

An ACI Manual

# ACI Reinforced Concrete Design Handbook

A Companion to ACI 318-19



Volume 1: Member Design  
MNL-17(21)



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**ACI MNL-17(21)**

# **ACI REINFORCED CONCRETE DESIGN HANDBOOK**

**A Companion to ACI 318-19**

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**STRUCTURAL SYSTEMS**

**STRUCTURAL ANALYSIS**

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**ACI MNL-17(21)  
Volume 1**

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## DEDICATION



This edition of *The ACI Reinforced Concrete Design Handbook*, MNL-17(21), is dedicated to the memory of Daniel W. Falconer and his many contributions to the concrete industry. He was Managing Director of Engineering for the American Concrete Institute from 1998 until his death in July 2015.

Dan was instrumental in the reorganization of “Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)” as he served as ACI staff liaison to ACI Committee 318, Structural Concrete Building Code; and ACI Subcommittee 318-SC, Steering Committee. His vision was to simplify the use of the Code for practitioners and to illustrate the benefits of the reorganization with MNL-17. His oversight and review comments were instrumental in the development of the ninth edition of the Handbook.

An ACI member since 1982, Dan served on ACI Committees 344, Circular Prestressed Concrete Structures, and 373, Circular Concrete Structures Prestressed with Circumferential Tendons. He was also a member of the American Society of Civil Engineers. Prior to joining ACI, Dan held several engineering and marketing positions with VSL Corp. Before that, he was Project Engineer for Skidmore, Owings, and Merrill in Washington, DC. He received his BS in civil engineering from the University at Buffalo, Buffalo, NY and his MS in civil and structural engineering from Lehigh University, Bethlehem, PA. He was a licensed professional engineer in several states.

In his personal life, Dan was an avid golfer, enjoying outings with his three brothers whenever possible. He was also an active member of Our Savior Lutheran Church in Hartland, MI, and a dedicated supporter and follower of the Michigan State Spartans basketball and football programs. Above all, Dan was known as a devoted family man dedicated to his wife of 33 years, Barbara; his children Mark, Elizabeth, Kathryn, and Jonathan; and two grandsons, Samuel and Jacob.

In his memory, the ACI Foundation has established an educational memorial. For more information visit <http://www.scholarshipcouncil.org/Student-Awards>. Dan will be sorely missed for many years to come.

# FOREWORD

*The ACI Reinforced Concrete Design Handbook* provides assistance to professionals engaged in the design of reinforced concrete buildings and related structures. This edition is a major revision that brings it up-to-date with the approach and provisions of “Building Code Requirements for Structural Concrete” (ACI 318-19).

*The ACI Reinforced Concrete Design Handbook* provides dozens of design examples of various reinforced concrete members, such as one- and two-way slabs, beams, columns, walls, diaphragms, footings, and retaining walls. For consistency, many of the numerical examples are based on a fictitious seven-story reinforced concrete building. There are also many additional design examples not related to the design of the members in the seven-story building that illustrate various ACI 318-19 requirements.

Each example starts with a problem statement, then provides a design solution in a three-column format—Code provision reference, short discussion, and design calculations—followed by a drawing of reinforcing details, and finally a conclusion elaborating on a certain condition or comparing results of similar problem solutions.

In addition to examples, almost all chapters in *The ACI Reinforced Concrete Design Handbook* contain a general discussion of the related ACI 318-19 chapter.

This edition of *The ACI Reinforced Concrete Design Handbook* was updated and enhanced by ACI staff engineers under the auspices of the ACI Technical Activities Committee (TAC). Each chapter was reviewed by at least two reviewers, who provided valuable comments, suggestions, and insights. The following reviewers are gratefully acknowledged and thanked:

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*The ACI Reinforced Concrete Design Handbook* is published in two volumes: Chapters 1 through 11 are published in Volume 1 and Chapters 12 through 15 are published in Volume 2. Design aids and a moment interaction diagram Excel spreadsheet are available for free download from the following ACI webpage links:

<https://www.concrete.org/MNL1721Download1>

<https://www.concrete.org/MNL1721Download2>

**Keywords:** anchoring to concrete; beams; columns; cracking; deflection; diaphragm; durability; flexural strength; footings; frames; pile caps; piles; post-tensioning; punching shear; retaining wall; shear strength; seismic; slabs; splicing; stiffness; structural analysis; structural systems; strut-and-tie; walls.

Trey Hamilton  
*Managing Editor*

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# CHAPTER 1—INTRODUCTION

## 1.1—Introduction

This Manual is intended to assist with the design of reinforced concrete structures using ACI 318-19 (hereinafter referred to as the Code). The focus is on the application of the Code requirements to the individual members with respect to both structural design requirements and detailing provisions. As with the Code, the design procedures and detailing practices illustrated in this Manual do not replace sound professional judgment or the licensed design professional's (LDP's) knowledge of the specific factors surrounding a project.

To illustrate the procedures and details, it is necessary to generate the member actions for which the design will be conducted. Although this Manual provides background and context regarding the analysis of structural concrete systems, it is assumed that the user of this Manual has a basic understanding of structural analysis and the development of the member design actions from such an analysis.

This chapter describes the overall organization of this Manual and additionally describes the loads, geometry, and other details of the example building used to generate actions for member or component design illustrated in subsequent chapters.

## 1.2—Organization and use

A structural system consists of members, joints, and connections, each performing a specific role or function. Structural systems and their component members must provide sufficient stability, strength, and stiffness so that overall structural integrity is maintained, design loads are resisted, and serviceability limits are met.

This Manual is organized into chapters listed below that follow the general progression of the structural design of a building. The early chapters describe the overall building configuration, loads, and development of actions from structural analysis followed by chapters devoted to the design of the individual members within the example structure.

- Horizontal floor and roof members (one-way and two-way slabs, Chapters 7 and 8)
- Horizontal support members (beams and joists, Chapter 9)
- Vertical members (columns and structural walls, Chapters 10 and 11)
- Diaphragms and collectors (Chapter 12)
- Foundations—isolated footings, mats, pile caps, and piles (Chapter 13)
- Plain concrete—unreinforced foundations, walls, and piers (Chapter 14)
- Joints and connections (Chapters 15 and 16)

In Table 1.2, Code chapters are correlated with the chapters in Volumes 1 and 2 of this Manual.

The fictitious example building depicted in Fig. 1.2a through 1.2d was created to demonstrate how, by various examples in this Manual, to design and detail a typical struc-

**Table 1.2—Member chapters**

| Volume No.<br>ACI MNL-17(21) | Chapter name<br>ACI MNL-17(21) | Chapter No. |                |
|------------------------------|--------------------------------|-------------|----------------|
|                              |                                | ACI 318-19  | ACI MNL-17(21) |
| I                            | Building system                | —           | 1              |
|                              | Structural systems             | 4 and 5     | 2              |
|                              | Structural analysis            | 6           | 3              |
|                              | Durability                     | 19          | 4              |
|                              | One-way slab                   | 7           | 5              |
|                              | Two-way slab                   | 8           | 6              |
|                              | Beams                          | 9           | 7              |
|                              | Diaphragm                      | 12          | 8              |
|                              | Columns                        | 10          | 9              |
|                              | Walls                          | 11          | 10             |
|                              | Foundations                    | 13          | 11             |
| II                           | Retaining walls                | 7 and 13    | 12             |
|                              | Serviceability                 | 24          | 13             |
|                              | Strut and tie                  | 23          | 14             |
|                              | Anchoring to concrete          | 17          | 15             |

tural concrete building according to the Code. This example building is seven stories above ground and has a one-story basement. The building has evenly spaced columns along the grid lines in both directions. One column has been removed along Grid C on the second level to provide open space for the lobby. The building dimensions are:

- Width (north/south) = 72 ft (5 bays @ 14 ft)
- Length (east/west) = 218 ft (6 bays @ 36 ft)
- Height (above ground) = 92 ft
- Basement height = 10 ft

The basement is used for storage, building services, and mechanical equipment. It is 10 ft high and has an extra column in every bay along Grids A through F to support a two-way slab at the second level. There are basement walls at the perimeter.

A single specific gravity load system is not specified herein but rather is left unknown to enable demonstration of the design of several structural systems including nonprestressed and prestressed one-way beam and slab systems; nonprestressed and prestressed two-way slab systems; nonprestressed and prestressed transfer girder to accommodate column removal; and nonprestressed and prestressed beams of various types and sizes. Lateral loads are resisted by concrete shear walls in the north/south direction and concrete moment frames in the east/west direction; both systems are designated as *ordinary* for the purposes of seismic design and detailing. In some cases, member examples are expanded to demonstrate the change in design and detailing procedures when elements or systems are designated as *intermediate* or *special*, but using the requirements from the original structural

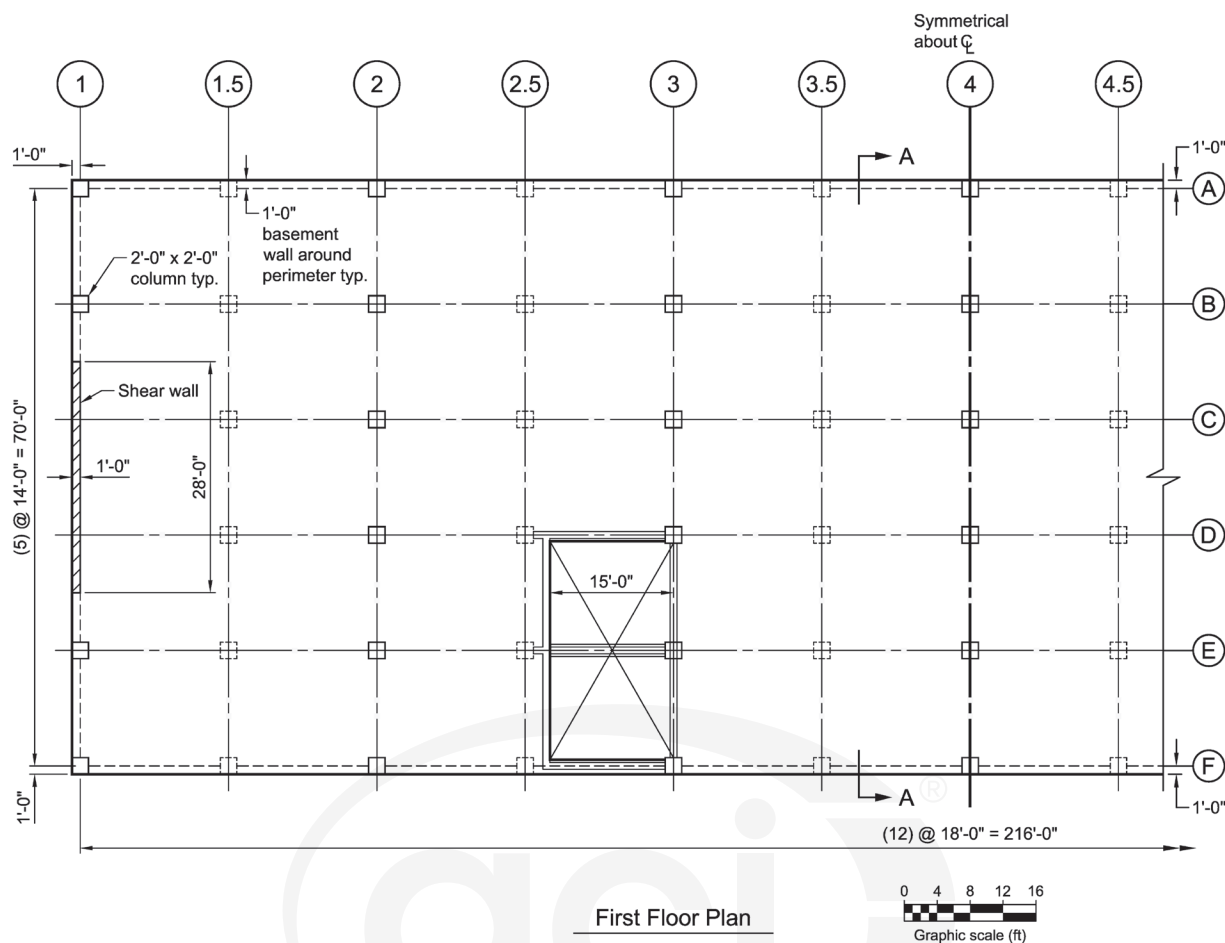


Fig. 1.2a—Example building first floor plan.

analysis. Those examples may modify this initial data to demonstrate some specific code requirement.

This building example was created for the purpose of illustrating design of structural concrete members and systems. Other aspects of building design that may affect the structural layout such as occupancy, egress, fire protection, or other architectural constraints have not been addressed.

### 1.3—Building plans and elevation

The following building plans and elevation illustrate the structural layout and some details of the example building.

#### 1.4—Loads

The following loads for the example building are generated in accordance with ASCE/SEI 7. Risk Category II is assumed.

##### Gravity Loads

Dead Load,  $D$ :

- Self-weight
- Additional  $D = 15 \text{ lb/ft}^2$
- Perimeter walls =  $15 \text{ lb/ft}^2$

Live Load:

- First and Second Floors: Lobbies, public rooms, and corridors serving them =  $100 \text{ lb/ft}^2$

- Typical Floor: Private rooms and corridors serving them =  $65 \text{ lb/ft}^2$

Roof Live Load:

- Unoccupied =  $20 \text{ lb/ft}^2$

Snow Load:

- Ground load,  $P_g = 20 \text{ lb/ft}^2$
- Thermal,  $C_t = 1.0$
- Exposure,  $C_e = 1.0$
- Importance,  $I_s = 1.0$
- Flat roof load,  $P_f = 20 \text{ lb/ft}^2$

##### Lateral Loads

Wind Load:

- Basic (ultimate) wind speed = 115 mph
- Exposure category =  $C$
- Wind directionality factor,  $K_d = 0.85$
- Topographic factor,  $K_{st} = 1.0$
- Gust-effect factor,  $G_f = 0.85$  (rigid)
- Internal pressure coefficient,  $GC_{pi} = \pm 0.18$

Directional Procedure Seismic Load:

- Importance,  $I_e = 1.0$
- Site class =  $D$
- $S_S = 0.15$ ,  $S_{DS} = 0.16$
- $S_1 = 0.08$ ,  $S_{D1} = 0.13$
- Seismic design category =  $B$
- Equivalent lateral force procedure

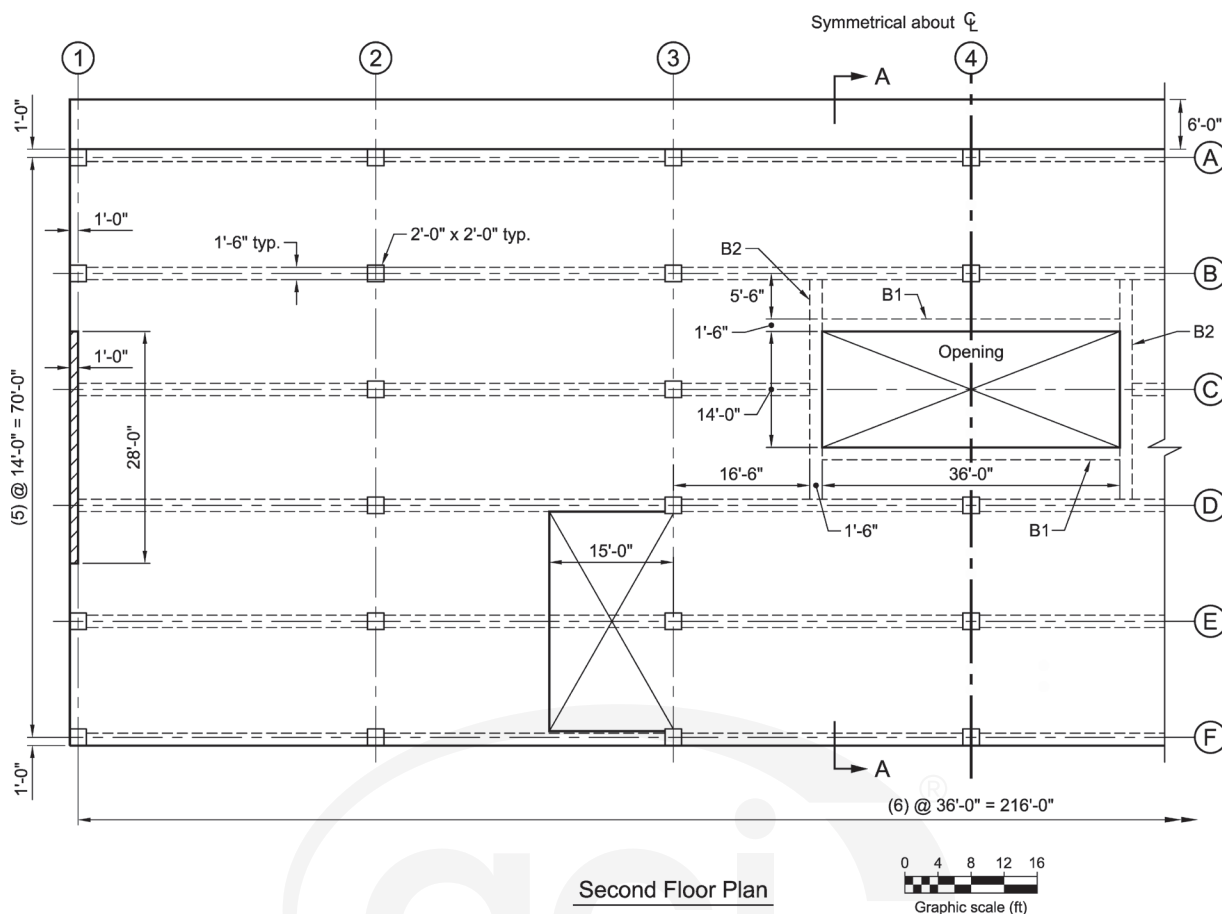


Fig. 1.2b—Example building second floor plan.

- Ordinary reinforced concrete shear walls in the north-south direction
  - $R = 5$
  - $C_s = 0.046$
- Ordinary reinforced concrete moment frame in the east-west direction
  - $R = 3$
  - $C_s = 0.032$

### 1.5—Material properties

The designer should investigate and acquire a reasonable knowledge of locally available concrete and steel materials. Concrete properties are typically selected based on both mechanical properties and durability. Code Chapter 19 provides limitations, requirements, and guidance on the selection of  $f'_c$ . The chapter also provides requirements for durability of concrete, which will be discussed further in Chapter 4 of this Manual.

Code Table 19.2.1.1 provides minimum  $f'_c$  for use with a variety of structural systems and seismic design categories (SDCs). Minimum required strength for general use in SDC A, B, or C is 2500 psi. For typical floor spans and loads, however, an  $f'_c$  of 4000 psi is usually sufficient to satisfy strength requirements. For the example used in this Manual, the building height is moderate and the loads are typical. Assume that the locally available aggregate is a

durable dolomitic limestone. Thus, the concrete can readily have a higher  $f'_c$  than the initial assumption of 4000 psi. A check of the durability requirements of Code Table 19.3.2.1 shows that 5000 psi will satisfy minimum  $f'_c$  for all exposure classes. The following concrete material properties are chosen:

- $f'_c = 5000$  psi
- Normalweight  $w_c = 150$  lb/ft<sup>3</sup>
- $E_c = 4,030,000$  psi (Code Eq. (19.2.2.1b))
- $\nu = 0.2$
- $e_{th} = 5.5 \times 10^{-6}/F$  (ACI 209R)

To minimize space occupied by heavily loaded columns and walls in multi-story buildings, the designer may choose to use a larger  $f'_c$  for columns than is used for the floor system. Concrete placement usually proceeds in two stages for each story; first, the vertical members, such as columns, and second, the floor members, such as beams and slabs. This results in column loads being transferred through the lower-strength concrete of the floor system. It may be desirable to specify  $f'_c$  of the floor system to be greater than  $0.7 \times f'_c$  of the vertical members to avoid having to place higher-strength concrete in the floor system in the area of contact between floor and column (Code Sections 15.5.1 and 15.5.1a). Usually this situation only becomes an issue for taller buildings.

