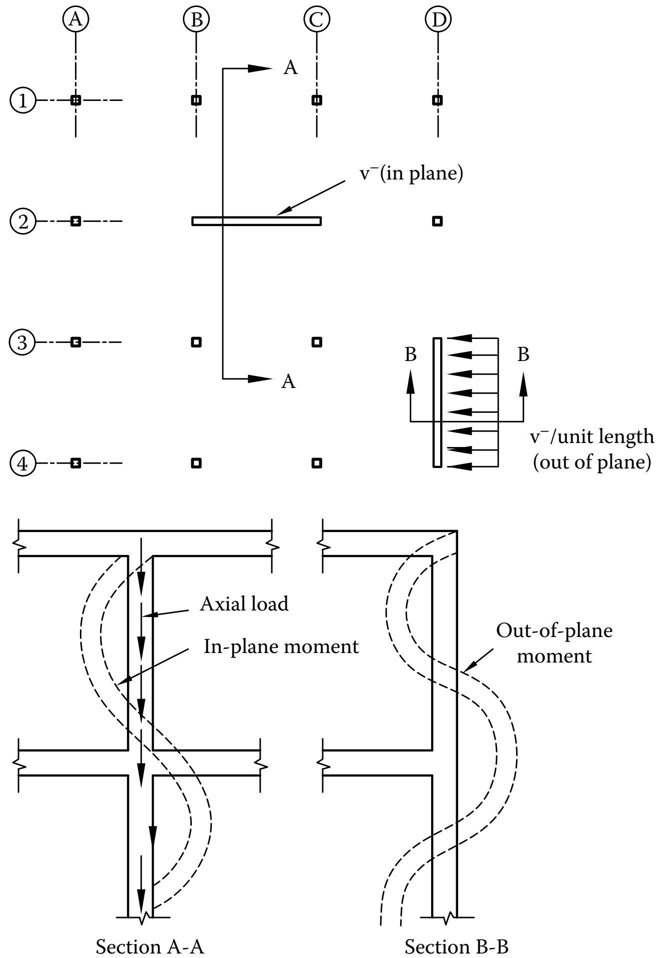


FIGURE 9.3 Wall system.

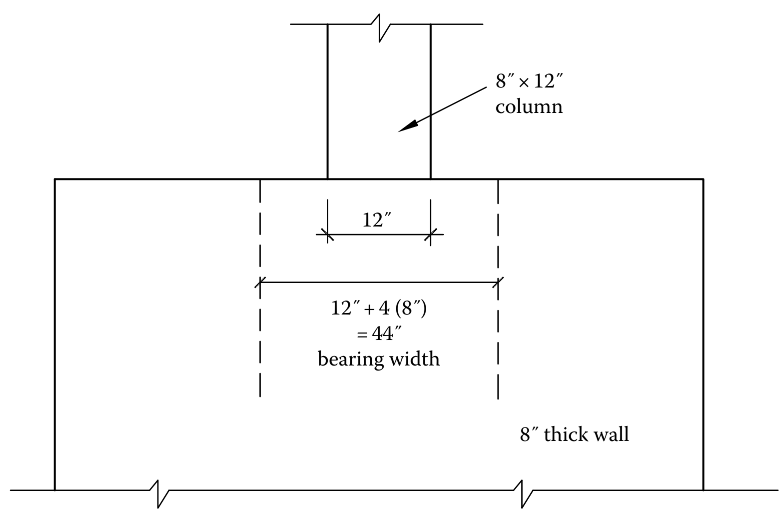


FIGURE 9.4 Concentrated load on walls.

9.5.1 Design Strength

The design strength of the columns shall be verified as follows:

$$\Phi P_n \geq P_u$$
$$\Phi M_n \geq M_u$$
$$\Phi V_n \geq V_u$$

where

P_n is the nominal axial strength (Section 4.3 of this book)
M_n is the nominal flexural strength (Section 4.2 of this book)
V_n is the nominal shear strength (Section 4.4 of this book)
Φ is the strength reduction factor (Section 5.14 of this book)

Discussions regarding the calculations of P_n and M_n are provided in Chapter 1 of this book.

9.5.2 In-Plane Shear

The nominal in-plane shear strength of the wall shall not exceed $10\sqrt{f_c'}\,hd$, where 'h' is the thickness of the wall and 'd' is 0.8 times the length of the wall (l_w). If 'd' is calculated using strain compatibility, a larger value of 'd' equal to the distance from the extreme compression fiber to the center of the force of all reinforcement in tension can be taken (Section 11.5.4 of the code).

$$V_n = V_c + V_s \qquad \text{(ACI Equation 11.5.4.4)}$$

V_c (for walls subject to axial load) shall not exceed $2\lambda\sqrt{f_c'}\,hd$ unless table 11.5.4.6 of the code is used to calculate Vc.

$$V_s = \frac{A_v f_{yt} d}{s} \qquad \text{(ACI Equation 11.5.4.8)}$$

where

V_s is the nominal shear strength provided by shear reinforcement (lb)
A_v is the area of shear reinforcement within spacing "s" (in.)
f_{yt} is the specified yield strength of the shear reinforcement (psi)

The nominal strength of the out-of-plane shear is discussed in Section 4.4 of this book.

9.5.3 Reinforcement Limits

Section 11.6 of the code specifies limits for both minimum longitudinal (ρ_l) and transverse (ρ_t) reinforcement for walls.

If $V_u \leq 0.5\phi V_c$, then $\rho_l = 0.0012$ and $\rho_t 0.0020$ for #5 bars or less and $\rho_l = 0.0015$ and $\rho_t 0.0025$ for bars greater than #5.

$$\text{If } V_u > 0.5\Phi V_c, \quad \text{then } \rho_l \geq 0.0025 + 0.5\left(2.5 - h_w/l_w\right)\left(\rho_t - 0.0025\right) \text{ and } \rho_t \geq 0.0025.$$

9.5.4 Reinforcement Detailing

The concrete cover (Section 5.10 of this book), development lengths (Section 11.4 of this book), and the splice lengths (Section 11.5 of this book) of the reinforcing bars have been discussed in the book.

According to section 11.7 of the code, the spacing of the longitudinal reinforcing bars shall not exceed the lesser of three times the thickness of the wall (h) and 18 in^2 for exterior walls and 30 in^2 for the interior walls. If shear reinforcement is required, then the spacing of the longitudinal reinforcing bars shall not exceed the smallest of three times the thickness of the wall, 18 in^2, and one-third of the length of the wall (l_w). Except for basement and cantilever retaining walls, for walls with thickness (h) greater than 10 in^2, there shall be two layers of distributed reinforcement for each direction. One layer shall consist of at least one-half and not exceeding two-thirds of total reinforcement required for each direction placed at least 2 in^2 but not exceeding one-third of the thickness of the wall (h) from the exterior surface. The other layer shall consist of the balance of the required reinforcement placed at least ¾ in^2 but not exceeding one-third of the thickness of the wall (h) from the interior surface. If longitudinal reinforcement is required for axial strength of the wall and the area of the longitudinal reinforcement (A_{st}) exceeds 1% of the gross cross-sectional area of the wall, then the longitudinal reinforcement must be laterally supported by the transverse reinforcement.

The spacing of transverse reinforcing bars in reinforced concrete walls shall not exceed the lesser of three times the thickness of the wall (h) and 18 in^2. If shear reinforcement is required, then the spacing of the transverse reinforcing bars shall not exceed one-fifth of the length of the wall (l_w).

In addition to the minimum reinforcement for the walls (discussed above), at least two #5 bars in walls having two layers of reinforcement and one #5 bar in wall having one layer of reinforcement shall be provided around openings such as doors and windows in the wall.

9.6 ALTERNATIVE METHOD FOR OUT-OF-PLANE SLENDER WALL ANALYSIS

According to section 11.8 of the code, when the following conditions are met, this method can be used to analyze out-of-plane slender walls:

a. Cross section of wall is constant over the height.
b. Wall is tension controlled for out-of-plane moment effect.
c. ϕM_n is at least equal to M_{cr}. The calculations of M_{cr} based upon f_r are discussed in Section 6.2.4 of this book.
d. P_u at the mid-height does not exceed $0.06f_c'A_g$.
e. Calculated out-of-plane deflection due to service load (Δ_s), including P-Δ effects, does not exceed $l_c/150$, where l_c is length of the wall, centerline to center of the joints.

The wall is modeled as an axially loaded simply supported wall subject to an out-of-plane uniformly distributed lateral load, with maximum bending moment and deflections occurring at mid-height. When concentrated gravity loads are applied to the wall, they shall be distributed over a width of bearing width plus a width on each side that increases at a slope of two vertical to one horizontal. However, the width shall not extend beyond the adjacent concentrated load or edge of the wall panel.

The bending moment at the mid-height is evaluated by one of the following two methods:

$$\text{a.}\quad M_u = M_{ua} + P_u\Delta_u \qquad \text{(ACI Equation 11.8.3.1a)}$$

where M_{ua} is the maximum factored moment at the mid-height of wall due to lateral and eccentric vertical loads, not including the PΔ effects.

$$\Delta_u = \frac{5M_u l_c^2}{(0.75)48E_cI_{cr}} \qquad \text{(ACI Equation 11.8.3.1b)}$$

where

$$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_u}{f_y}\frac{h}{2d}\right)(d-c)^2 + \frac{l_w c^3}{3} \qquad \text{(ACI Equation 11.8.3.1c)}$$

E_s/E_c shall be at least 6.

$$\text{b.}\quad M_u = \frac{M_{ua}}{\left(1 - \frac{5P_u l_c^2}{(0.75)48E_cI_{cr}}\right)} \qquad \text{(ACI Equation 11.8.3.1d)}$$

9.6.1 Out-of-Plane Service Loads Deflections

The out-of-plane service load deflections are calculated in accordance with table 11.8.4.1 of the code, as follows:

$$\text{If } M_a \le \left(\frac{2}{3}\right)M_{cr},\quad \text{then } \Delta_s = \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr}$$

$$\text{If } M_a > \left(\frac{2}{3}\right)M_{cr},\quad \text{then } \Delta_s = \left(\frac{2}{3}\right)\Delta_{cr} + \frac{\left(M_a - (2/3)M_{cr}\right)}{\left(M_n - (2/3)M_{cr}\right)}\left(\Delta_n - \left(\frac{2}{3}\right)\Delta_{cr}\right)$$

The maximum moment (M_a) at mid-height of the wall due to service lateral and eccentric vertical loads including the $P_s\Delta_s$ effects is

$$M_a = M_{sa} + P_s\Delta_s \quad \text{(ACI Equation 11.8.4.2)}$$

where M_{sa} is the maximum moment in walls due to service load excluding the $P_s\Delta_s$ effects.

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} \quad \text{(ACI Equation 11.8.4.3a)}$$

$$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}} \quad \text{(ACI Equation 11.8.4.3b)}$$

9.7 ASSIGNMENTS

1. Design and detail the parapet wall shown in Figure 9.5. The thickness of the parapet wall is 8 in². Use $f_c' = 4{,}000$ psi and $f_y = 60{,}000$ psi.
2. Design and detail the retaining wall shown in Figure 9.6. The thickness of the retaining wall is 10 in². The wall is also supporting a service live load of 500 lb/feet and a service dead load of 2000 lb/feet from the slab. Use $f_c' = 4{,}000$ psi and $f_y = 60{,}000$ psi. Use a soil density of 120 pcf.
3. Design and detail the 10′ high load-bearing concrete wall shown in Figure 9.7. The wall is 12 in² thick and supports steel columns A, B, C, D, and E with loads (in lbs) shown in the table below. Use $f_c' = 5{,}000$ psi and $f_y = 60{,}000$ psi.

Column	Dead Load	Live Load
A	60,000	23,000
B	85,000	42,000
C	85,000	42,000
D	85,000	42,000
E	60,000	23,000

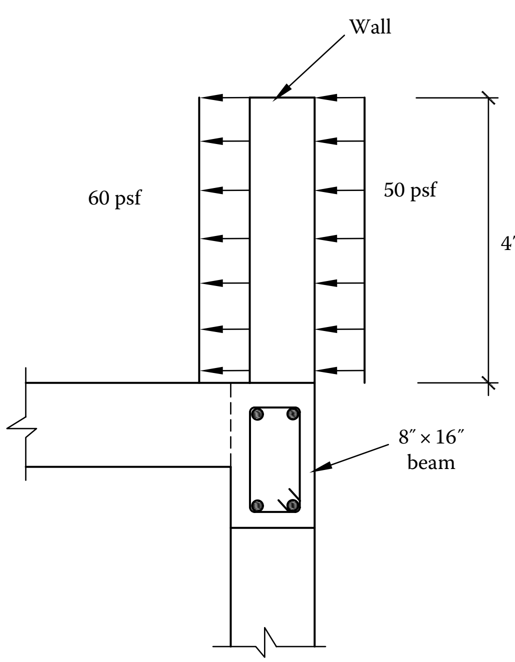


FIGURE 9.5

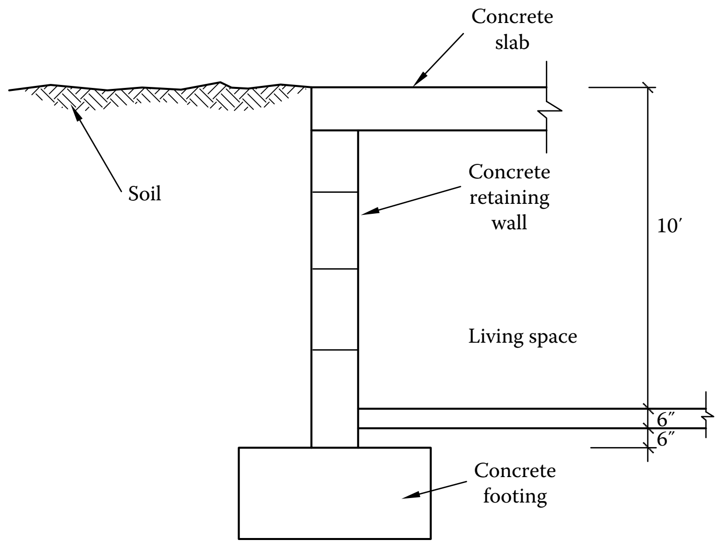


FIGURE 9.6

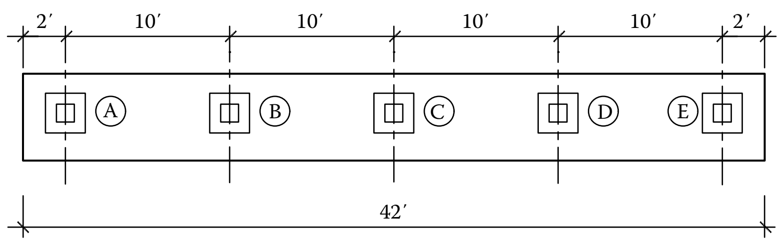


FIGURE 9.7

4. Design and detail a 15 feet high, 22 feet long, and 10 in^2 thick concrete wall restrained by slabs at the top and bottom. Apart from its self-weight, the wall supports a uniform dead load of 2500 plf and a uniform live load of 1800 plf. Use $f_c' = 5{,}000$ psi and $f_y = 60{,}000$ psi.
5. The shear wall at the ground floor of the building shown in figure 1.7 is 10 feet high and 8 in^2 thick. Apart from its self-weight, the wall supports a uniform dead load of 1200 plf and a uniform live load of 800 plf. The shear at the top and bottom of the wall due to wind are 350,000 lb and 380,000 lb, respectively. The moment at base due to wind is 300,000 lb-feet. Design and detail the shear wall using $f_c' = 7{,}000$ psi and $f_y = 60{,}000$ psi.

10 Foundations

10.1 INTRODUCTION

The main function of foundations is to transmit the load of the superstructure applied on them through columns or load-bearing walls to the ground. When loads of structures are transferred to the ground through the foundations, the ground gets compressed and settles. Either the entire ground enclosed by the periphery of the structure settles (called the total settlement) or there is partial settlement of the ground (called the differential settlement). The total settlement should be minimal and not noticeable. The partial settlement should be avoided because it causes damage to the structure.

The type of foundation (footing) selected depends upon various factors. Some of them are as follows:

a. Type of load (whether uniform, concentrated, or dynamic)
b. Magnitude of load
c. Type of soil
d. Availability of the construction equipment for foundations such as pile footings

If the loads are of low magnitude, then for columns, individual pad footings are adequate. For walls carrying uniform loads, continuous wall footings are adequate. If the loads on the walls are high and the soil is weak, then walls are supported on grade beams that are supported on piles. The selection of the piles depends upon the type of piles and the coefficient of friction of the soil. The end bearing of the pile contributes considerably little to the pile capacity.

If the soil is good, meaning that it has a high bearing capacity, then individual pad foundations can be used to support columns. This is economical till about a 10-storied building. Above that, a pile cap foundation supported on three or more piles is used.

If two or more columns are closely spaced and their footings overlap, then a combined footing is used. For high loads and weak soils, a mat footing, spreading over the entire footprint of the building is used to support all columns and walls.

10.2 GEOTECHNICAL REPORT

The geotechnical report is prepared by a specialty geotechnical engineer after performing the soil boring test. Typically in the report, the geotechnical engineer provides recommendation of the foundation, which includes the safe bearing capacity of soil, the depth of the soil where the safe bearing capacity can be obtained, or the use of other types of deep foundations. The geotechnical engineer may also provide the treatment of soil to obtain the specified safe bearing capacity of soil. When piles are recommended, then the type and dimension of the piles, length to which the pile needs to be driven in the soil, and the compressive, tensile, and lateral capacity of the piles are specified.

10.3 SLAB ON GRADE

If the soil is so weak that it cannot support a slab on grade, then suspended slabs are specified by the geotechnical engineers. These slabs could be designed supported on grade beams or designed as flat plates supported on the pile caps. These slabs are treated as elevated slabs, as discussed in Chapter 6 of the book. Slab on grade are ground slabs fully supported on the soil. Sometimes the geotechnical engineer specifies compaction of soil before casting the concrete of the slab on grade. The slab on grade is reinforced with very light reinforcement such as welded wire fabrics to withstand stresses due to temperature and shrinkage, and avoid cracking. Control joints are provided in slabs on grade to account for expansion and contraction.

10.4 PAD FOOTINGS

Individual pad footings are designed to resist concentrated loads from the columns. The pads are sized, based upon the service loads and the safe bearing capacity of the soil (Figure 10.1).

If the service load from the column at its base is P, then area of the footing (A) required is P/SBC, where SBC is the safe bearing capacity of the soil. Then the footing is sized and the weight of the footing (W) is added to the service load from

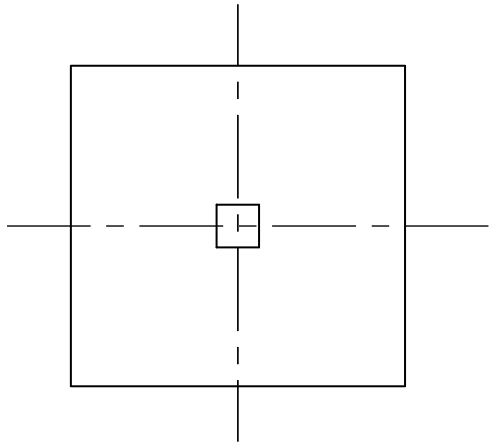


FIGURE 10.1 Pad footing.

column (P). In order to calculate the weight of the footing, the thickness of the footing is assumed. Then a second iteration is performed.

Safe bearing capacity of soil	SBC
Service load from the column	P
Weight of the footing (from first iteration)	W
Total load on the soil	P + W
Area of the footing	A = (P + W)/SBC

Then the net upward pressure is calculated based upon the total factored load. The footing is checked for moment and shear created by the net upward pressure and the punching shear created by the column load on the footing.

Mostly, individual pad footings for a single column are designed as square footings, but sometimes due to restrictions at the construction site or to avoid overlap with an adjacent footing, rectangular footings are also designed. The procedure for detailing the reinforcement of a rectangular isolated footing is discussed later in this chapter.

10.5 CONTINUOUS FOOTINGS

Continuous wall footings are usually provided for load-bearing walls. The service load at the base of the wall is calculated per unit length. This load is divided by the safe bearing capacity of the soil to obtain the width of the footing. Then the weight of the footing per unit length is added to the service load at the base of the wall. In order to calculate the weight of the footing, the thickness of the footing is assumed. The total load is divided by the safe bearing capacity of the soil to obtain the width of the footing in the second iteration, which is acceptable. Then the total factored load (load at the base of the wall plus the weight of the footing) is calculated. It is divided by the width of the footing to obtain the net upward pressure. The footing is designed for moment and shear due to the net upward pressure (Figure 10.2).

Continuous footings can also be used for lightly loaded columns. In this case, the width (b) of the footing should be checked in the following way.

A 45° angle be subtended from the edges of the columns parallel to the width of the footing till the base of the footing. The distance between these two points is calculated as ‘l.’ Calculate the weight of the footing of the region of the continuous footing

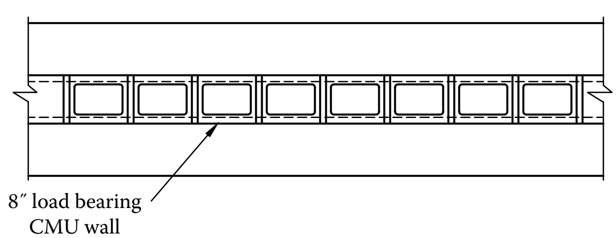


FIGURE 10.2 Wall footing.

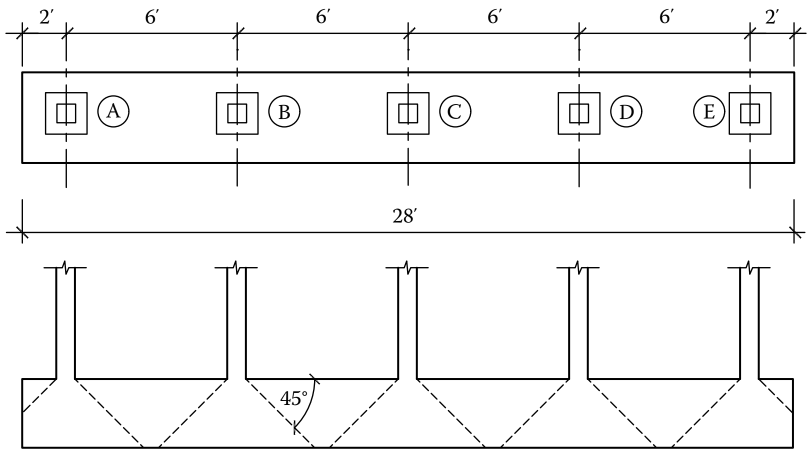


FIGURE 10.3 Continuous foot supporting columns.

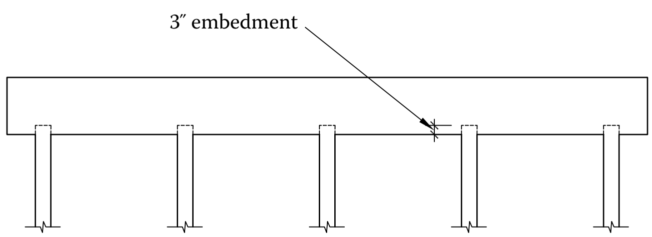


FIGURE 10.4 Grade beams on supported piles.

of length ‘l’ and width ‘b,’ after assuming the thickness of the footing. The total load on the soil (load from the column plus the weight of the portion of the footing) is divided by width (b) X length (l) to check that the safe bearing capacity is not exceeded. The footing is checked for the longitudinal and transverse moments, shear, and punching shear, as demonstrated in Figure 10.3.

Continuous footing could also be in the form of grade beams, when the safe bearing capacity of the soil is low. Grade beams are supported on piles and designed as continuous beams (Figure 10.4).

10.6 COMBINED FOOTINGS

If individual pad footings for two or more adjacent columns overlap, then combined footing is designed. The same procedure, as discussed for pad and continuous footings, is adopted to determine the size of the footing. The footing is designed for moments, shear, and punching shear. Complications arise when columns are not distributed evenly on the footing, while establishing the equilibrium of the footing. This is demonstrated in Figure 10.5.

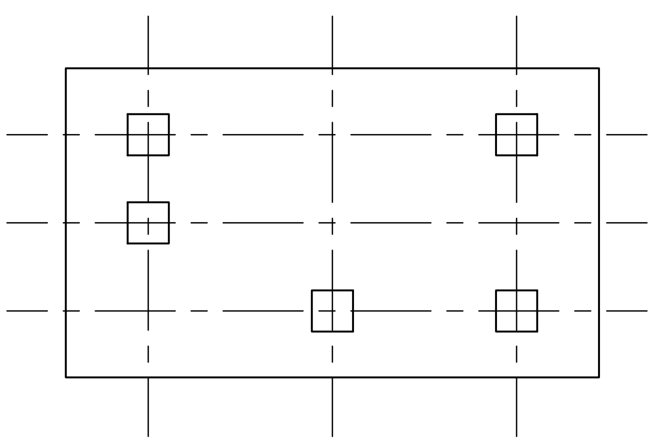


FIGURE 10.5 Combined footing.

10.7 STRAP FOOTINGS

When an individual pad footing is eccentrically loaded, the pressure on the soil is checked using the following equation:

$$f_{max,min} = \frac{P}{A} \pm \frac{M}{s}$$

In this equation,

$$f_{max} = \frac{P}{A} + \frac{M}{s} \quad \text{and} \quad f_{min} = \frac{P}{A} - \frac{M}{s}$$

The value of f_{max} shall not exceed the safe bearing capacity of soil and f_{min} should be a positive number not exceeding the safe bearing capacity of soil. If f_{min} is a negative number, it implies that the soil is undergoing tension. The footing needs to be strapped in the direction of the eccentricity to another footing capable of pushing down the uplift caused due to the moment created by the eccentricity of the load (Figure 10.6). If the footing is loaded eccentrically in both directions, then a diagonal strap beam is used (Figure 10.7). If there is no footing adjacent to the eccentric footing to connect the strap, then a mass of concrete is poured to strap the beam (Figure 10.8).

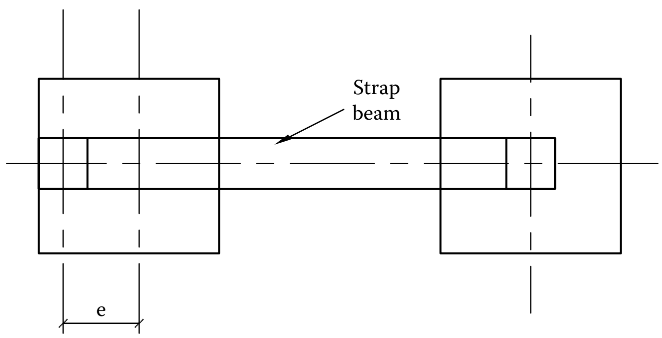


FIGURE 10.6 Strapped footing.

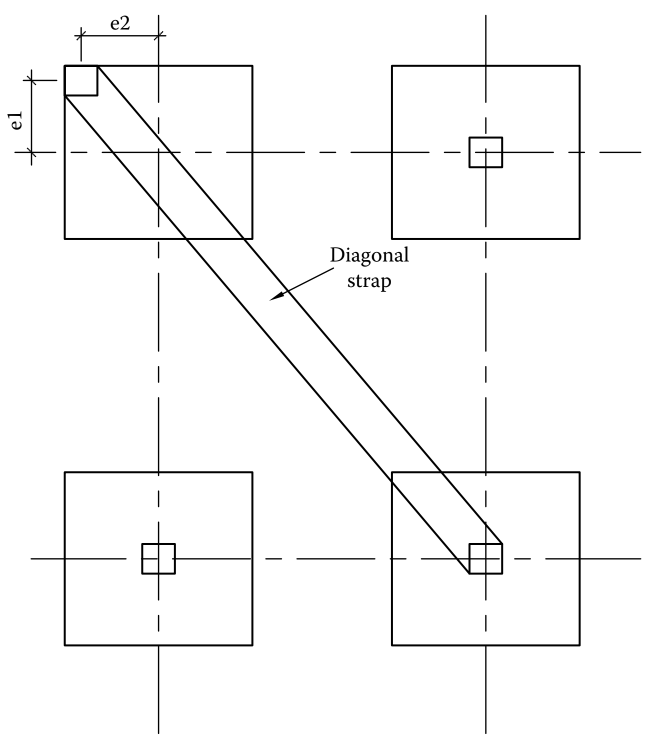


FIGURE 10.7 Diagonally strapped footing.

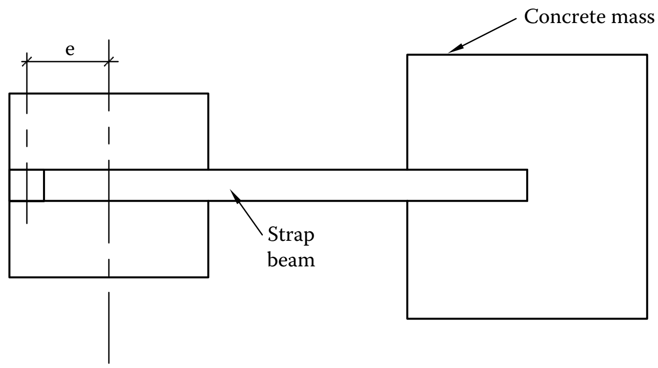


FIGURE 10.8 Eccentric footing strapped to mass of concrete.

Consider an eccentrically loaded footing with an eccentricity 'e.' The factored load of the column is P_u. Then the moment created due to eccentricity is $P_u e$. If a strap beam of span 'l' is used to strap the footing to another footing or a mass of concrete, then it should be capable of withstanding an uplift of $P_u e/l$. If a mass of concrete is used to resist this uplift, then it should be dimensioned to have a volume of $P_u e/(1.4)(150)(l)$, where 1.4 is the dead load combination factor and 150 is the weight of concrete (pcf). Here the unit for P_u is lbs and the unit for eccentricity is feet. The strap is designed for a moment equal to $P_u e$.

10.8 PILE CAPS

Pile caps are supported on piles and carry the load from columns or walls and transfer them to the piles (Figure 10.9). Piles are used when the magnitude of the load of the columns and walls is high and the soil bearing capacity is low. Piles are driven to a firm stratum of soil. The capacity of the piles is calculated based upon the soil friction acting on the surface area of the pile and the end bearing. Typically, piles support columns and walls of high-rise buildings. As discussed in the chapters on columns and walls, a moment accompanies the axial load. Hence, if in the interaction equation to calculate the stresses ($f_{max,\,min} = (P/A) \pm M/s$), if f_{min} is a negative number, piles are subject to uplift and are designed as tension piles.

Loads on pile caps are applied at the column and pile cap interface. The pile cap is assumed to be rigid. When there is no moment or eccentricity of load, then each pile is assumed to support equal load.

$$P_p = \frac{P}{n}$$

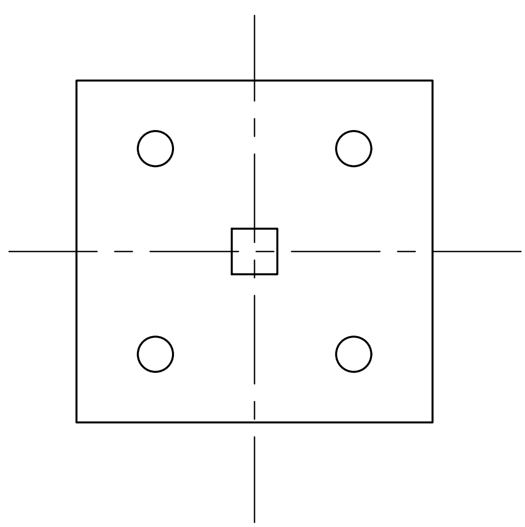


FIGURE 10.9 Pile cap.

When the pile cap is eccentrically loaded or subject to moment along one or both axes, then the load shared by the pile depends on its location.

$$P_p = \frac{P}{n} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2}$$

where

P_p is the load on pile under consideration
P is the total vertical load from column or wall
M_x, M_y is the bending moment with respect to X- and Y-axes
x, y is the distance from 'Y' and 'X' axes to the piles
n is the number of piles

10.9 MAT FOUNDATIONS

Mat foundations are considered when loads acting on the foundations are very high and the soil is weak. Individual pad footings become exceptionally large. If their area is more than one-third of the area of the building footprint, then mat foundation is a better solution. If the soil is prone to differential settlements, then flexural strength of the mat foundation will absorb these irregularities to a large extent and resist the differential settlement to a good degree. However, if the loads are very high, then portions of the mat, or the entire mat, need to be supported on piles.

The other advantages of mat foundation over individual pad footings are as follows:

1. If the lateral loads are not uniformly distributed over the columns supported on individual pad footings, then the stresses on the foundations will be different leading toward differential settlements. Since mat foundations provide one common base for all columns and walls, the differential settlement due to the lateral loads can be averted.
2. If uplift loads on certain individual pad footings are high, mat foundation is a good solution because of the large downward load available to resist the uplift.
3. If the bottom elevation of the foundation is below the groundwater table, water proofing of the footings is required. Since mat is one large foundation, water proofing is easy.
4. The resistance of hydrostatic uplift by a mat footing is high.

Generally, mats are considered rigid so the bending moment acting on the mat does not affect the bearing pressure. The magnitude of the bearing pressure depends upon the loads acting and the weight of the mat. If the loads acting are only vertical loads, the distribution of the pressure is typically uniform, but if moments act along with the vertical loads, then the distribution varies linearly across the mat.

However, portions of mats under heavily loaded columns and walls will settle more than portions with small loads. Hence, the bearing pressures under the heavily loaded zones are high. The solution to this problem is either increasing the stiffness of the mat or providing piles under these zones.

10.10 DESIGN REQUIREMENTS

The chapter 13 of the code provides design specifications for footings.

The critical section for the calculations of factored moments and shears on the footings, in accordance with table 13.2.7.1 of the code, is as follows:

1. Column or pedestal—at the face of the column or pedestal
2. Column with steel base plate—halfway between the face of the column and the edge of the steel base plate
3. Concrete wall—face of the wall
4. Masonry wall—halfway between the center and face of the wall

The design of one-way and two-way pad footings; combined footings and grade beams are related with the design of one-way slabs, two-way slabs and beams. The code provisions of slabs and beams are discussed in Chapters 6 and 7 of this book. The reinforcement of one-way and two-way square footings is uniformly distributed. For rectangular footings, the reinforcement is distributed uniformly in the long direction, but in the shorter direction, a portion of the total reinforcement ($\Upsilon_s A_s$) is distributed over a bandwidth of the short span of the footing, centered over the centerline of the column. The remainder of the reinforcement $[(1-\Upsilon_s)\,A_s]$ is distributed over the remaining width.

$$\Upsilon_s = \frac{2}{(\beta+1)} \qquad \text{(ACI Equation 13.3.3.3)}$$

where β is the ratio of the long side to the short side of the footing.

The effective depth of pad footings shall not be less than 6 in^2, and the effective depth of pile caps shall not be less than 6 in^2. Though pile caps are required to be designed with the code provisions used for one-way and two-way slabs (discussed earlier), section 13.4.2.5 of the code requires that:

a. Entire reaction of any pile with its center located at half the diameter ($d_{pile}/2$) or more outside the section shall be considered as producing shear on that section.
b. Reaction of any pile with its center located ($d_{pile}/2$) or more inside the section shall be considered as producing no shear on that section.
c. For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on a linear interpolation between full value at $d_{pile}/2$ outside that section and zero value at $d_{pile}/2$ inside the section.

10.11 ASSIGNMENTS

1. Design and detail a square footing supporting a column.

Dead load from the column	75,000 lbs
Live load from the column	40,000 lbs
Safe bearing capacity of the soil	3,000 psf
f'_c of concrete	3,000 psi
f_y of steel	60,000 psi

2. Design and detail a 12 feet long wall footing supporting a load bearing CMU wall.

Dead load from the wall	5,000 plf
Live load from the column	4,000 plf
Safe bearing capacity of the soil	2,000 psf
f'_c of concrete	3,000 psi
f_y of steel	60,000 psi

3. Design and detail the combined footing shown in Figure 10.10.

Dead load from each column	95,000 lbs
Live load from the column	60,000 lbs
Safe bearing capacity of the soil	3,000 psf
f'_c of concrete	4,000 psi
f_y of steel	60,000 psi

4. Design and detail the combined footing shown in Figure 10.11.

Dead load from each column	55,000 lbs
Live load from the column	30,000 lbs
Safe bearing capacity of the soil	3,000 psf
f'_c of concrete	4,000 psi
f_y of steel	60,000 psi

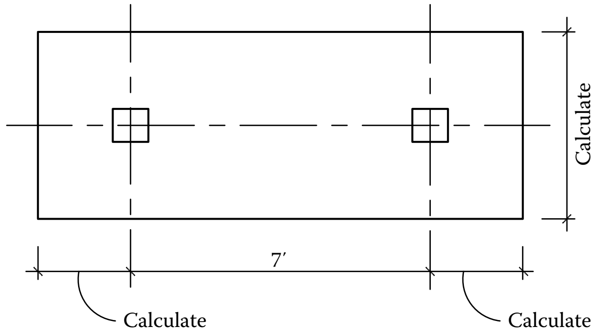


FIGURE 10.10

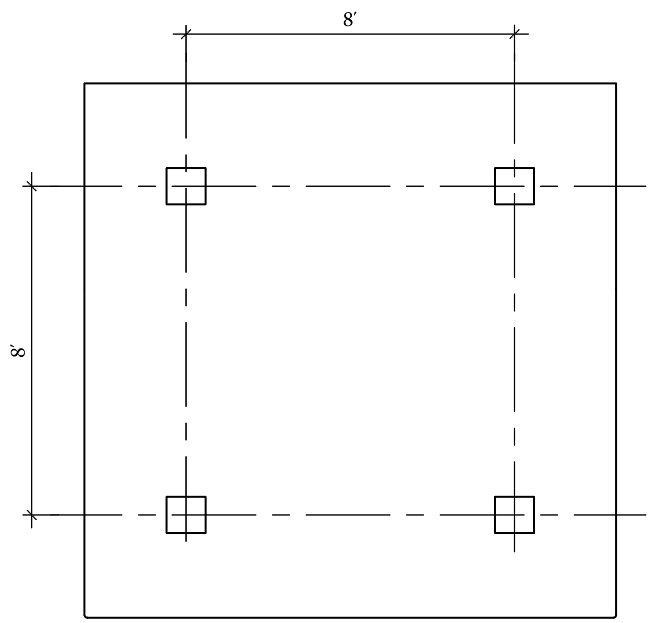


FIGURE 10.11

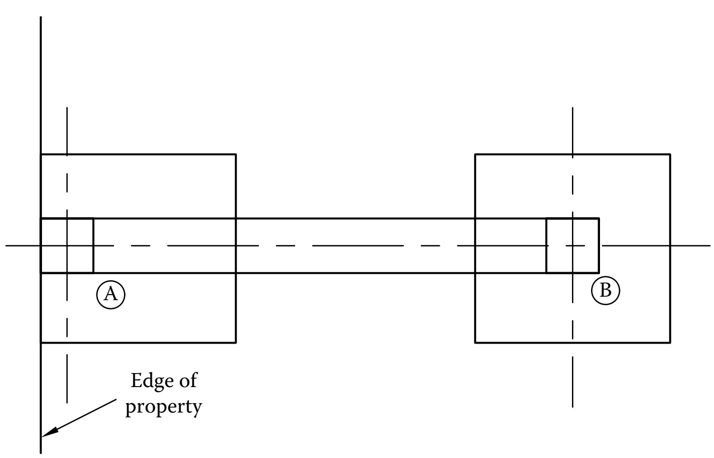


FIGURE 10.12 Strapped footing.

5. Design and details the two pad footings and the strap beam shown in Figure 10.12. The soil cannot be excavated beyond the edge of the property due to municipal restrictions. Check if column (B) has the capacity to resist the uplift created at its base by the strap beam.

Dead load from each column	64,000 lbs
Live load from each column	32,000 lbs
Safe bearing capacity of the soil	3,000 psf
f_c' of concrete	4,000 psi
f_y of steel	60,000 psi
Size of the columns	12 in^2 × 12 in^2

6. Size the footing for the 10 columns shown in Figure 10.13 based upon the unfactored. Draw the bending moment and shear force diagram along each axis.

Safe bearing capacity of the soil	3,000 psf
f_c' of concrete	4,000 psi
f_y of steel	60,000 psi

Column	Dead Load (K)	Live Load (K)
1	23	12
2	46	37
3	24	13
4	35	22
5	52	42
6	54	42
7	31	18
8	30	20
9	34	20
10	24	12

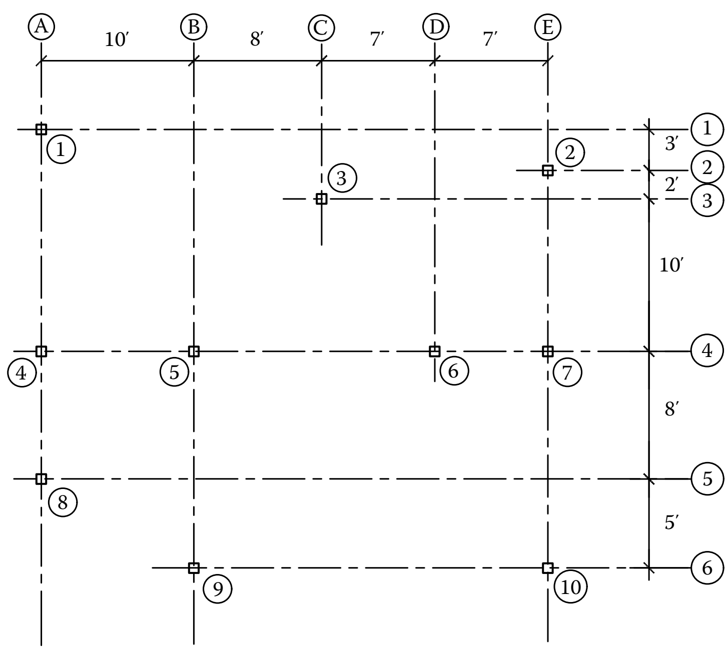


FIGURE 10.13

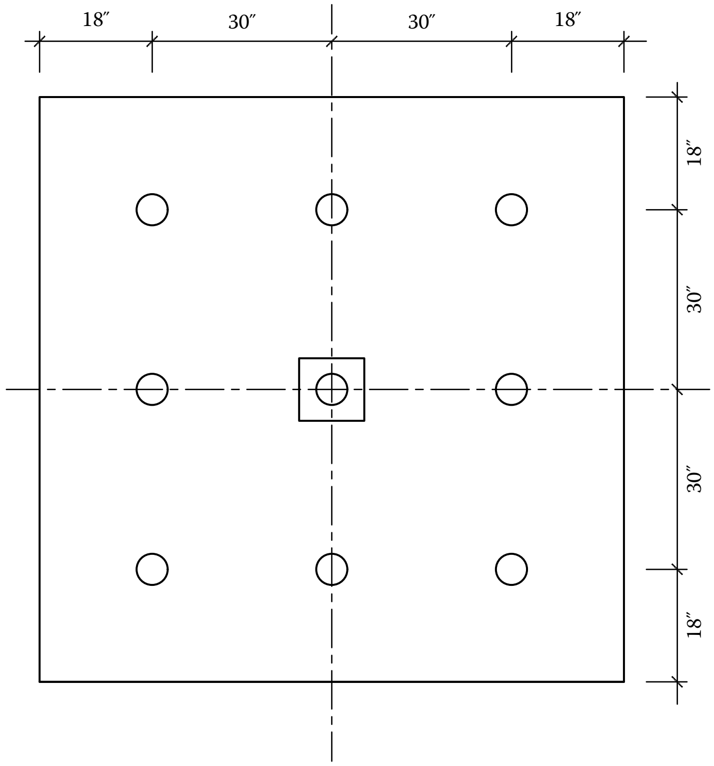


FIGURE 10.14

7. A pile cap supports a column carrying an axial dead load of 300 K and an axial live load of 250 K. The dead load moment acting at the base of the column is 145 K feet, and the live load moment is 120 K feet. Calculate the axial forces on each of the nine piles shown in Figure 10.14.

11 Reinforcement Details

11.1 INTRODUCTION

The job of a structural engineer does not finish with good analysis and sizing of the members. At the construction site, these documents are not even required. The detailing of the reinforced concrete members is the key to good design and execution of work at the site. One of the major items under detailing of the reinforced concrete members is the detailing of the reinforcing bars. Structural integrity of the reinforcement is required to enhance continuity and improve the redundancy and ductility of structures. Continuous reinforcement or tie between horizontal framing members provides this redundancy. Chapter 25 of the code provides the requirements of reinforcement detailing. By providing good reinforcement detailing, the engineer ensures that his/her analysis and design is accurately executed at the site. Bad detailing of reinforcement makes the structure undergo cracking, excessive deflection, or even collapse. Detailing of the reinforced concrete members assists the construction of the structure in accordance with the design intent and the design parameters. For example, if a structure is in a seismic zone, then appropriate reinforcement detailing would be required to resist those seismic forces. As discussed in the previous chapters, reinforcements resist tensile forces and they may also be required in the compression zones to increase the compression capacity, enhance ductility, reduce long-term deflections, or increase the flexural capacity for beams. They prevent cracking of concrete due to shrinkage and temperature stresses. Even a slab directly supported on well-compacted soil and not subject to any bending, shear, or torsion may require nominal reinforcement to prevent shrinkage and temperature cracks. Shear stirrups help resist the principal tensile stresses resulting from shear. Beam and column joints may have highly stressed compression zones and may require hoops. Bars in tension may not be fully effective unless they are developed. Bars need to be embedded in concrete to provide sufficient bonding on each side of the critical section in the form of embedment length, hooks, mechanical anchorage devices, headed deformed reinforcement, or a combination of these methods. As discussed in Section 5.7 of this book, small detailing could help reduce the effect of some high abnormal loading not considered in the design.

11.2 MINIMUM SPACING OF REINFORCEMENTS

Section 25.2 of the code specifies the requirement of minimum spacing of reinforcement. For parallel bars in beams and slabs, the clear spacing shall be at least the greatest of 1 in., bar diameter (d_b), and 4/3 times the size of the aggregate (d_{agg}). If multiple layers of bars are provided in a beam or a slab, the bars of the upper layers shall be directly above the bars of the lower layer and a spacing of 1 in. shall be maintained between them (Figure 11.1).

The clear spacing between the longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls shall be at least the greatest of 1.5 in^2, 1.5 times the bar diameter (d_b), and 4/3 times the size of the aggregate + one bar diameter (4/3 $d_{agg} + d_b$) (Figure 11.2).

A minimum spacing of reinforcing bar is required to allow concrete to flow freely between and around the bars. According to ACI, workability of concrete is the property of freshly mixed concrete that determines the ease and homogeneity with which it can be mixed, placed, consolidated, and finished. ASTM defines it as the property of concrete determining the effort required to manipulate a freshly mixed quantity of concrete with minimum loss of homogeneity. Good workability of concrete reduces segregation, maintains homogeneity, and makes it easily compactible and finishable. If workability of concrete is limited, it leads to honeycombs, which are rough, pitted surface or voids in concrete resulting from incomplete filling of concrete against the formwork. Though the causes of honeycomb include stiff concrete, over-vibration, improper placement of concrete, and addition of water beyond the designed water–cement ratio, providing a minimum spacing between the reinforcement between reinforcing bars helps reduce the formation of honeycombs.

11.3 STANDARD HOOKS

Hooking of bars in reinforced concrete structural elements is done to facilitate the anchorage of a member to the supporting member. Bars can be hooked at 90°, 180°, or 135°. The code specifies the diameter of the bend and the extension of the bar after the bend. The design of the hook is code-specific and the code does not require any calculations. However, Osamn and

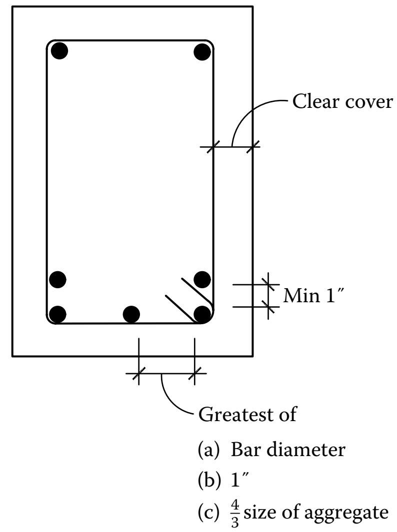


FIGURE 11.1 Minimum spacing of bar in beam.

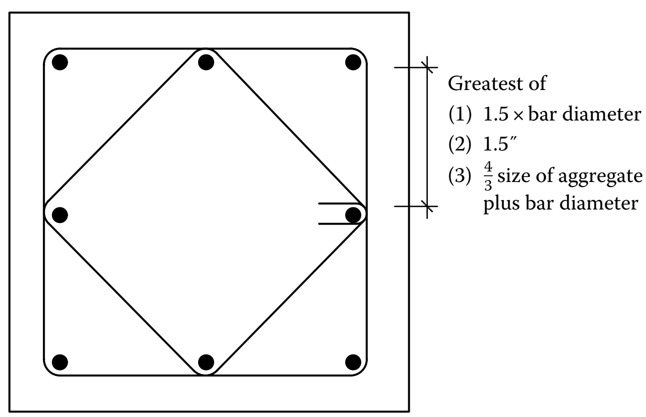


FIGURE 11.2 Minimum spacing of bar in column.

Yasuki conclude that the hooked bar should be checked for side split failure, local compression failure, and raking out failure (Figure 11.3). When a sufficient development length cannot be provided, then the bar is anchored. Hooking is a type of anchoring of the bar. Three parameters are used to define hooks and are a function of the size of the bar.

1. l_{dh}—development length of the hook
2. l_{ext}—straight extension of the hook
3. Inside bend diameter

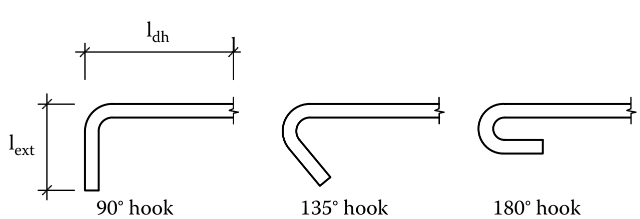


FIGURE 11.3 Hooked bars.

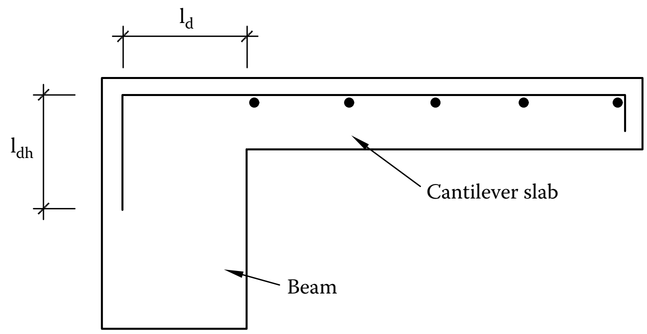


FIGURE 11.4 Development of bar in tension.

The minimum inside diameter and straight extension for the bars in tension are specified in table 25.3.1 of the code. The development length of the hooked bar (l_{dh}) shall be the greatest of

a. 6 in^2
b. 8 times the bar diameter ($8d_b$)
c. $\left(\dfrac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f_c'}}\right)$

The modification factors ψ_c, ψ_e, and ψ_r are specified in table 25.4.3.2 of the code. The development length (l_{dh}) can be reduced with a factor (A_s, required/A_s, provided), if excessive reinforcement than required by the design is provided (Figure 11.4).

11.4 DEVELOPMENT LENGTH

Developing the proper length of concrete-embedded reinforcing bar is crucial for obtaining its full tensile capacity. If the distance is less than the defined development length, the bar will pull out of the concrete. The reinforcing bar bonds with the concrete over the development length. The bond between the reinforcement and the concrete is influenced by various factors: diameter and spacing of the bar, cover to the bar, transverse reinforcement, grade and confinement of the concrete around the bars, aggregate used in the concrete, and coating applied to the bar to resist corrosion. The reinforcing bars that are highly stressed tend to split the concrete to which they are bonded. A length of bar needs to be developed beyond the point at which this stress in the concrete does not split the concrete. It is important to check the development length for cases like the following:

- Flexural members that have relatively short length to make sure that bars are fully developed.
- At simple supports and at points of infliction (contraflexure where moment changes sign). These are locations where bars can be curtailed as they may not be required.
- At points of bar cutoff.
- At beam–column joints in lateral load resisting system (for wind and earthquake).
- At cantilever supports.
- For stirrups and transfer ties.
- At lap splices.

Section 25.4 of the code addresses the development length. It discusses the following cases:

1. Development of deformed bars and deformed wires in tension
2. Development of standard hooks in tension
3. Development of headed deformed bars in tension
4. Development of mechanically anchored deformed bars in tension
5. Development of welded deformed wire reinforcement in tension
6. Development of welded plain wire reinforcement in tension
7. Development of prestressed seven-wire strand in tension
8. Development of deformed bars and deformed wires in compression
9. Reduction of development for excess reinforcement

In this chapter, development of deformed bars in tension and compression, development of standard hooks in tension, and deduction of development for excess reinforcement are discussed.

11.4.1 Deformed Bars in Tension

If the clear spacing of the bars being developed is not less than the diameter of the bar (d_b), the clear cover to the bar is not less than the diameter of the bar (d_b), and there are shear stirrups designed according to the code; or if the clear spacing of the bars being developed is at least twice the diameter of the bar ($2d_b$) and the clear cover is at least equal to the diameter of the bar, then according to table 25.4.2.2 of the code:

$$l_d = \left(\frac{f_y \psi_t \psi_e}{25\lambda\sqrt{f_c'}}\right) d_b \quad \#6 \text{ bar or less}$$

$$= \left(\frac{f_y \psi_t \psi_e}{20\lambda\sqrt{f_c'}}\right) d_b \quad \text{for } \#7 \text{ bar or more}$$

For other cases,

$$l_d = \left(\frac{3 f_y \psi_t \psi_e}{50\lambda\sqrt{f_c'}}\right) d_b \quad \text{for } \#6 \text{ bar or less}$$

$$= \left(\frac{3 f_y \psi_t \psi_e}{40\lambda\sqrt{f_c'}}\right) d_b \quad \text{for } \#7 \text{ bar or more}$$

where
- λ is 1.0 for normal weight concrete
- ψ_e is 1.0 for uncoated bars
- ψ_t is 1.3 if 12 in^2 of concrete is placed below the bar
  or = 1.0 for all other conditions
- $\sqrt{f_c'} \leq 100$

The modification factors λ, ψ_c, ψ_t, and ψ_s are specified in table 25.4.2.4 of the code. However, in accordance with section 25.4.2.1 of the code, a minimum of 12 in^2 development length must be specified.

