

Guide to Simplified Design for Reinforced Concrete Buildings

(For Buildings of Limited Size and Height,
based on ACI 318-14 and ACI IPS-1, “Essential
Requirements for Reinforced Concrete
Buildings”)

Reported by ACI Committee 314



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Guide to Simplified Design for Reinforced Concrete Buildings

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(For Buildings of Limited Size and Height, based on ACI 318-14 and ACI IPS-1, “Essential Requirements for Reinforced Concrete Buildings”)

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This document is dedicated to the memory of late subcommittee member W. Gene Corley.

This guide presents simplified methods and design techniques that facilitate and speed the engineering of low-rise buildings within certain limitations. Material is presented in an order that follows typical design process with procedures introduced as the designer will need them in the course of a building design. Much of the information presented in this guide is derived from ACI 318, ASCE 7, and the 2015 International Building Code (IBC) (International Code Council 2015). The quality and testing of materials used in construction are covered by references to the appropriate ASTM standard specifications.

Whereas many of the tables, charts, and values included in this guide originated from the aforementioned reference documents, they have been modified or reorganized to be more conservative, to match design process flow, or better support the holistic and simplified design approach presented.

Although this guide is not written in mandatory language, the information is presented in such a manner that a structure designed following this guide will, in principle, comply with the codes and

standards on which it was based. Although this guide is written in nonmandatory language, it is meant to be applied as a whole, because the simplified provisions are interdependent, and it would be unsafe to employ only a portion of this guide and disregard the remainder. This guide is not a code and is not deemed to satisfy ACI 318, ASCE 7, and the International Building Code (International Code Council 2015). This guide is expected to be especially useful in the education and training of engineers in reinforced concrete design of low-rise structures of small to medium floor areas.

There are many options within these standards that are not considered in this guide, such as the use of supplementary cementitious materials in concrete mixtures. As this guide will be used as a design aid, it is the licensed design professional’s responsibility to ensure that the structure design satisfies the requirements of ACI 318, ASCE 7, the International Building Code (International Code Council 2015), and the legal requirements of the local jurisdiction. The original draft of the guide, published as ACI IPS-1 (2002), was produced by a Joint Committee of Instituto Colombiano de Normas Técnicas y Certificación (Colombian Institute for Technical Standards and Certification) (ICONTEC) and Asociación Colombiana de Ingeniería Sísmica (Colombian Association for Earthquake Engineering) (AIS).

The initial drafting of ACI IPS-1 (2002) was motivated by frequent worldwide discussions that reinforced concrete codes might be unnecessarily sophisticated for some applications, such as small low-rise buildings. Current knowledge of reinforced concrete behavior obtained through experimentation and experi-

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ACI 314R-16 supersedes ACI 314R-11 and became effective June 2016.

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ence, and its status and dissemination as a structural material used worldwide, made developing a simplified design and construction guide feasible. This guide used *ACI IPS-1 (2002)* as a basis, with information derived from *ACI 318*, *ASCE 7*, and the *International Building Code (International Code Council 2015)*.

This guide presents simplified approaches to assist engineers in designing low-rise buildings within certain limitations, in addition to the following:

- (a) Information on the order needed in the course of a design
- (b) Explanatory material at appropriate places
- (c) Computations only requiring a hand calculator
- (d) Graphs and graphical explanations
- (e) Design information based on simplified strength models
- (f) Other limit states accounted for by minimum dimensions
- (g) Conservative loads and simplified analysis guidelines
- (h) Simplified geotechnical information to help define soil-bearing capacity
- (i) Shear walls as the seismic-force-resisting system
- (j) Material and construction guidelines based on commonly available steel grades and medium-strength concrete that can be site mixed.

Keywords: concrete quality; foundation design; frame analysis; inspection; low-rise building construction; low-rise structure; mixing; placing; section analysis; seismic design; simplified design; specifications; structure design; structure layout.

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CHAPTER 17—REFERENCES, p. 124**APPENDIX A—COMPARISON OF ACI 314R-16 TO ACI 318-14, INTERNATIONAL BUILDING CODE (2015), AND ASCE 7-10, p. 125****CHAPTER 1—GENERAL****1.1—Scope**

This guide is intended for the planning, design, and construction of reinforced concrete structures in new low-rise buildings of restricted occupancy, number of stories, and area. Although the information presented was developed to produce, when properly used, a reinforced concrete structure with an appropriate margin of safety, this guide is not a replacement for a licensed design professional's experience and working knowledge. For the structure designed by this guide to attain the intended margin of safety, the guide should be used as a whole, and alternative procedures should be used only when explicitly permitted herein. The minimum dimensioning prescribed in the guide replace, in most cases, more detailed procedures prescribed in ACI 318, ASCE 7, and the International Building Code (International Code Council 2015).

1.2—Purpose

This guide provides a licensed design professional with sufficient information to design structural reinforced concrete members that comprise the structural framing of a low-rise building with the limits set in 1.3. Design rules set forth in this guide are simplifications that, when used together, comply with the more detailed requirements of ACI 318, ASCE 7, and the International Building Code (International Code Council 2015).

1.3—Limitations

This guide is only meant for buildings meeting all the limitations set forth in 1.3.1 to 1.3.10. These limits maintain the guide scope in close adherence to the collective experience of the original drafting committee (ICONTEC-AIS). Buildings within this scope are expected to have a normal rectangular footprint with simple standard geometries and member dimensions in both plan and elevation. Such buildings also depend primarily on reinforced concrete

Table 1.3.1.1—Permitted uses and occupancies

Occupancy group	Occupancy subgroup		Permitted
Group A—Assembly	A-1	Fixed-seating theaters, television, and radio studios	NO
	A-2	Building having an assembly room with capacity less than 100 persons and not having a stage	YES
	A-3		
	A-4	Arenas, skating rinks, swimming pools, and tennis courts	NO
	A-5	Amusement parks, bleachers, grandstands, and stadiums	NO
Group B—Business	B	Building for use as offices, or professional services containing eating and drinking establishments with less than 50 occupants	YES
Group E—Educational	E	Educational purposes with less than 500 students and staff	YES
Group F—Factory	F-1	Light industries not using heavy machinery	YES
	F-2	Heavy industries using heavy machinery	NO
Group H—Hazardous	H	Manufacturing, processing, generation, or storage of materials that constitute a physical or health hazard	NO
Group I—Institutional	I-1	Residential board and care facilities	YES
	I-2	Hospitals	NO
	I-3	Prisons, jails, reformatories, and detention centers	YES
	I-4	Daycare facilities	YES
Group M—Mercantile	M	Display and sale of merchandise	YES
Group R—Residential	R-1	Hotels having an assembly room with capacity less than 100 persons and not having a stage	YES
	R-2	Apartment buildings and dormitories	YES
	R-3	Houses	YES
	R-4	Residential care and assisted-living facilities	YES
Group S—Storage	S-1	Storage of heavy or hazardous materials	NO
	S-2	Storage of light materials	YES
Group U—Utility and miscellaneous	U	Utilities, water supply systems, and power-generating plants	NO
	U	Garages for vehicles with carrying capacity up to 4000 lb (1800 kg)	YES
	U	Garages for trucks of more than 4000 lb (1800 kg) carrying capacity	NO

structural walls for lateral load resistance. Observing these limits justifies the simplified analysis and design methods herein without the need for special analyses, including slenderness and second-order effects. Buildings with offsets, reentrant corners, and vertical or horizontal irregularities are outside the scope of this guide.

1.3.1 Use and occupancy

1.3.1.1 Permitted uses and occupancies—Table 1.3.1.1 lists building occupancy groups and subgroups, indicating for each whether the use of this guide is permitted.

1.3.1.2 Mixed occupancy—Recommendations described in this guide apply to cases involving only combinations for which the use of this guide is permitted, as identified in Table 1.3.1.1.

1.3.2 Maximum number of stories—Recommendations described in this guide apply to buildings with five or fewer stories above ground and no more than one basement level.

1.3.3 Maximum area per floor—The area per floor should not exceed 10,000 ft² (1000 m²).

1.3.4 Maximum story height—Story height, measured from floor finish to floor finish, should not exceed 13 ft (4 m).

1.3.5 Maximum span length—The span length for girders, beams, and slab-column systems, measured center-to-center of the supports, should not exceed 30 ft (10 m).

1.3.6 Maximum difference in span length—Spans should be approximately equal, and the shorter of two adjacent spans should be at least 80 percent of the larger span, except in elevator and stair cores. Refer to 7.9.1 for cores.

1.3.7 Minimum number of spans—There should be at least two spans in each of the two principal directions of the building in plan. Single spans may be permitted in one- and two-story buildings if the span length does not exceed 15 ft (5 m).

1.3.8 Maximum overhang—For girders, beams, and slabs with overhangs, the length of the overhang should not exceed one-third of the length of the first interior span of the member.

1.3.9 Maximum slope for slabs, girders, beams, and joists—When sloping slabs, girders, beams, or joists are used, the slope of the member should not exceed 15 degrees.

1.3.10 Maximum slope of the terrain—The slope of the terrain surrounding the building should not exceed 30 degrees (Fig. 1.3.10) or the ratio of the height of the first story to the smaller dimension of the building in plan.

1.4—Supporting codes and standards

For cases within the limits described in 1.3, this guide is intended to be a simplification complying with the following supporting codes and standards:

a) **ACI 318**

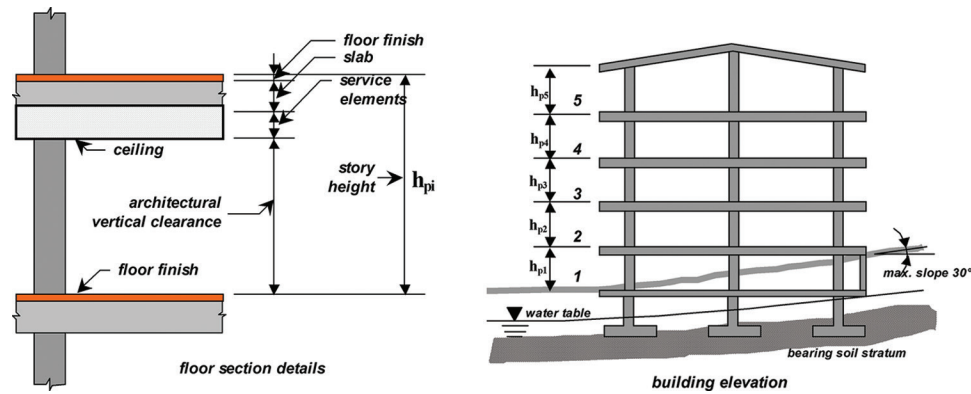


Fig. 1.3.10—General structural layout in elevation.

Table 1.5.1—Design and construction procedure steps

Step	Description	Related chapter(s)
A	Verification that the limitations for using the guide are met. Definition of the layout in plan and height of the structure.	1 and 3
B	Calculation of all gravity loads that act on the structure, excluding the self-weight of the structural members.	4
C	Definition of an appropriate floor system, depending on the span lengths and the magnitude of the gravity loads.	6
D	Selection of trial dimensions for the slab of the floor system. Calculation of the self-weight of the system and design of the members that comprise it, correcting the dimension if needed by the strength and serviceability limit states, complying with the limits for slab systems with beams, or slab-column systems.	6, 7, and 9
E	Trial dimensions for the beams and girders (if needed). Calculation of the self-weight of girders, beams, and joists. Flexural and shear design of the beams and girders, correcting the dimensions if needed by the strength and serviceability limit states.	6 and 8
F	Trial dimensions for the columns. Verification of column slenderness through the use of minimum dimensions. Calculation of the self-weight of the columns. Design of the columns for combination of axial load, moment, and shear. Correcting the dimensions if needed by the strength and serviceability limit states.	10
G	If lateral load, such as earthquake, wind, or lateral earth pressure, is beyond nominal, magnitude and application point are to be established; otherwise the designer may proceed to Step I.	4
H	Preliminary location and trial dimensions for reinforced concrete walls capable of resisting lateral loads. For earthquake loads, the influence of wall self-weight is evaluated. Flexure and shear design of the reinforced concrete walls.	11 and 12
I	Design of the stairways, ramps, small potable water tanks, and retaining walls.	13
J	Loads at the foundation level are determined. Definition of the foundation system is performed. Design of the structural members of the foundation.	14
K	Production of the structural drawings and specifications.	15
L	The structure is built complying with the construction and inspection requirements.	16

b) **ASCE 7**c) International Building Code (**International Code Council 2015**)

Other cases are not covered by this guide. Please refer to Table A.1 in **Appendix A** for a guide by section to corresponding topics in the supporting codes and standards.

1.5—Design and construction procedure

1.5.1 Procedure—The design procedure comprises the steps listed in Table 1.5.1. Refer also to Fig. 1.5.1a and 1.5.1b. Note that by conforming to the dimensional limits and cover of this guide, a 1-hour fire rating is achieved. This rating is usually sufficient for the permitted occupancies in this guide. Other fire

ratings are beyond the scope of this guide, and such designs should be performed using **ACI 318**, **ASCE 7**, and the International Building Code (International Code Council 2015).

1.5.2 Design documentation—The design steps should be recorded as follows.

1.5.2.1 Calculation record—The licensed design professional should document all design steps in a calculation record. This record should contain, at a minimum, the following:

- General structural program, as defined in **Chapter 3**
- Description of the structural system
- Loads
- Characteristics, strength, and fabrication standards for all structural materials

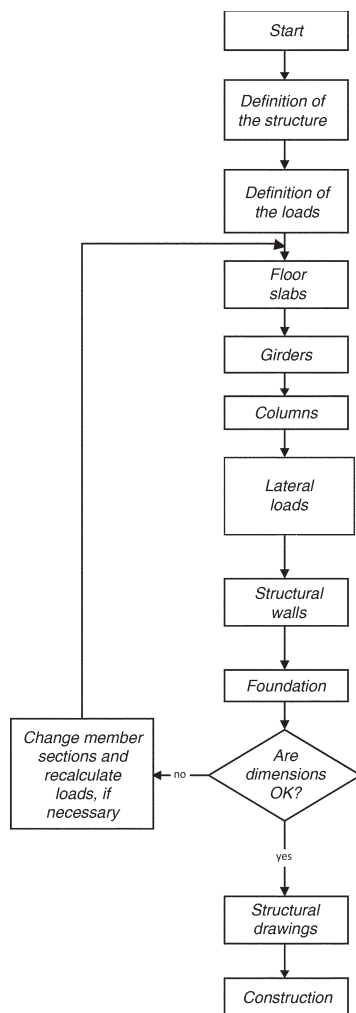


Fig. 1.5.1a—Design and construction procedure.

- (e) Justification of all design calculation
- (f) Sketches of the reinforcement layout for all structural members

1.5.2.2 Geotechnical report—The geotechnical report should record, at a minimum, the soil investigation performed, selected allowable bearing capacity of the soil, soil profile type, lateral soil pressures anticipated for design of any soil-retaining structures, and all other information indicated in **Chapters 4 and 14**.

1.5.2.3 Structural drawings—Structural drawings should include, at a minimum, all the plans indicated by Chapter 15 for construction of the building.

1.5.2.4 Project specifications—Project specifications should include, at a minimum, all the construction specifications described in **Chapter 15**.

1.5.3 Precast concrete components—Precast concrete components may be used, including prestressed concrete manufactured in offsite facilities. Such components should be designed by a licensed design professional in accordance with **ACI 318, ASCE 7**, and the International Building Code (**International Code Council 2015**). Calculations should be reviewed by the licensed design professional of record (1.2) and included in the calculation record (1.5.2.1). Detailing and placing drawings conforming to **15.2.2** should be

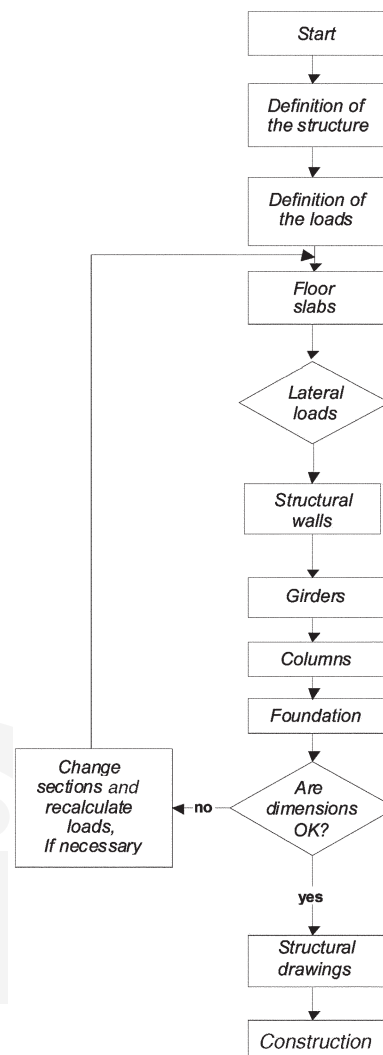


Fig. 1.5.1b—Design and construction procedure for earthquake regions.

furnished and included as part of the structural drawings (1.5.2.3). Manufacture of precast components should be done in a facility with demonstrated capability of producing quality products.

1.6—Limit states

The design approach of this guide is based on limit states, where a limit state is a condition beyond which a structure or member becomes unfit for service and is judged to be unsafe or no longer useful for its intended function. The designer should verify that the strength and serviceability limit states are accounted for in the resulting structure. The following are considered implicitly in the design procedure:

- (a) Structural integrity
- (b) Lateral load story drift
- (c) Durability
- (d) Fire resistance

1.7—Strength design

1.7.1 General—In strength design, the structure and the structural members are dimensioned to have design strengths

at all sections at least equal to the demands calculated for the combinations of factored loads described in Chapter 4.