The most common and most available nonprestressed reinforcement is ASTM A615 Grade 60. Higher grades are

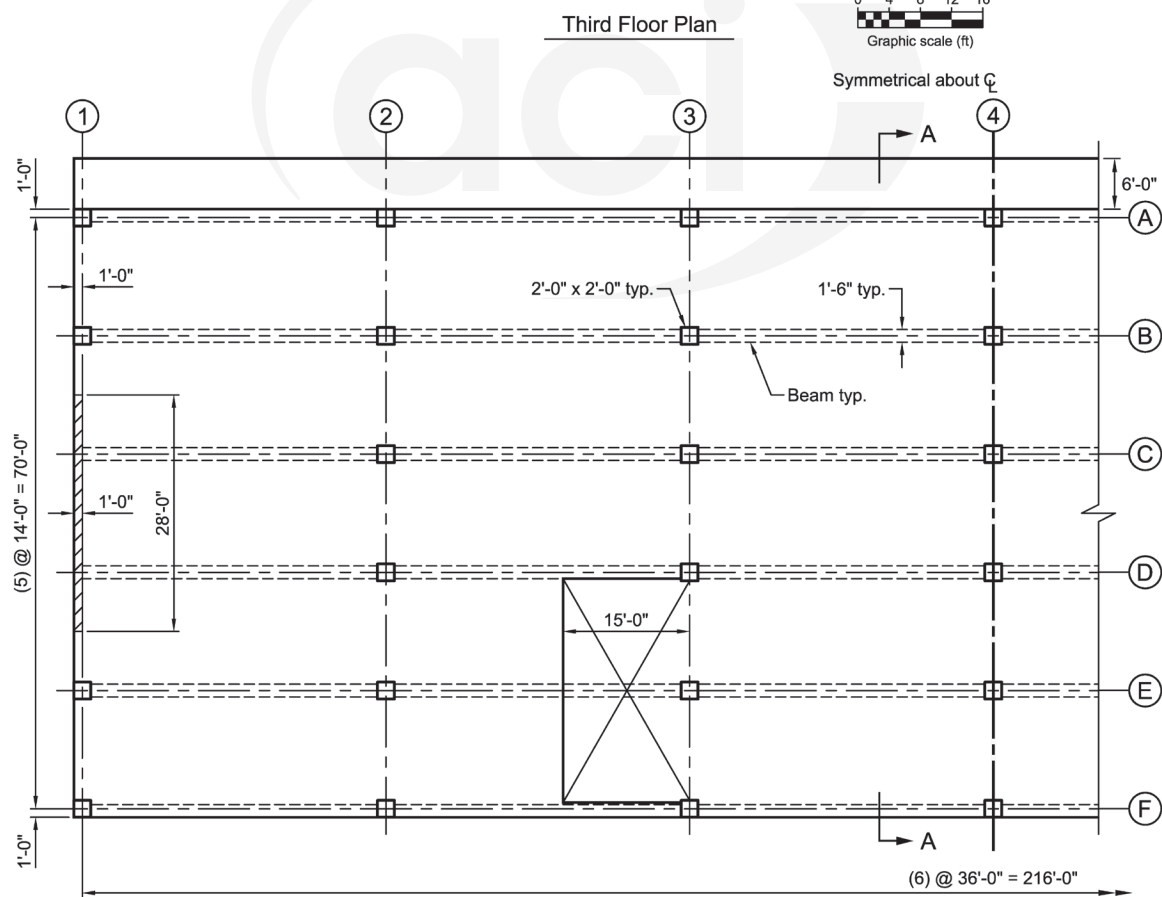
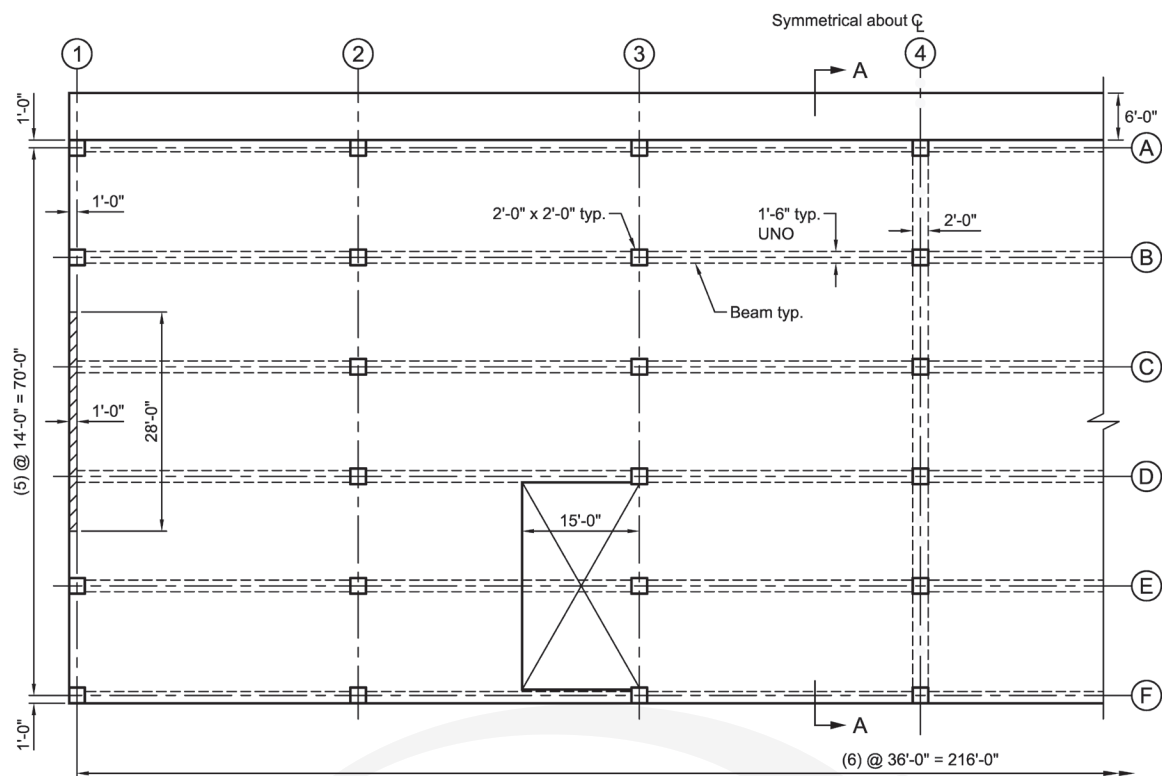


Fig. 1.2c—Example building floor plans for third through seventh floor.

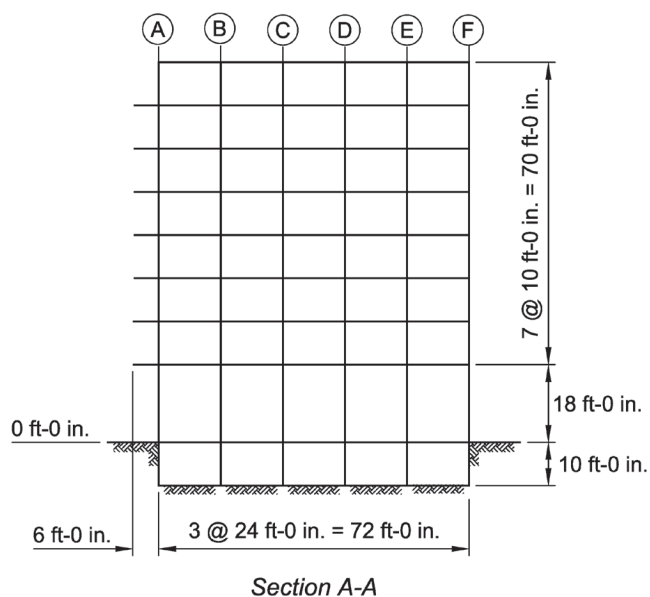


Fig. 1.2d—Example building section.

available but are subject to some limitations by the Code. The modulus of elasticity for reinforcement,  $E_s$ , is given in Code Section 20.2.2.2. The Code allows the use of wire, strand, or bar as prestressed reinforcement. The most commonly used prestressed reinforcement, however, is ASTM A416 Grade 270 seven-wire prestressing strand.

Assumed reinforcement material properties

- $f_y = 60,000$  psi
- $f_{yt} = 60,000$  psi
- $E_s = 29,000,000$  psi
- $f_{pu} = 270,000$  psi
- $E_p = 28,500,000$  psi

The use of lightweight concrete can reduce seismic forces, column loads, and foundation loads, which will allow savings in both concrete and reinforcement in taller or heavier buildings. Because this building is of moderate height, carries modest loading, and is designed for low seismic risk, the use of lightweight concrete is not likely to provide an economic advantage over normalweight concrete.

## REFERENCES

*American Concrete Institute*

ACI 209R-92(08)—Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

*American Society of Civil Engineers*

ASCE/SEI 7-16—Minimum Design Loads for Buildings and Other Structures

*ASTM International*

A416/A416M-18—Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

A615/A615M-18e1—Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement



## CHAPTER 2—STRUCTURAL SYSTEMS

### 2.1—Introduction

Structural concrete design has evolved from emphasizing the design of individual members to designing the structure as an entire system. Ultimately, however, the individual members must be designed and detailed according to their distinctive role in the system. Chapter 4 of the Code addresses the role and use of an individual member in the context of the entire structural concrete system. This chapter of the Manual provides an introduction and overview of the design lateral loads that might be encountered in design. In addition, common lateral load and gravity load systems used in structural concrete buildings are described.

Prior to the 1970s, reinforced concrete buildings that were of moderate height (less than 20 stories), not in seismically active areas, or constructed with nonstructural masonry walls and partitions, were seldom explicitly designed for lateral forces (ACI Committee 442 1971). Continuing research, advancement in materials science, and improvements in analysis tools have allowed structural engineers to develop economical building designs with more reliable structural performance.

For a typical structural concrete building, the structural system can be subdivided into the gravity load-resisting system and the lateral-force resisting system. Elements such as the floor system, walls, columns, and foundation contribute to one or both of the load-resisting systems. A structural engineer's primary concern is to design these systems to harmoniously support the anticipated design loads in a safe and serviceable manner.

### 2.2—Design lateral loads

The Code provisions are intended to address dead, live, earthquake, and wind loads such as those recommended in ASCE/SEI 7. These loads are applied to the structural system or directly to individual members, as applicable. Gravity loads are typically assumed to be applied vertically. Earthquake and wind loads are typically applied horizontally and are assumed to act in orthogonal directions. This section covers the following loads as they relate to Code provisions:

1. Wind loading (Code Chapter 6, elastic analysis)
2. Earthquake loading (Code Chapter 18)

Wind and earthquake loads are dynamic in nature; however, they differ in the manner in which these loads are induced on a structure. Wind loads are externally applied loads and, hence, are related to the structure's exposed surface. Earthquake loads are inertial forces related to the magnitude and distribution of the mass in the structure.

**2.2.1 Wind loading**—Wind kinetic energy is transformed into potential energy when it is resisted by an obstruction. Wind pressure is related to the wind velocity, building height, building surface, the surrounding terrain, and the location and size of other local structures. The structural response to a turbulent wind environment is predominantly in the first mode of vibration.

The quasi-static approach to wind load design has generally proved sufficient. Aeroelastic effects in tall buildings, however, may cause vibrations that cause occupant discomfort or damage partitions or glass. Therefore, to determine design wind loads for very tall buildings, wind tunnel testing may be required.

**2.2.2 Earthquake loading**—The main objective of structural design is life safety—that is, preserving the lives of occupants and passersby. Serviceability and minimizing economical loss, however, are also important objectives. By studying the results of previous earthquakes on various structural systems, improvements to code provisions and design practices have been achieved. These improvements have led to a reduction in damage of reinforced concrete structures that experience an earthquake. Some code improvements for members that resist the load effects of significant seismic accelerations include design and detailing requirements that:

1. Ensure that columns in a frame are flexurally stronger than beams—the so-called “strong column-weak beam” concept.
2. Increase ductility and improve energy dissipation (with less deterioration in stiffness and strength).
3. Ensure members yield in flexure before reaching nominal shear strength, thus protecting the energy dissipation capability of the plastic hinge.
4. Ensure connections are stronger than the members framing into them.
5. Limit structural system irregularities.

For most structures, the equivalent lateral force procedure given in ASCE/SEI 7 is used.

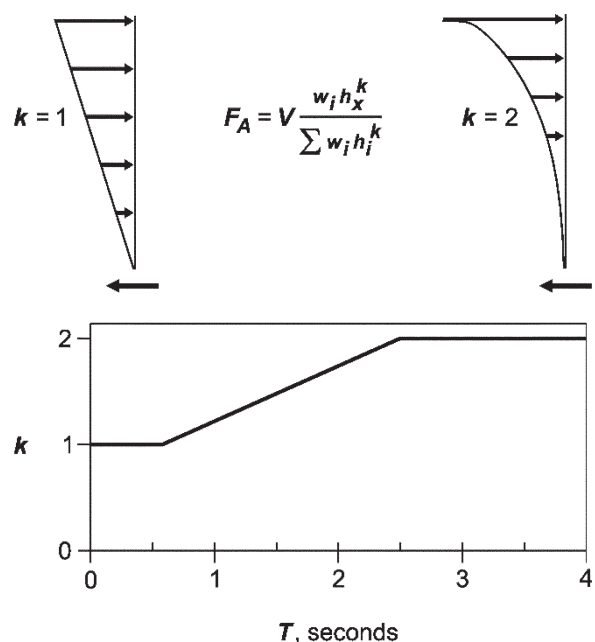


Fig. 2.2.2—Typical distribution of equivalent lateral forces representing seismic forces (adapted from ASCE/SEI 7)

Based on this procedure, the distribution of design forces along the height of a building roughly approximates the building's fundamental mode of vibration (Fig. 2.2.2).

Applying recorded earthquake motions to a structure through elastic dynamic analyses usually result in greater force demands than from the earthquake design forces specified by most codes. This is because codes generally account for force reductions due to inelastic response. For example, ASCE/SEI 7 applies an  $R$  factor (response modification factor), which accounts for the ductility of a building, system overstrength, and energy dissipation through the soil-foundation system (ASCE/SEI 7). It simplifies the seismic design process such that linear static elastic analysis can be used for building designs. The  $R$  factor reduces the calculated lateral loads and assumes a building may be damaged during an earthquake event, but will not collapse. The higher the  $R$ -value, the lower the lateral design load on a structure.  $R$ -values range from 1-1/2 for structures with stiff systems having low ductility to 8 for ductile systems having significant ductility. In a design-level earthquake, it is expected that some building members will yield. To promote appropriate inelastic behavior, the Code contains provisions meant to ensure inelastic deformation capacity in regions where yielding is likely, which then protects the overall integrity and stability of the building.

Dynamic (modal) analysis is commonly used for larger structures, important structures, or for structures with an irregular vertical or horizontal distribution of stiffness or mass. For very important and potentially critical structures—for example, nuclear power plants—inelastic dynamic analysis may be used (ACI Committee 442 1988).

## 2.3—Structural systems

All structures must have a continuous load path that can be traced from all load sources or load application to the foundation. The joints between the vertical members (columns and walls) and the horizontal members (beams, slabs, diaphragms, and foundations) are crucial to this concept. Properly detailed cast-in-place (CIP) reinforced concrete joints transfer moments and shears from the floor into columns and walls, thus creating a continuous load path. Joint design strength is covered in Chapter 15 of the Code. Further design and detailing information can be found in ACI 352R.

Engineers commonly refer to a structure's gravity-load-resisting system (GLRS) and lateral-force-resisting system (LFRS). All members of a CIP reinforced concrete structure contribute to the GLRS and most contribute to both systems. For low-rise structures, the inherent lateral stiffness of the GLRS is often sufficient to resist the design lateral forces without changes to the design or detailing of the GLRS members. As the building increases in height, however, the importance of designing and detailing the LFRS to resist lateral loads increases. When sufficient building height is reached, stiffness rather than strength will govern design of the LFRS. In the design process, the type of LFRS is usually influenced by architectural considerations and construction requirements.

There are several types of structural systems or a combination thereof to resist gravity, lateral, and other loads, with deformation behavior as follows:

1. **Frames**—Lateral deformations are primarily due to story shear. The relative story deflections therefore depend on the horizontal shear applied at each story level.

2. **Walls**—Lateral deformations are due to both shear and bending. The predominate behavior mode depends on the wall's height-to-width aspect ratio.

3. **Dual systems**—Dual systems are a combination of moment-resisting frames and structural walls. The moment-resisting frames support gravity loads, and up to 25 percent of the lateral load. The structural walls resist the majority of the lateral loading.

4. **Frames with closely spaced columns, known as cantilevered column system or a tube system**—Lateral deformations are due to both shear and bending, similar to a wall. Wider openings in a tube, however, can produce a behavior intermediate between that of a frame and a wall.

Regardless of the system, a height is reached at which the resistance to lateral sway will govern the design of the structural system. At such a height, stiffness, not strength, controls the building design.

ASCE/SEI 7 provides provisions for assigning a structure to a Seismic Design Category (SDC A through F). As a building's Seismic Design Category increases, from A through F, ASCE/SEI 7 requires a progressively more rigorous seismic design and a more ductile system to maintain an acceptable level of seismic performance.

The Code provides three categories of earthquake detailing: ordinary, intermediate, and special. These categories provide an increasing level of system ductility and toughness.

Building height limits in ASCE/SEI 7 are related to the LFRS.

For buildings in SDC A and B, wind load will usually control the design of the LFRS.

For buildings in SDC C, seismic loads are likely to control design forces, and seismic detailing is required. LFRSs are not limited in height for most systems for this SDC, but interstory drift limits from ASCE/SEI 7 must be met. Again, stiffness, not strength, will likely control the LFRS design.

For buildings in SDC D, E, and F, seismic loads almost always control design forces, and increased seismic detailing is required. LFRSs often have height limitations based on assumed structural performance. Figure 2.3 shows approximate height limits for different structural systems.

Table 2.3 provides ASCE/SEI 7 limits for choosing a structural system for a particular building. The ranges of applicability shown are influenced by occupancy requirements, architectural considerations, internal traffic flow (particularly in the lower floors), the structure's height and aspect ratio, and load intensity and types (live, wind, and earthquake).

**2.3.1 Gravity-load-resisting systems**—A GLRS is composed of horizontal floor members and vertical members that support the horizontal members.

Gravity loads are resisted by reinforced concrete members through axial, flexural, shear, and torsional stiffness and

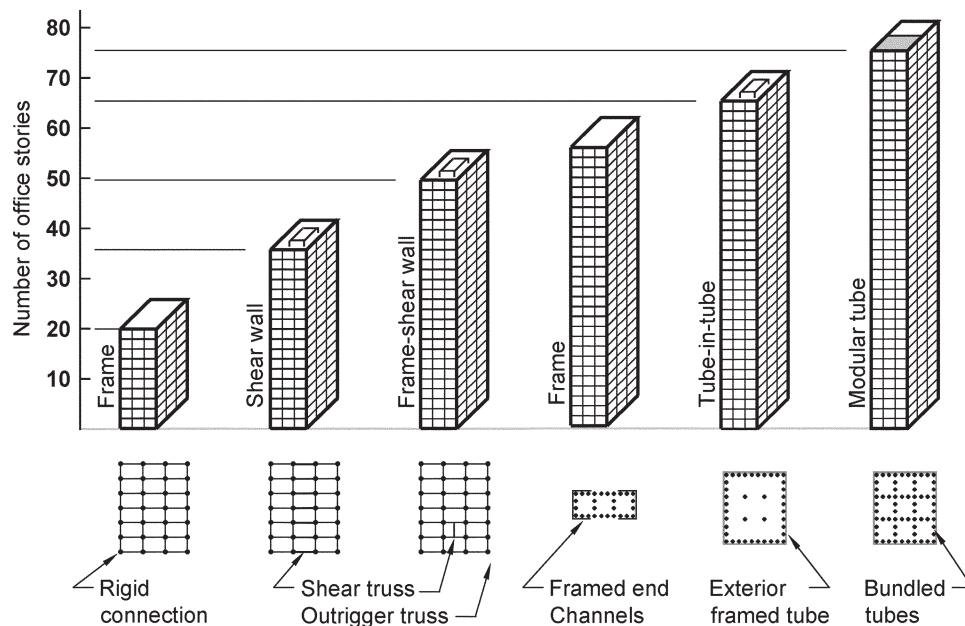


Fig. 2.3—Structural systems and optimum height limitations (Ali and Moon 2007).

Table 2.3—Approximate building height limits for various LFRS

| Practical limit of system (ASCE/SEI 7 limit according to SDC)                                                                 |         |     |        |        |        |
|-------------------------------------------------------------------------------------------------------------------------------|---------|-----|--------|--------|--------|
| Type of LFRS                                                                                                                  | SDC     |     |        |        |        |
|                                                                                                                               | A and B | C   | D      | E      | F      |
| Moment-resisting frames (only):                                                                                               |         |     |        |        |        |
| Ordinary moment frame (OMF)                                                                                                   | NL*     | NP* | NP     | NP     | NP     |
| Intermediate moment frame (IMF)                                                                                               | NL      | NL  | NP     | NP     | NP     |
| Special moment frame (SMF)                                                                                                    | NL      | NL  | NL     | NL     | NL     |
| Structural walls (only):                                                                                                      |         |     |        |        |        |
| Building frame systems (structural walls are the primary LFRS and frames are the primary GLRS):                               |         |     |        |        |        |
| Ordinary structural wall (OSW)                                                                                                | NL      | NL  | NP     | NP     | NP     |
| Special structural wall (SSW) <sup>†</sup>                                                                                    | NL      | NL  | 160 ft | 160 ft | 100 ft |
| Bearing wall systems (structural walls are the primary lateral- and gravity-load-resisting system):                           |         |     |        |        |        |
| OSW                                                                                                                           | NL      | NL  | NP     | NP     | NP     |
| SSW <sup>†</sup>                                                                                                              | NL      | NL  | 160 ft | 160 ft | 100 ft |
| Dual systems (structural walls are the primary LRFS, and the moment-resisting frames carry at least 25% of the lateral load): |         |     |        |        |        |
| OSW with OMF                                                                                                                  | NL      | NP  | NP     | NP     | NP     |
| OSW with IMF                                                                                                                  | NL      | NL  | NP     | NP     | NP     |
| OSW with SMF                                                                                                                  | NL      | NL  | NP     | NP     | NP     |
| SSW with OMF                                                                                                                  | NP      | NP  | NP     | NP     | NP     |
| SSW with IMF                                                                                                                  | NL      | NL  | 160 ft | 100 ft | 100 ft |
| SSW with SMF                                                                                                                  | NL      | NL  | NL     | NL     | NL     |

\*NL is no limit; NP is not permitted.

<sup>†</sup>Height limits can be increased per ASCE/SEI 7, Section 12.2.5.4.

strength. The related deformations are exaggerated and shown in Fig. 2.3.1.

**2.3.2 Lateral-load-resisting system**—An LFRS must have an adequate toughness to maintain integrity during high wind loading and design earthquake accelerations. Buildings are basically cantilevered members designed for strength (axial, shear, torsion, and moment) and serviceability (deflection and creep must be considered for tall buildings).

Code Section 18.2.1 lists the design and detailing requirements for each SDC as it applies to a specific seismic-force-resisting system. The following LFRSs are addressed as follows.

**2.3.2.1 Moment-resisting frames**—Cast-in-place moment-resisting frames derive their load resistance from member



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strengths and connection rigidity. In a moment-resisting frame structure, the lateral displacement (drift) is the sum of three parts: 1) deformation due to bending in columns, beams, slabs, and joint deformations; 2) deformation due to shear in columns and joints; and 3) deformations due to axial force in columns.

Yielding and plastic hinge formation in frame members can significantly increase the lateral displacement. The effect of secondary moments caused by column axial forces multiplied by lateral displacement ( $P\text{-}\Delta$  effect) will further increase the lateral displacement.

In buildings, moment-resisting frames are usually arranged parallel to the principal orthogonal axes of the structure and the frames are interconnected by floor diaphragms (Code Chapter 12). Moment-resisting frames usually allow the maximum flexibility in space planning and are an economical solution up to a certain height.

**2.3.2.2 Shear walls**—Reinforced concrete shear walls are often introduced into multi-story buildings because of their high in-plane stiffness and strength to resist lateral forces or when the building program is conducive to layout of structural walls. For buildings without a significant moment frame, shear walls behave as vertical cantilevers. Walls can be designed with or without openings (Fig. 2.3.2.2a). Separate walls can be coupled to act together by beams/slabs or deep beams, depending on design forces and architectural

requirements. Coupling of shear walls introduces frame action to the LFRS and thus reduces lateral deflection of the system. Reinforced concrete walls are often used around elevator and stair shafts to achieve the required fire rating. For shear wall types and functions, refer to Table 2.3.2.2.

A shear wall building usually consists of a series of parallel shear walls in orthogonal directions that resists lateral loads and supports vertical loads.

In multi-story bearing wall buildings, significant discontinuities in mass, stiffness, and geometry should be avoided. Bearing walls should be located close to the plan perimeter if possible and should preferably be symmetric in plan to reduce torsional effects from lateral loading (refer to Fig. 2.3.2.2b).

**2.3.2.3 Staggered wall-beam system**—This system uses story-high solid or pierced walls extending across the entire width of the building and supported on two lines of columns placed along exterior faces (Fig. 2.3.2.3). By staggering the locations of these wall beams on alternate floors, large clear areas are created on each floor.

The staggered wall-beam building is suitable for multi-story construction having permanent interior partitions such as apartments, hotels, and student residences.

An advantage of the wall-beam building is the large open area that can be created in the lower floors when needed for parking, commercial use, or even to allow a highway to pass under the building. This system should be considered in low seismic areas because of the stiffness discontinuity at each floor.

**2.3.2.4 Tubes**—A tube structure consists of closely spaced columns in a moment frame, generally located around the perimeter of the building (Fig. 2.3.2.4(a)).

Because tube structures generally consist of girders and columns with low span-to-depth ratios (in the range of 2 to 4), shearing deformations often contribute to lateral drift and should be included in analytical models. Tubes are often thought of as behaving like a perforated diaphragm.