The development length is also calculated by using the ACI Equation 25.4.2.3a:

$$l_d = \left(\frac{3 f_y \, \psi_t \psi_e \psi_s}{4\lambda \sqrt{f_c'} \left(\frac{c_b + K_{tr}}{d_b}\right)}\right) d_b$$

where
- λ, ψ_t, ψ_e, and ψ_s are obtained from table 25.4.2.4 of the code
- d_b is the diameter of the bar
- c_b is lesser of (a) the distance between center of a bar to the nearest concrete surface and (b) the center-to-center spacing of bars or wires being developed (in.)
- $K_{tr} = \dfrac{40A_{tr}}{sn}$ (can be taken as "0" for simplification)
- A_{tr} is the total cross-sectional area of all transverse reinforcement within spacing (s) that crosses the potential plane of splitting through the reinforcement being developed (in^2)
- s is the center-to-center spacing of bars
- n is the number of bars

However, in either case the development length (l_d) shall not be less than 12 in^2.

11.4.2 Development of Standard Hooks in Tension

Standard hooks are discussed earlier in Section 11.3 of this book. Section 25.4.3 of the code has provisions for the design of the development length for standard hooks in tension. The development length (l_{dh}) for bars in tension terminating into a standard hook is the greatest of

a. $\left(\dfrac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f_c'}}\right) d_b$

b. $8d_b$

c. 6 in^2

The values of λ, ψ_e,ψ_c, and ψ_r are obtained from table 25.4.3.2 of the code.

If the bars have the two sides and top or bottom cover less than 2.5 in^2, then the hook shall be enclosed along l_{dh} within the ties, perpendicular to l_{dh} at $s \leq 3d_b$, the first tie should enclose the bent portion of the hook within $2d_b$ of the outside of the bent, and ψ_r shall be taken as 1.0 in the above calculations.

11.4.3 Deformed Bars in Compression

In accordance with section 25.4.9 of the code, the development length l_{dc} of the bar in compression shall be greater of

a. $\left(\dfrac{f_y \psi_r}{50\lambda\sqrt{f_c'}}\right) d_b$

b. $0.003 f_y \psi_r d_b$

c. 8 in^2

The values of λ and ψ_r are obtained from table 25.4.9.3 of the code.

11.4.4 Reduction of Development Length for Excess Reinforcement

In accordance with section 25.4.10 of the code, the development length discussed in the above three cases can be reduced by a factor ($A_{s,required}/A_{s,provided}$). However, the minimum length specified for each case (l_d = 12 in^2, l_{dh} = 6 in^2, and l_{dc} = 8 in^2) shall be maintained. Further, reduction in the development length is not permitted at noncontinuous supports, at locations where anchorage or development for f_y is required, and where bars are required to be continuous.

11.5 SPLICES

Splices are provided to reinforcement when the length of the bar is shorter than the span or a concrete pour is stopped short or if there is a change of structural element, such as a column supported on a footing. For example, if a combined length of a five-span continuous slab is 100 feet and if the bar length available is not more than 25 feet, the continuous bottom reinforcement will be spliced at three locations. For the bottom reinforcement of the slab, splicing near the midspan, where there is the maximum positive bending moment, is discouraged. Another example is the connection between the footing and the column. While placing the reinforcement of the footing, L-shaped dowels are placed in the footing. The length of the dowel above the top surface of the footing is equal to the splice length required for the column reinforcement. Splices are also provided on the top of the poured column for the column above. For a beam–column joint, the length of the bar above the top surface of the poured concrete of the column is equal to the depth of the beam plus the splice length of the column reinforcement. For a slab–column joint, the length of the bar above the top surface of the poured concrete of the column is equal to the thickness of the slab plus the splice length of the column reinforcement. Splices are specified in section 25.5 of the code.

Lap splices for bars larger than #11 bars are not permitted by the code and mechanical connections such as couplings are used. The spacing between the adjacent contact splices shall be in accordance with section 25.2.1 of the code, as discussed in Section 11.2 of this chapter. For noncontact splices in flexural members, the transverse center-to-center spacing of spliced bars shall not exceed the lesser of one-fifth the required lap splice length and 6 in^2. Reduction of development length, as discussed in the previous section of this chapter, is not permitted in the calculations of splice length.

Tension lap splice length (l_{st}) is determined in accordance with table 25.5.2.1 of the code. The factors included in the table for the calculations of the splice length (l_{st}) are the ratio of area of steel provided and area of steel required ($A_{s,required}/A_{s,provided}$), development length ($l_d$), and the percent of the area of steel spliced within the required lap length. If bars proposed to be spliced are of different diameters, then the splice length (l_{st}) shall be greater of the development length of the larger bar and the smaller bar.

Compression lap splice length (l_{sc}) of #11 or smaller bars for grade 60 steel is greater of $0.005f_yd_b$ and 12 in^2, according to section 25.5.5.1 of the code. But if the compressive strength of concrete is less than 3000 psi, then the length shall be increased by one-third. The code does not allow compression splices for bars larger than #11, unless the larger bars are grouped with smaller bars. End-bearing splices are permitted if they are contained in stirrups, ties, spirals, or hoops.

If mechanical or welded splices are used, they shall be developed at least 1.25 times the strength of the main bar to ensure sufficient strength in splices so that yielding can be achieved in a member and brittle failure is avoided.

Bars in structural elements are placed as singles, but sometimes groups of two or three or four bars are placed together, called bundled bars. They shall be enclosed within stirrups or ties to ensure that they remain together. Bundling of bars becomes essential when a large number of bars are required to be accommodated in a structural element. We have discussed the minimum spacing of bars in Section 11.2 of this chapter to allow the flow of concrete. If the reinforcement requirement of a structural element is high, by bundling bars the minimum spacing between the bars (between singles or bundles) is maintained and the size of the member is not increased. The provisions of bundled bars are discussed in section 25.6 of the code. Bundled bars in compression shall be enclosed in transverse bars, #4 or greater. Bars larger than #11 are not bundled. Bar cutoffs within the bundled bars shall be staggered at different points at least 40 times the diameter of bar. Development length of individual bars within a bundle shall be that of the individual bar, increased by 20% in a three-bar bundle and 33% in a four-bar bundle, and the lap splices are calculated based upon these values. A unit of bundled bar shall be treated as a single bar with an area equivalent to that of the bundle and a centroid coinciding with that of the bundle. This equivalent diameter is used for various parameters such as spacing between the bars or cover requirement.

Detailing of transverse reinforcement such as stirrups, ties, and spirals is discussed in section 25.7 of the code. The various requirements of detailing of the transverse reinforcement are discussed in the figures and the examples provided in Chapters 7 and 8 of this book (Figures 11.5 through 11.8).

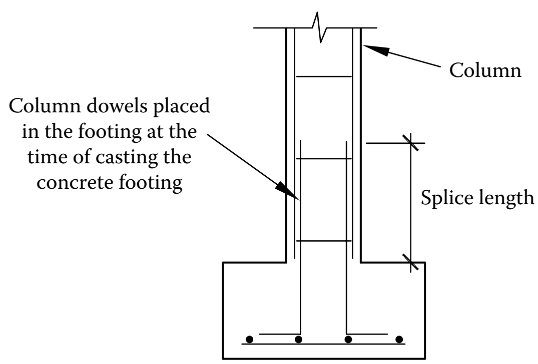


FIGURE 11.5 Splicing detail for columns.

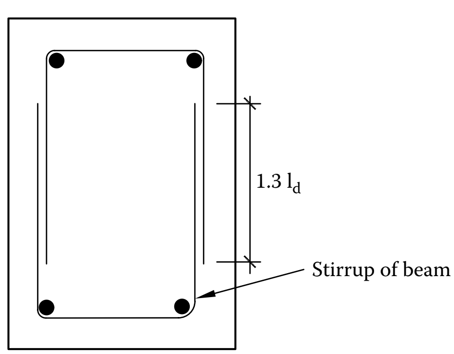


FIGURE 11.6 Splicing stirrups of beams.

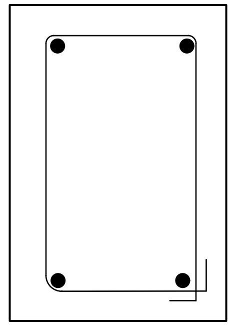


FIGURE 11.7 Hooked stirrups of beams.

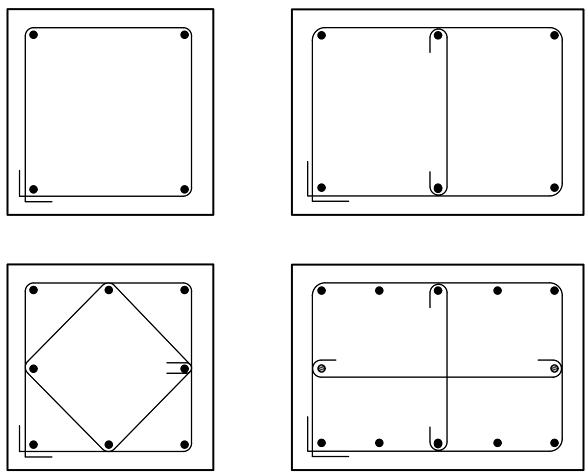


FIGURE 11.8 Column tie examples.

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12 Drawings, Inspections, and Testing

12.1 INTRODUCTION

An engineer's knowledge of structural engineering is finally transferred to the construction site, where the final product of that knowledge is seen. The complete process of building design is as follows:

1. Architect plans the building for utility and aesthetics.
2. Other disciplines involved are structural, civil, mechanical/electrical/plumbing (MEP), landscape architecture, interior designing, acoustics, and many other specialties based on the function of the building.
3. Architect coordinates with the professionals of various disciplines.
4. Structural engineer (SE) selects the structural system and prepares the layout of the columns and walls, provides initial sizes of the members, and coordinates with the architect and MEP engineers to finalize them.
5. Design load criteria and materials specifications are established.
6. Analysis is performed.
7. Members are sized.
8. Design is performed wherein the members are checked for adequacy to resist the applied forces.
9. Preliminary drawings are prepared.
10. Final coordination with architect and MEP engineers.
11. Drawings are finalized.
12. Drawings are issued for building permit to verify code compliance.
13. Comments from the Building Department, if any, are addressed.
14. Final construction drawings for building permits are issued.
15. Bidding process and response to request for information (RFI) from the contractors.
16. Contract is awarded.
17. Construction begins.
18. Inspections during construction for compliance with the drawings and applicable codes.
19. RFI during construction is responded to.
20. Testing performed during construction.
21. Construction ends.
22. Final inspections.
23. The SE issues a Certificate of Compliance for the contractor to obtain the Certificate of Occupancy from the Building Department.
24. Building is occupied.

During their practice, a SE should be well aware of these steps and their methodologies. However, three important steps for a SE are preparing a good set of structural working drawings, monitoring the construction with inspections, and reviewing of the test reports. This chapter deals with all these three criteria.

No matter how well engineers analyze and design a structure, good execution of work at site does not take place unless a good set of construction documents are prepared. Good construction documents enable ease and coordination of construction at site, reduce the number of RFIs (memos sent by the contractor or construction administrator requesting missing information on the drawings or requesting to resolve an erroneous information), and reduce change orders (an extra amount claimed by the contractor for the errors and omissions made by the engineer on the drawings), time delays, and litigation. Hence, the engineer must practice care during the preparation of the drawings, and finally the reputation of the engineer is at stake. With experience, engineers develop their own methodology of preparation of drawings. If a pattern is followed with a checklist, then most of the design requirements are covered and a final check of the drawings then enables a substantial completion of the drawings.

Typically, inspection of work at site is performed by the engineer's representative, who by qualification may be a licensed engineer, an engineering intern, or a general contractor with a building inspector's license, depending upon the law of the state. The Engineer of Record typically visits the construction sites for cursory inspections, to participate in important construction meetings, or if a technical issue needs to be resolved. The inspection plan must be clearly laid out for the inspector and as a guide to the contractors for their expectations. In the state of Florida, for threshold buildings (building that are more than three stories or 50 feet in height or 5000 feet^2 in area with an occupancy of 500 or more), a threshold inspection plan is required to obtain a building permit. The building is inspected by a threshold inspector according to the threshold inspection plan prepared by the Engineer of Record. During construction, testing of concrete such as the compression strength test and slump cone test is performed according to the requirements specified on the drawings or in the technical specifications manual.

12.2 PREPARATION OF STRUCTURAL DRAWINGS

The structural design process starts with the review of the architectural drawings. The SE meets with the architect and the MEP engineer. Based upon the architectural drawings, the SE establishes the load path of how the load from the roof traverses to the ground collecting the entire load in its path and proposes a structural system that is strong and adequate for the loads and economical and whose materials are readily available in the local market. Load path has been discussed in Chapter 1. During this process, the SE establishes the locations of the columns and walls and also tentative sizes of the structural members such as columns and beams. The locations and sizes of the columns are typically based upon the architectural planning requirements; the sizes of some of the interior beams are based upon the MEP requirements, and the sizes of the exterior beams are based upon the façade of the structure. During placement and sizing of the columns, it should be noted that columns do not hinder the open space requirements of the architectural plans. Sometimes, columns cannot be taken to a floor below and they are supported either by the slab or the beam at that level, which are called "transfer slabs" and "transfer beams." This situation occurs in cases like parking spaces below a floor. The sizes of the interior beams sometimes get restricted by the air-conditioning ducts that need to pass below the beams, requiring they be made shallower. Beams may also be required to account for embedment of plumbing pipes and electrical conduits. There might be restriction of beam depths because of the ceiling heights from the finished floor below. The sizes of the exterior beams are governed by the façade of the structure. Sometimes, the façade may require deeper drop beams and sometimes shallower beams. However, the SE ensures that the integrity of the structure is not compromised by these requirements.

Then, according to the utility of the building, the live load and other superimposed loads are determined from the ASCE 7-10 or the general building codes. The superimposed dead loads of the floors are calculated according to the locations of the partition walls and types of floor finishes. The weights of the air-conditioning, plumbing, and electrical systems are obtained from the MEP consultants and incorporated in the load estimates. Some residential buildings have rooftop swimming pools and landscaping and also landscaping on the balconies. These loads are incorporated in the analysis. Multistoried buildings are required to have elevators and their weights are accounted for.

Once all the member sizes are established and information about the loads is collected, the SE analyzes the structure. After the analysis process, the SE verifies the adequacy of the member sizes. If changes to the member sizes are required, then the SE discusses it with the architect and other consultants before finalizing the sizes of the members. The SE then performs the final analysis and the design, which has to comply with the prevailing codes (in our case the ACI 318-14) and the general building codes (in our case the IBC, 2015). Once the design process is completed, the SE prepares the drawings with the help of a draftsperson (commonly called CAD technician). In fact, the preparation of the drawings can begin along with the preliminary analysis. Drawings such as the floor plans and structural notes can be put together at the beginning of the analysis process.

The construction documents of reinforced concrete structures at a minimum include the following drawings:

1. Roof plan
2. All typical floor plans (there may be multiple typical floors according to the architectural requirements)
3. Ground floor plan
4. Foundation plan
5. Beams schedule
6. Typical details of beams
7. Typical details of slabs
8. Column schedules
9. Column details
10. Foundation schedules
11. Foundation details
12. Wall sections
13. Special details required to complete the drawings
14. Structural notes

The roof plan shows the layout of the roof slab, layout of the slab reinforcing bars (top and bottom), thickness of the slab, layout of the beams (if any), and the layout of the rooftop units (RTUs) for air conditioning. The RTUs need to be located and their loads need to specified on the roof plan because the slabs are designed according to these loads and their locations need to be firmed up. If there are several RTUs, then a separate roof plan needs to be provided in order to avoid too much information on one drawing. Some engineers prefer showing the roof slab reinforcing bars and beams marking on the separate drawings.

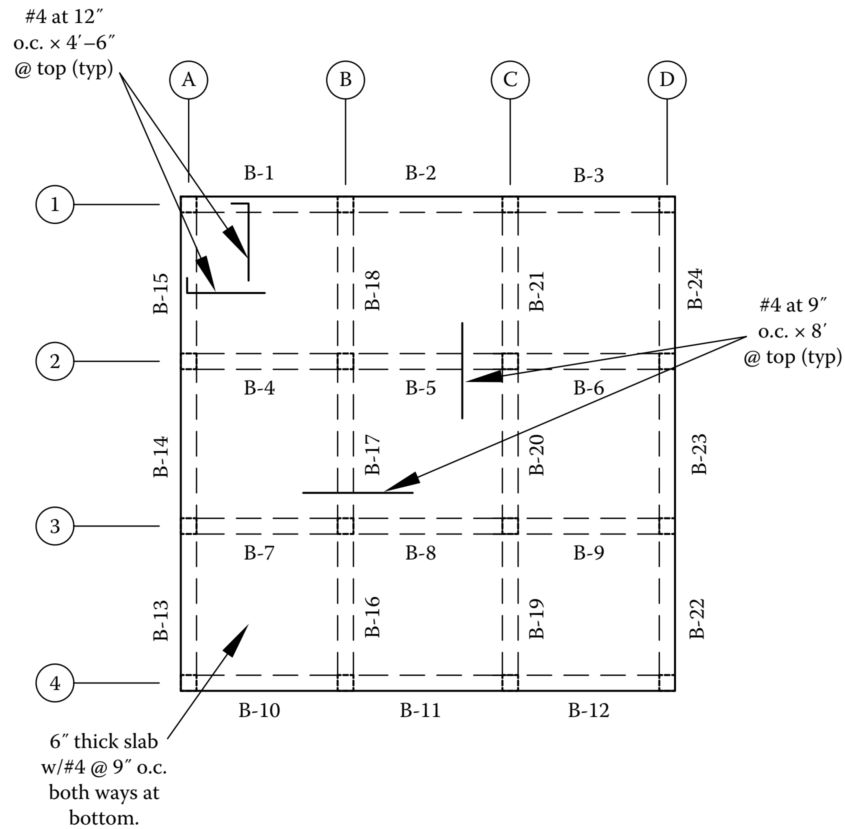

FIGURE 12.1 Typical floor plan.

They provide two drawings—one for the slab reinforcing bars and the other for the identification of the beams. Usually, the roof beams are assigned labels RB1, RB2, RB3, and so on. On the roof plan, columns are shown in dashed lines because they terminate below the roof. However, there may be small structures, which may go a floor or two above the roof, such as the elevator machine rooms. Then separate roof plans need to be provided for them. If there are swimming pools and other landscape structures, then special detailing is required.

The typical floor plan (Figure 12.1) shows the layout of the slab, layout of the slab reinforcing bars (top and bottom), thickness of the slab, and layout of the beams. Usually, beams are assigned labels like B-1, B-2, B-3, and so on. The columns are shown in bold because the columns are both above and below the typical floor. There may be one or more typical floors depending upon the columns layout. A typical floor is a set of identical floors with regard to the architectural layout and the structural system. Again, as with the roof plan, some engineers prefer showing the slab reinforcing bars and beams markings on separate drawings.

The ground floor plan shows the ground slab details and the columns layout. If the soil conditions require the slab to be suspended (supported on ground beams also called "grade beams"), then the grade beams layout with their labels GB-1, GB-2, GB-3, and so on are also provided. In case of ground slabs not suspended (slab on grade), then only the thickness of the slab and control joints are shown on the ground floor plan. Control joints are joints used to prevent cracking in the slab. After the concrete of the slab on grade is cast, a saw cut (usually 0.25 in. × 0.75 in.) is made in the concrete after its initial hardening.

The foundation plan shows the columns layout and the foundation layout. In some one- or two-storied single family homes, due to the soil conditions, the load-bearing walls may be supported on grade beams. The ground slab may be suspended on the grade beams, which are supported on piles at regular intervals. The piles are spaced according to their capacity and the loads transferred to the grade beams. A pile foundation system for low-rise buildings is used when the soil has very low safe bearing capacity. However, in multistoried framed structures, grade beams span the footings provided for the columns. As discussed in Chapter 10, foundations could be individual pad footings, continuous footings for load-bearing walls, strapped footings for eccentrically loaded footings, pile caps supported on a group of piles for highly loaded columns, mat footings for all the columns of the structure, and combined footings for two or more columns. The footings should be identified on the plan for referencing to details of the footings provided on the same drawing or another drawing.

The beam details are typically provided in a schedule in the form of a table. The columns of the table shall consist of beam identification mark, size of the beam (width × thickness), bottom reinforcing bars, top reinforcing bars (continuous reinforcing

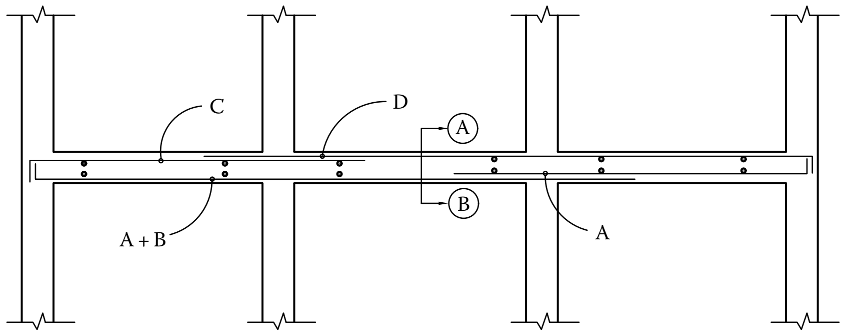

FIGURE 12.2 Typical longitudinal section of beam.

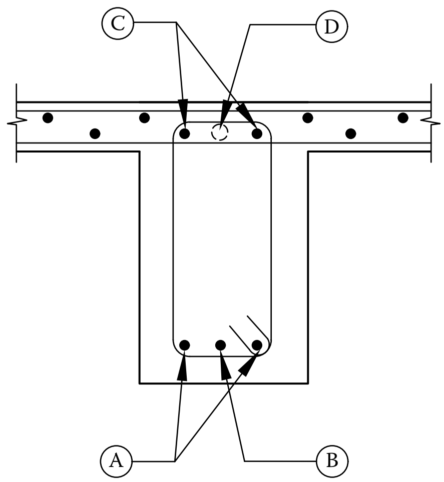

FIGURE 12.3 Typical section of beam—interior.

bars throughout the span + extra reinforcing bars at the support), reinforcing bars at mid-depth, if any (for the large depth of the beam or torsion), and stirrup size and spacing.

Typical longitudinal and cross sections of the beams must be provided (Figures 12.2 through 12.5). The longitudinal sections shall show the typical details of the continuous top and bottom reinforcing bars, typical details of reinforcing bars to be curtailed, typical details of extra reinforcing bars at the supports, standard hooks, development length, bending of reinforcing bars at the end support, placement of shear stirrups, and typical details of reinforcing bars at the face of the beam. The cross-sectional

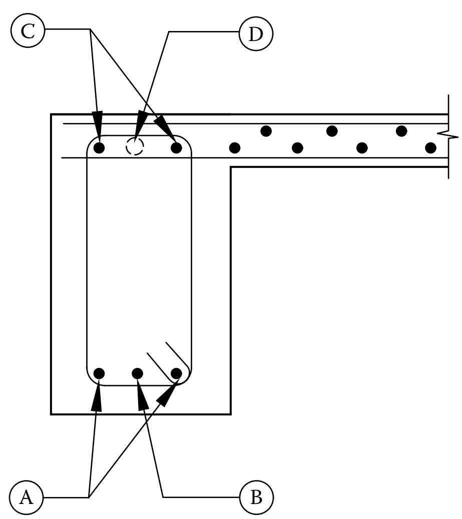

FIGURE 12.4 Typical section of beam—exterior frame.

Beam	Size	Bottom Reinforcement		Top Reinforcement			Stirrups
		A	B	C	D		
					Left	Right	

FIGURE 12.5 Beam schedule.

details shall focus on the location of the longitudinal reinforcing bars, hooks for shear reinforcement, and concrete cover to the longitudinal reinforcing bars.

Even though, the top and bottom reinforcing bars are shown on the plan view, a typical detail longitudinal section of the slab should be provided. The detail of columns shall show the continuous reinforcing bars and the bottom reinforcing bars to be curtailed when they are no longer required, top reinforcing bars at support, temperature and shrinkage reinforcing bars for one-way slab, splice in reinforcing bars, end anchorages, and hooks.

The columns schedule provided in the form of a table. The first row in the table shall be for column identification marks (C-1, C-2, C-3, etc.). Then each subsequent row shall be for the column details of each floor. The cross-sectional detail of columns for a floor shall be provided in a cell of the scheduled table related to the floor and the column type. The details of columns shall include dimensions of the column, longitudinal reinforcing bars and their placements, and the stirrups with their hook details. The grade of the concrete of the column shall also be provided. The columns schedule provided in Figure 12.6

26th				
26th				
24th				
23th				
22th				
21th				
20th				
19th				
18th				
17th				
16th				
15th				
14th				
13th				
12th				
11th				
10th				
9th				
8th				
7th				
6th				
5th				
4th				
3rd				
2nd				
Ground				
Column	**C-1**	**C-2**	**C-3**	**C-1**

FIGURE 12.6 Column schedule—high-rise building.

Column	Size	Longitudinal Bars	Stirrups

FIGURE 12.7 Column schedule—low-rise building.

is for mid-rise and high-rise buildings because as the column goes up, typically the section, longitudinal reinforcing bars, and the grade of the concrete get reduced. For low-rise buildings, the dimensions of the column, longitudinal reinforcing bars, and the grade of the concrete remain the same. Hence, only two rows are provided in the table: one row for the column identification and the other for the details (Figure 12.7).

A typical longitudinal section of the columns shall be provided to show the longitudinal reinforcing bars, connections with the footings (including the splice lengths of the dowels), how the longitudinal reinforcing bars are spliced with the columns below, and how the reinforcing bars at the top of the column are bent for upper floor columns and placement of lateral ties. If cross-sectional details are not provided in the column schedule, then typical cross-sectional details are provided based on different shapes of the columns used in the building and also the number of reinforcing bars along with the details of lateral ties to detail their arrangements. For example, cross-sectional details showing bars 4, 6, 8, 10, and so on. The clear cover shall be shown in the column details.

A schedule is provided for individual pad footings and continuous wall footings. The schedule of pad footings shall show the footing identification mark, base dimensions, thickness of footing, and the reinforcement size and spacing for each direction. Sometimes, the number of reinforcing bars along with their size in each direction is provided instead of spacing of the reinforcing bars. Pad footings are typically labeled F1, F2, F3, and so on. For continuous footings, the schedule consists of footing identification mark, dimensions (width × thickness), size and number of reinforcing bars in the longitudinal direction, and size and spacing of transverse reinforcing bars (Figure 12.7).

Typical sections for individual pad footings and continuous footings shall be provided, showing the dimensions and the placement of reinforcing bars with clear cover to the reinforcing bars. For combined footings, strap footings, and pile caps, dimensioned plans and sections showing the reinforcement details must be provided. It is recommended not to provide schedules for these footings. For mat foundations, reinforcing bars are shown on the foundation plan and sectional details are provided as supplements to show placement, hooks, and anchorage of the reinforcing bars.

Finally, structural sections and details pertaining to architectural features in the building and members other than the main structural members, such as lintels and sills, shall be provided to complete the information on the drawings. Details such as anchoring cantilever slabs in beams, connections of CMU parapets with beams or slabs, connections of members made of other materials with concrete members (such as metal rails attached to concrete beams), and typical cross sections of the buildings shall be provided. The SE must make sure that these details are coordinated with the architectural and MEP drawings.

Finally, the drawings are not complete without thorough structural notes, which should at a minimum contain design code references, design load criteria, grades of concrete and steel, concrete covers to reinforcing bars, concrete mix design criteria, and requirements of in situ and laboratory tests.

12.3 INSPECTIONS

The primary inspections in concrete structures include

- Excavations for footings
- Formwork
- Reinforcing steel
- Embedded items
- Construction joints

Excavation is typically performed for all types of footings inclusive of the mat footings. The depth of excavation depends upon the recommendations of the geotechnical engineer for the location of the bottom surface of the footing and the thickness of the footing calculated during the design process. The IBC (2015) requires that a minimum footing depth of 12 in^2 is provided below the undisturbed soil. The excavated area of the footing shall be devoid of pockets of soft materials. Soft zones found shall be undercut and backfilled. In excavations to rock, the surface of the rock shall be sound and level. The base and the walls of the excavated areas shall be clean and moist and free of frost, ice, or mud prior to the placement of concrete. If there is water present in the excavated area, it should be pumped out.

Inspecting forms is the next inspection task. Concrete forms are encasements where the reinforcing bars are placed and the concrete is cast to give the required shape to the concrete member after hardening. For footings only wall sheathing are provided for the forms. The concrete is placed on compacted soil surfaces. For columns, the cross-sectional shape of the form is the same as the cross-sectional shape of the column (rectangular, square, circular, L-shaped, polygon, etc.). Beams have formwork with two walls and a base sheathing. If the beams are tie beams, then there is no base sheathing in the formwork and the concrete is cast against the top of the masonry, where the openings of the blocks are capped to prevent concrete from spilling into the cell of the masonry. Forms are available in different materials such as plywood, steel sheets, and plastic. The contractor shall make sure that forms are strong enough to hold the weight of the concrete, equipment, and labor, and must be mortar-tight to avoid leaks of concrete. The forms must be clean of all foreign materials such as sawdust, chips, blocks, dried mortar, ice, and water. Before the placement of the concrete, they must be inspected for any irregularities, dents, sags, or holes. The surfaces of the form should be coated with a light non-staining oil or lacquer before the placement of the reinforcing bars. During the casting of the concrete, forms must be continuously monitored for any movement.

During the inspection of the reinforcing bar, the inspector shall verify the grade, bending, spacing, location, firmness of installation, and surface condition of the bar. The lengths, and the radii of the reinforcing bars must be checked. Reinforcing bars shall be spaced, spliced, and anchored; embedded at a specified minimum distance from the surface; and accurately located according to the design documents. They must be firmly held in place before and during the casting of concrete, by means of built-in concrete blocks, metallic supports, spacer bars, wires, or other devices adequate to ensure against displacement during construction and to keep the reinforcing bars at the proper distances from the forms. The concrete cover to the reinforcing bars must be in accordance with the specifications on the drawings. A tolerance of ±0.5 in. is allowed for the horizontal spacing of the bar, a tolerance of ±0.375 in. is allowed for the concrete cover of the bar for members with depths 8 in^2 or less, and a tolerance of ±0.5 in. is allowed for the concrete cover of the bar for members with depths more than 8 in^2 in accordance with table 26.6.2.1 (a) of the code. The lengths and locations of the splices must be according to the drawings. For example, the bottom bar of a slab must not be spliced at the midspan because typically that is the location of the maximum bending moment. Heating of reinforcing bars shall not be permitted to bend the reinforcing bars at the site. Rusted bars must not be allowed before cleaning them. Bars shall be free of paint, oil, grease, dried mud, and weak dried mortar that has been splashed on the bars before the concrete is placed.

Ties, anchor bolts, inserts, brick, flashing, pipe sleeves, conduits, and conduit fittings are typical items that are placed on the forms before concrete is cast. Embedment of these items should not alter the location of the reinforcing bars. If an alteration to the location of the reinforcing bar is required, then an RFI must be sent to the SE.

Control joints are installed in concrete structures to control the cracking of the concrete. Expansion joints are installed to allow movements between sections of the structure. They are used when the length of the building is greater than 100 feet. At the location of the expansion joints, structures are separated into parts above the foundations to allow for their movements. Additionally, cold joints are provided during breaks in concrete placement operations. The locations and the details of the joints are important because they weaken the sections of concrete where they are formed. The inspector shall ensure that the location and details of the joints conform to the drawings. High slump concrete mixtures must be avoided at the joints because they segregate and bleed badly, resulting in weak concrete at the surface. Coarse aggregates must be well consolidated near joints, and the surface of the concrete must be relatively smooth. It must be ensured that the area of the joints is very clean before casting the concrete.

12.4 TESTING OF CONCRETE

The SE determines the type and frequency of testing required during the construction of concrete structures. However, before the casting of the concrete, the inspector checks the consistency of the concrete and determines if water needs to be added. This can be done by performing the slump cone test. The inspector should also check the temperature of the concrete, which should range between 50°F and 90°F. For ready-mix concrete transported from a plant, the elapsed time between the batching of the concrete till the placement of the concrete shall not exceed 90 minutes in accordance with ASTM C94. The inspector shall retain copies of the tickets provided by the concrete company and attach it to the daily report.

Slump test is used to determine the consistency of concrete and to give an indication of the amount of water used in the concrete mix. They are performed at intervals specified by the SE. Slump testing shall be performed in accordance with ASTM C143. The concrete shall be sampled according to ASTM C172. The minimum and maximum slump requirements for concrete shall be specified by the SE. If the slump measured at the site is below the minimum requirement, water may be added to the truck according to the specifications on the drawings to increase the slump to within required limits. Concrete with slumps higher than the specified slumps shall be rejected.

Air content testing is used to determine the amount of entrained air in a concrete mix. Air content tests shall be performed in accordance with ASTM C173, or ASTM C231. The number and frequency of air content tests required for a particular project shall be specified by the SE. Concrete with higher quantity of air than specified by the SE shall be rejected because it reduces the strength of the concrete. Concrete with lower quantity of air than specified by the SE shall also be rejected because concrete will be subject to cracking caused by freezing and thawing.

Concrete test cylinders shall be made and tested according to ASTM C31 and ASTM C39, respectively. The number of test cylinders, the frequency at which test cylinders are to be made, and the age at which test cylinders are to be tested shall be specified by the SE. Usually, the 7-day and the 28-day compressive strength testing are specified by the SE. The concrete cylinders are typically 6 in^2 in diameter and 12 in^2 in height. The cylinders shall be carefully stored and protected from jarring, vibration, and weather before sending them to the laboratory. Section 26.12.2.1 of the code specifies the minimum frequency of preparing the concrete cylinders for compressive strength test—at least once each day, at least once every 150 $yards^3$ of concrete, and at least once for 5000 sf of the slab or wall area. Typically, five (5) cylinders are made for each sampling.

12.5 CONSTRUCTION OF CONCRETE ELEMENTS

Concrete production and construction is specified in section 26.5 of the code. The concrete should be handled properly to provide a good quality of concrete. Inspections, sampling, and testing of concrete are very important. Section 26.5.2 (f) of the code specifies that supply of concrete shall be adequate for the placement of the concrete. For example, if the concrete of a very large slab is being cast in a busy metropolitan city, then in order to have continuity of the supply of concrete, it is better to cast the concrete in the night after obtaining the required permission from the local jurisdiction. If there is break in the supply of concrete, cold joints are created. The workability of the concrete should be good for proper consolidation with segregation. The concrete shall not be placed at an angle but dumped at a vertical position. Placing concrete at an angle causes mortar and aggregate to separate. Casting concrete shall start at one end and continue without break. Concrete shall not be cast at intermittent locations to avoid cold joints. Section 26.5.2 (e) of the code prohibits conveyance of concrete using pipes made of aluminum and its alloys.

Procedures of casting concrete in cold weather are specified in section 26.5.4 of the code. When concrete is placed in cold weather, the minimum temperature shall be maintained for 3 days because concrete gains its strength very slowly at low temperatures and the initial hydration should take normally. It must be protected from freezing. The surfaces to which the concrete is expected to come in contact shall be free from frost and ice. However, overheating of the concrete above 90°F is prohibited by ASTM C94. After the concrete is cast, it should be covered with material such as insulated curing blanket for 7 days, as required by section 26.5.3.2 of the code.

Procedures of casting concrete in hot weather are specified in section 26.5.5 of the code. When concrete is placed in hot weather approaching 90°F, special measures are necessary to obtain concrete of good quality. High temperature causes rapid evaporation, reduction in slump, plastic shrinkage cracks, excessive volume change, rapid hydration, and early setting of concrete, and these reduce the compressive strength of the concrete. Typically, water is sprayed on the forms and for foundations concrete, the soil is kept moist. For large concrete jobs, it is better to cast the concrete at night after obtaining permission from the local jurisdiction.

Curing is specified in section 26.5.3 of the code. Curing helps in preventing moisture loss from the concrete. After concrete is placed, water evaporates and sometimes concrete bleeds. Hence, use of curing compounds, spraying water on the exposed surfaces of the concrete elements and covering them with wet cloth are encouraged after the concrete is cast.

Finally, forms of concrete need to be removed. They are removed after the concrete acquires adequate strength to support self-weight and any construction load that acts on the slab. For multistoried buildings after the removal of forms, slabs are reshored. Shorings are wooden or steel props that are installed at designed intervals in both directions to support slabs. Soon after the forms are removed, the strength of concrete may still be less than designed 28-day strength. The construction loads are typically higher than the designed superimposed dead plus live loads. Hence, shoring is designed by a specialty engineer to shorten the spans of the slab during the construction, thus increasing the load carrying capacity of the slab.

12.6 INSPECTION REPORTS

Inspection reports shall address the observation of the following items. This list is based upon the discussions in the above paragraphs.

a. Condition of the soil
b. Compaction of soil
c. Locations, alignments, and dimensions of forms
d. Condition of forms

e. Grade of steel; size, spacing, and location of the reinforcing bars
f. Splices of reinforcing bars
g. Physical condition of the reinforcing bars
h. Locations and proper strapping of the embedded items
i. Locations and details of the joints
j. Grades of concrete
k. Testing of concrete
l. Time difference between batching and placement of concrete
m. Placement of concrete inclusive of consolidation and vibrations
n. Check of hot or cold weather
o. Removal time of forms
p. Reshoring
q. Curing
r. Defects, if any, after the placement of concrete

REFERENCES

306R-10: Cold Weather Concreting.
347R-14: Guide to Formwork for Concrete.
American Concrete Institute (ACI) Publications.
American Society for Testing and Materials (ASTM) Publications.
C31/C31M—12: Standard Practice for Making and Curing Concrete Test Specimens in the Field.
C39/C39M—16: Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.
C94/C94M—15: Standard Specification for Ready-Mixed Concrete.
C143/C143M—15: Standard Test Method for Slump of Hydraulic Cement Concrete.
C172/C172M—14: Standard Practice for Sampling Freshly Mixed Concrete.
C173/C173M—14: Standard Test Method for Air Content of Freshly-Mixed Concrete by the Volumetric Method.
SP-2(07): Manual of Concrete Inspection.
Threshold inspection in Florida. http://www.leg.state.fl.us/Statutes/index.cfm?App_mode=Display_Statute&Search_String=&URL=0500-0599/0553/Sections/0553.79.html (Accessed July 22, 2016).

Section III

Miscellaneous Chapters

13 Various Types of Buildings

13.1 BUILDING OCCUPANCY CLASSIFICATION

Building occupancy classifications refer to categorizing structures based on their usage and are primarily used for building and fire code enforcement. They are usually defined by model building codes. According to their use and occupancy, the IBC (2015) classifies buildings into 11 categories[1]:

1. *Assembly* ***(Group A)***—places used by people gathering for entertainment, worship, and eating or drinking. *Examples*: churches, restaurants (with 50 or more possible occupants), theaters, and stadiums.
2. *Business* ***(Group B)***—places where services are provided (not to be confused with mercantile, below). *Examples*: banks, insurance agencies, government buildings (including police and fire stations), and doctors' offices.
3. *Educational* ***(Group E)***—schools and day care centers up to the 12th grade.
4. *Factory* ***(Group F)***—places where goods are manufactured or repaired (unless considered "High-Hazard" [Group H]). *Examples*: factories and dry cleaners.
5. *High-hazard* ***(Group H)***—places involving production or storage of very flammable or toxic materials. Includes places handling explosives and/or highly toxic materials (such as fireworks, hydrogen peroxide, and cyanide).
6. *Institutional* ***(Group I)***—places where people are physically unable to leave without assistance. *Examples*: hospitals, nursing homes, and prisons. In some jurisdictions, Group I may also be used to designate industrial buildings.
7. *Mercantile* ***(Group M)***—places where goods are displayed and sold. *Examples*: grocery stores, department stores, and gas stations.
8. *Residential* ***(Group R)***—places providing accommodations for overnight stay (excluding Institutional). *Examples*: houses, apartment buildings, hotels, and motels.
9. *Storage* ***(Group S)***—places where items are stored (unless considered High-Hazard). *Examples*: warehouses and parking garages.
10. *Utility and miscellaneous* ***(Group U)*** *others—Examples*: water towers, barns, and towers.
11. *Day care* ***(Group D)****—Examples*: family day care homes, and nursery schools.

13.2 STRUCTURAL CLASSIFICATION OF BUILDINGS

For the requirements of structural design, buildings can be classified based upon material of construction (steel, concrete, wood, aluminum), height (low-rise, mid-rise, high-rise), loads (earthquake-resistant, wind-resistant), and sometimes use of the building. The use of some building requires more resistance to external forces such as wind, earthquake, and blast for them to be in operational condition after a natural or man-made disaster. A brief description of some of the types of buildings, in accordance with FEMA, is provided below.[2] However, load paths of only concrete buildings are discussed in Section 1.19.

13.2.1 Wood Buildings

1. Wood light-framed buildings are for single or multiple family dwellings of one or more stories in height. Building loads are light and the framing spans are short. Floor and roof framing consists of wood joists or rafters supported on wood studs spaced no more than 24 in^2 apart. The first floor framing is typically supported directly on the reinforced concrete wall footings, either directly or through masonry stem walls. If there are chimneys in the building, they are made of solid brick masonry, masonry veneer, or wood frame with internal metal flues. Lateral forces are resisted by wood frame diaphragms and shear walls. Floor and roof diaphragms consist of straight or diagonal lumber sheathing, tongue and groove planks, oriented strand board, or plywood. Shear walls consist of straight or lumber sheathing, plank siding, oriented strand board, plywood, stucco, gypsum board, particle board, or fiberboard. Interior partitions are sheathed with plaster or gypsum board.
2. In commercial or industrial wood buildings, there are very few or no interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Lateral forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or rod bracing. Wall openings for storefronts and garages are framed by post-and-beam framing (Figure 13.1).

FIGURE 13.1 Wood-framed building.

13.2.2 Steel Buildings

1. The Steel moment framed buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Lateral forces are resisted by steel moment frames that develop their stiffness through rigid or semirigid beam–column connections. When all connections are moment-resisting connections, the entire frame participates in lateral force resistance. When only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns may be oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill. They are stiff relative to the frames. Foundations typically consist of concrete spread footings or deep pile foundations (Figure 13.2).
2. The steel brace-frame buildings have a frame of steel columns, beams, and braces. Braced frames develop resistance to lateral forces by the bracing action of the diagonal members. The braces induce forces in the associated beams and columns such that all elements work together in a manner similar to a truss, with all element stresses being primarily axial. When the braces do not completely triangulate the panel, some of the members are subjected to shear and flexural stresses; eccentrically braced frames are one such case. Diaphragms transfer lateral loads to braced frames. The diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames (Figure 13.3).
3. The steel light-framed buildings are preengineered and prefabricated with transverse rigid steel frames. They are one story in height. The roof and walls consist of lightweight metal, fiberglass, or cementitious panels. The frames are designed for maximum efficiency, and the beams and columns consist of tapered, built-up sections with thin plates. The frames are built in segments and assembled in the field with bolted or welded joints. Lateral forces in the transverse direction are resisted by the rigid frames. Lateral forces in the longitudinal direction are resisted by wall panel shear elements or rod bracing. Diaphragm forces are resisted by untopped metal deck, roof panel shear elements, or a system of tension-only rod bracing (Figure 13.4).

FIGURE 13.2 Steel moment framed building.

FIGURE 13.3 Steel brace-frame building.

FIGURE 13.4 Steel light-framed building.

4. The steel frame with concrete shear wall buildings consist of a frame assembly of steel beams and steel columns. The floors and roof consist of cast-in-place concrete slabs or metal deck with or without concrete fill. Framing consists of steel beams, open web joists, or steel trusses. Lateral forces are resisted by cast-in-place concrete shear walls. These walls are bearing walls when the steel frame does not provide a complete vertical support system. Sometimes the steel frame is designed for vertical loads only, while in other cases, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidity (called the dual system). Diaphragms consist of concrete or metal deck with or without concrete fill. In the dual system, the steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam–column connections (Figure 13.5).
5. The steel frame with infill masonry shear walls consists of a frame assembly of steel beams and steel columns. The floors and roof consist of cast-in-place concrete slabs or metal deck with concrete fill. Framing consists of steel beams, open web joists, or steel trusses. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. Infill walls may completely encase the frame members and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The diaphragms consist of concrete floors and are stiff relative to the walls.

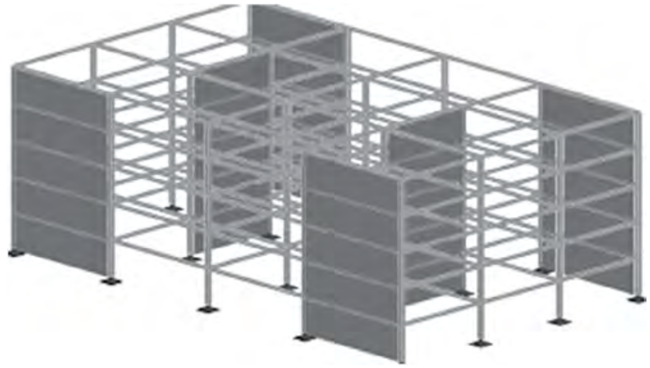

FIGURE 13.5 Steel-framed building with shear walls.

FIGURE 13.6 Concrete moment framed building.

13.2.3 Concrete Buildings

1. Concrete moment framed buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam–column connections. The moment frames may also consist of the column strips of two-way flat slab systems, when beams are eliminated. Foundations consist of concrete-spread footings or deep pile foundations (Figure 13.6).
2. Concrete shear wall buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on concrete columns or bearing walls. Lateral forces are resisted by cast-in-place concrete shear walls. Foundations consist of concrete-spread footings or deep pile foundations (Figure 13.7).
3. Concrete frame with infill masonry shear walls consists of a frame assembly of cast-in-place concrete beams and columns. The floors and roof consist of cast-in-place concrete slabs. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The diaphragms consist of concrete floors and are stiff relative to the walls.

13.2.4 Precast Concrete Buildings

1. Precast/tilt-up concrete shear wall buildings are one or more stories in height and have precast concrete perimeter wall panels that are cast on site and tilted into place. Floor and roof framing consists of wood joists, glulam beams, steel beams, or open web joists. Framing is supported on interior steel columns and perimeter concrete bearing walls. The floors and roof consist of wood sheathing or untapped metal deck. Lateral forces are resisted by the precast concrete perimeter wall panels. Wall panels may be solid or have large window and door openings that cause the panels to

FIGURE 13.7 Concrete shear wall building.

FIGURE 13.8 Precast tilt-up construction.

behave more as frames than as shear walls. Foundations consist of concrete-spread footings or deep pile foundations (Figure 13.8).

2. Precast concrete framed buildings consist of a frame assembly of precast concrete girders and columns with the presence of shear walls. Floor and roof framing consists of precast concrete planks, tees, or double-tees supported on precast concrete girders and columns. Lateral forces are resisted by precast or cast-in-place concrete shear walls. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs (Figure 13.9).

13.2.5 Masonry Buildings

1. Reinforced masonry bearing wall buildings with flexible or rigid diaphragms have bearing walls that consist of reinforced brick or concrete block masonry. Floors and roof may consists of wood trusses or wood joists or steel open web joists. Lateral forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms

FIGURE 13.9 Precast framed construction.

FIGURE 13.10 Masonry building.

consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. Foundations consist of brick or concrete-spread footings.

2. Unreinforced masonry bearing wall buildings have perimeter bearing walls that consist of unreinforced masonry. Interior bearing walls, when present, also consist of unreinforced clay brick masonry. Floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which are supported on posts. Floors may also consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms become flexible, relative to the walls. When they exist, ties between the walls and diaphragms consist of bent steel plates or anchors embedded in the mortar joints and attached to framing. Foundations consist of concrete-spread footings (Figure 13.10).

13.3 TALL BUILDINGS

Tall buildings have always been an expression of dreams, power, and technical advancement. Urbanization of major cities of the world has made the construction of tall buildings very viable for residential and business use. The first building with 100 or more floors is the Empire State Building constructed in 1931, and it held the record of the tallest building on earth for almost 36 years, followed by the World Trade Center (New York), Willis Tower (Chicago, formerly Sears Towers), Petronas Tower (Kuala Lumpur, Malaysia), Taipei 101 (Taiwan), and Burj Khalifa (Dubai). The Kingdom Tower being constructed in Jeddah, Saudi Arabia, would be 1 km tall and is scheduled to be completed in 2018. The Council on Tall Buildings and Urban Habitat (CTBUH) is an excellent source to gain information about tall buildings.

Structural design of tall buildings is an iterative process involving setting up the framework in coordination with the architectural design, deciding the structural system, assuming gravity loads, conceptual design, approximate analysis, preliminary

TABLE 13.1
Major Design Criteria of Tall Buildings

Design Criteria	Addressed by
Strength	Satisfaction of limit stress.
Serviceability	Limiting drift in the range of H/500 to H/1000, where H is the height of the building. Drift is the horizontal displacement of tall buildings caused by accumulated deformation of each member such as column, beam, brace, and shear wall.
Stability	Factor of safety against buckling and P-Delta effects.
Human comfort	Keeping acceleration of the building in the range of 10–25 mg, where g is 981 cm/s^2

design, and several iterations to prepare an optimized design. Setting up the framework consists of placing the columns and walls in such a way that the utility space is not hindered. The structural system consists of concrete slab as horizontal diaphragm and moment-resisting framed structure or shear walls structure or braced frames tubular structures or bundled tubes or outrigger system or buttressed core to resist the lateral and gravity load. The final optimized design should address the strength and stability along with serviceability and human comfort (Table 13.1).

How is a tall building classified? According to the CTBUH,[3] there is no definition of "tall building." CTBUH has provided some criteria for categorizing a building as a tall building:

1. *Height relative to context*: In a city such as New York or Chicago, a 15-storied building in the downtown area may not be considered a tall building, whereas in a rural area, a 5-storied building could be considered a tall building.
2. *Proportion*: If a building is not very high but slender, it can be addressed as a tall building, whereas a higher building with broad base may not be considered a tall building.
3. *Tall building technologies*: When the state-of-the-art tall building technologies are used in the design and construction of buildings, they provide the feeling of a tall building. Building elements such as wind braces or vertical transportation technologies, when used in buildings, classify the buildings as tall buildings. Number of floors is a weak criterion to classify a building as tall. A 500 feet high building may have 34 floors due to architectural and usage constrains, while another 500 feet high building may have 50 floors.

The CTBUH categorizes a building over 300 m (984 feet) in height as a "supertall" building, and a building over 600 m (1968 feet) in height as a "megatall" building. As of June 2015, there were 91 supertall and 2 megatall buildings fully completed and occupied globally.

Fazlur R. Khan, a twentieth-century pioneer of the design of tall buildings, provided his criteria of classifying buildings as tall buildings[4]:

1. *Height-to-width ratio*: Khan assumed buildings with same column layouts, dimensions, column spacing, and sizes of columns and beams but with different heights. When the amount of steel in columns and beams started to vary from the quantities required for shorter building, he classified the buildings as tall buildings because the lateral wind load started affecting the design.
2. *Forces and moments due to lateral loads*: Khan considered wind load as the lateral load in his explanation. For a regular building, the axial force and moment interaction equation is

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

For a tall building,

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} + \frac{f_{wa}}{F_{wa}} + \frac{f_{wbx}}{F_{wbx}} + \frac{f_{wby}}{F_{wby}} \leq 1.0$$

3. *Sway under lateral loads*: An engineer should limit the sway to a reasonable value. The maximum sway of a building is the lateral deflection of the building at the top. Sway needs to be controlled to avoid damages to the partition walls and discomfort to the occupants. Allowable sway or drift is expressed in terms of height of the building as shown in Table 13.1.

13.3.1 Types of Structures Used in Tall Buildings

Framing a building to transfer the gravity and lateral loads to the ground through a load path is based upon the engineer's perception, but in this chapter we will discuss the types of structural systems that have been universally adopted for various heights and type of loads. Some commonly used structural systems are moment-resisting framed structure, shear walls structure, braced frames, tubular structures, bundled tubes, outrigger system, and buttressed core.

Structure Type	Height (Feet)	Advantages	Example
Moment-resisting framed	200	Easy design and construction.	Home Insurance Building, Chicago (1884); 10 storied
Braced frames	1000	Due to bracing of the buildings, lateral story displacement, story drift as well as axial force and bending moment in columns reduces to a remarkable level. Reduction in lateral displacement is a major advantage. Braced frames resist the wind and seismic forces, much more than nonbraced buildings. Steel bracing is economical and easy to erect, occupies less space, and has flexibility to design for meeting the required strength and stiffness.	John Hancock Building (1969), Chicago; 100 storied
Shear walls	700	Past experience of good performance during earthquake and wind events. Provides large strength and stiffness to buildings in the direction of their orientation, significantly reducing the lateral sway and reducing damage to the structure and its content. Shear walls are easy to construct because reinforcement detailing is straightforward. They are efficient in terms of construction costs.	Four Seasons Hotel and Tower (2003), Miami, 64 storied. Because of high winds, shear wall construction is a preferred method for buildings in Miami.
Tubular	1000		
Bundled tubes	1100		
Outrigger	1500		
Buttressed core			

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3. http://www.ctbuh.org/HighRiseInfo/TallestDatabase/Criteria/tabid/446/language/en-GB/Default.aspx (Accessed May 3, 2016).
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14 Wind Load Analysis of Buildings

14.1 INTRODUCTION

This chapter addresses wind effects and provides guidelines for assessing design wind loads for buildings. The prevailing standard for the calculation for wind forces in the United States is the ASCE publication *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10). The fundamental goals of this chapter are outlined as follows:

- To present engineering understanding of wind, structural dynamics, and wind effects on buildings
- To describe how ASCE 7-10 interprets and incorporates the fundamentals of wind engineering in the document
- To apply the provisions of ASCE 7-10 with correct interpretation to assess wind loads on buildings
- To gain an in-depth understanding of ASCE 7-10 wind load provisions and wind loads on buildings
- To discuss interpretations and limitations of key provisions of ASCE 7-10

The chapter has several references to the sections, figures, tables, and equations of the ASCE 7-10. It is advised that the reader keep a copy of ASCE 7-10 while reading this chapter and frequently refer to it. An abbreviation of F for figures and T for tables is used in several portions of this chapter. Where the term ASCE is used along with equations, figures, tables, and sections, it implies ASCE 7-10; otherwise, it is a reference to a section, figure, or table of this book. In several sections of this chapter, tables are provided to explain concepts.

The chapter discusses hurricanes, the history of codes and standards pertaining to wind loads, and general wind concepts in the first three sections. In order to explain the use of ASCE 7-10, Section 14.5 of the book is dedicated to the organization of the code regarding information about wind. Sections 14.6 and 14.7 of the book deal with the general requirements for determining wind loads for various cases and the concept of velocity pressure, respectively. Sections 14.8 through 14.11 of the book deal with chapters 27 through 30 of ASCE 7-10. Section 14.12 of the book explains the major differences between ASCE 7-05 and ASCE 7-10. Some examples of the calculations of various factors of ASCE 7-10 are provided in section 14.13.