The basic expression for the strength limit state is

$$\text{resistances} \geq \text{load effects} \quad (1.7.1a)$$

Because resistances may be less than computed and the load effects could be larger than computed, strength reduction factors ϕ less than 1, and load factors γ generally greater than 1, are used

$$\phi R_n \geq \gamma_1 S_1 + \gamma_2 S_2 + \dots \quad (1.7.1b)$$

where R_n is nominal strength, and S is load effects based on the loads described in Chapter 4. Therefore, the strength design requires that

$$\text{design strength} \geq \text{required strength} \quad (1.7.1c)$$

$$\phi (\text{nominal strength}) \geq U \quad (1.7.1d)$$

where the required strength is $U = \gamma_1 S_1 + \gamma_2 S_2 + \dots$

1.7.2 Required strength—The required strength U should be computed for the combinations of factored loads listed in 4.2.

1.7.3 Design strength—The design strength provided by a member, its connections to other members, and its cross sections in terms of flexure, axial load, and shear, is the nominal strength multiplied by a strength reduction factor ϕ . Nominal strength should be calculated for each particular force effect in each of the member types at the defined critical sections. The following strength reduction factors ϕ should be used:

- a) Flexure, without axial load: $\phi = 0.90$
- b) Axial tension and axial tension with flexure: $\phi = 0.90$
- c) Axial compression and axial compression with flexure:
 - i. Columns with ties and reinforced concrete walls: $\phi = 0.65$
 - ii. Columns with spiral reinforcement: $\phi = 0.75$
- d) Shear and torsion: $\phi = 0.75$
- e) Bearing of concrete: $\phi = 0.65$

1.8—Serviceability design

To ensure adequate response during service, follow the recommendations in this guide for limiting dimensions, cover, detailing, and construction. These serviceability conditions include effects such as:

- (a) Long-term environmental effects, including exposure to aggressive environment or corrosion of the reinforcement
- (b) Dimensional changes due to variations in temperature, relative humidity, and other effects
- (c) Excessive cracking of the concrete
- (d) Excessive vertical deflections
- (e) Excessive vibration

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_a = effective seismic peak ground horizontal acceleration in rock for short periods of vibration, expressed as a fraction of gravity g

A_b = area of an individual reinforcing bar or wire, in.² (mm²)
 A_{cb} = bearing area of concrete, in.² (mm²)
 A_{cs} = area of core of spirally reinforced compression member measured to outside diameter of spiral, in.² (mm²)
 A_f = contact area of footing with soil, ft² (m²)
 A_g = gross area of section, or area of concrete only excluding area of voids, in.² (mm²)
 A_i = area of additional hanger reinforcement where beams are supported by girders or other beams, in.² (mm²)
 A_j = effective cross-sectional shear area within a joint, in.² (mm²)
 A_p = component, or cladding, wind-exposed surface area, ft² (m²)
 A_s = area of longitudinal tension reinforcement, in.² (mm²)
 A'_s = area of longitudinal compression reinforcement, in.² (mm²)
 A_{se} = steel area at the extreme face of column or reinforced concrete wall, in.² (mm²)
 $A_{s,min}$ = minimum area of longitudinal tension reinforcement, in.² (mm²)
 A_{ss} = steel area at the side face of column or reinforced concrete wall, in.² (mm²)
 A_{st} = total area of longitudinal reinforcement, in.² (mm²)
 A_{su} = wind-exposed surface area, ft² (m²)
 A_v = area of shear reinforcement, in.² (mm²)
 a = depth of equivalent rectangular compressive stress block, in. (mm)
 a_w = distance from edge of wall footing to the resultant of soil reaction in wall footing, in. (mm)
 B_f = short horizontal dimension of footing, in. (mm)
 b = width of compression flange of member, or width of member, in. (mm)
 b_c = width of column section, and for punching shear evaluation, the smallest plan dimension of pedestal, column capital, or drop panel, or thickness change in stepped footings, in. (mm)
 b_f = width of compression face of member, in. (mm)
 b_o = perimeter of critical section for two-way shear (punching shear) in slabs, in. (mm)
 b_w = web width of section, or wall width, in. (mm)
 C_p = component, or cladding, wind surface pressure coefficient
 C_{su} = wind surface pressure coefficient
 C_{vx} = coefficient defined in 4.11.4 for design of seismic loads
 c_c = least distance from surface of reinforcement to the side face, in. (mm)
 D = dead loads or related internal moments and loads
 d = effective depth of section, taken as distance from extreme compression fiber to centroid of tension reinforcement, in. (mm)
 d' = distance from extreme compression fiber to centroid of compression reinforcement, in. (mm)
 d_b = nominal diameter of reinforcing bar or wire, in. (mm)
 d_c = distance from extreme tension fiber to centroid of tension reinforcement, in. (mm)
 d_s = outside diameter of spiral reinforcement, in. (mm)
 E = seismic loads or related internal moments and loads

E_c =	modulus of elasticity of concrete, psi (MPa)	h_n =	clear vertical distance between lateral supports of columns and walls, in. (mm)
e_B =	eccentricity of resultant applied to footing in direction parallel to B_f , in. (mm)	h_{pi} =	story height of floor i measured from floor finish of story to floor finish of story immediately below, ft (m)
e_H =	eccentricity of resultant applied to footing in direction parallel to H_f , in. (mm)	h_r =	mean roof height for wind design, measured over terrain, ft (m)
e_x =	eccentricity, measured in x-direction, between story center of lateral stiffness and application point of story lateral loads acting in y-direction, in. (mm)	h_s =	depth of soil against retaining wall, in. (mm)
e_y =	eccentricity, measured in y-direction, between story center of lateral stiffness and application point of story lateral loads acting in x-direction, in. (mm)	h_w =	height of wall from base to top, in. (mm)
F =	loads due to weight and pressure of fluids with well-defined densities and controllable maximum heights or related internal moments and loads	I_c =	moment of inertia of column section, in. ⁴ (mm ⁴)
F_a =	seismic site coefficient for short periods of vibration	K_a =	active soil pressure coefficient
F_{ac} =	total lateral active soil force, lb (kN)	K_o =	at-rest soil pressure coefficient
F_i, F_x =	wind or seismic force applied at level i or x , respectively, lb (kN)	K_p =	passive soil pressure coefficient
F_o =	total lateral at-rest soil force, lb (kN)	k_r =	story total rotational stiffness
F_{pw} =	equivalent static wind force for components and cladding acting normal to wind-exposed surface, lb (kN)	k_x, k_y =	wall lateral stiffness in direction x or y , respectively, lb/in. (N/mm) (Eq. (4.14.5a(a)) and Eq. (4.14.5a(b)))
F_{su} =	equivalent static wind force acting normal to wind-exposed surface, lb (kN)	L =	live loads or related internal moments and loads
F_{ui}, F_{ux} =	factored lateral force applied to wall at level i or x , respectively, lb (kN)	L_r =	roof live load or related internal moments and loads
f'_c =	specified compressive strength of concrete, psi (MPa)	ℓ_0 =	column confinement length, in. (mm)
$\sqrt{f'_c}$ =	square root of specified compressive strength of concrete; the result has units of psi (MPa)	ℓ_1 =	length of span in direction of moments, measured center-to-center of supports, in. (mm)
f_{cr}' =	required average compressive strength of concrete used as the basis for selection of concrete proportions, psi (MPa)	ℓ_2 =	length of span transverse to ℓ_1 , measured center-to-center of supports, in. (mm)
f_{cu} =	extreme fiber compressive stress due to factored loads at edges of structural walls, psi (MPa)	ℓ_a =	length of clear span in short direction of two-way slabs or in direction of moments, measured face-to-face of beams or other supports, in. (mm)
f_y =	specified yield strength of reinforcement, psi (MPa)	ℓ_c =	length of clear span in long direction of two-way slabs or systems, measured face-to-face of beams or other supports, in. (mm)
f_{ypr} =	probable specified maximum strength of reinforcement, psi (MPa) ($f_{ypr} = 1.25f_y$)	ℓ_d =	development length, in. (mm)
f_{yt} =	specified yield strength of transverse or spiral reinforcement, psi (MPa)	ℓ_n =	length of clear span, in long direction for two-way systems, measured face-to-face of beams or other supports, in. (mm)
g =	acceleration due to gravity, 386 in./s ² (9.8 m/s ² $\approx$ 10 m/s ²)	ℓ_{ps} =	factor to calculate punching shear strength (9.5.4.3)
H =	loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and loads	ℓ_s =	center-to-center span length; shortest distance between adjacent parallel column centerlines, in. (mm)
H_f =	long horizontal dimension of footing, in. (mm)	ℓ_w =	horizontal length of wall, in. (mm)
h =	overall depth or thickness of member, or height of section of member, or outside diameter of circular section, in. or ft (mm or m)	M_a =	factored moment in short direction in two-way slabs, lb·in. (N·m) per unit slab width
h_b =	vertical distance measured from bottom of supporting girder to bottom of supported beam, ft (m)	$M_{a \text{ or } b}^+$ =	factored positive moment at section, lb·in. (N·m) per unit slab width
h_c =	depth of column, or dimension of column in direction parallel to girder span; and for punching shear evaluation, the largest plan dimension of capital, drop panel, pedestal, or thickness change in stepped footings, in. (mm)	$M_{a \text{ or } b}^-$ =	factored negative moment at section, lb·in. (N·m) per unit slab width
h_f =	flange thickness, in. (mm)	M_b =	factored moment in long direction in two-way slabs, lb·in. (N·m) per unit slab width
h_g =	total depth of supporting girder, in. (mm)	M_{bn} =	nominal moment strength at section at balanced conditions, lb·in. (N·m)
h_i, h_x =	height above base to level i or x , respectively, ft (m)	M_i, M_x =	unfactored overturning moment due to lateral loads for story i or x , respectively, lb·in. (kN·m)
		M_{iu}, M_{xu} =	factored story moment due to lateral loads at story i or x , respectively, lb·in. (kN·m)
		M_n =	nominal moment strength at section, lb·in. (N·m)
		M_o =	total factored moment at section, lb·in. (kN·m)
		M_{ot} =	unfactored overturning moment due to lateral loads at base of structure, lb·in. (kN·m)
		M_{otu} =	factored overturning moment due to lateral loads at base of structure, lb·in. (kN·m)
		M_{pr} =	probable flexural strength of member, lb·in. (N·m) computed using f_{ypr} and ρ_{cr}

M_u =	factored moment at section, lb·in. (N·m)	p_{uw} =	factored design horizontal pressure for retaining walls, lb/ft ² (kPa or kN/m ²)
M_u^+ =	factored positive moment at section, lb·in. (N·m)	p_z =	at-rest, or active, lateral soil pressure at depth x , lb/ft ² (kPa or kN/m ²)
M_u^- =	factored negative moment at section, lb·in. (N·m)	q_a =	unfactored allowable bearing capacity of soil, lb/ft ² (kPa or kN/m ²)
M_{ux} =	factored moment at section in x direction, lb·in. (N·m)	q_c =	standard cone penetration resistance in cone penetration test, lb/ft ² (kPa or kN/m ²)
M_{uy} =	factored moment at section in y direction, lb·in. (N·m)	q_d =	unfactored dead load per unit area, lb/ft ² (kN/m ²)
M_{xu} =	factored overturning moment due to lateral loads for story x , lb·in. (kN·m)	q_h =	wind velocity pressure at height h over terrain, lb/ft ² (kN/m ²)
N =	number of blows in standard penetration test (SPT)	q_ℓ =	unfactored live load per unit area, lb/ft ² (kN/m ²)
$\bar{N}$ =	average soil SPT resistance, measured in number of blows per ft (0.305 m) of penetration, averaged over upper 100 ft (30 m) of soil profile	q_o =	overburden pressure, or unfactored gravity loads applied directly to slab in mat foundations, lb/ft ² (kN/m ²)
n_c =	number of interior columns in story in direction under consideration, for the entire structure	q_u =	factored load per unit area, lb/ft ² (kN/m ²)
n_e =	number of edge columns in story in direction under consideration, for the entire structure	q_{uc} =	unconfined compression strength of soil, lb/ft ² (kPa or kN/m ²)
n_s =	number of stories in the building above the base	q_{un} =	factored net soil reaction pressure on footing, lb/ft ² (kN/m ²)
P_{bn} =	nominal compression axial compression strength at section at balanced conditions, lb (N)	R =	rain load or related internal moments and loads
P_{cu} =	factored compression load on wall boundary element, including seismic effects, lb (N)	R_n =	nominal strength in terms of flexure, axial load, shear or bearing strength
P_d =	unfactored dead load axial force at section, lb (kN)	R_{pi} =	reaction from lateral soil pressure at story i , lb (kN)
P_ℓ =	unfactored live load axial force at section, lb (kN)	R_s =	response modification factor related to energy dissipation capacity in inelastic range of seismic-resistant structural system
P_n =	nominal axial compression strength at given eccentricity, lb (N)	R_u =	factored reaction from supported structural member, lb (kN)
$P_{n(max)}$ =	maximum compression nominal axial compression strength at section, lb (N)	r_u =	factored uniformly distributed reaction from slab on supporting girder, beam, or reinforced concrete wall, lb/ft (kN/m)
P_{on} =	nominal compression, without flexure, or axial compression strength at section, lb (N)	S =	snow load or related internal moments and loads
P_{ov} =	maximum vertical load applied to footing including wind or seismic overturning effects, lb (N)	S_a =	value of elastic acceleration design response spectrum, for damping ratio of 5 percent of critical, expressed as fraction of acceleration of gravity
P_m =	nominal tension, without flexure, or axial tension strength at section, lb (N)	S_{DS} =	value of design earthquake spectral acceleration parameter at short period
P_{tu} =	factored tension force on wall boundary element, including seismic effects, lb (N)	S_i =	nominal load effect based on load i
P_u =	factored axial or concentrated load, or factored axial load at given eccentricity, lb (N or kN)	s =	spacing of transverse reinforcement, stirrups, or wall reinforcement measured along axis of member, in. (mm)
P_{ub} =	factored axial load at base of column, lb (N)	s_j =	center-to-center spacing between parallel joists, in. (mm)
P_v =	maximum vertical load applied to footing not including wind or seismic, lb (N)	s_s =	sample standard deviation, psi (MPa)
PI =	soil plasticity index, equal to difference in percentage of moisture content at liquid limit and at plastic limit	s_{sk} =	spacing of skin reinforcement, in. (mm)
p =	unfactored pressure for braced retaining walls, psi or lb/ft ² (kPa or kN/m ²)	s_u =	shear strength of undrained cohesive soil, lb/ft ² (kPa or kN/m ²)
p_a =	unfactored active pressure, psi or lb/ft ² (kPa or kN/m ²)	T =	cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete
p_d =	unfactored concentrated dead load applied directly to member, lb (kN)	T_i, T_x =	unfactored story torsional moment due to lateral loads at story i or x , respectively, lb·in. (kN·m)
p_ℓ =	unfactored concentrated live load applied directly to member, lb (kN)	T_o =	unfactored story torsional moment due to lateral loads at base of structure, lb·in. (kN·m)
p_o =	unfactored pressure at rest, psi or lb/ft ² (kPa or kN/m ²)	T_{ou} =	factored story torsional moment due to lateral loads at base of structure, lb·in. (kN·m)
p_p =	unfactored passive pressure, lb/ft ² (kPa or kN/m ²)	T_u =	factored torsional moment at section, lb·in. (N·mm)
p_t =	unfactored vertical surcharge pressure on top of retaining wall backfill, lb/ft ² (kPa or kN/m ²)		
p_u =	factored concentrated load applied directly to member, lb (kN)		
p_{uw} =	factored design horizontal pressure for retaining walls caused by surcharge pressure on top of retaining wall backfill, lb/ft ² (kPa or kN/m ²)		

T_{xu}, T_{yu} = factored story torsional moment due to lateral loads at story i or x , respectively, lb·in. (kN·m)
 t_x, t_y = structural vertical member cross-sectional dimension in direction x or y , respectively, in. (mm)
 U = required strength to resist factored loads or related internal moments and loads
 V = basic wind speed, mph (m/s), corresponding to 3-second gust speed at 33 ft (10 m) above ground
 V_{bs} = seismic design base shear, lb (kN)
 V_c = nominal shear strength provided by concrete, lb (N)
 V_i, V_x = unfactored story shear due to lateral loads at story i or x , respectively, lb (kN)
 V_{iu}, V_{xu} = factored story shear due to lateral loads at story i or x , respectively, lb (kN)
 V_n = nominal shear strength at section, lb (N)
 V_o = unfactored story shear due to lateral loads at base of structure, lb (kN)
 V_{ou} = factored story shear due to lateral loads at base of structure, lb (kN)
 V_s = nominal shear strength at section provided by transverse reinforcement, lb (kN)
 V_{sw} = contribution of the horizontal wall reinforcement to the nominal shear strength at section, lb (kN)
 V_u = factored shear, lb (N)
 V_w = wind design base shear, lb (kN)
 W = wind loads or related internal moments and loads
 W_s = total building weight for seismic design, lb (kN)
 W_u = total factored uniformly distributed design load per unit member length, lb/in. (kN/m)
 W_{uf} = total factored uniformly distributed design load per unit member length, lb/ft (kN/m)
 w = moisture content of the soil, percentage
 w_d = unfactored dead load per unit member length applied directly to the member, lb/in. (kN/m)
 w_{df} = unfactored dead load per unit member length applied directly to the member, lb/ft (kN/m)
 w_i, w_x = part of W_s corresponding to story i or x , respectively, lb (kN)
 w_ℓ = unfactored live load per unit member length applied directly to the member, lb/in. (kN/m)
 w_u = factored load per unit member length applied directly to the member, lb/ft (kN/m)
 $\bar{x}, \bar{y}$ = story lateral stiffness center coordinates in directions x and y , respectively, in. (mm)
 z = depth of soil, ft (m)
 α_a = fraction of load acting in short direction in two-way slabs-on-girders
 α_b = fraction of load acting in long direction in two-way slabs-on-girders
 α_c = coefficient defining the concrete strength contribution to wall shear strength
 α_f = parameter of Eq. (5.11.4.2)
 α_{sh} = factor affecting equivalent shear force due to unbalanced moment at column-slab connection in Eq. (9.5.4.4)
 α_w = horizontal angle between normal to wind exposed surface and wind direction, degrees