Frames parallel to direction of force act like webs to carry the shear from lateral loads, while frames perpendicular to the direction of force act as flanges to carry the moment from lateral loads. Gravity loads are resisted by the exterior frames and interior columns.

A reinforced concrete braced tube is a system in which a tube is stiffened and strengthened by infilling in a diagonal pattern over the faces of the building (Fig. 2.3.2.4(b)). This

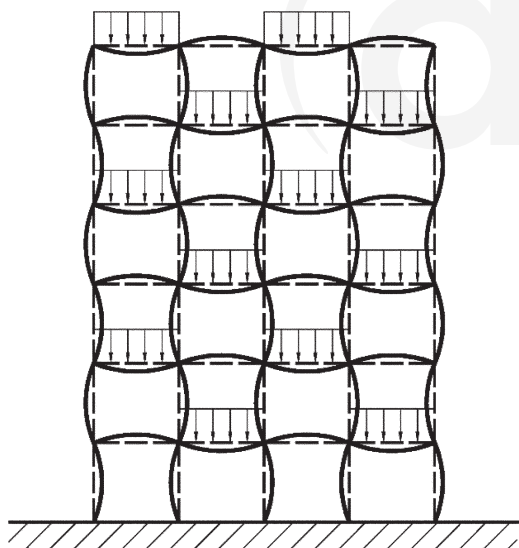


Fig. 2.3.1—Deflections due to gravity load.

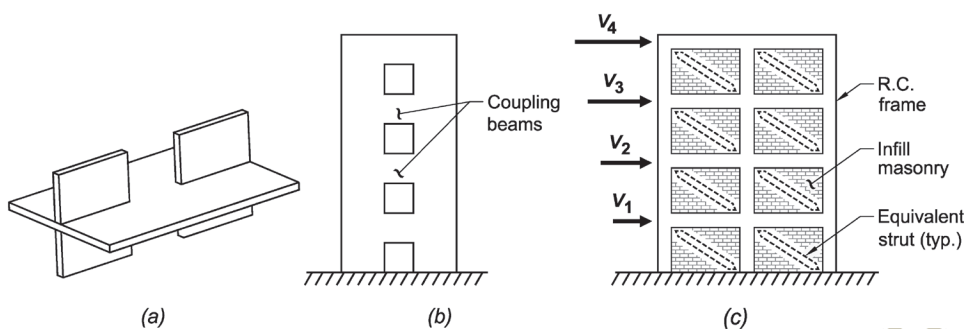


Fig. 2.3.2.2a—Coupled and infill walls: (a) shallow coupling beams or slabs; (b) coupling beams; and (c) infill walls.

**Table 2.3.2.2—Shear wall types and functions**

| Wall type                                                             | Behavior                                                                                                                                                                                              | Reinforcement                                                                                                                                                                                                                        | Remarks                                                                                                                                                                                                           |
|-----------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Short—height-to-length ratio does not exceed 2                        | Lateral design is usually concerned only with shear strength.                                                                                                                                         | Bars evenly distributed horizontally and vertically.                                                                                                                                                                                 | Wall foundation must be capable of resisting the actions generated in the wall. Consider sliding resistance provided by foundation.                                                                               |
| Height-to-length ratio is greater than 2                              | Lateral design must consider both the wall's shear and moment strength.                                                                                                                               | Evenly distributed vertical and horizontal reinforcement. Part of the vertical reinforcement may be concentrated at wall ends—boundary elements. Vertical reinforcement in the web contributes to the flexural strength of the wall. | Wall foundation must be capable of resisting the actions generated in the wall. Consider overturning resistance provided by foundation.                                                                           |
| Ductile structural wall                                               | Lateral design is heavily influenced by flexure stiffness and strength.                                                                                                                               | Flexural bar spacing and size should be small enough so that flexural cracking is limited if yielding occurs. Over-reinforcing for flexure is discouraged because flexural yielding is preferred over shear failure.                 | Acceptable ductility can be obtained with proper attention to axial load level, confinement of concrete, splicing of reinforcement, treatment of construction joints, and prevention of out-of-plane buckling.    |
| Coupled walls with shallow coupling beams or slabs (Fig. 2.3.2.2a(a)) | Link slab flexural stiffness deteriorates quickly during inelastic reversed loading.                                                                                                                  | Place coupling slab bars to limit slab cracking at the stress concentrations at the wall ends.                                                                                                                                       | Punching shear stress around the wall ends in the slab needs to be checked.                                                                                                                                       |
| Coupled walls with coupling beams (Fig. 2.3.2.2a(b))                  | Depending on span-to-depth ratio, link beams may be designed as deep beams.                                                                                                                           | Main reinforcement placed horizontally or diagonally. Diagonal reinforcement is placed corner to corner of the beam and may be confined by spirals or closed ties and designed to resist flexure and shear directly.                 | Properly detailed coupling beams can achieve ductility. Coupling beams should maintain their load-carrying capacity under reverse inelastic deformation.                                                          |
| Infilled frames (structural or nonstructural) (Fig. 2.3.2.2a(c))      | Frames behave as braced frames, increasing the lateral strength and stiffness. The infilling acts as a strut between diagonally opposite frame corners, and creates high shear forces in the columns. | Infill walls should either be sufficiently separated from the moment frame (making them nonstructural), or detailed to be connected structurally with the moment frame.                                                              | Uneven infilling can cause irregularities of the moment frame. If there are no infills at a given story level, that story acts as a weak or soft story that is vulnerable to concentrated damage and instability. |

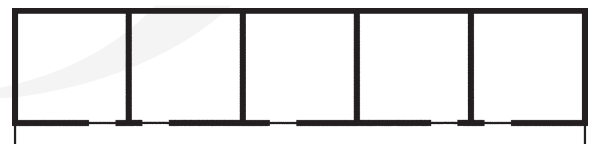
bracing increases the structure's lateral stiffness, reduces the moments in the columns and girders, and reduces the effects of shear lag.

**2.3.3 Dual systems**—Dual systems consist of combined shear walls and moment-resisting frames. They are used to achieve specific response characteristics, particularly with respect to seismic behavior. Some of the more common dual systems are discussed in Sections 2.3.3.1 through 2.3.3.6.

**2.3.3.1 Wall-frame systems**—Rigid-jointed frames and isolated or coupled structural walls can be combined to produce an efficient LFRS. Because of the different shear and flexural lateral deflection characteristics of moment frames and structural walls, careful attention to the interaction between the two systems can improve the structure's lateral response to loads by reducing lateral deflections (Fig. 2.3.3.1).

The wall's overturning moment is greatly reduced by interaction with the frame. Because drift compatibility is forced on both the frame and the wall, and the frame-alone and wall-alone drift modes are different, the building's overall lateral stiffness is increased. Design of the frame columns for gravity loads is also simplified in such cases, as the frame columns are assumed to be braced against side-sway by the walls.

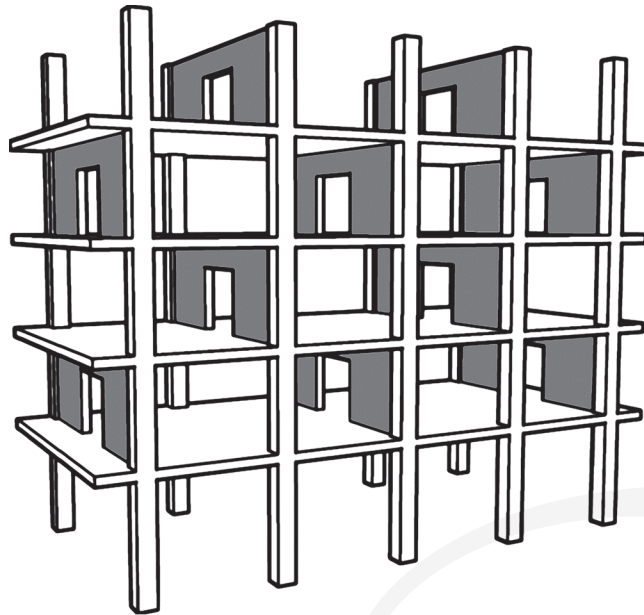
The wall-frame dual system permits the structure to be designed for a desired yielding sequence under strong ground motion. Beams can be designed to experience signif-

*(a) No openings in walls; access from outside**(b) Coupled walls**(c) Longitudinal corridor walls***Fig. 2.3.2.2b—Example of shear wall layouts.**

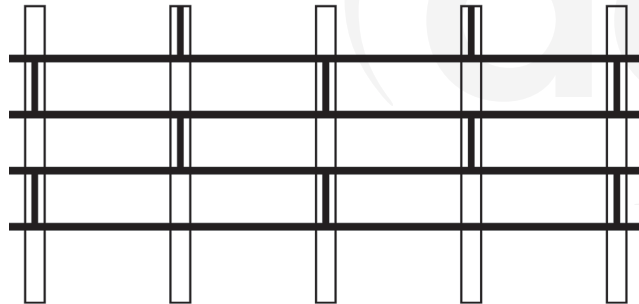
icantly yielding before inelastic action occurs at the bases of the walls. By creating a hinge sequence, and considering the relative economy with which yielded beams can be repaired, wall-frame structures are appropriate for use in higher seismic zones. However, note that

of shears and overturning moments over the height of the wall and frame is very different under inelastic versus elastic response conditions.

**2.3.3.2 Outrigger systems**—An outrigger system uses orthogonal walls, girders, or trusses, one or two stories in



*Perspective*



*Longitudinal section*

Fig. 2.3.2.3—Staggered wall-beam system.

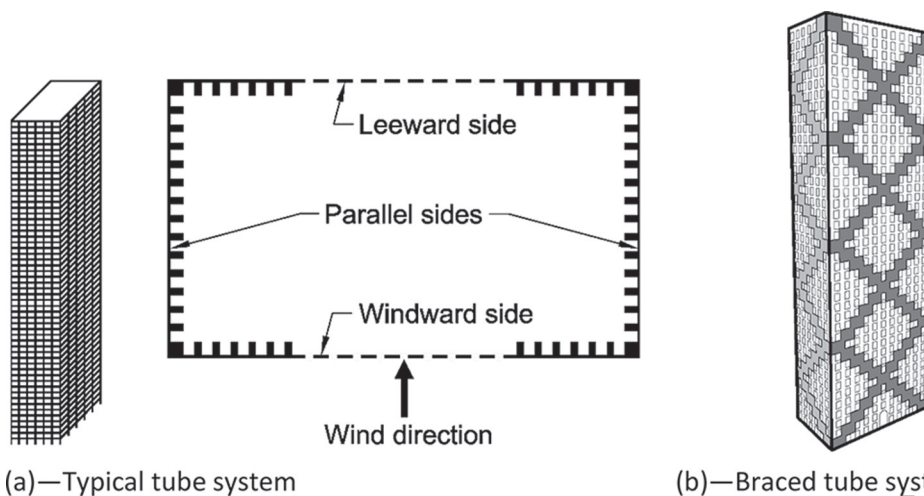
height, to connect the perimeter columns to central core walls, thus enhancing the structure lateral stiffness (Fig. 2.3.3.2).

In addition to the outrigger girders that extend out from the core, girders or trusses are placed around the perimeter of the structure at the outrigger levels to help distribute lateral forces between the perimeter columns and the core walls. These perimeter girders or trusses are called “hat” or “top-hat” bracing if located at the top, and “belt” bracing if located at intermediate levels. Some further reductions in total drift and core bending moments can be achieved by increasing the cross section of the columns and, therefore, the axial stiffness, and by adding outriggers at more levels. Outriggers are effective in increasing overall building stiffness and, thus, resist wind loads with less drift. Design of outrigger-type systems for SDC D through F must consider the effect of the high local stiffness of the outriggers on the inelastic response of the entire system. Members framing into the outriggers should be detailed for ductile response.

**2.3.3.3 Tube-in-tube**—For tall buildings with a reasonably large service core, it is generally advantageous to use shear walls enclosing the entire service core (inner tube) as part of the LFRS. The outer tube is formed by the closely spaced column-spandrel beam frame. A bundled tube system consists of several framed tubes bundled into one larger structure that behaves as a multicell perforated box (Fig. 2.3.3.3).

The tube-in-tube system combines the advantages of both the perimeter-framed tube and the inner shear walls. The inner shear walls enhance the structural characteristics of the perimeter-framed tube by reducing the shear deformation of the columns in the framed tube. The tube-in-tube system can be considered a refined version of the shear wall-frame interaction type structure.

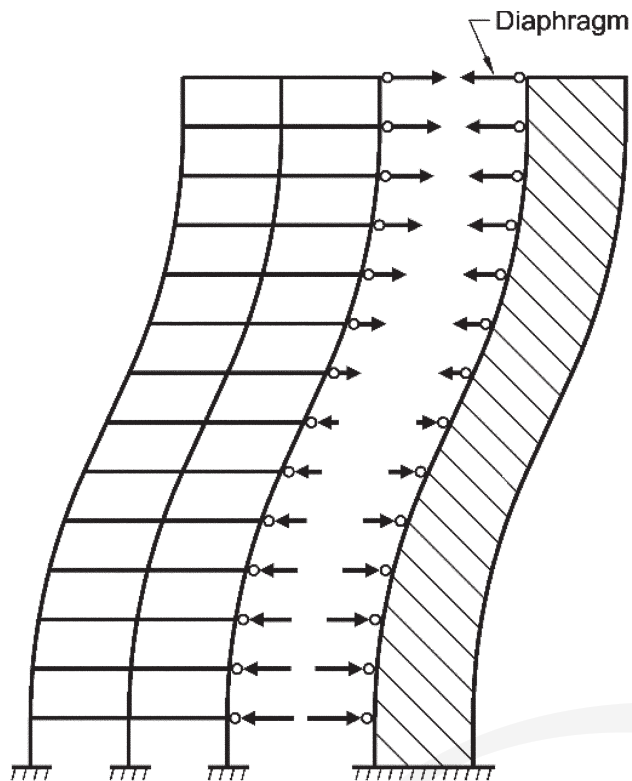
**2.3.3.4 Bundled tubes**—A bundled tube system consists of several framed tubes bundled into one larger structure that behaves as a multicell perforated box. Individual tubes can be terminated at different heights. The bundled tube system offers considerable flexibility in layout and possesses large torsional and flexural stiffness.



(a)—Typical tube system

(b)—Braced tube system

Fig. 2.3.2.4—Tube systems.



Frame and shear wall connected by floor diaphragm (equal lateral deflection at each level)

Fig. 2.3.3.1—Shear wall and moment frame system.

**2.3.3.5 Mixed concrete-steel structures**—Mixed concrete-steel systems consist of interacting concrete and steel assemblies. The resulting composite structure displays most or all of the advantages of steel structures (large spans and lightweight construction) as well as the favorable characteristics of concrete structures (high lateral stiffness of shear walls and cores, and high damping). Engineers must address the differential vertical creep and shrinkage between steel and concrete to prevent uneven displacement. Because the erection of steel and concrete structures involves different building trades and equipment, engineers who design mixed construction should consider scheduling issues.

**2.3.3.6 Precast structures**—Precast concrete members are widely used as components in frame, wall, and wall-frame systems. Mixed construction, consisting of precast concrete assemblies connected to a cast-in-place concrete core, is also used. The efficiency of such systems depends on the extent of standardization, the ease of manufacture, the simplicity of assembly, and the speed of erection.

Precast floor systems include large standardized reinforced (and usually prestressed) concrete slabs, with or without interior cylindrical voids (hollow core), as well as prefabricated rib slabs. Rigid-jointed frames are usually assembled from H- or T-units, and shear walls and cores are assembled from prefabricated single-story panels. Planning and designing appropriate connection details for panels, frame members, and floor assemblies is the single most important operation related to precast systems.

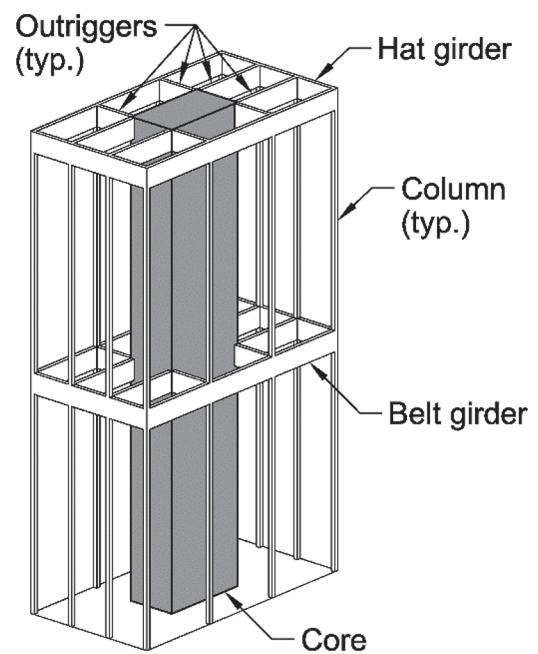


Fig. 2.3.3.2—Outrigger system.

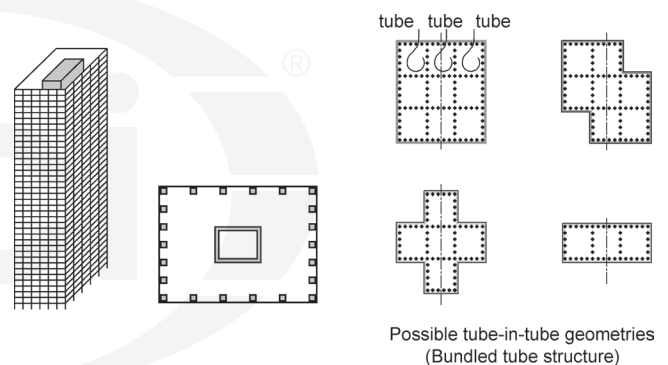


Fig. 2.3.3.3—Tube-in-tube and bundled tube systems.

Three main types of connections are described as follows:

1. Steel reinforcing bars protruding from adjacent precast members are made continuous by mechanical connectors, welding, or lap splices, and the joint between the members is filled with cast-in-place concrete. If welding is used, the engineer should specify weldable reinforcement and appropriate reinforcement material and welding procedures to ensure connections with suitable ductility.

2. Steel inserts (plates and angles) cast into the precast members are bolted or welded together and the gaps are grouted.

3. The individual precast units are post-tensioned together across the joint, with or without a mortar bed.

The behavior of a precast system subjected to seismic loading depends to a considerable degree on the characteristics of the connections. Connection details can be developed that ensure satisfactory performance under seismic loadings, provided that the engineer pays particular attention to steel ductility and positive confinement of concrete in the joint area.

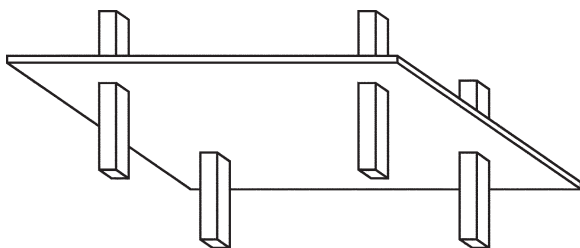


Fig. 2.4.1—Two-way flat plate system.

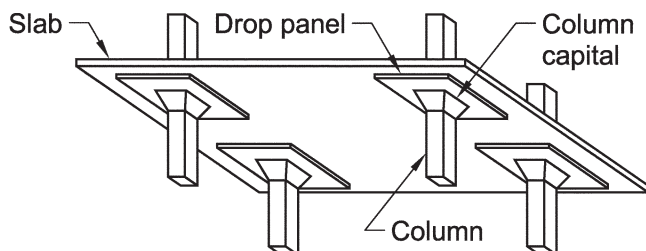


Fig. 2.4.2—Flat slab with drop panels and capitals.

## 2.4—Floor framing systems

Selection of the floor system significantly affects a structure's cost as well as the performance of its lateral-force-resisting system. The primary function of a floor system is to resist gravity load. Additional important functions in most buildings are:

(a) Diaphragm action: The slab's in-plane stiffness maintains the plan shape of the structure and distributes horizontal forces to the LFRS.

(b) Moment resistance: The flexural stiffness of the floors may be an integral and necessary part of the LFRS.

Concrete structures are commonly analyzed for lateral loads assuming the floor system acts as a diaphragm, infinitely stiff in its plane. This assumption is not valid for all configurations and geometries of floor systems. Factors affecting diaphragm stiffness are span-to-depth ratio of the slab's plan dimensions relative to the location of the lateral-load-resisting members; slab thickness; locations of slab openings and discontinuities; and type of floor system used. The floor system flexural stiffness can add to the lateral stiffness of the structure. If the slab is assumed to act as part of a frame to resist lateral moments, engineers usually limit the effective slab width (acting as a beam within the frame) to between 25 and 50 percent of the bay width.

**2.4.1 Flat plates**—A flat plate is a two-way slab supported by columns, without column capitals or drop panels (Fig. 2.4.1).

The flat-plate system is a very cost-effective floor for commercial and residential buildings. Simple formwork and reinforcing patterns, as well as lower overall building height, are advantages of this system. In designing and detailing plate-column connections, particular attention must be paid to the transfer of shear and unbalanced moment between the slab and the columns (Chapters 8 and 15 of the Code). This is achieved by using a sufficient slab thickness or shear reinforcement (stirrups or headed shear studs) at the slab-column joint, and by concentrating slab flexural reinforcement over the column area.

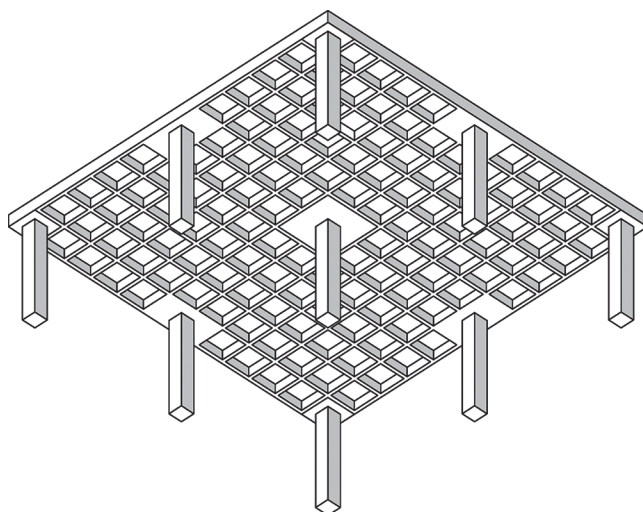


Fig. 2.4.3—Two-way grid (waffle) slab.

**2.4.2 Flat slabs with drop panels, column capitals, or both**—The shear strength of flat slabs can be improved by thickening the slab around columns with drop panels, column capitals (either constant thickness or tapered), shear caps, or a combination (Fig. 2.4.2). Like flat plates, flat slab systems normally act as diaphragms transmitting lateral forces to columns and walls.

Drop panels increase a slab's flexural and shear strength at the column, and thus improve the ability of the flat slab to participate in the LFRS. Shear caps and column capitals improve the slab shear strength by increasing the slab thickness around the column. To improve the slab shear strength without increasing the slab thickness, engineers can provide closely spaced stirrups or shear studs radiating out from the column.