14.2 MAJOR CAUSES OF WIND FORCES

Lateral forces caused by winds is a major factor in the design of tall buildings. Even in locations that are in low-wind zones, tall buildings are designed factoring wind effects. The greatest wind effects are caused by hurricanes and tornadoes. Hurricanes are the most intense type of wind storm on earth, and few natural disasters cause as much calamity as a hurricane does. A hurricane can cause a landfall with sustained winds greater than 155 mph. During their lifetime, hurricanes can expend as much energy as that of 10,000 nuclear bombs. They are called by different names in different parts of the world. They are called “typhoons” in the western Pacific and China Sea area. In Australia, Bangladesh, India, and Pakistan, they are called “cyclones.” They are named “bagyos” in the Philippines. Their scientific name is “tropical storm.” Tropical storms are storm systems consisting of a large low-pressure center and numerous thunderstorms that produce strong winds and heavy rain. When saturated air rises, water evaporated from the ocean is released, and the storms and water vapor contained in the moist air condenses. At any height in the atmosphere, the center of a tropical cyclone will be warmer than its surroundings. In general, it is a large system of spinning air that rotates around a point of low pressure.

The first sign of hurricane formation is the appearance of a cluster of thunderstorms over the tropical oceans. It is called a tropical disturbance. When winds converge, the collision forces the air to rise, initiating thunderstorms. These convergences take place either at the meeting point of the northern and southern hemispheres at the eastern side of the equator or along the boundary between masses of warm and cold water. The thunderstorms created get organized into a more unified storm system and result in the fall of surface air pressures in the area around them. Winds begin to spin. Water vapors condense in the rising air and release energy, which increases the buoyancy of air and makes it rise. To compensate for this rising air, the surrounding air sinks and is compressed by the air above it and warms. The pressure rises at the top layer of warm air, pushing the air at the top layer outward. Now, there is less air in the layer, making the pressure of the ocean surface to drop, thus drawing more air at the surface which converges near the center of the storm to form more clouds. This becomes a chain reaction and the storm gets intensified. The lower the surface pressure, the more rapidly the air flows into the storm at the surface, increasing the wind and causing more thunderstorms. Stronger winds are triggered. When the wind speed is about 25 mph, it is called a tropical depression; at about 40 mph, it is called a tropical storm; and at 75 mph, it is called a hurricane. However, if the atmospheric condition 3–6 miles above the surface is not favorable, the storm withers away. Hurricanes can diminish in strength when the

TABLE 14.1
How Is a Hurricane Formed?

Number	Description of Event
1	Warm, moist air moves over the ocean.
2	Water vapor rises into the atmosphere.
3	As the water vapor rises, it cools and condenses into liquid droplets.
4	Condensation releases heat into the atmosphere, making the air lighter.
5	The warmed air continues to rise, with moist air from the ocean taking its place and creating more wind.

storm moves over cooler water that cannot supply warm air or when it moves over land or into an area where strong winds high in the atmosphere disperse latent heat, reducing the warm temperatures aloft and raising the surface pressure.

Tropical cyclones produce extremely powerful winds and torrential rain and are also able to produce high waves and damaging storm surges, as well as tornadoes. Once they make landfall, they lose their strength due to increased surface friction with the ground and the absence of the warm ocean as an energy source. The coastal regions receive significant damage from tropical cyclones, while the inland regions experience winds with less velocity. Heavy rains produce significant flooding inland and storm surges, flooding up to 25 miles from the coastline. A storm surge is the most destructive force accompanying hurricanes, a rise in the ocean levels of up to about 33 feet.

A major hurricane is a Category 3, 4, or 5 hurricane on the Saffir–Simpson hurricane wind scale, capable of inflicting great damage and loss of life. This scale provides specific wind values for each hurricane category. The original Saffir–Simpson hurricane wind scale category assignment of U.S. hurricanes was based on a combination of wind, central pressure, and storm surge values. It consists of five categories (1 being the weakest and 5 being the strongest).

A Scale 1 hurricane has a wind velocity of 75–95 miles per hour (mph). The accompanying storm surge is generally 4–5 feet above normal. There is no real damage to building structures. The damage is primarily to unanchored mobile homes, shrubbery, and trees. There is some damage to poorly constructed signs. Also, some coastal road flooding can occur, and minor piers can get damaged.

A Scale 2 hurricane has a wind velocity of 96–110 mph. The accompanying storm surge is generally 6–8 feet above normal. There is some damage to the roofing material, doors, and windows of buildings. There is considerable damage to shrubbery and trees, with some trees blown down. There is also considerable damage to mobile homes, poorly constructed signs, and to piers. Coastal and low-lying escape routes flood 2–4 hours before arrival of the hurricane center. Small craft in unprotected anchorages break the moorings.

A Scale 3 hurricane has a wind velocity of 111–130 mph. The accompanying storm surge is generally 9–12 feet above normal. There is some structural damage to small residences and utility buildings, with a minor amount of curtain wall (non-load-bearing exterior wall) failures. There is damage to shrubbery and trees, with foliage blown off the trees and large trees blown down. Mobile homes and poorly constructed signs are destroyed. Low-lying escape routes are cut off by rising water 3–5 hours before arrival of the hurricane center. Flooding near the coast destroys smaller structures, with larger structures damaged by battering from floating debris.

A Scale 4 hurricane has a wind velocity of 131–155 mph. The accompanying storm surge is generally 13–18 feet above normal. There are more extensive curtain wall failures, with some complete roof structure failures on small residences. Shrubs, trees, and all signs are blown down. Mobile homes are completely destroyed. There is extensive damage to doors and windows. Low-lying escape routes may be cut off by rising water 3–5 hours before arrival of the hurricane center. There is major damage to lower floors of the structures near the shore. Terrain lower than 10 feet above sea level may be flooded.

A Scale 5 hurricane has a wind velocity of greater than 155 mph. The accompanying storm surge is generally greater than 18 feet above normal. Roofs could completely fail in single-family residences and industrial buildings. Complete failures of one- or two-storied buildings have occurred. All shrubs, trees, and signs are blown down. There is severe and extensive window and door damage. Low-lying escape routes are cut by rising water 3–5 hours before arrival of the hurricane center. There is major damage to the lower floors of all structures located less than 15 feet above sea level and within 500 yards of the shoreline.

The National Hurricane Center has recorded historic information about hurricanes that made landfall in the United States. Florida, Texas, Louisiana, and North Carolina are the states with higher frequencies of hurricane landfall. Katrina, Andrew, Ike, Wilma, Ivan, Charley, Hugo, Rita, Agnes, and Betsy are among the most destructive hurricanes in recent history.

When it comes to hurricanes, wind speeds do not tell the whole story. Hurricanes produce storm surges, tornadoes, and, often the deadliest of all, inland flooding. While a storm surge is always a potential threat, more people have died from inland flooding. Intense rainfall is not directly related to the wind speed of tropical cyclones. In fact, some of the greatest rainfall amounts occur from weaker storms that drift slowly or stall over an area. Inland flooding can be a major threat to communities hundreds of miles from the coast, as intense rain falls from these huge tropical air masses. Persistent high wind and changes in air pressure push water toward the shore, causing a storm surge, which can be several feet high. Waves can be highly destructive as they move inland, battering structures in their path. On open coasts, the magnitude varies with the tides. An increase in the level of the ocean during high tide will flood larger areas than a storm that strikes during low tide. Major coastal storms can significantly change the shape of shoreline landforms, making sandy coastal floodplains unstable places for development. Wind and waves shape sand dunes, bluff, and barrier islands. The preservation of the landforms is important for the internal development because they form a protection from the effects of the storm.

14.3 BUILDING CODES ADDRESSING WIND AND FLOOD LOADS

In the twenty-first century, Hurricane Katrina has been the most devastating natural event in the United States, with a life loss of more than 1200 and a financial damage of $108 billion in Florida, Louisiana, and Mississippi. In 1900, a Category 4 hurricane hit Galveston in Texas with a sustained wind of more than 140 mph, killing more than 8000 people. There were no preparations made to resist the impact of the hurricane. The hurricane covered the buildings like an ocean and cost several million dollars to recuperate. The year 1992 experienced the most devastating hurricane in the modern times when, in August, Hurricane Andrew hit South Florida. People in Miami and Homestead were unprepared, as Andrews changed its route. Building codes were rewritten due to Andrew. There was a major change to both the Miami-Dade and Broward County edition of the South Florida Building Code, which was later incorporated in the Florida Building Code as its High Velocity Hurricane Zone (HVHZ) portion.

The two standards published by the American Society of Civil Engineers—ASCE 7-10 “Minimum Design Loads for Buildings and Other Structures” and ASCE 24-14 “Flood Resistant Design and Construction”—address the loading requirements for wind and flood. Wind and flood along with earthquake, fire, and snow are the main reasons for building fiasco in the United States.

The first code related to wind loads was published as ANSI A58.1-1972 by the American National Standards Institute (ANSI) in 1972. A subsequent edition of the ANSI A58.1 was published in 1982. Then the American Society of Civil Engineers started publishing the ASCE 7. Editions of the standard were published in 1988, 1993, 1995, 1998, 2002, 2005, and 2010. The ASCE 7-10 has been significantly re-organized in comparison with the previous codes. It has six chapters as compared to the previous versions, which had only chapter for wind loads. In the 1995 edition, the wind speed was changed from the fastest wind speed to the 3-second gust. Each revision included changes to several different factors such as the importance factor, terrain factor, directionality factor, gust effect factor, and the pressure/force coefficients.

Flood Resistant Design and Construction—ASCE 24—was published in 1998 and subsequently in 2005, 2010, and 2014. ASCE 24 deals with the minimum requirements and the expected performance for the design and construction of buildings and structures in flood hazard areas. It is not a re-statement of all of the National Flood Insurance Program (NFIP) regulations but offers additional specificity, some additional requirements, and some limitations. The parts 59, 60, 65, and 70 of chapter 44 of the Code of Federal Regulations (CFR) deal with the NFIP. These parts of the CFR chapter 44 describe the program, floodplain management criteria, identification and mapping of special hazard areas, and procedures of map corrections.

The International Building Code (2009) uses ASCE 7-05 for wind loads and ASCE 24-05 for flood loads. The Florida Building Code (2007) uses ASCE 7-05 for wind loads. However, the freeboard of ASCE 24-05 was not adopted in the Florida Building Code (2007). The Florida Building Code (2011) has adopted ASCE 7-10 for wind loads and ASCE 24-05 for flood loads. The International Building Code (2015) uses ASCE 7-10 for loads and ASCE 24-14 for floods.

In general, building envelope is the physical separator between the interior and the exterior environments of a building. Another emerging term is “building enclosure.” It serves as the outer shell to help maintain the indoor environment (together with the mechanical conditioning systems) and facilitate its climate control. Building envelope design is a specialized area of architectural and engineering practice that draws from all areas of building science and indoor climate control. The building envelope provides an air barrier system in buildings, blast safety, seismic safety, wind safety, CBR safety, and flood resistance and also provides indoor air quality and mold prevention, sustainability, and HVAC integration. In terms of wind engineering, a building envelope provides protection against strong wind actions to the interior environment and the occupants. The building envelope generally consists of the cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.

Various states and the International Code Council have their product evaluation agencies that review and approve products for the building envelopes. Like in Florida, the Miami-Dade County Product Control Division and the Florida Department of Business and Professional Regulation are the two agencies that approve products to be used in the envelope of the structure to resist wind. The International Code Council Evaluation Services is the agency that gives approvals of envelope products in accordance with the International Building Code.

The state of Florida has its own code based on the International Building Code with local amendments. There are two portions of the code—the HVHZ and the rest of Florida. The HVHZ consists of Miami-Dade and Broward Counties. Each section of the Florida Building Code has similar requirements for product approvals that include approved testing laboratories, testing standards, evaluation criteria, and quality assurance verification. There are several Roofing Application Standards and Testing Application Standards supplementing the Florida Building Code for envelope product approvals.

14.4 BASIC WIND ENGINEERING CONCEPTS

There are static, dynamic, and aerodynamic effects of wind on structures. In general, an analysis of static effects of wind is sufficient in the design of low-rise buildings. In tall buildings, the dynamic and aerodynamic effects, along with the static effects, are required to be analyzed. Flexible slender structures and structural elements such as tall buildings are subjected to wind induced along and across the direction of wind. The along-wind effects are mainly due to buffeting effects caused by turbulence, and across-wind effects are mainly due to alternate-side vortex shedding. The across-wind effect can sometimes become primary because it could exceed along-wind accelerations if the building is slender about both axes.

Galloping and flutter are two important wind-induced motions. Galloping is a transverse oscillation of a structure due to the development of aerodynamic forces, which are in phase with the motion. It is demonstrated by the progressively increasing amplitude of transverse vibration with increase of wind speed. The structural elements that are not circular are more prone to galloping. Flutter is an unstable oscillatory motion of a structure due to the coupling between aerodynamic force and elastic deformation of the structure. Combined bending and torsion are among the most common forms of oscillatory motion.

Gust, vortex shedding, and buffeting are three major dynamic components of wind that cause the oscillation of structures.

During a hurricane, the wind velocity is not constant. There is a steady component of wind, and there are effects of gusts that last for a few seconds. The wind velocity in the American standards is based on a 3-second gust (explained later). Gust gives a more realistic assessment of wind load. The intensity of gusts is related to the duration of gusts that affects the structures. In comparison with smaller structures, larger structures are affected by larger duration gusts and are subjected to less pressure. The gust effect factor accounts for additional dynamic amplification of loading in the along-wind direction due to wind turbulence and structure interaction. It does not include allowances for crosswind loading effects, vortex shedding, and instability due to galloping or flutter, or dynamic torsional effects. Where crosswind loading effects, vortex shedding, galloping, flutter, and dynamic torsion are anticipated, wind tunnels are used to determine wind pressures on buildings, which take into consideration random wind gusts acting for short durations over an entire structure or part of it; fluctuating pressures induced in the wake of the structure, including vortex shedding forces; and fluctuating forces induced by the motion of a structure.

When wind acts on a building, forces and moments in three mutually perpendicular directions are generated (three translations and three rotations). Since the weight of a building is high compared to wind pressure in the upward direction, only the along-wind response and across-transverse wind responses are considered. Only on the roof elements is the uplift due to wind considered. The across-wind response causing motion in a plane perpendicular to the direction of wind typically dominates over the along-wind response for tall buildings. In a building subjected to a smooth wind flow, the originally parallel upwind streamlines are displaced on either side of the building due to boundary layer separation. This results in spiral vortices being shed periodically from the sides into the downstream flow of wind creating a low-pressure zone due to the shedding of eddies called the wake. When the vortices are shed, crosswind components are generated in the transverse direction. At low wind speeds, since the shedding occurs at the same instant on either side of the building, there is no tendency for the building to vibrate in the transverse direction. It is therefore subject to along-wind oscillations parallel to the wind direction. At higher speeds, the vortices are shed alternately, first from one and then from the other side. When this occurs, there is a force in the along-wind direction as before, but in addition, there is a force in the transverse direction. This type of shedding, which gives rise to structural vibrations in the flow direction as well as in the transverse direction, is called vortex shedding. The frequency of shedding depends mainly on the shape and size of the structure, velocity of flow, and, to a lesser degree, on the surface roughness and turbulence of the flow. Changing the cross-sectional shape of the building over its height can ensure that vortices are broken up and cannot be shed coherently over the entire height of the building, thus reducing across-wind loading. The Sears Tower in Chicago and the Burj Khalifa in Dubai use this technique to great effect.

Large buildings affect the wind loading of nearby low buildings. There are a significant adverse effects for particular building proximity configurations. These effects are called buffeting. A downwind structure could oscillate due to the vortex shedding of adjacent large structure.

14.5 ORGANIZATION OF ASCE 7-10 FOR WIND LOAD CALCULATIONS

There is a significant difference in the organization of ASCE 7. In ASCE 7-05, the entire wind load information is in chapter 6, and in ASCE 7-10, the wind load information is in chapters 26 through 31. Table 14.2 familiarizes the reader with the chapters of ASCE 7-10 related to wind loads.

14.6 GENERAL REQUIREMENTS OF WIND LOAD CALCULATIONS

The building codes in the United States have adopted ASCE 7-10, "Minimum Design Loads for Building and Other Structures," to design the Main Wind Force Resisting System (MWFRS) and components and cladding (C & C) of buildings and other structures to resist wind loads.

TABLE 14.2
Organization of ASCE 7-10 (in Relation to Wind Pressures Determination)

Chapter	Title and Intent	Content with the Section, Figure, and Table Numbers of ASCE 7-10
1	General MWFRS/C & C	Definitions (Section 1.2) Risk Categories (Table 1.5-1)
2	Combination of loads MWFRS/C & C	Strength Design (Section 2.3) Allowable Stress Design (Section 2.4)
26	Wind loads: general requirements MWFRS/C & C	Definitions (Section 26.2), Wind Hazard Maps (Figure 26.5-1A, 1B and 1C), Directionality Factor (Section 26.6), Exposure Category (Section 26.7), Topographic Factor (Section 26.8), Gust Effect Factor (Section 26.9), Enclosure Classifications (Section 26.10), Internal Pressure Coefficient (Section 26.11)
27	Wind loads on building—directional procedure MWFRS	*Part (1)*: Enclosed, Partially Enclosed, and Open Low-Rise Buildings of All Heights *Part (2)*: Enclosed Simple Diaphragm Buildings
28	Wind loads on buildings—envelope procedure MWFRS	*Part (1)*: Enclosed, Partially Enclosed, and Open Low-Rise Buildings *Part (2)*: Enclosed Simple Diaphragm Low-Rise Buildings
29	Wind loads on other structures and building appurtenances MWFRS	Solid Freestanding Walls and Attached Sign (Section 29.4) Rooftop Structures and Equipment (Section 29.5) Parapets (Section 29.6) Roof Overhangs (Section 29.7)
30	Wind loads C & C	*Part (1)*: Low-Rise Buildings *Part (2)*: Low-Rise Buildings (Simplified) *Part (3)*: Buildings with Height > 60 feet *Part (4)*: Buildings with Height ≤ 160 feet *Part (5)*: Open Buildings *Part (6)*: Building Appurtenances and Rooftop Structures and Equipment
31	Wind tunnel procedures MWFRS/C & C	Wind Tunnel Procedure

14.6.1 Main Wind Force Resisting System and Components and Cladding

The MWFRS in accordance with chapter 26 of the ASCE 7-10 is defined as an assemblage of structural systems assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface. In simpler terms, an MWFRS consists of an entire assembly that is used to transfer the wind loads to the ground. The elements of the building envelope that do not qualify as part of the MWFRS are the C & C. They transfer the load to the MWFRS. Claddings receive wind loads directly and components receive wind loads either directly or from cladding. Components can also be part of the MWFRS when they act as a roof diaphragm or as shear walls (see Table 14.3 for examples of components and cladding).

14.6.2 General Requirements

In order to calculate wind loads for any case, chapter 26 of the ASCE 7-10 is used to determine the basic parameters for both MWFRS and C & C. The important change made to the 2010 version of the ASCE 7 is the elimination of the importance factor. For all occupancy categories, the importance factor is 1.0, and it is not used in the calculations.

A risk category is assigned to a building occupancy in table 1.5-1 of the ASCE 7-10 to determine the basic wind speed. The basic wind speed can be determined from the ASCE maps in figures 26.5-1A (Risk Category II), 26.5-1B (Risk Category III

TABLE 14.3
Examples of C & C

Element	Example
Components	Fasteners, purlins, girts, studs, roof decking, roof trusses
Claddings	Wall coverings, curtain walls, roof coverings, exterior doors and windows

TABLE 14.4
General Wind Load Parameters

Factor	Notation	ASCE 7-10 Reference
Wind directionality	K_d	Section 26.6
Exposure category		Section 26.7
Topographic factor	K_{zt}	Section 26.8
Gust effect factor	G or GC_p, GC_p, and GC_{pf}	Section 26.9
Exposure classification		Section 26.10
Internal pressure coefficient	GC_{pi}	Section 26.11

and IV), and 26.5-1C (Risk Category I). These are maps with isotachs (lines of equal pressure) representing a 3-second gust speed at 33 feet above the ground. The maps for Occupancy Category I, II, and III buildings are standardized for 300-, 700-, and 1700-year recurrence intervals, respectively, for Exposure C topography (flat, open, country, and grasslands with open terrain and scattered obstructions generally less than 30 feet in height). The minimum wind speed provided in the standard is 100 miles per hour (mph) for a mean recurrence interval of 300 years. Increasing the minimum wind speed for special topographies such as mountain terrain, gorges, and oceanfronts is recommended. The abandonment of the fastest-mile speed in favor of a 3-second gust speed first took place in the ASCE 7-1995 edition primarily due to the following reasons:

1. Modern weather stations no longer measure wind speeds using the fastest-mile method.
2. The 3-second gust speed is closer to the sensational wind speeds often quoted by news media.
3. It matches closely the wind speeds experienced by small buildings and components of all buildings.

The wind directionality factor, exposure categories, topographic factor, gust effect factor, exposure classification, and internal pressure coefficient in ASCE 7-10 have no significant changes from ASCE 7-05. The references to the ASCE 7-10 to determine these factors are provided in Table 14.4. These factors are discussed in Sections 14.6.3 through 14.6.8.

14.6.3 Wind Directionality Factor (K_d)

The wind directionality factor (K_d) accounts for the reduced probability of maximum winds coming from any given direction and for the reduced probability of maximum pressure coefficient occurring for any given wind direction. The factor K_d accounts for the directionality of wind. Directionality refers to the fact that wind rarely, if ever, strikes along the most critical direction of a building. Wind direction changes from one instant to the next. Wind can be only instantaneous along the most critical direction because at the very next instant, it will not be from the same direction. It can only be used with load combinations of ASCE sections 2.3 and 2.4.

14.6.4 Exposure Category

There are three Exposure categories B, C, and D defined in accordance with three categories of surface roughness (B, C, and D). There is an additional Exposure category A, which is used in the wind tunnel testing. The exposure category of a building or other structure should be very carefully selected, as the velocity pressure coefficient (K_h or K_z) depends on the exposure category. The velocity pressure is directly proportional to the velocity pressure coefficient. There is a significant numerical difference between the velocity coefficients for different exposures. The exposure categories are explained in Table 14.5 and the surface roughness categories are explained in Table 14.6 in accordance with the ASCE 7-10.

The ground surface roughness is measured in terms of a roughness length parameter called z_0 and can be estimated by the following equation:

$$Z_0 = 0.5H_{ob}\frac{S_{ob}}{A_{ob}} \qquad \text{(ASCE Equation C26.7-1)}$$

where

H_{ob} is the average height of roughness in the upwind stream
S_{ob} is the average vertical frontal area per obstruction presented to the wind
A_{ob} is the average area of ground occupied by each obstruction, including the open area surrounding

TABLE 14.5
Exposure Categories

Exposure Category	Description
B	Mean roof height ≤ 30 feet, if surface roughness B prevails > 1500 feet length in the upwind direction
	Mean roof height > 30 feet, if surface roughness B prevails > 2600 feet length in the upwind direction
D	If surface roughness D prevails > 5000 feet length or 20 times the height of building in the upwind direction
	If surface roughness D prevails > 600 feet or 20 times the height of building and surface roughness B or C is immediately upwind of the site
C	Where Exposure categories B and D do not apply

TABLE 14.6
Surface Roughness

Surface Roughness	Description
B	Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstruction having the size of single-family dwellings or larger
C	Open terrain with scattered obstructions having heights generally <30 feet. Includes flat open country and grasslands
D	Flat unobstructed areas and water surfaces. Includes smooth mud flats, salt flats, and unbroken ice

TABLE 14.7
Range of z_0

Exposure Category	Range of z_0 (feet)
A	>2.3
B	(0.5–2.3)
C	(0.033–0.5)
D	<0.033

14.6.5 Topographic Factor (K_{zt})

The topographic factor (K_{zt}) is used to include the wind speed-up effect in the calculations of the design wind loads. Wind speed-up effects at isolated hills, ridges, and escarpments with abrupt changes in topography. Escarpment is defined as cliff or steep slope generally separating two levels or gently sloping areas. Topographic effects are considered if all of the following ASCE 7-10 conditions listed in the ASCE section 26.8.1 are met. It is not the intent of ASCE section 26.8 to address the general case of wind flow over hilly or complex terrain for which engineering judgment, expert advice, or a wind tunnel procedure may be required.

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100 H) or 2 miles, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mile radius in any quadrant by a factor of two or more.
3. The structure is located as shown in ASCE figure 26.8-1 in the upper one-half of a hill, ridge, or near the crest of an escarpment.

Factors K_1, K_2, and K_3 are calculated from figure 26.8-1, and the topographic factor K_{zt} is calculated using the following equation. If the site conditions do not meet all of the above conditions, then K_{zt} is 1.0:

$$K_{zt} = \left(1 + K_1K_2K_3\right)^2 \quad \text{(ASCE Equation 26.8-1)}$$

14.6.6 Gust Effects

Gust is a sudden, brief increase in the speed of wind. According to the U.S. weather observing practice, gusts are reported when the peak wind speed reaches at least 17.5 mph and the variation in wind speed between the peaks and lulls is at least 10 mph. The duration of a gust is usually less than 20 seconds. ASCE 7-10 considers 3-second gusts. The "gust effect factor" is an increasing function of speed. In addition, most structures will experience yielding as "pushover" loading is increased, resulting in a reduced natural frequency and, therefore, an even higher load factor.

To determine whether a building is rigid or flexible, the fundamental natural frequency, n_a, is established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. Low-rise buildings are permitted to be considered rigid.

Rigid structures are those with the fundamental frequency ≥1 Hz. If the fundamental frequency <1 Hz, then the structure is flexible. ASCE section 26.9.3 can be used to determine the approximate natural frequency (n_a) if

Building height ≤ 300 feet (and)
Building height < 4× effective length (L_{eff})

The effective length is determined using ASCE equation 26.9-1; the approximate natural frequencies for structural steel moment-resisting frame buildings, concrete moment-resisting frame buildings, structural steel and concrete buildings with other lateral-force-resisting systems, and concrete or masonry shear wall buildings are determined using ASCE equations 26.9-2, 26.9-3, 26.9-4, and 26.9-5, respectively.

The gust effect factor of rigid buildings can be assumed as 0.85 or can be calculated using ASCE equations 26.9-6 through 26.9-9. The gust effect factor of flexible buildings can be calculated using ASCE equations 26.9-10 through 26.9-16.

As stated in the previous paragraph, ASCE 7 contains a single gust effect factor of 0.85 for rigid buildings. As an option, the designer can incorporate specific features of the wind environment and building size to more accurately calculate a gust effect factor. One such procedure is located in the body of the standard. A procedure is also included for calculating the gust effect factor for flexible structures. The gust factor for a rigid structure is 0%–10% lower than the simple, but conservative, value of 0.85 permitted in the standard without calculation. The procedures for both rigid and flexible structures (1) provide a superior model for flexible structures that displays the peak factors g_Q and g_R and (2) cause the flexible structure value to match the rigid structure as resonance is removed. A designer is free to use any other rational procedure from the approved literature.

The gust effect factor accounts for the loading effects in the along-wind direction due to wind turbulence–structure interaction and along-wind loading effects due to dynamic amplification for flexible buildings. It does not include allowances for crosswind loading effects, vortex shedding, and instability due to galloping or flutter, or dynamic torsional effects. For structures susceptible to loading effects that are not accounted for in the gust effect factor, information should be obtained from wind tunnel tests.

14.6.7 Enclosure Classifications

A structure can be classified as "Open," "Partially Enclosed" or "Enclosed" in accordance with ASCE section 26.2. A building having each wall at least 80% open is an "Open Structure." A "Partially Enclosed Structure" complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%.
2. The total area of openings in a wall that receives positive external pressure exceeds 4 feet^2 or 1% of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20%.

If a structure is not an "Open" or "Partially Enclosed" structure, then it is an "Enclosed" structure. This concept does not work properly for some structures for example a 10 feet × 10 feet building with a roof and two walls without openings. Since there are no openings in two walls, it cannot be an "Open" structure. If the two walls without openings receive the positive external pressure, the total area of the openings on these walls is zero and does not exceed 4 feet^2 or 1% of the area of the wall. Hence, it cannot be a "Partially Enclosed" structure. So, is the building "Enclosed?" In the commentary of the ASCE section 26.2, this type of building is classified as "Enclosed."

Hurricane-prone regions are areas vulnerable to hurricanes, and they are defined in ASCE section 26.2 as the U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II buildings is greater than 115 mph, including Hawaii, Puerto Rico, Guam, the Virgin Islands, and American Samoa.

Wind-borne debris regions as defined in ASCE section 26.10.3.1 are located within 1 mile of the coastal mean high water line where the basic wind speed is equal to or greater than 130 mph, or in areas where the basic wind speed is equal to or greater than 140 mph. The expanded Risk Category III wind-borne debris region in ASCE figure 26.5-1B is applied only to healthcare facilities and not to any other type of building of Risk Category III. However, the wind-borne debris region in ASCE figure 26.5-1B is applied to all of the of Risk Category IV buildings.

Glazed opening shall be protected with impact-resistant glass or shutters, tested in accordance with ASTM E1886 to comply with ASTM E1996. Glazing shall be protected up to the height of 60 feet above the ground and up to the height of 30 feet above an aggregate-surface roof located within 1500 feet of the building.

14.6.8 Internal Pressure Coefficient (GC_{pi})

The magnitude and sense of internal pressure is dependent on the magnitude and location of openings around the building envelope with respect to a given wind direction. In accordance with table 26.11-1, the internal pressure coefficients (GC_{pi}) are as follows:

Open buildings	0.0
Partially enclosed buildings	±0.55
Enclosed buildings	±0.18

(The value of GC_{pi} shall be used with two cases of positive and negative with q_z or q_h to determine the design wind pressures.)

14.6.9 Structural Damping

Structural damping is a measure of energy dissipation in a vibrating structure that brings the structure to a quiescent state. The damping is defined as the ratio of the energy dissipated in one oscillation cycle to the maximum amount of energy in the structure in that cycle. There are as many structural damping mechanisms as there are modes of converting mechanical energy into heat. The most important mechanisms are material damping and interfacial damping.

In engineering practice, the damping mechanism is often approximated as viscous damping because it leads to a linear equation of motion. This damping measure, in terms of the damping ration, is usually assigned based on the construction material. The calculation of dynamic load effects requires the damping ration as an input. In wind applications, damping ratios of 1%–2% are typically used in the United States for steel and concrete buildings at serviceability levels, respectively. Damping values for steel support structures for signs, chimneys, and towers may be much lower than buildings and may fall in the range of 0.15%–0.5%. Damping values of special structures such as steel stacks can be as low as 0.2%–0.6% and 0.3%–1.0% for unlined and lined steel chimneys, respectively. These values may provide some guidance for design. Damping levels used in wind load applications are smaller than the 5% damping ratios common in seismic applications because buildings subjected to wind loads respond essentially elastically, whereas buildings subjected to design-level earthquakes respond inelastically at higher damping levels. Because the level of structural response in the serviceability and survivability states is different, the damping values associated with these states may differ.

In addition to structural damping, aerodynamic damping may be experienced by a structure oscillating in air. In general, the aerodynamic damping contribution is quite small compared to the structural damping, and it is positive in low-to-moderate wind speeds. Depending on the structural shape, at some wind velocities, the aerodynamic damping may become negative, which can lead to unstable oscillations. In these cases, reference should be made to a wind tunnel study.

14.7 WIND VELOCITY PRESSURE

Wind velocity pressure can be calculated at a height "z" above the ground (q_z) or at the mean roof height (q_h). The basic wind speed (V) in mph is converted to a velocity pressure in psf using the basic wind parameters (velocity pressure coefficient, topographic factor, directionality factor, and basic wind speed):

$$Q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE Equations 27.3-1, 28.3-1, 29.3-1, 30.3-1)}$$

(Refer to Section 14.6 for the notation of terms.)

In the foot-pound system, the constant 0.00256 reflects the mass density of air for the standard atmosphere, that is, temperature of 59°F and sea level pressure of 29.92 in^2 of mercury and dimensions associated with wind speed in mph. The constant is obtained as follows:

$$\text{Constant} = 1/2\left[\left(0.0765\,\text{lb/feet}^3\right)/\left(32.2\,\text{feet/s}^2\right)\right]$$
$$\times\left[(\text{mph})(5280\,\text{feet/mile})\times(1\,\text{h}/3600\,\text{s})\right]^2 = 0.00256$$

0.0765 lb/feet^3	Average ambient air density
32.2 feet/s^2	Acceleration due to gravity
5280	Used to convert miles to feet
3600	Used to convert hours to second

The constant 0.00256 takes into account the equation, pressure (p) = ½ × ρ × V^2, and the conversion of mph to feet/s^2.

In the equation, ρ is the mass density and V is velocity in mph. The basic wind velocity is multiplied by the constant 0.00256, velocity pressure coefficient, topographic factor, and directionality factor to obtain the wind velocity pressure.

14.8 DIRECTIONAL PROCEDURE (CHAPTER 27, ASCE 7-10)

The Directional Procedure of the ASCE 7-10 is used for the determination of wind pressures of the MWFRS on enclosed, partially enclosed, and open buildings of all heights. It is the Analytical Method of the ASCE 7-05. A simplified method is added as Part (2) for special buildings less than 160 feet high. There are two parts in chapter 27 of ASCE 7-10. Part (1) deals with buildings of all heights where it is necessary to separate applied wind loads onto the windward, leeward, and side walls, and Part (2) deals with special buildings designated as enclosed simple diaphragms with h ≤ 160 feet. Both parts are explained below. The Directional Procedure prescribes minimum design load of 16 psf for the walls and 8 psf of the roof for Enclosed and Partially Enclosed MWFRS buildings, respectively, and 16 psf for Open buildings. The tables and figures used in Part (1) of chapter 27 are described in Table 14.9. The conditions and limitations of the Directional Procedure are listed below:

- For regular-shaped buildings.
- For buildings not subject to across-wind loading, vortex shedding, or instability due to galloping or flutter.
- The building is not located at a site for which channeling effects or buffeting in the wake of upwind obstructions warrants special consideration.
- The load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings is considered.
- No reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

14.8.1 Part (1): Enclosed, Partially Enclosed, and Open Buildings of All Heights

1. Determine V, K_d, Exposure category, K_{zt}, G, enclosure classification, and GC_{pi}.
2. Determine velocity pressure coefficient (K_z or K_h) from ASCE table 27.3-1.
3. Determine C_P from ASCE figures 27.4-1 through 27.4-3 for the cases of wall & flat, gable, hip, monoslope or mansard roof; domed roof, arched roof, monoslope roof/open building, pitched roof/open building, troughed roof/open building and along-ridge/valley wind load for monoslope, pitched or troughed roof of open building.
4. Calculate velocity pressure (q_z):

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE Equation 27.3-1)}$$

5. Calculate wind force for each case:
   A. For Enclosed and Partially Enclosed Rigid Buildings

$$p = qGC_p - q_i\left(GC_{pi}\right) \qquad \text{(ASCE Equation 27.4-1)}$$

   B. For Enclosed and Partially Enclosed Flexible Buildings

$$p = qG_fC_p - q_i\left(GC_{pi}\right) \qquad \text{(ASCE Equation 27.4-1)}$$

   C. Open Buildings With Monoslope, Pitched, Troughed Free Roofs

$$p = q_h GC_N \qquad \text{(ASCE Equation 27.4-3)}$$

   (q_h is evaluated at mean roof height and C_N is the net pressure coefficient determined from ASCE figures 27.4-4 through 27.4-7.)

   D. Roof Overhangs
   The positive external pressure on the bottom surface of the windward roof overhangs is determined using $C_p = 0.8$ and combined with the top surface pressures determined using ASCE figure 27.4-1.

E. Parapets

$$p_p = q_p\left(GC_{pn}\right) \quad \text{(ASCE Equation 27.4-4)}$$

(q_p is evaluated at the top of the parapet, and GC_{pn} is 1.5 for a windward parapet and (−) 1.0 for a leeward parapet.)
where
- $q = q_z$ for windward walls at height z
- $q = q_h$ for leeward walls, side walls, and the roof at mean roof height
- $q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of an enclosed building and negative internal pressure evaluation in partially enclosed buildings
- $q_i = q_z$ for positive internal pressure evaluation in partially enclosed building where height z is the height of the highest opening in the building that could affect the positive internal pressure.

There are four design load cases in the Directional Procedure for buildings of all heights where it is necessary to separate applied wind loads onto the windward, leeward, and side walls. Case (1) is the full design wind pressure acting on the project area perpendicular to each principal axis of the structure along the principal plane. Case (2) is 75% of the design wind pressure acting on the project area perpendicular to each principal axis of the structure in conjunction with torsional moment along the principal plane. Case (3) is 75% of the design wind pressure acting on the project area perpendicular to each principal axis of the structure along the principal plane. Case (4) is 56.3% of the design wind pressure acting on the project area perpendicular to each principal axis of the structure in conjunction with torsional moment along the principal plane. Forces obtained in each of the cases are shown in Table 14.8.

For flexible structures, the eccentricities shall be modified using the following equation:

$$e = \frac{e_Q + 1.7I_z(\mathrm{SQRT}\left(\left(q_Q Q e_Q\right)^2 + \left(g_R\, R e_R\right)^2\right)}{1 + 1.7I_z(\mathrm{SQRT}\left(\left(q_Q Q\right)^2 + \left(g_R R\right)^2\right)} \quad \text{(ASCE Equation 27.4-5)}$$

where
- e_Q is the eccentricity e for the rigid structure as defined in the table
- e_R is the distance between the elastic shear center and center of mass of each floor
- I_z, q_Q, Q, g_R, and R are determined from ASCE section 26.9 dealing with the gust effect factor

TABLE 14.8
Design Wind Load Cases

Case	Description	Action	Value
1	Full design wind pressure acting on the project area perpendicular to each principal axis of the structure along the principal plane	Separately on each plane	Direct forces—P_{WX}, P_{WY}
2	75% of the design wind pressure acting on the project area perpendicular to each principal axis of the structure in conjunction with torsional moment along the principal plane	Separately on each plane	Direct forces—$0.75P_{WX}$, $0.75P_{WY}$ Torsion $M_{TX} = 0.75(P_{WX} + P_{LX})B_X e_X$ $e_X = \pm 0.15B_X$ $M_{TY} = 0.75(P_{WY} + P_{LY})B_Y e_Y$ $e_Y = \pm 0.15B_Y$
3	75% of the design wind pressure acting on the project area perpendicular to each principal axis of the structure along the principal plane	Simultaneously on both planes	Direct forces—$0.75P_{WX} + 0.75P_{WY}$
4	56.3% of the design wind pressure acting on the project area perpendicular to each principal axis of the structure in conjunction with torsional moment along the principal plane	Simultaneously on both planes	Direct forces—$0.563P_{WX} + 0.563P_{WY}$ Torsion $M_{TX} = 0.75(P_{WX} + P_{LX})B_X e_X + 0.75(P_{WY} + P_{LY})B_Y e_Y$ $e_X = \pm 0.15B_X$, $e_Y = \pm 0.15B_Y$

Notation:
P_{WX}, P_{WY}: Windward face design pressure acting in the x and y principal axes, respectively.
P_{LX}, P_{LY}: Leeward face design pressure acting in the x and y principal axes, respectively.
e (e_X, e_Y): Eccentricity for the x and y principal axes of the structure, respectively.
M_T: Torsional moment per unit height acting about a vertical axis of the building.

TABLE 14.9
Explanation of Tables and Figures of Part (1) of Chapter 27

Table/Figure	Description
T27.2-1	Steps to determine wind loads
T27.3-1	Velocity pressure Exposure coefficients (K_z or K_{h+}) for B, C, and D up to a height of 500 feet
F27.4-1	External pressure coefficient (C_p) for walls and a gable/hip/monoslope/mansard roof (enclosed and partially enclosed structures)
F27.4-2	External pressure coefficient (C_p) for a domed roof (enclosed and partially enclosed structures)
F27.4-3	External pressure coefficient (C_p) for an arched roof (enclosed and partially enclosed structures)
F27.4-4	External pressure coefficient (C_p) for monoslope free roofs (open structures)
F27.4-5	External pressure coefficient (C_p) for pitched free roofs (open structures)
F27.4-6	External pressure coefficient (C_p) for troughed free roofs (open structures)
F27.4-7	External pressure coefficient (C_p) for free roofs (open structures)
F27.4-8	Design wind load Cases 1, 2, 3, and 4 with equations for eccentricities and moments due to torsion

ASCE Appendix D lists buildings exempted from torsional wind cases. They include one- and two-storied buildings with flexible diaphragms or light-frame constructions, seismic-controlled buildings, torsionally regular buildings, flexible diaphragm buildings that are designed for increased wind loadings, and Class 1 and Class 2 simple diaphragm buildings. There are certain criteria listed in appendix D for the exemption.

14.8.2 Part (2): Enclosed Simple Diaphragm Buildings with Heights ≤ 160 Feet

In accordance with ASCE chapter 26, a simple diaphragm building is one in which both windward and leeward wind loads are transmitted by roof assemblies and vertically spanning wall assemblies, through continuous floor and roof diaphragms, to the MWFRS. The tables and figures used in Part (2) of ASCE chapter 27 are described in Table 14.10.

1. Determine V, K_d, the Exposure Category, K_{zt}, G, the Enclosure Classification, and GC_{pi}.
2. From ASCE table 27.6-1, determine net pressures on walls at the top and base (p_h, p_0, respectively) of a building.
3. From ASCE table 27.6-2, determine net roof pressures (p_z).
4. Apply topographic factors to the wall and roof pressures (if applicable).
5. Apply loads to walls and roof simultaneously.
6. Where two load cases are shown in the table of roof pressures, the effects of each load case shall be dealt separately. The MWFRS shall be designed for the four wind load cases of figure 27.4-8 with the exceptions of appendix D.

The building shall meet the class 1 or class 2 requirements of the code.

Class (1)—Simple diaphragm building, $h \leq 60$ feet, range of L/B (0.2–5.0), $K_{zt} = 1.0$
For L/B < 0.5, use tabulated value of L/B = 0.5.
For L/B > 2.0, use tabulated value of L/B = 2.0.

TABLE 14.10
Explanation of the Tables and Figures of Part (2) of Chapter 27

Table/Figure	Description
T27.5-1	Steps to determine wind loads
F27.5-1	Geometry requirements of classes 1 and 2 enclosed simple diaphragm buildings
F27.6-1	Application of wind pressures for enclosed simple diaphragm buildings
F27.6-2	Application of parapet wind loads for enclosed simple diaphragm buildings
F27.6-3	Application of roof overhang wind loads for enclosed simple diaphragm buildings
T27.6-1	Application of wall pressures for enclosed simple diaphragm buildings MWFRS wind loads—walls—Exposures B, C, and D
T27.6-2	Application of roof pressures for enclosed simple diaphragm buildings MWFRS wind loads—walls—Exposure C Exposure adjustment factors for Exposures B and D Description of flat/gable/hip/monoslope/mansard roofs

TABLE 14.11
Explanation of Tables and Figures of Part (1) of Chapter 28

Table/Figure	Description
T28.2-1	Steps to determine wind loads
T28.3-1	Velocity pressure Exposure coefficients (K_z or K_{h+}) for B, C, and D up to a height of 60 feet
F28.4-1	External pressure coefficient (GC_{pf}) for low-rise walls and roofs (enclosed and partially enclosed structures)

Class (2)—Simple diaphragm building
60 feet < h ≤ 160 feet, range of L/B (0.5–2.0)
Fundamental natural frequency (f) ≥ 75/h, K_{zt} = 1.0
Parapets: Wind pressure = 2.25 × wind pressure for the wall with L/B = 1.0 applied simultaneously with wall and roof pressures. The height to determine the parapet wind pressures is the height of the building at the top of the parapet.*
Roof overhangs: Positive wind pressure on the underside of the roof overhang is 75% of the roof edge pressure for the applicable zone applied on the windward roof overhang.

14.9 ENVELOPE PROCEDURE (CHAPTER 28, ASCE 7-10)

The Envelope Procedure is used to determine the MWFRS wind loads on low-rise buildings. There are two parts of this procedure. Part I is the former "low-rise buildings" provision of Method 2 of ASCE 7-05. Part 2 is derived from the MWFRS provisions of Method 1 for simple diaphragm buildings up to 60 feet in height.

There are two parts in chapter 28 of ASCE 7-10. Part (1) deals with low-rise buildings where it is necessary to separate applied wind loads onto the windward, leeward, and side walls of the building to properly assess the internal forces in the MWFRS members, and Part (2) deals with a special class of low-rise buildings designated as the enclosed simple diaphragm building. The tables and figures used in Part (1) of chapter 28 are described in Table 14.11. The conditions and limitations of the Envelope Procedure are as follows:

- For regular-shaped buildings.
- For buildings not subject to across-wind loading, vortex shedding, or instability due to galloping or flutter.
- The building is not located at a site for which channeling effects or buffeting in the wake of upwind obstructions warrants special consideration.
- The load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings is considered.
- No reduction in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

14.9.1 Part (1): Enclosed and Partially Enclosed Low-Rise Buildings

1. Determine V, K_d, Exposure Category, K_{zt}, Enclosure Classification, and GC_{pi}.
2. Determine the velocity pressure coefficient (K_z or K_h) from ASCE table 28.3-1.
3. Calculate velocity pressure (q_z):

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE Equation 28.3-1)}$$

4. Calculate the wind pressure (p) for each case as shown below:
   a. For low-rise buildings

$$p = q_h\left[\left(GC_{pf+}\right) - \left(GC_{pi+}\right)\right] \qquad \text{(ASCE Equation 28.4-1)}$$

where
q_h is the velocity pressure evaluated at mean roof height
GC_{pf} is the external pressure coefficient (ASCE figure 28.4-1)

TABLE 14.12
Explanation of Tables and Figures of Part (2) of Chapter 28

Table/Figure	Description
T28.5-1	Steps to determine wind loads
F28.6-1	Design wind pressures for walls and roofs of enclosed buildings
	Tables for wind pressures
	Table for adjustment factor for height and exposure

b. Parapets

$$P_P = q_p\left(GC_{pn}\right) \quad \text{(ASCE Equation 28.4-2)}$$

where
q_p is the velocity pressure evaluated at top of the parapet
GC_{pn} is 1.5 for windward parapet and (−) 1.0 for leeward parapet

c. Roof overhangs
The positive external pressure on the bottom surface of the windward roof overhangs shall be determined using $C_p = 0.7$ and combined with the top surface pressures determined using figure 28.4-1.

The minimum design load in the design of MWFRS for an enclosed or partially enclosed building shall not be less than 16 psf.

14.9.2 Part (2): Enclosed Simple Diaphragm Low-Rise Buildings

In accordance with ASCE chapter 26, a simple diaphragm building is one in which both the windward and leeward wind loads are transmitted by roof assemblies and vertically spanning wall assemblies, through continuous floor and roof diaphragms, to the MWFRS. The tables and figures used in Part (2) of ASCE chapter 28 are described in Table 14.12.

1. Determine V, the Exposure Category, K_{zt}, and the Enclosure Classification.
2. From figure 28.6-1, determine the wind pressure (p_{s30}) for h = 30 feet and Exposure B.
3. From figure 28.6-1, determine the adjustment factor (λ) for height and exposure.
4. Determine adjusted wind pressure.

$$p_s = \lambda K_{zt} p_{s30} \quad \text{(ASCE Equation 28.6-1)}$$

The conditions of use for Part (2) are as follows:

- Simple diaphragm, low-rise, Enclosed, regular-shaped, rigid, and conforms to the wind-borne debris provisions of ASCE section 26.10.3.
- For buildings not subject to across-wind loading, vortex shedding or instability due to galloping or flutter.
- The building is not located at a site for which channeling effects or buffeting in the wake of upwind obstructions warrants special consideration.
- The building has an approximately symmetrical cross section in each direction with a flat roof, gable roof, or hip roof with $\theta \leq 45°$.
- The building is exempted from torsional load cases as indicated in Note 5 of figure 28.4-1, or the torsional load cases defined in Note 5 do not control the design of any of the MWFRSs of the building.

14.10 OTHER STRUCTURES AND BUILDING APPURTENANCES (CHAPTER 29, ASCE 7-10)

Building appurtenances generally consist of rooftop structures and rooftop equipment. Other structures generally consist of solid freestanding walls, freestanding solid signs, chimneys, tanks, open signs, lattice frameworks, and trussed towers. Equations for determining the wind forces for solid freestanding walls, solid signs, other structures, and lateral and vertical forces acting on rooftop structures are provided in the ASCE chapter 29. The tables and figures used in ASCE chapter 29 are described in Table 14.13. The conditions and limitations of ASCE chapter 29 are listed below:

- For regular-shaped buildings.
- For buildings not subject to across-wind loading, vortex shedding, or instability due to galloping or flutter.

TABLE 14.13
Explanation of Tables and Figures of Chapter 29

Table/Figure	Description
T29.1-1	Steps to determine wind loads
T29.3-1	Velocity pressure Exposure coefficients (K_z or K_{h+}) for B, C, and D up to a height of 500 feet
F29.4-1	Force coefficients (C_f) for solid freestanding walls and signs
F29.5-1	Force coefficients (C_f) for chimneys, tanks, rooftop equipment, and similar structures
F29.5-2	Force coefficients (C_f) for open signs and lattice frameworks
F29.5-3	Force coefficients (C_f) for trussed towers

- The building is not located at a site for which channeling effects or buffeting in the wake of upwind obstructions warrants special consideration.
- The load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings is considered.
- No reduction in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

The procedure to calculate the forces is as follows:

1. Determine V, K_d, G, the Exposure Category, K_{zt}, and the Enclosure Classification.
2. Determine the velocity pressure coefficient (K_z or K_h) from table 29.3-1.
3. Calculate velocity pressure (q_z):

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE Equation 28.3-1)}$$

4. Calculate wind pressure (p) as shown below for each case:
   a. Solid freestanding walls and solid signs

$$F = q_h G C_f A_s \qquad \text{(ASCE Equation 29.4-1)}$$

where
C_f is the net force coefficient from ASCE figure 29.4-1
A_s is the gross area of the structure

   b. Other structures

$$F = q_z G C_f A_s \qquad \text{(ASCE Equation 29.5-1)}$$

where
C_f is obtained from ASCE figures 29.5-1 (for chimneys, tanks, rooftop equipment, and similar structures), 29.5-2 (for open signs and lattice work), and 29.5-3 (for trussed towers)
A_f is the projected area normal to wind except where C_f is specified for actual surface area

   c. Rooftop structures and equipment for H ≤ 60 feet (lateral force)

$$F_h = q_h \left(GC_r\right) A_f \qquad \text{(ASCE Equation 29.5-2)}$$

where
(GC_r) is 1.9 for rooftop structures and equipment with $A_f < (0.1Bh)$ and is reduced from 1.9 to 1.0 if A_f increased from (0.1Bh) to (Bh)
A_f is the vertical projected area normal to wind

   d. Rooftop structures and equipment for H ≤ 60 feet (vertical uplift force)

$$F_v = q_h \left(GC_r\right) A_f \qquad \text{(ASCE Equation 29.5-3)}$$

(GC_r) is 1.5 for rooftop structures and equipment with $A_f < (0.1Bh)$ and is reduced from 1.5 to 1.0 if A_f increased from (0.1Bh) to (Bh)
A_f is the horizontal projected area normal to wind

Use the methods of Directional and Envelope procedures to calculate wind loads for parapets and roof overhangs. The minimum design wind force shall be not less than 16 psf multiplied by the area A_f.

14.11 COMPONENTS AND CLADDING (CHAPTER 30, ASCE 7-10)

The conditions and limitation for the use of chapter 30 of ASCE 7-10 are as follows:

- For regular-shaped buildings.
- For buildings not subject to across-wind loading, vortex shedding, or instability due to galloping or flutter.
- The building is not located at a site for which channeling effects or buffeting in the wake of upwind obstructions warrants special consideration.
- The load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings is considered.
- No reduction in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.
- Used for air-permeable cladding unless lower loads are demonstrated by tests or research.
- Elements with tributary areas greater than 700 feet^2 can be designed as MWFRS.

The general requirements of chapter 30 of ASCE 7-10 are as follows:

1. Determine V, K_d, the Exposure Category, K_{zt}, G, the Enclosure Classification, and GC_{pi}.
2. Determine the velocity pressure coefficient (K_z or K_h) from table 30.3-1.
3. Calculate velocity pressure (q_z):

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE Equation 28.3-1)}$$

4. Determine the design wind pressure (p) for each case as defined in Table 14.14 and as explained below.

14.11.1 Part (1): Low-Rise Buildings

$$p = q_h\left[\left(GC_p\right) - \left(GC_{pi}\right)\right] \qquad \text{(ASCE Equation 30.4-1)}$$

GC_p is the external pressure coefficients given in ASCE figure 30.4-1 for walls; ASCE figures 30.4-2A to 30.4-2C for flat, gable roof, and hip roofs; ASCE figure 30.4-3 for stepped roofs; ASCE figure 30.4-4 for multispan gable roofs; ASCE figures 30.4-5A and 30.4-5B for monoslope roofs; ASCE figure 30.4-6 for sawtooth roofs; ASCE figure 30.4-7 for domed roofs; and ASCE figure 27.4-3, footnote 4, for arched roofs. q_h is evaluated at mean roof height.

14.11.2 Part (2): Low-Rise Building

$$p_{net} = \lambda K_{zt}\, p_{net30} \qquad \text{(ASCE Equation 30.5-1)}$$

λ is the adjustment factor for building height and exposure from ASCE figure 30.5-1
p_{net30} is the net design wind pressure for Exposure B, at h = 30 feet, from ASCE figure 30.5-1

TABLE 14.14
Arrangement of Chapter 30

Part	Height	Applicability	Description
1	≤60 feet	Enclosed or partially enclosed	Flat, gable, multispan gable, hip, monoslope, stepped, or sawtooth roof
2	≤60 feet	Enclosed	Flat, gable, or hip roof
3	>60 feet	Enclosed or partially enclosed	Flat, pitched, gable, hip, mansard, arched, or domed roof
4	≤160 feet	Enclosed	Flat, gable, hip, monoslope, or mansard roof
5	All	Open	Pitched free, monoslope free, or trough-free roof
6	All		Building appurtenances such as roof overhangs and parapets and rooftop equipment

TABLE 14.15
Explanation of Tables and Figures of Part (1) of Chapter 30

Table/Figure	Description
T30.4-1	Steps to determine wind loads
F30.4-1	External pressure coefficient (GC_p) for walls with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-2A	External pressure coefficient (GC_p) for a gable roof (Θ ≤ 7°) with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-2B	External pressure coefficient (GC_p) for gable/hip roof (7° < Θ ≤ 27°) with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-2C	External pressure coefficient (GC_p) for gable/hip roof (27° < Θ ≤ 45°) with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-3	External pressure coefficient (GC_p) for stepped roof with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-4	External pressure coefficient (GC_p) for a multispan gable roof with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-5A	External pressure coefficient (GC_p) for a monoslope roof (3° < Θ ≤ 10°) with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-5B	External pressure coefficient (GC_p) for monoslope roof (10° < Θ ≤ 30°) with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-6	External pressure coefficient (GC_p) for a saw-tooth roof with h ≤ 60 feet (enclosed and partially enclosed structures)
F30.4-7	External pressure coefficient (GC_p) for a domed roof for all heights (enclosed and partially enclosed structures)

TABLE 14.16
Explanation of Tables and Figures of Part (2) of Chapter 30

Table/Figure	Description
T30.5-1	Steps to determine wind loads
F30.5-1	Design wind pressures of walls and roofs of enclosed structures with h ≤ 60 feet Tables include those for Net design wind pressures for walls and roofs Flat roof Hip roof (7° < Θ ≤ 27°) Gable roof (Θ ≤ 7°) Gable roof (7° < Θ ≤ 45°) Net design wind pressures for roof overhang Adjustment factor for building height and exposure

14.11.3 Part (3): Buildings with h > 60 Feet

$$p = q_h\left[\left(GC_p\right)-\left(GC_{pi}\right)\right] \quad \text{(ASCE Equation 30.6-1)}$$

where

- $q = q_z$ for windward walls at height z
- $q = q_h$ for leeward walls, side walls, and roof at mean roof height
- $q_i = q_h$ for windward walls, side walls, leeward walls of an enclosed building, and negative internal pressure evaluation in partially enclosed buildings
- $q_i = q_z$ for positive internal pressure evaluation in a partially enclosed building where height z is the height of the highest opening in the building that could affect the positive internal pressure.