β = ratio of clear spans in long to short direction of two-way slabs
 β_f = ratio of long side to short side of footing
 β_w = vertical angle between normal to wind exposed surface and horizontal line, degrees
 ΔM_u = factored unbalanced moment at column-girder joint or wall-girder joint, lb·in. (N·mm)
 ΔM_{u-ad} = additional factored unbalanced moment at column-slab connection, lb·in. (N·mm)
 ΔV_e = factored design shear force from development of probable flexural capacity of member at faces of joints, lb (N)
 ΔV_u = factored shear force due to unbalanced moment at column-slab connection, lb (N)
 ΔV_{ut} = increase in wall factored shear due to torsional effects, lb (kN)
 ϕ = strength reduction factor
 ϕ_s = angle of internal friction of soil
 γ = unit weight of material or soil, lb/ft³ (kN/m³)
 γ_i = load factor for load effect i
 ρ = ratio of longitudinal tension reinforcement $A_s/(bd)$
 ρ' = ratio of longitudinal compression reinforcement $A_s'/(bd)$
 ρ_ℓ = ratio of total longitudinal reinforcement area to gross concrete section area $A_{st}/(bh)$
 ρ_{max} = maximum ratio of longitudinal flexural tension reinforcement
 ρ_{min} = minimum ratio of longitudinal flexural tension reinforcement
 ρ_s = ratio of volume of spiral reinforcement to core volume confined by spiral reinforcement (measured out-to-out)
 ρ_t = ratio of horizontal shear reinforcement area to gross concrete area of vertical section
 ρ_{vw} = ratio of vertical reinforcement in reinforced concrete walls
 $\sum M_c$ = sum of nominal flexural strengths (M_n) of columns framing into joint, lb·in. (N·m)
 $\sum M_g$ = sum of nominal flexural strengths (M_n) of girders framing into joint, lb·in. (N·m)
 $\sum P_u$ = sum of factored concentrated loads within span, lb (kN)
 $\sum R_u$ = sum of factored reactions from supported structural members at same story, lb (kN)

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <https://www.concrete.org/store/productdetail.aspx?ItemID=CT13>. Definitions provided herein complement that resource.

admixture—material other than water, aggregate, or hydraulic cement, used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.

aggregate—granular material, such as sand, gravel, crushed stone, and iron blast-furnace slag, used with a cementing medium to form a hydraulic cement concrete or mortar.

allowable bearing capacity—maximum pressure to which a soil or other material should be subjected to guard against shear failure or excessive settlement.

anchorage—device embedded in concrete for the purpose of providing a connection to another member or structure.

base of structure—level at which the horizontal earthquake ground motions are assumed to be imparted to a building; this level does not necessarily coincide with the ground level.

beam—structural member subjected to axial load and flexure but primarily to flexure. See also **girder**.

bending moment—bending effect at any section of a structural element; it is equal to the algebraic sum of the moments of the internal compression and tension forces acting at the section, with respect to the centroidal axis of a member, on a free body of the member.

boulders—coarse materials more than 8 in. (200 mm) in diameter.

boundary element—portion along wall edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in thickness of the wall.

cement—any of a number of materials that are capable of binding aggregate particles together.

clay—natural mineral material having plastic properties and composed of very fine particles; the clay mineral fraction of a soil is usually considered to be the portion consisting of particles finer than 8×10^{-5} in. (2 μ m); clay minerals are essentially hydrous aluminum silicates or occasionally hydrous magnesium silicates.

coarse-grained soil—soil in which the larger grain sizes, such as sand and gravel, predominate.

column—member with a ratio of height-to-least lateral dimension exceeding 3 used primarily to support axial compressive load.

collector element—element that acts in axial tension or compression to transmit earthquake-induced loads between a structural diaphragm and a vertical element of the seismic-force-resisting system.

combined footing—structural unit or assembly of units supporting more than one column.

compression flange—the widened portion of an I, T, or similar cross section that is shortened or compressed by bending under normal loads, such as the horizontal portion of the cross section of a simple span T-beam.

compression reinforcement—reinforcement designed to carry compressive stresses.

concrete—mixture of portland cement and any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

concrete cover—least distance between the surface of embedded reinforcement and the closest outer surface of the concrete.

concrete mixture proportioning—proportions of ingredients that make the most economical use of available materials to produce mortar or concrete of the required properties.

confinement stirrup or tie—see also **hoop**.

construction documents—written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with the standard.

contraction joint—formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

corrosion—destruction of metal by a chemical, electrochemical, or electrolytic reaction within its environment.

crosstie—continuous reinforcing bar having a seismic hook at one end and a hook not less than 90 degrees with at least a six-diameter extension at the other end; the hooks shall engage peripheral longitudinal bars; the 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

curing—action taken to maintain moisture and temperature conditions in a freshly placed cementitious mixture to allow hydraulic cement hydration and (if applicable) pozzolanic reactions to occur so that the potential properties of the mixture may develop.

curtain wall—walls that are part of the façade or enclosure of the building that do not form part of the gravity- or lateral-load-resisting system.

deformed reinforcement—metal bars, wire, or fabric with a manufactured pattern of surface ridges that provide a locking anchorage with surrounding concrete (deformed reinforcement includes deformed reinforcing bars, welded plain wire fabric, and welded deformed wire fabric conforming to the appropriate ASTM standards).

depth of member, h —distance in a flexural member, measured from extreme compression fiber to the extreme tension fiber.

design strength—nominal strength multiplied by a strength reduction factor ϕ .

development length—length of embedded reinforcement required to develop the design strength of reinforcement at a critical section.

development length for a bar with a standard hook—shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90- or 180-degree hook.

diaphragm—structural member, such as a floor or roof slab, that transmits loads acting in the plane of the member to the vertical elements of the seismic-force-resisting system.

differential settlement—lowering in elevation of various parts of a foundation by different amounts.

effective depth of section, d —distance measured from extreme compression fiber to centroid of tension reinforcement.

embedment length—length of embedded reinforcement provided beyond a critical section.

factored load—load, multiplied by appropriate load factors, used to proportion members by the strength design method of this guide.

fine-grained soil—soil in which the smaller grain sizes predominate, such as fine sand, silt, and clay.

fire resistance—property of a material or assembly to withstand fire or give protection from it; as applied to elements of buildings, it is characterized by the ability to confine a fire or, when exposed to fire, to continue to perform a given structural function, or both.

flexural reinforcement—reinforcement provided to resist the tension and compression stresses induced by flexural moments acting on the member section.

floor system—structural members that comprise the floor of a story in a building, including girders, beams, joists, and the slab that spans between them, or the slab only where it is directly supported on columns, as in slab-column systems.

footing—structural element of a foundation that transmits loads directly to the soil.

formwork—total system of support for freshly placed concrete, including the mold or sheathing in contact with the concrete as well as supporting members, hardware, and necessary bracing.

foundation—system of structural elements that transmit loads from the structure above to the earth.

foundation beam (grade beam)—reinforced concrete beam, usually at ground level, that strengthens or stiffens the foundation or supports overlying construction.

girder—large beam, usually horizontal, that serves as a main structural member often supporting reactions from other beams or girders. See also **beam**.

gravel—

1. Granular material predominantly retained on the No. 4 (4.75 mm) sieve and resulting either from natural disintegration and abrasion of rock or processing of weakly bound conglomerate; and

2. That portion of an aggregate retained on the No. 4 (4.75 mm) sieve resulting either from natural disintegration and abrasion of rock or processing of weakly bound conglomerate.

gravity loads—loads that act downward and are caused by the acceleration of gravity acting on the mass of the elements and content that cause the dead and live loads.

hook—bend in the end of a reinforcing bar.

hoop—closed tie or continuously wound tie; a closed tie can be made up of several reinforcement elements each having seismic hooks at both ends, and a continuously wound tie shall have a seismic hook at both ends.

isolation joint—separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

joist—comparatively narrow beam used in closely spaced parallel arrangements to support floor or roof slabs.

lap splice—connection of reinforcing steel made by lapping the ends of bars.

lateral-force-resisting system—portion of the structure composed of members proportioned to resist loads related to lateral loads.

lateral seismic load—lateral load corresponding to the appropriate distribution of the base shear force for seismic-resistant design.

licensed design professional—individual who is licensed to practice structural design as defined by the statutory requirements of the professional licensing laws of the state or jurisdiction in which the project is to be constructed and who is in responsible charge of the structural design.

lightweight aggregate—aggregate meeting the requirements of **ASTM C330/C330M** and having a loose bulk density of 70 lb/ft³ (1120 kg/m³) or less, determined in accordance with **ASTM C29/C29M** (concrete manufactured using lightweight aggregate is not covered by this guide).

lightweight concrete—concrete containing lightweight aggregate and an equilibrium density, as determined by **ASTM C567/C567M**, between 90 and 115 lb/ft³ (1440 to 1840 kg/m³) (this type of concrete is not covered by this guide).

limit state—condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

live load—live load specified by general building code (without load factor).

load—loads or other actions that result from the weight of all building materials, occupants and other variable or permanent contents, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads.

load combinations—combinations of factored loads and loads.

load effects—loads and deformations produced in structural members by the applied loads.

load factor—factor by which a service load is multiplied to determine a factored load used in the strength design method.

longitudinal reinforcement—reinforcement parallel to the length of a concrete member.

mat foundation—continuous footing supporting an array of columns in several rows in each direction, having a slab-like shape with or without depressions or openings.

modulus of elasticity—ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

negative moment—condition of flexure in which top fibers of a horizontally placed member, or external fibers of a vertically placed exterior member, are subjected to tensile stresses.

negative reinforcement—steel reinforcement for negative moment.

nominal bar diameter—value computed using the nominal bar area.

nominal strength—strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of this guide before application of any strength reduction factors.

nonstructural elements—architectural, mechanical, and electrical components and systems permanently attached to the building.

occupancy—purpose for which a building or other structure, or part thereof, is used or intended to be used.

partitions—nonstructural interior wall that spans horizontally or vertically from support to support; supports may be the basic building frame, subsidiary structural members, or other portions of the partition system.

pedestal—upright compression member with a ratio of unsupported height to average least lateral dimension not exceeding 3.

permanent load—load in which variations over time are rare or of small magnitude.

pile—structural element that is driven, jetted, or otherwise embedded on end in the ground to resist loads or compact the soil.

pile cap—concrete element that transfers load from a column or pedestal to the top of one or more supporting piles.

plain concrete—structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete (plain concrete is not covered by this guide).

plain reinforcement—reinforcement without surface deformations, or having deformations that do not conform to the applicable requirements.

positive moment—condition of flexure in which, for a horizontal simply supported member, the deflected shape is normally considered to be concave downward and the top fibers subjected to compression stresses; for other members and other conditions, consider positive and negative as relative terms.

positive reinforcement—steel reinforcement to resist positive bending moment.

precast concrete—structural concrete element cast elsewhere than its final position in the structure (precast concrete is not covered by the guide, except as described in 1.5.3).

prestressed concrete—structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads (prestressed concrete is not covered by the guide, except as described in 1.5.3).

project drawings—drawings that, along with the project specifications, complete the descriptive information for constructing the work required in the construction documents.

project specifications—written documents that specify requirements for a project in accordance with the service parameters and other specific criteria established by the owner.

reinforced concrete—structural concrete reinforced with no less than the minimum amounts of reinforcement.

reinforced concrete wall—structural concrete wall reinforced with no less than the minimum amount of prestressing steel or nonprestressed reinforcement as specified in the applicable building code; a shear wall is a reinforced concrete wall.

reinforcement—deformed steel bars, wire, or wire mesh, embedded in concrete in such a manner that it and the concrete act together in resisting loads.

reshores—shores placed snugly under a concrete slab or other structural member after the original forms and shores have been removed to allow the new slab or structural member to deflect and support its own weight.

required strength—strength of a member or cross section required to resist factored loads or related internal moments and loads in specified load combinations.

retaining wall—wall built to hold back earth.

sand—

1. Granular material passing the 3/8 in. (9.5 mm) sieve and almost entirely passing the No. 4 (4.75 mm) sieve and predominantly retained on the No. 200 (75 μ m) sieve, and resulting either from natural disintegration and abrasion of rock or processing of completely friable sandstone; and

2. That portion of an aggregate passing the No. 4 (4.75 mm) sieve and predominantly retained on the No. 200 (75 μ m) sieve, and resulting either from natural disintegration and abrasion of rock or processing of completely friable sandstone.

seismic hook—hook on a stirrup, or crosstie having a bend not less than 135 degrees, except that circular hoops shall have a bend not less than 90 degrees; hooks shall have a $6d_b$ (but not less than 3 in. [75 mm]) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

self-weight—weight of the structural member, caused by the material that composes the member.

service load—all loads, static or transitory, imposed on a structure, or element thereof, during operation of a facility (without load factors).

settlement—downward movement of the supporting soil.

shear—internal force tangential to the plane on which it acts.

shear reinforcement—reinforcement designed to resist shear or diagonal tension stresses.

shore—vertical or inclined support member designed to carry the weight of the formwork, concrete, and construction loads above.

shrinkage and temperature reinforcement—reinforcement provided to resist shrinkage and temperature stresses in concrete.

silt—granular material resulting from the disintegration of rock, with grains largely passing a No. 200 (75 μ m) sieve; alternatively, such particles in the range from 8×10^{-5} to 0.002 in. (2 to 50 μ m) diameter.

slab—molded layer of reinforced concrete, flat, horizontal (or nearly so), usually of uniform but sometimes of variable thickness, either on the ground or supported by beams, columns, walls, or other framework.

slab-on-ground—shallow foundation consisting of a continuous concrete slab, placed over native soil or engineered subgrade, where loads are locally distributed and transmitted to the ground.

soil—generic term for unconsolidated natural surface material above bedrock.

soil-bearing capacity—maximum stress under a foundation that provides adequate safety against soil failure and against excessive soil settlement.

solid slab—slab of uniform thickness.

span length—horizontal distance between supports of a horizontal structural member such as a slab, joist, beam, or girder, measured center to center of support.

specifications—written document describing in detail the scope of work, materials to be used, method of installation, and quality of workmanship.

specified compressive strength of concrete—compressive cylinder strength of concrete at 28 days used in design and evaluated in accordance with the appropriate ASTM standards, expressed in psi (MPa); whenever the quantity f'_c is under a radical sign ($\sqrt{f'_c}$), square root of numerical value only is intended, and the result maintains the units as psi (MPa).

spread footing—generally rectangular prism of concrete, larger in lateral dimensions than the column or wall it supports, to distribute the load of a column or wall to the subgrade.

spiral reinforcement—continuously wound reinforcement in the form of a cylindrical helix.

stirrup—reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire fabric (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement; usually applied to lateral reinforcement in beams and girders, ties usually refers to those in columns.

story height—vertical distance from the floor finish of a story to the floor finish of the story below.

strength design method—method of member proportioning based on ensuring that the design strength obtained by reducing the nominal strength is larger than the required strength obtained by applying load factors to service loads.

strength reduction factor ϕ —capacity reduction factor in structural design; a number less than 1.0 by which the nominal strength of a structural member or element in terms of load, moment, shear, or stress is required to be multiplied to determine design strength or capacity.

stress—intensity of force per unit area.

structural concrete—all concrete used for structural purposes, including prestressed and reinforced concrete, and, under special circumstances, plain concrete.

structural integrity—design concept that after an overload event or after damage occurs to a major supporting member, the structure has sufficient toughness to confine the damage to a local area and sufficient overall stability to prevent immediate collapse.

tank—container for the storage of water or other fluids (this guide only covers tanks used for storage of potable water in locations where the water supply system is unreliable).

tension reinforcement—reinforcement designed to carry tensile stresses such as those in the bottom of a simple beam.

tie—loop of reinforcing bar or wire enclosing longitudinal reinforcement; a continuously wound bar or wire in the form of a circle, rectangle, or other polygon shape without reentrant corners is acceptable.

tie elements—elements that serve to transmit inertia loads and prevent separation of building components such as footings and walls.

transverse reinforcement—reinforcement located perpendicular to the longitudinal axis of the member, comprising stirrups, ties, and spiral reinforcement.

wall—member, usually vertical, used to enclose or separate spaces. See also **reinforced concrete wall**.

web—thin vertical portion of an I-shaped section that connects the flanges.

wind load—nominal pressure of wind to be used in design.

wire—reinforcing bar of small diameter.

wire mesh—welded wire reinforcement.

work—entire construction, or separately identifiable parts thereof, that are required to be furnished under the construction documents.

working stress—maximum permissible design stress using working-stress design methods.

yield strength f_y —specified minimum yield strength or yield point of reinforcement; yield strength or yield point should be determined in tension according to applicable ASTM standards.

CHAPTER 3—STRUCTURAL SYSTEM LAYOUT

3.1—Description of structural components

The building structure should be divided into components as described by 3.1.1 through 3.1.5.

3.1.1 Floor system—The floor system consists of structural members that comprise the floor of a story in a building. **Chapter 6** describes different types of floor systems. The floor system can include girders, beams, joists, and the slab that spans between them or the slab only, where it is directly supported by columns, as in slab-column systems.

3.1.2 Vertical supporting members—Support the floor system at each story and carry accumulated gravity loads down to the foundation of the structure. Vertical supporting members should be either columns or reinforced concrete walls.

3.1.3 Foundation—Comprises structural elements through which the load of a structure is transmitted to the earth. It includes members such as spread footings, combined footings, foundation mats, basement and retaining walls, and grade beams. Foundation members are described in **Chapter 14**. Deep foundations, such as piles, caissons, pile footings, and caps, are beyond the scope of the guide.

3.1.4 Lateral-force-resisting system—Comprises the structural members that, acting jointly, resist and transmit to the ground the lateral loads arising from seismic motions, wind, and lateral earth pressure. The floor system acts as a diaphragm that carries in its plane the lateral force from the application point to the vertical members of the lateral-force-resisting system. Vertical members of the lateral-force-resisting system, in turn, collect the loads arising from all floors and transmit them down to the foundation and through the foundation to the underlying soil. For areas with moderate or high seismic risk, the main vertical members

of the lateral-force-resisting system should be reinforced concrete walls.

3.1.5 Other structural members—Other parts of the building structure that are covered in this guide are stairways, ramps, small potable water tanks, and slabs-on-ground.