LFRSs consisting only of flat slab or flat plate frames, without ductile frames, structural walls, or other bracing members, are unsuitable in high seismic areas (SDC D through F).

**2.4.3 Two-way grid (waffle) slabs**—For longer spans, a slab system consisting of a grid of ribs intersecting at a constant spacing can be used to achieve an appropriate slab depth for the longer span with much less dead load than a solid slab (Fig. 2.4.3).

The ribs are formed by standardized dome or pan forms that are closely spaced. The slab thickness between the ribs is thin and normally governed by fire rating requirements. Some pans adjacent to the columns are omitted to form a solid concrete drop panel, to satisfy requirements for transfer of shear and unbalanced moment between the slab and columns.

A waffle slab provides an adequate shear diaphragm. The solid slab adjacent to the column provides significant two-way shear strength. Slab flexural and punching shear strength can be increased by the addition of closely spaced stirrups radiating out from the column face in two directions. Stirrups may also be used in the ribs.

Because a waffle slab behaves similarly to a flat slab, LFRSs consisting only of waffle slab frames are unsuitable in high seismic design areas (SDC D through F).

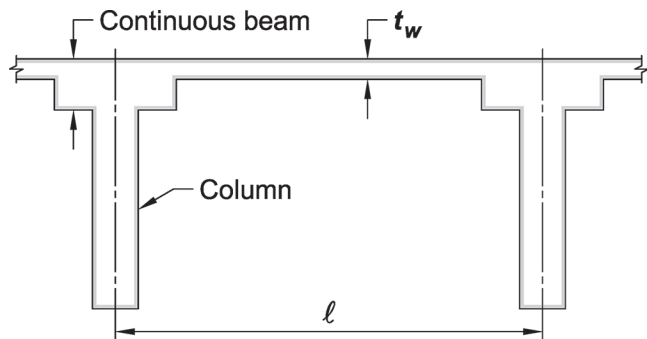


Fig. 2.4.6—One-way banded one-way slab.

**2.4.4 One-way slabs on beams and girders**—One-way slabs on beams and girders consist of girders that span between columns and beams that span between the girders. One-way slabs span between the beams. This system provides a satisfactory diaphragm, and uses the girder-column frames and beam-column frames to resist lateral loads. Adequate flexural ductility can be obtained by proper detailing of the beam and girder reinforcement.

The beams and slabs can be placed in a composite fashion (with precast elements). If composite, shear connectors are placed at the beam-slab interface to ensure composite action. This system can provide good lateral force resistance, provided that the shear connectors are detailed with sufficient strength and ductility. Some examples of this type of slab system include:

(a) Precast concrete joists with steel shear connectors between the top of the beam and a cast-in-place concrete slab. The concrete joists are usually fabricated to readily support the formwork for the cast-in-place slab. In this system, the joists are supported on walls or cast-in-place concrete beams framing directly into columns.

(b) Steel joists with the top chord embedded in a cast-in-place concrete slab. The slab formwork is supported from the joists, which supports the fresh slab concrete.

(c) Steel beams supporting a noncomposite steel deck with a cast-in-place concrete slab. Note that the Code does not govern the structural design of concrete slabs for composite steel decks.

**2.4.5 One-way ribbed slabs (joists)**—One-way ribbed slab (joist) systems consist of concrete ribs in one direction, spanning between beams, which span between columns. The size of pan forms available usually determines rib depth and spacing. As with a two-way ribbed system, the thickness of the thin slab between ribs is often determined by the building's fire rating requirements.

This system provides an adequate shear diaphragm and is used in a structure whose lateral resistance comes from a moment-resisting frame or shear walls. One row of pans can be eliminated at column lines, giving a wide, flat beam that may be used as part of the LFRS. Even if the slab system does not form part of the designated LFRS, the engineer should investigate the actions induced in the ribs by building drift.

**2.4.6 One-way banded slabs**—A one-way banded slab is a continuous drop panel (shallow beam) spanning between columns, usually in the long-span direction, and a one-way slab spanning in the perpendicular direction (Fig. 2.4.6). The

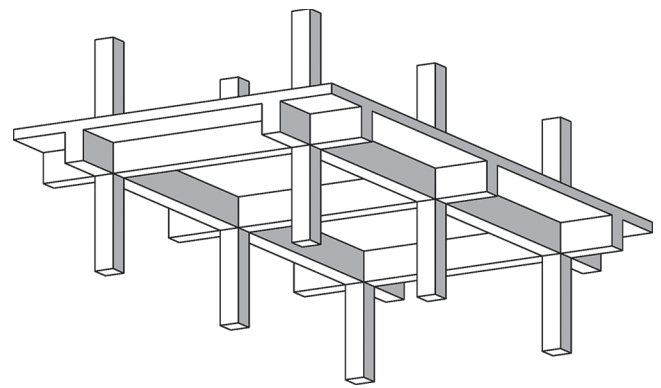


Fig. 2.4.7—Two-way slab with beams.

shallow beam can be reinforced with closely spaced stirrups near the support to increase the slab's shear strength. This system is also sometimes referred to as wide-shallow beams with one-way slabs.

A structure using this type of floor system is less stiff laterally than a structure using a ductile moment frame with beams of normal depth. LFRSs consisting only of flat slab or flat plate frames, without ductile frames, structural walls, or other bracing members, are not suitable in SDC D through F.

**2.4.7 Two-way slabs with beams**—As shown in Fig. 2.4.7, the slab is supported by beams in two directions on the column lines. This system is useful where a beam-column frame is required as part of the LFRS. The slab provides high diaphragm stiffness, and the perimeter beams can provide sufficient lateral stiffness and strength through frame action for use in SDC D through F.

For longer spans, a two-way grid (Section 2.4.3) slab with beams may be used rather than a slab with beams.

**2.4.8 Precast slabs**—Precast, one-way slabs are usually supported by bearing walls, precast beams, or cast-in-place beams. Precast slabs may be solid, hollow-core slabs, or single- or double-T sections. They are sometimes topped by a thin cast-in-place concrete layer, referred to as a "topping slab."

Welded connections are normally used to transfer in-plane shear forces between precast slabs and their supports. Because precast slabs are individual units interconnected mechanically, the ability of the assembled floor system to act as a shear diaphragm must be examined by the engineer. Boundary reinforcement may be required, particularly where the lateral-force-resisting members are far apart. In areas of high seismicity, the connections between the precast slab system and the LFRS must be carefully detailed. A concrete topping bonded to the precast slab improves the ability of the slab system to act as a shear diaphragm, and can be used in SDC D through F.

## 2.5—Foundations

Foundation design must consider the weight of the building, live loads, and the transmission of lateral forces to the ground. A distinction should be drawn between external forces, such as wind, and inertia forces that result from the building's response to ground motions during an earthquake.

External lateral forces can include static pressures due to water, earth or fill, and equivalent static forces representing the effects of wind pressures, where a gust factor or impact factor is included to account for their dynamic nature.

The soil type and strata usually dictate whether a deep or shallow foundation is required. A soils report from a licensed geotechnical engineer provides the detailed information and foundation recommendations that the licensed design professional (LDP) needs to design the foundation. For shallow footings, the geotechnical engineer provides an allowable soil bearing pressure for the soil at the foundation elevation. That pressure limit targets a certain amount of soil deflection and includes consideration of the anticipated use of the building. If allowable soil pressure is less than 2500 lb/ft<sup>2</sup>, the soil is very soft and deep foundation options are usually considered. Other soil situations, such as expansive clay or nonstructural fill, may preclude the use of shallow foundations. If the building is below grade, concrete earth retaining walls can be part of the foundation system.

The two types of deep foundations are caissons (also known as piers) and piles. If hard rock is not far below existing grade, caissons can transfer a column load directly to the bedrock. Bearing values for solid rock can be more than 10 kip/ft<sup>2</sup>. Caissons are large in diameter, usually starting at approximately 30 in. Piles are generally smaller in diameter, starting at approximately 12 in., and can be cast-in-place in drilled holes or precast piles that are driven into place. Piles are usually designed for lighter loads than caissons. Groups of piles may be used where bedrock is too deep for a caisson. Tops of piles or caissons are bridged by pile caps and grade beams to distribute column loads as needed.

Shallow foundations are referred to as footings. Types of footings are isolated, combined, and mat. Isolated rectangular or square footings are the most common types. Combined footings are often needed if columns are too close together for two isolated footings, if an exterior column is too close to the boundary line, or if columns are transmitting moments to the footing, such as if the column is part of an LFRS. If the column loads are uniformly large, such as in multi-story buildings, or if column spacing is small, mat foundations are considered.

**2.5.1 Resistance to lateral loads**—The vertical foundation pressures resulting from lateral loads are usually of short duration and constitute a small percentage of the total vertical load effects that govern long-term soil settlements. Allowing a temporary peak in vertical bearing pressures under the influence of short-term lateral loads is usually preferred to making the footing areas larger.

The geotechnical engineer should report the likelihood of liquefaction of sands or granular soils in areas with a high groundwater table, or the possibility of sudden consolidation of loose soils when subjected to jarring. The capacity of friction piles founded in soils susceptible to liquefaction or consolidation should be checked.

**2.5.2 Resistance to overturning**—The engineer should investigate the safety factor of the foundation against overturning and ensure it is within the limits of the local building code. Overturning calculations should be made with remov-

able soil fill or live load completely removed and should be based on a safe (low) estimate of the building's actual dead load.

## 2.6—Structural analysis

The analysis of concrete structures “shall satisfy compatibility of deformations and equilibrium of forces,” as stated in Section 4.5.1 of the Code. The LDP may choose any method of analysis as long as these conditions are met. This discussion is intended to be a brief overview of the analysis process as it relates to structural concrete design. For more detailed information on structural analysis, refer to Chapter 3 of this Manual.

## 2.7—Durability

Reinforced concrete structures are expected to be durable. The design of the concrete mixture proportions should consider exposure to temperature extremes, snow and ice, and ice-removing chemicals. Chapter 19 of the Code provides mixture requirements to protect concrete and reinforcement against various exposures and deterioration. Chapter 20 of the Code provides concrete cover requirements to protect reinforcement against steel corrosion. For more information, refer to Chapter 4 of this Manual.

## 2.8—Sustainability

The Code provisions for strength, serviceability, and durability are minimum requirements to achieve a safe and durable concrete structure. To improve sustainability of the structure, however, the LDP or owner are permitted to specify requirements that are more stringent than those mandated by the Code. For example, the LDP may choose to specify a higher concrete strength or design with a more restrictive deflection limit to improve service life, which is one aspect of sustainability. The strength, serviceability, and durability requirements specified in the Code, however, are required to take precedence over sustainability considerations.

For more information, the reader is directed to ACI 130R, which provides in-depth discussion on the connection between materials selection and their impact on sustainable development. Topics include efficient use of materials, life-cycle assessment, energy, and replacement materials, among others.

## 2.9—Structural integrity

Code provisions for *structural integrity* are intended to “improve the redundancy and ductility in structures so that in the event of damage to a major supporting element or an abnormal loading event, the resulting damage may be confined to a relatively small area and the structure will have a better chance to maintain overall stability” (ACI Committee 442 1971). It is currently defined in the Code as the “ability of a structure through strength, redundancy, ductility, and detailing of reinforcement to redistribute stresses and maintain overall stability if localized damage or significant overstress occurs.” The means by which the Code ensures structural integrity varies depending on the building or system type, load characteristics, and details. Table 2.9.1

shows the different member types that have specific structural integrity requirements.

Many of the Code sections quoted in the table are focused on reinforcement detailing to maintain overall stability in the event of an overload or the loss of a supporting element. These detailing provisions are relatively inexpensive and unobtrusive methods that are intended to restrict the extent of damage and provide overall stability. The

**Table 2.9.1—Structural integrity Code provisions for specific member types**

| Member type                                | Section  |
|--------------------------------------------|----------|
| Nonprestressed one-way cast-in-place slabs | 7.7.7    |
| Nonprestressed two-way slabs               | 8.7.4.2  |
| Prestressed two-way slabs                  | 8.7.5.6  |
| Nonprestressed two-way joist slabs         | 8.8.1.6  |
| Cast-in-place beam                         | 9.7.7    |
| Nonprestressed one-way joist system        | 9.8.1.6  |
| Precast joints and connections             | 16.2.1.8 |

fundamental mechanism is illustrated in Fig. 2.9a. In the event a supporting element is lost, whether through a shear failure or outright loss of a column, the flexural strength of the horizontal element will likely be exceeded by doubling of the span length. The failed element will fall on the floor below and precipitate failure of that element, resulting in a progressive collapse. Reinforcing bars that are properly anchored and spliced, however, can behave as a cable and through catenary action support the element, thus stabilizing the structure and preventing further collapse. Mitchell and Cook (1984) cite experimental evidence that this mechanism can stabilize two-way slab systems by developing a “net” of reinforcing bars that can prevent progressive collapse.

Rather than attempting to compute the loads associated with a collapse scenario, most of the Code provisions in the table call for a specific number of bars that are already in place for positive reinforcement to be continuous. The use of bottom bars is critical to the success of these provisions. For example, Code Section 9.7.7 requires that one-quarter of the positive reinforcement of interior beams be continuous, with

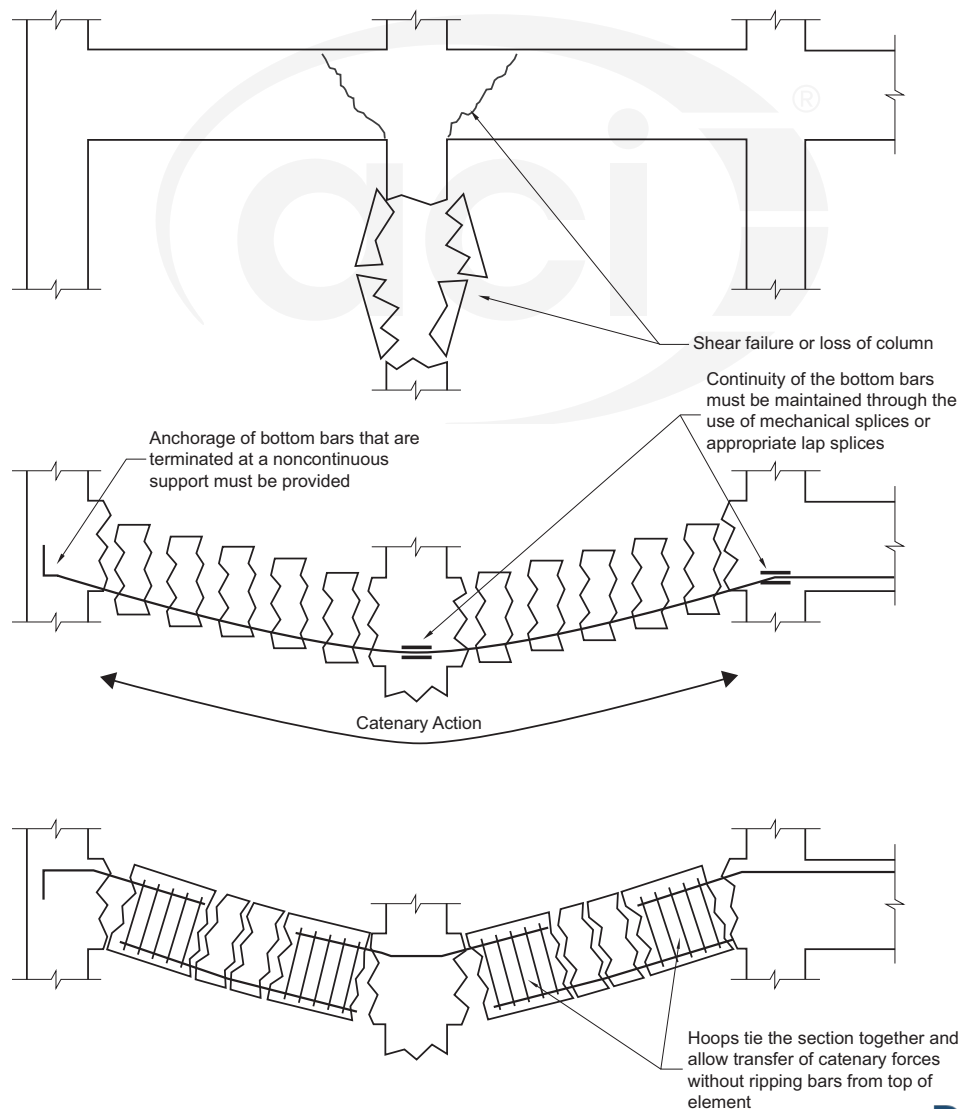


Fig. 2.9a—Role of integrity reinforcement in maintaining overall structural stability following loss of support

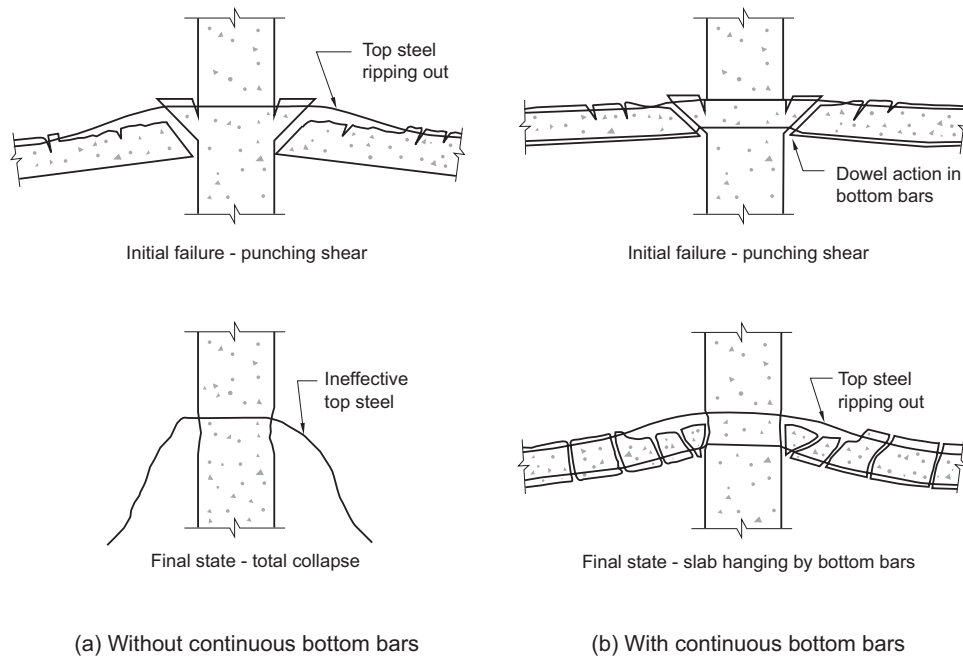


Fig. 2.9b—Catenary action of bottom flexural reinforcement after punching failure.

a minimum of two bars or strands. Alternatively, top reinforcement can be engaged by enclosing both top and bottom steel with closed stirrups in accordance with 25.7.1.6.

Figure 2.9b illustrates the effect of reliance on top bars to behave as catenary support for the horizontal element. Following a shear failure or other loss of support, the slab or beam can hang from the top bars. This will result in the top bars ripping out of the top of the slab or beam and allowing collapse to continue. As illustrated in Fig 2.9a, using bottom bars that are continuous from end to end of the structure ensures that a continuous catenary element is available at any location in the structure. Furthermore, if the continuous bars are properly anchored in the exterior supports and mechanically spliced or appropriately lap spliced in other locations, then the approach will be successful.

## 2.10—Fire resistance

Minimum cover specified in Chapter 20 of the Code is intended to protect reinforcement against fire; however, the Code does not provide a method to determine the fire rating of a member. The International Building Code (IBC) 2015 Section 722 permits calculations that determine fire ratings to be performed in accordance with ACI 216.1 for concrete, concrete masonry, and clay masonry members.

## 2.11—Post-tensioned/prestressed construction

The introduction of post-tensioning/prestressing to concrete floor, beams, and wall elements imparts an active, permanent force within the structural system. Because cast-in-place structural systems are monolithic, this force affects the behavior of the entire system. The engineer should consider how elastic and plastic deformations, deflections, changes in length, and rotations due to post-tensioning/prestressing affect the entire system. Special attention must be given to the connection of post-tensioned/prestressed

members to other members to ensure the proper transfer of forces between, and maintain a continuous load path. Because the post-tensioning/prestressing force is permanent, the system creep and shrinkage effects require attention.

## 2.12—Quality assurance, construction, and inspection

Code Section 4.13 requires that the specifications used for the execution of construction must be prepared in accordance with the provisions in Chapter 26. In addition, inspection must be conducted in accordance with the provisions in Chapter 26 and the general building code. These two general provisions, along with the more specific provisions provided in Chapter 26, establish the minimum level of quality. In the context of the Code, quality means that mechanical and durability properties of the construction materials meet or exceed either values explicitly assumed by the designer or values that are implicit in the Code provisions. Quality also denotes the level of workmanship used in assembling the materials into their final form. Chapter 26 also requires verification, typically through inspection, that the Work was constructed in accordance with the construction documents and thus meets the intent of the structural design.

At first glance, Chapter 26 may appear to apply directly to contractors. As indicated in the scope of the chapter, however, it is “directed to the LDP responsible for incorporating project requirements into the construction documents.” Construction documents, which contain written and graphic instructions for the construction of the Work, are prepared by the LDP to ensure that the structural system meets the structural system requirements of Code Chapter 4. As such, the LDP should neither expect nor require the Contractor to read and interpret the Code. Construction documents should be free of general or specific references to the Code. Such provisions as are necessary to ensure compli-

ance with the Code should be interpreted by the designer and included in the construction documents. For instance, use of material standards published by ASTM International should be included in the construction documents to ensure that materials used by the contractor are manufactured correctly. Other standards, such as ACI 301, which was written to be consistent with the requirements of the Code, can be used by the designer to aid in preparation of their construction specifications. ACI 301 is a reference specification that the designer can apply by citing it in the construction documents. A mandatory requirements checklist and an optional requirements checklist are provided to assist the designer in implementing this Specification.

Chapter 26 also provides the LDP with minimum information that should be included in the construction documents such as information developed in the structural design that must be conveyed to the contractor, provisions directing the contractor on required quality, and inspection requirements to verify compliance with the construction documents.

## REFERENCES

### *American Concrete Institute*

ACI 130R-19—Report on the Role of Materials in Sustainable Concrete Construction

ACI 216.1-14—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

ACI 301-16—Specifications for Structural Concrete

ACI 352R-02—Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures

ACI 442R-71—Response of Buildings to Lateral Forces

ACI 442R-88—Response of Concrete Buildings to Lateral Forces

### *American Society of Civil Engineers*

ASCE/SEI 7-16—Minimum Design Loads for Buildings and Other Structures

### *International Code Council*

IBC 2015—International Building Code

## Authored documents

Ali, M. M., and Moon, K. S., 2007, “Structural Development in Tall Buildings: Current Trends and Future Prospects,” *Architectural Science Review*, V. 50, No. 3, pp. 205-223.

Mitchell, D., and Cook, W. D., 1984, “Preventing Progressive Collapse of Slab Structures,” *Journal of Structural Engineering*, ASCE, V. 110, No. 7, pp. 1513-1532.