GC_p is the external pressure coefficient given in ASCE figure 30.6-1 for walls and flat roofs; 27.4-3, footnote 4, for arched roofs; 30.4-7 for domed roofs; – Note 6 of 30.6-1 for other roof angles and geometries (Table 14.17).

TABLE 14.17
Explanation of Tables and Figures of Part (3) of Chapter 30

Table/Figure	Description
T30.6-1	Steps to determine wind loads
F30.6-1	External pressure coefficient (GC_p) for walls and roofs with h > 60 feet (enclosed and partially enclosed structures)

14.11.4 Part (4): Buildings with h ≤ 160 Feet

$$p = p_{table}\left(EAF\right)\left(RF\right)K_{zt} \quad \text{(ASCE Equation 30.7-1)}$$

RF is the effective area reduction factor from Table 30.7-2, and EAF is the exposure adjustment factor from Table 30.7-2.

14.11.4.1 Windward Parapet (Load Case A)

Windward parapet pressure is determined using the positive wall pressure zone 4 or 5 from ASCE table 30.7-2.

Leeward parapet pressure is determined using the negative roof pressure zone 2 or 3 from ASCE table 30.7-2.

14.11.4.2 Leeward Parapet (Load Case B)

Windward parapet pressure is determined using the positive wall pressure zone 4 or 5 from ASCE table 30.7-2.

Leeward parapet pressure is determined using the negative wall pressure zones 4 or 5 from ASCE table 30.7-2.

Height is specifically the height of the top of the parapet. A wind effective area of 10 feet^2 is used in the calculations of wind pressures of ASCE table 30.7-2, and the RF can be used to calculate the wind pressure for higher tributary area.

14.11.4.3 Roof Overhang

The net outward pressure of the roof overhang is the edge zone pressure of zones 1 and 2 read from ASCE table 30.7-2. For zone 3, the wind pressure read from ASCE table 30.7-2 shall be increased by 15%.

14.11.5 Part (5): Open Buildings

$$p = q_h G C_N \quad \text{(ASCE Equation 30.8-1)}$$

C_N is evaluated from ASCE figure 30.8-1 for a monoslope roof, ASCE figure 30.8-2 for pitched roof, and ASCE figure 30.8-3 for troughed roof.

TABLE 14.18
Explanation of Tables and Figures of Part (4) of Chapter 30

Table/Figure	Description
T30.7-1	Steps to determine wind loads
T30.7-2	Wall and roof pressures for enclosed buildings with h ≤ 160 feet for flat, gable, monoslope, hip, and mansard roofs Tables include those for C & C exposure adjustment factor Effective wind area for reduction factors C & C wind pressures for different heights and velocities
F30.7-1	Parapet wind loads for enclosed simple diaphragm buildings with h ≤ 160 feet
F30.7-2	Roof overhang wind loads for enclosed simple diaphragm buildings with h ≤ 160 feet

TABLE 14.19
Explanation of Tables and Figures of Part (5) of Chapter 30

Table/Figure	Description
T30.8-1	Steps to determine wind loads
F30.8-1	Net pressure coefficient C_N for Open buildings monoslope free roof ($\Theta \leq 45°$) ($0.25 \leq h/L \leq 1.0$)
F30.8-2	Net pressure coefficient C_N for Open buildings pitched free roof ($\Theta \leq 45°$) ($0.25 \leq h/L \leq 1.0$)
F30.8-3	Net pressure coefficient C_N for Open buildings troughed free roof ($\Theta \leq 45°$) ($0.25 \leq h/L \leq 1.0$)

14.11.6 Part (6): Building Appurtenances and Rooftop Structures and Equipment

14.11.6.1 Part (6)(a): Parapets

$$p = q_P\left[\left(GC_p\right)-\left(GC_{pi}\right)\right] \quad \text{(ASCE Equation 30.9-1)}$$

where q_P is the velocity pressure evaluated at the top of the parapet.

GC_p is the external pressure coefficients given in ASCE figure 30.4-1 for walls with $h \le 60$ feet; 30.4-2A to 30.4-2C for flat, gable, and hip roofs; 30.4-3 for stepped roofs; 30.4-4 for multispan gable roofs; 30.4-5A and 30.4-5B for monoslope roofs; 30.4-6 for sawtooth roofs; 30.4-7 for domed roofs of all heights; 30.6-1 for walls and flat roofs with $h > 60$ feet; and 27.4-3, footnote 4, for arched roofs.

Load cases of ASCE figure 30.9-1 apply.

Load case (A): Apply positive wall pressure from ASCE figure 30.4-1 ($h \le 60$ feet) or from ASCE figure 30.6-1 ($h > 60$ feet) to the windward surface of the parapet. Apply negative edge or corner-zone roof pressure from ASCE figures 30.4-2 (A, B, or C), 30.4-3, 30.4-4, 30.4-5 (A or B), 30.4-6, and 30.4-7; ASCE figure 27.4-3, footnote 4; or figure 30.6-1 ($h > 60$ feet) to the leeward surface of the parapet.

Load case (B): Apply positive wall pressure from ASCE figure 30.4-1 ($h \le 60$ feet) or from ASCE figure 30.6-1 ($h > 60$ feet) to the windward surface of the parapet. Apply the negative wall pressure from ASCE figure 30.4-1 ($h \le 60$ feet) or ASCE figure 30.6-1 ($h > 60$ feet) to the leeward surface.

14.11.6.2 Part (6)(b): Roof Overhang

$$p = q_h\left[\left(GC_p\right)-\left(GC_{pi}\right)\right] \quad \text{(ASCE Equation 30.10-1)}$$

GC_p is the external pressure coefficient for overhangs from ASCE figures 30.4-2A to 30.4-2C (flat roofs, gable roofs, and hip roofs), including contributions from the top and bottom surfaces of the overhang. The external pressure coefficient for the covering on the underside of the roof overhang is the same as the external pressure coefficient on the adjacent wall surface, adjusted for effective wind area, determined from ASCE figure 30.4-1 or 30.6-1.

14.11.6.3 Part (6)(c): Rooftop Structures and Equipment for Buildings with $h \le 60$ Feet

Wall pressures of the rooftop structures are calculated in accordance with ASCE section 29.5.1 and divided by their respective wall surface areas. They shall be considered to act inward or outward.

Roof pressures of the rooftop structures are calculated in accordance with ASCE section 29.5.1 and divided by the horizontal projected area of the roof. They shall be considered to act upward.

14.12 SIGNIFICANT CHANGES IN ASCE 7-10 AS COMPARED TO ASCE 7-05

There have been significant changes between the 2005 and 2010 versions of ASCE 7. In ASCE 7-05, the entire wind load information is in chapter 6, and in ASCE 7-10 wind load information is in chapters 26 through 31. The changes are demonstrated in Tables 14.21 through 14.30.

TABLE 14.20
Explanation of Tables and Figures of Part (6) of Chapter 30

Table/Figure	Description
T30.9-1	Steps to determine wind loads
T30.10-1	Steps to determine wind loads
F30.9-1	Parapet wind loads for all buildings types
F30.10-1	Roof overhang wind loads for all buildings types

TABLE 14.21
Arrangement of the Code

ASCE 7-05	ASCE 7-10	
Only chapter 6—"Wind Loads"—deals with the complete information.	Chapters 26–31 have been introduced.	
	Chapter	Title
	26	Wind Loads—General Requirements
	27	Wind Loads on Buildings—MWFRS (Directional Procedure)
	28	Wind Loads on Buildings—MWFRS (Envelope Procedure)
	29	Wind Loads on Other Structures & Building Appurtenances—MWFRS
	30	Wind Loads—Components & Cladding
	31	Wind Tunnel Procedure

TABLE 14.22
Definitions

ASCE 7-05	ASCE 7-10
In the ASCE 7-10, the definitions for Building, Torsionally Regular under Wind Load, and Diaphragm; Directional Procedure, Envelope Procedure, and Wind Tunnel Procedure were added.	
The hurricane-prone regions are the U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed is greater than 90 mph.	The hurricane-prone regions are the U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II is greater than 115 mph.
Locations of wind-borne debris region areas within hurricane-prone regions: 1. Within 1 mile of coastal mean high water line where the basic wind speed ≥110 mph and in Hawaii 2. In areas where the basic wind speed ≥140 mph	Locations of wind-borne debris region areas within hurricane-prone regions: 1. Within 1 mile of coastal mean high water line where the basic wind speed ≥130 mph or 2. In areas where the basic wind speed ≥140 mph

TABLE 14.23
Basic Wind Speed

ASCE 7-05	ASCE 7-10	
Basic wind speed is determined using figure 6-1 for all exposure categories with the exception of Special Wind Regions and the estimation of basic wind speeds from Regional Climatic Data.	Basic wind speed is determined using figure 26.5-1 according to exposure categories with the exception of Special Wind Regions and estimation of basic wind speeds from Regional Climatic Data.	
	Figure	Risk Category
	26.5-1A	II
	26.5-1B	III and IV
	26.5-1C	I
In both versions, tornadoes have not been considered in developing the basic wind speed distributions.		

14.12.1 Some Similarities between ASCE 7-05 and ASCE 7-10

1. Wind directionality factor
2. Definitions of surface roughness
3. Information about topographic effects
4. Definitions of the Enclosure Classifications
5. The values of internal pressure coefficient for the various enclosure classifications and the equation for the calculation for the reduction factor for large-volume buildings

TABLE 14.24
Permitted Procedures

ASCE 7-05

For both MWFRS and C & C, the following procedures are allowed:

Method	Description
1	Simplified procedure
2	Analytical procedure
3	Wind tunnel procedure

ASCE 7-10

Directional, Envelope, and Wind Tunnel procedures have been used in five different chapters.

Chapter	Component	Description
27	MWFRS—buildings of all height	Directional procedure
28	MWFRS—low-rise buildings	Envelope procedure
29	Building appurtenances and other structures	Directional procedure
30	C & C	Envelope procedure (Parts 1 and 2) Directional procedure (Parts 3, 4, and 5) Building appurtenances (Part 6)
31	MWFRS/C & C	Wind Tunnel Procedure

TABLE 14.25
Importance Factor

ASCE 7-05	ASCE 7-10
Based on the building categories listed in table 1-1, importance factors differ.	The importance factor is the same for all building categories. It is not used in the calculations of the wind velocity pressure.

TABLE 14.26
Exposure

ASCE 7-05	ASCE 7-10
Exposure B is applied to buildings of all heights where Surface Roughness B prevails in the upwind direction for a distance of at least 2600 feet or 20 times the height of the building, whichever is greater.	For buildings of mean roof height ≤ 30 feet, Exposure B is applied where Surface Roughness B prevails in the upwind direction for a distance of at least 1500 feet. For buildings of mean roof height ≥30 feet, Exposure B is applied where Surface Roughness B prevails in the upwind direction for a distance of at least 2600 feet or 20 times the height of the building, whichever is greater.

TABLE 14.27
Gust Effects

ASCE 7-05	ASCE 7-10
	The calculations of the gust effect remaining the same, calculations for approximate natural frequency in sections 26.9.2 and 26.9.3 are added.

TABLE 14.28
Velocity Pressure

ASCE 7-05	ASCE 7-10
$q_z = 0.00256K_zK_{zt}K_dIV^2$ (lb/feet^2)	$q_z = 0.00256K_zK_{zt}K_dV^2$ (lb/feet^2) Since importance factor I = 1 for all building classifications, it is not used in the calculations of the velocity pressure.

TABLE 14.29
Analytical Procedure of ASCE 7-05 and Directional and Envelope Procedures of ASCE 7-10

ASCE 7-05 Method II (Analytical Procedure)	ASCE 7-10 Directional and Envelope Procedures
MWFRS rigid buildings of all heights	MWFRS enclosed and partially enclosed rigid buildings (Chapter 27/Part 1)
MWFRS low-rise buildings	MWFRS enclosed and partially enclosed low-rise buildings (Chapter 28/Part 1)
MWFRS flexible buildings	MWFRS enclosed and partially enclosed flexible buildings (Chapter 27/Part 1)
MWFRS parapets	MWFRS parapets (Chapter 27/Part 1)
C & C low-rise buildings, h ≤ 60 feet	C & C enclosed and partially enclosed low-rise buildings, h ≤ 60 feet (Chapter 30, Part 1)
C & C buildings, h > 60 feet	C & C enclosed and partially enclosed buildings, h > 60 feet (Chapter 30, Part 3)
MWFRS for monoslope, pitched, or troughed roof Open buildings	MWFRS open buildings with monoslope, pitched, or troughed roof (Chapter 27/Part 1)
C & C for monoslope, pitched, or troughed roof Open buildings	C & C open buildings with monoslope, pitched, or troughed roof (Chapter 30, Part 5)
Solid freestanding walls and solid signs	Solid freestanding walls and solid signs (Chapter 29)
Other structures	Other structures (Chapter 29)
Rooftop structures and equipment of buildings with h ≤ 60 feet use equation 6-28 in accordance with the note of section 6.5.15.1	The lateral force (F_h) and the vertical uplift force (F_v) are defined by equations 29.5-2 and 29.5-3 with the gust factor GC_r varying in accordance with the vertical and horizontal project areas (Chapter 29).

TABLE 14.30
Simplified Procedure of ASCE 7-05 and Directional and Envelope Procedures of ASCE 7-10

ASCE 7-05 Method I (Simplified Procedure)	ASCE 7-10 Directional and Envelope Procedures
Simplified method is used only for enclosed simple diaphragm buildings <60 feet high with either a flat or gable end roof with Θ ≤ 45° or a hip roof with Θ ≤ 45° for MWFRS and Θ ≤ 27° for C & C. Wind pressure values are tabulated for walls and roofs at 30 feet, 85–170 mph wind velocity, and Exposure B with the related adjustment factors.	For MWFRSs of enclosed simple diaphragm buildings using the Directional Procedure, the height has been extended to 160 feet. The buildings have been classified as Class 1 and Class 2 buildings. Class 1 buildings have a mean roof height ≤60 feet, while Class 2 buildings have 60 feet < h ≤ 160 feet. Separate tables for roofs (110–200 mph wind velocity at Exposure B) and walls (110–200 mph wind velocity at Exposure C) are provided with related adjustment factors (Chapter 27/Part 2). For MWFRSs of enclosed simple diaphragm low-rise buildings for height ≤60 feet, using the Envelope Procedure, tables (at h = 30 feet and Exposure B) are provided to determine wind pressures for basic wind speeds 110–200 mph. Adjustment factors are used for height and exposure (Chapter 28/Part 2). For the C & Cs of low-rise enclosed buildings with height ≤60 feet, the simplified procedure is used to tabulate wind pressures for roofs and walls evaluated at 30 feet height and Exposure B. The values are tabulated for basic wind speed 110–200 mph and for roofs (0°–7°, 7°–27° and 27°–45°) and walls for effective wind areas 10–100 feet^2. Adjustment factors are used for height and exposure (Chapter 30/Part 2). For the C & Cs of low-rise enclosed buildings with height ≤160 feet, the simplified procedure is used to tabulate wind pressures for roof and wall for Exposure C. The values are tabulated for basic wind speed 110–200 mph and for flat, gable, mansard, hip, and monoslope roofs. Reduction factors for effective wind area and adjustment factors for height and exposure are provided (Chapter 30/Part 4).

14.13 SOLVED EXAMPLES

Example 14.1—Roughness Length Parameter

Problem Statement: A building 50 feet wide and 20 feet high is located on an open lot of 10,000 feet^2. Calculate the surface roughness parameter and determine the exposure category for the wind load calculations.

Solution

Height of the building = 20 feet; width of the building = 50 feet; ground area of building = 10,000 feet^2

Hence, H_{ob} = 20 feet; S_{ob} = 20 × 50 = 1,000 feet^2, A_{ob} = 10,000 feet^2

$$\text{Roughness length parameter } (z_0) = \frac{(0.5)(20 \text{ feet})(1{,}000) = 1.0}{(10{,}000)}$$

Hence, building falls under Exposure B.

Example 14.2—Approximate Natural Frequency—Steel Moment-Resisting Frames

Problem Statement: Calculate the natural frequency of a steel moment-resisting frame building 300 feet tall. Determine the type of the building.

Solution

$$\text{Natural frequency } \eta_a = \frac{22.2}{h^{0.8}} \qquad \text{(ASCE Equation 26.9-2)}$$

$$= \frac{22.2}{300^{0.8}} = 0.232 \text{ Hz} < 1 \qquad \text{(ASCE Section 26.2)}$$

Hence, it is a flexible building.

Example 14.3—Approximate Natural Frequency—Concrete Moment-Resisting Frames

Problem Statement: Calculate the natural frequency of a concrete moment-resisting frame building 300 feet tall. Determine the type of the building.

Solution

$$\text{Natural frequency } \eta_a = \frac{43.5}{h^{0.8}} \qquad \text{(ASCE Equation 26.9-3)}$$

$$= \frac{43.5}{300^{0.8}} = 0.256 \text{ Hz} < 1 \qquad \text{(ASCE Section 26.2)}$$

Hence, it is a flexible building.

Example 14.4—Approximate Natural Frequency—Steel and Concrete Building with Other Lateral-Force Resisting Systems

Problem Statement: Calculate the natural frequency of a steel and concrete building with a masonry lateral resisting system. The building is 300 feet tall. Determine the type of the building.

Solution

$$\text{Natural frequency } \eta_a = \frac{75}{h}$$

$$= \frac{75}{300} = 0.25 \text{ Hz} < 1$$

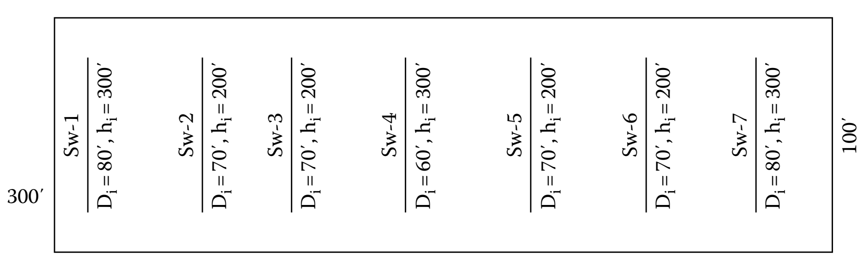
$$\text{Natural frequency } \eta_a = \frac{75}{h} \qquad \text{(Equation 26.9-4)}$$

$$= \frac{75}{300} = 0.25 \text{ Hz} < 1 \qquad \text{(Section 26.2)}$$

Hence, it is a flexible building.

Example 14.5—Approximate Natural Frequency—Concrete Shear Wall Building

Problem Statement: A 300 feet × 100 feet building has seven concrete shear walls in the shorter direction as shown in the figure. The lengths of shear walls are marked on the figure. Calculate the natural frequency and determine the type of the building.



Shear Wall	Length (Feet)	Cross Section Area (Feet²)	Height (Feet)	(Equation 26.9-5) for C_w
SW-1	80	53.6	300	4.23
SW-2	70	53.6	200	3.09
SW-3	70	53.6	200	3.09
SW-4	60	53.6	300	2.46
SW-5	70	53.6	200	3.09
SW-6	70	53.6	200	3.09
SW-7	80	53.6	300	4.23

$$\text{Natural frequency } \eta_a = \frac{385(C_w)^{0.5}}{h} \qquad \text{(ASCE Equation 26.9-5)}$$

$$\text{where } C_w = \frac{100}{AB}\sum_{i=1}^{n}\left(\frac{h}{h_i}\right)2\frac{Ai}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

$$\text{Hence, } C_w = \frac{100}{100\times300}\left[4.23+3.09+3.09+2.46+3.09+3.09+4.23\right] = 0.0776$$

$$\eta_a = \frac{385(0.0776)^{0.5}}{300} = 0.36$$

Hence, it is a flexible building.

Example 14.6—Approximate Natural Frequency—Gust Effect Factor

Problem Statement: A 300 feet × 300 feet building located in Exposure C is made of concrete moment-resisting frame and is 300 feet tall. The basic wind velocity in the region is 150 mph. Find the gust effect factor for concrete moment-resisting frame.

Solution:

$$\text{Approximate natural frequency}(n_a) = \frac{43.5}{h^{0.9}} \qquad \text{(ASCE Equation 26.9-3)}$$

Hence, it is a flexible building. Use ASCE section 26.9.5:

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} \qquad \text{(ASCE Equation 26.9-11)}$$

$$= \sqrt{2\ln(3600\times0.256)} + \frac{0.577}{\sqrt{2\ln(3600\times0.256)}} = 3.85 \qquad \text{(ASCE Section 26.9-4)}$$

$$\bar{z} = 0.6\ h = 0.6(300\ \text{feet}) = 180\ \text{feet}, \quad l = 500\ \text{feet}, \quad \varepsilon = 1/5.0 \qquad \text{(ASCE Table 26.9-1)}$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\varepsilon} = 500\left(\frac{180}{33}\right)^{\frac{1}{5.0}} = 702 \qquad \text{(ASCE Equation 26.9-9)}$$

$$\bar{b} = 0.65,\ \bar{\alpha} = \frac{1}{6.5}$$

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)Y$$

$$= 0.65\left(\frac{180}{33}\right)^{\frac{1}{6.5}}\left(\frac{88}{60}\right)150 = 185.6 \qquad \text{(ASCE Equation 26.9-16)}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\overline{V}_z} = 0.256 \quad \frac{(702)}{(185.6)} = 0.968 \qquad \text{(ASCE Equation 26.9-14)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(0.968)}{(1+10.3(0.968))^{5/3}} = 0.153 \qquad \text{(ASCE Equation 26.9-13)}$$

$$\text{where } R_l = R_h, \quad \eta = \frac{4.6 n_1 h}{\overline{V}_z} = \frac{(4.6)(0.256)(300)}{185.6} = 1.90$$

$$\text{where } R_l = R_B, \quad \eta = \frac{4.6 n_1 B}{\overline{V}_z} = \frac{(4.6)(0.256)(300)}{185.6} = 1.90$$

$$\text{where } R_l = R_L, \quad \eta = \frac{15.4 n_1 L}{\overline{V}_z} = \frac{(15.4)(0.256)(300)}{185.6} = 6.37$$

$$\text{In general, } R_l = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2n}\right) \quad \text{for } \eta > 0 \qquad \text{(ASCE Equation 26.9-15a)}$$

$$R_h = \frac{1}{1.9} - \frac{1}{2(1.9)^2}\left(1-e^{-2(1.9)}\right) = 0.390$$

$$R_B = \frac{1}{1.9} - \frac{1}{2(1.9)^2}\left(1-e^{-2(1.9)}\right) = 0.390$$

$$R_L = \frac{1}{6.37} - \frac{1}{2(6.37)^2}\left(1-e^{-2(6.37)}\right) = 0.144$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B \left(0.53 + 0.47 R_L\right)} \qquad \text{(ASCE Equation 26.9-12)}$$

Use 5% damping ratio, $\beta = 0.05$:

$$R = \sqrt{\left(\frac{1}{0.05}\right)(0.153)(0.39)(0.39)(0.53+0.47\times 0.144)} = 0.36$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1+0.63\left(\frac{600}{702}\right)^{0.63}}} = 0.798 \qquad \text{(ASCE Equation 26.9-8)}$$

$$\bar{I}_z = C\left(\frac{33}{z}\right)^{\frac{1}{6}} \quad C = 0.20 \qquad \text{(ASCE Equation 26.9-7)}$$

$$\bar{I}_z = 0.2\left(\frac{33}{180}\right)^{\frac{1}{6}} = 0.151 \qquad \text{(ASCE Table 26.9-1)}$$

$$G_f = 0.925\left[\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_r I_{\bar{z}}}\right]$$

$$= 0.925\left[\frac{1+1.7(0.151)\sqrt{(3.4)^2(0.798)^2+(3.85)^2(0.36)^2}}{1+1.7(3.4)(0.151)}\right]$$

$$= 0.952 \qquad \text{(ASCE Equation 26.9-10)}$$

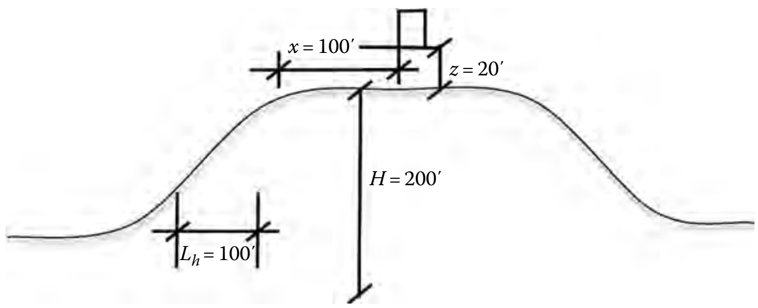
With a 2% damping, $R = 0.678$ and $G_f = 1$.

Now, assume the same building to be a rigid building:

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_r I_{\bar{z}}}\right)$$
$$= 0.925\left(\frac{1+(1.7)(3.4)(0.151)(0.798)}{1+(1.7)(3.4)(0.151)}\right) = 0.84$$

Example 14.7—Approximate Natural Frequency—Topographic Factor

Problem Statement: In a building shown with the profile in the diagram below, determine the topographic factor (K_{zt}).



Solution:

H is the height of hill
L_H is the distance upwind of crest to where the difference in ground elevation is half the height if the hill
x is the distance from the crest to the building site
z is the height above the ground surface at building site

Assume $H = 200$ feet, $L_H = 100$ feet, $x = 100$ feet, and $z = 20$ feet.

$$\left.\begin{aligned}\frac{H}{L_n} &= \frac{200}{100} = 2.0, \quad K_1 = 0.43\\ \frac{x}{L_n} &= \frac{100}{100} = 1.0, \quad K_2 = 0.75\end{aligned}\right\}\text{Figure 26.8-1}$$

$$\frac{z}{L_n} = \frac{30}{100} = 0.3 \quad K_3 = 0.47$$

$$\begin{aligned}K_{zt} &= (1+K_1K_2K_3)^2\\ &= [1+(0.43)(0.75)(0.47)]^2\\ &= 1.33\end{aligned}$$

15 Engineering in Popper's Three Worlds

Engineers deal with human-necessitated changes to the natural environment by means of engineering design and production. Engineering is the application of scientific, economic, social, and practical knowledge in order to invent, design, build, maintain, research, and improve structures, machines, devices, systems, materials, and processes. Engineering came into existence from the very early time after the evolution of humankind. Ancient philosophers like Thales of Miletus (620–546 BC) investigated engineering along with history, science, mathematics, geography, and politics (*Internet Encyclopedia of Philosophy*). However, in the last several centuries, neither philosophy paid much attention to engineering nor engineering paid much attention to philosophy. By and large, engineers have tended to ignore philosophical analysis and reflection as trivial to engineering practice. Some exceptions to this rule can be seen in engineers and philosophers like Leonardo da Vinci, Benjamin Franklin, and Ludwig Wittgenstein. Philosophers too did not make significant efforts to appreciate and assess the role of engineers in the advancement of the society and the world. Engineering and philosophy were almost mutually exclusive domains and were treated as if they did not have much in common between them. With rapid advancement in engineering in the modern era, the philosophy of engineering dealing with the ethics, ontology, and epistemology of engineering is emerging. The philosophy of engineering considers what engineering is, what engineers do, and how they impact the society. The engineering of artifacts has implications that extend into areas like psychology, finance, and sociology and should take into consideration the conditions of manufacture, use, and disposal. This should compel the philosophers to study the impact of engineering.

ENGINEERING AS A SOCIAL TOOL

Though engineering is part of the wider field of technology, it should be considered as a social discipline because of its influence on society. For example, engineers make elaborate attempts to align the environment by making sure that the use of carbon is less by involving more natural processes. The work of civil engineers improves public health and quality of life, which is their prime responsibility. They make a direct and immediate impact on the society by designing and constructing sustainable and green projects. Civil engineers develop our society by designing and building infrastructures that ensure high rigidity. They design and build schools, colleges, malls, theaters, bridges, water supply systems, drainage systems, sewage systems, and transportation systems. Reliability has become an important factor in the design of civil engineering structures for resistance against natural calamities like earthquakes, hurricanes, flood, tornadoes, snow storms, and tsunamis. As such, reliability is an important factor for all branches of engineering, whether hard or soft.

Some Major Fields of Engineering and Philosophy

Engineering	Philosophy
Civil	Ethics
Structural	Metaphysics
Mechanical	Political philosophy
Electrical	Epistemology
Metallurgical	Logic
Chemical	Social philosophy
Mining	Axiology
Nuclear	

SCIENCE AND ENGINEERING

> Scientists study the world as it is, engineers create the world that has never been seen before—Theodore von Karman[1]

An engineering project begins in response to social requirements. The necessity of traveling fast led to the invention of automobiles and airplanes. The necessity of talking to others located at an inaudible distance led to the invention of telephones. The necessity of fast data processing led to the invention of computers and software. This corroborates with the proverb, "Necessity is the mother of all inventions" (author unknown). However, there has been a criticism to this proverb. Alfred North Whitehead, in his address to the Mathematical Association of England in 1917, argued that the basis of invention is science, and science is almost wholly the outgrowth of pleasurable intellectual curiosity (Sigler, 1996). Science definitely aids engineering in technological inventions, but these inventions are made to fulfill the requirements of the society.

Engineering knowledge, though pursued at great effort and expense in schools of engineering, receives little attention from scholars of other disciplines.(2) Engineering was treated as an extension of sciences to make artifacts, even though engineering has a different ontological basis with its theories addressing different entities and being judged with different criteria than science. English historians Singer, Holmyard, and Hall (1954) and French historian Daumas (1969) have treated technology as a technique and technologists as technicians. They treat basic science as the source of all new technical knowledge, and the role of technologists is to apply this knowledge generated by the scientists. This conviction is because of their mistaken images about science and technology. However, in the past few decades, many books have been published in which the distinction between science and engineering as well as significant practical activities of engineering have been recognized.

Since the latter half of the nineteenth century, the fact that science has greatly influenced engineering emerged. For example, various concepts of physics like mechanics, dynamics, statics, cryogenics, and soil physics are extensively used in the engineering of building structures. There is a close relationship between engineering and science, medicine, biology, and art. Engineering and science overlap to a certain degree as both use mathematics and classification criteria to analyze and communicate observations. Medicine aims to sustain, enhance, and even replace functions of the human body through the use of technology. However, a broad distinction between engineering and science must be established.

Staples (2014) argued that the biggest misconception about engineering is that it is treated as an applied science. Though engineers use scientific theories, they also use theories that are not acceptable in science. Engineering theories could be phenomenological and approximate and have limited scope.

Engineering methods are driven based upon set parameters and the design criteria established before the process begins. If the parameters and the design criteria change, then the methodology might also change. Scientific methodologies are universal. Engineering targets requirements, whereas science targets truth.

EPISTEMOLOGY OF ENGINEERING

Philosophers and historians do not have sound knowledge of engineering and, as discussed in the last section, generally treat engineering as an applied science. Engineering knowledge has not fully emerged as an epistemology. The cognitive dimension of engineering has not been fully examined and recognized by the philosophers. At the best, the philosophy in engineering has been explored with regard to ethical questions until recently (van de Poel, 2010). At the same time, there is little philosophical introspection by engineers in the practice of their own profession. The challenge of engineering education today is not only to produce experts in engineering disciplines but also to produce engineers who can reflect and make conjectures to advance engineering knowledge. Even though engineering educators unanimously believe that critical thinking among students needs to be promoted, there are only a few engineering colleges that mandate courses in philosophy as core courses (Paul and Elder, 2002).

Several engineering scholars have come up with different definitions of engineering. The terms “design” and “technology” have been synonymously used with the term “engineering.” For example, the work done by structural engineers is called “structural design.” The most elite engineering colleges in India have the term “technology” in their names—“Indian Institute of Technology” and “National Institute of Technology.” However, in the United States, people holding a degree as a technologist are either not licensed as engineers in some states or have more rigorous experience requirements to be licensed as engineers in other states (Smith, Berson and Balasio, 2013). In the United States, a technologist can use the title “designer” but no person, unless licensed in engineering, can use the title “engineer.” Globally, there are mix-ups in the terminologies of design, technology, and engineering, but in this essay these terms are used synonymously.(3)

Layton (1974), a Case Western Reserve University professor, in his paper “Technology as Knowledge,” classifies engineering as having a significant component of thought of its own. Though engineering uses sciences, it is creative and constructive on its own. de Figueiredo (2008) defined engineering epistemology in four major dimensions—basic sciences, social sciences, design, and practical realization. Engineering design is a methodical series of steps engineers use in creating functional artifacts. When these artifacts are invented, they serve the society and facilitate its growth. Design is a highly iterative process, and historically, it is always evolving and improving upon its previous version. The Accreditation Board for Engineering and Technology (ABET) defines engineering design as an iterative process in which the basic sciences and mathematics are applied to convert resources optimally to meet a stated objective.(4) The design objectives are set; criteria and parameters are established; synthesis and analysis are performed; and then the product is constructed, tested, and evaluated.

According to Staples (2014), the five most recurring terms in engineering epistemology are artifacts, requirements, theories, design, and judgment. Artifacts include structures, machines, apparatus, manufacturing processes, and more recently robots, computers, and computer software. Artifacts are central to engineering, which are evaluated according to requirements. Requirements address intended functions, economics, life, and property safety, and also social necessities. Theory is contemplative and rational thinking, or the results of such thinking that support engineering design. Design is the creation of a plan to construct an artifact. It is a creative process that requires problem definition, idea searches, and solution development (Stephan, Bowman, Park, Sill and Ohland, 2013). It is performed using models and design philosophies that determine design goals.

These goals guide the design of solving a miniature problem of a small element to the most holistic element. Engineering judgment is the ability to make sound design choices based on experience and intuition. Engineers should be educated to understand what knowledge is and what they do intuitively.

The structure of engineering knowledge can be simple, consisting of isolated pieces of information, or complex, consisting of pieces of information that are dependent upon each other. The engineering knowledge is continuously evolving based upon social requirements. In the present world environment, I propose that the epistemology of engineering broadly falls into three paradigms:

1. Engineering that uses basic sciences and mathematics
2. Engineering as sociology
3. Engineering as a tool for design

Engineering uses both sciences and mathematics. Science sets the limits that cannot be breached. Engineers use the tool of math and scientific knowledge in their efforts to come up with novel solutions for the problems at hand. Science, math, and engineering complement each other in solving problems, but neither scientific knowledge nor math is directly applied to those problems without engineering. Engineers solve real-world problems. Throughout history, engineers have been "agents of change" and "civilization movers." Engineers and the engineering community generally display the professional standards of openness of mind and honesty, though they need to be certain about the outcome of their activities, which in some cases induces a degree of dogmatic conviction in them with regard to their abilities. Engineers themselves are influenced by cultural and personal factors, such as cultural norms and their own experiences. Engineers study the world of which they are a part, and as such their work is accessible and assessed by the public, making it objective. People from all cultures contribute to engineering. Hence, it is important that these convictions and certainties be moderated by the help of introducing discussions from humanities, particularly philosophy.

Engineering design is problem solving, where complex problems can be simplified to levels where they can satisfy minimal criteria for positivist solutions. Engineering design is problem setting, which views design as requiring the discovery and negotiation of unstated goals, implications, and criteria, following constructivist epistemologies. Engineering is about designing artifacts and systems to change the world and overcome resistance and ambiguity. Engineering artifacts can be viewed as entities developed by key engineering design processes. Engineers need to study aesthetic concepts and incorporate them in engineering design decisions. Engineering design refers to modern, industrialized design—distinct from preindustrial, craft-oriented design—based on scientific knowledge, but utilizing a mix of both intuitive and nonintuitive design methods. The development methods of engineering are design, invention, and production. Engineering artifacts should have the appeal of being well made and well functioning at its very early interaction with the customer. Engineering responds to noncognitive needs or acts as a tool for facilitating the cognitive pursuits of the customers, and in both cases they need to fulfill their demands.

Engineers have contributed a lot in the process of development of this world, and it is high time that engineers become reflective and philosophical. It will enhance their functionality in the society. Many philosophical concepts can be adopted for engineering thinking and judgment. Magee (1985) quoted medical Nobel laureate Sir John Eccles, "My scientific life owes so much to my conversion in 1945, if I may call it so, to Popper's teaching on the conduct of scientific investigations… I have endeavored to follow Popper in the formulation and in the investigation of fundamental problems in neurobiology." Eccles was a Cartesian dualist and later converted into a trialist, influenced by Popper's three world ontology, which influenced his attitude toward work and contributed to the success of his neuroscience research. There are other Nobel laureates like Sir Peter Medawar and Jacques Monod who have publicly acknowledged Popper's influence on their work and success (Magee, 1985). When doctors can get influenced by Popper and attain success of very high magnitude, engineers should also move toward philosophy to enrich their profession and take it to new heights. It is worth mentioning that one of the most prominent structural engineers of modern times, Fazlur Khan, looked for guidance in the work of poet and philosopher George Santayana. As Khan's interest in philosophy grew, he intertwined his philosophical sensibility with his practical and logical approach to life (Khan, 2004). With his simplistic approach to building design, he changed several concepts of building structural engineering. In the subsequent section, the application of Popper's three world ontology to engineering is discussed.

Popper's Three Worlds Applied to Engineering

Karl Popper introduced the concept of his three worlds (Popper, 1978). World 1 consists of physical objects and phenomena. World 2 consists of subjective experience and mental phenomena. World 3 consists of mathematical structures ("truth in themselves"). These three worlds form a base for developing his philosophical views. According to Popper, scientific hypothesis cannot be validated; they can only be falsified. Popper has repeated at least on three occasions in his *The Poverty of Historicism* that in the world described by physics nothing can happen that is truly and intrinsically new (Popper, 1957). Newness in physics is merely the newness of the physical arrangements of various components. However, Popper was always in favor of novelty and innovative ideas.

The significance of Popper's concept of three worlds in engineering lies in the potential of tentative theories or design being formulated and improved over time. Engineering theories can be refuted with better research. Designs could be enhanced to cater to the evolving social needs. An example of such processes is the design of high-rise buildings. The first high-rise building more than 100 stories tall is the Empire State Building constructed in 1931. The structural Engineer of Record used conventional beam-column design.[(5)] Engineer Fazlur Khan improved upon the conventional design of beam-column methodology when he designed the Sears Tower (now Willis Tower) in 1973 using tubular structure methodology.[(6)] Subsequently, this methodology was used in the design of high-rise buildings like the Petronas Tower in Malaysia and the Burj Khalifa in Dubai.[(7)] Currently being constructed, the kilometer-tall Kingdom Tower in Jeddah posed more problems in its design because of the height and action of wind. Engineers came with a solution of providing triangular shape to the building to reduce wind effects.[(8)]

In the design and construction of high-rise buildings, all physical objects like concrete, steel, wood, glass, and paper to prepare the drawings; equipment like cranes and compactors; computer to process the design information; and printer to plot the drawings belong to Popper's World 1. The established design theories, principles, and methodologies are decided based upon the conditions in which the projects are located.[(9)] They can be treated as subjective and the application requires human experience and intellect. This forms World 2. Now, with the interaction between these worlds, a third world is created. By applying World 2 to the physical objects of World 1, World 3 is realized. During this process, new engineering principles and methodologies may emerge, which are superior to the existing methodologies. Even though the physical aspect of the buildings designed and constructed belong to World 1, the functionality, the form, and the new methodologies developed while dealing with the problems during the design and construction belong to World 3.

The main point of World 3 objects is that they are human creations and hence they can be improved for most part through the dynamic interaction between the three worlds, as demonstrated in the example of the design of high-rise buildings. It would be ideal to introduce the concept of Popper's critical rationalism (CR) at this juncture. CR is a philosophy introduced by Karl Popper concerning how we approach knowledge. It takes the stance that there are no ultimate answers, but knowledge is nevertheless possible, and truth is an endless quest. Popper proposed that anything can never be fully justified, but we can merely filter bad ideas and work with the residue. He stated that science moves forward through a method of conjectures and refutations. It has an attitude that no one has a monopoly on the truth; therefore, let's work together to get closer to it. CR has not been tapped to its potential. It can be applied to most disciplines like engineering. The example of the development in the design methodology of high-rise building (discussed above) demonstrates Popper's philosophy that we learn through mistakes or by mistakes of others and improve upon prevailing theories and concepts.

Applying the concept of CR to Popper's World 3 objects, ideas, theories, and methods developed by engineers are not assessed for their truth values but for their utility and pragmatic values. These ideas, theories, and methods, after their creation, have a limited life duration till they are refuted or improved upon by engineers using them. For example, Newton's laws were undisputed for more than two centuries. Even though Newton's laws are very widely used even in the current times, Newtonian physics has been superseded by relativistic physics and quantum physics.[(10)] Likewise, in engineering, all design theories and methodologies are tentative and subject to error elimination under Popper's schema. Engineering design is more concerned with notions of utility and significance than with issues of truth. Even though the physical artifacts of design follow the laws of nature and the success of simulation, modeling and prototypes are physical, the idea that produces artifacts falls under the realm of Popper's World 3. Popper's three worlds provide engineers alternative ways to conceptualize engineering. Engineers should employ Popper's paradigm as a foundation for the pedagogical model for engineering epistemology. In current engineering practice, too much focus is given to World 2 objects with no emphasis on World 3 objects. There is very little emphasis during the training and education of engineers on questions such as follows: Why are we using this concept? What does this engineering concept do or fail to do? How can this engineering concept be improved? Engineers evaluate the success of ideas by subjecting them to criteria like feasibility, viability, and desirability (Brown, 2009). These criteria serve the social and commercial purposes but have very little contribution to the enhancement of the epistemology of engineering and the promotion of the development of the capacity of human judgment and self-reflection.

Popper's philosophy could help engineers by combining engineering objects of all three worlds. A model for engineering epistemology and practice is proposed using Popper's three worlds.

Issues, challenges, and problems arise in World 1. These problems in the field of engineering have commercial and social nature. As explained earlier, enhancement of the functionalities of artifacts or inventions of artifacts become social necessities. Engineers are employed to seek resolutions to these problems. Resolution begins with the process of forming initial ideas (World 3) with the aid of the available engineering epistemological resources. The epistemic stock is supplied from World 2. Through the rendering of the initial ideas on the physical objects (World 1) using the available epistemology (World 2), various methods of engineering are used to perform iteration of potential solutions. These initial ideas would also aid the engineers to understand the problem in a better manner. According to Popper, during these iterations, the ideas that are refuted or falsified lead to more knowledge.[(11)] This process in turns caters to the development of new epistemological resources, which can be consolidated into the epistemological stock. This stock is dynamic in nature because the knowledge that is falsified completely or enhanced upon gets eliminated from the epistemic stock and new knowledge becomes the member of this knowledge bank.

Basically, the three worlds of Popper interact with each other reciprocally through the conscious human mind (World 2). Changes in one world influence the other worlds. The key task of engineers is to appreciate the problems and the challenges at hand and adopt the appropriate epistemic frames for collaborative knowledge construction. In this proposed model, all engineering knowledge developed till now can be used to improve the engineering environment. All the elements of World 2 are epistemic resources, and they should be treated as areas that can be enhanced and improved.

CONCLUSION

This essay examines the epistemology of engineering and proposes a model to broaden the engineering epistemology to include engineering design epistemology that gives importance to creativity, collaboration, and design thinking. Engineering knowledge creation process can be explicated using Popper's ontology of three worlds of objects. Engineering concepts (World 3) like new engineering ideas are products of human minds that result from thinking, experience, and established theories and methodologies (World 2) and are encrypted through physical artifacts (World 1). Examining engineering knowledge from this perspective requires engineers to undergo basic courses of philosophy and critical thinking. Engineering has many philosophically interesting problems (other than the issues of ethics). Engineering is based upon rational reasoning of the physical world. Engineers should equip themselves to give solutions to these philosophical engineering problems. Engineers should develop methodologies based upon Popper's philosophy of falsification of engineering theories and the growth of knowledge in engineering. Philosophy should be introduced in engineering curricula as core courses. This will enhance the profession, raise the bar of practicing engineers, and eliminate the routine dialogue in engineering meetings, "Let's cut all this philosophy and get back to business."

GLOSSARY

Phenomenology is the study of structures of consciousness as experienced from the first-person point of view. The central structure of an experience is its intentionality, its being directed toward something, as it is an experience of or about some object. Founded by German philosopher Edmund Gustav Albrecht Husserl.

Critical rationalism is the proposition that scientific theories, and any other claims to knowledge, can and should be rationally criticized and, if they have empirical content, can and should be subjected to tests which may falsify them. It was proposed by Austrian philosopher Karl Popper.

Ontology is the philosophical study of the nature of being, becoming, existence, or reality, as well as the basic categories of being and their relations. Traditionally listed as a part of the major branch of philosophy known as metaphysics, ontology deals with questions concerning what entities exist, or may be said to exist, and how such entities may be grouped, related within a hierarchy, and subdivided according to similarities and differences. Although ontology as a philosophical realm is academic in the sense that it is inseparable from each thinker's epistemology, it has practical application in information science and information technology.

Epistemology is the branch of philosophy concerned with the nature and scope of knowledge and is also referred to as "theory of knowledge." It is the study of knowledge and justified belief. It questions what knowledge is and how it can be acquired and the extent to which knowledge pertinent to any given subject or entity can be acquired. Much of the debate in this field has focused on the philosophical analysis of the nature of knowledge and how it relates to connected notions such as truth, belief, and justification. The term was first used by the Scottish philosopher James Frederick Ferrier.

Objectivity is a central philosophical concept, related to reality and truth, which has been variously defined by sources. Generally, objectivity means the state or quality of being true even outside of a subject's individual biases, interpretations, feelings, and imaginings. A proposition is generally considered objectively true when its truth conditions are met and are "bias-free."

Subjectivity is a central philosophical concept, related to consciousness, agency, personhood, reality, and truth, which has been variously defined by sources. Something being a subject, narrowly meaning an individual who possesses conscious experiences, such as perspectives, feelings, beliefs, and desires. Something being a subject, broadly meaning an entity that has agency, meaning that it acts upon or wields power over some other entity (an object). Some information, idea, situation, or physical thing considered true only from the perspective of a subject or subjects.

Positivism is the philosophy of science that information derived from logical and mathematical treatments and reports of sensory experience is the exclusive source of all authoritative knowledge, and that there is valid knowledge (truth) only in this derived knowledge. Verified data received from the senses are known as empirical evidence. Positivism holds that society, like the physical world, operates according to general laws. Introspective and intuitive knowledge is rejected, as is metaphysics and theology. Positivism in the modern times was developed by French philosopher Auguste Comte.

Constructivism is a theory of knowledge that argues that humans generate knowledge and meaning from an interaction between their experiences and their ideas. It has influenced a number of disciplines, including psychology, sociology, education, and the history of science. Swiss philosopher Jean Piaget founded constructivism.

Realm is a community or territory over which a sovereign rules; it is commonly used to describe a kingdom or other monarchical or dynastic state.

Pedagogy is the discipline that deals with the theory and practice of education; it thus concerns the study and practice of how best to teach. Its aims range from the general (full development of the human being via liberal education) to the narrower specifics of vocational education (the imparting and acquisition of specific skills).

NOTES

(1) Alan L. Mackay has used this quote in his work *A Dictionary of Scientific Quotations* (1994), published by CRC Press, New York.

(2) Though knowledge is a term reserved for understanding reality, in engineering it is the know-how used to change reality.

(3) It is not the intention of the author to breed confusion among the terminologies of engineering, technology, and design. Here these terms are used to explain a demarcation between science and engineering. As such, design in engineering is different than in interior decoration. Technology covers a much wider range of activities than engineering. A bank clerk is a technologist, so is a cook.

(4) The Accreditation Board for Engineering and Technology (ABET), located in Washington, DC, is an institute that internationally evaluates the engineering academic curriculum.

(5) The conventional beams and columns structure has a load path of the slabs being supported by the beams, which in turn are supported by the columns. The columns transfer the load to the soil through the foundations. The lateral loads are resisted by the beam-column frames or shear walls.

(6) Khan invented the tubular structures, which consists of concentric shear walls that carry both the gravity and lateral loads.

(7) The Petronas Towers was designed by a New York–based structural engineering firm, Thornton Tomasetti. Burj Khalifa, which is currently the tallest building in the world, was designed by Skidmore, Owings & Merrill, LLP, where Fazlur Rahman Khan worked as a partner.

(8) The Kingdom Tower required a new thinking in the designing to reduce wind effects as the height of the building is 1 km. The two towers share a similar three-petal triangular footprint for stability and a tapering form, with sheer height and wind being the biggest structural design challenge. The smooth, sloped façade of Kingdom Tower particularly induces a beneficial phenomenon known as "wind vortex shedding," whereas normally when wind swirls around the leeward side of a building, rushing in from both sides to fill the low pressure zone, it would create tornado-like vortices, which would rock the building from side to side due to variations in pressure, direction, and velocity. The dynamic façade of Kingdom Tower creates an infinite timing differential in air pressure exertion in any one particular direction, thus creating a more stable structure, as there is no broad area of outstanding pressure or depression at any given time. Put simply, a smooth taper is more aerodynamic than an irregular or jagged taper, while both are advantageous over rectangular geometries. At Kingdom Tower's height, it is considered essentially unfeasible to use a traditional square design.

(9) The design theories of buildings at different locations are different. In Los Angeles, there is a dominance of the seismic effects on the design. In Miami, wind loads govern. In Chicago, the snow loads govern; and in Dubai, the protection of the structural elements against corrosion plays an important part of the design.

(10) Newton's laws assumed instantaneous effect of gravity, and Einstein's theory of relativity had gravity warping of space. At the time of Newton, the modern mathematics was rudimentary, statistics and probability had not been studied, and even equations of wave were unknown. In physics, the concept of energy had not been explored.

(11) For example, today is actually Sunday and someone says that today is Monday. Then during the falsification of the statement "Today is Monday," we gain knowledge that today is not Tuesday nor Wednesday nor Thursday nor Friday nor Saturday.

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Appendix A: Main Design Concepts

A.1 PROBLEM A.1: BENDING AND SHEAR STRESSES

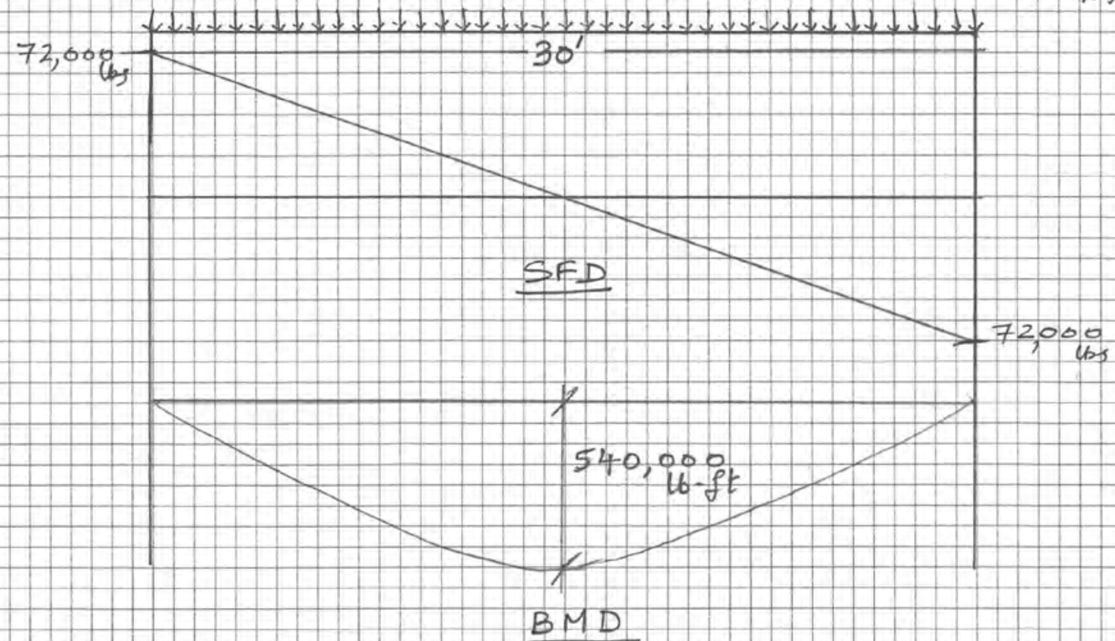
ACE CONSULTING ENGINEERS
(STRUCTURAL & CIVIL ENGINEERING, FORENSIC & EXPERT WITNESS)

1111 ABC Road | New York, NY
Phone: (000) 000-0000 | Fax: (000) 000-0000
www.abc.com

PROJECT: MAIN DESIGN CONCEPTS				Design
SUBJECT: BENDING & SHEAR STRESSES			SHEET NO. 1/1	Inspection
JOB NO: PROB: 1.1	DATE:	DESIGNED BY: MA	OF SHEETS	Investigation
				Reports

ACI Code

Dead Load — 2000 lb/ft
Live Load — 1500 lb/ft.
Factored Load — (1.2)(2000) + (1.6)(1500) — 4800 lb/ft.