3.2—General

3.2.1 Architectural program—A general architectural program of the building should be coordinated with the licensed design professional before the structural design begins. The general architectural program should include the following items as a minimum:

- Plan geometry and dimensions of all building floors
- Building elevation and the terrain, including the base-ment, if any
- Type of roof, its shape and slopes, the type of water-proofing, the means to facilitate the runoff of water from rain and melting snow or hail, and the location of drainage gutters
- Use of internal spaces of the building, its subdivision, and means of separation, in all stories
- Minimum architectural clear height in all floors
- Location and dimensions of stairways, ramps, and elevators
- Type of building enclosure, internal partitions, and architectural and nonstructural elements
- Locations of ducts and shafts for utilities such as power supply, lighting, thermal control, ventilation, water supply, and wastewater, including enough information to detect interference with the structural members
- Architectural features that may reduce effective concrete cover of reinforcement

3.2.2 Structural program—Based on the general architectural program information, the licensed design professional should define the general structural program for the building being designed. The general structural program includes the following items as a minimum:

- Intended use of the building
- Nominal loads related to the use of the building
- Special loads defined by the owner
- Design seismic loads, if the building is located in a seismic zone
- Wind loads appropriate for the site
- Loads from snow, hail, or rain
- Fire rating
- Type of roof and associated loads when not built of reinforced concrete
- Site information related to slopes and site drainage
- Allowable soil-bearing capacity and recommended foundation system derived from the geotechnical investigation and additional restrictions related to expected settlement
- Environmental conditions derived from local seasonal and daily temperature variations, humidity, presence of deleterious chemicals, and salts
- Availability, type, and quality of materials such as reinforcing bars, cement, and aggregates
- Availability of formwork materials

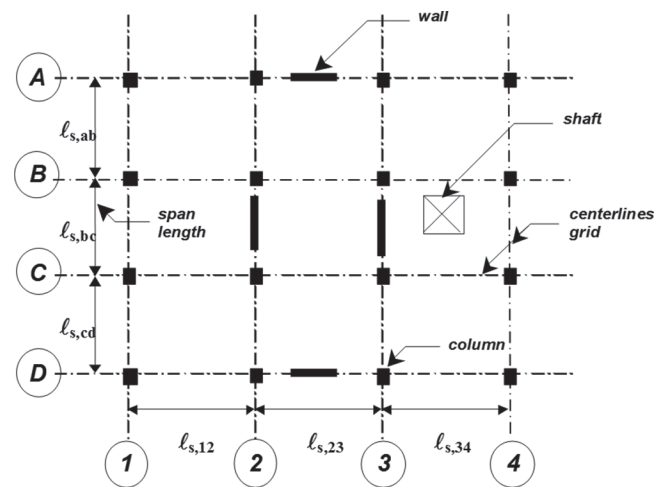


Fig. 3.3.1—General structural layout in plan.

- Availability of a testing lab for concrete mixture proportioning and quality control during construction
- Availability of qualified work force
- General and local sustainable construction practices

3.3—Structural layout

3.3.1 General structural layout—The licensed design professional should define a general structural layout in plan, including all information common to all structure levels of the structure (Fig. 3.3.1). The general structural layout in plan should include:

- A dimensioned axis grid, or centerlines, in both principal directions in plan, located at the intersection of the vertical supporting members (columns and reinforced concrete walls)
- Location in plan of all vertical supporting members, columns, and reinforced concrete walls. These vertical supporting members should be aligned vertically and be continuous to the foundation. Reinforced concrete walls are permitted if they are continuous to the foundation and have no openings for windows or doors.
- Location of all ducts, shafts, elevators, and stairways that are continuous from floor to floor
- Horizontal distance between centerlines, ℓ_s , which corresponds to the center-to-center span lengths of the floor system
- Location and distribution of all reinforced concrete walls

3.3.2 Floor layout—For each floor, the licensed design professional should develop a structural floor layout (Fig. 3.3.2). The structural floor layout includes:

- Location of the floor perimeter on the general axis grid
- Girder and beam locations, or column and middle strips for slab-column systems
- All substantial architectural openings in the floor
- An approximate load path from all floor areas to the supporting beams and girders

3.3.3 Vertical layout—The licensed design professional should define a general structural layout in elevation. (Fig. 1.3.10). The general structural layout in elevation includes:

- Number of stories



Documents

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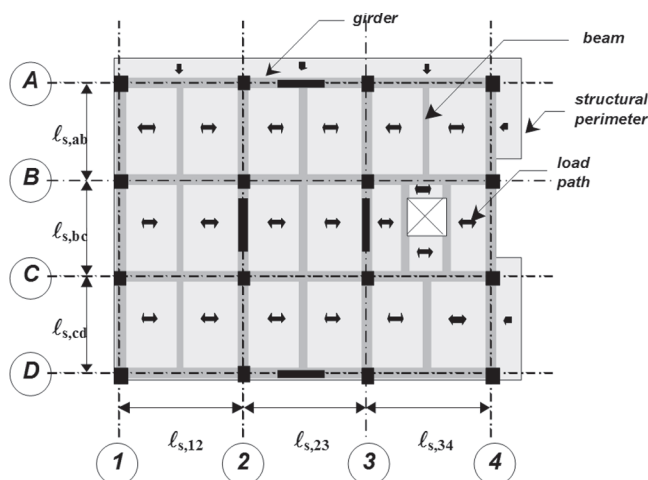


Fig. 3.3.2—Typical floor structural layout.

- b) Story height for each floor, defined as the vertical distance from floor finish to floor finish
- c) Slope and shape of the roof
- d) Architectural clearance from floor finish to ceiling for each floor
- e) Space necessary to accommodate power distribution, water supply and drainage, and heating, ventilation, and air conditioning
- f) Slope of the terrain and its relationship to the ground floor or basement
- g) Supporting soil stratum depth and water-table depth

3.4—Feasibility of guide usage

If any of the conditions stated in Chapter 3 are not met, the structural design should be performed using **ACI 318**, **ASCE 7**, and the International Building Code (**International Code Council 2015**).

CHAPTER 4—LOADS

4.1—General

The load information in this chapter is based on the requirements of the model code (International Building Code [International Code Council 2015]) and the reference standard (ASCE 7).

In jurisdictions where the local governing authority has adopted other standards, they should be used rather than the load information in this chapter.

4.2—Load factors and load combinations

The largest required factored strength U , as defined in 1.7.1, for the structural member should be determined by using load factors and load combinations of this section. The following apply:

- a) Each relevant strength limit state should be investigated, including effects of one or more loads not acting simultaneously.
- b) In load combinations where the symbol “ $\pm$ ” is used in factoring alternating loads that act in one direction or the opposite, it should be interpreted as the sign that leads to the maximum (positive) or minimum (negative) value of U .

c) The most unfavorable effects from both wind and seismic loads should be investigated, but they need not be considered to act simultaneously.

4.2.1 Dead and live load—Required factored strength U to resist dead load D and live load L should be the greater of

$$U = 1.4D \quad (4.2.1a)$$

$$U = 1.2D + 1.6L \quad (4.2.1b)$$

4.2.2 Rain load, snow load, and sloping roof live load—Required factored strength U to resist rain load R , snow load S , or sloping roof live load L_r should be evaluated based on the following load combinations

$$U = 1.2D + 1.6L + 0.5(R \text{ or } S \text{ or } L_r) \quad (4.2.2a)$$

$$U = 1.2D + 1.0L + 1.6(R \text{ or } S \text{ or } L_r) \quad (4.2.2b)$$

4.2.3 Wind loads—Required factored strength U to resist wind loads W should be evaluated based on the following load combinations

$$U = 1.2D + 1.0L \pm 1.0W + 0.5(R \text{ or } S \text{ or } L_r) \quad (4.2.3a)$$

$$U = 1.2D \pm 0.5W + 1.6(R \text{ or } S \text{ or } L_r) \quad (4.2.3b)$$

$$U = 0.9D \pm 1.0W \quad (4.2.3c)$$

Note: In ASCE 7-10, wind forces are defined at strength level. In previous versions of ASCE 7, wind forces were defined at working stress level. In jurisdictions where the local governing authority has adopted a standard that is different from ASCE 7, where wind forces are defined at working stress level, the factor for W in Eq. (4.2.3a) and Eq. (4.2.3c) should be 1.6 instead of 1.0, and in Eq. (4.2.3b) should be 0.8 instead of 0.5.

4.2.4 Seismic loads—Required factored strength U to resist seismic loads E should be evaluated based on the following load combinations

$$U = 1.2D \pm 1.0L + 0.2S \pm 1.0E \quad (4.2.4a)$$

$$U = 0.9D \pm 1.0E \quad (4.2.4b)$$

4.2.5 Earth pressure—For members resisting lateral earth pressure H , the required factored strength U should be at least equal to

$$U = 1.2D \pm 1.6L + 1.6H \quad (4.2.5)$$

When the building structure as a whole resists permanent uncompensated horizontal loads due to lateral soil pressure, $1.6H$ should be added to the right side of Eq. (4.2.3c) and Eq. (4.2.4b).

4.2.6 Weight and pressure of fluids—For members subjected to weight and pressure of fluids with well-defined densities

Table 4.4—Minimum unit weight γ for evaluation of dead and live loads from materials

Material	Unit weight γ , lb/ft ³	Mass density μ , kg/m ³	Material	Unit weight γ , lb/ft ³	Mass density μ , kg/m ³
Aluminum	170	2700	<i>Iron</i>		
<i>Bituminous products</i>			Cast	450	7200
Asphalt and tar	81	1300	Wrought	480	7700
Gasoline	42	700	Lead	710	11,400
Graphite	135	2160	<i>Lime</i>		
Paraffin	56	900	Hydrated, loose	32	500
Petroleum	53	850	Hydrated, compacted	45	800
Brass	526	8430	Masonry, brick (solid portion)	115	1850
Bronze	552	8850	Masonry, concrete (solid portion)	125	2150
Cement, portland, loose	90	1440	Masonry grout	140	2250
Ceramic tile	150	2400	Masonry, stone	162	2600
Charcoal	12	200	Mortar cement or lime	130	2100
Cinder fill	57	920	Particleboard	45	750
Coal, piled	50	800	Plywood	36	600
Concrete, plain	144	2300	<i>Sand</i>		
Concrete, reinforced	150	2400	Clean and dry	90	1440
Copper	556	9000	River, dry	106	1700
Cork, compressed	14	250	Steel	488	7800
<i>Earth</i>			<i>Stone</i>		
Clay, dry	63	1100	Basalt, granite, gneiss	169	2700
Clay, damp	110	1750	Limestone, marble, quartz	179	2850
Clay and gravel, dry	100	1600	Sandstone	169	2700
Silt, moist, packed	96	1550	Shale	163	2600
Silt, moist, loose	78	1250	<i>Terra cotta</i>		
Sand and gravel, dry, loose	100	1600	Voids filled	120	1950
Sand and gravel, dry, packed	110	1750	Voids unfilled	72	1150
Sand and gravel, wet	120	1900	Tin	459	7360
Glass	160	2600	<i>Water</i>		
Gravel, dry	104	1660	Fresh	62	1000
Gypsum, loose	70	1150	Sea	64	1030
Gypsum, wallboard	50	800	Wood, seasoned	28 to 47	450 to 750
Ice	57	920	Zinc, rolled sheet	449	7200

and controllable maximum heights, $1.4F$ in Eq. (4.2.1a) and $1.2F$ in Eq. (4.2.1b) should be added to the right side.

4.2.7 Other effects—Where structural effects T of differential settlement, shrinkage, or temperature change are significant in design, the design should not be performed using the guide, and **ACI 318**, **ASCE 7**, and the International Building Code (**International Code Council 2015**) should be used.

4.3—Mass and weight

A clear distinction between mass and weight should be made by the licensed design professional in all calculations.

4.4—Weight of materials

In determining dead loads, the actual weight of materials and constructions should be used. In the absence of definite information, the approximated values listed in Table 4.4 may be used.

4.5—Dead loads

4.5.1 Definition of dead loads—Dead loads consist of the weight of construction material incorporated into the building, including, but not limited to, structure, walls, floors, roofs, ceilings, stairways, ramps, finishes, cladding, other incorporated architectural and structural systems, and fixed service equipment. When determining dead loads for design purposes, the actual weights of materials and constructions should be used. For design purposes, the weight of fixed service equipment, such as plumbing stacks and risers, electrical feeders, and heating, ventilating, and air-conditioning systems, should be included with the dead loads.

4.5.2 Self-weight of concrete—The unit weight of reinforced concrete should be taken as 150 lb/ft^3 (2400 kg/m^3).

4.5.3 Dead loads from nonstructural elements—Dead loads from nonstructural elements should be divided into vertical and horizontal nonstructural elements.

**Table 4.5.3.1a—Flat nonstructural elements
minimum dead loads: ceilings**

Component	Load (lb/ft ²) per ft ² of floor area	Load (kN/m ²) per m ² of floor area
<i>Ceilings</i>		
Acoustical fiberboard	1	0.05
Gypsum board	0.55 (per 1/8 in. thickness)	0.0080 (per mm thickness)
Mechanical duct allowance	4	0.20
Sprinkler system allowance	6	0.30
Luminaries and electrical system allowance	1	0.05
Plaster on tile or concrete	5	0.25
Plaster on wood lath	8	0.40
Suspended steel channel system	2	0.10
Suspended metal lath and cement plaster	15	0.70
Suspended metal lath and gypsum plaster	10	0.50
Wood furring suspension system	2.5	0.15

**Table 4.5.3.1b—Flat nonstructural elements
minimum dead loads: floor fill**

Component	Load (lb/ft ²) per ft ² of floor area	Load (kN/m ²) per m ² of floor area
<i>Floor fill</i>	(per in. thickness)	(per mm thickness)
Cinder concrete	9	0.20
Lightweight concrete	8	0.015
Sand	8	0.015
Stone concrete	12	0.025

4.5.3.1 Flat nonstructural elements—Flat nonstructural elements should include construction material with a vertical dimension that is substantially less than the horizontal dimensions and are either applied, supported, attached, fixed, or suspended from the slabs or roof of the building. These elements should include, but are not limited to, permanent slab or joist forms, floor topping, floor fills, floor finishes, roof slope fills, roof coverings, tiles, waterproofing membranes, thermal insulation, skylights, ceilings, furring, and ducts for services.

For purposes of design, the dead loads due to flat nonstructural elements may be considered uniform vertical loads per unit area of horizontal surface or horizontal projection, applied to the corresponding zones or areas where the flat nonstructural elements are located. In determining the dead loads of flat nonstructural elements, the actual weight of materials and constructions and realistic thickness should be used. As a guide, Table 4.4 gives minimum unit weight. (Multiply the values provided by the corresponding thickness in ft [m] to obtain dead loads in lb/ft² [N/m²]). In Tables 4.5.3.1a to 4.5.3.1d, values are given for dead loads of typical flat nonstructural element construction materials. The values provided in Tables 4.5.3.1a to 4.5.3.1d correspond to average

**Table 4.5.3.1c—Flat nonstructural elements
minimum dead loads: floors**

Component	Load (lb/ft ²) per ft ² of floor area	Load (kN/m ²) per m ² of floor area
<i>Floors and floor finishes</i>		
Asphalt block (2 in. [50 mm]), 1/2 in. (12 mm) mortar	30	1.50
Cement finish (1 in. [25 mm]) on stone-concrete fill	32	1.50
Ceramic or quarry tile (3/4 in. [20 mm]) on 1/2 in. (12 mm) mortar bed	16	0.80
Ceramic or quarry tile (3/4 in. [20 mm]) on 1 in. (25 mm) mortar bed	23	1.10
Concrete fill finish	12 (per in. thickness)	0.020 (per mm thickness)
Hardwood flooring, 7/7 in. (25 mm)	4	0.20
Linoleum or asphalt tile, 1/4 in. (6 mm)	1	0.05
Marble and mortar on stone-concrete fill	33	1.60
Slate	15 (per in. thickness)	0.030 (per mm thickness)
Solid flat tile on 1 in. (25 mm) mortar base	23	1.10
Sub flooring, 3/4 in. (20 mm)	3	0.15
Terrazzo 1-1/2 in. (40 mm) directly on slab	19	0.90
Terrazzo 1 in. (25 mm) on stone-concrete fill	32	1.50
Terrazzo 1 in., (25 mm), 2 in. (50 mm) stone concrete	32	1.50
Wood block 3 in. (75 mm) on mastic, no fill	10	0.50
Wood block 3 in. (75 mm) on 1/2 in. (12 mm) mortar base	16	0.80

minimum values. The licensed design professional should consider the possibility of variation of these values from different local materials and local construction practices.

4.5.3.2 Standing nonstructural elements—Standing nonstructural elements should include construction material whose vertical dimension is substantially greater than the least horizontal dimension and are either free-standing, supported by vertical structural members or attached to them, or span vertically from floor-to-floor. These elements include, but are not limited to, façades, nonstructural walls, partitions, wall coverings, veneer, architectural ornaments, windows, doors, and vertical ducts for services. In office buildings or other buildings where partitions can be erected or rearranged, provisions for partition weight should be made, whether or not partitions are shown on the architectural drawings. Dead load should be included as uniform vertical loads per unit area of horizontal surface, in lb/ft² (kN/m²).