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# CHAPTER 3—STRUCTURAL ANALYSIS

## 3.1—Introduction

Structural engineers mathematically model reinforced concrete structures, in part or in whole, to calculate member moments, forces, and displacements under the design loads that are specified by a standard such as ASCE/SEI 7. In all conditions, equilibrium of forces and compatibility of deformations must be maintained. The stiffness values of individual members for input into the model, under both service loads and factored loads, are discussed in detail in Code Chapter 6. The factored moments and forces resulting from the analysis are used to determine the required strengths for individual members. The calculated displacements and drift are also checked against commonly accepted serviceability limits.

## 3.2—Overview of structural analysis

**3.2.1 General**—The analysis of concrete structures “shall satisfy compatibility of deformations and equilibrium of forces,” as stated in Section 4.5.1 of the Code. The licensed design professional (LDP) may choose any method of analysis as long as these conditions are met. The Code specifically recognizes four methods of analysis. Section 6.6 addresses the most common, which is linear elastic first-order analysis. Section 6.7 covers linear elastic second-order analysis, which is typically used to calculate slenderness effects. Section 6.8 addresses inelastic analysis, both first- and second-order. In addition, the Code permits the use of strut-and-tie modeling for the analysis of discontinuous regions.

Except as noted in Code Chapter 18, structural concrete members are typically assumed to be prismatic and behave elastically under design loads. Although these assumptions result in behavior that differs from the actual behavior of the concrete member, they provide a reasonable estimate of the distribution of the demands on the members. The Commentary suggests that both serviceability and strength requirements can be addressed by conducting separate analyses with varying stiffness assumptions to bound the actual solution, especially where stiffness can significantly affect results.

**3.2.2 Elastic analysis**—Load effects in most concrete structures are determined using elastic analysis techniques that are implemented with computer software. If a first-order analysis is used, then Code Section 6.6.1.1 indicates that slenderness effects should be considered by using the moment magnification approach covered in Section 6.6.4. In this approach, moments obtained from a first-order frame analysis are multiplied by a moment magnifier that is based on the axial load and buckling strength of the column. Frames that are braced against sway are treated differently from those that are not. Alternatively, a second-order analysis can be used to determine the slenderness effects directly by considering the loads applied to the deformed structure. This approach is typically implemented with computer software and involves iterative analyses where the stiffness matrix is reformulated as the geometry of the structure changes under load. A valid second-order analysis requires that the structure be in equilibrium in the final deformed shape.

**3.2.3 Inelastic analysis**—Inelastic analysis considers material nonlinearities such as concrete cracking and steel yielding. As with elastic analyses, a first-order inelastic analysis must satisfy equilibrium in the undeformed (original) geometry. A second-order inelastic analysis must satisfy equilibrium in the deformed configuration. Code Commentary R6.8.1.1 suggests that material nonlinearities may be affected by duration of loads, shrinkage, and creep. The Code indicates that inelastic analysis procedures should be validated by comparison of strength and deformation results with that of physical tests. Inelastic analyses are typically used for “push-over” analysis, which is used in seismic retrofit of existing buildings; design of materials and

**Table 3.2.5—Common analysis types and tools**

| Analysis type                                                                 | Applicable member or assembly    | Analysis tool                                                                         |
|-------------------------------------------------------------------------------|----------------------------------|---------------------------------------------------------------------------------------|
| First-order<br>Linear elastic<br>Static load<br>Hand calculations             | One-way slab                     | Analysis tables*                                                                      |
|                                                                               | Continuous one-way slab          | Simplified method in Section 6.5 of the Code                                          |
|                                                                               | Two-way slab                     | Direct design method in Section 8.10 of ACI 318-14†                                   |
|                                                                               |                                  | Equivalent frame method in Section 8.11 of ACI 318-14†                                |
|                                                                               | Beam                             | Analysis tables*                                                                      |
|                                                                               | Continuous beam                  | Simplified method in Section 6.5 of the Code                                          |
|                                                                               | Column                           | Interaction diagrams*                                                                 |
|                                                                               | Wall                             | Interaction diagrams*                                                                 |
|                                                                               |                                  | Alternative method for out-of-plane slender wall analysis in Section 11.8 of the Code |
|                                                                               | Two-dimensional frame            | Portal method in Section 3.3.3 of this chapter                                        |
| First-order<br>Linear elastic<br>Static load<br>Computer programs             | Gravity-only systems             | Spreadsheet program based on the analysis tools for hand calculations above           |
|                                                                               |                                  | Program based on matrix methods but only analyze floor assemblies for gravity loads   |
|                                                                               | Two-dimensional frames and walls | Program based on matrix methods without iterative capability                          |
| Second-order<br>Linear elastic<br>Static or dynamic load<br>Computer programs | Two-dimensional frames and walls | Programs based on matrix methods with iterative capability                            |
|                                                                               | Three-dimensional structure      | Programs based on finite element methods with iterative capability                    |
| Second-order<br>Inelastic                                                     | Three-dimensional structure      | Beyond the scope of this Manual                                                       |

\*Information can be downloaded from the ACI website [www.concrete.org/publications](http://www.concrete.org/publications).

†Direct design method and equivalent frame method are not included in ACI 318-19, but are still permitted.

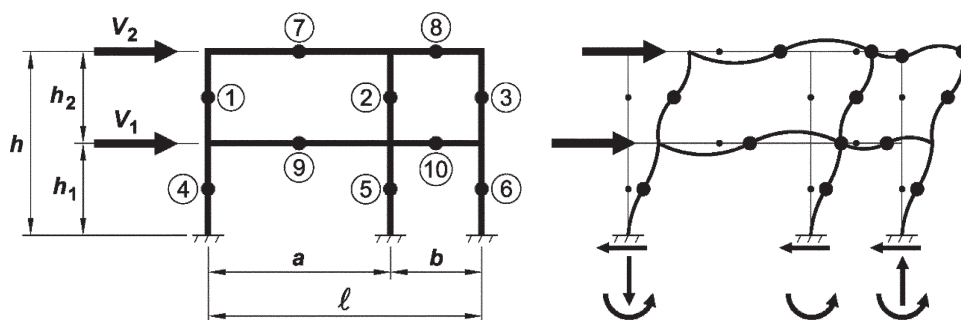


Fig. 3.3.3—Frame analyzed by portal method.

systems not covered by the Code; and evaluation of building performance above Code minimum requirements (Deierlein et al. 2010). This Manual does not include any examples of inelastic analyses.

**3.2.4 Strut-and-tie**—The strut-and-tie method in Code Chapter 23 is another analysis method that is permitted by the Code. This method does not assume that plane sections of unloaded members remain plane under loading. Because this method also provides design provisions, it is considered both an analysis and design method. This method is applicable where the sectional strength assumptions in Code Chapter 22 do not apply for a discontinuity region of a member or a local area.

**3.2.5 Analysis types and tools**—The Code identifies three general types of analysis; refer to Section 3.2.1: 1) first-order linear elastic; 2) second-order linear elastic; and 3) second-order inelastic. Table 3.2.5 shows some common analysis tools used for different analysis methods, loads, and systems.

### 3.3—Hand calculations

**3.3.1 General**—Before computers became widely available, designers used simplified analysis methods to calculate gravity design moments and shears (Code Section 6.5) along with a simplified frame analysis technique such as the portal method to calculate frame moments and shears due to lateral forces. In limited applications, the design of an entire building using hand calculations is still possible with today's set of building codes. For the large majority of building designs, however, a hand calculation design approach is not practical due to the large number and complexity of design load combinations necessary to fully meet ASCE/SEI 7 requirements.

**3.3.2 Code design equations for moment and shear**—The simplified Code equations are useful for purposes of preliminary estimating or member sizing, for designing isolated members or subassemblies, and to complete rough checks of computer program output. Because these equations and expressions are easy to incorporate into electronic spreadsheets and equation solvers, they continue to be helpful. In the member chapters of this Manual, examples of hand calculations are provided.

**3.3.3 Portal method**—The portal method was commonly used before computers were readily available to calculate a frame's moments, shears, and axial forces due to lateral forces (Hibbeler 2015). This method has been virtually abandoned as a design tool with the widespread use of commercial design software programs. The portal method has limitations as stated in the assumptions and considerations

that follow but it is still a useful tool for the designer. With complex, three-dimensional modeling becoming commonplace, there is always a chance of modeling error. The portal method allows the designer to independently and quickly find approximate moments and shears in a frame. This can be useful for spot-checking the program results (Fig. 3.3.3).

The basic assumptions in the portal method are:

- (a) Apply only the lateral load to the frame
- (b) Exterior columns resist the overturning from lateral loads
- (c) Shear at each column is based on plan tributary area
- (d) Inflection points are assumed to be located at midheight of column and midspan of beams
- (e) Shear in the beam is the difference between column axial forces at a joint
- (f) Beam axial force is to be zero

These assumptions reduce a statically indeterminate problem to a statically determinate one.

The following should be considered when using this method:

- (a) Discontinuity in geometry or stiffness—such as setbacks, changes in story height, and large changes in member sizes—can cause member moments to differ significantly from those calculated by a computer analysis.
- (b) The lateral deformation will be larger than the lateral deformation calculated by a computer analysis.
- (c) Axial column deformation is ignored.

### 3.4—Computer programs

**3.4.1 General**—Computing power and structural software have advanced significantly from the time computers were introduced to the designer. Numerous complex computer programs and specialized analysis tools have been developed, taking advantage of increasing computer speeds. Currently, designers commonly use finite element analysis to design structures. A multi-story building only takes minutes of computing time on a personal computer compared to the past, when it would take several hours on a large mainframe computer. Computer software has also greatly improved: user interfaces have become more intuitive, members can be automatically meshed, and input and output data can be reviewed in graphical and in tabular form in a variety of preprogrammed or user-defined menus. Sophisticated user interfaces, however, further disconnects the designer from the analysis process, which can lead to modeling errors or incorrect interpretation of output. It is important, therefore,

for the designer to have a solid understanding of structural behavior and design and of the software being used in the analysis. This makes hand calculations discussed in the previous section an important tool that can be used to verify that the designer has used the program properly and has interpreted the output correctly.

Although three-dimensional models are becoming commonplace, many engineers still analyze the building as a series of two-dimensional frames. Matrix methods mentioned in Table 3.2.5 are programs based on the direct stiffness method. Simpler programs model the structure as discrete members connected at joints. The members can be divided into multiple elements to account for changes in member properties along its length. The two-dimensional stiffness method is a key component of many commercially available software packages and is typically used to model frame and shear wall structural systems.

For complex geometric shapes, boundary conditions, or loads, the finite element analysis (FEA) can be used. FEA involves the discretization of a member or system into multiple discrete solid elements that are typically connected together at nodes (similar to frame members). Each member of the structure consists of multiple elements. Subdividing members into an assembly of elements is called “meshing” and can be a time-consuming task. A large amount of data is generated from this type of analysis, which may be tedious for the designer to review and process. For straightforward designs, a designer may more efficiently analyze the structure by dividing it into parts and using less complicated programs to analyze each part.

**3.4.2 Two-dimensional frame modeling**—A building can be divided into parts that are analyzed separately. For example, structures are often symmetrical with regularly spaced columns in both directions. There may be a few isolated areas of the structure where columns are irregularly spaced. These columns can be designed separately for gravity load and checked for deformation compatibility when subjected to the expected overall lateral deflection of the structure.

Buildings designed as moment-resisting frames can often be effectively modeled as a series of parallel planar frames. The complete structure is modeled using orthogonal sets of crossing frames. Compatibility of vertical deflections at crossing points is not required. The geometry of beams can vary depending on the floor system. For slab-column moment frames, the direct design method or equivalent frame method may be used (Code Section 8.2.1). For beam-column moment frames, it is permitted to model T-beams, with the limits on geometry given in Section 6.3.2 of the Code; however, it is often simpler to conservatively ignore the slab and use the section properties of the stem of the T-beam in the analysis. For beams in intermediate or special moment frames, however, the assumption of a rectangular section may not be conservative; refer to Sections 18.4.2.2, 18.6.5.1, and 18.7.3.2 in the Code.

The stiffness of the beam-column joint is underestimated if the beam is assumed to span between column centerlines and the beam is modeled as prismatic along the entire span. Consequently, most computer programs adjust their analysis procedures to account for the contribution of the column stiff-

ness to the overall beam stiffness. To do this, a program may add a rigid zone that extends from the face of the column to the column centerline. If the program does not provide this option, the designer can increase the beam stiffness in the column region 10 to 20 percent to account for this change of rigidity (ACI 442-71). Walls with aspect ratios of total height-to-width greater than 2 can sometimes be modeled as column elements. A thin wall may be too slender for a conventional column analysis, and a more detailed evaluation of the boundary elements and panels may be necessary. Where a beam frames into a wall that is modeled as a column element, a rigid link should be provided between the edge of the wall and centerline of the wall.

Wall elements can be modeled as either a frame element that is very stiff or, if the program has the capability, the wall can be modeled using FEA. Where a finite beam element frames into a finite wall element, rotational compatibility should be assured. Walls with openings can be more difficult to analyze with frame elements because the rigidity of the joints near the openings must be carefully considered (Fig. 3.4.2a(a)). Similar to the beam-column joint modeling discussed previously, a rigid link should be modeled from the centerline of the wall to the edge of the opening (Fig. 3.4.2a(b)). Alternatively, FEA can be used to analyze walls with openings (Fig. 3.4.2a(c)). Once the distribution of forces in the wall has been determined from FEA, the wall can be analyzed using the strut-and-tie method to check section size and reinforcement quantities and distribution.

For lateral load analysis, all the parallel plane frames in a building are linked into one plane frame to enforce lateral deformation compatibility. Alternately, two identical frames can be modeled as one frame with doubled stiffness; one method to double the stiffness is to double the modulus of elasticity in the concrete material model. Structural walls, if present, should be linked to the frames at each floor level (Fig. 3.4.2b). Note that torsional effects should be considered after the lateral deformation compatibility analysis is run. For seismic loads, accidental torsion must be considered when the diaphragm is rigid (ASCE/SEI 7 Section 12.8.4.2). For wind loads, a torsional moment should be applied according to Fig. 27.3-8 in ASCE/SEI 7.

**3.4.3 Three-dimensional modeling**—A three-dimensional model allows the designer to observe structural behavior that a two-dimensional model would not reveal. The effects of structural irregularities and torsional response can be directly analyzed. Current computer software that provides three-dimensional modeling are capable of running a modal response spectrum analysis, seismic response history procedures, and can perform a host of other time-consuming mathematical tasks.

To reduce computation time, concrete floors are sometimes modeled as rigid diaphragms, reducing the number of dynamic degrees of freedom to only three per floor (two horizontal translations and one rotation about a vertical axis). ASCE/SEI 7 allows for diaphragms to be modeled as rigid if following conditions are met:

(a) For seismic loading, no structural irregularities and the span-to-depth ratios are 3 or less (Section 12.8.4.2 of ASCE/SEI 7)

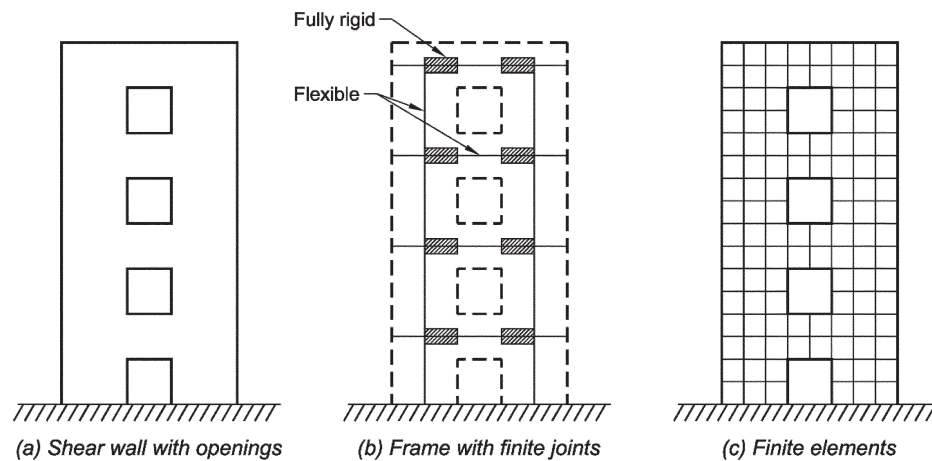


Fig. 3.4.2a—Element and frame analogies.

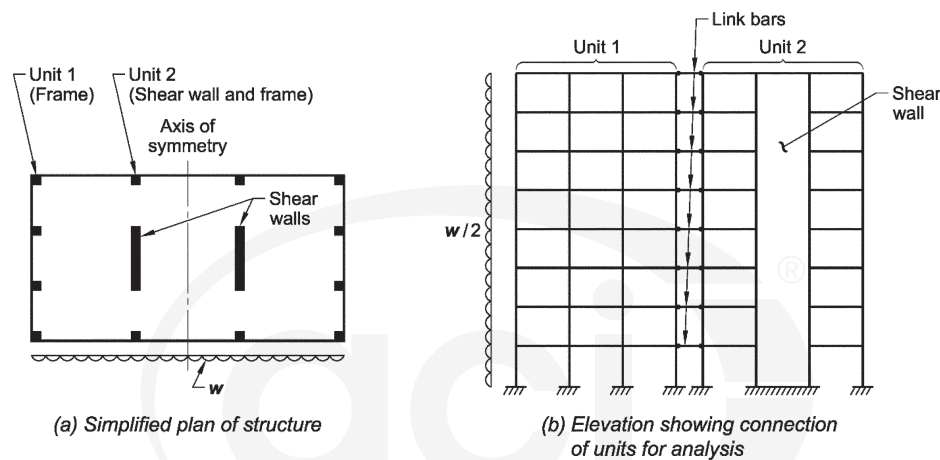


Fig. 3.4.2b—Idealization for plane frame analysis.

(b) For wind loading, the span-to-depth ratios are 2 or less (Section 27.4.5 in ASCE/SEI 7)

If a rigid diaphragm is assumed, the stresses in the diaphragm are not calculated and need to be derived from the reactions in the walls above and below the floor. A semi-rigid diaphragm requires more computation power but provides a distribution of lateral forces and calculates slab stresses. A semi-rigid diaphragm can also be helpful in analyzing torsion effects. For more information on torsion, refer to ASCE/SEI 7 Section 12.8.

### 3.5—Structural analysis in ACI 318

**3.5.1 Arrangement of live loads**—Section 4.3.3 in ASCE/SEI 7 states that the “full intensity of the appropriately reduced live load applied only to a portion of a structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member.” This is a general requirement that acknowledges greater moments and shears may occur with a pattern load than with a uniform load. There have been a variety of methods used to meet this requirement. Cast-in-place concrete is inherently continuous, and Section 6.4 in the Code provides acceptable arrangements of pattern live load for continuous one-way and two-way floor systems.

**3.5.2 Simplified method of analysis for nonprestressed continuous beams and one-way slabs**—Section 6.5 in the

Code provides approximate equations for conservative design moments and shears, which greatly simplifies the design of continuous floor members. This method is probably used more often to estimate initial member sizes for computer input, or for initial cost estimates, than for final design.

**3.5.3 First-order analysis**—Code requirements for first-order analysis are provided in Section 6.6 in the Code.

**3.5.3.1 Section properties**—Section properties for elastic analysis are given in Table 6.6.3.1.1(a) of the Code. The moment of inertia values have a stiffness reduction  $\phi_k$  of 0.875 already applied. These properties are acceptable for the analysis of the structure for strength design. For service level load analysis, the moment of inertia values in Table 6.6.3.1.1(a) can be multiplied by 1.4. Table 6.6.3.1.1(b) offers a more accurate estimation of stiffness by including the effects of axial load, eccentricity, reinforcement ratio, and concrete compressive strength. These equations can also be used to calculate member stiffness at factored load levels by using the factored axial load and moment, as presented, but the equations can be used to calculate member stiffness for any given axial load and moment. These moment-of-inertia equations also have the 0.875 stiffness reduction  $\phi_k$  already applied. Section 6.6.3.1.2 of the Code also allows a simplification of using  $0.5I_g$  for all members in a factored lateral load analysis. This is helpful for hand-calculation methods such as the portal method.

It is important to note that the stiffness reduction factor used for moment of inertia discussed previously is for global building behavior. The moment of inertia for second-order effects related to an individual column or wall should have a stiffness reduction  $\phi_k$  of 0.75, as discussed in R6.6.4.5.2 of the Code.

**3.5.3.2 Slenderness effects**—A first-order analysis in the Code assumes that only primary stresses are calculated. Secondary stresses caused by the lateral deflection caused by the design loads are not calculated. First-order analysis is typical when hand-calculation methods are used or basic matrix analysis computer programs are used that are not programmed for iterative analysis.

This method ignores both  $P$ - $\Delta$  and  $P$ - $\delta$  effects, which are the second-order moments caused by axial loads acting on the deformed geometry (Fig. 3.5.3.2).  $P$ - $\Delta$  is typically considered the second-order moment due to sidesway of the structure in which  $\Delta$  is the lateral relative story drift.  $P$ - $\delta$  is the second-order moment due to the flexural deformation,  $\delta$ , of the column relative to its supporting ends. To approximately account for these secondary effects, a moment magnifier is applied to first-order column design moments.

The designer must account for slenderness in a first-order analysis. Figure R6.2.5.3 in the Code provides a flowchart that illustrates the options to account for slenderness. In summary, slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in the Code. If slenderness cannot be neglected, the next step is to determine if the building story being analyzed is braced against sidesway or unbraced. If the story is braced, the column or wall end moments are magnified for moment effects along the member ( $P$ - $\delta$ ). If the story is unbraced, the column or wall end moments are magnified for moment effects along the member ( $P$ - $\delta$ ) and at the ends due to story drift ( $P$ - $\Delta$ ).

**3.5.3.3 Superposition**—Linear analysis allows for superposition to be used when combining loads. This is helpful when performing hand calculations. The designer calculates the member moment, shear, and axial load for each load. The reactions are then superimposed according to the applicable load combination. Many hand-calculation tools, such as the moment magnification method, assume that the designer is performing a linear analysis with superposition of multiple load effects.

**3.5.3.4 Redistribution of moments**—The Code allows for the designer to adjust design slab and beam moments and shears by taking advantage of the ductility provided through Code detailing requirements. Ductile detailing is required for continuous members at supports and midspan. Moment redistribution can be helpful in creating economical designs. For example, in a final design, moment redistribution may permit the designer to specify uniform beam sizes over multiple beam spans. If column spacing is not uniform, some beam design moments may be slightly lower than the beam required moments. Once steel yielding has developed at factored loads, however, additional moments will be carried by regions that have not yet yielded, and the beam design moments will satisfy the beam required moments throughout the multiple beam spans.