Max: S.F = $4800 \times \frac{30}{2}$ = 72,000 lbs

Max: B.M = $4800 \times \frac{30^2}{8}$ = 540,000 lb-ft

Size of beam = 12" × 48"
Reinforcement — 4 #8 bars
Clear cover — 1.5"
Effective depth (d) — 48" − 1.5" − 1/2" = 46"
Ratio of steel (ρ) — $\frac{As}{bd}$ — $\frac{4 \times 0.786}{12 \times 46}$ = 0.0057

Use 6,000 psi concrete
Modulus of Elasticity (E_c) = $57{,}000\sqrt{f'_c}$ = 4.42×10^6 psi 19.2.2.1b

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Modular Ratio $(n) = E_s/E_c = \dfrac{29\times10^6}{4.42\times10^6} = 6.56$

$\rho n = (0.0057)(6.56) = 0.0374$

$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$

$= \sqrt{(0.0374)^2 + 2(0.0374)} - 0.0374 = 0.239$

$j = 1 - k/3 = 0.921$

stress in concrete $(f_c) = \dfrac{2M}{kjbd^2} = \dfrac{(2)(540{,}000)(12)}{(0.239)(0.921)(12)(46)^2}$

$= 2319\text{ psi} < 6000\text{ psi}$

stress in steel $(f_s) = \dfrac{M}{A_s jd} = \dfrac{(540{,}000)(12)}{(4)(0.786)(0.921)(46)}$

$= 48{,}649\text{ psi} < 60{,}000\text{ psi}$

Average Shear Stress $= \dfrac{S.F.}{bh} = \dfrac{72{,}000}{12\times48} = 125\text{ psi}$

Max. Shear Stress $= \dfrac{3}{2}\dfrac{S.F.}{bh} = \dfrac{3}{2}\times\dfrac{72{,}000}{12\times48} = 187.5\text{ psi}$

A.2 PROBLEM A.2: BENDING STRESS

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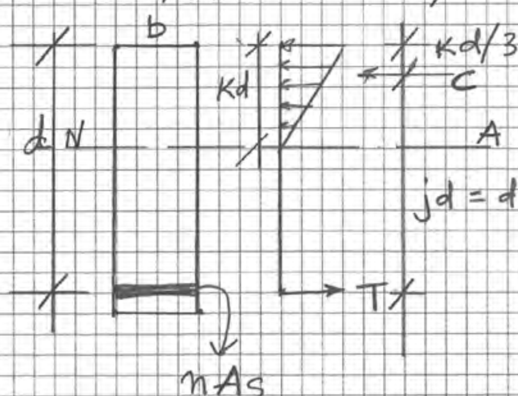
Size of beam — 8" × 36"
Condition — One-span simply supported
Reinforcement — 2#8
f'_c — 4,000 psi
f_y — 60,000 psi
Clear cover — 1.5"

Determine ultimate moment at which beam will fail?

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4{,}000} = 3.6\times10^6$ psi

$n = E_s/E_c = 29\times10^6 / 3.6\times10^6 = 8.06$



$jd = d - Kd/3$

$d = 36'' - 1.5'' - \frac{1''}{2}$
$= 34''$
$b = 8''$

nAs — cracked transformed section

Taking moment about the neutral axis (N–A);

$$b\frac{(Kd)^2}{2} = nA_s(d - Kd)$$

$$\frac{8K^2(34)^2}{2} = 8.06(2)(0.786)(34 - 34K)$$

$$4624K^2 = 430.8 - 430.8K$$

$$4624K^2 + 430.8K - 430.8 = 0$$

$$10.73K^2 + K - 1 = 0 \;;\quad K = 0.262$$

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$j = 1 - k/3 = 1 - (0.262/3) = 0.913$

Compression $(C) = \frac{f'_c}{2} bKd = \left(\frac{4000}{2}\right)(8)(0.262)(34)$

$= 142{,}528$ lbs

Tension $(T) = A_s f_y j d = (2)(0.786)(60{,}000)(0.913)(34)$

$= 2{,}927{,}881$ lbs

$M = Cjd = \frac{(142{,}528)(0.913)(34)}{12 \times 1000} = 368.9$ K-ft

$M = Tjd = \frac{(2{,}927{,}881)(0.913)(34)}{12 \times 1000} = 7{,}573.9$ K-ft

Governing moment — 368.9 K-ft

Beam will fail in compression

Beam is over reinforced

A.3 PROBLEM A.3: BENDING STRESS

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Solve problem 1.2 using equivalent rectangular stress block.

Effective depth $(d) = 36'' - 1.5'' - \frac{1''}{2} = 34''$

Area of steel $(A_s) = (2)(0.786) = 1.572 \text{ in}^2$

Using a rectangular stress block,

Ultimate moment capacity $(M_n) = A_s f_y \left(d - \frac{0.59 A_s f_y}{f'_c b}\right)$

$= (1.572)(60{,}000)\left[34 - \frac{(0.59)(1.572)(60{,}000)}{(4{,}000)(8)}\right]$

$= 3{,}042{,}855$ lb-in $= 254$ K-ft

Appendix B: Materials

B.1 PROBLEM B.1: CONCRETE MIX DESIGN

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Prepare a mix design for concrete to be used for parking garage floor in an extreme ocean front condition in a tropical zone.

The 28 days compressive strength of concrete shall be 5000 psi.

Assume that the concrete production facility does not have field strength data available

Required average field strength

$f'_{cr} = 1.10 f'_c + 700$ — Table 5.3.2.2

STEP(1)

Maximum slump of concrete – 3" — Table 6.3.1 ACI 211.1

STEP(2)

Choose a nominal aggregate size of 3/4". It shall be ascertained that it is locally available

STEP(3)

Since the building is located in a tropical zone, air-entrained concrete is not required

Water required = 340 lb/yd^3 — Table 6.3.3 ACI 211.1

For non-entrained concrete with $f'_{cr} = 6200$ psi

water-cement ratio = 0.41 — Table 6.3.4(a) ACI 211.1

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STEP(5)

$$\text{Cement content} = \frac{\text{water content}}{\text{water-cement ratio}}$$

$$= \frac{340}{0.41} = 829.3\ lb/yd^3$$

STEP(6)

Assume a fineness moduli of fine aggregate as 2.6

Volume of 3/4" nominal maximum size coarse aggregate = 0.64 of total volume of concrete — Table 6.3.6 ACI 211.1

For $1\,yd^3$, coarse aggregate $= 0.64\,yd^3$

$= 17.28\,ft^3$

Unit weight of dry coarse aggregate $\simeq 100\ \frac{lb}{ft^3}$

Weight of dry coarse aggregate for $1\,yd^3$ of concrete = 1728 lbs

STEP(7): (Based on weight)

$1\,ft^3$ of concrete weighs — 144 lbs

$1\,yd^3$ of concrete weighs — 144×27

— 3888 lbs

Weight of water — 340 lbs (step 3)

Weight of cement — 829 lbs (step 5)

Weight of coarse aggregate — 1728 lbs (step 6)

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Hence, weight of fine aggregate

$= 3888 - 340 - 829 - 1728 = 991$ lbs

STEP (7) : (Based on volume)

Volume of water $= \frac{340}{62.4} = 5.45\ ft^3$ (Step 3)

Volume of cement $= \frac{829.3}{3.15 \times 62.4} = 4.22\ ft^3$ (Step 4)

(Specific gravity of Portland cement = 3.15)

Volume of coarse aggregate $= \frac{1728}{2.68 \times 62.4} = 10.33\ ft^3$

(To calculate the solid volume of coarse aggregate, the specific gravity of 2.68 is used)

Hence, volume of fine aggregate for 1 yd^3 of concrete $= 27 - 5.45 - 4.22 - 10.33 = 7\ ft^3$

Weight of fine aggregate $= 7 \times 2.64 \times 62.4$

$= 1153$ lbs

(Specific gravity of fine aggregate = 2.64)

STEP (8)

If tests indicate that the coarse aggregates contain 3% moisture and fine aggregates contain 5% moisture, then adjusted weights:

Coarse aggregate, wet = (1728)(1.03) = 1780 lbs

Fine aggregate, wet = (991)(1.05) = 1040 lbs

Weight of water needs to be adjusted

340 − (1728)(0.03) − (1040)(0.05) = 235 lbs

Appendix C: Design Loads

C.1 PROBLEM C.1: BUILDING (1) LOADS

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Use the fig: 1.5.1 through 1.5.4 to perform the load calculations of this building. Read sections 1.5 and 1.6 of the book. In this problem only live loads and dead loads are considered.

There is no allowance for live reduction

All beams are 8"x16" except second floor beams at grids B, C & D

All columns are 16"x16"

Clear floor height between bottom of the beam and top of the slab below is 9'

Height of parapet wall is 3' (8" C.M.U.)

All slabs are 6" thick

Assume density of concrete 150 pcf

Roof dead load is 25 psf

Roof live load is 30 psf

Floor dead load is 25 psf (inclusive of partitions)

Floor live load is 40 psf

Weight of masonry is 65 psf.

Clear height of exterior masonry walls is 9'

Since the second floor beams at grid lines 'B', 'C' and 'D' are supporting columns from above they are transfer beams – 16"x 36"

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(a) Calculate the service loads and factored loads of roof slabs S-1 and S-2 (one-way)

Self wt. of slabs $- \frac{6''}{12''} \times 150$ pcf $- 75$ psf

Super-imposed dead loads $- 25$ psf

Total dead load $- 100$ psf

Live load $- 30$ psf

Total service load $- 100 + 30 - 130$ psf

Factored Load $- (1.2)(100) + (1.6)(30) - 168$ psf

(b) Calculate loads on the roof beams at grid lines '1' and '2'

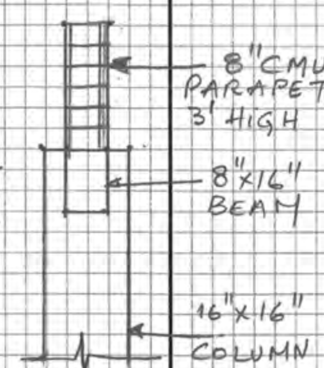
These beams do not support the slab. Supports self weight and the masonry parapets.

Self weight $- \frac{8'' \times 16''}{12'' \times 12''} \times 150$ pcf $- 133$ plf

Parapet $- 3' \times 65$ psf $- 195$ plf

Total dead load $- 133 + 165 - 328$ plf

Factored load $- (1.2)(328) - 394$ plf



(c) Calculate loads on the roof beams at grid lines 'A' and 'F'

These are 5-span beams supporting the parapet walls, self-weight and half the dead and live loads from one-way slab panels (S-1) like above,

Self weight $- \frac{8'' \times 16''}{12'' \times 12''} \times 150$ pcf $- 133$ plf

Parapet $- 3' \times 65$ psf $- 195$ plf

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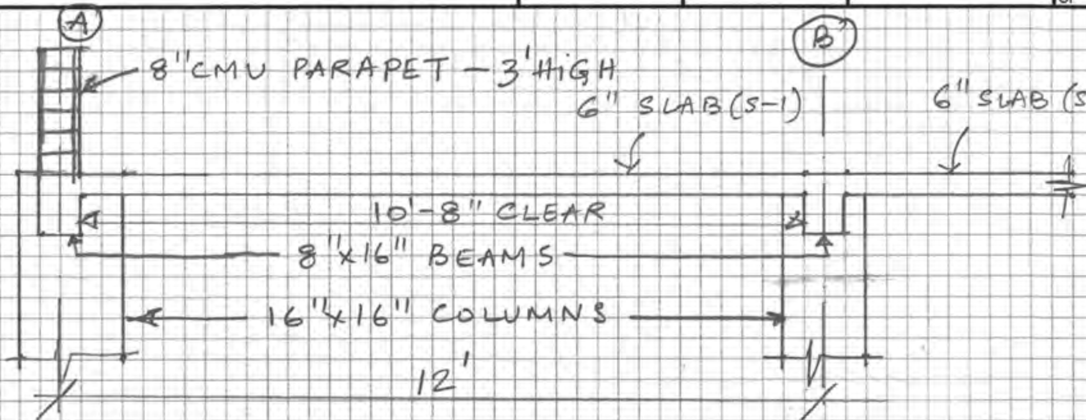

Dead load from slab — 100 psf × $\frac{10'\text{-}8''}{2}$ — 534 plf   from prob: (a)

Total dead load — 133 + 195 + 534 — 862 plf

Live load from slab — 30 psf × $\frac{10'\text{-}8''}{2}$ — 160 plf

Total Service load — 862 + 160 — 1,022 plf

Factored Load — (1.2)(862) + (1.6)(160) — 1,290 plf

Loads on beams of grid lines 'A' and 'F' are identical

(d) Calculate loads on the roof beams at grid lines 'B' and 'E'

There are no parapets on these beams. From the slabs, they receive half the load of S-1 and half the load of S-2, the adjacent slab panels.

Tributary width from slab (S-1) — 10'-8"/2 — 5.34'   Total width = 5.34+6.34' = 11.68'

Tributary width from slab (S-2) — 12'-8"/2 — 6.34'

Self wt. of beam — $\frac{8'' \times 16''}{12'' \times 12''}$ × 150 pcf — 133 plf

Dead load from slab — 100 psf × 11.68' — 1168 plf   from prob: (a)

Total Dead Load — 133 + 1168 — 1301 plf

Live load from slab — 30 psf × 11.68' — 350 plf

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Total service load = 133 + 1168 + 350 = 1651 plf

Factored load = (1.2)(1301) + (1.6)(350) = 2121 plf

Loads on roof beams of grid lines 'B' and 'E' are identical because of symmetry in the building.

(d) Calculate the loads on roof beams at grid lines 'C' and 'D'

There are no parapets on these beams. They receive equal loads from the two adjacent panels because the clear spans of each panel is 12'-8".

Self wt. of beam = $\frac{8'' \times 16''}{12'' \times 12''} \times 150$ pcf = 133 plf

Dead load from slab = 100 psf × 12'-8" = 1267 plf

Total dead load = 133 + 1267 = 1400 plf

Live load from slab = 30 psf × 12'-8" = 380 plf

Total service load = 1400 + 380 = 1780 plf

Factored load = (1.2)(1400) + (1.6)(380) = 2288 plf

(e) Calculate loads on the base of the second floor columns.

The column layout is typical for 9 floors. Eight of the columns curtail at the second floor — grid lines '2' and '5'. Please see Fig: 1.5.3 and 1.5.4.

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we first need to identify typical columns in accordance with tributary areas and load. we will identify the columns according to the grid intersections. like the corner column at intersection of grid lines 'A' and '1' is A-1.

Column Type	Columns
A	A-1, A-6, F-1, F-6
B	A-2, A-5, F-2, F-5
C	A-3, A-4, F-3, F-4
D	B-1, E-1, B-6, E-6
E	B-2, B-5, E-2, E-5
F	B-3, B-4, E-3, E-4
G	C-2, C-5, D-2, D-5
H	C-3, C-4, D-3, D-4
I	C-1, C-6, D-1, D-6

let's start with column type 'A'

At the roof, the loads acting are:

Clear span of be RB-1 and RB-3 = $12' - 2(8'') = 10'\text{-}8''$

Clear span of each parapet = $10'\text{-}8''$

Clear span of slab S-1 = $12' - 2(4'') = 11'\text{-}4''$

<u>Dead Load</u>

From beams (RB-1 and RB-3) = $2 \times \frac{10'\text{-}8''}{2} \times \frac{8'' \times 16''}{144''} \times 150\ \text{pcf}$ — Beams are $8'' \times 16''$

$= 1{,}427$ lbs

From parapet $= 2 \times \frac{10'\text{-}8''}{2} \times 65\ \text{psf} \times 3'$ — Parapets are 3' high

$= 2{,}081$ lbs

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From slab $= \frac{11'-4''}{2} \times \frac{11'-8''}{2} \times 100\ psf = 3306$ lbs — from prob (a)

(Length of slab $= (12') - (4'') = 11'-8''$)

Column from top of parapet to roof top

$= \frac{16'' \times 16''}{144''} \times 3' \times 150\ psf = 800$ lbs

Total dead load $= 1,427 + 2,081 + 3,306 + 800 = 7,614$ lbs

Live load from slab $= \frac{11'-4''}{2} \times \frac{11'-8''}{2} \times 30\ psf = 992$ lbs

Total service load $= 7,614 + 992 = 8606$ lbs

Now there are eight typical slabs above second floor. Each floor is 10' high from top of the lower slab to top of the next upper slab.

The thickness of the slab is also 6" and the super imposed dead load is also 25 psf

Hence dead of slab is 100 psf as for the roof slab

The live load for the typical floor is 40 psf.

Dead load for each floor

From beams (B-1 & B-5) $= 2 \times \frac{10'-8''}{2} \times \frac{8'' \times 16''}{144} \times 150\ psf$

$= 1,427$ lbs

Masonry walls $= 2 \times \frac{10'-8''}{2} \times 65\ psf \times 8'-8''$ — Clear ht. of masonry $= 10' - 16'' = 8'-8''$

$= 6,013$ lbs.

From slab $= \frac{11'-4''}{2} \times \frac{11'-8''}{2} \times 100\ psf = 3306$ lbs.

Column $= \frac{16'' \times 16''}{144} \times 10' \times 150\ psf = 2667$ lbs

Total Dead Load $= 1,427 + 6,013 + 3,306 + 2,667 = 13,413$ lbs

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Live load $= \frac{11'-4''}{2} \times \frac{11'-8''}{2} \times 40\,psf = 1{,}322$ lbs

Adding loads,

	Dead	Live
Roof	7,614 lbs	992 lbs.
Typical floor	13,413 lbs ×8	1,322 lbs ×8
	107,304 lbs	10,576 lbs

Loads at base of column A-1 at second floor

Dead − 7,614 + 107,304 − 114,918

Live − 992 + 10,576 − 11,568 lbs.

Service − 114,918 + 11,568 − 126,486 lbs

factored − (1.2)(114,918) + (1.6)(11,568) − 156,410 lbs

This example was to illustrate the calculation of loads on slabs, beams and columns. This is a very tedious job. After understanding the concept, a student can use spread sheets to calculate the loads on the columns. For building (A), spreadsheets are provided The loads calculated on the columns shall be used in the design of columns and footings in subsequent chapters.

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COLUMN TYPE - A; A-1, A-6, F-1, F-6

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	10.67	1422	33.1	3310	800	10.67	2081	33.1	993
9th Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
8th Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
7th Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
6th Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
5th Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
4th Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
3rd Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324
2nd Floor	10.67	1422	33.1	3310	2667	10.67	6018	33.1	1324

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	7613	993	8606	10724	8606	10724
9th Floor	13417	1324	14741	18219	23347	28943
8th Floor	13417	1324	14741	18219	38088	47162
7th Floor	13417	1324	14741	18219	52830	65381
6th Floor	13417	1324	14741	18219	67571	83600
5th Floor	13417	1324	14741	18219	82312	101819
4th Floor	13417	1324	14741	18219	97053	120039
3rd Floor	13417	1324	14741	18219	111794	138258
2nd Floor	13417	1324	14741	18219	126535	156477

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - B; A-2, A-5, F-2, F-5

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	11.67	1556	72.67	7267	800	11.67	2276	72.67	2180
9th Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
8th Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
7th Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
6th Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
5th Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
4th Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
3rd Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830
2nd Floor	17.33	2310	70.75	7075	2667	11.67	6582	70.75	2830

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	11898	2180.1	14078	17766	14078	17766
9th Floor	18634	2830	21464	26889	35542	44655
8th Floor	18634	2830	21464	26889	57006	71544
7th Floor	18634	2830	21464	26889	78470	98432
6th Floor	18634	2830	21464	26889	99934	125321
5th Floor	18634	2830	21464	26889	121398	152210
4th Floor	18634	2830	21464	26889	142862	179099
3rd Floor	18634	2830	21464	26889	164326	205987
2nd Floor	18634	2830	21464	26889	185790	232876

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - C; A-3, A-4, F-3, F-4

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	12.67	1689	79.34	7934	800	12.67	2471	79.34	2380
9th Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
8th Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
7th Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
6th Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
5th Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
4th Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
3rd Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020
2nd Floor	18.33	2443	75.51	7551	2667	12.67	7146	75.51	3020

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	12894	2380	15274	19281	15274	19281
9th Floor	19807	3020	22828	28601	38101	47882
8th Floor	19807	3020	22828	28601	60929	76483
7th Floor	19807	3020	22828	28601	83757	105085
6th Floor	19807	3020	22828	28601	106584	133686
5th Floor	19807	3020	22828	28601	129412	162287
4th Floor	19807	3020	22828	28601	152240	190889
3rd Floor	19807	3020	22828	28601	175067	219490
2nd Floor	19807	3020	22828	28601	197895	248091

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - D; B-1, E-1; B-6; E-6

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	11.67	1556	71.89	7189	800	11.67	2276	71.89	2157
9th Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
8th Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
7th Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
6th Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
5th Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
4th Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
3rd Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4
2nd Floor	17	2266	70.76	7076	2667	11.67	6582	70.76	2830.4

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	11820	2157	13977	17635	13977	17635
9th Floor	18591	2830	21421	26838	35398	44473
8th Floor	18591	2830	21421	26838	56820	71311
7th Floor	18591	2830	21421	26838	78241	98148
6th Floor	18591	2830	21421	26838	99662	124986
5th Floor	18591	2830	21421	26838	121084	151824
4th Floor	18591	2830	21421	26838	142505	178662
3rd Floor	18591	2830	21421	26838	163927	205500
2nd Floor	18591	2830	21421	26838	185348	232338

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft

Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs

3' high parapets weigh (3' x 65 psf) = 195 lbs

Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs

Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - E; B-2, E-2; B-5; E-5

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	11.67	1556	160.56	16056	800	0	0	160.56	4817
9th Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
8th Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
7th Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
6th Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
5th Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
4th Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
3rd Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8
2nd Floor	23.34	3111	152.92	15292	2667	0	0	152.92	6116.8

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	18412	4817	23228	29801	23228	29801
9th Floor	21070	6117	27187	35071	50415	64872
8th Floor	21070	6117	27187	35071	77602	99943
7th Floor	21070	6117	27187	35071	104789	135014
6th Floor	21070	6117	27187	35071	131976	170085
5th Floor	21070	6117	27187	35071	159164	205157
4th Floor	21070	6117	27187	35071	186351	240228
3rd Floor	21070	6117	27187	35071	213538	275299
2nd Floor	21070	6117	27187	35071	240725	310370

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - F; B-3, E-3; B-4; E-4

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	12.67	1689	177.34	17734	800	0	0	177.34	5320
9th Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
8th Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
7th Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
6th Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
5th Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
4th Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
3rd Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744
2nd Floor	24.34	3245	168.6	16860	2667	0	0	168.6	6744

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	20223	5320	25543	32780	25543	32780
9th Floor	22772	6744	29516	38116	55059	70896
8th Floor	22772	6744	29516	38116	84574	109012
7th Floor	22772	6744	29516	38116	114090	147128
6th Floor	22772	6744	29516	38116	143605	185245
5th Floor	22772	6744	29516	38116	173121	223361
4th Floor	22772	6744	29516	38116	202636	261477
3rd Floor	22772	6744	29516	38116	232152	299593
2nd Floor	22772	6744	29516	38116	261667	337710

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - G; C-2, D-2, C-5, D-5

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	11.67	1556	175.78	17578	800	0	0	175.78	5273
9th Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
8th Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
7th Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
6th Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
5th Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
4th Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
3rd Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574
2nd Floor	23.34	3111	164.34	16434	2667	0	0	164.34	6574

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	19934	5273	25207	32358	25207	32358
9th Floor	22212	6574	28786	37172	53993	69530
8th Floor	22212	6574	28786	37172	82779	106703
7th Floor	22212	6574	28786	37172	111564	143875
6th Floor	22212	6574	28786	37172	140350	181047
5th Floor	22212	6574	28786	37172	169136	218220
4th Floor	22212	6574	28786	37172	197922	255392
3rd Floor	22212	6574	28786	37172	226708	292565
2nd Floor	22212	6574	28786	37172	255494	329737

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - H; C-3, D-3, C-4, D-4

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	11.67	1556	196	19600	800	0	0	196	5880
9th Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
8th Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
7th Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
6th Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
5th Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
4th Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
3rd Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107
2nd Floor	23.34	3111	177.68	17768	2667	0	0	177.68	7107

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	21956	5880	27836	35755	27836	35755
9th Floor	23546	7107	30653	39627	58489	75382
8th Floor	23546	7107	30653	39627	89142	115009
7th Floor	23546	7107	30653	39627	119796	154636
6th Floor	23546	7107	30653	39627	150449	194263
5th Floor	23546	7107	30653	39627	181103	233890
4th Floor	23546	7107	30653	39627	211756	273517
3rd Floor	23546	7107	30653	39627	242410	313144
2nd Floor	23546	7107	30653	39627	273063	352771

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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COLUMN TYPE - I; C-1, D-1, C-6, D-6

FLOOR	BEAM		SLAB DEAD		COLUMN	WALL		SLAB LIVE	
	SPAN	LOAD	AREA	LOAD	LOAD	LENGTH	LOAD	AREA	LOAD
Roof	11.67	1556	77.74	7774	800	11.67	2276	77.74	2332
9th Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
8th Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
7th Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
6th Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
5th Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
4th Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
3rd Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020
2nd Floor	18	2399	75.5	7550	2667	11.67	6582	75.5	3020

FLOOR	TOTAL DEAD	TOTAL LIVE	SERVICE LOAD	FACTORED LOAD	CUMMALATIVE LOAD	
					SERVICE	FACTORED
Roof	12405	2332	14737	18618	14737	18618
9th Floor	19198	3020	22218	27870	36956	46488
8th Floor	19198	3020	22218	27870	59174	74358
7th Floor	19198	3020	22218	27870	81392	102228
6th Floor	19198	3020	22218	27870	103611	130098
5th Floor	19198	3020	22218	27870	125829	157968
4th Floor	19198	3020	22218	27870	148047	185837
3rd Floor	19198	3020	22218	27870	170265	213707
2nd Floor	19198	3020	22218	27870	192484	241577

All beams are 8" x 16" weighing (8" x 16" x 150 pcf / 144 sq.in) = 133.3 lbs/ft
Column from roof till top of parapet weighs (16" x 16" x 3' x 150 pcf/ 144 sq.in) = 800 lbs
3' high parapets weigh (3' x 65 psf) = 195 lbs
Floor columns weigh (16' X 16" x 10' x 150 pcf/144 sq.in) = 2667 lbs
Exterior masonry walls weigh (8'-8" x 65 psf) = 564 lbs/ft

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GROUND FLOOR COLUMN LOAD CALCULATIONS

(GRID LINES 'A' AND 'F')

COLUMN	BEAM		DEAD LOAD SLAB		LIVE LOAD SLAB		COLUMN	DEAD LOAD	LIVE LOAD	TOTAL	TOTAL	TOTAL	FACTORED
	LENGTH	LOAD	AREA	LOAD	AREA	LOAD	LOAD	FROM ABOVE	FROM ABOVE	DEAD	LIVE	SERVICE	LOAD
A-1, F-1, A-6, F-6	10.33	1374	33	3300	33	1320	2667	114950	11585	122291	12905	135196	167397
A-2, F-2, A-5, F-5	11.67	1552	72.67	7267	72.67	2907	2667	160970	24820	172456	27727	200183	251310
A-3, F-3, A-4, F-4	12.67	1685	79.34	7934	79.34	3174	2667	171352	26543	183638	29717	213355	267912

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GROUND FLOOR COLUMN LOAD CALCULATIONS
(GRID LINES 'B' through 'E')

COLUMNS	SHALLOW BEAM		DEEP BEAM		DEAD FROM SLAB		COLUMN	LIVE FROM SLAB	
	LENGTH	LOAD	LENGTH	LOAD	AREA	LOAD	LOAD	AREA	LOAD
B-1, B-6, E-1, E-6	11.67	1552	5.34	2136	69.03	6903	2667	69.03	2071
B-3, B-4, E-3, E-4	0		12.67	5068	165.7	16570	2667	165.7	4971
C-1, C-6, D-1, D-6	12.67	1685	5.34	2136	73.98	7398	2667	73.98	2219
C-3, C-4, D-3, D-4	0		12.67	5068	177.4	17740	2667	177.4	5322

COLUMNS	REACTION FROM COLUMN ABOVE		DEAD LOAD FROM ABOVE	LIVE LOAD FROM ABOVE	TOTAL DEAD LOAD	TOTAL LIVE LOAD	SERVICE LOAD	FACTORED LOAD
	DEAD	LIVE						
B-1, B-6, E-1, E-6	93487	26875	160548	24800	264626	53746	318372	403545
B-3, B-4, E-3, E-4	93487	26875	202395	59272	317520	91118	408638	526813
C-1, C-6, D-1, D-6	98816	28931	165992	26492	276027	57642	333670	423460
C-3, C-4, D-3, D-4	98816	28931	210325	62738	331949	96991	428940	553524

Shallow beams are 8" x 16" weighing 133 lbs/ft
Deep beams are 16" x 24" weighing 400 lbs/ft

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SUMMARY OF COLUMN LOADS

COLUMN	DEAD	LIVE	SERVICE	FACTORED
A-1, A-6, F-1, F-6	122291	12905	135196	167397
A-2, A-5, F-2, F-5	172456	27727	200183	251310
A-3, A-4, F-3, F-4	183638	29717	213355	267913
B-1, B-6, E-1, E-6	264626	53746	318372	403545
B-3, B-4, E-3, E-4	317530	91118	408648	526825
C-1, C-6, D-1, D-6	276027	57642	333669	423460
C-3, C-4, D-3, D-4	331949	96991	428940	553524

C.2 PROBLEM C.2: COLUMN LOADS AND GRAVITY LOADS

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PROJECT: DESIGN LOAD				Design
SUBJECT: COLUMN LOADS			SHEET NO. 3/20	Inspection / Investigation
JOB NO: PROB: 3.2	DATE:	DESIGNED BY: MA	OF SHEETS	Reports

ACI Code

The partial layout of a 30-storied building with a flat plate system is provided. Calculate the gravity load acting on the footings of columns 'A' and 'B'. The ground floor slab is suspended on piles and is used for parking. Ignore lateral loads.

Roof live loads	– 30 psf
Roof super-imposed dead loads	– 15 psf
Floor live loads	– 40 psf
Floor finishes and partions	– 25 psf
Ground floor live loads	– 50 psf
Floor finishes for ground floor	– 0 psf
Typical clear floor height	– 9′
Thickness of roof slab	– 8″
Thickness of floor slab	– 9″
Thickness of ground slab	– 8″

Column dimensions

	A	B
Ground – 10^{th} floor	12″×48″	12″×60″
11^{th} – 20^{th} floor	12″×42″	12″×54″
20^{th} – 30^{th} floor	12″×36″	12″×48″

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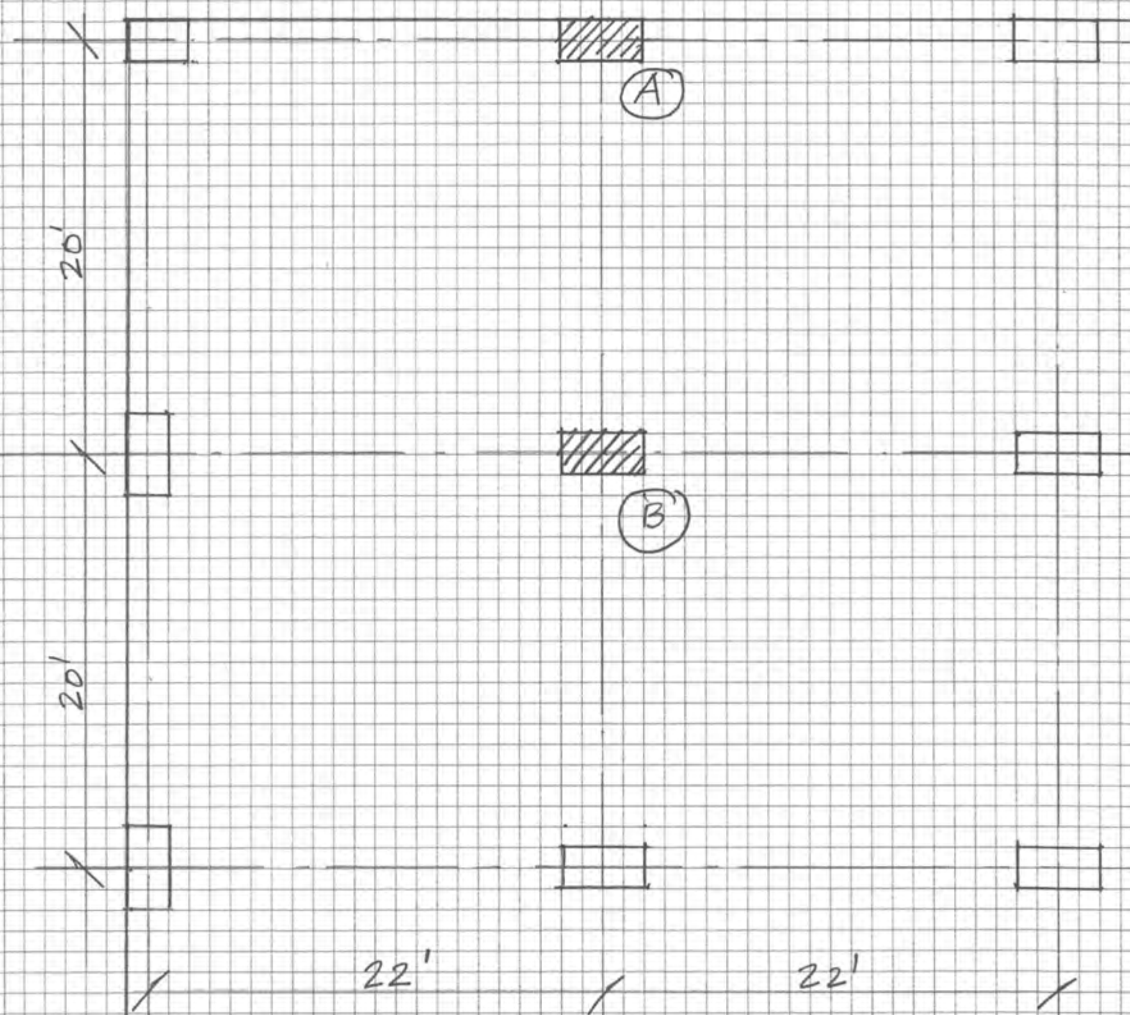
PROJECT: DESIGN LOADS
SUBJECT: GRAVITY LOADS
SHEET NO. 3/21 OF SHEETS
JOB NO: Prob: 3.2
DATE:
DESIGNED BY: MA

Design
Inspection
Investigation
Reports

ACI Code



Column A

Tributary area $= 22' \left(\frac{20'}{2} + 6''\right) = 231$ sft

Dead load

Roof self weight $= \frac{8''}{12''} \times 150\text{ psf} \times 231 = 23{,}100$ lbs

Roof super-imposed dead $= 15\text{ psf} \times 231 = 3{,}465$ lbs

29 typical floor self weight $= \frac{9''}{12''} \times 150\text{ psf} \times 231 \times 29$
$= 753{,}638$ lbs

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PROJECT:				Design
SUBJECT:			SHEET NO. 3/22	Inspection Investigation
JOB NO:	DATE:	DESIGNED BY:	OF SHEETS	Reports

ACI Code

29 typical floors finishes & partitions

$= 25\,psf \times 231 \times 29 = 167{,}475$ lbs

Column weight (Ground – 10th floor

$= \frac{12'' \times 48''}{144} \times 9' \times 150\,psf \times 10 = 54{,}000$ lbs

Column weight (11th – 20th floor)

$= \frac{12'' \times 42''}{144} \times 9' \times 150\,psf \times 10 = 47{,}250$ lbs

Column weight (21st – 30th floor)

$= \frac{12'' \times 36''}{144} \times 9' \times 150\,psf \times 10 = 40{,}500$ lbs

Ground floor self weight

$= \frac{8''}{12''} \times 150\,psf \times 231 = 23{,}100$ lbs

Total dead load $= \underline{1{,}112{,}518 \text{ lbs}}$

Live Load

Roof $= 30\,psf \times 231 = 6{,}930$ lbs

29 typical floors $= 29 \times 40\,psf \times 231 = 267{,}960$ lbs

Ground floor $= 50\,psf \times 231 = 11{,}550$ lbs

Total live load $= \underline{286{,}440 \text{ lbs}}$

Total service load $= 1{,}112{,}518 + 286{,}440$

$= \underline{1{,}398{,}958 \text{ lbs}}$

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ACI Code

Column 'B'

Tributary area = 22'×20' = 440 sft

Dead load

Roof slab weight = $\frac{8''}{12''}$ × 150 psf × 440 = 44,000 lbs

Roof super-imposed dead = 15 psf × 440 = 6,600 lbs

29 typical floor self weight = $\frac{9''}{12}$ × 150 psf × 440 × 29
= 1,435,500 lbs

29 typical floor finishes & partions = 25 psf × 440 × 29
= 319,000 lbs

Column weight (ground − 10th floor)
= $\frac{12'' \times 60''}{144}$ × 9' × 150 psf × 10 = 67,500 lbs

Column weight (11th − 20th floor)
= $\frac{12'' \times 54''}{144}$ × 9' × 150 psf × 10 = 60,750 lbs

Column weight (20th − 30th floor)
= $\frac{12'' \times 48''}{144}$ × 9' × 150 psf × 10 = 54,000 lbs

Ground floor self weight = $\frac{8''}{12''}$ × 150 psf × 440 = 44,000 lbs

Total dead load = 2,031,350 lbs

Live load

Roof = 30 psf × 440 = 13,200 lbs

29 typical floor = 29 × 40 psf × 440 = 510,400 lbs

Ground floor = 50 psf × 440 = 22,000 lbs

Total live load = 545,600 lbs

Total service load = 2,031,350 + 545,600
= 2,576,950 lbs

C.3 PROBLEM C.3: LATERAL SOIL PRESSURE

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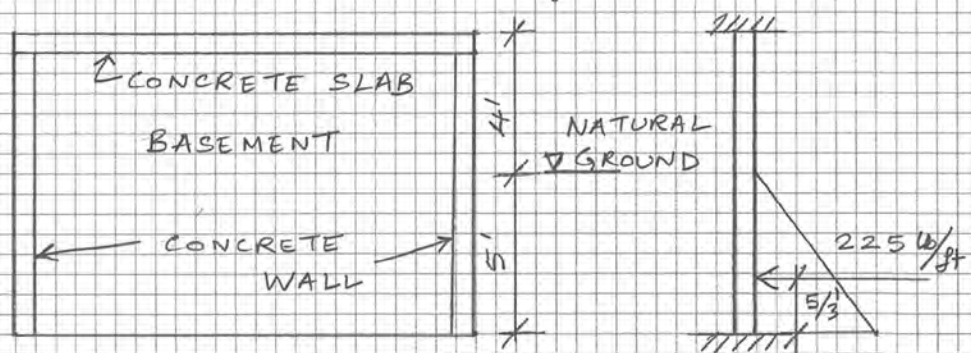
ACI Code

The roof of a 9' tall basement is 4' above the adjacent natural ground. The concrete walls of the basement are retaining 5' of silty sand. Draw a free body diagram indicating the lateral force due to the soil acting on the wall.



Free body diagram

Design lateral soil load of silty sand = 45 psf/ft — ASCE 7-10 Table 3.2-1

Height of soil being retained = 5'

Lateral pressure of soil on the wall = 45 psf × 5'
= 225 lb/ft
acting at 5/3' from base

C.4 PROBLEM C.4: HYDROSTATIC UPLIFT

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SUBJECT: HYDROSTATIC UPLIFT			SHEET NO. 3/25	Inspection
JOB NO: Prob: 3.4	DATE:	DESIGNED BY: MA	OF SHEETS	Investigation
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ACI Code

The finished floor elevation of an 8" thick slab of a utility room in a single family home is 7'. The base flood elevation of the property is 8'. Calculate the hydrostatic uplift on the slab. (All elevations are NGVD — National Geodetic Vertical Datum of 1929)

Finished floor elevation — 7'-0"

Thickness of slab — 0'-8"

Elevation at the underside of slab — 6'-4"

Hydrostatic uplift head = (Base flood elevation) − (Elevation at underside of slab)

= 8'-0" − 6'-4" = 1'-8" = 1.67'

Hydrostatic uplift = 1.67' × 62.4 psf = 104.2 psf

(Density of water is 62.4 psf)

Either the slab should be thick enough to resist this uplift force by virtue of its weight or the slab should be anchored to piles and designed as a suspended slab.

C.5 PROBLEM C.5: SNOW LOADS

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Calculate the snow load on a sheltered roof located in Anchorage, Alaska (Exposure B). The building is intentionally kept below freezing and categorized as type II

		ACI Code
		ASCE 7-10 References
Exposure factor (C_e)	— 1.2	Table 7-2
Thermal factor (C_t)	— 1.3	Table 7-3
Importance factor (I_s)	— 1.0	Table 1.5-2
Ground Snow Load (P_g)	— 50 psf	Table 7-1
Flat roof snow load (P_f) — $0.7 C_e C_t I_s P_g$		Equation 7.3-1
$= (0.7)(1.2)(1.3)(1.0)(50)$	— 54.6 psf	

Appendix D: ACI Strength Requirements

D.1 PROBLEMS D.1 AND D.2: BENDING STRENGTH—β_1 FACTOR AND MOMENT CAPACITY

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	SUBJECT: BENDING STRENGTH – β_1 FACTOR			SHEET NO. 4/1	Inspection / Investigation
	JOB NO: PROB: 4·1/2	DATE:	DESIGNED BY: MA	OF SHEETS	Reports

ACI Code

1) A rectangular beam (8″ × 24″) is reinforced with 3#5 bars at the bottom. The clear cover is 1.5″. $f'_c = 4{,}000$ psi; $f_y = 60{,}000$ psi. Using a rectangular stress block, calculate the ultimate capacity of the beam.

Effective depth $(d) = 24'' - 1.5'' - \frac{0.625''}{2} = 22.19''$

Area of steel $= 3 \times 0.31 = 0.93\ in^2$

Ultimate Moment capacity $(M_n) = A_s f_y \left(d - \frac{0.59 A_s f_y}{f'_c b}\right)$

$= (0.93)(60{,}000)\left(22.19 - \frac{(0.59)(0.93)(60{,}000)}{(4000)\,8}\right)$

$= 1180794.2$ lb-in $= \underline{98.4\ \text{K-ft}}$

2) Prepare a table showing β_1 values. β_1 is the factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis

$\beta_1 = 0.85$ for $2500\,psi \le f'_c \le 4000\,psi$ — Equation 22.2.2.4.1

$= 0.85 - \left[0.05 - \left(\frac{f'_c - 4000}{1000}\right)\right]$ for $4000\,psi \le f'_c \le 8000\,psi$

$= 0.65$ for $f'_c \geq 8000$ psi

f'_c	β_1
≤ 4000 psi	0.85
5000 psi	0.80
6000 psi	0.75
7000 psi	0.70
8000 psi	0.65
> 8000 psi	0.65

D.2 PROBLEM D.3: COMPRESSIVE STRENGTH

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ACI Code

Calculate the maximum axial compressive capacity of a 12″ × 48″ rectangular column, reinforced with 8#8 bars. $f'_c = 8$ Ksi, $f_y = 60$ Ksi

Gross area of column (A_g) $= 12 \times 48 = 576$ in^2

Total area of steel (A_{st}) $= 8 \times 0.786 = 6.288$ in^2

Nominal capacity of column (P_o) — Equation 22.4.2.2

$= 0.85 f'_c (A_g - A_{st}) + f_y A_{st}$

$= 0.85 (8000)(576 - 6.288) + (60{,}000)(6.288)$

$= 4244522$ lbs $= 4244.5$ Kips

$P_{n,max} = 0.80 P_o$ for ties — Table 22.4.2.1

$= (0.8)(4244.5)$

$= 3395.6$ Kips

$P_{n,max} = 0.85 P_o$ for spirals

$= (0.85)(4244.5)$

$= 3607.8$ Kips

D.3 PROBLEM D.4: SHEAR STRENGTH

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ACI Code

A rectangular beam (8"x16") is reinforced with 3#5 at the bottom.

Factored Moment (M_u) = 44,640 lb-ft

Factored Shear (V_u) = 14,880 lb-ft

$f'_c = 5{,}000$ psi , $f_y = 60{,}000$ psi

Design the shear stirrups of the beam.

Assuming a clear cover of 1.5",

Effective depth $(d) = 16'' - 1.5'' - (0.625''/2) = 14.2''$

Shear capacity of concrete $(V_c) = 2\sqrt{f'_c}\, b_w d$ — Equation 22.5.5.1

$= 2\sqrt{5000}\,(8)(14.2) = 16065$ lbs

Steel ratio $(\rho_w) = \dfrac{(3)(0.31)}{(8)(14.2)} = 0.00819$

Also, V_c is minimum of: — Table 22.5.5.1

(1) $\left[1.9\sqrt{f'_c} + 2500\,\dfrac{\rho_w V_u d}{M_u}\right] b_w d$

$= \left[1.9\sqrt{5000} + \dfrac{2500 \times 0.0819 \times 14{,}880 \times 14.2}{44{,}640 \times 12}\right](8)(14.2)$

$= 24{,}437$ lbs

(2) $\left[1.9\sqrt{f'_c} + 2500\,\rho_w\right] b_w d$

$= \left[1.9\sqrt{5000} + 2500(0.00819)\right](8)(14.2) = 17{,}588$ lbs

(3) $3.5\sqrt{f'_c}\, b_w d = 3.5\sqrt{5000}\,(8)(14.2) = 28{,}115$ lbs

Hence, the least value of the shear capacity of concrete $(V_c) = 16065$ lbs

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	ACI Code
Shear to be resisted by stirrups	
$V_s \geq \dfrac{V_u}{\phi} - V_c$	Equation 22.5.10.1
Shear reduction factor $(\phi) = 0.75$	Table 21.2.1
$V_s \geq \dfrac{14{,}880}{0.75} - 16{,}065$	
≥ 3775 lbs	
Using #3 double legged stirrups,	Equation 22.5.10.5.3
$s = \dfrac{A_v f_y d}{V_s}$	
$= \dfrac{(2)(0.11)(60{,}000)(14.2)}{3775} = 49''$	
$V_s = 3775$ lbs $\leq 4\sqrt{f'_c}\, b_w d$, stirrups to be at a $d/2$ spacing	Section 9.7.6.2.2
$d = 14.2/2 = 7.1''$	
Provide #3 stirrups @ 7'' o.c.	

Appendix E: Slabs

E.1 PROBLEM E.1: MOMENTS—CONTINUOUS SLAB

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SUBJECT: MOMENTS – CONT: SLAB			SHEET NO. 6/1	Inspection
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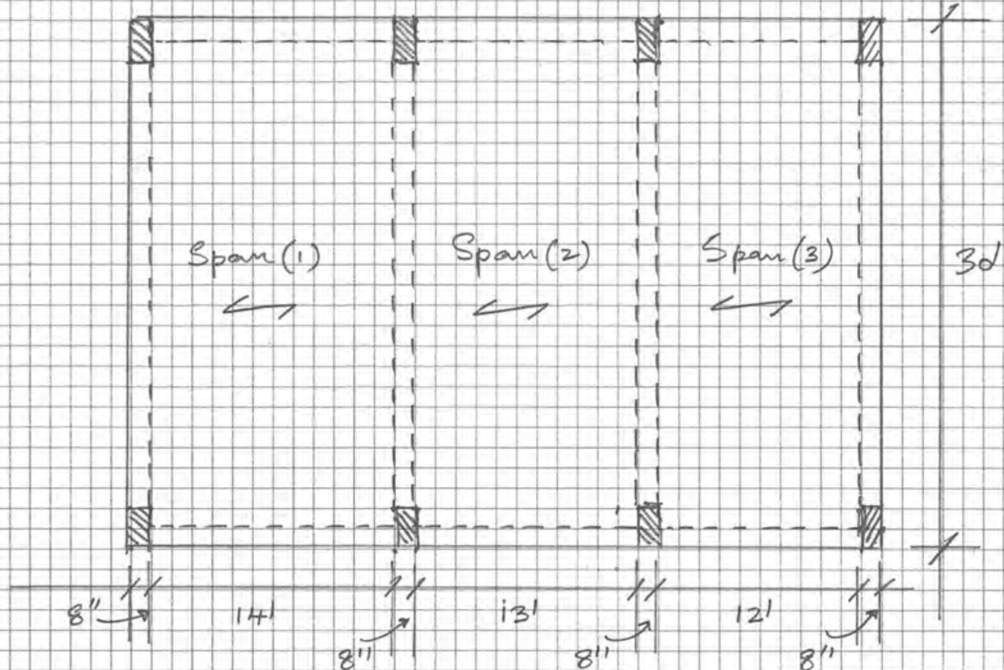
ACI Code

The live load for an office building slab is 50 psf. The super-imposed dead load is 25 psf. The slabs are 6" thick. Using the ACI co-efficients, calculate moments at different locations.



$\frac{30'}{14'} > 2$ $\frac{30'}{13'} > 2$ $\frac{30'}{12'} > 2$

Hence all slab panels are one-way

Slab weight – $\frac{6''}{12''} \times 150$ – 75 psf

Super-imposed dead load – 25 psf

Total dead load – 100 psf

Live load – 50 psf

Factored load (w_u) – $(1.2 \times 100) + (1.6 \times 50)$ – 200 psf

ACI Code: Sec: 8.3.3

(a) There are 3 spans.

(b) 14' compared to 13' is less than 20%

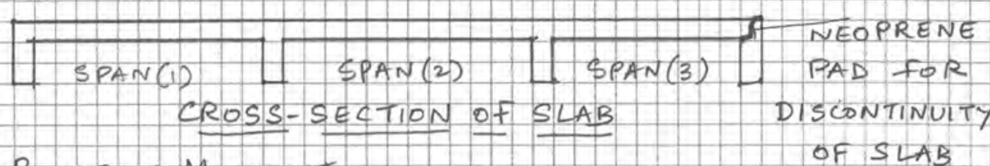
13' compared to 12' is less than 20%

(c) Load is uniform 200 psf

(d) Unfactored live load (50 psf) < dead load (120 psf)

(e) All elements are rectangular in cross-section and hence, prismatic.

Moment co-efficient of Table 6.5.2 can be used.



CROSS-SECTION OF SLAB

POSITIVE MOMENT

Span (1) – End span – discontinuous end integral with support $(M_U) = \frac{w_u l_n^2}{14} = \frac{200 \times 14^2}{14}$

$= 2800$ lb-ft

Span (2) – Interior span $(M_U) = \frac{w_u l_n^2}{16} = \frac{200 \times 13^2}{16}$

$= 2112.5$ lb-ft

Span (3) – End span – discontinuous end unrestrained $(M_U) = \frac{w_u l_n^2}{11} = \frac{200 \times 12^2}{11} = 2618.2$ lb-ft

NEGATIVE MOMENT

Span (1) – at interior face of external support where support is a spandrel beam $(M_U) = \frac{w_u l_n^2}{24}$

$= \frac{200 \times 14^2}{24} = 1633.3$ lb-ft

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ACI Code

Span (1) – at exterior face of first interior support for more than 2 spans $(M_U) = \frac{w_u l_n^2}{10} = \frac{200 \times 13.5^2}{10}$

(l_n – average of adjacent spans) $= 3645$ lb-ft

$\frac{13' + 14'}{2} = 13.5'$

Span (2) – at other face of interior supports (left)

$M_U = \frac{w_u l_n^2}{11} = \frac{200 \times 13.5^2}{11} = 3313.6$ lb-ft

(l_n – average of adjacent spans)

$\frac{13' + 14'}{2} = 13.5'$

Span (2) – at other face of interior support (right)

$M_U = \frac{w_u l_n^2}{11} = \frac{200 \times 12.5^2}{11} = 2840.9$ lb-ft

(l_n – average of adjacent spans)

$\frac{13' + 12'}{2} = 12.5'$

Span (3) – at exterior face of first interior support for more than two spans $(M_U) = \frac{w_u l_n^2}{10}$

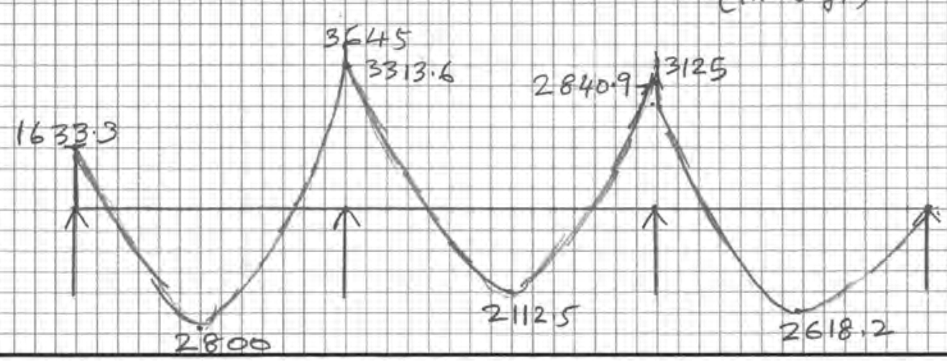
(l_n – average of adjacent spans) $= \frac{200 \times 12.5^2}{10} = 3125$ lb-ft

$\frac{13' + 12'}{2} = 12.5'$

Span (3) – at interior face of exterior support

$M_U = \frac{w_u l_n^2}{24} = 0$ [because, as shown in the section, span is not built integrally with the beam]

BENDING MOMENT DIAGRAM
(in lb-ft)



E.2 PROBLEM E.2: ONE-WAY SLAB

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ACI Code

Design the one-way slab of Fig. 1.5.1

$f'_c = 4{,}000$ psi ; $f_y = 60{,}000$ psi, slab thickness – 6"

Super-imposed loads – dead – 25 psf ; live – 30 psf.

Self wt. of slab – $\frac{6''}{12''} \times 150$ pcf – 75 psf

Super-imposed dead load – 25 psf

Total dead load – 100 psf

Live load – 30 psf

Factored load (w_u) – $1.2 \times 100 + 1.6 \times 30$ – 168 psf

The five adjacent spans are 12', 14', 14', 14' & 12'.

Differences in the span is not greater than 20%

Uniform load is 168 psf

Unfactored live load (30 psf) < Unfactored dead L

All elements have a rectangular cross section

Hence moment co-efficients of table 6.5.2 apply.

Averages of adjacent spans for (–ve) moments

Span 1–2 $= \frac{12' + 14'}{2} = 13'$

Span 2–3 $= \frac{14' + 14'}{2} = 14'$

Span 3–4 $= \frac{14' + 14'}{2} = 14'$

Span 4–5 $= \frac{14' + 12'}{2} = 13'$

POSITIVE MOMENTS

Span (1) & (5) – discontinuous ends integrals with supports – $\frac{w_u l_n^2}{14} = \frac{(168)(12)^2}{14} = 1{,}728$ lb-ft

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ACI Code

Spans (2), (3) & (4) — $\frac{w_u l_n^2}{16} = \frac{(168)(14)^2}{16} = 2{,}058$ lb-ft

NEGATIVE MOMENTS

Spans (1) & (5) — interior face of exterior support

$M_u = \frac{w_u l_n^2}{24} = \frac{(168)(12)^2}{24} = 1{,}008$ lb-ft

Spans (1) & (5) — exterior face of first interior support (avg. l_n taken)

$M_u = \frac{w_u l_n^2}{10} = \frac{(168)(13)^2}{10} = 2{,}839$ lb-ft

Spans (2) & (4) — faces adjacent to 12' span

$M_u = \frac{w_u l_n^2}{11} = \frac{(168)(13)^2}{11} = 2{,}581$ lb-ft

Spans (2), (3) & (4) — All adjacent span of 14'

$M_u = \frac{w_u l_n^2}{11} = \frac{(168)(14)^2}{11} = 2{,}994$ lb-ft

SHEAR

All shear forces are calculated at face of support — Sect. 7.4.2.1

Spans (1) & (2), $V_u = \frac{(168)(11.33)}{2} = 952$ lbs

Spans (3), (4) & (5), $V_u = \frac{(168)(13.33)}{2} = 1120$ lbs

CHECK FOR THICKNESS OF SLAB — Table 7.3.1.1

Spans (1) & (5), $h = l/24 = 12/24 = 0.5'$

Spans (2), (3) & (4), $h = l/28 = 14/28 = 0.5'$

Hence thickness of 6" provided is OK!