Dead loads from standing nonstructural elements may be considered concentrated loads or uniform line loads per unit of length of nonstructural element. As a guide, minimum unit weights are given in Table 4.4. Multiply values by the

Table 4.5.3.1d—Flat nonstructural elements minimum dead loads: roof coverings

Component	Load (lb/ft ²) per ft ² of floor area	Load (kN/m ²) per m ² of floor area
<i>Roof covering</i>		
Asbestos-cement shingles	4	0.20
Asphalt shingles	2	0.10
Cement tile	16	0.80
Clay tile including mortar	25	1.20
Copper or tin	1	0.05
Corrugated asbestos-cement roofing	4	0.20
Deck, metal, 20 gauge (0.9 mm nominal thickness)	2.5	0.12
Deck, metal, 18 gauge (1.2 mm nominal thickness)	3	0.15
Decking, 2 in. (50 mm) wood	5	0.25
Decking, 3 in. (75 mm) wood	8	0.40
Fiberboard, 1/2 in. (12 mm)	0.75	0.05
Gypsum sheathing, 1/2 in. (12 mm)	2	0.10
<i>Insulation, roof boards</i>		
Fibrous or cellular glass	1.0 (per in. thickness)	0.0020 (per mm thickness)
Fiberboard	1.5 (per in. thickness)	0.0030 (per mm thickness)
Perlite	0.8 (per in. thickness)	0.0015 (per mm thickness)
Polystyrene foam	0.2 (per in. thickness)	0.0005 (per mm thickness)
Urethane foam with skin	0.5 (per in. thickness)	0.0010 (per mm thickness)
Plywood (per 1/8 in. [3 mm] thickness)	0.4 (per 1/8 in. thickness)	0.0100 (per mm thickness)
Skylight, metal frame, 3/8 in. (10 mm) wire-glass	8	0.40
<i>Waterproofing membranes</i>		
Bituminous, gravel-covered	5.5	0.25
Bituminous, smooth surface	1.5	0.10
Liquid applied	1	0.05
Single-ply, sheet	0.7	0.03
Wood sheathing	3.0 (per in. thickness)	0.0060 (per mm thickness)
Wood shingles	3	0.15

Table 4.5.3.2a—Standing nonstructural elements minimum dead loads: coverings for walls

Component	Load (lb/ft ²) per ft ² of vertical surface (multiply by the height of element in ft to obtain line loads in lb/ft)	Load (kN/m ²) per m ² of vertical surface (multiply by the height of element in m to obtain line loads in kN/m)
<i>Covering for walls</i>		
Cement tile	16	0.80
Fiberboard, 1/2 in. (12 mm)	0.75	0.05
Gypsum sheathing, 1/2 in. (12 mm)	2	0.10
<i>Insulation, wall boards</i>		
Fibrous or cellular glass	1.0 (per in. thickness)	0.0020 (per mm thickness)
Fiberboard	1.5 (per in. thickness)	0.0030 (per mm thickness)
Perlite	0.8 (per in. thickness)	0.0015 (per mm thickness)
Polystyrene foam	0.2 (per in. thickness)	0.0005 (per mm thickness)
Urethane foam with skin	0.5 (per in. thickness)	0.0010 (per mm thickness)
Plywood (per 1/8 in. [mm] thickness)	0.4 (per 1/8 in. thickness)	0.0100 (per mm thickness)
Wood sheathing	3.0 (per in. thickness)	0.0060 (per mm thickness)

corresponding thickness in ft (m) and by the element height in ft (m) to obtain uniform line dead loads, in lb/ft (N/m). In Tables 4.5.3.2a to 4.5.3.2e, values are given for dead loads of typical standing nonstructural elements construc-

tion materials, in lb (kN) per unit vertical area in ft² (m²). To obtain uniform line dead loads, in lb/ft (kN/m), multiply values suggested in Tables 4.5.3.2a to 4.5.3.2e by the height of the standing nonstructural element in ft (m). The values

Table 4.5.3.2b—Standing nonstructural elements minimum dead loads: light partitions

Component	Load (lb/ft ²) per ft ² of vertical surface (multiply by the height of element in ft to obtain line loads in lb/ft)	Load (kN/m ²) per m ² of vertical surface (multiply by the height of element in m, to obtain line loads in kN/m)
<i>Light partitions</i>		
Movable steel partitions (non-full height)	6	0.30
Movable steel partitions (full height)	4	0.20
Wood or steel studs, 1/2 in. (12 mm) gypsum board each side	8	0.80
Wood studs, 2 x 4 in. (50 x 100 mm), unplastered	4	0.30
Wood studs, 2 x 4 in. (50 x 100 mm), plastered one side	12	0.60
Wood studs, 2 x 4 in. (50 x 100 mm), plastered two sides	20	1.00

Table 4.5.3.2c—Standing nonstructural elements minimum dead loads: veneer

Component	Load (lb/ft ²) per ft ² of vertical surface (multiply by the height of element in ft to obtain line loads in lb/ft)	Load (kN/m ²) per m ² of vertical surface (multiply by the height of element in m, to obtain line loads in kN/m)
<i>Veneer</i>		
Granite	9 (per in. thickness)	0.017 (per mm thickness)
Limestone	8 (per in. thickness)	0.015 (per mm thickness)
Sandstone	7 (per in. thickness)	0.013 (per mm thickness)
Ceramic	8 (per in. thickness)	0.015 (per mm thickness)

Table 4.5.3.2d—Standing nonstructural elements minimum dead loads: walls

Component	Load (lb/ft ²) per ft ² of vertical surface (multiply by the height of element in ft to obtain line loads in lb/ft)	Load (kN/m ²) per m ² of vertical surface (multiply by the height of element in m to obtain line loads in kN/m)
<i>Walls</i>		
<i>Exterior stud walls (steel or wood studs):</i>		
2 x 4 in. (50 x 100 mm) at 16 in. (400 mm), 5/8 in. (15 mm) gypsum, insulated, 3/8 in. (10 mm) siding	11	1.00
2 x 6 in. (50 x 150 mm) at 16 in. (400 mm), 5/8 in. (15 mm) gypsum, insulated, 3/8 in. (10 mm) siding	12	
Exterior stud walls with brick veneer	48	2.30
Clay solid masonry unit walls:	Wall thickness, in. 4 8 12 16	Wall thickness, mm 100 150 200 250 300
Unplastered	39 79 115 155	1.90 2.90 3.80 4.70 5.50
Clay tile masonry unit walls:	Wall thickness, in. 4 6 8 10 12	Wall thickness, mm 100 150 200 250 300
Plaster in both faces	37 52 64 79 91	1.80 2.50 3.10 3.80 4.40
Unplastered	27 41 54 68 81	1.30 2.00 2.60 3.30 3.90
Hollow concrete masonry unit walls:	Wall thickness, in. 4 6 8 10 12	Wall thickness, mm 100 150 200 250 300
No grout	29 30 39 47 54	1.40 1.45 1.90 2.25 2.60
48 in. (1.2 m) on center grout spacing	36 47 57 66	1.70 2.25 2.70 3.15
40 in. (1.0 m) on center grout spacing	37 48 59 69	1.80 2.30 2.80 3.30
32 in. (0.8 m) on center grout spacing	38 50 62 72	1.80 2.40 3.00 3.45
24 in. (0.6 m) on center grout spacing	41 54 67 78	2.00 2.60 3.20 3.75
16 in. (0.4 m) on center grout spacing	46 61 76 90	2.20 2.90 3.60 4.30
Full grout	62 83 105 127	3.00 4.00 5.00 6.10
Solid concrete masonry unit walls:	Wall thickness, in. 4 6 8 10 12	Wall thickness, mm 100 150 200 250 300
Unplastered	41 64 87 110 133	2.00 3.10 4.20 5.30 6.40

Table 4.5.3.2e—Standing nonstructural elements minimum dead loads: windows

Component	Load (lb/ft ²) per ft ² of vertical surface (multiply by the height of element in ft to obtain line loads in lb/ft)	Load (kN/m ²) per m ² of vertical surface (multiply by the height of element in ft to obtain line loads in kN/m)
<i>Windows</i>		
Glass curtain walls, glass, frame and sash	10	0.50
Windows, glass, frame and sash	8	0.40

Table 4.5.3.3—Alternative minimum values for dead load of nonstructural elements when no detailed analysis is performed

Occupancy			Façade and partitions, lb/ft ² (kN/m ²) per ft ² (m ²) of floor area	Floor finish and ceiling, lb/ft ² (kN/m ²) per ft ² (m ²) of floor area
<i>Group A—Assembly</i>	A-2 A-3	Building having an assembly room with capacity of less than 100 persons and not having a stage	20 (1.0)	40 (1.8)
<i>Group B—Business</i>	B	Movable full height partitions	20 (1.0)	40 (1.8)
		Fixed masonry partitions	45 (2.0)	40 (1.8)
<i>Group E—Educational</i>	E	Classrooms	45 (2.0)	35 (1.5)
<i>Group F—Factory</i>	F-1	Light industries	18 (0.8)	35 (1.6)
<i>Group I—Institutional</i>	I-1	Residential board and care facilities	45 (2.0)	35 (1.6)
	I-3	Prisons, jails, reformatories, detention centers	55 (2.5)	40 (1.8)
	I-4	Daycare facilities	45 (2.0)	35 (1.6)
<i>Group M—Mercantile</i>	M	Display and sale of merchandise	35 (1.5)	30 (1.4)
<i>Group R—Residential</i>	R	Masonry façade and partitions	65 (3.0)	35 (1.6)
		Light façade and partitions	45 (2.0)	30 (1.4)
<i>Group S—Storage</i>	S-2	Storage of light materials	35 (1.5)	35 (1.5)
<i>Group U—Utility and Miscellaneous</i>	U	Garages for vehicles with carrying capacity up to 4000 lb (2000 kg)	5 (0.2)	20 (1.0)

provided in Tables 4.4 and 4.5.3.2a to 4.5.3.2e correspond to typical average minimum values; the licensed design professional should consider the variation of these values due to local materials and local construction practices.

Dead load of internal standing nonstructural elements, such as internal walls and partitions, may be considered uniform vertical dead loads per unit area when secondary structural members of the floor system are capable of supporting the resulting concentrated or uniform line loads, without impairment to strength and serviceability of the floor system or the nonstructural element. If standing, nonstructural elements are more than one story tall, their dead loads should be considered concentrated loads or line loads. The dead load of façades and elements of the building enclosure should be considered line loads on the slab edge.

4.5.3.3 Alternative minimum values for dead loads of nonstructural elements—For buildings with story heights not exceeding 10 ft (3 m), the minimum values, in lb/ft² (kN/m²), of horizontal floor area given in Table 4.5.3.3 for the occupancies listed may be used instead of a detailed analysis of dead loads caused by nonstructural elements.

4.5.4 Fixed equipment—Fixed equipment dead load should be provided by the equipment manufacturer.

4.6—Live loads

Live loads are produced by the use and occupancy of the building and do not include construction or environmental loads such as wind, snow, rain, seismic, or dead loads. Live loads should be the maximum loads expected for the intended use or occupancy but should not be less than the minimum uniformly distributed unit loads given in Table 4.6 for the occupancies listed.

4.7—Roof live loads

Roof live loads should not be less than the maximum live load used for the rest of the building, and when of mixed occupancy, the maximum of the different occupancies. While this roof live load is not required by [ACI 318](#), [ASCE 7](#), and the International Building Code ([International Code Council 2015](#)), buildings in some countries commonly add an additional floor level during the life of the structure. This live load value addresses this possibility. When the roof is a nonconcrete roof with a slope greater than 15 degrees, supported on a steel or wood structure, it is permitted to use a roof live load of 20 lb/ft² (0.80 kN/m²).

Table 4.6—Minimum uniformly distributed live loads

Occupancy or use		Uniform load (lb/ft ²) per ft ² of floor area	Uniform load (kN/m ²) per m ² of floor area
Group A—Assembly	Balconies	125	6.0
	Corridors and stairways	125	6.0
	Gymnasiums	125	6.0
	Lobbies	125	6.0
	Movable seats	125	6.0
	Recreational areas	100	5.0
	Platforms	125	6.0
Group B—Business	Corridors and stairways	115	5.5
	Offices	50	2.5
	Restaurants	115	5.5
Group E—Educational	Classrooms	65	3.0
	Corridors and stairways	125	6.0
	Libraries	—	—
	Reading rooms	75	3.5
	Stack rooms	170	8.0
Group F—Factory	Light industries	150	7.0
Group I—Institutional	Operating rooms, laboratories	85	4.0
	Wards and private rooms	50	2.5
	Corridors and stairways	115	5.5
Group M—Mercantile	Retail	115	5.5
	Wholesale	150	7.0
Group R—Residential	Balconies	115	5.5
	Private rooms and corridors serving them	50	2.2
	Public rooms and corridors serving them	115	5.5
	Stairways	125	6.0
Group S—Storage	Light	140	6.5
Group U—Utility and Miscellaneous	Garages for passenger cars only	60	2.8

4.8—Rain load

Each portion of the roof should be designed to sustain the accumulated rainwater if the drainage system for that portion is blocked.

4.9—Snow load

Where snow is expected, the loads caused by its accumulation should be taken into account when designing the roof. The loads should be calculated using the local code provisions. In the absence of a governing local code, the minimum snow load at latitudes greater than 30 degrees in lb/ft² (kN/m²) is given in Fig. 4.9. Where records or experience near the site indicate that the value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval) exceeds the value from Fig. 4.9 or the building is subject to significant ice loads, the design should not be performed using the guide, and ACI 318, ASCE 7, and the International Building Code (International Code Council 2015) should be used.

4.10—Wind loads

4.10.1 General—This section provides a method to calculate wind loads on a building and on individual structural members and cladding. The procedures and values provided by a local building code should be used to obtain wind loads.

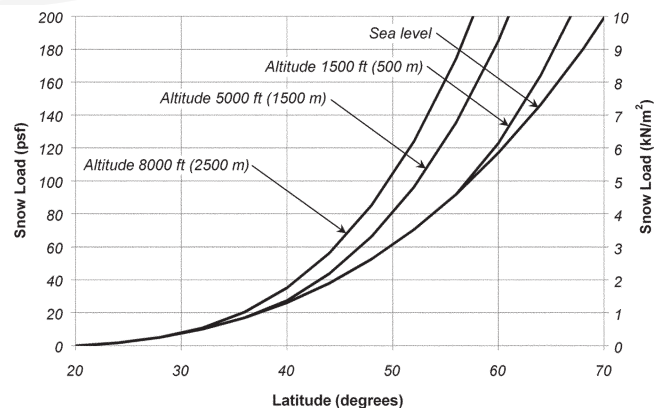


Fig. 4.9—Minimum snow load.

In the absence of a local building code, for buildings with regular geometric shapes, response characteristics, and site locations, the following should apply. Buildings that do not comply with these features should be considered outside the scope of the guide.

4.10.1.1 Procedure—Wind pressures and wind loads at strength level should be determined in accordance with Steps 1 through 6, keeping in mind that building wind loads

are based on the gross exposed areas in the direction of the wind whereas component and cladding wind loads are based on the element area under study.

Step 1: Velocity pressure q_h should be determined in accordance with 4.10.2.1 and 4.10.2.2.

Step 2: Appropriate C_{su} pressure coefficients should be selected from 4.10.3, depending on exposed building surfaces, assuming that the wind acts in the positive and negative sense in both principal directions in plan of the building.

Step 3: Velocity pressure q_h affected by the force coefficients and multiplied by the exposed area of the surface A_{su} should be used to determine the equivalent static wind design force F_{su} that acts in a direction normal to the exposed building surfaces.

Step 4: Components in the direction of the wind of the static wind loads that act on the surfaces, F_{su} , should be algebraically added to obtain the wind base shear V_w for the building in each principal direction in plan, in both senses.

Step 5: Static wind loads that act on the structural members and cladding should be used to design the individual elements and their attachments to the structure.

Step 6: Static wind loads that act on the building should be used to design the members of the lateral-load-resisting system and the overall stability of the structure.

4.10.1.2 Application of pressures and loads—Static wind design loads F_{su} should be assumed to act in a direction normal to the surface considered at the centroid of its area. F_{su} used in calculating building loads should be expressed as the resultant along the principal axis of the building (Eq. (4.10.2.3b)).

4.10.2 Calculation of the wind loads

4.10.2.1 Velocity pressure calculation—Velocity pressure q_h at the mean roof height h_r over the terrain should be calculated from Eq. (4.10.2.1a) and Eq. (4.10.2.1b)

$$q_h = \frac{1}{400} V^2 \left(\frac{h_r}{33} \right)^{\frac{2}{7}} \quad (q_h \text{ in lb/ft}^2, h_r \text{ in m, and } V \text{ in mph}) \quad (4.10.2.1a)$$

$$\left[q_h = \frac{1}{1630} V^2 \left(\frac{h_r}{10} \right)^{\frac{2}{7}} \quad (\text{SI}) \right] \quad (q_h \text{ in kN/m}^2, h_r \text{ in ft, and } V \text{ in m/s}) \quad (4.10.2.1b)$$

4.10.2.2 Selection of basic wind speed—Basic wind speed at strength level, V , corresponds to a 3-second gust speed at 33 ft (10 m) above ground measured at open terrain with scattered obstructions having heights less than 33 ft (10 m), such as open country, with a 300-year mean return period. The 15 percent exceedance probability in a 50-year period or the annual probability that the basic wind speed will be exceeded is 0.00333.

Table 4.10.2.2 gives typical values of basic wind speed at strength level given as a guide for regions outside of the United States and countries where wind speed velocity information is not available. The user should refer to histor-

Table 4.10.2.2—Typical values of strength level basic wind speed, V

Region	V , mph	V , m/s
Islands and coastal regions exposed to tropical storms	150 to 200	67 to 89
Islands and coastal regions exposed to subtropical storms	110 to 150	49 to 67
Islands and coastal regions not exposed to tropical or subtropical storms	90 to 110	40 to 49
Tornado-prone regions	90 to 160	40 to 70
Continental regions, more than 100 miles (160 km) from the coast	70 to 110	30 to 50

ical records or local regulations for actual design speed. A distinction should be made between the type of basic wind speed (3-second gust or fastest mile), the mean return period, and the type of exposure where the wind velocity is defined. Given the lateral-load building stiffness obtained from the guide, a gust effect factor is not included in the procedure.

4.10.2.3 Wind loads on a building—The equivalent static strength level wind force that acts in the direction normal to the wind-exposed surface should be calculated using Eq. (4.10.2.3a). Regardless of the openness of the exposed surface, the gross area should be used.

$$F_{su} = q_h A_{su} C_{su} \quad (4.10.2.3a)$$

where A_{su} is the exposed surface area, and C_{su} is the value given in 4.10.3.

A positive value of F_{su} indicates that the force is acting toward the surface, and a negative value indicates that the force is acting away from the surface (suction). The wind design force at floor level x , F_x , due to wind in a principal direction in plan should be calculated as the sum of all wind-exposed surfaces of the story using Eq. (4.10.2.3b).

$$F_x = \sum_{w=1}^n \left(F_{su}^{(w)} \cos \alpha_w \cos \beta_w \right) \quad (4.10.2.3b)$$

where α_w is the horizontal angle between the normal to the surface and the principal direction in plan under consideration, and β_w is the vertical angle between the normal to the surface and a horizontal line.

In Eq. (4.10.2.3b), care should be used in defining the sign to be used for F_{su} , with respect to the direction in which the force is acting.

The wind base shear V_w in a principal direction in plan corresponds to the sum of all story wind loads F_x in the principal direction in plan under study.

4.10.2.4 Wind loads on components and cladding—Calculation of the strength level wind force on components and cladding should be performed using Eq. (4.10.2.4).

$$F_{pw} = q_h A_p C_p \quad (4.10.2.4)$$

where A_p is the component exposed surface area, and C_p is the component surface pressure coefficient.