**3.5.4 Second-order analysis**—Code Section 6.7 indicates that a second-order analysis must consider the effect

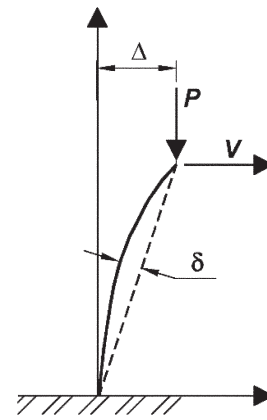


Fig. 3.5.3.2— $P$ - $\Delta$  effects.

of loads on the laterally deformed structure. The initial  $P$ - $\Delta$  effects on the member due to story drift are computed. A computer algorithm then automatically carries out a series of iterative analyses using the new deflection values until the solution converges to the final secondary moments. Note that linear material properties are used with this method, but the results of a second-order analysis is a nonlinear solution. This is referred to as “geometric nonlinearity.” This means that the load cases cannot be computed separately and then combined for the calculation of the secondary moments.

Software should be checked to determine if it accounts for both  $P$ - $\Delta$  and  $P$ - $\delta$  effects. Software can easily calculate the additional moment due to building lateral deformation but some software does not calculate the secondary moments along the member length. The designer may have to model the column as at least two segments to capture this effect. Even though a column member is modeled by two elements, the designer must account for the smaller stiffness reduction factor for the moment of inertia (refer to Section 3.5.3.1) because deflection along the member is a local effect. Because of the difficulty of appropriately capturing the secondary moment along the column length, many programs calculate the secondary moments due to  $P$ - $\Delta$  effects and then use Code Section 6.6.4.5 in a post-processing program to account for the secondary moments due to  $P$ - $\delta$  effects.

**3.5.5 Inelastic second-order analysis**—The consideration of the nonlinear behavior of structures arising from nonlinearity of the stress-strain curve for concrete and steel reinforcement, particularly under large deformations, may be important in seismic analysis. Nonlinearities in structural response, whether arising from material properties as for concrete or steel, loading conditions (for example, axial load effects on bending stiffness), or geometry (for example, second moments) are best handled by numerical iterative or step-by-step procedures. For inelastic second-order analysis, the principle of superposition should not be used. Nonlinear analysis is beyond the scope of this Manual. Several references that provide further information on nonlinear analysis are ASCE 41, FEMA 440, and Deierlein et al. (2010).

**3.5.6 Finite element analysis**—The finite element analysis of concrete structures is permitted by the Code and can be used to satisfy any of the modeling approaches in the Code as long as the element types are compatible with the response required.



## CHAPTER 4—DURABILITY

### 4.1—Introduction

Durability of structural concrete is its ability, while in service, to resist possible deterioration due to the surrounding environment, and to maintain its engineering properties. This can be accomplished by proper proportioning and selection of materials for the concrete mixture design. Other aspects influencing durability include reinforcing bar selection, detailing, and construction practices. The Code provides minimum requirements to protect the structure against early serviceability deterioration. Depending on exposure conditions, structural concrete may be required to resist chemical or physical attack,

or both. The attack mechanisms the Code covers include exposure to freezing and thawing, soil and water sulfates, wetting and drying, and reinforcement corrosion due to chlorides. All these failure mechanisms depend on transport of water through concrete. For this reason, it is essential to understand the mechanisms themselves and how different concrete-making materials, including admixtures and their proportions, influence concrete's resistance to these mechanisms.

**4.1.1 Permeability**—Permeability can be defined as “the ease with which a fluid can flow through a solid” or as “the ability of concrete to resist penetration by water or other substances (liquid, gas, or ions)” (ACI 365.1R; Kosmatka and Wilson 2011). Low-permeability concretes are more resistant to resaturation, freezing and thawing, sulfate and chloride ion penetration, and other forms of chemical attack (Kosmatka and Wilson 2011). Concrete permeability is related to porosity (volume of voids/pores in concrete) and connectivity of these pores. Out of the pores present in concrete, capillary pores in cement paste are most relevant to concrete durability, as they are responsible for the transport properties of concrete (ACI 201.2R; Kosmatka and Wilson 2011). The influence of capillary porosity in cement paste on permeability was reported by Powers (1958) (Fig. 4.1.1a).

Concrete permeability, diffusivity, and electrical conductivity can be reduced with lower water-cement ratios ( $w/c$ ), the use of supplementary cementitious materials (SCMs), and extended moist curing (Kosmatka and Wilson 2011). Effects of the  $w/c$  and duration of the moist curing on permeability is presented in Fig. 4.1.1b.

**4.1.2 Freezing and thawing**—When water freezes in concrete, it causes cement paste to dilate destructively by generating hydraulic and osmotic pressure. While hydraulic pressure forces water away from the freezing water-filled capillary cavities, osmotic pressure is produced by water entering partly frozen capillary cavities (Powers 1958). Hydraulic pressures in cement paste are generated by the 9 percent increase in volume of water when it freezes and

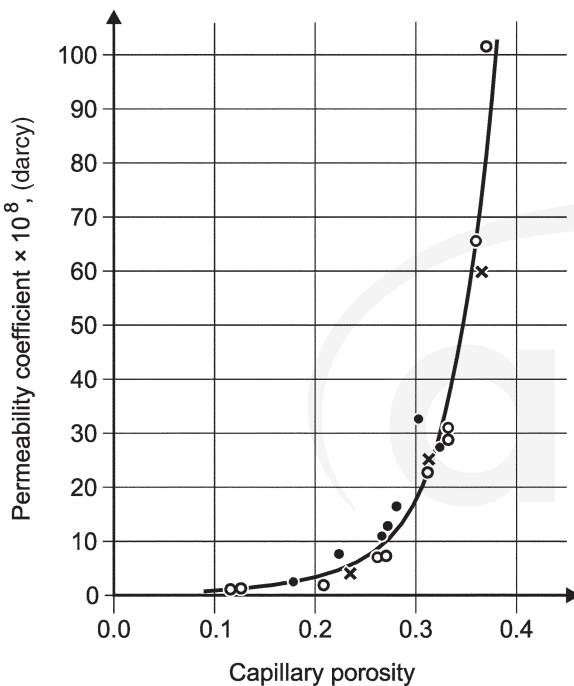


Fig. 4.1.1a—Permeability versus capillary porosity for cement paste. Different symbols designate different cement pastes (Powers 1958).

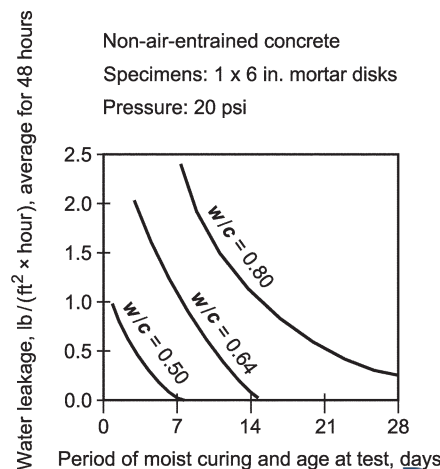
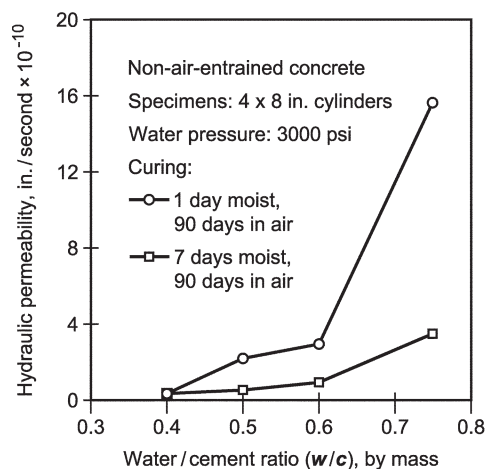


Fig. 4.1.1b—(left) Effect of  $w/c$  and initial curing on hydraulic (water) permeability; and (right) effect of  $w/c$  and curing duration on permeability (leakage) of mortar (Kosmatka and Wilson 2011).

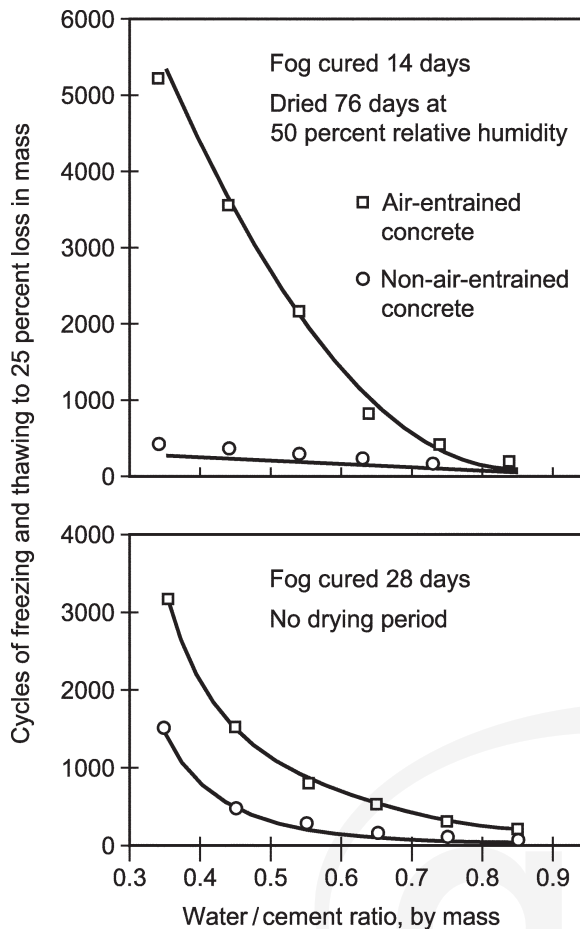


Fig. 4.1.2—Effect of w/c, air entrainment, and curing/drying on resistance to freezing and thawing of concrete (Kosmatka et al. 2008).

changes to ice. For the freezing to take place, a capillary has to reach its critical saturation: 91.7 percent filled with water (Kosmatka and Wilson 2011). Osmotic pressures develop due to differential concentrations of ionic species in the pore solution within the paste.

When pressure in concrete due to freezing exceeds the tensile strength of concrete, some damage occurs, especially if concrete is saturated with water and exposed to repeated cycles of freezing and thawing. The resulting damage is cumulative as it increases with additional repetitions of freezing and thawing cycles. Deterioration due to freezing and thawing can appear in the form of cracking, scaling, or disintegration, or all three of these (Kosmatka and Wilson 2011).

Low permeability and low absorption are main characteristics needed for concrete to be frost resistant, while air-entraining admixtures are used to control the pressure generated in concrete paste during freezing-and-thawing cycles. In other words, high resistance to freezing and thawing is associated with entrained air, low w/c, and a drying period prior to freezing-and-thawing exposure, which is demonstrated by data presented in Fig. 4.1.2.

**4.1.3 Sulfates**—Sulfates present in soil and water can react with hydrated compounds in the hardened cement paste and induce sufficient pressure to disintegrate the concrete. However, formation of new crystalline substances due to

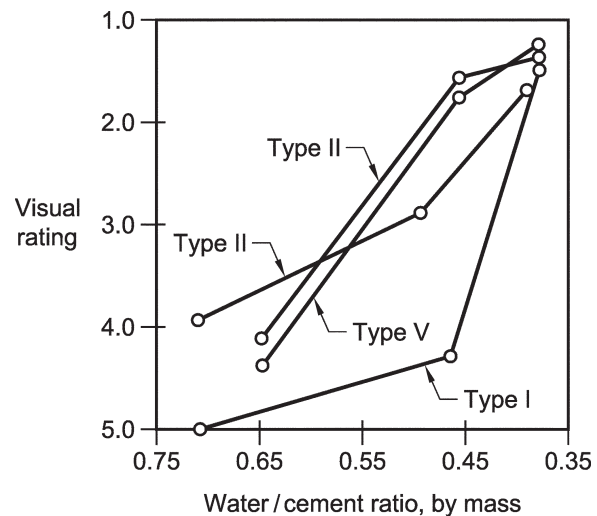


Fig. 4.1.3—Effect of w/c on sulfate resistance for different ASTM C150/C150M types of cement (lower visual rating indicates better resistance) (Stark 1989).

those reactions is partly responsible for the expansion. If water can freely diffuse out of capillaries and into the cement paste, the volume of growing crystals cannot exceed the space available to them. Additionally, the swelling pressure can arise from the diffusion of the sulfate salts into the gel pores, which disturbs the equilibrium between the gel and its surrounding liquid phase, resulting in the movement of external water into the gel pores (Hewlett 1998).

Although ordinary portland cements are most susceptible to sulfate attack, the use of sulfate-resistant cements will not necessarily prevent sulfate attack, either (Fig. 4.1.3). Resistance to sulfate attack can be greatly increased by decreasing the permeability of concrete through reduction of the water-cementitious materials ratio (w/cm) (Stark 1989).

**4.1.4 Corrosion**—The alkaline nature of concrete (pH greater than 13) will induce formation of a passive, noncorroding layer on reinforcing steel. If, however, chloride ions are present in concrete, they can reach and disrupt that layer, which can lead to corrosion of steel in the presence of water and oxygen. Once corrosion initiates, corrosion products form and may cause cracking, spalling, or delamination of concrete. This allows for easier access of aggressive agents to the steel surface and increases the rate of corrosion. Cross-sectional area of the corroding steel will decrease and the load-carrying capacity of the member will be reduced (Neville 2003).

Chlorides can be introduced to concrete with materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or through marine exposure (seawater or brackish water). To reduce likelihood of corrosion initiation, total chloride-ion content should not exceed a certain concentration value, referred to as the chloride threshold. A literature review of reported chloride threshold values revealed “that there is no single threshold value, but a range based on the conditions and materials in use” and were found to vary from 0.1 to 1 percent by mass of cement (Taylor et al. 1999). The Code limits water-soluble chlorides to 0.15 percent by mass of cement for nonprestressed concrete and 0.06 percent for

for prestressed concrete exposed to external chlorides or seawater. Value of 0.40 percent total chloride by mass of cement is given in British and European Standards (Neville 2003; Whiting et al. 2002). Corrosion of the reinforcing steel in concrete can be reduced or prevented by minimizing the  $w/cm$  (permeability), ensuring maximum cover depth of concrete over steel (Stark 2001), use of corrosion inhibitors, corrosion-resistant reinforcement, and membranes and sealers, which are popular in Europe.

## 4.2—Background

To produce durable structural concrete, concrete materials and mixture proportions are selected based on design strength requirements, anticipated exposure conditions, and required service life of the structure. The selection of materials and mixture proportions has to be accompanied by appropriate field practices, such as quality control, testing, inspection; and proper placement, finishing, and curing practices.

As stated in Section 1.3.1 of the Code, “The purpose of this Code is to provide for public health and safety by establishing minimum requirements for strength, stability, serviceability, durability, and integrity of concrete structures.” Section 4.8 of the Code addresses global durability requirements related to material selection for concrete mixtures and corrosion protection of reinforcement. Code Chapters 19 and 20 provide detailed durability requirements for concrete and reinforcing steel, respectively. Code Chapter 26 discusses what durability requirements must be specified in a project’s construction documents.

The durability section of the Code focuses mainly on concrete resistance to fluid penetration, which is primarily affected by the  $w/cm$  and the composition of the constitutive materials of concrete. The use of SCMs, such as Type F and Type C fly ashes, slag cement, silica fume, calcined shale, calcined clay or metakaolin, or their combinations, can result in a significant improvement in concrete durability. The SCMs affect concrete properties in many ways, depending on their type, dosage, and other mixture proportions and composition. In general, SCMs have the following impacts on hardened concrete properties (Kosmatka and Wilson 2011):

- Increase long-term strength
- Have varied effect on early-age strength gain (Type F fly ash, calcined shales, and calcined clays lower early strength; silica fume and metakaolin increase early strength gain)
- Reduce permeability and absorption
- Increase sulfate resistance (with the exception of Type C fly ash, which may have either a positive or negative effect)
- Have no significant impact on abrasion resistance, drying creep and shrinkage, and freezing and thawing
- May reduce resistance to deicer scaling

The Code does not cover all topics related to concrete durability. It does not include recommendations for extreme exposure conditions (that is, acids, high temperature, or exposure to fire), alkali-aggregate reaction, or abrasion. The Code commentary (R4.8) identifies the importance of preventive maintenance; however, the topic is not explicitly

addressed in the Code. Additionally, the Code does not cover waterproofing, routine inspections, condition assessment, or service life prediction. Information related to these topics is found in other ACI documents, including:

- ACI 201.2R—Guide to Durable Concrete
- ACI/TMS 216.1—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies
- ACI 221.1R—Report on Alkali-Aggregate Reactivity
- ACI 222R—Protection of Metals in Concrete Against Corrosion
- ACI 222.2R—Report on Corrosion of Prestressing Steels
- ACI 222.3R—Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures
- ACI 224.1R—Causes, Evaluation, and Repair of Cracks in Concrete Structures
- ACI 311.4R—Guide for Concrete Inspection
- ACI 362.1R—Guide for the Design and Construction of Durable Concrete Parking Structures
- ACI 365.1R—Report on Service Life Prediction
- ACI 515.2R—Guide to Selecting Protective Treatments for Concrete
- ACI 562—Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures and Commentary

## 4.3—Requirements for concrete in various exposure categories

The Code addresses durability by requiring that four exposure categories be assigned to each concrete member. The four exposure categories are:

1. F: concrete exposed to moisture and cycles of freezing and thawing (with or without deicing chemicals);
2. S: concrete in contact with soil or water containing deleterious amounts of water-soluble sulfate ions;
3. W: concrete in contact with water but not exposed to freezing and thawing, chlorides, or sulfates;
4. C: concrete exposed to conditions that require additional protection against corrosion of reinforcement.

Each exposure category is divided into exposure classes that define severity of the exposure, starting with 0 for a negligible exposure. Once all structural members are assigned exposure classes and the concrete mixtures for these members satisfy the requirements outlined in those exposure classes, the Code’s minimum durability requirements are met.

**4.3.1 Freezing and thawing (F)**—The volume of ice is 9 percent larger than water. As water freezes in saturated concrete, cement phase and aggregates are subject to internal pressure, which then causes concrete tensile stresses. If those stresses are greater than the tensile strength of concrete, cracking will occur. The cumulative expansion after many cycles of freezing and thawing may lead to significant concrete damage. One method to protect concrete from freezing-and-thawing damage is to reduce moisture penetration so it does not become critically saturated. However, this is not always possible. The other method is to generate small

air bubbles in fresh concrete by addition of an air-entraining admixture, which creates voids for the freezing water to expand into without creating internal stress.

The Code requires the use of air entrainment in concrete in structural members exposed to cycles of freezing and thawing. Air entrainment significantly improves resistance of saturated concrete to freezing and thawing. ACI 212.3R provides an in-depth discussion on these air-entraining admixtures and their applications, dosage rates, effects on fresh and hardened concrete, and other factors they influence.

Although the specified amount of air entrainment depends primarily on the frequency of exposure to water (exposure class), but also on nominal maximum aggregate size and concrete compressive strength. To achieve similar freezing-and-thawing protection, higher air content is generally required for concrete mixtures with smaller nominal maximum aggregate size. For example, concrete with 3/8 in. aggregate requires 50 percent higher air content than concrete with 2 in. aggregate (Code Table 19.3.3.1). The Code requires that the licensed design professional (LDP) specify the nominal maximum aggregate size for each concrete mixture in the construction documents. Nominal maximum aggregate size depends on locally available aggregates, as well as construction issues such as size of formwork, member depth, and clear bar spacing. The criteria for maximum size selection are given in Code Section 26.4.2.1. Table 19.3.3.1 lists target air content for Exposure Classes F1, F2, and F3, which depends on the nominal maximum aggregate size. Code Table 19.3.3.3 also lists target air content for shotcrete.

Another factor affecting selection of target air content is compressive strength. An air content reduction of 1 percent is allowed for concrete with specified compressive strength exceeding 5000 psi (Code Section 19.3.3.6). The reason for air content reduction is that concretes with higher strengths are characterized by lower  $w/cm$  and reduced porosity, which improve resistance to freezing-and-thawing cycles.

For example, a structural member in Exposure Class F2 with 1/2 in. nominal maximum aggregate size requires concrete with a target air content of 7 percent (or 6 percent for concrete with compressive strength exceeding 5000 psi). Because exact air content is difficult to achieve, the Code allows tolerance for air content in as-delivered concrete of  $\pm 1.5$  percentage points. This is consistent with the tolerances in ASTM C94/C94M and ASTM C685/C685M (Section R26.4.2.1(a)(5)). The required air content range, therefore, is from 5.5 to 8.5 percent (or 4.5 to 7.5 percent for concrete with compressive strength exceeding 5000 psi).

Additional requirements or limitations, such as minimum compressive strength, minimum  $w/cm$ , or limits on cementitious materials depend on the exposure class assigned to a particular member. Interior members, foundations below the frost line, or structures in climates where freezing temperatures are not anticipated are assigned Exposure Class F0. These conditions therefore do not require air entrainment and there is no limit on maximum  $w/cm$  or on the use of cementitious materials. The minimum compressive strength for concrete in Exposure Class F0 is the Code minimum: 2500 psi.

Freezing-and-thawing cycles have little effect on concrete that is not critically saturated. Structural members exposed to freezing-and-thawing cycles, but with low likelihood of being saturated, are assigned Exposure Class F1. Concrete for this exposure must be air-entrained (Code Table 19.3.3.1) in case there is occasional saturation during freezing. In addition, the concrete should have a maximum  $w/cm$  of 0.55 and at least 3500 psi compressive strength.

Exposure Classes F2 and F3 are assigned to concrete in structural members having frequent exposure to water during freezing. The distinction between the two classes is that Class F2 anticipates no exposure to deicing chemicals or seawater, while Class F3 does. Concrete in F2 and F3 exposure classes must be air-entrained (Code Table 19.3.3.1) and have a maximum  $w/cm$  of 0.45 and 0.40, respectively. The minimum concrete compressive strengths for F2 and F3 classes are 4500 and 5000 psi, respectively. The most severe class of exposure, F3, also has a limit on cementitious materials in concrete mixtures, given in Code Table 26.4.2.2(b).

Requirements for concrete in Exposure Category F are listed in Table 19.3.2.1 of the Code.

**4.3.2 Sulfate (S)**—All soluble forms of sulfate, sodium, calcium, potassium, or magnesium have a detrimental effect on concrete. Depending on the sulfate form, they react with hydrated cement phases and result in formation of ettringite or gypsum. Depending on the reaction product, the concrete either expands and cracks (ettringite), or softens and loses strength (gypsum). The most effective measure to reduce the effects of sulfate reactions, apart from reducing moisture ingress, is to use cements with a low content of tricalcium aluminate ( $C_3A$ ). A more detailed discussion on sulfate's effect on concrete can be found in ACI 201.2R.

Exposure Category S applies to structural members that will likely be affected by external source of sulfates, which predominantly come from exposure to soil, groundwater, or seawater. The exposure classification (class) is selected based on the concentration of sulfate ions ( $SO_4^{2-}$ ), which should be determined in accordance with ASTM C1580 for soil samples and with ASTM D516 or ASTM D4130 for water samples. The Code requires the LDP to specify the exposure class by comparing field test results with concentration ranges in Table 19.3.1.1 of the Code. Note that seawater exposure is classified as S1 even though the sulfate concentration (in seawater) is usually higher than 1500 ppm. The reason for lower class for seawater is the presence of chloride ions, which inhibit expansive reaction due to sulfate attack.