DESIGN OF REINFORCEMENT

The span and support design moments are exhibited on the figure on the next page

PROJECT:

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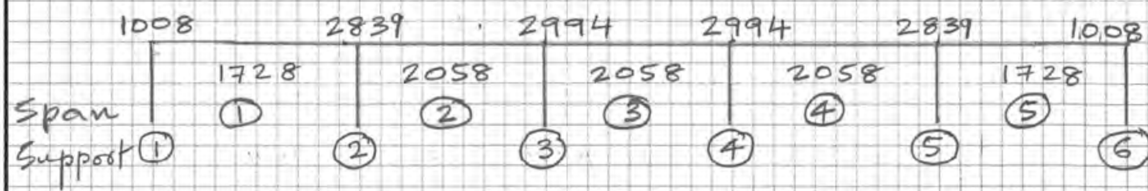

All moments shown in lb-ft units

For exhibiting the reinforcement design in this example, only the maximum (+ve) & (−ve) moments are used.

Maximum (+ve) moment — 2,058 lb-ft (span)

Maximum (−ve) moment — 2,994 lb-ft (support)

Assume 3/4" clear cover to reinforcement (not exposed to weather) — Table 20.6.1.3.3

Assume #4 bars

Effective depth $(d) = 6'' - 0.75'' - 0.5/2 = 5''$

β_1 (for $f'_c = 4{,}000$ psi) $= 0.85$ — Equation 22.2.2.4.1

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{\rho d f_y}{0.85 f'_c} \quad (\text{Since } \rho = A_s/bd)$$

$$\text{Tension } (T) = A_s f_y = \rho b d f_y$$

$$M_n = T\left(d - \frac{a}{2}\right) = \rho b d f_y \left[d - \frac{0.5 \rho d}{0.85}\frac{f_y}{f'_c}\right]$$

$$= \rho b d^2 f_y \left[1 - \frac{0.5 \rho f_y}{0.85 f'_c}\right]$$

$$\text{Hence, } \frac{M_n}{b d^2 f'_c} = \frac{\rho f_y}{f'_c}\left[1 - \frac{0.5 \rho f_y}{0.85 f'_c}\right]$$

$$\text{Let } \omega = \frac{\rho f_y}{f'_c} \quad \text{and} \quad M_n = \frac{M_u}{\phi}$$

$$\text{Hence, } \frac{M_u}{\phi b d^2 f'_c} = \omega(1 - 0.59\omega)$$

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ACI Code

$$\frac{M_u}{\phi b d^2 f'_c} = \frac{2058 \times 12}{(0.9)(12)(5)^2(4000)} = 0.0229 \quad \left[\text{for +ve moment}\right]$$

$(\phi = 0.9$ for bending$)$ — Table 21.2.1

Hence, $\omega(1-0.59\omega) = 0.0229$

$-0.59\omega^2 + \omega - 0.0229 = 0$

$0.59\omega^2 - \omega + 0.0229 = 0$

Solving the quadratic equation

$$\omega = \frac{1 \pm \sqrt{1-(4)(0.59)(0.0229)}}{(2)(0.59)} = \frac{1.973 \text{ (or) } 0.0274}{1.18}$$

$= 1.672$ (or) 0.0232

$\omega = 1.672$ is very high – ignore

$$\rho \frac{f_y}{f'_c} = \rho \frac{60{,}000}{4{,}000} = 15\rho = 0.0232$$

Hence $\rho = 0.0232/15 = 0.00155$

$A_{st} = 0.00155 \times 12 \times 5 = 0.0928$ in^2/ft

$$\frac{M_u}{\phi b d^2 f'_c} = \frac{2994 \times 12}{(0.9)(12)(5)^2(4000)} = 0.0333 \quad \left[\text{for -ve moment}\right]$$

Hence, $\omega(1-0.59\omega) = 0.0333$

Solving the quadratic equation, as explained above, $\omega = 0.0339$

$$\rho \frac{f_y}{f'_c} = \rho \frac{60{,}000}{4{,}000} = 15\rho = 0.0333$$

Hence, $\rho = 0.0333/15 = 0.00225$

$A_{st} = 0.00225 \times 12 \times 5 = 0.136$ in^2/ft

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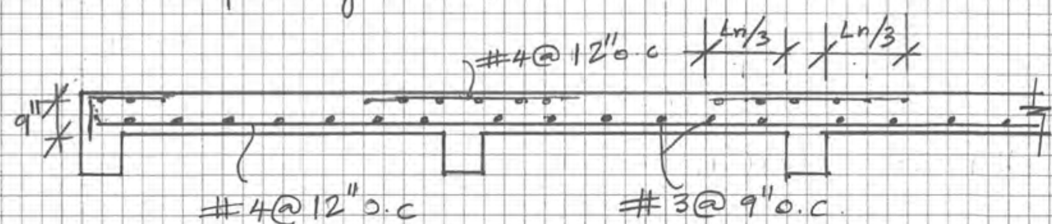
	ACI Code
Min. reinforcement = 0.0018 Ag	Table
= 0.0018 × 12 × 6 = 0.13 in²/ft	7.6.1.1
Provide #4 @ 12" o.c , As = 0.20 in²/ft at top and bottom	
Max. spacing = 2h = 2 × 6 = 12" < 3h	Section 7.7.2.3
Distribution steel (As) = 0.0018 Ag = 0.0018 × 12 × 6	Table
= 0.13 in²/ft	24.4.3.2
Provide #3 @ 9" o.c.	
Max. spacing = 18" > 9"	Section 24.4.3.3



E.3 PROBLEM E.3: FLAT PLATES

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Fig: 1.5.6 shows the typical floor plan of building 'B'. The slab is 7" thick flat plate supported on columns and walls. $f'_c = 4$ ksi; $f_y = 60$ ksi

Super-imposed loads — dead – 25 psf; Live – 50 psf

Self weight of slab – $\frac{7''}{12''} \times 150$ pcf = 88 psf

Super-imposed dead load = 25 psf

Total dead load = 113 psf

Live load = 50 psf

Factored load = (1.2)(113) + (1.6)(50) = 215 psf

Minimum thickness (h) = $l_n/30$ = $17 \times 12/30$ = 6.8" < 7.0" — Table 8.3.1.1

<u>Conditions for Direct Design Method</u> — Section 8.10.2

(a) There are 5 continuous equal span

(b) Span ratio = 1.0

(c) No column offsets.

(d) Not design for lateral loads (Assumption)

(e) Unfactored live load (50 psf) < dead load (113 psf)

For this example, assume there are no walls in fig: 1.5.6, instead there are 16"×16" columns at grid intersections, B-2, B-3, B-5, C-5, D-2, E-2, E-4 and E-5.

l_n – clear span = (17') – (16") = 15.67'

l_2 – Span perpendicular to l_n, c/c = 17'

Total factored static moment in each direction

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(215)(17)(15.67)^2}{8 \times 1000} = 112.2 \text{ K-ft}$$

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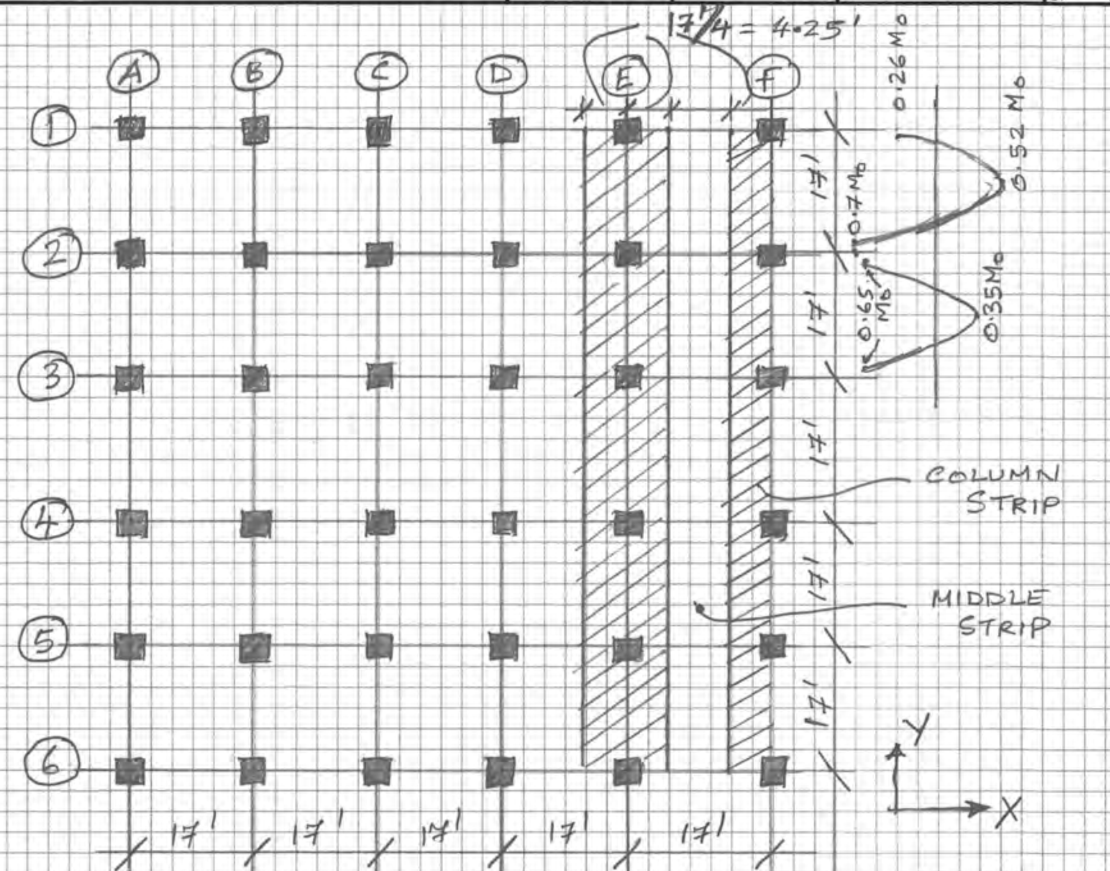
Section 8.4.1.5



Section 8.10.4

The moments acting along the Y-axis are shown. Corresponding column and middle strips along the X-axis are shown. Critical section for the negative moment is at the face of the column.

Tables

DISTRIBUTION OF MOMENTS

Exterior (−ve) moment — column strip (100%) = $0.26 M_o$ 8.10.5.2
(first column) $= 0.26 \times 112.2 = 29.2$ K-ft

Exterior (−ve) moment — column strip (100%) = $0.70 M_o$ 8.10.5.2
(first interior column) $= 0.70 \times 112.2 = 78.5$ K-ft

Interior (−ve) moment — column strip (75%) = $0.49 M_o$ 8.10.5.1
$= 0.49 \times 112.2 = 55.0$ K-ft

$(0.75 \times 0.65 = 0.49)$

Section 8.10.4.1

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Exterior (+ve) moment – column strip (60%) – $0.31 M_o$	8.10.5.5
$= 0.31 \times 112.2 = 34.8$ K-ft	
Exterior (+ve) moment – middle strip (40%) – $0.21 M_o$	8.10.5.5
$= 0.21 \times 112.2 = 23.6$ K-ft	
Interior (+ve) moment – column strip (60%) – $0.21 M_o$	8.10.5.5
$= 0.21 \times 112.2 = 23.6$ K-ft	
Interior (+ve) moment – middle strip (40%) – $0.14 M_o$	8.10.5.5
$= 0.14 \times 112.2 = 15.7$ K-ft	
Since the width used in the calculation of q_u was taken as 17', these calculations apply for grid lines 'B', 'C', 'D' & 'E'. For grid lines 'A' and 'F', width of (17'/2) shall be used	
DESIGN OF REINFORCEMENT	
EXTERIOR COLUMN STRIP (M_u) = 29.2 K-ft	
Width of column strip (b) = 17/2 = 8.5'	
Using clear cover of 3/4" and #4 bars,	
$d_{eff} = 7'' - 0.75'' - 0.5'' = 5.75''$	
$\frac{M_u}{\phi b d^2} = \frac{29.2 \times 12{,}000}{0.9 \times 8.5 \times 12 \times 5.75^2} = 115 \text{ psi} = A$	
$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2A}{0.85 f'_c}}\right] = \frac{0.85 \times 4000}{60{,}000}\left[1 - \sqrt{1 - \frac{(2)(115)}{(0.85)(4000)}}\right]$	
$= 0.0017$	
Provide $\rho_{(min.)} = 0.0018$	Table 7.6.1.1
$A_{st} = 0.0018 \times 12 \times 5.75 = 0.124 \text{ in}^2/\text{ft}$	
Max. spacing $= 2h = 2 \times 7 = 14'' < 3h$	Section 7.7.2.3
Provide #4 @ 14" o.c.	

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The reinforcement calculations for all locations of the design strip are tabulated below

	LOCATION	TYPE OF MOMENT	M_u (K-ft)	STRIP WIDTH (inch)	$\frac{M_u}{\phi bd^2}$ (psi)	ρ	A_s in²/ft	REBAR DESIGN
*	COLS 1,6 C.S.	−ve	29.2	102	115	0.0007 (0.0018)	0.124	#4 @ 12"
	COLS 2,5 C.S	−ve	78.5	102	310	0.0054	0.373	#4 @ 6"
	COL: 3,4 C.S	−ve	55.0	102	217	0.0037	0.255	#4 @ 9"
	SPANS 1,5 C.S	+ve	34.8	102	138	0.0023	0.159	#4 @ 12"
*	SPANS 2,3,4 C.S	+ve	23.6	102	93	0.0015 (0.0018)	0.124	#4 @ 12"
*	SPANS 1,5 M.S	+ve	23.6	102	93	0.0015 (0.0018)	0.124	#4 @ 12"
*	SPANS 2,3,4 M.S	+ve	15.7	102	62	0.0010 (0.0018)	0.104	#4 @ 12"

C.S. – COLUMN STRIP; M.S – MIDDLE STRIP

* Minimum steel $\rho = 0.0018$ is used — Table 7.6.1.1

CHECK MOMENT TRANSFER TO COLUMN

To illustrate this concept, the column at grid E-1 is selected. Column at grid E-6 is identical

Column strip moment (M_u) = 29.2 K-ft

Fraction of slab moment resisted by column

$$\gamma_f = \frac{1}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}}$$ Equation 8.4.2.3.2

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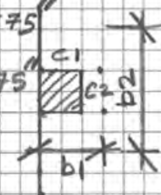
$b_1 = c_1 + d/2 = 16'' + \frac{5.75}{2} = 18.875''$

$b_2 = c_2 + d = 16'' + 5.75 = 21.75''$



Hence, $\gamma_f = \dfrac{1}{1 + (2/3)\sqrt{18.875/21.75}} = 0.62$

$\gamma_f M_u = (0.62)(29.2) = 18.1$ K-ft

Based upon the reinforcement calculations on the previous page, this is a smaller moment and would require minimum steel.

$\rho = 0.0018$

Effective slab width $= c_1 + 3h = 16 + 3(7) = 37''$ — Section 8.4.2.3.3

$A_s = 0.0018\,(5.75)(37) = 0.383''$

Provide 2#4 extra at column strip.

To demonstrate the moment transfer at an interior column, columns at grids E-2 & E-5 are selected as the moments at that location is higher (78.5 K-ft) as compared to columns at grids E-3 and E-4 (55.0 K-ft).

$b_1 = b_2 = c + d = 16'' + 5.75'' = 21.75''$

$\gamma_f = \dfrac{1}{1 + 2/3\sqrt{21.75/21.75}} = 0.6$

$\gamma_f M_u = (0.6)(78.5) = 47.1$ K-ft

$\dfrac{M_u}{\phi b d^2} = \dfrac{47.1 \times 12000}{(0.9)(37)(5.75)^2} = 513 = A$

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$$\rho = \frac{0.85f'_c}{f_y}\left[1-\sqrt{1-\frac{2A}{0.85f'_c}}\right]$$

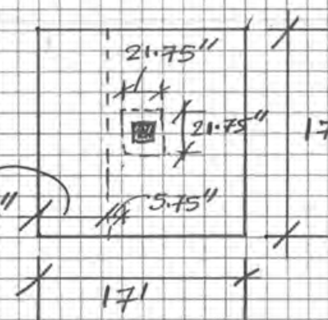
$$= \frac{0.85\times4{,}000}{60{,}000}\left[1-\sqrt{1-\frac{(2)\quad(513)}{(0.85)(4{,}000)}}\right] = 0.0093$$

$A_{st} = 0.0093(37)(5.75) = 1.98\ in^2$

Provide 7#5 extra @ 6" o.c.

For the middle strip, top reinforcement add minimum steel, #4@12" o.c.

WIDE BEAM SHEAR



$V_u = 0.215\times\frac{85.4}{12} = 1.53\ K/ft$ — Equation 22.5.5.1

$V_c = 2\sqrt{f'_c}\,b_w d$

$= \frac{2\sqrt{4000}\,(12)(5.75)}{1000}$

$= 8.72\ K/ft$ — Table 21.2.1

$\phi V_c = (0.75)(8.72) = 6.55\ K/ft > 1.53\ K/ft$

TWO-WAY SHEAR — Table 22.6.5.2

$V_u = 0.215\left[17^2-\left(\frac{21.75}{12}\right)^2\right] = 61.4\ K$

$V_c = 4\sqrt{f'_c}\,b_o d = \frac{4\sqrt{4000}\,(4\times21.75)(5.75)}{1000}$

$= 126.6\ K$

$\phi V_c = 0.75\times126.6 = 94.1\ K > 61.4\ K$

COMBINED SHEAR STRESS

To illustrate this concept, the columns at grid E-1 and E-6 are chosen.

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According to commentary R8.4.4.2.2, 40% of the moment is M_{sc} acting at the centroid of the critical section.

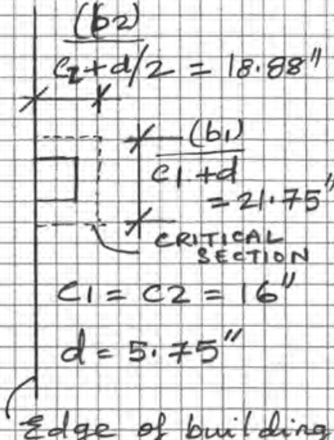

Figure R8.4.4.2.3

$M_{sc} = 0.4\,M_o = 0.4(112.2)$

$= 44.88$ K-ft

$\gamma_v = 1 - \gamma_f = 1 - 0.62 = 0.38$ — Equation 8.4.4.2.2

Area of concrete at critical section

$= (2 \times 18.8 + 21.75)\,5.75 = 341.3 \text{ in}^2$

$v_u = v_{ug} + \dfrac{\gamma_v M_{sc}\, c}{J}\;;\quad \dfrac{J}{c} = \dfrac{2b_1^2 d(b_1 + 2b_2) + d^3(2b_1 + b_2)}{6b_1}$ — Commentary R8.4.4.2.3

$$\frac{J}{c} = \frac{2(18.88)^2(5.75)[18.88 + (2 \times 21.75)] + 5.75^3[(2 \times 18.88) + 21.75]}{6(18.88)}$$

$= 2357 \text{ in}^3$

Shear for exterior column

$= 0.215\left[(17')\left(\frac{17'}{2} + \frac{8''}{12}\right) - \left(\frac{18.88 * 21.75}{144}\right)\right] = 32.9\text{k}$

$A = (2 \times 18.88 + 21.75)(5.75) = 342.2 \text{ in}^2$

$v_u = \dfrac{32900}{342.2} + \dfrac{(0.38)(44.88)(12{,}000)}{2357} = 182.9 \text{ psi}$

$v_n = 4\sqrt{f'_c} = 4\sqrt{4000} = 253 \text{ psi}$

$\phi v_n = 0.75(253) = 190 \text{ psi} > 182.9 \text{ psi}$

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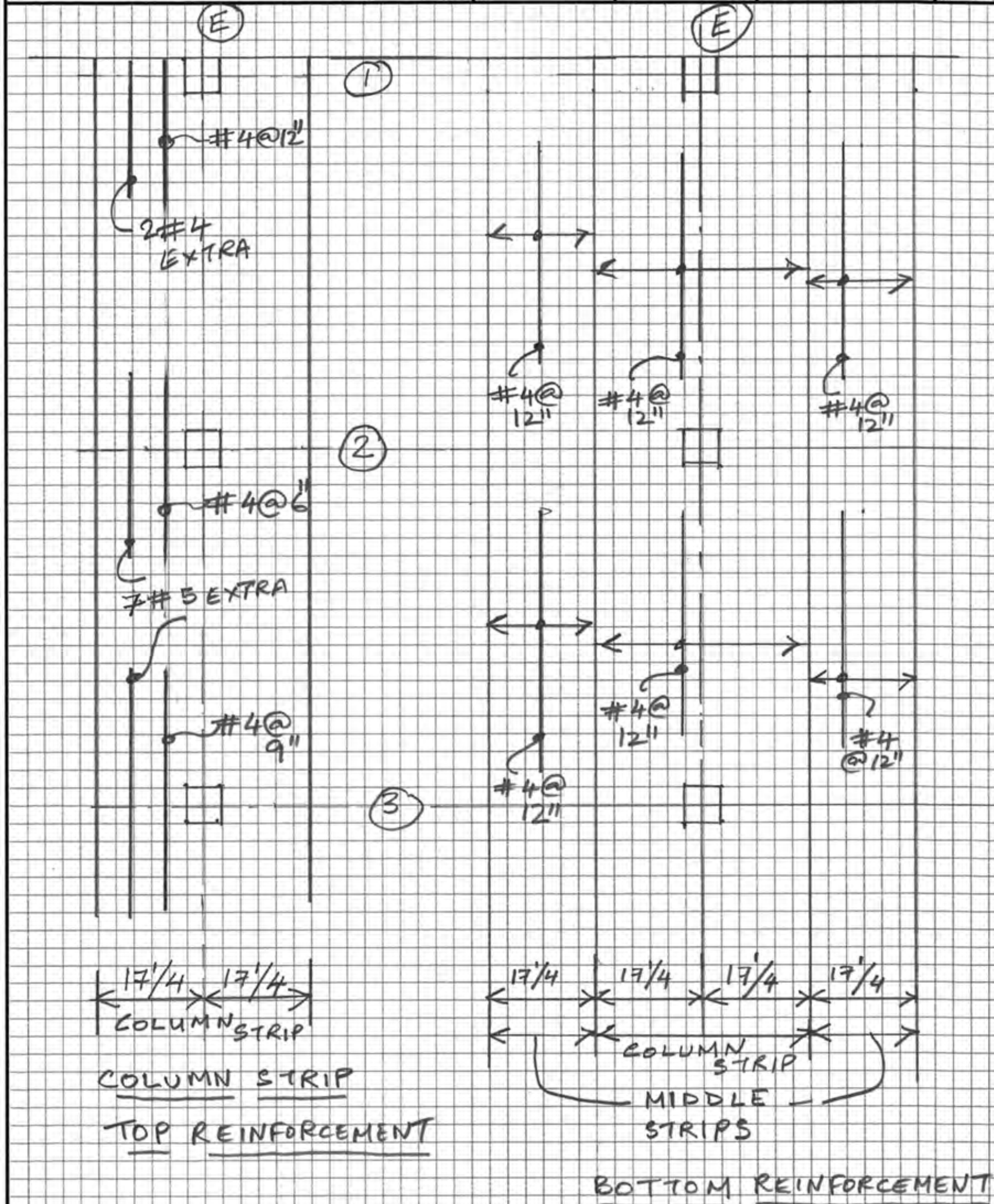

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In this example all spans in both directions are equal (17'). If spans in two directions are different (say l_1 and l_2), then the width of the column strip is smaller span/4.	
In the above example, the thickness selected is 7" in accordance with Table 8.3.1.1. Hence, deflection calculations are not required. But, in order to demonstrate procedure, calculation for deflections are performed.	
MODULUS OF RUPTURE	
$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4000} = 474$ psi	Equation 19.2.3.1
$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4{,}000} = 3.6\times10^6$ psi	Equation 19.2.2.1b
Modular ratio $(n) = E_s/E_c = \dfrac{29\times10^6}{3.6\times10^6} = 8.1$	
SERVICE LOAD MOMENTS	
Dead load (d) = 113 psf	
Live load (ℓ) = 50 psf	
Sustained load (s) = 113 + 0.4(50) = 133 psf	
$M_o(d) = 113\times17\times15.67^2/(8\times1000) = 59$ K-ft	
$M_o(d+L) = 163\times17\times15.67^2/(8\times1000) = 85$ K-ft	
$M_o(s) = 133\times17\times15.67^2/(8\times1000) = 69$ K-ft	
DISTRIBUTION OF MOMENT	
Unlike for the design of flexural moment, only the moments at grids '1' and '2' are considered.	

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For Dead Load ($M_o(d) = 59$ K-ft

Exterior (−ve) moment − $0.26\ M_o(d)$ − 15.34 K-ft
(first column)

Exterior (−ve) moment − $0.70\ M_o(d)$ − 41.30 K-ft
(first interior column)

Exterior (+ve) moment − $0.31\ M_o(d)$ − 18.29 K-ft
(column strip)

Exterior (+ve) moment − $0.21\ M_o(d)$ − 12.39 K-ft
(middle strip)

Interior (+ve) moment − $0.21\ M_o(d)$ − 12.39 K-ft
(column strip)

Interior (+ve) moment − $0.14\ M_o(d)$ − 8.26 K-ft
(middle strip)

For Dead + Live Load $M_o(d+l) = 85$ K-ft

Exterior (−ve) moment − $0.26\ M_o(d+l)$ − 22.10 K-ft
(first column)

Exterior (−ve) moment − $0.70\ M_o(d+l)$ − 59.50 K-ft
(first interior column)

Exterior (+ve) moment − $0.31\ M_o(d+l)$ − 26.35 K-ft
(column strip)

Exterior (+ve) moment − $0.21\ M_o(d+l)$ − 17.85 K-ft
(middle strip)

Interior (+ve) moment − $0.21\ M_o(d+l)$ − 17.85 K-ft
(column strip)

Interior (+ve) moment − $0.14\ M_o(d+l)$ − 11.90 K-ft
(middle strip)

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For sustained load $M_o(s) = 69$ K-ft

Exterior (-ve) moment – 0.26 $M_o(s)$ – 17.94 K-ft
(first column)

Exterior (-ve) moment – 0.70 $M_o(s)$ – 48.30 K-ft
(first interior column)

Exterior (+ve) moment – 0.31 $M_o(s)$ – 21.39 K-ft
(column strip)

Exterior (+ve) moment – 0.21 $M_o(s)$ – 14.49 K-ft
(middle strip)

Interior (+ve) moment – 0.21 $M_o(s)$ – 14.49 K-ft
(column strip)

Interior (+ve) moment – 0.14 $M_o(s)$ – 9.66 K-ft
(middle strip)

Gross moment of inertia of panel (17' wide)

$= (17\times12)(7)^3/12 = 5831$ in^4

Gross moment of inertia of the sum of one column strips or one middle strips in the panel $= \frac{1}{2}(5831) = 2915.5$ in^4

Cracking moment $(M_{cr}) = f_r \cdot I_g / y_t$ — Equation 24.2.3.5b

$= (474)(2915.5)\left(\frac{1}{3.5}\right)\left(\frac{1}{12{,}000}\right) - 32.9$ K-ft

(for each of middle or column strip)

If the cracking moment is compared with moments at different locations calculated for dead, dead + live, and sustained loads, there are moments less than the cracking moments

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The worst case is the first interior column. Hence deflections are checked at first interior column.

EFFECTIVE MOMENT OF INERTIA

Area of steel provided (A_s) – #4 @ 6" o.c + 6#5

$= (0.4)(8.5) + (3)(0.31) = 4.33\ in^2$

$$B = \frac{b}{n A_s} = \frac{(8.5 \times 12)}{(8.1)(4.33)} = 2.91\ (1/in.)$$

$$Kd = \frac{\sqrt{2dB+1} - 1}{B} = \frac{\sqrt{(2)(5.75)(2.91)+1} - 1}{2.91} = 1.64''$$

$$I_{cr} = \frac{b(Kd)^3}{3} + nA_s(d-Kd)^2$$

$$= \frac{(8.5 \times 12)(1.64)^3}{3} + (8.1)(4.33)(5.75-1.64)^2$$

$$= 742.5\ in^4$$

CALCULATE M_{cr}/M_a AT COLUMN STRIP – GRID '2'

For dead load, $M_{cr}/M_{a(d)} = (32.9/41.3)$

$[M_{cr}/M_{a(d)}]^3 = 0.51$

For dead + live, $M_{cr}/M_{a(d+l)} = (32.9/59.5)$

$[M_{cr}/M_{a(d+l)}]^3 = 0.17$

For sustained load, $M_{cr}/M_{a(s)} = (32.9/48.30)$

$[M_{cr}/M_{a(s)}] = 0.32$

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EFFECTIVE MOMENT OF INERTIA

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr}$$ Equation 24.2.3.5a

for dead load

$$I_e = (0.51)(2915.5) + (1-0.51)(742.5) = 1850.7 \text{ in}^4$$

for dead + live load

$$I_e = (0.17)(2915.5) + (1-0.17)(742.5) = 1111.9 \text{ in}^4$$

for sustained load

$$I_e = (0.32)(2915.5) + (1-0.32)(742.5) = 1437.9 \text{ in}^4$$

DEFLECTIONS

$$\Delta = \frac{5wL^4}{384 E I_e}$$

$$\Delta_d = \frac{5(113/12)(15.67\times12)^4}{384(3.6\times10^6)(1850.7)} = 0.023''$$

$$\Delta_{d+l} = \frac{5(163/12)(15.67\times12)^4}{384(3.6\times10^6)(1111.9)} = 0.055''$$

$$\Delta_s = \frac{5(133/12)(15.67\times12)^4}{384(3.6\times10^6)(1437.9)} = 0.035''$$

LONG-TERM DEFLECTIONS

$$\lambda_\Delta = \frac{\xi}{1+50\rho'}$$ Equation 22.2.4.1.1

$\because$ compressive steel $(\rho') = 0.0$; $\lambda_\Delta = \xi$.

$\xi = 2.0$ Table 22.2.4.1.3

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Multiply calculated deflections by 2.

$\Delta_d = 2(0.023) = 0.046''$

$\Delta_{d+l} = 2(0.055) = 0.110''$

$\Delta_s = 2(0.035) = 0.070''$

$(L/\Delta)_d = (15.67 \times 12)/0.046 = 4088$

$(L/\Delta)_{d+l} = (15.67 \times 12)/0.110 = 1709$

$(L/\Delta)_s = (15.67 \times 12)/0.070 = 2686$

Compare these values with allowable values in table 24.2.2 of the code. As expected the deflections are very small.

Appendix F: Beams

F.1 PROBLEM F.1: L-SHAPED BEAM

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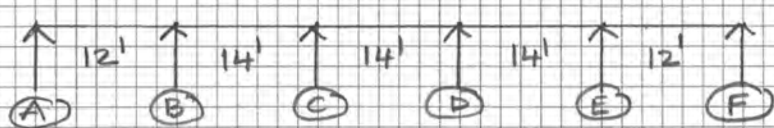
Design the beam on grids 'A' and 'E' of Fig: 1.15.3
as L-beam, $f'_c = 4$ ksi ; $f_y = 60$ ksi
Assume a beam size of 8" × 12"

Loads:

Slab weight − $5.87' \times \frac{6''}{12''} \times 150$ psf − 440 lb/ft
Super-imposed dead load − slab − 25×5.87 − 147 lb/ft
Wall weight − 9' × 65 psf − 585 lb/ft
Self weight of beam − $\frac{8'' \times 12''}{144} \times 150$ pcf − 100 lb/ft
Total dead load − 1272 lb/ft
Live load − 5.87' × 40 psf − 235 lb/ft
Factored Load − (1.2)(1272) + (1.6)(235) − 1902 lb/ft



The end supports are considered fixed
Moment distribution method is used to solve the beam.

Fixed end moments

for AB, EF − $1902 \times \frac{12^2}{12}$ − 22,824 lb-ft
− 22.9 K-ft

for BC, CD, DE − $1902 \times \frac{14^2}{12}$ − 31,066 lb-ft
− 31 K-ft

Distribution factors

All beams have same cross-section. Hence, stiffness would depend upon span

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Joint "A", $d_{AB} = 1$

Joint "B", $d_{BA} = 14'/26 = 0.54$; $d_{BC} = 12/26 = 0.46$

Joint "C", $d_{BC} = 0.5$, $d_{CD} = 0.5$

Joint "D", $d_{DC} = 0.5$, $d_{DE} = 0.5$

Joint "E", $d_{ED} = 0.46$; $d_{EF} = 0.54$

Joint "F", $d_{EF} = 1.0$

	A		B		C		D		E	F	
D.F	1	0.54	0.46	0.5	0.5	0.5	0.5	0.46	0.54	1	Distribution factor
FEM	-22.9	+22.9	-31	+31	-31	+31	-31	+31	-22.9	+22.9	fixed end moment
Dist.		+4.4	+3.7	0	0	0	0	-3.7	-4.4		Distribute
C.O	+2.2			+1.85			-1.85			-2.2	Carry over
Dist.				-0.925	-0.925	+0.925	+0.925				
Final moment	-20.7	+27.3	-27.3	+31.925	-31.925	+31.925	-31.925	+27.3	-27.3	+20.7	

Span moments are calculated by subtracting the average of the support moments from the simply supported moment, which is 1.5 times the fixed end moment

Span Moments

Spans AB, EF $= (1.5)(22.9) - \dfrac{(20.7 + 27.3)}{2} = 10.35$ K-ft

Spans BC, DE $= (1.5)(31.0) - \dfrac{(27.3 + 31.925)}{2} = 16.9$ K-ft

Span CD $= (1.5)(31.0) - \dfrac{(31.925 + 31.925)}{2} = 14.6$ K-ft

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The maximum support moment of 31.925 K-ft is used to design the top steel at support

The maximum span moment of 16.9 K-ft is used to design the bottom steel at span

SUPPORT REINFORCEMENT — Design as a rectangular section

Effective depth $(d) = 12 - 1.5 - 0.625/2 \cong 10''$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{31.925 \times 12000}{0.9 \times 8 \times 10^2} = 532.0 \text{ psi}$$

$$\rho = 0.85 \frac{f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right] =$$

$$= \frac{(0.85)(4000)}{60{,}000}\left[1 - \sqrt{1 - \frac{(2)(532)}{0.85 \times 4000}}\right] = 0.0103$$

$A_{st} = 0.0103 \times 8 \times 10 = 0.82 \text{ in}^2$ (3#5)

Check if beam is tension controlled,

Modulus of elasticity of concrete $- 57000\sqrt{f'_c}$ Equation 19.2.2.1.b

$= 57000\sqrt{4000} = 3.6 \times 10^6$ psi

Modular ratio $(n) = E_s/E_c = \dfrac{29 \times 10^6}{3.6 \times 10^6} = 8.1$

Depth of compression block $- Kd$, where

$\rho n = (0.0103)(8.1) = 0.0834$

$$K = \sqrt{(\rho n)^2 + 2\rho n} - \rho n = \sqrt{(0.0834)^2 + 2(0.0834)} - 0.0834$$

$$= 0.333$$

$$\rho_{bal.} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{87{,}000}{87{,}000 + f_y}$$

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$\beta_1 = 0.85$

$$\rho_{bal} = \frac{(0.85)(0.85)(4000)}{60{,}000}\frac{(87{,}000)}{(87{,}000+60{,}000)} = 0.0285$$ Equation 22.2.2.4.1

$\rho < \rho_{bal}$, hence compression controlled.

SPAN REINFORCEMENT — design as L-section.

flange width of L-section – least of: Table 6.3.2.1

(a) $6h$ (b) $s_w/2$ (c) $l_n/12$

(a) $6 \times 6 = 36''$

(b) $(12'-8'')/2 = 68''$

(c) $(12'-16'')/12 = 10.7''$

Hence flange width of L-section – $10.7''+8''$

– $18.7''$

Assuming a stress block depth of $6''$ (thickness of slab)

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{16.9 \times 12{,}000}{0.9\,(60{,}000)(10 - 6/2)} = 0.53\,in^2$$

Equivalent stress block depth $(a) = \dfrac{A_s f_y}{0.85 f'_c b}$

$$= \frac{0.53 \times 60{,}000}{0.85 \times 4{,}000 \times 8} = 1.2'' < 6''$$

Hence beam has the neutral axis in the flange and can be designed as a rectangular section.

$$R_n = \frac{M_u}{\phi b d^2} = \frac{16.9 \times 12000}{0.9 \times 8 \times 10^2} = 282\ psi$$

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$$\rho = \frac{(0.85)(4000)}{60{,}000}\left[1-\sqrt{1-\frac{(2)(282)}{(0.85)(4000)}}\right] = 0.00491$$

$$> \rho_{min} = \frac{200}{60{,}000} = 0.00333$$

$A_{st} = 0.00491 \times 8 \times 10 = 0.393\ in^2$ (2#5)

Provide 2#5 top and bottom

and 1#5 extra at support

Shear Force

For simply supported condition of span(1)

$$V_{span(AB)\,L,R} = 1.902 \times \frac{12'}{2} = 11.41\ K$$

For simply supported condition of span(2)

$$V_{span(BC)\,L,R} = 1.902 \times \frac{14'}{2} = 13.31\ K$$

End moment reaction (couple)

$$R_A' = \frac{27.3 - 20.7}{12} = 0.55^K$$

$$R_B' = \frac{27.3 - 27.3}{12} = 0$$

Support reactions

$R_A = 11.41 - 0.55 = 10.86^K$

$R_B = 11.41 + 0.55 + 13.31 = 25.27^K$

$R_C = 13.31^K$

$R_D = 25.27^K$

$R_E = 10.86^K$

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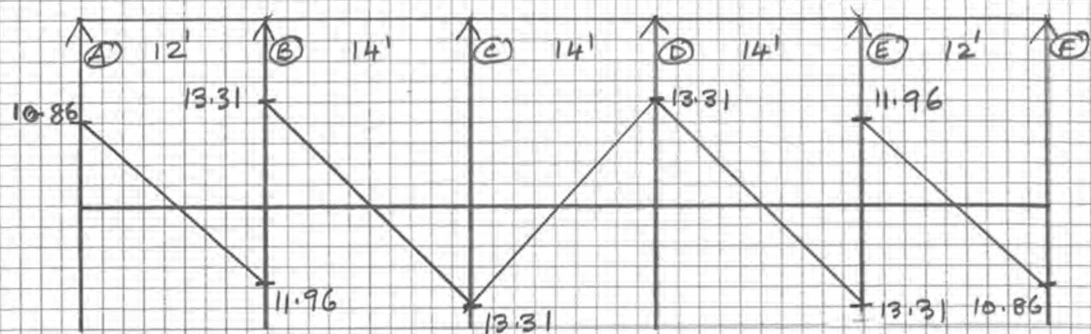

Design for maximum shear at a location 'd' from the face of the support. — Section 9.4.3.2

$V_u = 13.31 - \left(\frac{16''}{2} + 10''\right)\frac{(1.902)}{12} = 10.5\text{ K}$

Shear capacity of concrete $(V_c) = 2\sqrt{f'_c}\,bwd$ — Equation 22.5.5.1

$= \frac{2\sqrt{4000}\,(8)(10)}{1000} = 10.12\text{ K}$

Shear to be provided by stirrups $(V_s) \geq \frac{V_u}{\phi} - V_c$ — Equation 22.5.10.1

$V_s \geq \frac{10.5}{0.75} - 10.12 = 3.88\text{ K}$

Using #3 two-legged stirrups, $A_v = 2 \times 0.1 = 0.2\text{ in}^2$ — Equation 22.5.10.3

Spacing of stirrups $(S) = \frac{A_v f_{yt} d}{V_s} = \frac{(0.2)(60)(10)}{3.88}$

$= 30''$

$4\sqrt{f'_c}\,bwd = \frac{4\sqrt{4000}\,(8)(10)}{1000} = 20.24\text{ K}$

$V_s = 3.88 < 4\sqrt{f'_c}\,bwd$ — Table 9.7.6.2.2

Hence max. spacing of stirrups $= \frac{d}{2} = \frac{10''}{2} = 5''$

Cross-sectional dimensions shall be selected such that: $V_u \leq \phi(V_c + 8\sqrt{f'_c}\,bwd)$ — Equation 22.5.1.2

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$$\phi(V_c + 8\sqrt{f'_c}\, b_w d) = 0.75\left(10{,}120 + \frac{8\sqrt{4000}\,(8)(10)}{1000}\right)$$

$$= 37.9 > 10.5\text{K}$$

Beam details 8"×12" w/2#5 bars (top & bottom)
w/#3 stirrups @ 5" o.c

F.2 PROBLEM F.2: BEAM WITH CONCENTRATED LOAD

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Refer to fig: 1.15.3 – second floor plan of building 'B'. On grid lines 'C' and 'D', the columns at grid lines '2' and '5' terminate at second floor, thus making the beams at 'C' and 'D' (3 spans continuous beams), a transfer beam. From page 3/14 of problem 3.1 the loads of the columns at their base (at the second floor) are:

Service load – 255,494 lbs

Factored load – 329,737 lbs ≃ 330 K

The beam is 16" wide and the columns are 16"×16". Use $f'_c = 10$ ksi, $f_y = 60$ ksi

Analyze and design the beam.

UNIFORM DEAD LOAD

Self weight of slab – $\frac{6''}{12''} \times 150\text{ pcf} \times 12.67'$ (clear) – $950\ \frac{lb}{ft}$

Super-imposed dead load of slab – $25\text{ psf} \times 14'$ – $350\ \frac{lb}{ft}$

Self weight of beam – $\frac{16'' \times 48''}{144} \times 150\text{ pcf}$ – $800\ \frac{lb}{ft}$

Total dead load – $2100\ \frac{lb}{ft}$

Live load – $40\text{ psf} \times 14'$ – $560\ \frac{lb}{ft}$

Factored load – $(1.2)(2100) + (1.6)(560)$ – $3416\ \frac{lb}{ft}$

$\simeq 3.4\ K/ft$

The beam is analyzed using the three moments equation, where the end supports are assumed fixed

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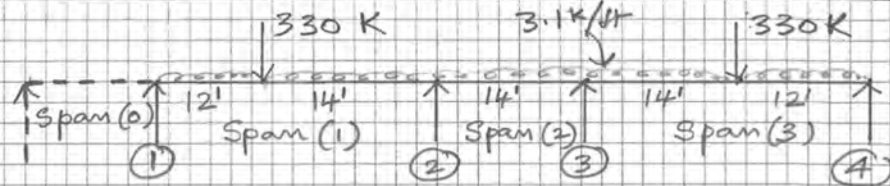

By symmetry, $M_4 = M_1$ and $M_3 = M_2$

Applying Three moments equation between span(0) and span (1)

$$M_0 L_0 + 2M_1(L_0 + L_1) + M_2 L_1 + \frac{6A_0\bar{a}_0}{L_0} + \frac{6A_1\bar{b}_1}{L_1} = 0$$

$$0 + 2M_1(0+26) + M_2(26) + 0 + \frac{1}{4}(3.4)(26)^3$$
$$+ \frac{330(14)(26^2 - 14^2)}{26} = 0$$

Simplifying, $2M_1 + M_2 + 3855 = 0$ Eqn (1)

For span (1) and span(2)

$$M_1 L_1 + 2M_2(L_1 + L_2) + M_3 L_2 + \frac{6A_1\bar{a}_1}{L_1} + \frac{6A_2\bar{b}_2}{L_2} = 0$$

$$M_1(26) + 2M_2(26+14) + M_2(14) + \frac{1}{4}(3.4)(26)^3 \qquad M_3 = M_2$$
$$+ \frac{330(12)(26^2 - 12^2)}{26} + \frac{1}{4}(3.4)(14)^3 = 0$$

Simplifying, $M_1 + 3.62 M_2 + 3691 = 0$ Eqn (2)

Solving eqn(s) (1) & (2),

$M_1 = -1645$ K-ft ; $M_2 = -565$ K-ft

Shear force

For simply supported condition of span(1)

$$V_{span(1)L} = 3.4 \times 13 + 330 \times \frac{14}{26} = 221.9 \text{ K}$$

$$V_{span(2)R} = 3.4 \times 13 + 330 \times \frac{12}{26} = 196.5 \text{ K}$$

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For simply supported condition of span (2)

$V_{span(2)L} = V_{span(2)R} = 3.4 \times \frac{14'}{2} = 23.8$ K

End moment reaction (couple)

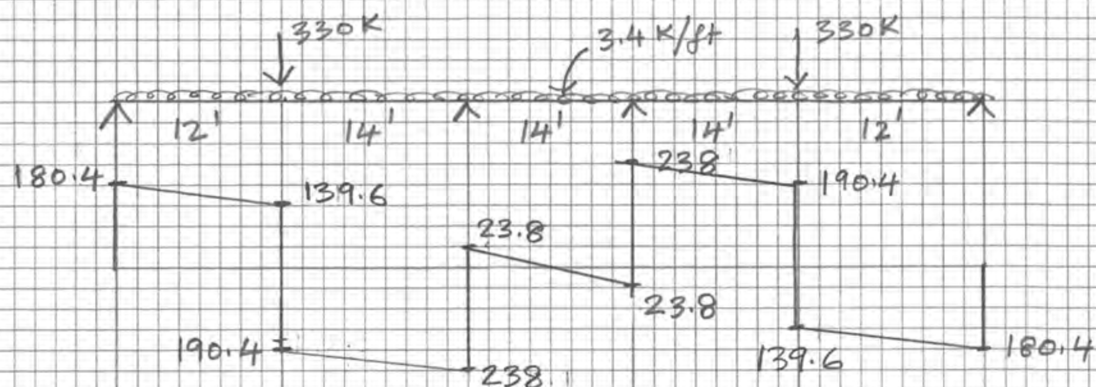
$R_1' = (1645 - 565)/26 = 41.5$ K.

$R_2' = (565 - 565)/14 = 0$ K

Support reactions

$R_1 = R_4 = 221.9 - 41.5 = 180.4$ K

$R_2 = R_3 = 23.8$ K.



Max: B.M for span (1) $= 180.4 \times 12 - 3.4 \times \frac{12^2}{2} = 1920$ K-ft

Max: B.M for span (2) $= 3.4 \times \frac{14^2}{8} - 565 = -482$ K-ft

Hence span (2) is completely in hogging.

Use # 10 bars and 1.5" clear cover

$d_{eff} = 48 - 1.5 - \frac{1.25''}{2} = 45.8''$

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Designing reinforcement

Support moments $M_1 = M_4$ and $M_3 = M_2$

There is no positive moment in span (2)

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

$$R_n = \frac{M_u}{\phi b d^2}$$

Location	M_u (K-ft)	R_n psi	ρ	A_{st} (in²)	Bars
M_1 & M_4 Top bars	1645	619	0.0107	7.84	5#11
M_2 & M_3 Top bars	565	212	0.0036	2.64	2#11
Span 1 Bottom bars	1920	722	0.0126	9.23	2#18 + 1#10

For M_2 and M_3 top bars, $\rho_{min} = 200/f_y$ — Section 9.6.1.2

$= 200/60000 = 0.00333$

Designing stirrups

Shear capacity of concrete $(V_c) = 2\sqrt{f'_c}\, b_w d$ — Equation 22.5.5.1

$= 2\sqrt{10{,}000}\,(16)(45.8)/10{,}000 = 146.6$ K

Since there is a stress concentration under the concentrated load, the shear force used to design under the load — 330 K

$$V_s \geq \frac{V_u}{\phi} - V_c = \frac{330}{0.75} - 146.6 = 293.4 \text{ K}$$

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Select #5 2 legged stirrups spaced @ 5" o.c

$V_s = \frac{(2\times 0.31)(60)(45.8)}{5} = 340.7 > 293.4\ k$

In order to economize the design, stirrups at other locations can be spaced at a longer distance.

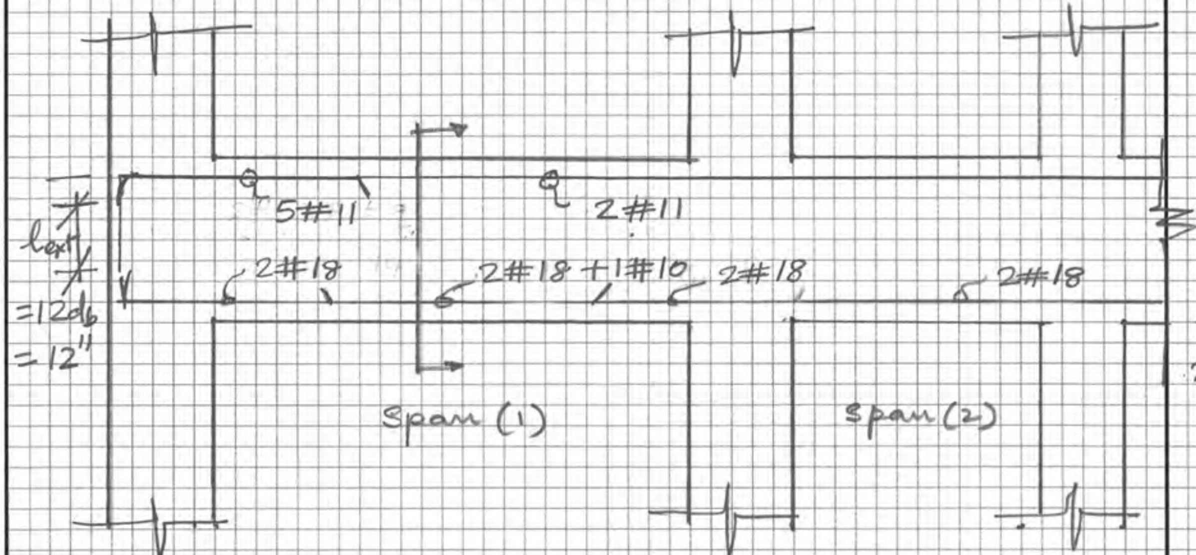
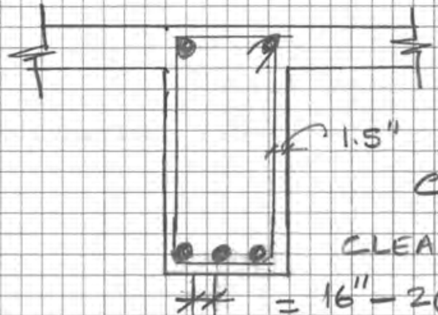


Table 25.3.1



CLEAR COVER = 1.5"

CLEAR SPACING BETWEEN BARS

$= \frac{16'' - 2(1.5'') - 2(0.625) - 2(2.25) - 1.75}{2}$

$= 4.25'' > 1'',\ d_b,\ (4/3)(d_{agg})$ Section 25.2.1

$d_b = 1''$

$(4/3)(d_{agg}) = \frac{4}{3}\times\frac{3}{4} = 1''$

(Hence ok)

F.3 PROBLEM F.3: TORSION

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For the figure shown below, design the beam for torsion and shear. The beam is supporting a 36" cantilever with a super-imposed dead load of 20 psf and a live load of 25 psf. Use $f'_c = 5$ Ksi ; $f_y = 60$ Ksi

Assume a beam depth of 24"

Self weight of beam $= \dfrac{12'' \times 24''}{144} \times 150\,\text{pcf} = 300\ \dfrac{lb}{ft}$

Live load on beam $= 1' \times 25\,\text{psf} = 25\ \dfrac{lb}{ft}$

Factored load $= (1.2)(300) + (1.6)(25) = 400\ \dfrac{lb}{ft}$

<u>Torsional load</u>

Weight of slab $= \dfrac{6''}{12''} \times 150\,\text{pcf} \times 3' = 225\ \dfrac{lb}{ft}$

Super-imposed dead load $= 3' \times 20\,\text{psf} = 60\ \dfrac{lb}{ft}$

Total dead load $= 285\ \dfrac{lb}{ft}$

Live load $= 3' \times 25\,\text{psf} = 75\ \dfrac{lb}{ft}$

Factored load $= (1.2)(285) + (1.6)(75) = 462\ \dfrac{lb}{ft}$

Total factored load $= 400 + 462 = 862\ \dfrac{lb}{ft}$

End Shear $(V_u) = 862 \times 16'/2 = 6896$ lbs.