Table 4.10.3.1—Pressure coefficients for the building as a whole

Type of surface	C_{su} Total building	C_{su} Windward	C_{su} Leeward
<i>Vertical surfaces</i>			
Prismatic rectangular buildings	1.30	0.80	−0.50
<i>Partially open surfaces</i>			
<i>(C_{su} applies over gross area)</i>			
10 percent to 80 percent open	1.30	0.80	−0.50
>80 percent open	0.35	0.25	−0.10
<i>Horizontal roof surfaces</i>			
Enclosed buildings		−1.30	−0.70
Buildings with one or two open sides		−1.50	−1.25
<i>(C_{su} applies over 1/3 of the roof area for windward)</i>			
<i>(C_{su} applies over 2/3 of the roof area for leeward)</i>			
<i>Sloping roof surfaces</i>			
<i>Roof angle measured with respect to a horizontal line</i>			
10		−1.30	−0.70
15		−1.00	−0.60
20		−0.70	−0.60
25		−0.50	−0.60
30		−0.30	−0.60
35		−0.20	−0.60
45		0.30	−0.60
$Q > 60$		$0.01Q$	−0.60

Table 4.10.3.2—Pressure coefficients for components and cladding

Type of surface	C_p
<i>Vertical surfaces</i>	
Exterior walls of enclosed buildings	1.80
Doors and windows	1.80
Exterior walls of buildings with one or more sides open	1.80
<i>Horizontal surfaces</i>	
Away from edges and corners	1.40
Edges of building and away from corners	2.30
Corners of building	3.20
Edge distance is 10 percent of length of surface in consideration.	

4.10.3 Pressure coefficients

4.10.3.1 Pressure coefficients for a building—Pressure coefficients in Table 4.10.3.1 should be used for the calculation of the wind loads on a building. Note that coefficients reflect the direction of the force relative to the direction of the members. In terms of the building total load effect for vertical surfaces, the absolute values should be added.

4.10.3.2 Pressure coefficients for components and cladding—Pressure coefficients in Table 4.10.3.2 should be used for the

calculation of the strength level wind loads on the components and cladding, positive for pressure and negative for suction.

4.11—Seismic loads

4.11.1 General—Resistance to seismic loads should be provided by using a sufficient number of reinforced concrete walls continuous from the roof to the foundation in both principal directions in plan. Reinforced concrete walls produce stiff structures with a short fundamental period of vibration and the seismic loads given in this guide reflect this type of structure. The calculation of seismic loads for more flexible structures is beyond the scope of this guide. Additional seismic information is presented in [Chapter 11](#).

4.11.2 Seismic ground motions

4.11.2.1 Seismic loads and seismic design—It is intended that where the governing authority has adopted, by local ordinance, the model code (International Building Code [[International Code Council 2015](#)]) and reference standard ([ASCE 7](#)), seismic design loads and seismic design prescribed under the model code (International Building Code [[International Code Council 2015](#)]) and reference standard ([ASCE 7](#)) should be used meeting 4.11.2.2 and 4.11.2.3. Where these standards have not been adopted by the governing authority, 4.11.2.4 through 4.11.2.6 should be used.

4.11.2.2 Design spectral acceleration parameters—The design earthquake spectral acceleration parameter at short period, S_{DS} , should be determined from the procedure established in [ASCE 7](#). The design spectral ordinates for determining seismic loads, S_a , of the guide are equal to S_{DS} .

4.11.2.3 Seismic design categories—The following distinction of seismic design categories as associated with seismic risk should be used in those cases where [ASCE 7](#) has not been adopted by local authorities:

a) Low or no seismic risk zones are where the seismic design category assigned to the structure in the procedure in [ACI 318](#), [ASCE 7](#), and the International Building Code (International Code Council 2015) for short-period response acceleration is either A or B.

b) Moderate or high seismic risk zones are where the seismic design category assigned to the structure in the procedure in [ACI 318](#), [ASCE 7](#), and the International Building Code (International Code Council 2015) for short-period response acceleration is C, D, E, or F.

4.11.2.4 Seismic loads and seismic design when [ACI 318](#), [ASCE 7](#), and the International Building Code (International Code Council 2015) have not been adopted—Where the governing authority has not adopted the model code (International Building Code [[International Code Council 2015](#)]) and reference standard ([ASCE 7](#)), the ground motion should be described through an effective peak ground horizontal acceleration in rock for short periods of vibration, A_a , expressed as a fraction of gravity g . The values given in the general building code having jurisdiction shall be used for obtaining the corresponding values.

Where the building code having jurisdiction defines a maximum considered seismic ground motion based on spectral response accelerations at 5 percent of critical damping, the short period (0.2 second) values should be employed.

and the value of A_a may be obtained as $A_a = S_s/3.75$. Where the building code having jurisdiction defines a seismic zone factor Z , the value of A_a should be taken equal to Z .

4.11.2.4.1 Soil profile types—Based on the soil type present at the building site, the soil profile should be classified as one of the following:

a) Soil profile S_A : hard rock with a measured shear wave velocity $\bar{v}_s > 5000$ ft/s (1500 m/s)

b) Soil profile S_B : rock with moderate fracturing and weathering with a measured shear wave velocity in the range of 5000 ft/s $\geq \bar{v}_s > 2500$ ft/s (1500 m/s $\geq \bar{v}_s > 760$ m/s)

c) Soil profile S_C : soft weathered or fractured rock, or dense or stiff soil, where the measured shear wave velocity is in the range of 2500 ft/s $\geq \bar{v}_s > 1200$ ft/s (760 m/s $\geq \bar{v}_s > 370$ m/s) or, in the upper 100 ft (30 m), the standard penetration test (SPT) resistance has an average value of $\bar{N} > 50$ or a shear strength for clays $s_u \geq 2000$ lb/ft² ($s_u \geq 100$ kPa)

d) Soil profile S_D : predominately medium-dense to dense, or medium-stiff to stiff soil, where the measured shear wave velocity is in the range of 1200 ft/s $\geq \bar{v}_s > 600$ ft/s (370 m/s $\geq \bar{v}_s > 180$ m/s) or where, in the upper 100 ft (30 m), the SPT resistance has an average value in the range of $15 < \bar{N} \leq 50$, or a shear strength for clays in the range of 1000 lb/ft² $\leq s_u < 2000$ lb/ft² (50 kPa $\leq s_u < 100$ kPa);

e) Soil profile S_E : a soil profile where the measured shear wave velocity $\bar{v}_s \leq 600$ ft/s (180 m/s), or the SPT resistance has an average value $\bar{N} \leq 15$ in the upper 100 ft (30 m), or has more than 10 ft (3 m) of plastic (soil plasticity index [PI] > 20), high moisture content ($w > 40$ percent), and low shear strength ($s_u < 500$ lb/ft² [$s_u < 25$ kPa]) clays

f) Seismically vulnerable soils: sites where the soil profile contains soil having one or more of the following characteristics are beyond the scope of the guide:

- i. Soils vulnerable to potential failure or collapse under seismic motions, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soil
- ii. Peats, highly organic clays, or both, with more than 10 ft (3 m) of thickness
- iii. Very-high-plasticity clays (PI > 75) with more than 25 ft (8 m) of thickness
- iv. Soft to medium-stiff clays with more than 120 ft (40 m) of thickness.

When the soil profile and properties are not known well enough in detail to determine the soil profile type, a soil profile S_D should be used.

4.11.2.4.2 Site effects—Site effects should be described through the site coefficient for short periods of vibration, F_a . Values of the site coefficient for short periods of vibration, F_a , should be determined from Table 4.11.2.4.2 as a function of the effective peak ground horizontal acceleration in rock for short periods of vibration, A_a , and the soil profile type from 4.11.2.4. Linear interpolation can be used between values of A_a in Table 4.11.2.4.2.

Site effect of seismically vulnerable soils, as described in 4.11.2.4, is beyond the scope of this guide, and designs should not be made under the guide for buildings located on such soil.

Table 4.11.2.4.2—Values of the site coefficient F_a

Soil profile	Site coefficient F_a for short periods of vibration				
	$A_a \leq 0.1$	$A_a = 0.2$	$A_a = 0.3$	$A_a = 0.4$	$A_a \geq 0.5$
S_A	0.8	0.8	0.8	0.8	0.8
S_B	1.0	1.0	1.0	1.0	1.0
S_C	1.2	1.2	1.1	1.0	1.0
S_D	1.6	1.4	1.2	1.1	1.0
S_E	2.5	1.7	1.2	0.9	0.9

4.11.2.5 Seismic risk zones when ACI 318, ASCE 7, and the International Building Code (International Code Council 2015) have not been adopted—The following distinction of seismic risk should be used:

a) Low and no seismic risk zones: zones where A_a is less than or equal to 0.10

b) Moderate or high seismic risk zones: zones where A_a exceeds 0.10

4.11.2.6 Design response spectral ordinates when ACI 318, ASCE 7, and the International Building Code (International Code Council 2015) have not been adopted—Ordinates of the elastic design response spectrum S_a for a damping ratio of 5 percent of critical, expressed as a fraction of the acceleration of gravity, should be calculated as

$$S_a = 2.5A_aF_a \quad (4.11.2.6)$$

4.11.3 Seismic design base shear

4.11.3.1 Seismic-resistant structural system—The seismic-resistant structural system used in this guide is classified as a dual building frame system, where an essentially complete moment-resistant space frame supports gravity loads, and resistance to lateral loads is provided by reinforced concrete walls, with the moment-resisting space frame providing a minimum collateral lateral-load resistance. Reinforced concrete walls resisting lateral loads are not permitted to carry vertical axial loads greater than the balanced point axial strength (Eq. (5.12.4.1a)).

4.11.3.2 Energy-dissipation capacity of the seismic-resistant structural system—The energy-dissipation capacity in the inelastic range of the seismic-resistant structural system, described by the response modification factor, should have a value of $R_s = 5.0$.

4.11.3.3 Calculation of seismic design base shear—The seismic design base shear V_{bs} , equivalent to the total horizontal inertial effects caused by seismic ground motions, should be determined using Eq. (4.11.3.3).

$$V_{bs} = \frac{S_a W_s}{R_s} \quad (4.11.3.3)$$

where S_a should be determined from Eq. (4.11.2.6); R_s is the response modification factor determined from 4.11.3.2; and W_s corresponds to the total weight of the building. W_s should include the total weight of the structure plus the weight of all nonstructural elements, such as walls and partitions, permanent equipment, tanks, and the contained liquid. In storage occupancies, W_s should also include 25 percent of the live load and the snow load when the snow load exceeds 30 lb/ft² (1.4 kPa).

4.11.4 Vertical distribution of design seismic loads—Total seismic design base shear should be distributed over the building height using Eq. (4.11.4a) and Eq. (4.11.4b). At each floor level designated as x , F_x should be applied over the building area in accordance with the mass distributions at each level.

$$F_x = C_{vx} V_{bs} \quad (4.11.4a)$$

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^n (w_i h_i)} \quad (4.11.4b)$$

4.12—Soil weight and lateral pressure

The design of basement walls and similar vertical elements below grade should include the lateral pressure of adjacent soil following 4.13.2.3.

4.13—Lateral loads

4.13.1 General—All applicable lateral loads as described in Chapter 4 should be used in the design. The simultaneous occurrence of lateral loads with other loads should be evaluated using the load combination of 4.2. A continuous load path from the lateral force application point to the lateral-force-resisting structural elements should be identified, and all elements along the load path should have adequate strength.

4.13.2 Application of lateral loads

4.13.2.1 Wind loads—Wind loads should be determined by 4.10. Horizontal strength level wind loads, F_x , at each level x and in each main direction in plan should be the sum of the windward and leeward wind components as determined by 4.10.2.3. A wind base shear V_o and its distribution over the building height should be determined for both main directions in plan, as determined by 4.13.3. The overturning moment M_{ot} , caused by the wind loads in both main directions in plan, should be determined as determined by 4.13.4. The application of the resultant wind loads on each floor and principal direction should be calculated, and the torsional moments on the structure, T_i , about a vertical axis should be evaluated as determined by 4.13.5 for each story.

4.13.2.2 Seismic loads—Seismic loads should be determined by 4.11. Horizontal seismic loads F_x at each level x and in each main direction in plan should be determined by 4.11.4. A lateral base shear V_o and its distribution over the building height should be determined by 4.11.3.3 and 4.13.3 for both main directions in plan. The overturning moment M_{ot} due to the seismic loads in both main directions in plan should be determined by 4.13.4. Application point of the seismic force at each floor should be the center of mass of the floor diaphragm. The effect of an eccentricity between the center of mass of the floor diaphragm and the center of stiffness corresponding to the lateral-force-resisting structural members should be calculated, and the torsional moments on the structure, T_i , about a vertical axis should be evaluated by 4.13.5 for each story.

4.13.2.3 Soil lateral loads—Lateral loads due to soil pressure should be determined by Chapter 14. Due to the restric-

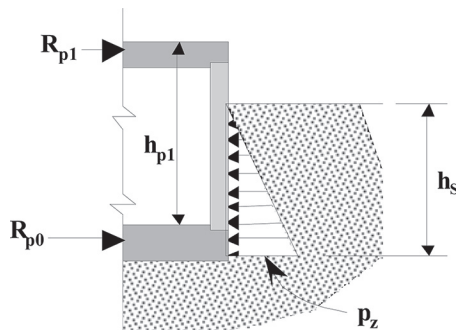


Fig. 4.13.2.3—Lateral reaction due to soil pressure.

tions of 1.3.2 and 1.3.10, retaining walls laterally supported by the building should be only one story in height. The reaction from the lateral soil pressure on the first story per unit horizontal length of retaining wall should be computed by Eq. (4.13.2.3).

$$R_{p1} = \frac{p_z h_s^2}{6h_{p1}} \quad (4.13.2.3)$$

where p_z is the lateral pressure at depth of soil, h_s (14.11.2); and h_{p1} is the first-story height (Fig. 4.13.2.3).

Two different cases should be considered when designing the structure as a whole for the effects of lateral soil pressure:

1. **Case 1**—A building located in flat or nearly flat terrain does not need a lateral analysis of the overall structure due to soil pressure, because lateral reactions due to soil pressure on basement walls in opposite sides are approximately equal, compensate each other, and introduce a nearly uniform in-plane compression on the first-floor slab. In this case, lateral soil pressure should be considered only when designing the soil-retaining foundation members, as noted in Chapter 14, and the members framing into it.

2. **Case 2**—A building located on sloping terrain supports uncompensated lateral loads, where lateral reaction due to soil pressures on basement walls in opposite sides are different. In this case, a lateral base shear V_o should be determined by 4.13.3 for both main directions in plan from the first-floor reaction. The overturning moment M_{ot} due to soil lateral pressure in both main directions in plan should be determined by 4.13.4. The location in plan of the resultant lateral soil loads on the first floor should be calculated and the torsional moments on the structure, T_i , about a vertical axis should be evaluated by 4.13.5. The occurrence of wind or seismic loads simultaneously with this soil pressure permanent lateral force should be evaluated using load combinations prescribed for this case in 4.2.5.

4.13.2.4 Fluid pressure—Auxiliary structures subjected to fluid pressure, such as water tanks, should be self-contained and fluid pressure should be compensated within the auxiliary structure. The main building structure should not be used to resist lateral loads derived from contained liquids. Weight from auxiliary tank and liquid content should be considered in the calculation of weight on the roof level.

4.13.3 Factored story shear and base shear—Base shear due to wind, seismic force, and lateral soil pressure should be determined independently for the two main directions in plan. The story shear at level x for each lateral force (wind, seismic, and soil) corresponds to the sum of the lateral loads applied to the structure in the main direction under consideration, from story x to the roof, as determined from Eq. (4.13.3) (Fig. 4.13.3).

$$V_x = \sum_{i=x}^n F_i \quad (4.13.3)$$

The base shear V_o for each lateral force (wind, seismic, and soil) corresponds to the sum of the lateral loads applied to the structure in the main direction under study (Fig. 4.13.3).

Factored story shear V_{xu} and factored base shear V_{ou} should be determined by multiplying V_x and V_o by the appropriate load factor from 4.2.

4.13.4 Factored overturning moment—The overturning moment due to wind, seismic force, and lateral soil pressure should be determined independently for the two main directions in plan. The story overturning moment at level x for each lateral force (wind, seismic, and soil) corresponds to the sum of the moments due to lateral loads at that story from level x to the roof, as determined from Eq. (4.13.4a) (Fig. 4.13.4).

$$M_x = \sum_{i=x}^n [F_i (h_i - h_x)] \quad (4.13.4a)$$

Base overturning moment M_{ot} for each lateral force (wind, seismic, and soil) corresponds to the sum of the product of the floor level lateral loads by the height to floor level

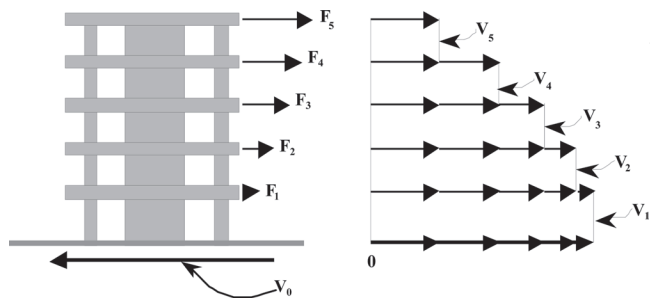


Fig. 4.13.3—Story shear and base shear calculation.

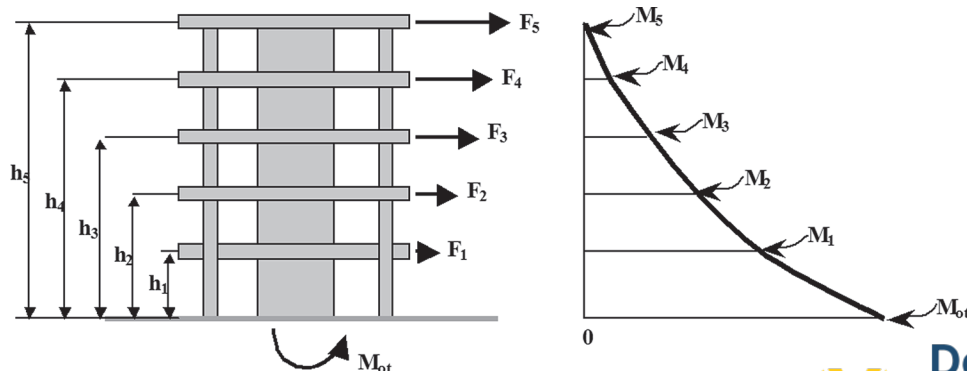


Fig. 4.13.4—Overturning moment calculation.

measured from the base of the structure (Fig. 4.13.4), as defined by Eq. (4.13.4b).

$$M_{ot} = \sum_{i=1}^n [F_i h_i] \quad (4.13.4b)$$

Factored story overturning moment M_{xu} and factored base overturning moment M_{ou} should be determined by multiplying M_x and M_{ot} by the appropriate load factor from 4.2.