Class S0 is assigned to concrete in members not exposed to sulfates and there is no restriction on  $w/cm$ , or type or limit on cementitious materials. The only requirement for concrete classified as S0 is the minimum compressive strength be at least 2500 psi. Greater minimum compressive strength and maximum  $w/cm$  limits are imposed on concrete in Exposure Classes S1 through S3. For these exposure classes, the type of cement is the major requirement.

A summary of all requirements for concrete in Exposure Category S is listed in Table 19.3.2.1 of the Code.

**4.3.3 In contact with water (W)**—The durability of structural members in direct contact with water, such as foundation walls below the groundwater table, may be affected by water penetration into or through concrete. Apart from external systems, such as drainage systems or waterproofing membranes for foundations, the most effective way to reduce concrete permeability is to keep the  $w/cm$  low.

Concrete for members assigned to Exposure Class W0 is required to have a minimum compressive strength of 2500 psi, but no additional requirements. Concrete in structural members assigned to Exposure Class W1 does not have specific requirements for low permeability, but because of exposure to water. No direct Code requirements are given for addressing aggregate reactivity. Instead, Code Section 26.4.2.1(d) requires that evidence be submitted documenting either that the aggregates are not alkali-silica reactive or the intended mitigation measures if they are reactive. Further documentation must be provided that the aggregates are not alkali-carbonate reactive. Such documentation is typically provided by the concrete supplier. Commentary R26.4.2.2(d) suggests consulting ASTM C1778 for methods and criteria for determining the reactivity of aggregates and guidance for reducing the risk of occurrence. Use of aggregates that are alkali-carbonate reactive is also prohibited. Concrete in structural members assigned to Exposure Class W2 requires low permeability. Table 19.3.2.1 of the Code requires  $w/cm$  not to exceed 0.50 and compressive strength to be at least 4000 psi. Exposure Class W2 has the same aggregate reactivity requirements as that of W1. Note that additional requirements are imposed if the member's durability is to be affected by reinforcement corrosion, sulfate exposure, or exposure to cycles of freezing and thawing. Recommendations for the design and construction of water tanks and reservoirs are provided in ACI 350.4R, ACI 334.1R, and ACI 372R.

Requirements for concrete in Exposure Class W are listed in Table 19.3.2.1 of the Code.

**4.3.4 Corrosion (C)**—Corrosion of reinforcement may significantly affect durability and structural capacity of a member. Reinforcement corrosion usually occurs as a result of the presence of chlorides or steel depassivation due to carbonation. Corrosion products (rust) are larger in volume than the original steel and therefore exert internal pressure on the surrounding concrete, causing it to crack or delaminate. A significant loss of reinforcing bar cross section leads to increased steel stresses under service load and reduced member nominal strength. Because moisture and oxygen must be present at the steel surface for corrosion to occur, the quality of concrete and the reinforcing bar cover are of great importance. Corrosion can be mitigated by proper mixture design and construction practices; application of sealers, coatings, or membranes that protect concrete from moisture and chloride penetration; use of corrosion-resistant reinforcement; or inclusion of corrosion inhibitors in the mixture to elevate the corrosion threshold concentration. Refer to ACI 222R, ACI 222.3R, and ACI 212.3R for additional information.

Each exposure class within the corrosion exposure category has a limit on water-soluble chloride-ion content in concrete. The chloride-ion content is measured in accor-

dance with ASTM C1218/C1218M, which requires the sample be representative of the concrete constituents—that is, cementitious materials, fine and coarse aggregate, water, and admixtures. Because chloride limits are imposed on concrete in Exposure Class C0, all structural concrete must comply with the Code's maximum chloride ion limits. Chloride limits for nonprestressed concrete, expressed as percent of cementitious materials weight, are 1 percent for Class C0, 0.30 percent for Class C1, and 0.15 percent for Class C2. Chloride limit for prestressed concrete is 0.06 percent by cement weight, regardless of exposure class. Apart from chloride limits, Exposure Classes C0 and C1 have no additional requirements, as there is no limit on  $w/cm$  and the minimum compressive strength is 2500 psi.

Class C2 requires concrete strength of at least 5000 psi, a maximum  $w/cm$  of 0.40, and reinforcing steel specified cover to satisfy the Code's minimum concrete cover provisions. The minimum concrete cover depends on exposure to weather, contact with ground, type of member, type of reinforcement, diameter and arrangement (bundling) of reinforcement, method of construction (cast-in-place or precast), and if the member is prestressed. Tables 20.5.1.3.1, 20.5.1.3.2, 20.5.1.3.3, and 20.5.1.3.4 of the Code provide cover provisions for cast-in-place nonprestressed; cast-in-place prestressed; precast nonprestressed or prestressed concrete members, produced under plant conditions; and deep foundation members, respectively. If the design requires bundled bars, check Section 20.5.1.3.5 of the Code for specific requirements. Concrete cover requirements in corrosive environments or other severe exposure conditions are more stringent and are provided in Section 20.5.1.4 of the Code.

Requirements for concrete in Exposure Class C are listed in Table 19.3.2.1 of the Code.

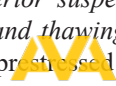
#### 4.4—Concrete evaluation, acceptance, and inspection

Durability requirements are met once concrete proportions and properties satisfy the minimums set by the Code. To assure that the delivered concrete achieves the desired durability, the LDP should specify concrete evaluation and acceptance criteria consistent with Code Section 26.12 and field inspection consistent with Code Section 26.13.

#### 4.5—Examples

The following examples illustrate one approach of implementing minimum durability requirements of the Code. In some cases, durability requirements for material properties may exceed those of the structural design. This is more likely for severe exposure conditions, which require a minimum compressive strength of 5000 psi. In some cases, SCMs may be required, which may extend setting time and reduce early-age strength, and result in modifications to construction schedule. For these reasons, consultation with engineers experienced with concrete materials and mixture proportioning, and with concrete suppliers is recommended.

**4.5.1 Example 1: Interior suspended slab not exposed to moisture or freezing and thawing**—On the design of a cast-in-place, nonprestressed slab in a multi-story



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office building. It is located in a climate zone with frequent freezing-and-thawing cycles; however, the slab will be constructed during summer and the temperatures at night during construction are expected to remain above 40 to 45°F. It is desirable for the slab to quickly gain strength to meet the construction schedule. For this reason, calcium chloride was proposed as an accelerating admixture. The required minimum compressive strength, from structural analysis, is 4000 psi. The slab is 7 in. thick with top and bottom mats of No. 5 bars spaced at 8 in. What additional information should be specified for the slab concrete to meet durability requirements?

**Answer:** The first step is to assign exposure classes within every exposure category to each structural member or group of members. Once exposure classes are assigned, the Code guides the LDP to satisfy the durability requirements. The step-by-step instructions are as follows:

| Step description/<br>action item                                                                              | Selection and discussion                                                                                                                                                                                                                                                            | Code reference   |
|---------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------|
| Assign exposure classes within each exposure category                                                         | <b>F0</b> (concrete not exposed to freezing-and-thawing cycles); <b>S0</b> (soil not in contact with concrete; injurious sulfate attack is not a concern); <b>W0</b> (there are no specific requirements for low permeability); <b>C0</b> (concrete dry or protected from moisture) | Table 19.3.1.1   |
| Assign required minimum compressive strength                                                                  | <b>2500 psi</b> (based on F0)                                                                                                                                                                                                                                                       | Table 19.3.2.1   |
| Assign maximum w/cm                                                                                           | <b>Not limited</b> (based on F0)                                                                                                                                                                                                                                                    | Table 19.3.2.1   |
| Assign minimum concrete cover                                                                                 | <b>0.75 in.</b> (not exposed to weather, slabs..., No. 11 bars and smaller)                                                                                                                                                                                                         | Table 20.5.1.3.1 |
| Assign nominal maximum size of aggregate                                                                      | <b>2 in.</b> (3/4 x 3 in. clear bar spacing – top and bottom mat, or 1/3 x 7 in. – slab thickness); <b>use 1 in.</b> as readily available                                                                                                                                           | 26.4.2.1(a)(5)   |
| Assign alkali-aggregate reaction documentation                                                                | <b>Not exposed to moisture.</b> No additional documentation required.                                                                                                                                                                                                               | Table 19.3.2.1   |
| Assign required air content                                                                                   | <b>Not air-entrained</b>                                                                                                                                                                                                                                                            | Table 19.3.2.1   |
| Assign limits on cementitious materials                                                                       | <b>No limits</b>                                                                                                                                                                                                                                                                    | Table 19.3.2.1   |
| Assign limits on calcium chloride admixture                                                                   | <b>No restriction</b> (based on S0) [Note: chloride ions from the admixture will significantly affect measured chloride ion content in concrete.]                                                                                                                                   | Table 19.3.2.1   |
| Assign maximum water-soluble chloride ion (Cl <sup>-</sup> ) content in concrete, percent by weight of cement | <b>1.00</b> (based on C0, water-soluble chloride-ion content from all concrete ingredients determined by ASTM C1218/ C1218M at age between 28 and 42 days)                                                                                                                          | Table 19.3.2.1   |

**4.5.2 Example 2: Balcony slab exposed to moisture and freezing and thawing**—An LDP designs a cast-in-place, nonprestressed balcony slab in a multi-story office building, located in a climate zone with frequent freezing-and-thawing cycles. It is anticipated that the balconies will be exposed to moisture, but not deicing salts. The required minimum compressive strength, from structural analysis, is 4000 psi. The balcony slabs are 6 in. thick with top mat of No. 4 bars spaced at 6 in. What additional information should be specified in the contract documents to ensure that the balcony concrete meets Code durability requirements?

**Answer:** Durability requirements are met once the most rigorous requirements of the Code are satisfied. The first step is to assign exposure classes within each exposure category to each structural member or group of members. Once exposure classes are assigned, the Code guides the LDP to set the minimum durability requirements. The step-by-step instructions are as follows:

| Step description/<br>action item                      | Selection and discussion                                                                                                                                                                                                                                                                                                                                                                                                      | Code reference                             |
|-------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------|
| Assign exposure classes within each exposure category | <b>F2</b> (concrete exposed to freezing-and-thawing cycles with frequent exposure to water); <b>S0</b> (soil not in contact with concrete; injurious sulfate attack is not a concern); <b>W1</b> (no specific requirements for low permeability; exposure to moisture triggers Code requirements to address alkali-aggregate reactivity); <b>C1</b> (concrete exposed to moisture but not to an external source of chlorides) | Table 19.3.1.1                             |
| Assign required minimum compressive strength          | <b>4500 psi</b> (based on F2); because 4500 psi is greater than design strength of 4000 psi, the 4500 psi governs                                                                                                                                                                                                                                                                                                             | Table 19.3.2.1                             |
| Assign maximum w/cm                                   | <b>0.45</b> (based on F2)                                                                                                                                                                                                                                                                                                                                                                                                     | Table 19.3.2.1                             |
| Assign minimum concrete cover                         | <b>1.5 in.</b> (exposed to weather, No. 5 bar and smaller)                                                                                                                                                                                                                                                                                                                                                                    | Table 20.5.1.3.1                           |
| Assign nominal maximum size of aggregate              | <b>2 in.</b> (1/3 x 6-in. – slab thickness, 3/4 x 6-in. clear bar spacing – top mat bars); <b>use 1 in.</b> as readily available                                                                                                                                                                                                                                                                                              | 26.4.2.1(a)(5)                             |
| Assign alkali-aggregate reaction documentation        | For concrete exposed to water, confirm low alkali-aggregate reactivity or provide mitigation. ASTM C1778 is referenced in R26.5.2.2(d) for this use.                                                                                                                                                                                                                                                                          | Table 19.3.2.1<br>26.4.2.2(d)              |
| Assign required air content                           | <b>6% ± 1.5%</b> (for 1 in. aggregate and F2 class) [Note: 1 in. aggregate can be substituted for 3/4 in. aggregate with no air content change]                                                                                                                                                                                                                                                                               | Table 19.3.3.1 and Section R26.4.2.1(a)(6) |
| Assign limits on cementitious materials               | <b>No limits</b>                                                                                                                                                                                                                                                                                                                                                                                                              | Table 19.3.2.1                             |

cont. from previous page

|                                                                                                                             |                                                                                                                         |                                                       |
|-----------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------|
| Assign maximum water-soluble chloride ion (Cl <sup>-</sup> ) content in concrete, percent by mass of cementitious materials | <b>0.30</b>                                                                                                             | Table 19.3.2.1<br>26.4.2.2(e)(1)<br>or 26.4.2.2(e)(2) |
| Provide guidance on cold weather construction                                                                               | Consult ASTM C94/C94M, ACI 306R, and ACI 301 for guidance on temperature limits for concrete delivered in cold weather. | Section 26.5.4.1                                      |

**4.5.3 Example 3: Wall foundation exposed to sulfate soil and deicing salts while in service**—An LDP designs a cast-in-place, nonprestressed foundation wall of a partially underground parking structure. The structure is located in a northern climate zone with frequent freezing-and-thawing cycles and high sulfate soil content (6 percent SO<sub>4</sub><sup>2-</sup> by mass). Exposure to deicing salts is anticipated from melt-water runoff from the nearby streets and a sidewalk. It is desirable for the foundation wall to quickly gain strength to reduce possible frost damage and to meet the construction schedule. The required minimum compressive strength from structural design is 4000 psi. The foundation wall is 8 in. thick with inside face and outside face mats of No. 4 bars spaced at 12 in. What additional information should be specified in the contract documents to ensure that the foundation wall concrete meets Code durability requirements?

**Answer:** The first step is to assign exposure classes within every exposure category to each structural member or group of members. Once exposure classes are assigned, the Code guides the LDP to set the minimum durability requirements. The step-by-step instructions are as follows:

| Step description/<br>action item                      | Selection and discussion                                                                                                                                                                                                                                                                                                                                                                                                                                                                                              | Code reference |
|-------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------|
| Assign exposure classes within each exposure category | <b>F3</b> (concrete exposed to freezing-and-thawing cycles with frequent exposure to water and exposure to deicing chemicals); <b>S3</b> (structural concrete members in direct contact with elevated soluble sulfates in soil or water); <b>W2</b> (concrete in contact with moisture and low permeability is required; exposure to moisture triggers Code requirements to address alkali-aggregate reactivity); <b>C2</b> (concrete exposed to moisture and an external source of chlorides from deicing chemicals) | Table 19.3.1.1 |
| Assign minimum compressive strength                   | <b>5000 psi</b> (based on F3 and C2); because 5000 psi is greater than design strength of 4000 psi, 5000 psi governs                                                                                                                                                                                                                                                                                                                                                                                                  | Table 19.3.2.1 |
| Assign maximum w/cm                                   | <b>0.40</b> (based on F3 and C2)                                                                                                                                                                                                                                                                                                                                                                                                                                                                                      | Table 19.3.2.1 |

|                                                                                                                             |                                                                                                                                                                                                                                                                                                                             |                                                          |
|-----------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------|
| Assign minimum concrete cover                                                                                               | <b>2.0 in.</b> – outside face of wall (1.5 in. cover is listed in Table 20.5.1.3.1 for exposure to weather or in contact with ground for No. 5 bar and smaller; cover increased to 2.0 in. based on 20.5.1.3.1)<br><b>3/4 in.</b> – inside face of wall (side of the wall not exposed to weather or in contact with ground) | Table 20.5.1.3.1<br>20.5.1.4.1                           |
| Assign alkali-aggregate reaction documentation                                                                              | For concrete exposed to water, confirm low alkali-aggregate reactivity or provide mitigation. ASTM C1778 is referenced in R26.4.2.2(d) for this use.                                                                                                                                                                        | Table 19.3.2.1<br>26.4.2.2(d)                            |
| Assign nominal maximum size of aggregate                                                                                    | <b>1.5 in.</b> (1/5 x 8 in. – wall thickness, 3/4 x 3-1/4 in. clear bar spacing – between interior and exterior mats of reinforcing steel); <b>use 1.5 in.</b>                                                                                                                                                              | Section<br>26.4.2.1(a)(5)                                |
| Assign limits on calcium chloride admixture                                                                                 | Not permitted (based on S2 and C2)                                                                                                                                                                                                                                                                                          | Table 19.3.2.1                                           |
| Assign required air content                                                                                                 | <b>5.5% ± 1.5%</b> (for 1.5 in. aggregate and F3 class) [Notes: 1. Changing to lower nominal maximum aggregate size will require higher air content; 2. Air content reduction of 1% (to 4.5% ± 1.5%) is allowable if concrete compressive strength exceeds 5000 psi; refer to 19.3.3.3]                                     | Table 19.3.3.1                                           |
| Assign limits on cementitious materials                                                                                     | S3 Option 2 limited to ASTM C595 HS or ASTM C1157 HS. Can use ASTM C150 Type V as sole cementitious material if it meets limitations on expansion.                                                                                                                                                                          | Table 19.3.2.1<br>Table 26.4.2.2(b)<br>Table 26.4.2.2(c) |
| Assign limits on calcium chloride admixture                                                                                 | Not permitted (based on S3)                                                                                                                                                                                                                                                                                                 | Table 19.3.2.1                                           |
| Assign maximum water-soluble chloride-ion (Cl <sup>-</sup> ) content in concrete, percent by mass of cementitious materials | 0.15                                                                                                                                                                                                                                                                                                                        | Table 19.3.2.1<br>26.4.2.2(e)(1) or<br>26.4.2.2(e)(2)    |

## REFERENCES

*American Concrete Institute*

ACI 201.2R-08—Guide to Durable Concrete

ACI 212.3R-10—Report on Chemical Admixtures for Concrete

ACI 222R-01—Protection of Metals in Concrete Against Corrosion

ACI 222.3R-11—Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures

ACI 301-10—Specification for Structural Concrete

ACI 306R-10—Guide to Cold Weather Concrete

ACI 334.1R-92—Concrete Shell Structures Practice and Commentary

ACI 350.4-04—Design Considerations for Environmental Engineering Concrete Structures

ACI 372R-13—Guide to Design and Construction of Circular Wire-and-Strand-Wrapped Prestressed Concrete Structures

#### *ASTM International*

ASTM C94/C94M-15—Standard Specification for Ready-Mixed Concrete

ASTM C1012/C1012M-13—Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution

ASTM C1218/C1218M-99—Standard Test Method for Water-Soluble Chloride in Mortar and Concrete

ASTM C150/C150M-12—Standard Specification for Portland Cement

ASTM C685/C685M-14—Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing

ASTM C1580-09—Standard Test Method for Water-Soluble Sulfate in Soil

ASTM D516-11—Standard Test Method for Sulfate Ion in Water

ASTM D4130-15—Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines

#### **Authored documents**

Hewlett, P. C., ed., 1998, *Lea's Chemistry of Cement and Concrete*, fourth edition, John Wiley & Sons, Inc., New York, 1057 pp.

Kosmatka, S. H.; Kerkhoff, B.; and Panarese, W. C., 2002, *Design and Control of Concrete Mixtures (EB001)*, 14th edition, fourth printing (rev.), Portland Cement Association, Skokie, IL, Feb., 358 pp.

Kosmatka, S. H., and Wilson, M. L., 2011, *Design and Control of Concrete Mixtures (EB001)*, 15th edition, Portland Cement Association, Skokie, IL, 444 pp.

Neville, A., ed., 2003, *Neville on Concrete: An Examination of Issues in Concrete Practice*, American Concrete Institute, Farmington Hills, MI, 510 pp.

Powers, T. C., 1958, "Structure and Physical Properties of Hardened Portland Cement Paste," *Journal of the American Ceramic Society*, V. 41, No. 1, pp. 1-6.

Stark, D., 1989, "Durability of Concrete in Sulfate-Rich Soils (RD097)," Portland Cement Association, Skokie, IL, 14 pp.

Stark, D., 2001, "Long-Term Performance of Plain and Reinforced Concrete in Seawater Environments (RD119)," Portland Cement Association, Skokie, IL, 14 pp.

Taylor, P. C.; Nagi, M. A.; and Whiting, D. A., 1999, "Threshold Chloride Content for Corrosion of Steel in Concrete: A Literature Review (RD2169)," Portland Cement Association, Skokie, IL, 32 pp.

Whiting, D. A.; Taylor, P. C.; and Nagi, M. A., 2002, "Chloride Limits in Reinforced Concrete (RD2438)," Portland Cement Association, Skokie, IL, 72 pp.

# CHAPTER 5—ONE-WAY SLABS

## 5.1—Introduction

One-way slabs are generally used in buildings with vertical supports (columns or walls) that are unevenly spaced, creating a longer span in one direction and a shorter span in the perpendicular direction. One-way slabs typically span in the short direction and are supported by beams that span in the long direction (refer to the building example in Chapter 1 of this Manual). During preliminary design, the designer determines the loads and spans, reinforcement type (prestressed or nonprestressed), and slab thickness. The designer determines the concrete strength based on experience and the exposure and durability provisions of the Code.

One-way slabs are designed in accordance with Code Chapter 7 for strength and serviceability. Generally, one-way slabs are designed for distributed loads specified by the building code. In some cases, such as parking garages, point loads must also be considered. These result in local shear forces on the slab, requiring verification of punching shear strength of the slab. Punching shear for slabs is addressed in Chapter 6 for two-way slabs in this Manual.

For relatively small slab openings, trim bars can be used to limit crack widths caused by geometric stress concentrations. For larger openings, a local increase in slab thickness, as well as additional reinforcement, may be necessary to provide adequate serviceability and strength.

## 5.2—Analysis

The Code allows for the designer to use any analysis procedure that satisfies equilibrium and geometric compatibility, as long as design strength and serviceability requirements are met. The Code includes a simplified method of analysis for one-way slabs that relies on coefficients to calculate moments and shears.

## 5.3—Service limits

**5.3.1 Nonprestressed slab – Minimum thickness**—For nonprestressed slabs, the Code allows the designer for slabs not supporting or attached to partitions or other construction likely to be damaged by large deflection to either calculate deflections or simply satisfy a minimum slab thickness (Code Section 7.3.1). In the case where loads are heavy, nonuniform, or deflection is a concern, deflections should be calculated to verify that short- and long-term deflections are within Code limits (Section 24.2.2).

**5.3.2 Prestressed slabs – Minimum thickness**—For prestressed slabs, the Code does not provide a minimum span-to-depth ratio, but rather requires that both immediate and time-dependent deflections be calculated in accordance with Code Section 24.2 and checked against the limits in Code Section 24.2.2. Table 9.3 of the *Post-Tensioning Manual* (Post-Tensioning Institute [PTI] 2006) lists span-to-depth ratios that have been found from experience to provide satisfactory structural performance.

**5.3.3 Deflections**—For all prestressed slabs and nonprestressed slabs that have depths less than those in Code Table 9.3.1.1, deflections must be calculated. The calculated deflections must not exceed the limits given in Code Section 24.2. Deflections can be calculated by any appropriate method, such as classical equations or structural analysis software.

Note that the spacing of slab reinforcing bars, timing of form removal, concrete quality, timing of construction loads, and other construction variables all could affect the actual deflection. These variables should be considered when assessing the accuracy of deflection calculations. In addition, creep will increase the immediate deflections.