Eccentricity of slab load $= \dfrac{12''}{2} + \dfrac{36''}{2} = 24'' = 2'$

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End torsional moment $(M_u) = 2'\left(\frac{16'}{2}\right)(462) = 7392$ lb-ft

Effective depth $= 24'' - 1.5'' - \frac{0.75''}{2} = 22.125''$

(Assuming 1.5" clear cover and #6 bars for longitudinal reinforcement)

Adopt an effective depth $(d) = 22''$

Enclosed area of outside perimeter of beam (A_{cp})

$= 12'' \times 24'' = 288 \text{ in}^2$

Outside perimeter of beam $(A_{cp}) = 2(12+24) = 72''$

Threshold torsion $(T_{th}) = \phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$ — Table 22.7.4

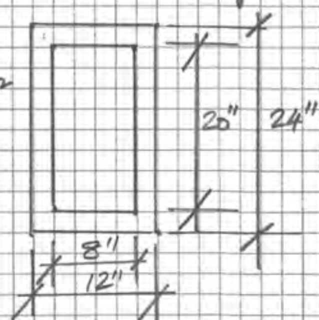
$= 0.75\sqrt{5000}\left(\frac{288^2}{72}\right) = 61094$ lb-in $= 5091$ lb-ft

$\phi T_{th} < T_u$, hence torsion cannot be neglected — Section 9.5.4.1

Using #4 stirrups and 1.5" clear cover to stirrups

$A_{oh} = 8 \times 20 = 160 \text{ in}^2$ — Section 22.7.6.1

$A_o = 0.85 A_{oh} = (0.85)(160) = 136 \text{ in}^2$



$T_n = \frac{2 A_o A_t f_{yt} \cot\theta}{s}$

$\frac{A_t}{s} = \frac{T_u}{2\phi A_o f_y \cot\theta}$

θ is assumed as $45°$ for solid section

$\therefore \frac{A_t}{s} = \frac{7392 \times 12}{(2)(0.75)(136)(60{,}000)(1)} = 0.00725 \text{ in}^2/\text{in}$

Shear capacity of concrete $(V_c) = 2\sqrt{f'_c}\, b_w d$ — Equation 22.5.5.1

$= 2\sqrt{5000}\,(12)(22) = 37{,}335$ lbs

$0.5(\phi)V_c = 0.5(0.75)(37{,}335) = 14{,}000$ lbs

$V_u < 0.5(\phi)V_c$, Min. shear reinforcement not required — Section 9.6.3.1

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	ACI Code
Maximum spacing of torsional reinforcement is less of (a) $P_h/8$ (b) 12"	Section 9.7.6.3.3
$P_h = 2(20+12) = 64''$; $P_h/8 = 8''$	
Provide 2 legged #3 stirrups @ 8" o.c	
Check for crushing of concrete compression struts	
$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)} \leq \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$	Equation 22.7.7.1a
$\sqrt{\left(\frac{6896}{12\times22}\right)^2 + \left(\frac{7392\times12\times64}{1.7\times160^2}\right)^2} = 133\text{ psi}$	
$0.75\left(\frac{37335}{12\times22} + 8\sqrt{5000}\right) = 530\text{ psi}$	OK
Longitudinal Torsional Reinforcement	Equation 22.7.6.1.b
$T_n = \frac{2A_oA_\ell f_{yt}}{P_h}\tan\theta$	
$A_\ell = \frac{7392\times12\times64}{2(0.75)(160)(60{,}000)(1)} = 0.39\text{ in}^2$	
A_ℓ shall be at least the lesser of:	Section 9.6.4.3
(a) $\frac{5\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{A_t}{s}\right)P_h\left(\frac{f_{yt}}{f_y}\right)$	
(b) $\frac{5\sqrt{f'_c}A_{cp}}{f_y} - \left(\frac{25b_w}{f_y}\right)P_h\left(\frac{f_{yt}}{f_y}\right)$	
(b) $\frac{5\sqrt{5000}(288)}{60{,}000} - \left(\frac{25\times12}{60{,}000}\right)(64)\left(\frac{60{,}000}{60{,}000}\right) = 1.38\text{ in}^2$	
(a) $\frac{5\sqrt{5000}(288)}{60{,}000} - (0.00725)(64)\left(\frac{60{,}000}{60{,}000}\right) = 1.23\text{ in}^2$	
Hence, $A_\ell = 1.23\text{ in}^2$ Use 2#5 at each face	

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ACI Code

Moment acting on the beam $(M_u) = \frac{862 \times 16^2}{8}$

$= 27{,}584$ lb-ft

$R_n = \frac{M_u}{\phi b d^2} = \frac{27{,}584 \times 12}{(0.9)(12)(22)^2} = 63.3$ psi

$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$

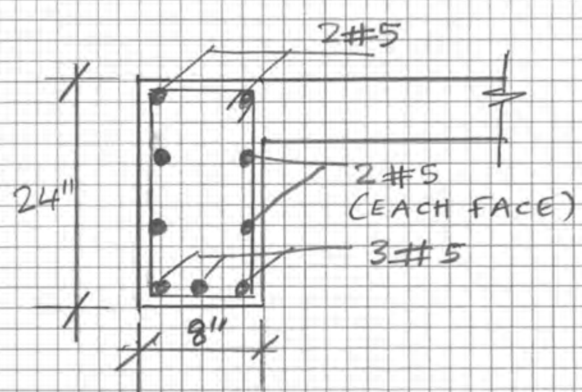
$= \frac{(0.85)(5000)}{60{,}000}\left[1 - \sqrt{1 - \frac{(2)(63.3)}{(0.85)(5000)}}\right] = 0.001$

Minimum steel is larger of (a) $\frac{3\sqrt{f'_c}}{f_y}$ (b) $\frac{200}{f_y}$ Section 9.6.12

$\frac{3\sqrt{5000}}{60{,}000} = 0.00354$; $\frac{200}{60{,}000} = 0.00333$

$A_{st} = 0.00333 \times 12 \times 22 = 0.879$ in^2

Provide 3#5 bars



F.4 PROBLEM F.4: CORBEL

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ACI Code

Design a corbel for a 16" × 16" column to support a structural steel beam, 16" wide

Dead load reaction of beam = 100^K

Live load reaction of beam = 80^K

Horizontal reaction of beam = 36^K

$f'_c = 5$ Ksi ; $f_y = 60$ Ksi

Factored shear $V_u = (1.2)(100) + (1.4)(80) = 232^K$

$$\frac{V_u}{\phi} = \frac{232}{0.75} = 309^K$$

Select a corbel of size 16" × 24"

Effective depth of corbel $= 24" - 1.5" - \frac{1"}{2} = 22"$

(clear cover − 1.5", #8 bars)

Max: V_u shall be less than the least of: — Section 16.5.2.4

(a) $0.2 f'_c b_w d = (0.2)(5000)(16)(22) = 352{,}000$ lbs

(b) $(480 + 0.08 f'_c) b_w d = (480 + 0.08 \times 5000)(16)(22)$
$= 309{,}760$ lbs

(c) $1600 b_w d = (1600)(16)(22) = 563{,}200$ lbs — Hence OK

Length of bearing

$V_u \le \phi B_n$ — Equation 22.8.3.1

Assuming 16" wide steel beam

$\phi B_n = \phi (0.85 f'_c A_1)$

$$A_1 = \frac{V_u}{\phi (0.85) f'_c} = \frac{232}{(0.65)(0.85)(5)} = 84 \text{ in}^2$$

Length of bearing $= \frac{84}{16} = 5.25"$ Adopt 6"

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	ACI Code
Shear span (a_v) = 6"/2 = 3"	
$N_{uc} = (1.6)(36) = 57.6$ K (horizontal reaction)	Commentary R16.5.3.4
$0.2 V_u = 0.2 \times 232 = 46.4$ K.	Section 16.5.3.5
Hence $N_u > 0.2 V_u$	
Design Moment (M_u) = $[V_u a_v + N_{uc}(h-d)]$	Section 16.5.3.1
$= (232)(3) + (57.6)(24-22) = 811.2$ k-in $= 67.6$ k-ft	

Flexural reinforcement

$$R_n = \frac{M_u}{\phi b d^2} = \frac{67.6 \times 12000}{0.75 \times 16 \times 22^2} = 140 \text{ psi}$$

(ϕ of 0.75 is used for corbel calculations)

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

$$= \frac{0.85 \times 5000}{60{,}000}\left[1 - \sqrt{1 - \frac{2 \times 140}{0.85 \times 5000}}\right] = 0.0023$$

Use $\rho_{min} = 0.00333$

$A_f = 1.17$ in^2

Shear friction reinforcement (Equation 22.9.4.2)

$$V_n = \frac{V_u}{\phi} = A_{vf} f_y$$

$$A_{vf} = \frac{V_u}{\phi f_y} = \frac{232}{(0.75)(60)} = 5.16 \text{ in}^2$$

Direct tension reinforcement (Equation 16.5.4.3)

$$N_n = \frac{N_{uc}}{\phi} = A_n f_y$$

$$A_n = \frac{N_{uc}}{\phi f_y} = \frac{57.6}{(0.75)(60)} = 1.28 \text{ in}^2$$

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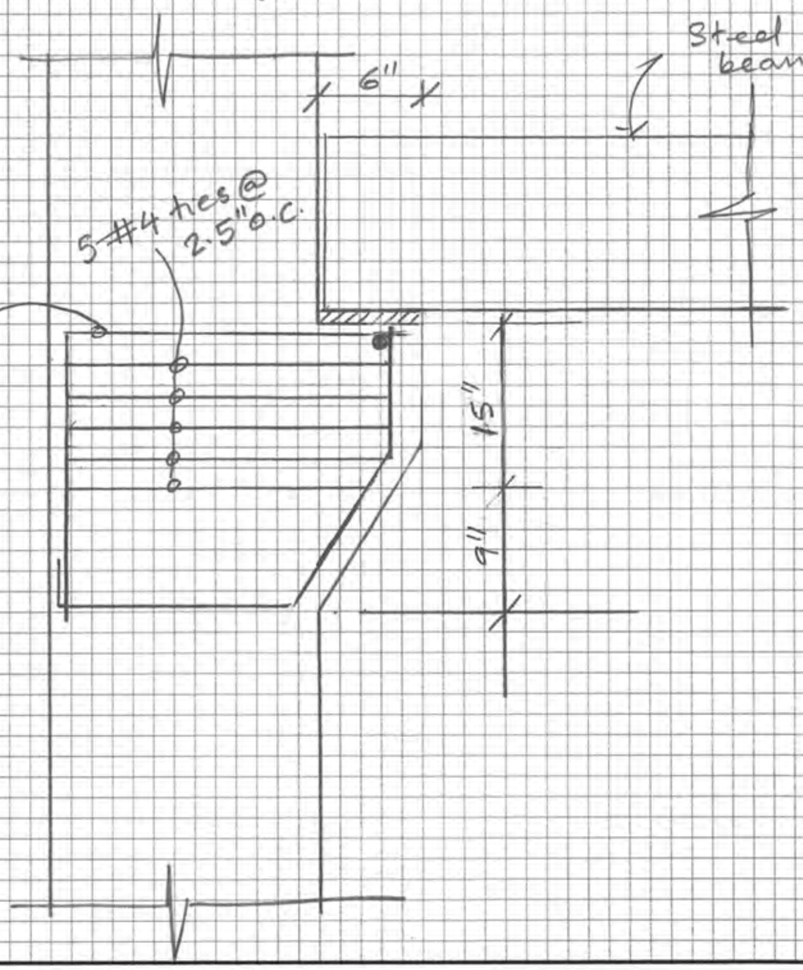
	ACI Code
Primary tension reinforcement	
Area of primary tension reinforcement (A_{sc}) shall be at least the greatest of:	Section 16.5.5.1
(a) $A_f + A_n = 1.17 + 1.28 = 2.45\ in^2$	
(b) $(2/3)A_{vf} + A_n = (2/3)(5.16) + 1.28 = 4.72\ in^2$	
(c) $0.04(f'_c/f_y)(b_w d) = (0.04)(5/60)(16)(22) = 1.17\ in^2$	
Shear reinforcement (A_h) $= 0.5(A_{sc} - A_n)$	Equation 16.5.5.2
$A_h = 0.5(4.72 - 1.28) = 1.72\ in^2$	
Provide 5 #4 2 legged ties @ 2.5" o.c.	



Appendix G: Columns

G.1 PROBLEM G.1: SWAY CONSIDERED

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ACI Code

Refer to Fig: 1.15.3 of the book for the second floor layout of the building type 'A'. The factored axial load of the column located at the intersection of grid lines 'A' and '3' is $P_U = 248^K$ (See example 3 of chapter 1).

The axial loads at the base of the columns located on the second floor are as follows:

A-1, A-6, F-1, F-6	– 4 × 156,477	– 625,908 lbs
A-2, A-5, F-2, F-5	– 4 × 232,876	– 931,504 lbs
A-3, A-4, F-3, F-4	– 4 × 248,091	– 992,364 lbs
B-1, E-1, B-6, E-6	– 4 × 232,338	– 929,352 lbs
B-2, E-2, B-5, E-5	– 4 × 310,370	– 1,241,480 lbs
B-3, E-3, B-4, E-4	– 4 × 337,710	– 1,350,840 lbs
C-1, D-1, C-6, D-6	– 4 × 241,577	– 966,044 lbs
C-2, D-2, C-5, D-5	– 4 × 329,737	– 1,318,948 lbs
C-3, D-3, C-4, D-4	– 4 × 352,771	– 1,411,084 lbs

Total – 9,767,524 lbs – 9768 K

Sway moment due to wind at top & bottom of column A-3: M_1 – 175 K-ft, M_2 – 195 K-ft

Total shear due to wind at second floor – 33,713 lbs

Factored shear at second floor $= \dfrac{0.8 \times 33{,}713}{1000}$

– 27.0 K

Relative lateral deflection between second and third floors of the building – 0.1"

f'_c for column – 10 ksi, f'_c for beams – 5 ksi

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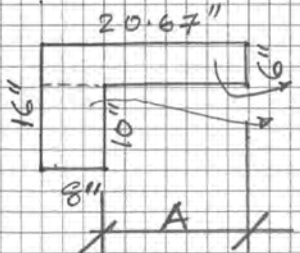
ACI Code

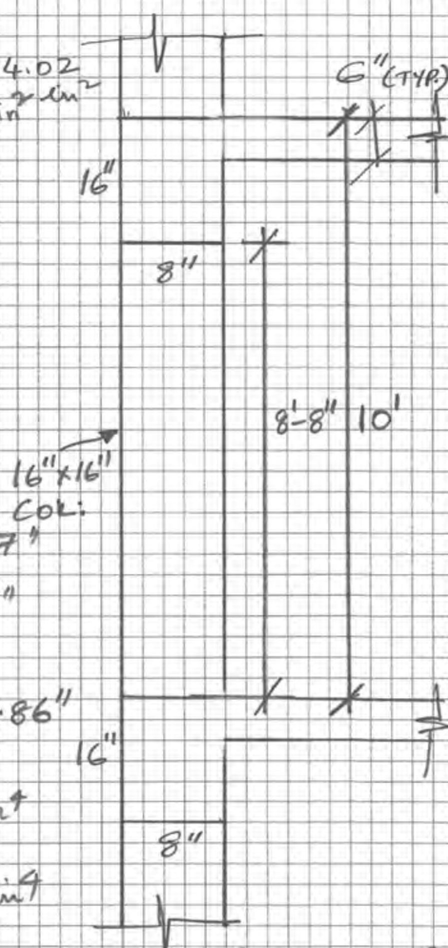
$f_y = 60$ ksi. Design the column "A-3" located in the building not protected against side sway.

Stability index $(Q) = \dfrac{\Sigma P_u \Delta_o}{V_u l_c} = \dfrac{9768 \times 0.1}{270 \times 10' \times 12''} = 0.03$ — Equation 6.6.4.4.2



$A = 20.67 \times 6 = 124.02 \text{ in}^2$

$A = 8 \times 10 = 80 \text{ in}^2$



Distance 'A' is least of: — Table 6.3.2.1

(a) $6h = 6 \times 6 = 36''$

(b) $S_w/2 = 12' \times 12''/2 = 72''$

(c) $l_n/12 = (14' - 16'')/12 = 12.67''$

Hence flange width $= 8'' + 12.67'' = 20.67''$

Centroid of L-beam (y_b)

$$= \frac{(8 \times 10 \times 5) + (20.67 \times 6 \times 13)}{(8 \times 10) + (20.67 \times 6)} = 9.86''$$

$$I_1 = \frac{20.67 \times 6^3}{12} = 372.06 \text{ in}^4$$

$$I_2 = \frac{8 \times 10^3}{12} = 666.67 \text{ in}^4$$

$$I_{beam} = 666.67 + 372.06 + 124.02 \times 3.14^2 + 80 \times 4.86^2 = 4151 \text{ in}^4$$

$$I_{col} = \frac{16 \times 16^3}{12} = 5461 \text{ in}^4$$

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ACI Code

$E_c = 57{,}000\sqrt{f'_c}$ — Equation 19.2.2.1b

E_c for column $= 57{,}000\sqrt{6{,}000} = 4.41 \times 10^6$ psi

E_c for beam $= 57{,}000\sqrt{5{,}000} = 4.03 \times 10^6$ psi

β_{dns} – ratio of maximum factored sustained load to maximum factored axial load associated with the same load combination

$\beta_{dns} = 0$ because wind is not a sustained load

$(EI)_{eff}$ for column $= \dfrac{0.4 E_c I_g}{1+\beta_{dns}}$ — Equation E6.6.4.4a

$= \dfrac{(0.4)(4.41\times10^6)(0.7\times5461)}{(1+0)(1000)} = 6.74\times10^6$ K-in²

$(EI)_{eff}$ for beam $= E_c(0.35 I_g)$

$= \dfrac{4.03\times10^6\times0.35\times4151}{1000} = 5.9\times10^6$ K-in²

$(EI/l_c)_{col} = \dfrac{6.74\times10^6}{10'\times12''} = 56.1\times10^3$ K-in

$(EI/l)_{beam} = \dfrac{5.9\times10^6}{10'\times12''} = 49.2\times10^3$ K-in

All beams and columns have the same value of (EI/l)

At top of the column,

$G_A = (\Sigma EI/l_c)_{col} / (\Sigma EI/l)_{beam}$

$= \dfrac{2\times56.1\times10^3}{2\times49.2\times10^3} = 1.14$

At bottom of the column,

$G_B = \dfrac{2\times56.1\times10^3}{2\times49.2\times10^3} = 1.14$

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From the alignment chart for sidesway — Figure R.6.2.5

$K = 1.45$

Critical load $(P_c) = \dfrac{\pi^2 (EI)_{eff}}{(K l_u)^2}$ — Equation 6.6.4.4.2

Dimensions and concrete strength of all columns are same. Dimensions of all beams are same implying the unsupported length (l_u) of all columns are same. Hence, P_c is calculated for one column and multiplied by 36 to obtain ΣP_c.

$P_c = \dfrac{\pi^2 (6.74 \times 10^6)}{(1.45 \times 8.67 \times 12)^2} = 2925\ K.$

Radius of gyration $(r) = \sqrt{\dfrac{I_g}{A_g}} = \sqrt{\dfrac{\frac{1}{12} \times 16^4}{16^2}} = 4.62$ — Equation 6.2.5.1

$(K l_u / r) = \dfrac{1.45 \times 8.67 \times 12}{4.62} = 32.7 < 40$ — Equation 6.2.5c

Hence, slender effect is neglected

Moment magnification factor (δ_s) is calculated — Equation 6.6.4.6.2.a

(i) $\delta_s = \dfrac{1}{1 - Q} = \dfrac{1}{1 - 0.03} = 1.03$

(ii) $\delta_s = \dfrac{1}{1 - \dfrac{\Sigma P_u}{0.75 \Sigma P_c}} = \dfrac{1}{1 - \dfrac{9768}{0.75 \times 36 \times 2925}} = 1.13$

Second order analysis not performed

Adopt $\delta_s = 1.13$

Non-sway moments are not calculated in this problem. Only sway moments considered

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Magnified Moment

Lesser moment $(M_1) = M_{1ns} + \delta_s M_{1s}$ — Equations 6.6.4.6.1.a

$= 0 + (1.13)(175)(1.0) = 198$ K-ft

Greater moment $(M_2) = M_{2ns} + \delta_s M_{2s}$ — 6.6.4.6.1b

$= 0 + (1.13)(195)(1.0) = 220$ K-ft

Wind load factor of 1.0 as worst case — Table 5.3.1

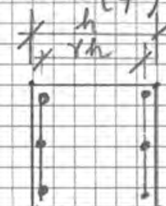
Column design for:

$P_u = 248$ K; $M_{1u} = 198$ K-ft; $M_{2u} = 220$ K-ft

$f'_c = 6$ ksi; $f_y = 60$ ksi

All the load combination can be tried but here only one load combination is considered to design the column.

Strength reduction factor for column $(\phi) = 0.65$ — Table 21.2.2



$h = 16"$; $\gamma h = 12"$

$\therefore \gamma = \frac{12"}{16"} = 0.8$

$K_n = \frac{P_n}{f'_c A_g} = \frac{248}{(0.65)(6)(16^2)} = 0.25$

$R_n = \frac{M_n}{f'_c A_g h} = \frac{220 \times 12}{(0.65)(6)(16^2)(16)} = 0.17$

from ACI interaction diagram

$\rho_g = 0.035$; $A_{st} = 0.035 \times 16^2 = 8.96$ in^2 — 10#9

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ACI Code

Design for tie

Use #3 ties for #9 longitudinal bars — Section 25.7.2.2

Spacing of ties — Section 25.7.2.1

(a) Clear spacing at least (4/3) size of aggregate

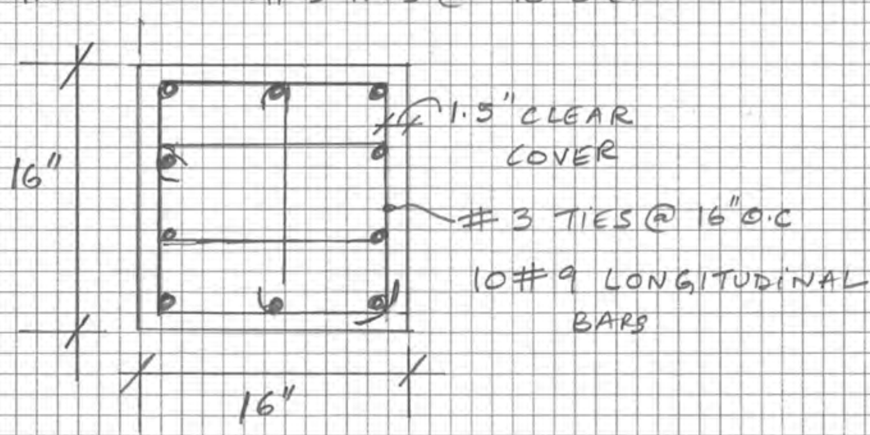
$\frac{4}{3} \times \frac{3}{4} = 1''$

(b) Center to center spacing not to exceed 16 times bar diameter of longitudinal bar, 48 times bar diameter of tie and smallest dimension of column

$16 \times \frac{9}{8} = 18''$; $48 \times \frac{3}{8} = 18''$; $\underline{16''}$

governs

Hence use #3 ties @ 16" o.c.



G.2 PROBLEM G.2: LONG COLUMN

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ACI Code

In a multi-storied building, braced against side sway, the atrium column is 30' high. The atrium is at the ground floor and has a suspended slab supported on grade beams. Beams frame into top of column (third floor) and beams frame into bottom of column (grade beam) are 16"×24". Design a 24"×24" column to support the following loads:

Use $f'_c = 5$ Ksi, $f_y = 60$ Ksi

Type of Load	Dead Load	Live Load	Factored Load
Axial Load	600^K	400^K	1386.64K
Top Moment	+100 K-ft	+80 K-ft	248 K-ft
Bottom Moment	+80 K-ft	+60 K-ft	192 K-ft

Weight of column − 28.67×2'×2'×150 pcf = 17,200 lbs

Use load combination of 1.2D + 1.6L to calculate factored loads, which are tabulated

Unsupported height of the column (l_u) = 28.67'

c/c length of column (l_c) = 28.67' + 2' = 30.67'

(Refer to the figure on next page)

Radius of gyration (r) $= \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{1}{12}\times 24^4}{24^2}} = 6.93$ — Equation 6.2.5.1

Overhanging width of T-beam flange is least of: — Table 6.3.2.1

(a) $8h = 8\times 8'' = 64''$

(b) $S_w/2 = 16'/2 = 4' = 48''$

(c) $l_n/4 = (25'-2')/4 = 5.75' = 69''$ l_n – clear span

Hence adopt a flange width of 48+16 = 64"

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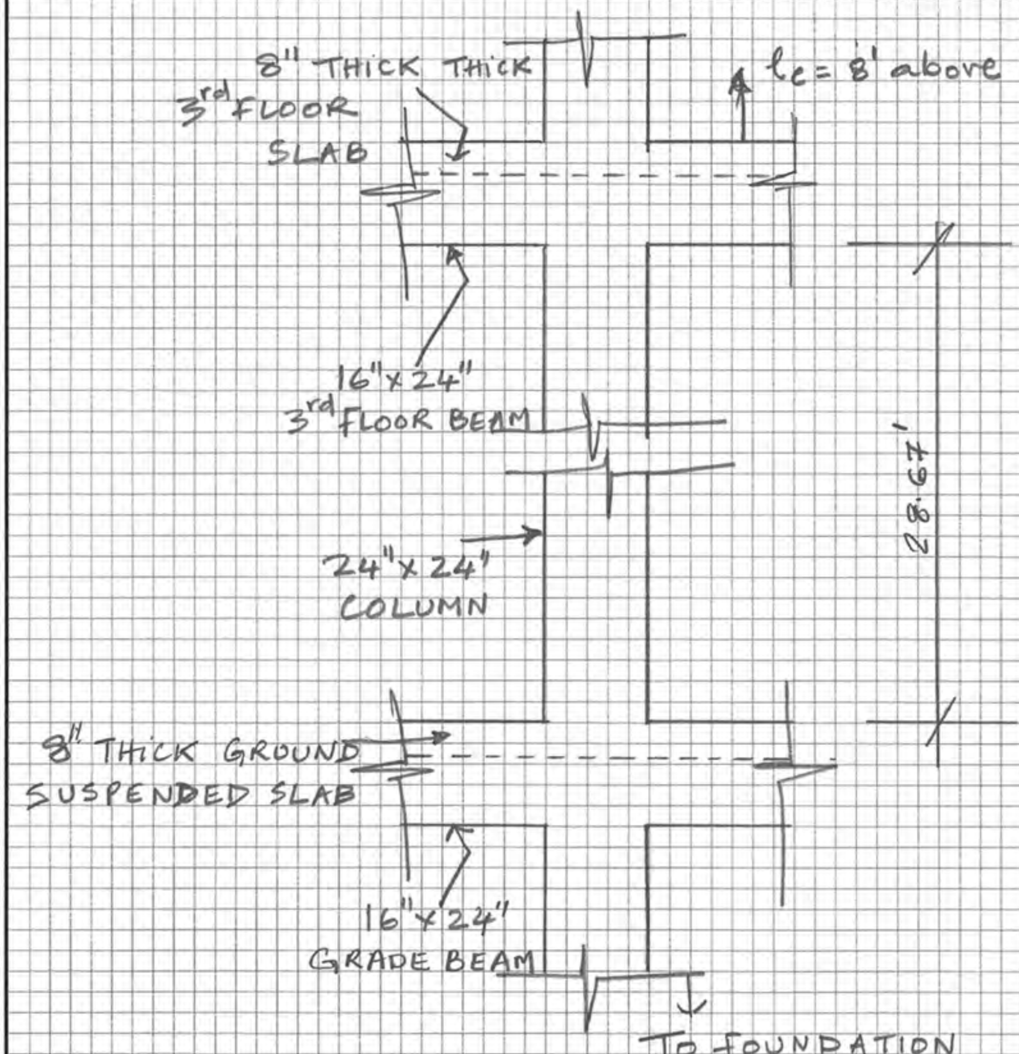

I_{x-x} for T-beam $= 32768 \text{ in}^4$

I_g for column $= \frac{1}{12}(24)(24)^3 = 27{,}648 \text{ in}^4$

Modulus of elasticity of concrete $(E_c) = 57{,}000\sqrt{f'_c}$ — Equation 19.2.2.1.b

$= 57{,}000\sqrt{5000} = 4.03 \times 10^6$ psi

β_{dns} – ratio of maximum factored sustained load to maximum factored axial load associated with same load combination $\beta_{dns} = 0.6$ — Section R6.6.4.4.4

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ACI Code

$(EI)_{eff}$ for column $= \dfrac{0.4 E_c I_g}{1+\beta_{dns}}$ — Equation 6.6.4.4.4a

$= \dfrac{(0.4)(4.03\times10^6)(27{,}648)}{(1+0.6)(1000)} = 27.9\times10^6$ K-in^2

$(EI)_{eff}$ for beam $= E_c(0.35 I_g)$

$= \dfrac{4.03\times10^6\times0.35\times32768}{1000} = 46.2\times10^6$ K-in^2

$(EI/l_c)_{col} = \dfrac{27.9\times10^6}{28.67'\times12} = 81.1\times10^3$ K-in

$(EI/l)_{beam} = \dfrac{46.2\times10^6}{25'\times12} = 154\times10^3$ K-in.

(both sides of the column)

$(EI/l_c)_{col:\ above} = \dfrac{27.9\times10^6}{8'\times12} = 291\times10^3$ K-in

$(EI/l_c)_{col:\ below} = 0$ (connected to footing)

At top of atrium column

$G_A = (\Sigma EI/l_c)_{col}/(\Sigma EI/l)_{beam}$

$= \dfrac{(81.1+291)\times10^3}{(154+154)\times10^3} = 1.21$

At bottom of atrium column

$G_B = \dfrac{(81.1+0)\times10^3}{(154+154)\times10^3} = 0.26$

From the alignment chart - sidesway inhibited — Figure R6.2.5

$K = 0.67$

$(Kl_u)/r = \dfrac{0.67\times28.67\times12}{6.93} = 33.3 < 40$ — Equation 6.2.5C

Hence, slenderness effect neglected

Considering bent in double curvature

$(Kl_u)/r \le 34+12(M_1/M_2) = 34+12\left(\dfrac{192}{248}\right) = 43.5$ (O.K) — Equation 6.2.5b

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Critical buckling load $(P_c) = \frac{\pi^2 (EI)_{eff}}{(Kl_u)^2}$ — Equation 6.6.4.4.2

$= \frac{\pi^2 (27.9 \times 10^6)}{(0.67 \times 28.67 \times 12)^2} = 5177K$

Factor relating to actual bending moment diagram to an equivalent bending moment diagram $(C_m) = 0.6 - 0.4 (M_1/M_2)$ — Equation 6.6.4.5.3a

$= 0.6 - 0.4 (+192/+248) = 0.29$

Moment magnification factor $(\delta) = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} \geq 1.0$ — 6.6.4.5.2

$= \frac{0.3}{1 - \frac{1380.64}{(0.75)(5177)}} = 0.47 < 1.0$ Use $\delta = 1.0$

Minimum $M_2 = P_u (0.6 + 0.03h)$ — Equation 6.6.4.5.4

$= 1380.64 (0.6 + 0.03 \times 24) = 1822$ K-in $= 151.9$ K-ft < 248 K-ft

Design factored moment $(M_c) = \delta M_2 = 248$ K-ft

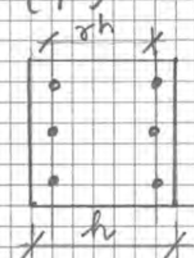
Design the column for $P_u = 1380.64$ K

$f'_c = 5$ ksi, $f_y = 60$ ksi, $M_u = 248$ K-ft

Use strength reduction factor $(\phi) = 0.65$ — Table 21.2.2

$h = 24''$; $\gamma h = 20''$

$\gamma = \frac{20}{24} = 0.83$



$K_n = \frac{P_n}{f'_c A_g} = \frac{1380.64}{0.65 \times 5 \times 24^2} = 0.74$

$R_n = \frac{M_n}{f'_c A_g h} = \frac{248 \times 12}{0.65 \times 5 \times 24^2 \times 24} = 0.07$

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From ACI interaction diagram

$\rho_g = 0.01$

$A_{st} = 0.01 \times 24^2 = 5.76 \text{ in}^2$

Provide 8#8 bars

Design for ties

Use #3 ties for #8 longitudinal bars — Section 25.7.2.2

Spacing of ties — Section 25.7.2.1

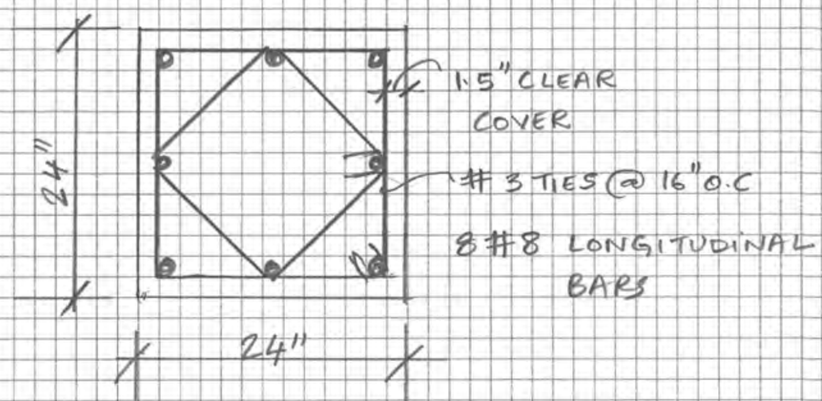
(a) clear spacing at least (4/3) size of aggregate

$\frac{4}{3} \times \frac{3}{4} = 1''$

(b) Center to center spacing not to exceed 16 times bar diameter of longitudinal bar, 48 times bar diameter of tie and smallest dimension of column

$\underbrace{16 \times 1 = 16''}_{\text{governs}}; \quad 48 \times \frac{3}{8} = 16''; \quad 24''$

Provide #3 ties @ 16" o.c.



G.3 PROBLEM G.3: PURE COMPRESSION

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Design the column located at the intersection of grid line 'B' and '3' at second floor shown in fig: 1.15.3 for axial compression $P_u = 337.7$ K (obtained from page 3/8 of problem (3) of chap: 1)

Use $f'_c = 5$ Ksi; $f_y = 60$ Ksi

$P_u = f'_c(A_g + (n-1)A_s)$

Modulus of elasticity of concrete $(E_c) = 57{,}000\sqrt{f'_c}$ — Equation 19.2.2.1.b

$= 57{,}000\sqrt{5000} = 4.03 \times 10^6$ psi

Modular ratio $(n) = E_s/E_c = 29\times10^6 / 4.03\times10^6$

$= 7.2$

$\phi P_n \geq P_u$ — Section 10.5.1.1

Strength reduction factor for column $(\phi) = 0.65$ — Table 21.2.2

Gross area of concrete $(A_g) = 16 \times 16 = 256$ in^2

$P_n \geq \dfrac{P_u}{\phi} \geq \dfrac{337.7}{0.65} \geq 519.5$ K

Hence, column should have a capacity of 520 K

$520 = 5(256 + (7.2-1)A_s)$

Solving for this equation will give a negative value of As indicating that concrete is adequate to support the compression

But, minimum longitudinal reinforcement is required $(A_s) = 0.01A_g = (0.01)(256)$ — Section 10.6.1.1

$= 2.56$ in^2 Provide 8#5 bars

There is no shear acting on the column

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Design for ties

Use #3 tie for #5 longitudinal bars — Section 25.7.2.2

Spacing for ties — Section 25.7.2.1

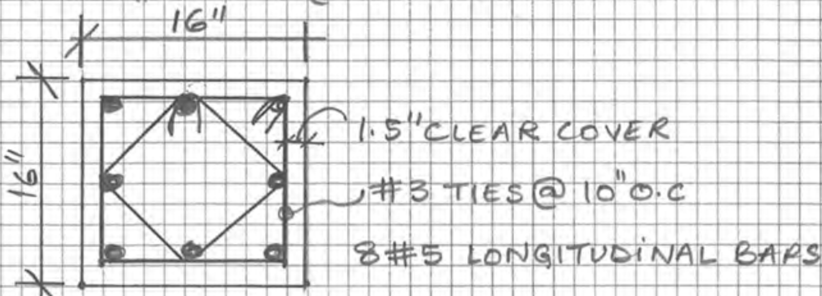
(a) clear spacing at least (4/3) size of aggregate

$\frac{4}{3} \times \frac{3}{4} = 1''$

(b) Center to center spacing not to exceed 16 times bar diameter of longitudinal bar; 48 times bar diameter of tie and smallest dimension of column

$16 \times \frac{5}{8} = \underbrace{10''}_{\text{governs}}$; $48 \times \frac{3}{8} = 18''$; $16''$

Hence use #3 ties @ 10" o.c.



Appendix H: Walls

H.1 PROBLEM H.1: SHEAR WALL

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	SUBJECT: SHEAR WALL		SHEET NO. 9/1	Inspection Investigation
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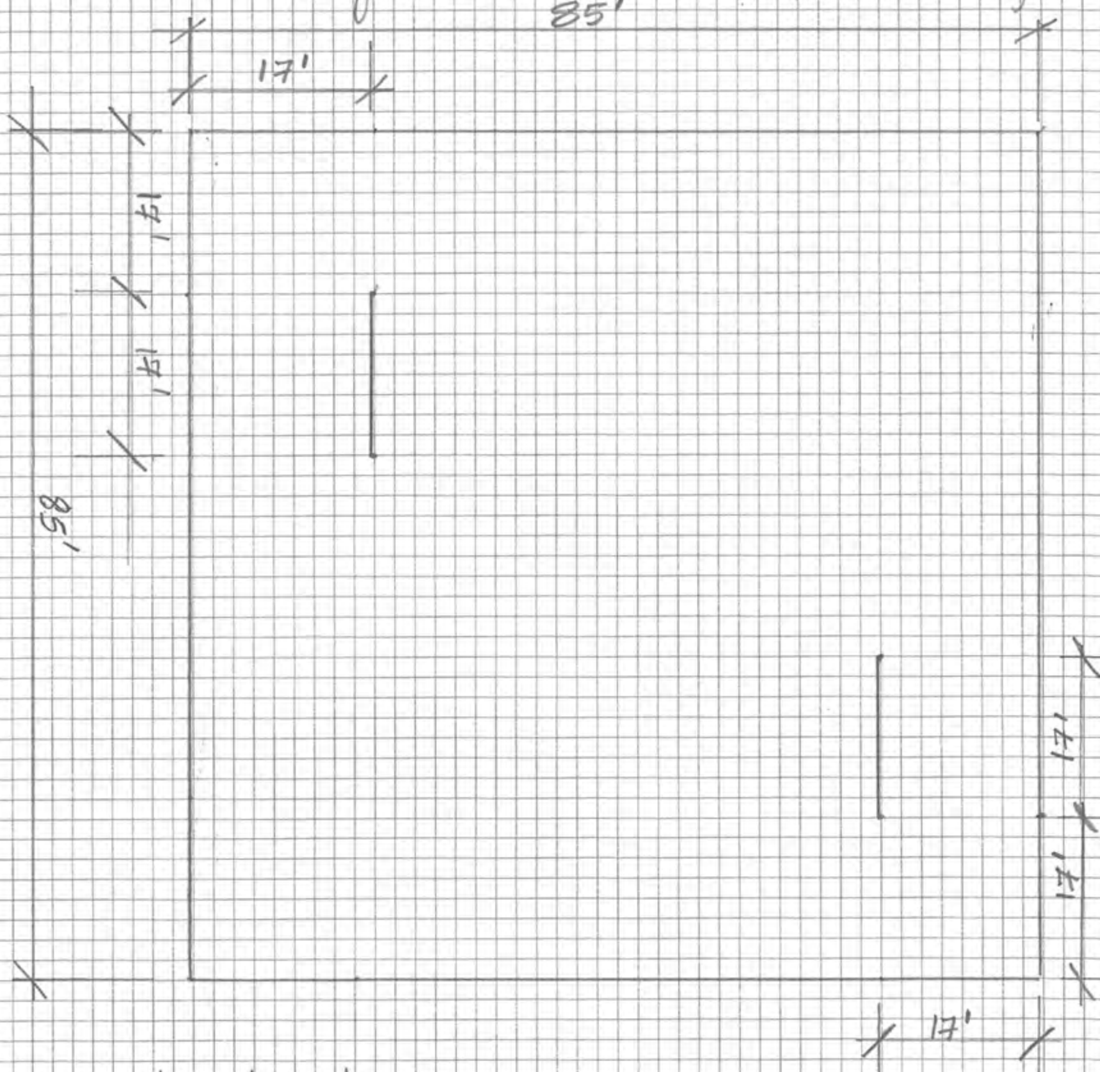
Plan of building type 'B' of chapter (1) is shown below. Since the building is symmetric about the two orthogonal axes, shear walls are shown only in one direction. $f'_c = 5$ ksi; $f_y = 60$ ksi



Wind Data:

Height of the building — 100'

Height of parapet — 3'

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Now, all the references provided till the end of wind analysis pertain to ASCE 7-10 — ASCE 7-10

Type of building – Residential

Risk Category – II

Location – Miami, FL

Basic wind speed – 175 mph — figure 26.5-1A

Exposure Category – C

Type of building – Enclosed

Internal pressure Coefficient (GC_{pi}) – ± 0.18 — Table 26.11-1

Velocity pressure (q_z) – $0.00256\, K_z K_{zt} K_d V^2$ — Equation 27.3.1

Directionality factor (K_d) – 0.85 — Table 26.6-1

Topographic Factor (K_{zt}) – 1.0 — Table 26.8.1

Velocity pressure (q_z) – $(0.00256)(K_z)(1)(0.85)(175)^2$

$= 66.64\, K_z$

Velocity pressure co-efficient (K_z) [shown in table attached] — Table 27.3-1

Wind pressure for Main Wind force Resisting System (MWFRS) (p) – $qGC_p - q_i G C_{pi}$ — Equation 27.4-1

$q = q_z$ for windward wall at height above ground (z)

$= q_h$ for leeward wall & side walls evaluated at height of the building (h)

$q_i = q_h$ for windwall walls, side walls & leeward walls.

G = gust effect factor — Section 26.9

C_p = External pressure coefficient — figure 27.4-1

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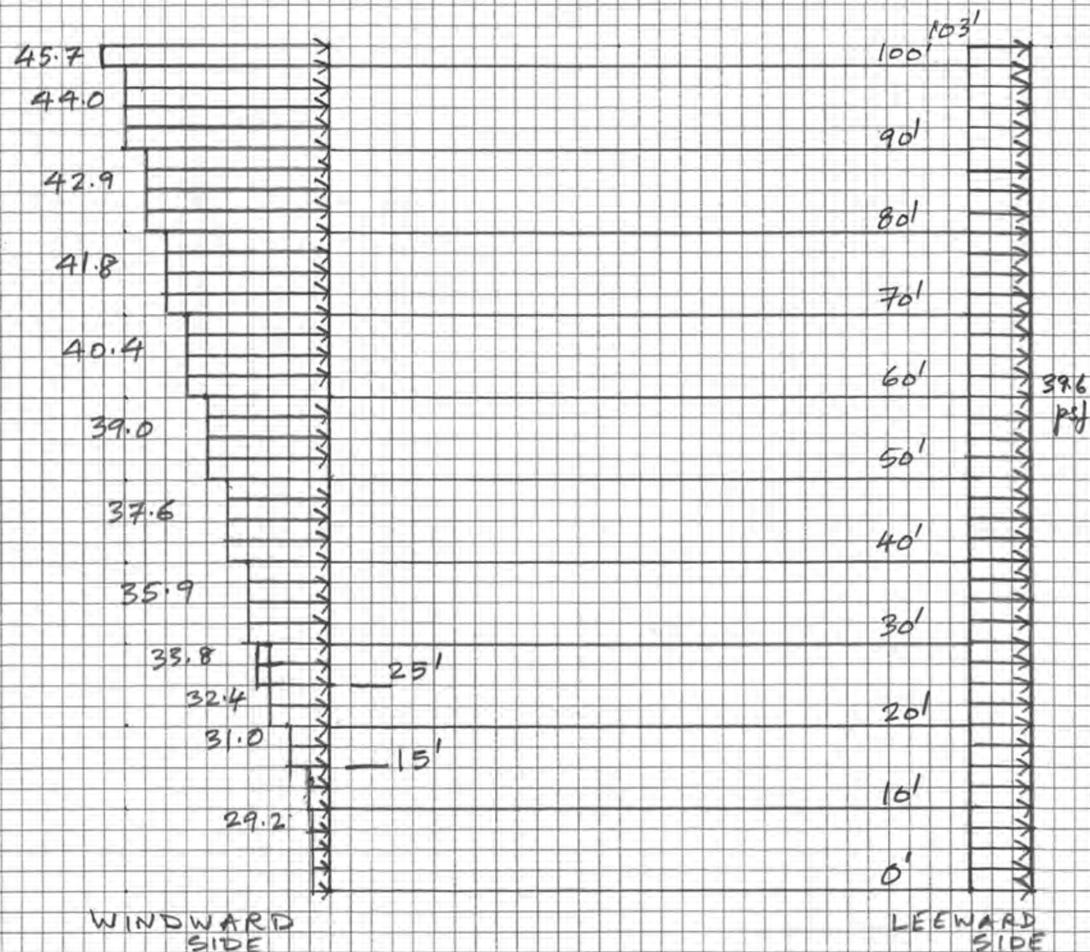
	ACI Code
Cp for windwall wall – 0.8	ASCE 7-10
Cp for leeward wall – −0.5; L/B = 1	
Cp for side wall – −0.7	
In order to simplify the calculations, it is assumed that the ten-storied building with concrete frames and shear walls is rigid	
Hence, gust effect factor (G) = 0.85	Section 26.9
Refer to the wind load calculations table	page 9/4



<u>**WIND LOAD CALCULATIONS - BUILDING "B"**</u>

Height	Velocity Pressure Exposure Coefficient	Velocity Pressure	Windward wall pressure			Leeward wall pressure			Side wall pressure			Allowable Wind Pressures			Total Wind Pressure	
(z)	(Kz)	(qz)	qGC_p	q_iGC_{pi}	(p) = $qGC_p - q_iGC_{pi}$	qGC_p	q_iGC_{pi}	(p) = $qGC_p - q_iGC_{pi}$	qGC_p	q_iGC_{pi}	(p) = $qGC_p - q_iGC_{pi}$	Windward Walls	Leeward Walls	Side Walls	X - Direction	Y - Direction
0 - 15	0.57	38.0	25.8	-11.9	37.7	-28.1	11.9	-39.9	-39.27	11.9	-51.2	29.2	-30.9	-39.6	60.1	-39.6
20	0.62	41.3	28.1	-11.9	40.0	-28.1	11.9	-39.9	-39.27	11.9	-51.2	31.0	-30.9	-39.6	61.9	-39.6
25	0.66	44.0	29.9	-11.9	41.8	-28.1	11.9	-39.9	-39.27	11.9	-51.2	32.4	-30.9	-39.6	63.3	-39.6
30	0.70	46.6	31.7	-11.9	43.6	-28.1	11.9	-39.9	-39.27	11.9	-51.2	33.8	-30.9	-39.6	64.7	-39.6
40	0.76	50.6	34.4	-11.9	46.3	-28.1	11.9	-39.9	-39.27	11.9	-51.2	35.9	-30.9	-39.6	66.8	-39.6
50	0.81	54.0	36.7	-11.9	48.6	-28.1	11.9	-39.9	-39.27	11.9	-51.2	37.6	-30.9	-39.6	68.6	-39.6
60	0.85	56.6	38.5	-11.9	50.4	-28.1	11.9	-39.9	-39.27	11.9	-51.2	39.0	-30.9	-39.6	70.0	-39.6
70	0.89	59.3	40.3	-11.9	52.2	-28.1	11.9	-39.9	-39.27	11.9	-51.2	40.4	-30.9	-39.6	71.4	-39.6
80	0.93	62.0	42.1	-11.9	54.0	-28.1	11.9	-39.9	-39.27	11.9	-51.2	41.8	-30.9	-39.6	72.8	-39.6
90	0.96	64.0	43.5	-11.9	55.4	-28.1	11.9	-39.9	-39.27	11.9	-51.2	42.9	-30.9	-39.6	73.8	-39.6
100	0.99	66.0	44.9	-11.9	56.7	-28.1	11.9	-39.9	-39.27	11.9	-51.2	44.0	-30.9	-39.6	74.9	-39.6
103	1.04	69.3	47.1	-11.9	59.0	-28.1	11.9	-39.9	-39.27	11.9	-51.2	45.7	-30.9	-39.6	76.6	-39.6

The wind pressure calculations are based upon ultimate wind speed. In order to convert it to allowable speed , Vasd = sqrt (0.6) times Vult

X-direction wind pressures is the summation of the windward and leeward wall pressures

Y-direction wind pressures are the side wall pressures

All wind pressures in psf units

All heights in foot unit

9/4

9/5

Height (z)	X - Direction Pressures
0 - 15	60.1
20	61.9
25	63.3
30	64.7
40	66.8
50	68.6
60	70.0
70	71.4
80	72.8
90	73.8
100	74.9
103	76.6

SUMMATION OF WIND PRESSURES

Segment	Height of Segment	Average Pressure	Pressure at height		Cummulative Pressure (psf)
			(ft)	(psf)	
100' - 103'	3	75.8	100'	227.3	227.3
90' - 100'	10	74.4	90'	743.5	970.8
80' - 90'	10	73.3	80'	733.0	1703.8
70' - 80'	10	72.1	70'	720.7	2424.5
60' - 70'	10	70.7	60'	706.7	3131.2
50' - 60'	10	69.3	50'	692.6	3823.8
40' - 50'	10	67.7	40'	676.8	4500.6
30' - 40'	10	65.8	30'	657.5	5158.1
25' - 30'	5	64.0	25'	320.0	5478.1
20' - 25'	5	62.6	20'	313.0	5791.1
15' - 20'	5	61.0	15'	305.1	6096.1
0' - 15'	15	60.1	0'	901.5	6997.6

Hence, wind pressure at the base of the building is 6997.6 lb/ft = 7 K/ft

Floor	Level	Wind Pressure (plf)	Wind Force per Wall (K)
Roof	100'	599.0	25.5
10th	90'	738.3	31.4
9th	80'	726.8	30.9
8th	70'	713.7	30.3
7th	60'	699.6	29.7
6th	50'	684.7	29.1
5th	40'	667.2	28.4
4th	30'	648.7	27.6
3rd	20'	632.9	26.9
2nd	10'	601.0	25.5

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Since the shear walls are symmetrically placed, the center of rigidity of the system will coincide with the center of load. Hence, there will not be any torsion	
From table on page 9/5, total shear force at base = 6997.6 lb/ft = 7.0 K/ft	
Hence shear force on each $= \frac{85' \times 7\,K/ft}{2} = 297.5^K$	
A wind load factor of 1.0 is used with no other load combination for shear	
Now, references for ACI 318-14	
Check for in-plane shear	
Thickness of wall (h) = 12″	
depth of wall (d) = $0.8\,l_w$ $= 0.8 \times 17' \times 12' = 163.2''$	Section 11.5.4.2
Shear capacity of wall $(V_c) = 2\sqrt{f'_c}\,h\,d$ $= \frac{(2)\sqrt{5000}\,(12)(163.2)}{1000} = 276.3\,K$	Section 11.5.4.5
$0.5\,\phi V_c = (0.5)(0.75)(276.3) = 103.6 < 297.5^K$	Section 11.6.2(b)
Hence shear reinforcement required	
$V_s = \frac{V_u}{\phi} - V_c = \frac{297.5}{0.75} - 276.3 = 120.4\,K$	Equation 11.5.4.4
$V_s = \frac{A_v f_{yt} d}{s}$	Equation 11.5.4.8
$s = \frac{A_v f_{yt} d}{V_s} = \frac{2 \times 0.1 \times 60 \times 163.2}{120.4} = 16.3''$	2 layers of #3 @ 16″ o.c.

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Maximum spacing is least of 3h and 18" — Section 11.7.2.1

Provided 2 layers of #3 bars @ 16" o.c.

Vertical steel

Minimum $\rho_t = 0.0025$

$A_{st} = 0.0025 \times 12'' \times 12'' = 0.36\ in^2/ft$

Provide 2 layers of #4 bars @ 12" o.c.

Approximate Estimate of gravity loads

Roof live load − 30 psf

Floor live load − 40 psf (residential)

Tributary area for wall load per floor

$- (17' \times 17') - \underbrace{(17' \times 1')}_{\text{area of wall}}$

Dead load

Roof slab weight $- \dfrac{8''}{12''} \times \dfrac{272 \times 150}{1000} - 27.3\ K$

Roof superimposed dead load $- \dfrac{272 \times 30}{1000} - 8.16\ K$

Slab wt. for 9 typical floors $- \dfrac{9''}{12''} \times \dfrac{272 \times 150 \times 9}{1000}$

$- 275.4\ K$

Typical floor superimposed dead load

$- \dfrac{272 \times 25 \times 9}{1000} - 61.2\ K$

Self weight of wall $- \dfrac{12''}{12} \times \dfrac{10' \times 10 \times 150 \times 17'}{1000} - 255\ K$

Total dead load − 627.1 K

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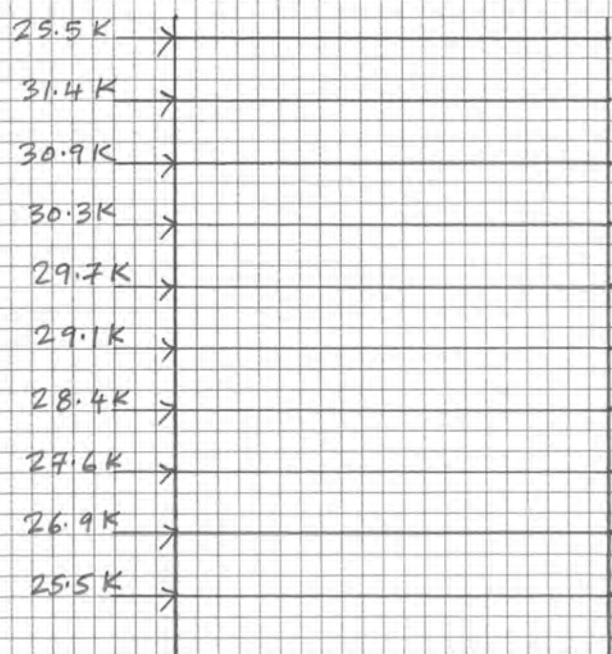
Live load

Roof $- \frac{272 \times 30}{1000} - 8.16$ K

Floors $- \frac{272 \times 4 \times 9}{1000} - 97.92$ K.

Refer to page 9/5 for the calculation of wind force at each floor



Moment due to wind at the base

$= (25.5 \times 100) + (31.4 \times 90) + (30.9 \times 80) +$
$(30.3 \times 70) + (29.7 \times 60) + (29.1 \times 50) +$
$(28.4 \times 40) + (27.6 \times 30) + (26.9 \times 20) +$
$(25.5 \times 10) = 15963$ k-ft

Moment resisted by each shear wall

$= 15963/2 = 7982$ k-ft

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Design the wall for axial load and in-plane moment

In order to demonstrate the design of this shear wall, only the load combination 5.3.1.d of the code is considered

$1.2D + 1.0W + 1.0L + 0.5L_r$

$P_u = (1.2)(627.1) + (1.0)(97.92) + (0.5)(8.16)$

$= 854.5$ K (Dead & live loads)

$M_u = (1.0)(7982) = 7982$ K-ft (Wind Load).