4.13.5 Torsional moments—For each story and in each principal direction, the unfactored torsion T_i should be computed as the product of the story lateral force F_i times an eccentricity equal to the transverse distance between the line of action of the lateral force and the story lateral stiffness center (Fig. 4.13.5). The story stiffness center should be determined by 4.14.5. For wind and seismic loads, T_i should be taken as the greater value determined for both principal directions. For lateral soil pressure, T_i should be taken as the algebraic sum of the values determined for both principal directions.

The story total unfactored torsion at level x for each lateral force (wind, seismic, and soil) corresponds to sum of the story unfactored torsion applied to the structure from story x to the roof, as determined by Eq. (4.13.5).

$$T_x = \sum_{i=x}^n T_i \quad (4.13.5)$$

Base total unfactored torsion T_o for each lateral force (wind, seismic, and soil) corresponds to the sum of the story unfactored torsion applied to the structure.

Factored story torsional moment T_{iu} and factored base torsional moment T_{ou} should be computed by multiplying T_i and T_o by the appropriate load factor from 4.2.

4.14—Lateral-force-resisting system

4.14.1 General—The lateral-force-resisting system is composed of structural members acting jointly that resist and transmit lateral loads resulting from seismic motions, wind, and lateral earth pressure to the ground (Fig. 4.14.1).

The floor system should act as a diaphragm that carries in its plane the lateral force from the application point to the vertical members of the lateral-force-resisting system. Vertical members of the lateral-force-resisting system, in turn, collect loads arising from all floors affected and

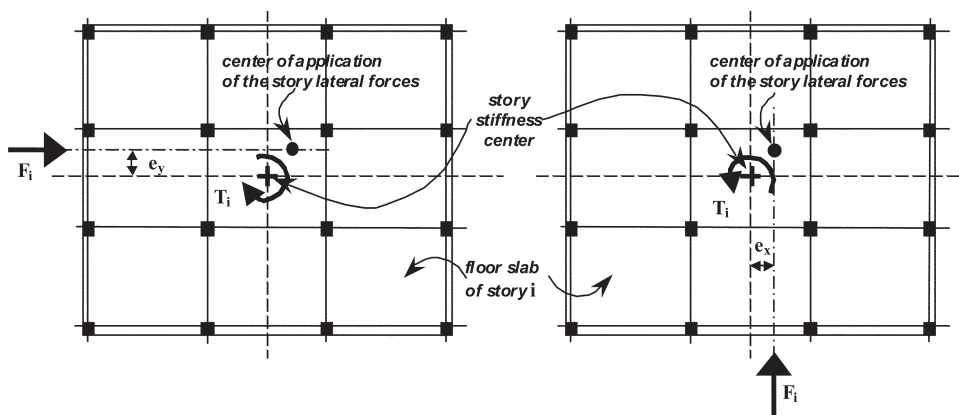


Fig. 4.13.5—Story torsion.

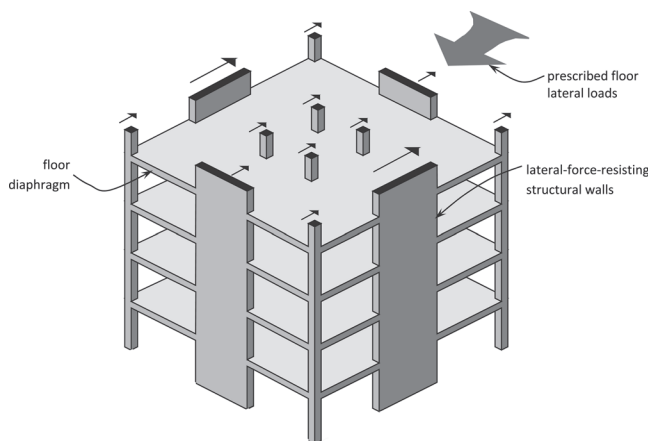


Fig. 4.14.1—Lateral-force-resisting structural system.

transmit them down to the foundation and to the underlying soil. The main vertical members of the lateral-force-resisting system should be a number of reinforced concrete walls in both principal directions in plan. These reinforced concrete walls should have no openings for windows or doors.

4.14.2 Restrictions on structural system for seismic resistance

4.14.2.1 General—The following restrictions apply to structural systems used for the seismic risk zones as defined in 4.11.2.3.

4.14.2.2 Zones of low and no seismic risk—There are no restrictions on which structural systems can be used in zones of low or no seismic risk, as defined in 4.11.2.3. In these zones, structural walls for resistance to earthquake seismic loads may not be needed, but their need to resist wind and soil pressure lateral loads should be verified.

4.14.2.3 Moderate or high seismic risk zone—Lateral-force resistance should be provided by reinforced concrete walls, continuous from the foundation to the roof, in both principal directions in plan. The special reinforcement details of Chapter 11 should be used.

4.14.3 Diaphragm effect

4.14.3.1 General—To obtain proper lateral-force resistance, the building structure should act as a single unit through the diaphragm effect of the floor slabs. The lateral loads should be transferred from the application point to the lateral-force-

resisting structural members (structural walls and moment-resisting frames) as in-plane loads in the diaphragm.

4.14.3.2 Diaphragm—The diaphragm should comply with (a) through (e):

a) The shape in plan of the floor slab diaphragm should be as regular and symmetrical as practicable. Preferred forms should be square or rectangular. Rectangular diaphragms should have a maximum ratio of shorter to longer side of 4.

b) Floor slab structural members should comply with the minimum thickness of Chapter 6.

c) No opening in the slab should be greater than 25 percent of the floor area, and the sum of all openings in the slab should not exceed 50 percent of the floor area.

d) Diaphragm collector elements should be provided, as indicated in 4.15.5, and be capable of transmitting lateral loads from the floor slab system to the lateral-force-resisting structural walls.

e) Diaphragm members can be at a slope but should comply with 1.3.9.

4.14.4 Floor mass centroid—The floor mass centroid should correspond to the centroid of the area of the floor slab. The area for computing each floor mass centroid should be bounded by slab borders, minus the area of all openings having a side greater than 6 ft (1.8 m). The lateral seismic loads should be applied at the floor mass centroid for each story.

4.14.5 Story lateral stiffness center—Story lateral stiffness center corresponds to the point where the floor diaphragm will tend to rotate about a vertical axis when subjected to a story torsional moment. The location of the story lateral stiffness center in plan should be calculated using the lateral stiffness of the walls only, disregarding the stiffness contribution of the columns of the structure. If the location of the structural walls is symmetric with respect to the center of mass of every story in the structure, it should be assumed that the lateral stiffness center for all stories is located at the center of mass. Where the walls are not symmetric, procedures (a) through (d) should be used to calculate the location of the story lateral stiffness center (Fig. 4.14.5).

(a) An arbitrary origin location is assumed at point 0.

(b) Lateral stiffnesses, k_x and k_y , of structural walls should be computed using Eq. (4.14.5a(a)) and Eq. (4.14.5a(b)) for both principal directions x and y .

For structural walls with lengths parallel to the x axis,

$$k_x = \frac{\ell_w^3 b_w}{h_{pi}^3} \text{ and } k_y = \frac{\ell_w b_w^3}{h_{pi}^3} \quad (4.14.5a(a))$$

or, for structural walls with lengths parallel to the y axis

$$k_x = \frac{\ell_w b_w^3}{h_{pi}^3} \text{ and } k_y = \frac{\ell_w^3 b_w}{h_{pi}^3} \quad (4.14.5a(b))$$

where b_w is the web width of section, or wall width; ℓ_w is the horizontal length of wall; and h_{pi} is the story height;

(c) The story lateral stiffness center coordinates, $\bar{x}$ and $\bar{y}$, with respect to origin 0, should be

$$\bar{x} = \frac{\sum(k_y x_i)}{\sum k_y} \text{ and } \bar{y} = \frac{\sum(k_x y_i)}{\sum k_x} \quad (4.14.5b)$$

where x_i is the distance from origin 0 to the wall cross-sectional centroid measured in a direction parallel to x , and y_i is the distance from origin 0 to the wall cross-sectional centroid measured in a direction parallel to y .

(d) Total story rotational stiffness about the story lateral stiffness center should be computed using Eq. (4.14.5c).

$$k_r = \left[\sum(k_y x_i^2 + k_x y_i^2) \right] - \left[\bar{x}^2 \sum k_y \right] - \left[\bar{y}^2 \sum k_x \right] \quad (4.14.5c)$$

4.15—Minimum amount of reinforced concrete structural walls

4.15.1 General—A minimum amount of reinforced concrete structural walls should be provided for factored lateral-force resistance. These structural walls should comply with (a) through (g):

(a) Structural walls should have rectangular cross sections. Other cross sections are beyond the scope of this guide, except core walls as prescribed in 12.8.

(b) Structural walls should be vertically continuous from foundation to roof, except where walls are needed only for resistance for unbalanced lateral soil pressure in a basement. In this case, the wall may be stopped at the first story level.

(c) Structural walls should be aligned vertically. Where section reductions occur, the wall cross section underneath should be greater than the one above and the centroid of the wall cross section above should be in the middle third, in both directions, of the one below.

(d) Structural walls should not have openings for windows or doors.

(e) In both principal directions in plan, there should be at least two parallel walls in different planes, and the planes should be as far apart as practicable. The walls should be placed as close to the periphery of the building as practicable.

(f) Walls should be located as symmetrically as possible with respect to the centers of mass and stiffness of each floor.

(g) Structural wall dimensions should comply with 4.15.2 and 4.15.3.

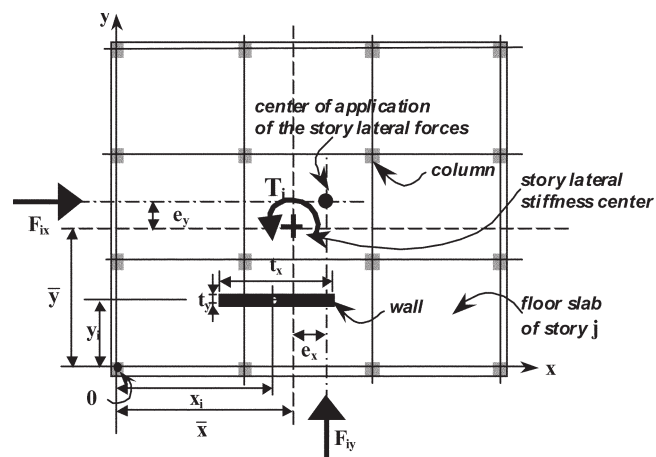


Fig. 4.14.5—Calculation of the story lateral stiffness center.

4.15.2 Minimum wall area for shear strength—At any floor i , for the two principal directions, x and y , the minimum cross-sectional area ($A_g = \ell_w b_w$) for all reinforced concrete walls acting in the principal direction under consideration should be determined from Eq. (4.15.2).

$$\sum(\ell_w b_w) \geq \frac{V_{iu}}{2\sqrt{f'_c}} \quad (4.15.2)$$

$$\left[\sum(\ell_w b_w) \geq \frac{6V_{iu}}{\sqrt{f'_c}} \text{ (SI)} \right]$$

In Eq. (4.15.2), only walls where the horizontal length of wall, ℓ_w , is parallel to the direction under consideration should be included; b_w corresponds to the wall thickness, and V_{iu} should be determined from 4.13.3. As noted by 4.15.1(e), each principal direction should have at least two walls.

4.15.3 Minimum wall dimensions for lateral stiffness—Slenderness ratio h_w/ℓ_w for any individual wall should comply with Eq. (4.15.3), and the wall thickness b_w should comply with Chapter 12.

$$\left(\frac{h_w}{\ell_w} \right) \leq \frac{3 + n_s}{2} \quad (4.15.3)$$

where h_w is the wall height from the foundation to the roof, and ℓ_w is the horizontal length of wall.

In Eq. (4.15.3) n_s corresponds to the total number of stories of the building above the base.

4.15.4 Lateral strength in vertical structural members

4.15.4.1 Structural walls—At any story i , the factored lateral shear V_u that a wall should sustain is determined from Eq. (4.15.4.1a). The summation in Eq. (4.15.4.1a) should be performed for all walls with ℓ_w parallel to V_{iu} .

$$V_u = V_{iu} \frac{b_w \ell_w^3}{\sum(b_w \ell_w^3)} + \Delta V_{iu} \quad (4.15.4.1a)$$

where V_{iu} is the factored story shear (4.13.3) in the direction parallel to ℓ_w ; b_w and ℓ_w are the wall cross-sectional

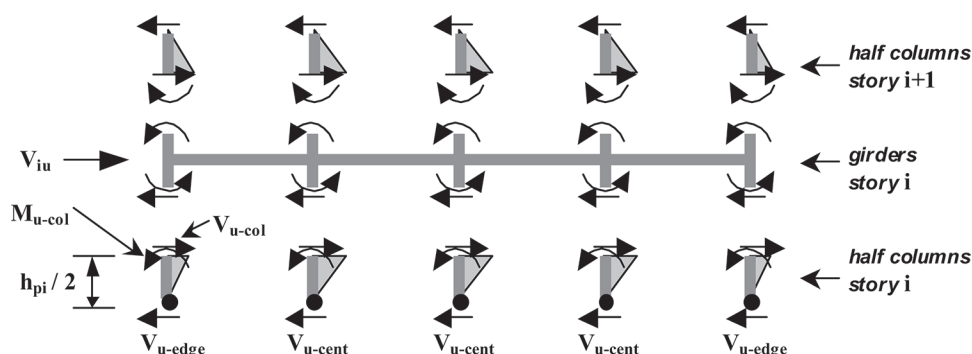


Fig. 4.15.4.2a—Column lateral shears and moments in frames.



Fig. 4.15.4.2b—Girder lateral force moments in frames.

dimensions; and ΔV_{ut} is the increase in shear due to torsion, determined from Eq. (4.15.4.1b). The value of ΔV_{ut} from Eq. (4.15.4.1b), parallel to ℓ_w , should be used in Eq. (4.15.4.1a).

$$\Delta V_{ux} = T_{iu} \frac{\bar{y}k_x}{k_r} \text{ and } \Delta V_{uy} = T_{iu} \frac{\bar{y}k_y}{k_r} \quad (4.15.4.1b)$$

where T_{iu} is determined from 4.13.5; $\bar{x}$ and $\bar{y}$ are determined from Eq. (4.14.5b); k_x and k_y are determined from Eq. (4.14.5a); and k_y is determined from Eq. (4.14.5c).

4.15.4.2 Frames—The following procedure should be used to assign factored story shears to the structure frame through the columns.

(a) In zones of low and no seismic risk, where reinforced concrete walls are not mandatory (4.14.2), total lateral story shear at any story should be distributed to the frames through the columns.

(b) In moderate or high seismic risk zones, 100 percent of factored lateral loads should be resisted by reinforced concrete walls. Frames should be proportioned to resist a minimum lateral force equal to 25 percent of the factored lateral force in each direction in plan to account for effects such as base rotation of the walls or a decrease in stiffness and strength due to inelastic response.

(c) The fraction of the story shear, V_{iu} , as described by (a) or (b), carried by an individual column in one plan direction for any story i , should be determined from Eq. (4.15.4.2a) for interior columns, and by Eq. (4.15.4.2b) for edge columns. An interior column is a column whose ends frame into two girders parallel to the direction under study, whereas edge columns frame into one girder on one side only.

$$V_u = \frac{2V_{iu}}{n_e + 2n_c} \quad (4.15.4.2a)$$

$$V_u = \frac{V_{iu}}{n_e + 2n_c} \quad (4.15.4.2b)$$

In Eq. (4.15.4.2a) and Eq. (4.15.4.2b), n_e and n_c are the total number of edge and central columns, respectively, in story i for the direction under consideration.

(d) The factored column moment due to lateral loads (Fig. 4.15.4.2a) should be determined from Eq. (4.15.4.2c).

$$M_u = V_u \frac{h_{pi}}{2} \quad (4.15.4.2c)$$

where h_{pi} corresponds to the story height of floor i , and V_u is the corresponding value determined in (c).

(e) For column-slab systems, the girder or slab moment due to lateral loads should be taken as equal to the factored column moment for interior column joints and twice the factored column moment for edge column joints. This results in the moments at all girders at the face of the column-girder joints. This moment acts in both the positive and negative sense in the direction under study, and it should be taken as an increase or decrease of the gravity load factored moments (Fig. 4.15.4.2b).

4.15.5 Collector elements—Collector elements within the diaphragm, usually consisting of beams or girders, transmit lateral loads in the story from application point to the lateral-force-resisting structural members. Collector element longitudinal reinforcement should be confined by closed stirrups that comply with the limits for tied columns of 10.4.3.2. The force p_u to be transferred by the collector element connected to a wall should be the difference in shear between the wall above and below the collector element. The wall shear difference is obtained from Eq. (4.15.4.1a).

The longitudinal reinforcement area in the collector element, A_{st} , should be equal to or greater than the area indicated by Eq. (5.12.3.1) and Eq. (5.12.5). For application of Eq. (5.12.3.1), A_g should be the gross cross-sectional area of the collector element.

CHAPTER 5—GENERAL REINFORCED CONCRETE INFORMATION

5.1—Scope

Chapter 5 contains general information for reinforced concrete structural members, including allowable materials, concrete cover to reinforcement, and procedures for defining design strength of members subjected to flexure, axial loads with or without flexure, and shear. Additional information for each individual member type is presented in **Chapters 6 to 16**.

5.2—Materials for reinforced concrete

5.2.1 Material standards—All materials used in construction of a structure designed following this guide should conform to the ASTM standards listed in **Chapter 17**.

5.2.2 Cement—Cement should conform to either **ASTM C150/C150M** or **ASTM C595/C595M**, excluding Types S and SA. ASTM C595/C595M Types S and SA are not intended as principal cementing constituents of structural concrete.

5.2.3 Aggregates—Aggregates should conform to **ASTM C33/C33M**. Concrete with lightweight aggregates are not within the scope of this guide.

5.2.4 Water—Water should conform to **ASTM C1602/C1602M**.

5.2.5 Steel reinforcement—Steel reinforcement should be deformed reinforcement, with exceptions noted in 5.2.5.3, conform to 5.2.5.1 through 5.2.5.3, and comply with the corresponding ASTM standards. Welded wire reinforcement should be considered deformed reinforcement.

5.2.5.1 Deformed reinforcement—Maximum specified yield strength for deformed reinforcement should be 60,000 psi (420 MPa). Deformed reinforcing bars should conform to **ASTM A615/A615M** or **ASTM A706/A706M**.