If prestressed slabs are designed to remain uncracked, then slab deflections are usually small because deflections can be calculated using gross section properties.

**5.3.4 Nonprestressed slabs – Concrete service stress**—Nonprestressed slabs are designed for strength but do not have limitations placed on concrete service flexural stress.

**5.3.5 Prestressed slabs – Concrete service stress**—For prestressed slabs, the analysis of concrete flexural tension stresses is a critical part of the design. The slab is classified according to Code 7.3.4.1 and 24.5.2 as Class U, T, or C based on the maximum net tensile stresses in the precompressed tensile zone. These classifications are used to determine the appropriate section properties for use in calculating stresses and deflections. Class U members are considered uncracked, Class C are considered cracked, and Class T is the transition between the two.

In post-tensioned (PT) slab construction, the tendon profile must be designed before the slab flexural stresses in a design strip can be calculated. Both profile and tendon force are directly related to slab forces and moments created by the prestressing force. A common approach to calculate PT slab moments is the use of the “load balancing” concept. Tendons are typically placed in a parabolic profile such that the tendon is at the minimum cover requirements at midspan and over supports; this maximizes the parabolic drape. Anchors are typically placed at middepth at the slab edge (Fig. 5.3.5).

The tendon exerts a uniform upward force along its length that counteracts a portion of the gravity loads, usually 60 to 80 percent of the slab self-weight according to Libby (1990); hence, the term “load balancing.” The load effect from the prestressing force in the tendon is then combined with the load effect of the gravity loads to determine net concrete stresses.

To achieve Code stress limits, the designer can use an iterative or direct approach. In the iterative approach, the

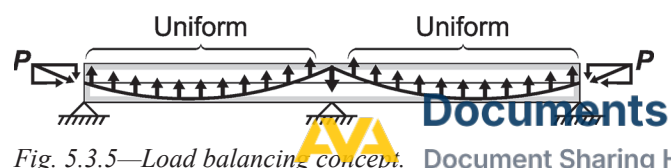


Fig. 5.3.5—Load balancing concept. Document Sharing Hub

tendon profile is defined and the tendon force is assumed. The analysis is executed, flexural stresses are calculated, and the designer then adjusts the profile or force or both, depending on results and design constraints.

In the direct approach, the designer determines the highest tensile stress permitted, then rearranges equations so that the analysis calculates the tendon force required to achieve the stress limit.

The Code does not impose a minimum concrete compressive stress due to the effective prestress force, but 125 psi is commonly used as a minimum for cast-in-place PT slabs, which is the same as the required amount for prestressed two-way slabs as noted in Code Section 8.6.2.1. For slabs exposed to aggressive environments, the minimum concrete compressive stress is usually set to a higher value.

## 5.4—Required strength

The design of one-way slabs is typically controlled by moment strength, not concrete stress or shear strength. Assuming a uniform load, the designer calculates the unit (usually a 1 ft width) factored slab moments. The required area of flexural reinforcement over a unit slab width is calculated with the same assumptions as a beam.

**5.4.1 Calculation of required moment strength**—For nonprestressed reinforced slabs, a quick way to calculate gravity design moments (if the slab meets the specified geometric and load conditions) is by using the moment coefficients in Section 6.5 of the Code. Chapter 6 of the Code permits other, more sophisticated analysis methods.

For indeterminate PT slabs, effects of reactions induced by prestressing (secondary moments) should be added to the factored gravity moments per Section 5.3.11 of the Code to calculate  $M_u$ . Secondary moments in the slab are a result of the beam's vertical restraint of the slab against the PT "load" at each support. Because the PT force and drape are determined during the service stress checks, secondary moments can be quickly calculated by the "load-balancing" analysis concept. A simple method for calculating the secondary moment is to subtract the tendon force multiplied by the tendon eccentricity (distance from the neutral axis) from the total balance moment, expressed mathematically as  $M_2 = M_{bal} - Pe$ .

The critical locations to calculate  $M_u$  along the span are usually at the support and midspan. Section 7.4.2.1 of the Code allows  $M_u$  to be calculated at the face of support rather than the support centerline.

**5.4.2 Calculation of required shear strength**—Assuming a uniform load, the designer calculates the unit (usually a 1 ft width) factored slab shear force by either the coefficient method or more sophisticated calculations.

## 5.5—Design strength

One-way slabs must have adequate one-way shear strength and moment strength in all design strips.

**5.5.1 Calculation of design moment strength**—The required area of flexural reinforcement for a nonprestressed and PT unit slab width are calculated using the same assumptions as for a beam, Chapter 7, of this Manual.

**Table 5.6.1a—Maximum spacing of bonded reinforcement in nonprestressed and Class C prestressed members (Code Table 24.3.2 excerpt for deformed bars or wires only)**

|            |                                              |
|------------|----------------------------------------------|
| Lesser of: | $15\left(\frac{40,000}{f_s}\right) - 2.5C_c$ |
|            | $12(40,000/f_s)$                             |

**5.5.2 Calculation of design shear strength**—Discussion for nominal one-way shear strength is the same as for a beam, Chapter 7, of this Manual.

## 5.6—Detailing of flexural reinforcement

The Code requires a minimum area of flexural reinforcement be placed in tension regions to ensure that the slab deformation and crack widths are limited when the cracking strength of the slab is exceeded. If more than the minimum area is required by analysis, that reinforcement area must be provided. Reinforcement in one-way slabs is usually uniformly spaced, unless there is a large point load or opening.

**5.6.1 Nonprestressed slab – Flexural reinforcement area and placing**—For nonprestressed slabs, the minimum area of flexural reinforcement,  $A_{s,min}$ , is  $0.0018A_g$  in Code Section 7.6.1.1.

The maximum spacing of deformed flexural bars is given in Table 24.3.2 of the Code (Table 5.6.1a of this Manual). Because the service load stress in the reinforcement ( $f_s$ ) is usually taken as 40,000 psi, the maximum spacing will not exceed 12 in.

The bar termination rules in Section 7.7.3 of the Code cover general conditions that apply to beams, but because one-way slab bars are usually placed at the maximum spacing, bars generally cannot be terminated without violating the maximum spacing. This usually results in all bottom bars extending full length into the beams.

**5.6.2 Prestressed slab – Flexural tendon area and placing**—For prestressed one-way slabs, the Code does not have a limit for minimum tendon area or a minimum compressive stress due to the effective prestress force. This is consistent with the flexible approach on service stresses.

**5.6.3 Prestressed slab – Flexural reinforcing bar area and placing**—The Code requires the minimum bonded reinforcement area,  $A_{s,min}$ , to be placed close to the slab face at the bottom at midspan and the top at the support. For one-way prestressed slabs,  $A_{s,min} = 0.004A_{cr}$ . Because the one-way slab strip is rectangular,  $A_{cr} = 0.5A_g$ . This minimum is independent of service stress level.

The maximum spacing of reinforcing bars in Class T or C prestressed slabs containing unbonded tendons is the lesser of  $3h$  and 18 in.

If the slab design moment strength is fully satisfied by the tendons alone, the termination length of  $A_{s,min}$  bars for bottom bars is (a) and for top bars is (b):

- (a) At least  $\ell_n/3$  in positive moment areas and be centered in those areas
- (b) At least  $\ell_n/6$  on each side of the face of support

The termination length for bars that are required for strength are the same as for nonprestressed slabs.

**5.6.4 Temperature and shrinkage reinforcement and placing**—Shrinkage and temperature slab reinforcement is required and could be either reinforcing bar or tendons placed perpendicular to flexural reinforcement.

If the designer uses reinforcing bars, the minimum area of temperature and shrinkage Grade 60 bar is  $0.0018A_g$ .

If the designer uses tendons to resist shrinkage and temperature stresses, the minimum slab effective compression force due to temperature and shrinkage tendons is 100 psi.

The purpose of this reinforcement is to restrain the size and spacing of slab cracks, which can occur due to volume variations caused by temperature changes and slab shrinkage over time. In addition, if the slab is restrained against movement, the Code requires the designer to provide reinforcement that accounts for the resulting tension stress in the slab.

#### Authored references

Libby, J., 1990, *Modern Prestressed Concrete: Design Principles and Construction Methods*, fourth edition, Springer, 871 pp.

Post-Tensioning Institute (PTI), 2006, *Post-Tensioning Manual*, sixth edition, PTI TAB.1-06, 354 pp.



## 5.7—Examples

### One-way Slab Example 1: *Nonprestressed one-way slab*—

Design and detail the second-story floor slab of the seven-story building. The one-way slab consists of five spans of 14 ft each. The slab is supported by 18 in. wide beams. A 6 ft cantilever extends at the left end of the slab (Fig. E1.1).

#### Given:

Load—

Service live load  $L = 100$  psf

Concrete—

$f'_c = 5000$  psi (normalweight concrete)

$f_y = 60,000$  psi

Geometry—

Span length: 14 ft

Beam width: 18 in.

Column dimensions: 24 in. x 24 in.

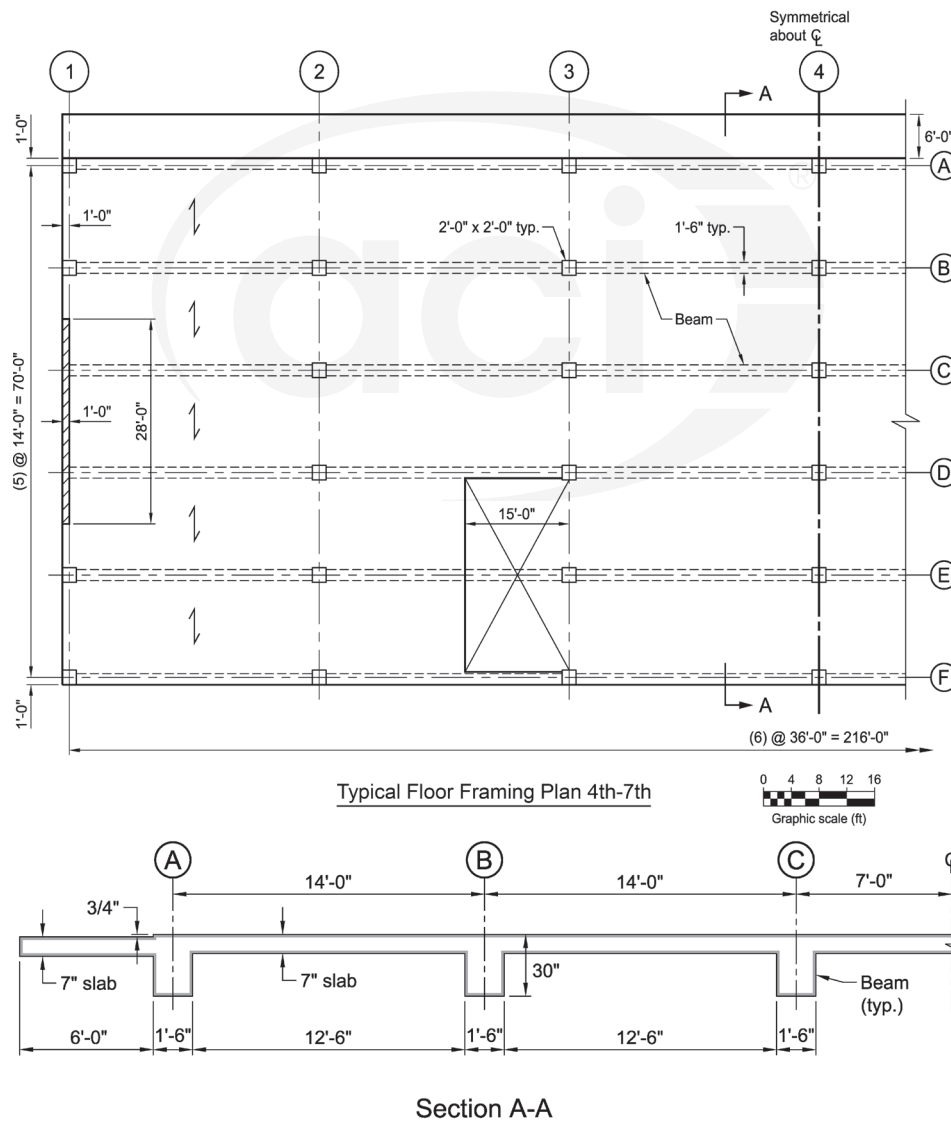
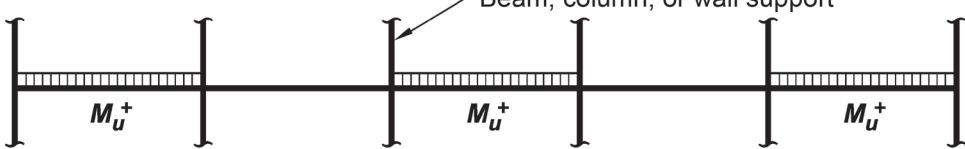
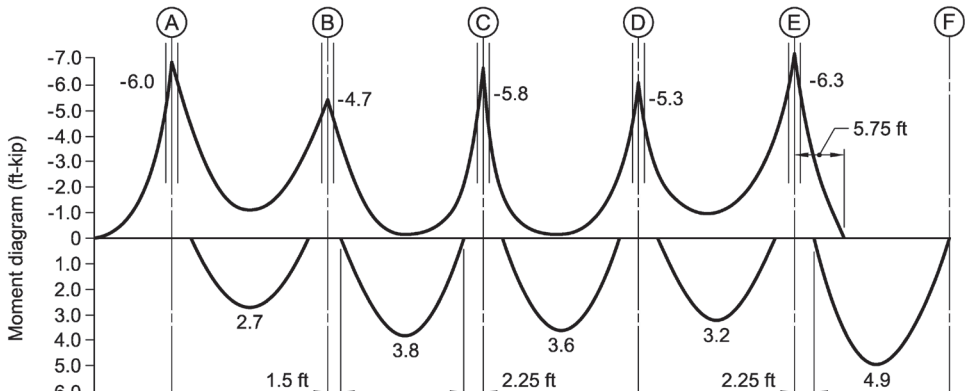


Fig. E1.1—Plan and section of five-span one-way slab.

| ACI 318                         | Discussion                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                              | Calculation                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
|---------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Step 1: Geometry                |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
| 7.3.1.1                         | <p>Determine slab thickness using ratios from Table 7.3.1.1.</p> <p>Determine cantilever thickness:</p>                                                                                                                                                                                                                                                                                                                                                                                                                                                                 | $h \geq \frac{\ell}{24} = \frac{(14 \text{ ft})(12 \text{ in./ft})}{24} = 7 \text{ in.}$ $h \geq \frac{\ell}{10} = \frac{(6 \text{ ft})(12 \text{ in./ft})}{10} = 7.2 \text{ in., say, 7 in.}$ <p>Because the slab and cantilever satisfy the Code span-to-depth ratios (Table 7.3.1.1), the designer does not need to check deflections, unless the slab is supporting or attached to partitions or other construction likely to be damaged by large deflections.</p> |
|                                 | Note: Architectural requirements specify a 3/4 in. step at the cantilever. Detail to maintain 7 in. slab thickness.                                                                                                                                                                                                                                                                                                                                                                                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
|                                 | Self-weight<br>Slab:                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                    | $w_s = (7 \text{ in.}/12 \text{ in./ft})(150 \text{ lb/ft}^3) = 87.5 \text{ psf}$                                                                                                                                                                                                                                                                                                                                                                                      |
| Step 2: Loads and load patterns |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
| 5.3.1                           | <p>The service live load is 100 psf in assembly areas and corridors per Table 4-1 in ASCE/SEI 7. For cantilever use 100 psf. To account for weights from ceilings, partitions, HVAC systems, etc., add 15 psf as miscellaneous dead load.</p> $U = 1.4D \quad (5.3.1a)$ $U = 1.2D + 1.6L \quad (5.3.1b)$ <p>The slab resists gravity only and is not part of a lateral force-resisting system, except to act as a diaphragm.</p>                                                                                                                                        | <p>The required strength equations to be considered are:</p> $U = 1.4(87.5 \text{ psf} + 15 \text{ psf}) = 143.5 \text{ psf}$ $U = 1.2(102.5 \text{ psf}) + 1.6(100 \text{ psf}) = 123 \text{ psf} + 160 \text{ psf} = 283 \text{ psf} \quad \textbf{Controls}$                                                                                                                                                                                                        |
| 6.4.2                           | <p>Both ASCE/SEI 7 and the Code provide guidance for addressing live load patterns. Either approach is acceptable.</p> <p>To simplify the analysis, the Code allows the use of the following two patterns, Fig. E1.2:</p> <p>Factored dead load is applied on all spans and factored live load is applied as follows:</p> <p>(a) Maximum positive <math>M_u</math> near midspan occurs with factored live load on the span and on alternate spans.</p> <p>(b) Maximum negative <math>M_u</math> at a support occurs with factored live load on adjacent spans only.</p> |                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
|                                 | <p>Maximum <math>M_u^+</math></p>  <p>Maximum <math>M_u^-</math></p>  <p>Fig. E1.2—Live load loading pattern.</p>                                                                                                                                                                                                                                                                               |                                                                                                                                                                                                                                                                                                                                                                                                                                                                        |

| Step 3: Concrete and steel material requirements                                                                       |                                                                                                                                                                                                                                                                                                                                                                                                     |                                                                                                                                                                                                                                   |
|------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 7.2.2.1                                                                                                                | The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements of the Code. The designer determines the durability classes. Please refer to Chapter 4 of this Manual for an in-depth discussion of the categories and classes.                                                                                                                  | By specifying that the concrete mixture must be in accordance with ACI 301-10 and providing the exposure classes, Code Chapter 19 requirements are satisfied.                                                                     |
| 26.4.3.1                                                                                                               | <p>ACI 301 is a reference construction specification that is coordinated with the Code. The Code allows the use of ACI 301 for compliance of concrete mixture proportioning.</p> <p>There are several mixture options within ACI 301, such as the use of admixtures and supplementary cementitious materials, which the designer can require, permit, or review if suggested by the contractor.</p> | Based on durability and strength requirements, and experience with local mixtures, the compressive strength of concrete is specified at 28 days to be 5000 psi.                                                                   |
| 7.2.2.2                                                                                                                | <p>The reinforcement must satisfy Chapter 20 of the Code.</p> <p>The designer specifies the grade of bar and whether the reinforcing bar should be coated by epoxy, galvanized, or both.</p>                                                                                                                                                                                                        | <p>By specifying the reinforcing bar grade and any coatings, and that the reinforcing bar must be in accordance with ACI 301-10, Chapter 20 requirements are satisfied.</p> <p>Use ASTM A615 Grade 60 uncoated reinforcement.</p> |
| Step 4: Slab structural analysis                                                                                       |                                                                                                                                                                                                                                                                                                                                                                                                     |                                                                                                                                                                                                                                   |
| 6.3                                                                                                                    | System is braced by shear walls and moment-resisting frames. Assume slab is braced and that moment effects in slab caused by lateral loads may be ignored.                                                                                                                                                                                                                                          | <p>Modeling assumptions:</p> <p>Assume a constant moment of inertia for the entire length of the slab.</p> <p>Ignore torsional stiffness of beams.</p> <p>Only the slab at this level is considered.</p>                          |
| 6.6                                                                                                                    | Figure E1.3 shows the results of the first-order analysis of the slab for moments due to pattern gravity load. The figure shows a moment envelope that was developed by the software for various pattern loadings.                                                                                                                                                                                  | <p>Analysis approach:</p> <p>The connection to the beams is monolithic; however, when the slab is fully loaded, flexural cracking will reduce joint stiffness.</p>                                                                |
| Step 5: Required moment strength                                                                                       |                                                                                                                                                                                                                                                                                                                                                                                                     |                                                                                                                                                                                                                                   |
| 7.4.2                                                                                                                  | The negative design moments are taken at the face of support as is permitted by the Code (Fig. E1.3).                                                                                                                                                                                                                                                                                               |                                                                                                                                                                                                                                   |
|  <p>Fig. E1.3—Moment envelope.</p> |                                                                                                                                                                                                                                                                                                                                                                                                     |                                                                                                                                                                                                                                   |

The maximum positive moment is located in the end span, EF, and is 4.9 ft-kip. The inflection points for positive moments are 0.0 ft from the exterior support centerline and 2.25 ft from the first interior support centerline, column line (CL) E.

The maximum negative moment is located at the face of the first interior support from the right end (CL E) and is 6.3 ft-kip. The negative moment inflection point is 5.75 ft from the support centerline. On the left side and for the full length of the slab, there is no inflection point. Make top bars continuous over the full length of the slab with the exception of span E-F.

The maximum positive moment in the interior span, CL BC, is 3.8 ft-kip. The inflection points for positive moments are 1.5 ft from the first interior support centerline and 2.25 ft from the second interior support centerline. Because of pattern loading, a small negative moment can exist across all spans with the exception of the last span.

The maximum negative moment at the exterior left support, CL A, is 6 ft-kip because of the cantilevered slab.

**Table E.1—Maximum moments at supports and midspans**

| Required strength | Location from left to right along the slab |            |           |            |           |            |
|-------------------|--------------------------------------------|------------|-----------|------------|-----------|------------|
|                   | Support A                                  | Midspan AB | Support B | Midspan BC | Support C | Midspan CD |
| $M_u$ , ft-kip    | -6.0                                       | +2.7       | -4.7      | +3.8       | -5.8      | +3.6       |

Continue:

| Required strength | Location from left to right along the slab |            |           |            |           |
|-------------------|--------------------------------------------|------------|-----------|------------|-----------|
|                   | Support D                                  | Midspan DE | Support E | Midspan EF | Support F |
| $M_u$ , ft-kip    | -5.3                                       | +3.3       | -6.3      | +4.4       | 0         |

**Step 6: Required shear strength**

7.4.3.1 The maximum shear is taken at the support centerline for simplicity. The maximum shear under any load pattern is 2.4 kip (Fig E1.4).

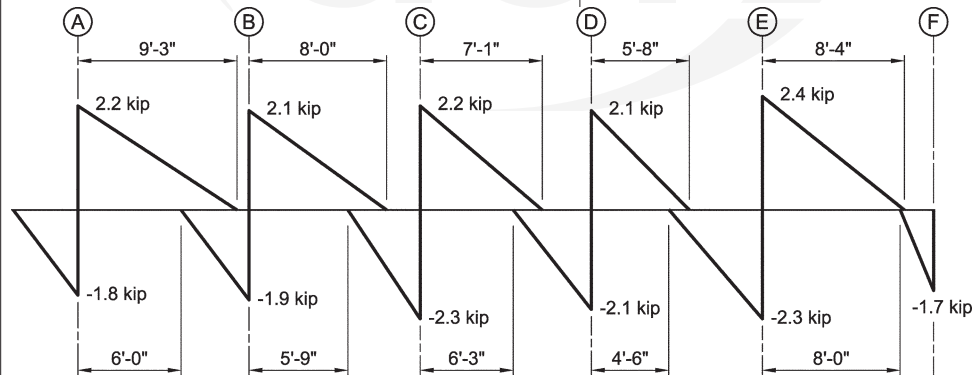


*Fig. E1.4—Shear envelope.*