Since the moment due to wind is high, the entire wall is considered in the design to resist the moment. If the moments are smaller, only a portion of the wall at the end called "boundry elements" are used to resist the moment

From the ACI interaction charts,

$K_n = P_n/f'_c A_g$

$$= \frac{854.5}{(0.9)(5)(12)(17' \times 12)} = 0.08$$

$R_n = M_n/f'_c A_g h$

$$= \frac{7982 \times 12}{(0.9)(5)(12)(17 \times 12)(17 \times 12)} = 0.04$$

From interaction diagram for $f'_c = 5$ ksi

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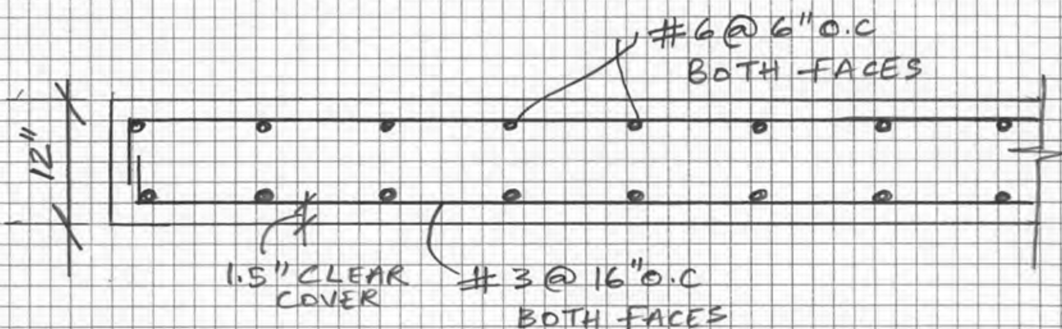
$f_y = 60$ ksi and $\gamma = 0.9$

$\rho = 0.01$

$A_{st} = 0.01 \times 12 \times 12 = 1.44\ in^2/ft$

Provide #6 bars @ 6" o.c at each

ρ_t calculated before (#4 @ 12" o.c.) is not required



H.2 PROBLEM H.2: RETAINING WALL

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ACI Code

Design a 10' high cantilever retaining wall

Weight of soil (γ) – 120 pcf

Angle of internal friction (ϕ) – 34°

Surcharge (q_{sur}) – 500 psf

Rankine's active pressure co-efficient (K_a)

$$= \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 34}{1+\sin 34} - 0.31$$

Lateral pressure on the wall due to soil retained (p_{soil}) $= \frac{1}{2} K_a \gamma h^2 - \frac{1}{2}(0.31)(120)(10^2)$

$- 1860$ lb/ft

Lateral pressure on the wall due to surcharge (p_{sur}) $- \frac{1}{2} K_a q_{sur} h - \frac{1}{2}(0.31)(500)(10') - 755$ lb/ft

Factored moment acting on the wall due to soil and surcharge (M_u) $-(1.6)(1860\frac{(10')}{3} + (1.6)(773)\frac{(10')}{2}$

$- 16120$ lb-ft

Assume a 10" thick wall

Effective depth (d) $= 10'' - 3'' - 0.5'' = 6.5''$

(Assume 3" clear cover and #8 bar)

$$R_n = \frac{M_n}{bd^2} = \frac{16120 \times 12}{0.9 \times 12 \times 6.5^2} = 424 \text{ psi}$$

$$\rho = 0.85\frac{f'_c}{f_y}\left[1-\sqrt{1-\frac{2R_n}{0.85 f'_c}}\right]$$

$$= 0.85\left(\frac{5}{60}\right)\left[1-\sqrt{1-\frac{(2)(424)}{(0.85)(5000)}}\right] = 0.0075$$

$A_{st} = 0.0075 \times 12 \times 6.5 = 0.58$ in²/ft

#5 @ 6" o.c

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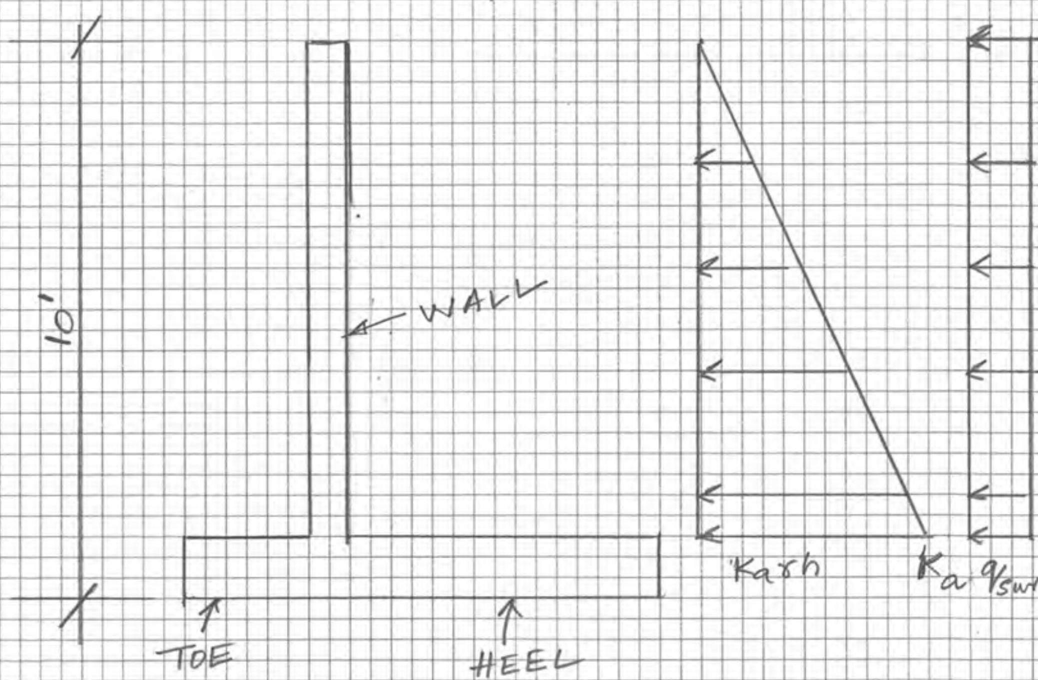
As provided = 0.62 in^2/ft

Min: steel = $\dfrac{0.0018 \times 60{,}000}{f_y} A_g$ — Table 7.6.1.1

$= \left(\dfrac{0.0018 \times 60{,}000}{60{,}000}\right)(12 \times 10)$

$= 0.22$ in^2/ft < 0.62 in^2/ft — OK



Design heel width for sliding

Neglect soil resistance in front of the wall

Factor of safety (FS) = $\dfrac{F_{res}}{F_{slid}} = 1.5$

F_{res} — Resisting force

F_{slid} — Sliding force

Co-efficient of friction = Tan (ϕ) = Tan (34) = 0.675

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F_{res} = Vertical force × Co-efficient of friction

Assume a heel width of "w"

Assume a heel thickness of 15"

Vertical force

Weight of soil $= (120)(10')(w) = 1200w$ lb/ft

Weight of stem $= \left(\frac{10''}{12''}\right)(10')(150) = 1250$ lb/ft

Weight of foundation $= \left(\frac{15''}{12''}\right)\left(w + 2' + \frac{10''}{12}\right)(150)$

$= (188w + 531)$ lb/ft

Total vertical force $= (1388w + 1781)$ lb/ft

$F_{res} = (1388w + 1781)(0.675)$

$= (937w + 1202)$ lb/ft

Sliding Force

Due to sand $= \frac{1}{2}(0.31)(120)(10)^2 = 1860$ lb/ft

Due to surcharge $= (0.31)(500)(10) = 1550$ lb/ft

Total sliding force $= 3410$ lb/ft

Hence, $\frac{937w + 1202}{3410} = 1.5$

Solving, $w = 4.18'$

Provide a heel width of 5' and toe width of 2'

Overturning

Overturning is caused by:

(a) Soil $= (1860)(10'/3) = 6200$ lb-ft/ft

(b) Surcharge $= (1550)(10'/2) = 7750$ lb-ft/ft

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Total overturning moment (M_{over}) – 13,950 lb-ft/ft

Overturning is resisted by:

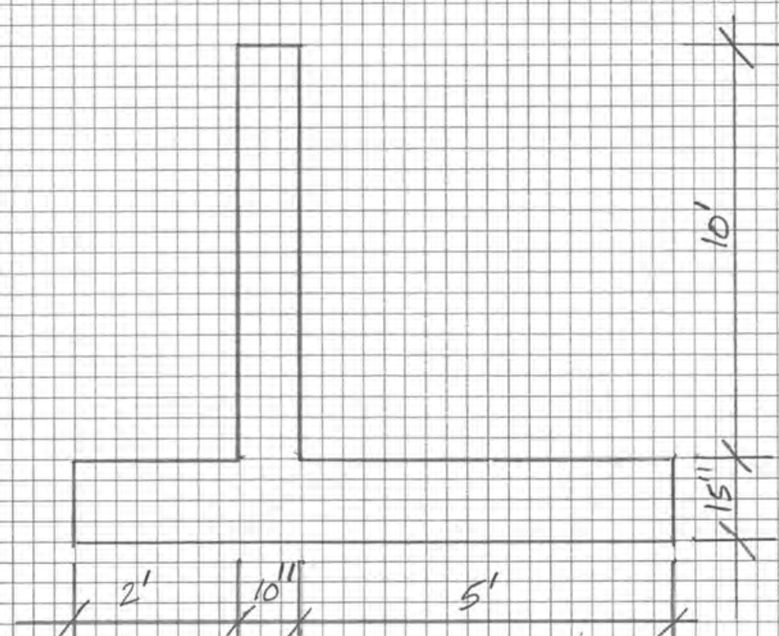
(a) Sand – $(120 \times 10 \times 5)(2' + 10'' + 5/2)$ = 32,000 lb-ft/ft

(b) Wall – $\left(\frac{10''}{12''} \times 10' \times 150\right)(2' + 5'')$ = 3,021 lb-ft/ft

(c) Footing – $\left(5' \times \frac{15''}{12''} \times 150\right)\left(2' + 10'' + \frac{5'}{2}\right)$ = 5,000 lb-ft/ft

Total resisting moment M_{res} = 40,021 lb-ft/ft

Factor of safety $= \frac{M_{res}}{M_{over}} = \frac{40,021}{13,950} = 2.87 > 2.0$



Bearing

Downward Loads

Weight of soil – (5')(10')(120) – 6000 lb/ft

Weight of stem – (10"/12")(10')(150) – 1250 lb/ft

Weight of foundation – $\left(2' + 5' + \frac{10''}{12}\right)\left(\frac{15''}{12}\right)(150)$ – 1469 lb/ft

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Total weight = 8719 lb/ft

Moment

$M_{overturning}$ = +13,950 lb-ft

$M_{sand} = -(5' \times 10' \times 120')\left(\frac{2'+10''+5'}{2} - \frac{5'}{2}\right) = -8500$ lb-ft

$M_{stem} = +(1250)\left(\frac{2'+10''+5'}{2} - \left(2' + \frac{10''}{2}\right)\right) = +1875$ lb-ft

$M_{net} = 13,950 - 8500 + 1875 = 7,325$ lb-ft/ft

Eccentricity $(e) = \frac{7325}{8719} = 0.84'$

$L/6 = \frac{1}{6}(2'+10''+5') = 1.31' > 0.84'$

Since eccentricity is less than L/6, there will be no tensile stress because M/S will not exceed P/A

Maximum stress on soil $= \frac{P}{A} + \frac{M}{S}$

$= \frac{8719}{1'\underbrace{(2'+5'+10'')}_{7.83'}} + \frac{7325}{1 \times \frac{7.83^2}{6}} = 1113.5 + 717.9$

$= 1831.4$ psf

< 2000 psf

(Safe bearing capacity soil – 2000 psf)

<u>Design of heel</u>

Load

Weight of heel – $\frac{15''}{12''} \times 150$ – 187.5 lb/ft

Weight of soil – $10' \times 120$ – 1200 lb/ft

Total dead load – 1387.5 lb/ft

Live load (surcharge) – 500 lb/ft

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Factored Load $= (1.2)(1387.5) + (1.6)(500)$
$= 2465$ lb/ft

Maximum Moment $= 2465 \times \frac{5^2}{2} = 30812.5$ lb-ft

$R_n = \frac{M_n}{bd^2} = \frac{30812.5 \times 12}{(0.9)(12)(11.5^2)} = 259$ psi

(Effective depth $(d) = 15'' - 3'' - 0.5'' = 11.5''$)
clear cover; Assume #8 bars

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

$$= \frac{0.85 \times 5}{60}\left[1 - \sqrt{1 - \frac{(2)(259)}{(0.85)(5000)}}\right] = 0.004$$

$A_{st} = (0.004)(12)(11.5) = 0.55$ in²/ft

Provide #5 @ 6" o.c.

A_{st} provided $= 0.62$ in²

Minimum Steel $= \left(\frac{0.0018 \times 60{,}000}{f_y}\right) A_g$ — Table 7.6.1.1

$A_{st} = \frac{0.0018 \times 60{,}000}{60{,}000} \times 12 \times 15 = 0.32$ in²/ft < 0.62 in²/ft

Shear $(V_u) = 2465 \times 5 = 12325$ lbs at the face of the wall

$\phi V_c = (\phi)(2)\sqrt{f'_c}\, b_w d$ — Section 11.5.4.5

$= (0.75)(2)\sqrt{5000}\,(12)(11.5)$

$= 14{,}637$ lbs $> 12{,}325$ lbs

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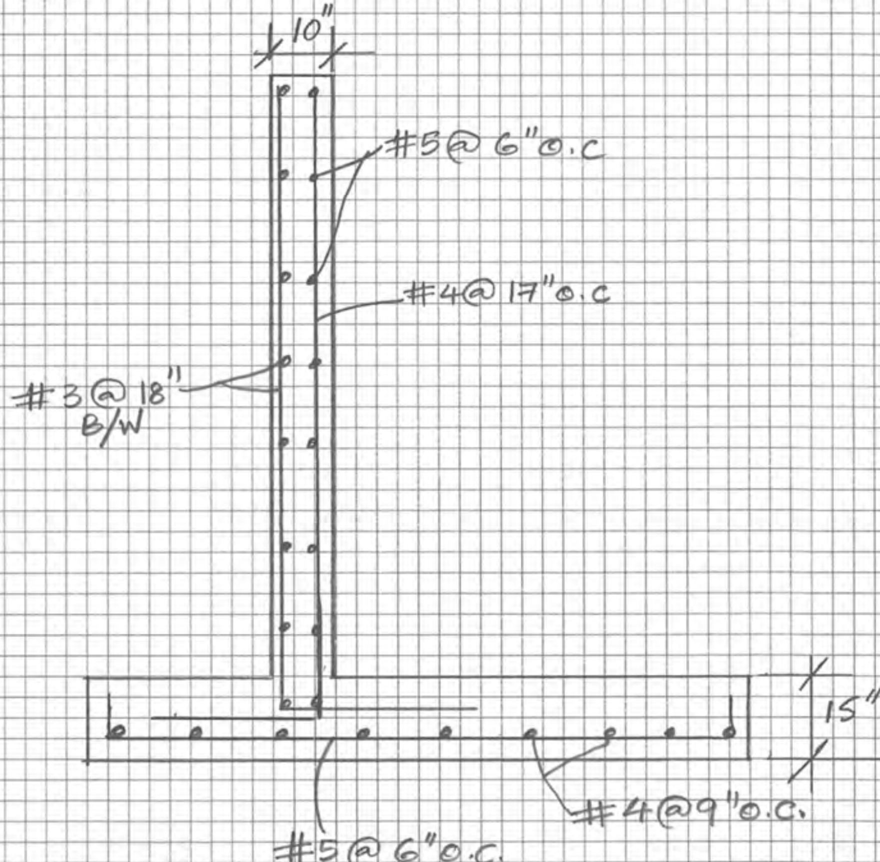
Transverse steel (A_{st}) $= 0.0018\,bd$ — Table 24.4.3.2

$= (0.0018)(12)(11.5)$

$= 0.25$ in^2/ft

Provide #4@9" o.c $< 18"$ — Section 7.7.6.2.1



Appendix I: Footings

I.1 PROBLEM I.1: ISOLATED FOOTING

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The footings of building type 'A' are shown in Fig: 1.15.4. Design the four typical footings on grid lines 'A', 'F', '1' and '6' labelled A-1, A-6, F-1 and F-6. The load transferred by these 16"x16" columns to the footing:

Service load : 135.2 K

Factored Load : 167.4 K

The safe bearing capacity of soil recommended by the Geotechnical Engineer is 4000 psf

Area of the footing required $= \dfrac{\text{Service Load}}{\text{Safe bearing capacity of soil}}$

$= \dfrac{135.2}{4} = 33.8$ sft

Select a footing of size 6'x6'x1.5'

Weight of the footing = 6'x6'x1.5'x150pcf = 8100lbs

Total weight on the soil = 135.2 + 8.1 = 143.3K

Check safe bearing capacity $= \dfrac{143.3}{6' \times 6'} = 3.98$ Ksf < 4.0 ksf.

Net upward pressure of soil

Factored wt: of footing = 1.2 x 8.1 = 9.72 K

Total factored load = 167.4 + 9.72 = 177.12K

Factored upward pressure of soil $= \dfrac{177.12}{6' \times 6'} = 4.92$ ksf

Clear cover to reinforcement = 3" — ACI Code: Table 20.6.1.3.1

Assume #5 bars

Effective depth $= 18" - 3" - \dfrac{0.625"}{2} = 14.7"$

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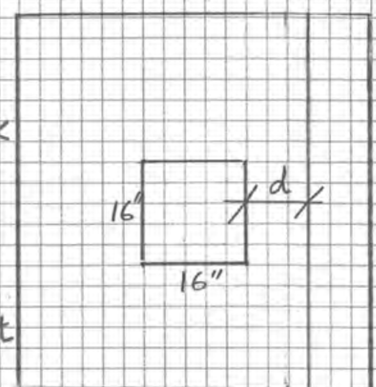
Maximum shear at a distance 'd' from the face of the support $= 4.92 \times 6' \times 13.3''/12 = 32.7$ K

Shear capacity of concrete — Equation 22.5.5.1

$V_c = 2\sqrt{f'_c}\, b_w d$

$= \dfrac{2\sqrt{5000}\,(6' \times 12')(14.7)}{1000} = 149.7$K



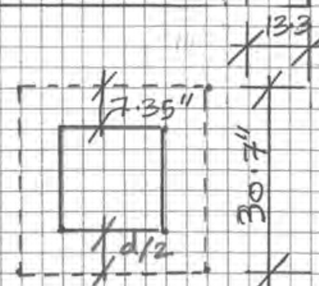
$0.5\phi V_c = 0.5(0.75)(149.7)$

$= 56.1$ K $> V_u$

Hence, no shear reinforcement — Section 9.6.3.1

Two Way Shear

$\dfrac{d}{2} = \dfrac{14.7}{2} = 7.35''$



Shear capacity of concrete for two-way shear is least of: — Table 22.6.5.2

(a) $4\sqrt{f'_c} = 4\sqrt{5000} = 282.8$ psi

(b) $(2 + 4/\beta)\sqrt{f'_c} = 6\sqrt{5000} = 424.3$ psi

(β for square column $= 16''/16'' = 1$)

(c) $(2 + \alpha_s d/b_o)\sqrt{f'_c} = \left[2 + \dfrac{(40)(14.7)}{122.8}\right]\sqrt{5000}$

$= 409.3$ psi

$\alpha_s = 40$, because the column is located at the center of the footing and treated as an interior load on the footing — Section 22.6.5.3

Shear perimeter $(b_o) = 4 \times 30.7 = 122.8''$

$V_c = 282.8$ psi governs

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Hence $\phi V_c = \dfrac{(0.75)(282.8)(122.8)(14.7)}{1000} = 382.9\,K$

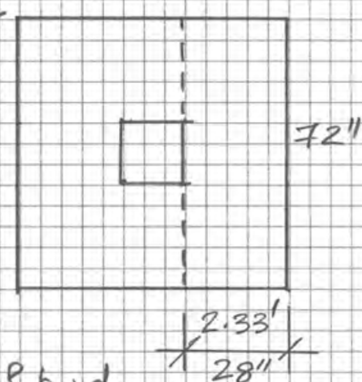
$V_u = \left[16' \times 16' - \dfrac{30.7^2}{144}\right] 4.92 = 149.9^K$

$\phi V_c > V_u$

Critical section for M_u for footing is at the face of the column $- \dfrac{4.92 \times 2.33^2}{2} = 13.36$ K-ft/ft — Table 13.2.7.1

$R_n = \dfrac{M_u}{\phi b d^2} = \dfrac{13.36 \times 12000}{(0.9)(12)(14.7)^2}$

$= 68.7$ psi



$\rho = \dfrac{0.85 f_c'}{f_y}\left[1 - \sqrt{1 - \dfrac{2R_n}{0.85 f_c'}}\right]$

$= \dfrac{0.85 \times 5000}{60{,}000}\left[1 - \sqrt{1 - \dfrac{2 \times 68.7}{0.85 \times 5000}}\right]$

$= 0.0012$

$A_{s,min} = 0.0018\,A_g > \rho b_w d$ — Table 7.6.1.1

$= 0.0018 \times 12 \times 18 = 0.39$ in^2/ft

Provide #5 @ 9" o.c. both ways

Bearing strength (B_n) is lesser of: — Table 22.8.3.2

(a) $\sqrt{A_2/A_1}\,(0.85 f_c' A_1)$ (b) $2(0.85) f_c' A_1$

$A_1 = 16'' \times 16'' = 256$ in^2; $A_2 = 72'' \times 72'' = 5184$ in^2

(a) $\sqrt{\dfrac{5184}{256}}\,(0.85 \times 5000 \times 256) = 4896000$ lbs $= 4896$ K

(b) $(2)(0.85)(5000)(256) = 2176000 = 2176$ K.

$\phi B_n = (0.65)(2176) = 1414\,K > \underset{P_u}{167.4\,K}$

I.2 PROBLEM I.2: WALL FOOTING EXPANSION

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An existing load bearing 8" CMU wall was supported on a 24"x 18" wall footing. The existing load on the footing is concentric

The load on the footing was:

Dead load: 2500 lb/ft (inclusive of footing weight)

live load : 1500 lb/ft

Due to an addition on the building, load was added to the wall so that the total additional load on the footing is:

Dead load : 700 lb/ft

live load : 300 lb/ft

Safe bearing capacity of soil – 2000 psf

A 6" addition was proposed on the exterior edge of the footing to avoid excavation inside the house. Assuming the centroid of load of the footing falls within the middle third of the footing (kern), design the footing for shear friction at the surface of addition of the footing.

Total dead load – 2500 + 700 = 3200 lb/ft

Total live load – 1500 + 300 = 1800 lb/ft

Total service load – 3200 + 1800 = 5000 lb/ft

width of footing required = $\frac{5000}{2000}$ = 2.5'

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Width of footing provided = 2.5'

Factored load = (1.2)(3200) + (1.6)(1800) = 6720 lb/ft

Net upward pressure of soil = $\frac{6700}{2.5}$ = 2680 lb/ft

Shear at the connection of old & new concrete
= (2680)(0.5) = 1344 lb/ft

$V_n = A_{vf} f_y$ — Equation 22.9.4.2

$A_{vf} = \frac{V_u}{\phi f_y} = \frac{1344/12}{(0.75)(60{,}000)}$ = 0.0025 in²/in
= 0.0298 in²/ft

Effective depth = 18" − 3" − 0.625"/2 = 14.7"

A_c = 12" × 14.7" = 176.4 in²

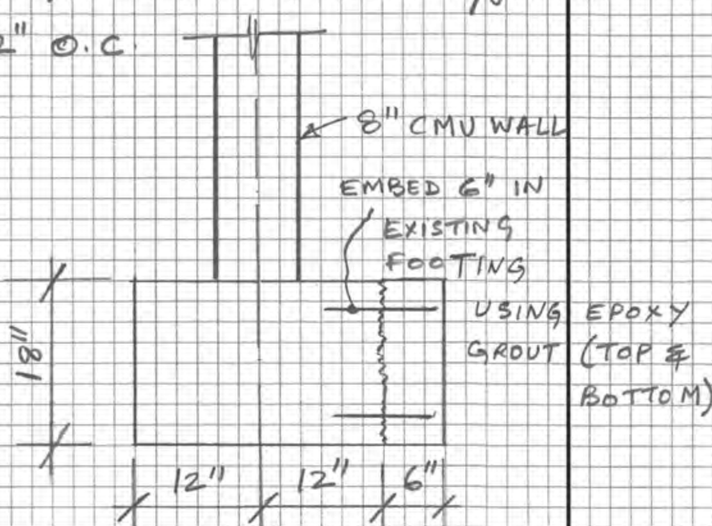
Maximum V_n across the shear plane is lesser of $0.2 f'_c A_c$ and $800 A_c$ — Table 22.9.4.4

0.2 (3000)(176.4) = 105840 lbs

(800)(176.4) = 141120 lbs

ϕV_c = (0.75)(105840) = 79110 > 1344 lb/ft

Use #3 bars @ 12" O.C
(TOP & BOTTOM)



I.3 PROBLEM I.3: COMBINED FOOTING

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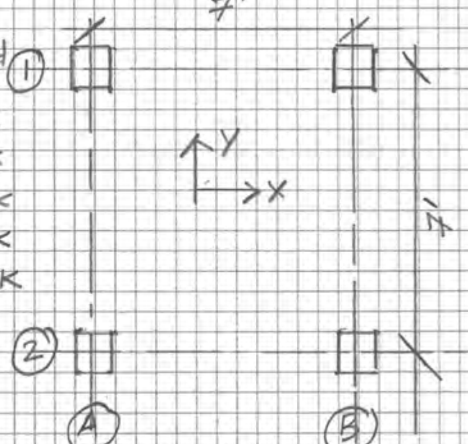
Design a combined footing for the four columns shown in the figure. All columns are 12"×12". The safe bearing capacity of soil is 3 Ksf

Use $f_c' = 5$ ksi, $f_y = 60$ ksi

Column	Service Load	Factored Load
A-1	89 K	120 K
B-1	72 K	97 K
A-2	92 K	120 K
B-2	78 K	105 K
Total	331 K	442 K



Size of footing required $= \frac{331}{3} = 110$ sft

Select a footing size of $11' \times 11' \times 40''$

Weight of footing $= 11' \times 11' \times \frac{40''}{12''} \times 150 = 60{,}500$ lbs

Total load $= 331 + 60.5 = 391.5$ K

Upward pressure of soil $= \frac{391.5}{11' \times 11'} = 3.24$ Ksf > 3.0 Ksf Not OK

Increase footing size to $12' \times 12' \times 40''$

Weight of footing $= 12' \times 12' \times \frac{40''}{12} \times 150 = 72{,}000$ lbs

Total load $= 331 + 72 = 403$ K

Upward pressure of soil $= \frac{403}{12' \times 12'} = 2.8$ Ksf < 3.0 Ksf OK

Since the four loads are different, an equilibrium in the size of footing needs to be established.

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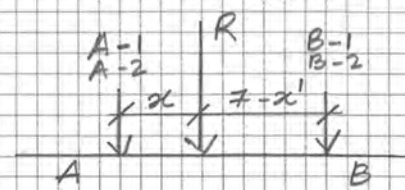
In X-direction



$A\text{-}1 + A\text{-}2 = 89 + 92 = 181\,K$

$B\text{-}1 + B\text{-}2 = 72 + 78 = 150\,K$

Taking moment about 'R'

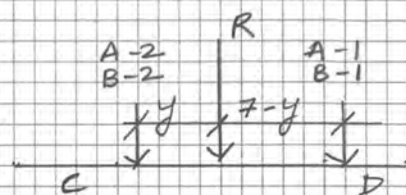
$(181)(x) = 150(7-x)$

Solving for 'x', $x = 3.17'$, $7-x = 3.83'$

Distance 'A' $= \frac{12'}{2} - 3.17' = 2.83'$

Distance 'B' $= \frac{12'}{2} - 3.83' = 2.17'$

In Y-direction



$A\text{-}2 + B\text{-}2 = 92 + 78 = 170\,K$

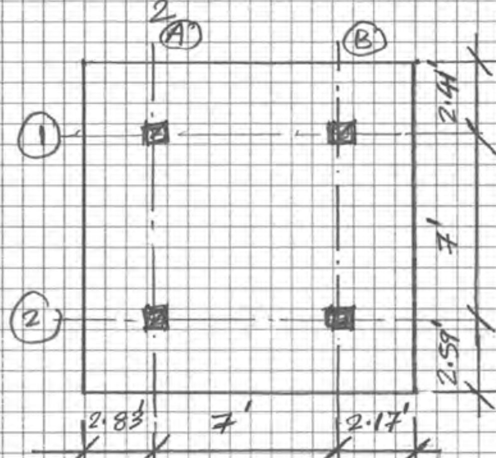
$A\text{-}1 + B\text{-}1 = 89 + 72 = 161\,K$

$(170)\,y = (7-y)(161)$

Solving for 'y', $y = 3.41'$, $7-y = 3.59'$

Distance 'C' $= \frac{12'}{2} - 3.41' = 2.59'$

Distance 'D' $= \frac{12'}{2} - 3.59' = 2.41'$



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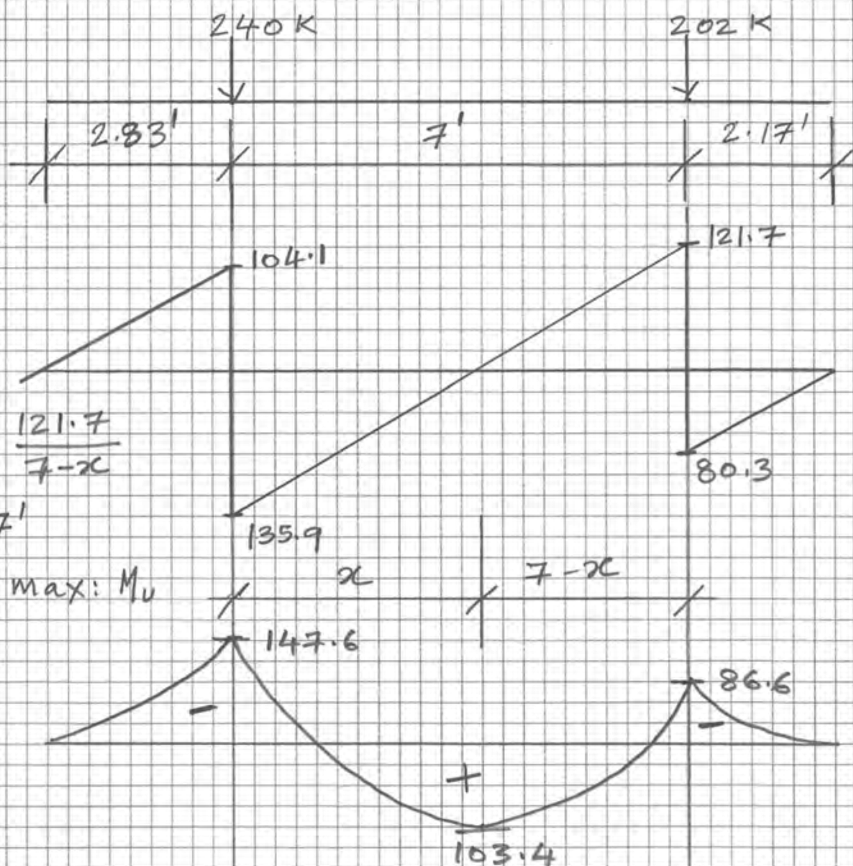
Factored net upward pressure of soil $= \dfrac{442}{12' \times 12'}$

$= 3.07$ psf $= 3.07 \times 12' = 36.8$ k/ft

X-direction – Factored loads



$\dfrac{135.9}{x} = \dfrac{121.7}{7-x}$

$x = 3.7'$

Point of max: M_u

Moment at left column $= -36.8 \times \dfrac{2.83^2}{2} = -147.6$ k-ft

Moment at right column $= -36.8 \times \dfrac{2.17^2}{2} = -86.6$ k-ft

Moment at span $= -\dfrac{(36.8)(2.83+3.7)^2}{2} + (240)(3.7) = +103.4$ k-ft

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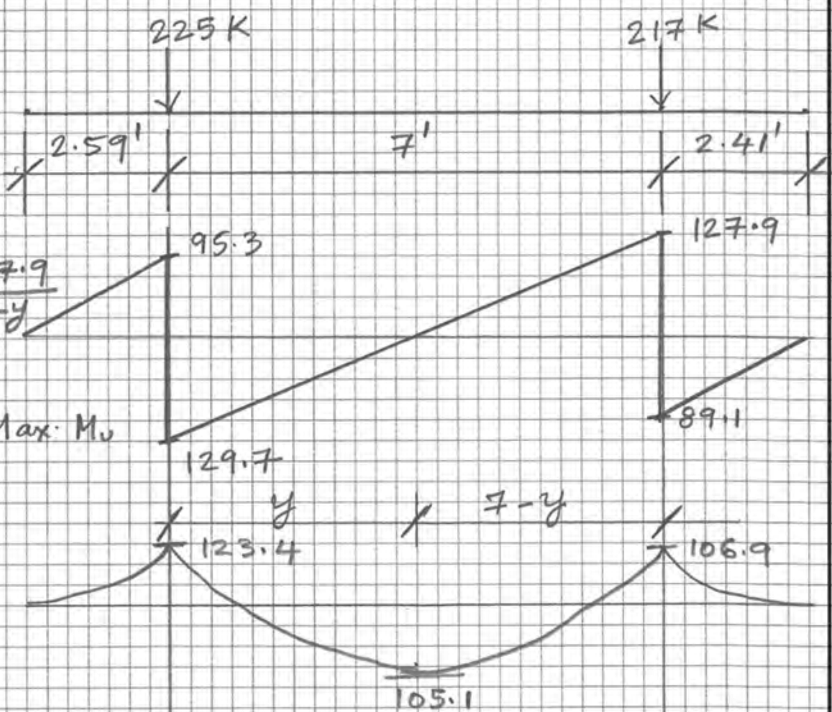
Y-direction – Factored load



$\frac{129.7}{y} = \frac{127.9}{7-y}$

$y = 3.52'$

Point of Max M_u

Moment at left column $= -36.8 \times \frac{2.59^2}{2} = -123.4$ K-ft

Moment at right column $= -36.8 \times \frac{2.41^2}{2} = -106.9$ K-ft

Moment at span $= -\frac{(36.8)(2.59+3.52)^2}{2} + (225)(3.52) = +105.1$ K-ft

The maximum shear force occuring at grid line (Y-direction) $= 127.9$ K/ft

Maximum one-way shear occurs at 'd' from the face of the support — Section 9.4.3.2

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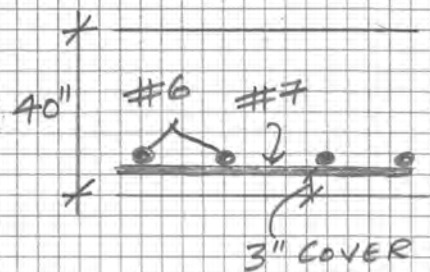
Effective depth (d) $= 40'' - 3'' - (0.875 + \frac{0.75}{2}) = 35.75''$

(0.875" for #7 bar at bottom and 0.75/2 for #6 bar in the other direction)



Max: $V_u = 127.9 - 36.8 \times \frac{35.75''}{12} = 18.3$ K/ft

(at 'd' from face of support)

Shear capacity of concrete $(V_c) = 2\sqrt{f'_c}\, b_w d$ — Equation 22.5.5.1

$= 2\sqrt{5000}\,(12)(35.75) = 60{,}670$ lb/ft $= 60.7$ K/ft

$\phi V_c = (0.75)(60.7) = 45.5$ K/ft > 31.9 K/ft

Either the thickness of footing or strength of concrete can be reduced

Using $f'_c = 3000$ psi

$\phi V_c = \frac{(0.75)(2)\sqrt{3000}\,(12)(35.75)}{1000} = 35.2$ K/ft > 18.3 K/ft

In practical world, it is more economical to reduce thickness of footing.

Two-way shear

$\frac{d}{2} = \frac{35.75''}{2} = 17.88'' = 1.49'$

Two-way shear is checked under column with highest load — A-1 (120 K). All columns have free corner location

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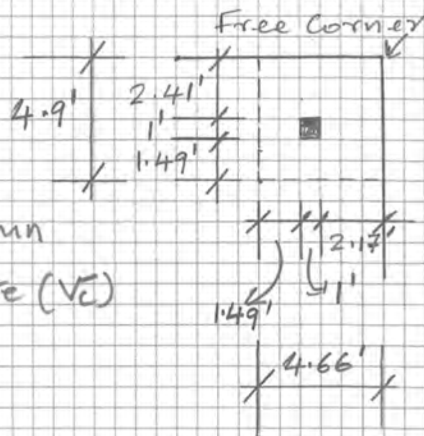
Shear perimeter (b_o)
$= 4.9 + 4.66 = 9.56'$



$\alpha_s = 20$ for corner column — Section 22.6.5.3

Shear capacity of concrete (V_c) — Section 26.6.5.2

Least of:

(a) $4\sqrt{f'_c}$

(b) $\left(2 + \frac{4}{\beta}\right)\sqrt{f'_c}$

(c) $\left(2 + \frac{\alpha_s d}{b_o}\right)\sqrt{f'_c}$

(a) $4\sqrt{f'_c}$

(b) $\left(2 + \frac{4}{1}\right)\sqrt{f'_c} = 6\sqrt{f'_c}$ $\beta = 1$ for square column

(c) $\left(2 + \frac{20 \times 35.75}{9.56 \times 12}\right)\sqrt{f'_c} = 8.23\sqrt{f'_c}$

Least value of $V_c = 4\sqrt{f'_c} = 4\sqrt{3000} = 219$ psi

Hence $\phi V_c = \frac{(0.75)(219)(9.56 \times 12)(35.75)}{1000}$

$= 673\text{ K} > 120\text{ K}$ OK

Check for bearing

Use the least value of B_n — Table 22.8.3.2

$B_n = 0.85 f'_c A_1 = \frac{0.85(3000)(12'' \times 12'')}{1000} = 367.2\text{ K}$

$\phi B_n = (0.65)(367.2) = 238.68\text{ K} > 120\text{ K}$

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Max: (−) M_u for bottom steel − 147.6 K-ft
Max: (+) M_u for top steel − 105.1 K-ft

Bottom Steel

$$R_n = \frac{M_u}{\phi b d^2} = \frac{147.6 \times 12000}{0.9 \times 12 \times 35.75^2} = 128.3 \text{ psi}$$

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

$$= \frac{0.85 \times 3000}{60{,}000}\left[1 - \sqrt{1 - \frac{2 \times 128.3}{0.85 \times 3000}}\right] = 0.0022$$

$A_s = 0.0022 \times 12 \times 35.75 = 0.94 \text{ in}^2/\text{ft}$ — #7@7" oc Both ways

Top Steel

$$R_n = \frac{105.1 \times 12000}{0.9 \times 12 \times 35.75^2} = 91.4 \text{ psi}$$

$$\rho = \frac{0.85 \times 3000}{60{,}000}\left[1 - \sqrt{1 - \frac{2 \times 91.4}{0.85 \times 3000}}\right] = 0.0016$$

Provide minimum reinforcement — Table 7.6.1.1

$A_s = 0.0018 \times 12 \times 35.75 = 0.77 \text{ in}^2/\text{ft}$ — #6 @ 6" Both way

Note: In the assumption to calculate the 'd' effective, #7 bar in one direction and #6 bar in the other direction was used but #7 in both direction at top and #6 in both direction at bottom is used

d for top steel $= 40 - 3'' - (0.875 + 0.875/2) = 35.7''$
d for bottom steel $= 40 - 3'' - (0.75 + 0.75/2) = 35.9''$ — Very close to 35.75".

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The thickness of the footing can also be reduced to 30"

Effective depth $= 30'' - 3'' - \left(0.75'' + \frac{0.75''}{2}\right) = 25.9''$

Two-way shear capacity of beam is very high and it would be sufficient to check one-way shear

$$\phi V_c = \frac{0.85(2)\sqrt{3000}(12)(25.9)}{1000} = 28.94\ K/ft$$
$$> 18.3\ K/ft \quad \text{O.K.}$$

Top Reinforcement

$$R_n = \frac{105.1 \times 12{,}000}{0.9 \times 12 \times 25.9^2} = 174\ psi$$

$$\rho = \frac{0.85 \times 3000}{60{,}000}\left[1 - \sqrt{1 - \frac{2 \times 174}{0.85 \times 3000}}\right] = 0.003$$

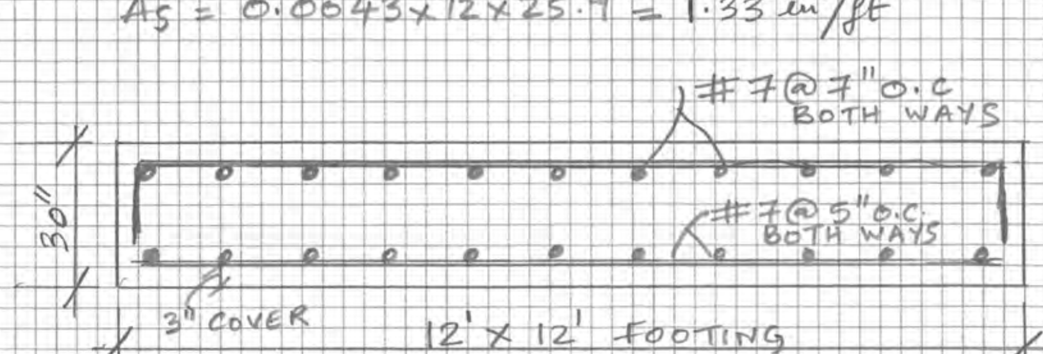
$A_s = 0.003 \times 12 \times 25.9 = 0.932\ in^2/ft$ #7@7"

Bottom Reinforcement

$$R_n = \frac{147.6 \times 12{,}000}{0.9 \times 12 \times 25.9^2} = 244\ psi$$

$$\rho = \frac{0.85 \times 3000}{60{,}000}\left[1 - \sqrt{1 - \frac{2 \times 244}{0.85 \times 3000}}\right] = 0.0043$$

$A_s = 0.0043 \times 12 \times 25.9 = 1.33\ in^2/ft$ #7@5"



I.4 PROBLEM I.4: STRAPPED FOOTING

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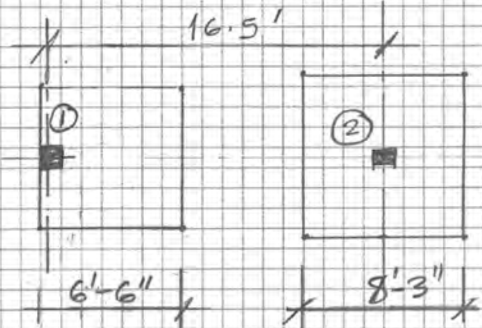
Design an eccentric footing and the strap beam shown in the sketch



COL (1) & (2) – 12"×12"

Bottom of footing – 4' below natural ground

Dead load of column (1) – 75 K

Live load of column (1) – 50 K

Dead load of column (2) – 100 K

Live load of column (2) – 150 K

Safe bearing capacity of soil – 4000 psf

Assume footing depth of 24"

Weight of footing – $\frac{24''}{12''} \times 150$ – 300 psf

Weight of soil – $2' \times 120$ – 240 psf

Net upward pressure of soil = 4000 − 300 − 240

= 3460 psf = 3.46 ksf

Area of footing (1) $= \frac{75+50}{3.46} = 36.1\ ft^2$

Area of footing (2) $= \frac{100+150}{3.46} = 72.3\ ft^2$

Provide 6'×6' footing (1)

8.5'×8.5' footing (2)

Eccentricity of footing (1) = (6'/2) − (1'/2) = 2.5'

Moment due to eccentricity = 125 × 2.5 = 312.5 K-ft

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Shear produced by moment $= \dfrac{312.5}{16.5} = 18.9\,K$

Reaction of footing (1) $= 75 + 50 + 18.9 = 143.9\,K$

Area of footing (1) $= \dfrac{143.9}{3.46} = 41.58\,ft^2$

Provide $6.5' \times 6.5' \times 2'$ footing.

Reaction of footing (2) $= 100 + 150 - 18.9 = 231.1\,K$

Area of footing (2) $= \dfrac{231.1}{3.46} = 66.79\,ft^2$

Provide $8.25' \times 8.25'$ footing.

Factored load of column (1) $= (1.2)(75) + (1.6)(50)$

$= 170\,K$

Factored load of column (2) $= (1.2)(100) + (1.6)(150)$

$= 360\,K$

Factored moment due to eccentricity $= 170 \times 2.75$

$= 467.5\,K/ft$

Factored shear due to moment $= \dfrac{467.5}{16.5}$

$= 28.3\,K$

Factored reaction at footing (1) $= 170 + 28.3$

$= 198.3\,K$

factored reaction at footing (2) $= 360 - 28.3$

$= 331.7\,K$

Factored footing (1) pressure $= \dfrac{198.3}{6.5} = 30.5\,\dfrac{K}{ft}$

Factored footing (2) pressure $= \dfrac{331.7}{8.25} = 40.2\,\dfrac{K}{ft}$

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Adopt a beam size of 20″ × 42″. The top surface of the beam shall at the ground level and the bottom surface 6″ above the bottom surface of the footing. The bottom surface of the beam is shored. After the casting of the concrete, the forms and the beams are removed.

Effective depth of beam = 42″ − 3″ − 0.5″ = 38.5″

$$R_n = \frac{M_U}{\phi b d^2} = \frac{467.5 \times 12000}{0.9 \times 20 \times 38.5^2} = 210.3 \text{ psi}$$

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

$$= \frac{0.85 \times 5000}{60{,}000}\left[1 - \sqrt{1 - \frac{(2)(210.3)}{(0.85)(5000)}}\right]$$

$$= 0.00359 \simeq \text{ min: } \rho \text{ of } 0.0033$$

Hence, $A_{st} = 0.00359 \times 24 \times 38.5 = 3.32 \text{ in}^2$

Provide 4 # 9 top and bottom

Maximum shear in beam = 28.3 K

Shear capacity of beam $(V_c) = 2\sqrt{f'_c}\, bwd$ — Equation 22.5.5.1

$$\phi V_c = \frac{(0.75)(108{,}894)}{1000} = 81.67 \text{ K} > 28.3 \text{ K}$$

Footing (1)

Check one-way shear at 'd' from face of column — Section 9.4.3.2

Effective depth (d) = 24″ − 3″ − 0.5″ = 20.5″

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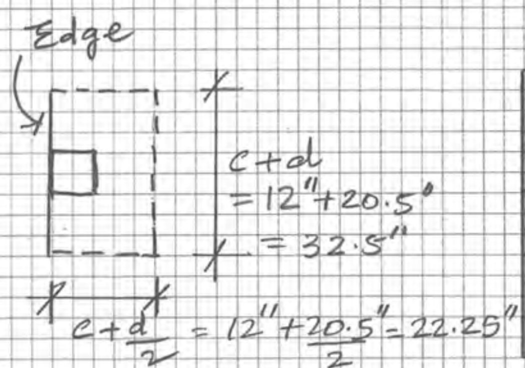
V_u at 'd' from face of column

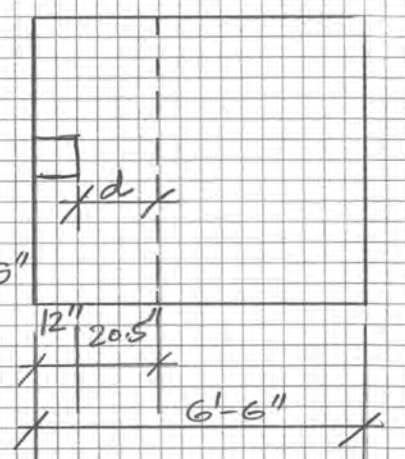
$= [6'\text{-}6'' - 12'' - 20.5''](30.5) = 115.7\text{K}$

Shear capacity of footing (ϕV_c)

$= \dfrac{(0.75)(2)\sqrt{5000}\,(6.5' \times 12'')(20.5)}{1000} = 170^{K} > V_u = 115.7\text{K}$



c – width of square column



ONE-WAY SHEAR

$b_o = 2(22.25) + 32.5$

$= 77''$

$\alpha_s = 30$ for edge column — Section 22.6.5.3

$\beta = 1$ for square column — Table 22.6.5.2

Two-way shear (v_c) is least of:

(a) $4\sqrt{f'_c}$ (b) $\left(2+\dfrac{4}{\beta}\right)\sqrt{f'_c}$ (c) $\left(2+\dfrac{\alpha_s d}{b_o}\right)\sqrt{f'_c}$

(a) $4\sqrt{f'_c}$

(b) $\left(2+\dfrac{4}{1}\right)\sqrt{f'_c} = 6\sqrt{f'_c}$

(c) $\left(2+\dfrac{30\times 20.5}{77}\right)\sqrt{f'_c} = 9.99\sqrt{f'_c}$

least (v_c) $= 4\sqrt{5000} = 282.8$ psi

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$\phi V_c = (0.75)(v_c) b_o d$

$= \dfrac{(0.75)(282.8)(77)(20.5)}{1000} = 334.8\ K > V_u = 170\ K.$

Calculate M_u at the face of the column — Table 13.2.7.1

Span of the footing along strap beam

$= 6.5' - 1' = 5.5'$

$M_u = 30.5 \times \dfrac{5.5^2}{2} = 461.3\ K\text{-}ft$

$R_n = \dfrac{461.3 \times 12000}{0.85 \times 6.5 \times 12 \times 20.5^2} = 198.8\ psi$

$\rho = \dfrac{0.85 \times 5000}{60{,}000}\left[1 - \sqrt{1 - \dfrac{(2)(198.8)}{(0.85)(5000)}}\right] = 0.033$

$\rho_{min} = 0.0018$

$A_s = 0.00333 \times 6.5 \times 12 \times 20.5 = 5.32\ in^2$

Provide 12 # 6 bar

Span of the footing perpendicular to strap beam $= \dfrac{6.5' - 6''}{2} = 3'$

$M_u = 30.5 \times \dfrac{3^2}{2} = 137.3\ K\text{-}ft$

$R_n = \dfrac{137.3 \times 12000}{0.85 \times 6.5' \times 12'' \times 20.5^2} = 59.7\ psi$ (Too low)

Use $\rho_{min} = 0.0018$

$A_{st} = 0.0018 \times 6.5 \times 12 \times 24 = 3.4\ in^2$

Provide 12 # 5 bars.

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Footing (2)

Check one-way shear at 'd' from the face of the column

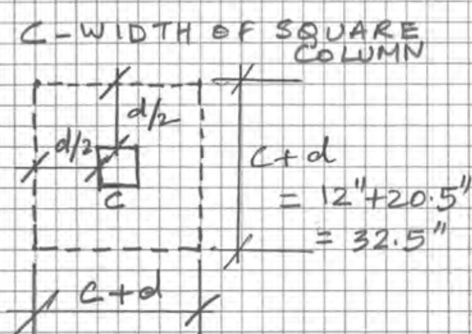
V_u at 'd' from face of the column

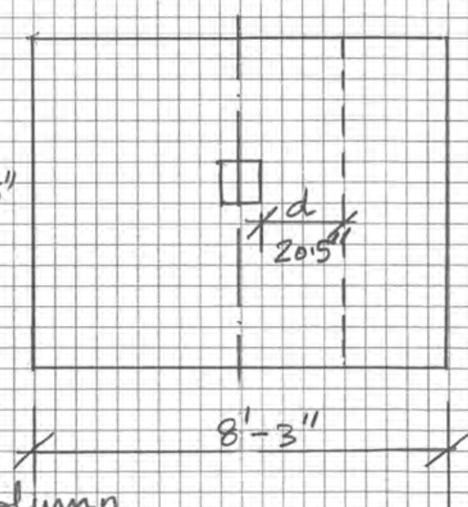
$$= \left(\frac{8'-3''}{2} - 0.5' - \frac{20.5''}{12}\right)(40.2) = 77.1\text{ K}$$

Shear capacity of footing (ϕV_c)

$$= \frac{(0.75)(2)\sqrt{5000}\,(8.25'\times 12'')(20.5)}{1000} = 215.3\text{K}$$

$$> 77.1\text{K}$$ OK.





Shear perimeter (b_o)

$= 4 \times 32.5 = 130''$

$\alpha_s = 40$ for interior column — Section 22.6.5.3

$\beta = 1$ for square column

Two-way shear is least of:

(a) $4\sqrt{f'_c}$

(b) $\left(2 + \frac{4}{1}\right)\sqrt{f'_c} = 6\sqrt{f'_c}$

(c) $\left(2 + \frac{40\times 20.5}{130}\right)\sqrt{f'_c} = 8.3\sqrt{f'_c}$

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Least $v_c = 4\sqrt{f'_c} = 4\sqrt{5000} = 282.8$ psi

$\phi V_c = \frac{(0.75)(282.8)(130)(20.5)}{1000} = 565K > 345K$

Maximum moment in the footing occurs at the face of the column and is same in both direction because of symmetry.

Span $= \frac{8.25'}{2} - 0.5' = 3.625'$

$M_U = 40.2 \times \frac{3.625^2}{2} = 266.8$ K-ft

$R_n = \frac{266.8 \times 12000}{0.85 \times 8.25 \times 12 \times 20.5^2} = 90.5$ psi

Too low

Use $\rho_{min} = 0.0018$

$A_{st} = 0.0018 \times 8.25' \times 12'' \times 24'' = 4.27$ in^2

Provide 14 #5 bars each way.

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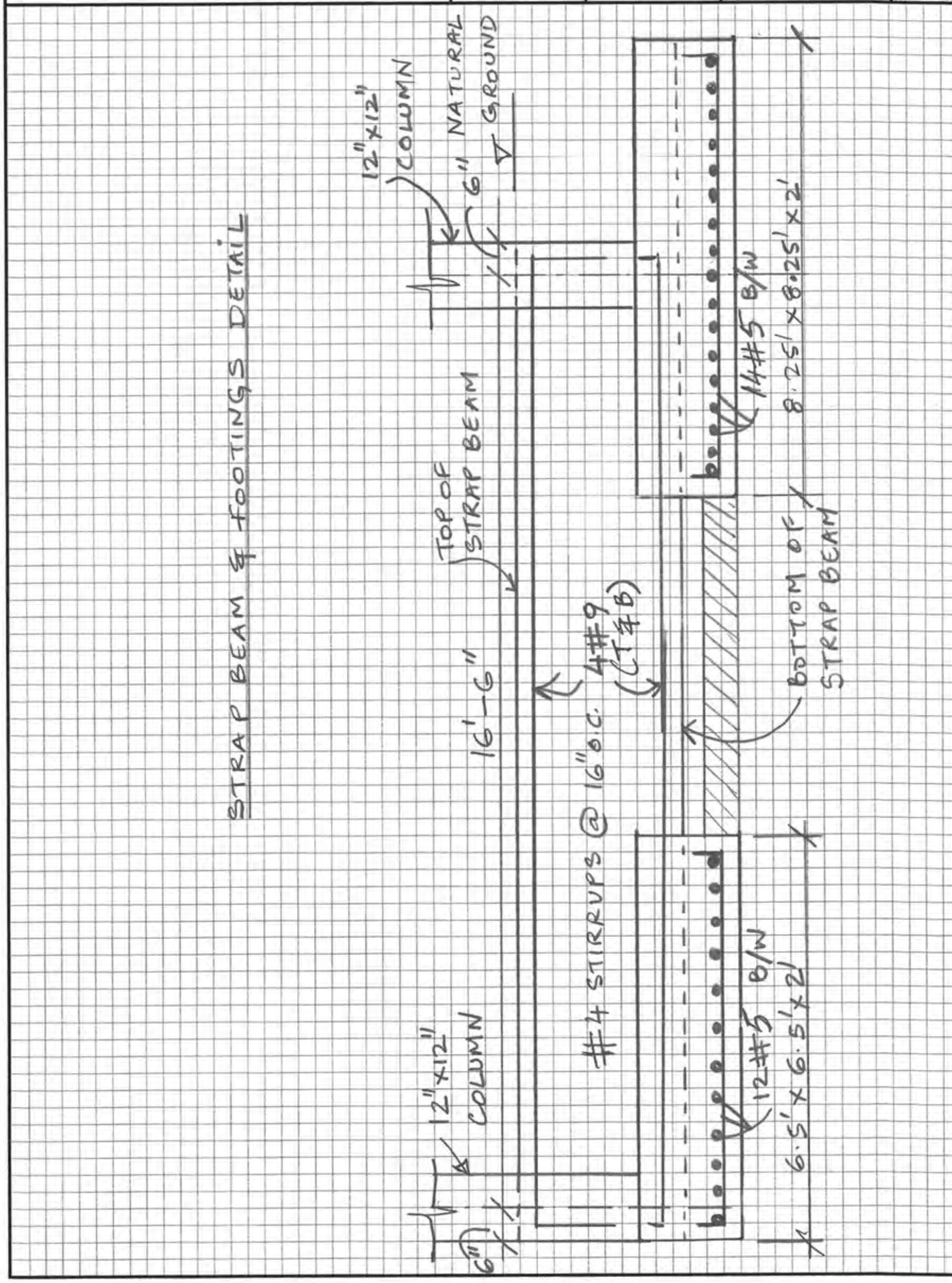

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