ASTM A615/A615M covers Grades 40, 60, and 75, with yield strengths of 40,000, 60,000, and 75,000 psi (280, 420, and 520 MPa), respectively, whereas ASTM A706/A706M covers Grades 60 (420) and 75 (520) with yield strengths of 60,000 and 75,000 psi (420 and 520 MPa). Under this guide, only yield strengths of 40,000 and 60,000 psi (280 and 420 MPa) are permitted.

Both standards cover the nominal diameters shown in Table 5.2.5.1. Under this guide, the maximum nominal diameter of deformed reinforcing bars is No. 8 (1 in. [25 mm]) (refer to 5.3). ASTM A615/A615M and A706/A706M standards do not include No. 2 (1/4 in.) or 6 mm diameter bars.

5.2.5.2 Welded wire reinforcement—Maximum specified yield strength for wires in welded wire reinforcement should be 70,000 psi (500 MPa). Welded wire reinforcement should conform to **ASTM A1064/A1064M**. Under the guide, the maximum nominal diameter of wire for welded wire reinforcement is 3/8 in. (10 mm) (5.3).

5.2.5.3 Plain reinforcement—Plain reinforcement may be used only for stirrups, ties, spirals, and when it is part of welded wire reinforcement, and should conform to the specifications listed in 5.2.5.1. Maximum specified yield strength for plain reinforcement should be 40,000 psi (280 MPa) and, under this guide, the maximum nominal diameter of plain

Table 5.2.5.1—Nominal characteristics for reinforcing bars

Bar denomination*	Nominal		
	Diameter, in. (mm)	Area, in. ² (mm ²)	Weight, lb/ft (kg/m)
No. 2 (6) [†]	0.250 (6.4)	0.05 (32)	0.170 (0.250)
No. 3 (10)	0.375 (9.5)	0.11 (71)	0.376 (0.560)
No. 4 (13)	0.500 (12.7)	0.20 (129)	0.668 (0.994)
No. 5 (16)	0.625 (15.9)	0.31 (199)	1.043 (1.552)
No. 6 (19)	0.750 (19.1)	0.44 (284)	1.502 (2.235)
No. 7 (22)	0.875 (22.2)	0.60 (387)	2.044 (3.042)
No. 8 (25)	1.000 (25.4)	0.79 (510)	2.670 (3.973)
No. 9 (29) [‡]	1.125 (28.7)	1.00 (645)	3.400 (5.060)
No. 10 (32) [‡]	1.250 (32.3)	1.27 (819)	4.303 (6.404)
No. 11 (35) [‡]	1.375 (35.8)	1.56 (1006)	5.313 (7.907)
No. 14 (43) [‡]	1.750 (43.0)	2.25 (1452)	7.650 (11.380)
No. 18 (57) [‡]	2.250 (57.3)	4.00 (2581)	13.600 (20.240)

*Bar No. indicates the number of 1/8 in. (3 mm) corresponding to the bar diameter and the metric denominations of the rounded bar diameter in mm as designated by ASTM.

[†]No. 2 (1/4 in. [6 mm]) bars are not manufactured routinely in the United States.

[‡]The maximum nominal bar diameter permitted by this guide is No. 8 (1 in. [25 mm]).

Table 5.2.6—ASTM standard references for admixtures

Purpose	Standard
Water reduction and setting time modification	ASTM C494/C494M
Produce flowing concrete	ASTM C1017/C1017M
Air entrainment	ASTM C260/C260M

reinforcing bars is 5/8 in. (16 mm) (5.3). Plain reinforcement is not allowed for use in the United States.

5.2.6 Admixtures—Admixtures should conform to the ASTM standards in Table 5.2.6. Additional admixtures may be used if approved by a licensed design professional. Calcium chloride or admixtures containing chlorides should not be used in concrete having prestressed steel, or in concrete that is in contact with galvanized steel.

The use of admixtures in concrete containing expansive cement, as specified by **ASTM C845/C845M**, should be used with caution. The admixture should be proven compatible with the cement without producing harmful effects.

5.2.7 Storage of materials—It is important that cement and aggregates be stored to prevent contamination, deterioration, and intrusion of foreign matter. Any material that has deteriorated or has been contaminated should not be used for concrete.

5.3—Minimum and maximum reinforcing bar diameter

Reinforcement used in structures designed under this guide should have a nominal diameter d_b not less than the minimum diameter, nor greater than the maximum diameter given in Table 5.3.



Documents

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5.4—Concrete cover for reinforcement

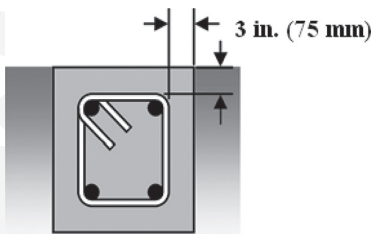
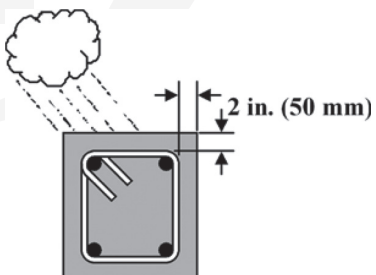
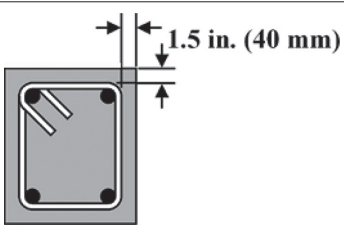
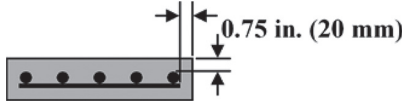
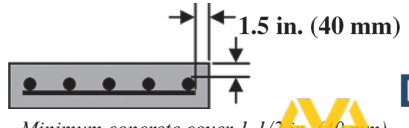
5.4.1 Minimum concrete cover—The minimum concrete cover to reinforcement that should be provided is shown in Table 5.4.1.

5.4.2 Special fire protection of the structure—When the designated fire rating, in hours, for the building is greater than 1 hour, the concrete cover of 5.4.1 should be increased by 1/4 in. (6 mm) per each additional hour of fire rating.

Table 5.3—Minimum and maximum reinforcing bar diameters used in structures

Reinforcement type (reference section)	Minimum bar diameter d_b	Maximum bar diameter d_b
Deformed reinforcing bars (5.2.5.1)	3/8 in. (10 mm)	1 in. (25 mm)
Wire for welded wire reinforcement (5.2.5.2)	0.16 in. (4 mm)	3/8 in. (10 mm)
For stirrups and ties (5.2.5.1)	3/8 in. (10 mm)	5/8 in. (16 mm)
Plain reinforcing bars (5.2.5.3)	3/8 in. (10 mm)	5/8 in. (16 mm)

Table 5.4.1—Minimum concrete cover to reinforcement

Members cast against and permanently exposed to earth	 Minimum concrete cover 3 in. (75 mm)
Members exposed to weather or earth	 Minimum concrete cover 2 in. (50 mm)
Girders, beams, or columns, when not exposed to weather or in contact with ground	 Minimum concrete cover 1-1/2 in. (40 mm)
Solid slabs, reinforced concrete walls, or joists, when not exposed to weather or in contact with ground	 Minimum concrete cover 3/4 in. (20 mm)
Solid slabs-on-ground	 Minimum concrete cover 1-1/2 in. (40 mm)

5.4.3 Additional corrosion protection—In very aggressive environments, such as direct contact with seawater, chemical facilities, or sewage water facilities, additional corrosion protection of the reinforcement should be specified, such as reduced water-cementitious material ratio (w/cm), epoxy-coated bars, air-entrained concrete, and other means. This type of protection is beyond the scope of this guide, **ACI 318**, **ASCE 7**, and the International Building Code (**International Code Council 2015**) should be used for design in that case.

5.5—Minimum reinforcement bend diameter

Bend diameter, measured on the inside of the bar, should not be less than the values shown in Table 5.5.

5.6—Standard hook dimensions

The term “standard hook” refers to one of the items in Table 5.6.

Table 5.5—Bend diameter minimum values

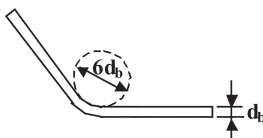
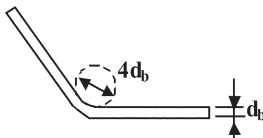
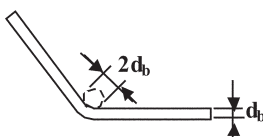
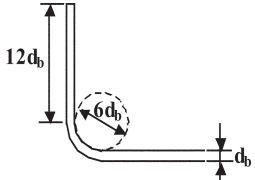
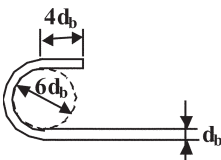
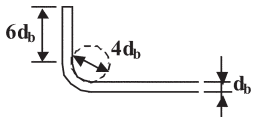
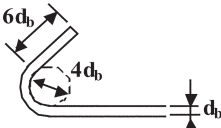
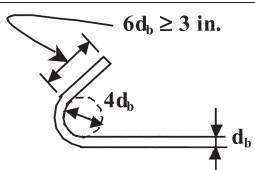
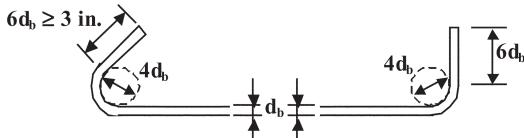
Reinforcement type	Diameter of bend	
Deformed reinforcing bars	$6d_b$	
Plain reinforcing bars	$6d_b$	
For stirrups and ties	$4d_b$	
Wire for welded wire reinforcement with $d_b > 1/4$ in. (6 mm)	$4d_b$	
Wire for welded wire reinforcement with $d_b \leq 1/4$ in. (6 mm)	$2d_b$	

Table 5.6—Standard hook description and dimensions

Hook designation	Description and dimensions	
90-degree hook	A 90-degree bend plus minimum $12d_b$ extension at free end of bar	
180-degree hook	A 180-degree bend plus minimum $4d_b$ extension at free end of bar	
For stirrup and tie hooks	A 90-degree bend plus minimum $6d_b$ extension at free end of bar, or	
	A 135-degree bend plus minimum $6d_b$ extension at free end of bar	
For hoops (confinement stirrups and ties) in seismic zones	A 135-degree bend plus minimum $6d_b$ extension at free end of bar, but not less than 3 in. (75 mm)	
For crossties in seismic zones	A 135-degree bend plus minimum $6d_b$ extension at one free end of bar, but not less than 3 in. (75 mm), and	

5.7—Maximum aggregate size

Maximum nominal coarse aggregate size should not be larger than (a), (b), or (c) (Fig. 5.7).

- One-fifth of the narrowest dimension between sides of forms
- One-third of the depth of slabs



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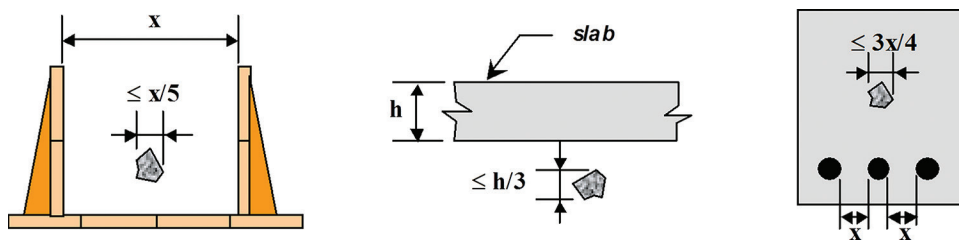


Fig. 5.7—Maximum nominal coarse aggregate size.

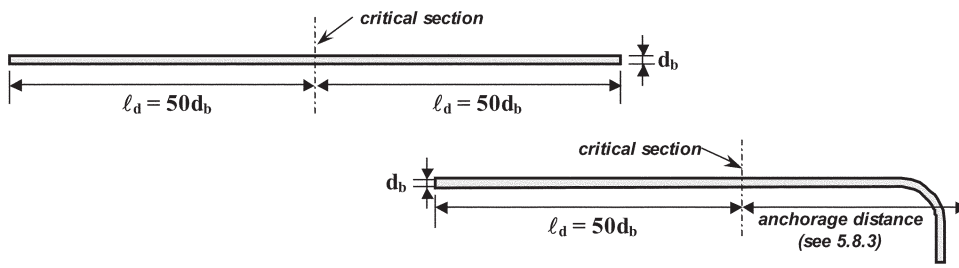
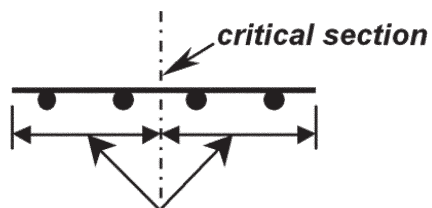


Fig. 5.8.1.1—Development length for reinforcing bars.



$\ell_d = 2 \text{ crosswires} \geq 8 \text{ in. (200 mm)}$

Fig. 5.8.1.2—Development length for welded wire reinforcement.

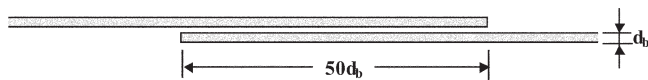


Fig. 5.8.2.1—Minimum lap splice length for reinforcing bars.

c) Three-fourths of the minimum clear spacing between parallel reinforcing bars

A maximum nominal size of 3/4 in. (19 mm) is recommended for columns, girders, beams, and joists. Except for structural slabs, a larger size can be used if it meets the limits given in (a) through (c).

5.8—Development length, lap splicing, and anchorage of reinforcement

5.8.1 Development length

5.8.1.1 Reinforcing bars—The minimum length of embedment, ℓ_d , on each side of a critical section for a reinforcing bar to develop its full strength should be $50d_b$. Development length on one side of the critical section may be replaced by a standard hook complying with the minimum standard hook anchorage distance of 5.8.3 (Fig. 5.8.1.1).

5.8.1.2 Welded wire reinforcement—Development length ℓ_d of welded wire reinforcement measured on each side of the critical section to the end of the wire should contain two crosswires, and should not be less than 8 in. (200 mm) (Fig. 5.8.1.2).

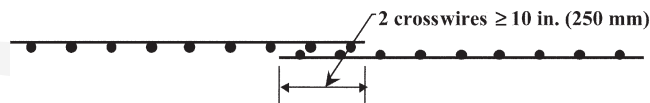


Fig. 5.8.2.2—Minimum lap splice length for welded wire reinforcement.

5.8.2 Lap splice dimensions

5.8.2.1 Reinforcing bars—Minimum lap length for splicing reinforcing bars should be $50d_b$. For buildings within the scope of this guide, this lap length calculation is a satisfactory simplification of the ACI 318 detailed calculation for splice length (Fig. 5.8.2.1). The maximum nominal bar diameter permitted by this guide is 1 in. (25 mm).

5.8.2.2 Welded wire reinforcement—A welded wire reinforcement splice should contain two crosswires from each sheet and should not be less than 10 in. (250 mm) (Fig. 5.8.2.2).

5.8.3 Minimum standard hook anchorage distance—The bent portion of the hook should be located as close to the outer face of concrete as cover permits. Minimum distance between the outer face of concrete and the critical section where the hooked bar develops its full strength is $25d_b$ (Fig. 5.8.3).

5.9—Longitudinal reinforcement

Longitudinal reinforcement in reinforced concrete members should be provided to resist axial tension, axial compression, flexural-induced tension and compression, and stresses induced by temperature variation and drying shrinkage of concrete.

The area of longitudinal reinforcement placed in reinforced concrete members should be sufficient to resist the factored loads but not less than the minimum value nor greater than the maximum value.

When the calculated area of longitudinal reinforcement is less than the minimum value, no less than the minimum value should be provided. If the calculated area exceeds the maximum limit, reinforced concrete member dimensions should be appropriately modified.

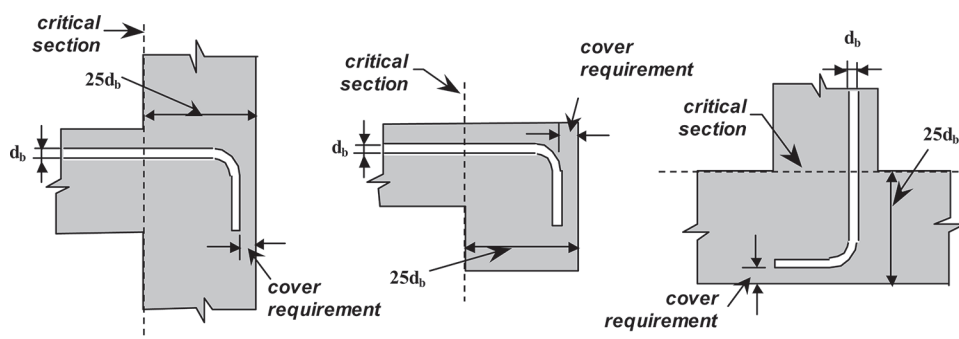


Fig. 5.8.3—Minimum standard hook anchorage distance

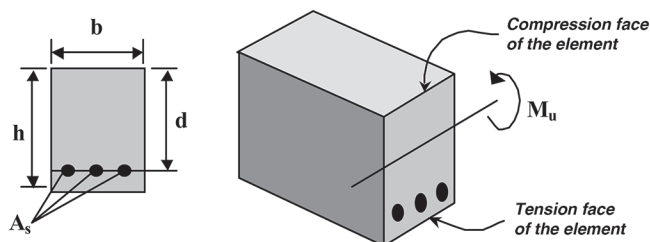


Fig. 5.11.3—Design dimensions for design moment strength.

5.10—Transverse reinforcement

Transverse reinforcement in reinforced concrete members resists shear and torsion loads, confines compression bars against buckling out of the concrete, and prevents displacement of longitudinal reinforcement during construction operations. In seismic zones, transverse reinforcement provides concrete confinement in special regions subjected to large strains. The area of transverse reinforcement placed in members should be sufficient to resist factored loads but not less than the minimum values.

When the calculated area of transverse reinforcement is less than the minimum value, no less than the minimum value should be placed. If the calculated area of transverse reinforcement exceeds the maximum limit, member dimensions should be appropriately modified.

5.11—Flexure

5.11.1 General—The design moment strength of sections should be computed by 5.11. Where the factored axial load on the member, P_u , exceeds $0.10f'_cA_g$ or produces axial tension, design strength should be computed by 5.12.

5.11.2 Required moment strength—The factored moment M_u (required moment strength) due to the applied factored loads should be determined for the particular member type from Chapters 7 to 14.

5.11.3 Design moment strength—The design moment strength of the section, ϕM_n , computed using the assumed dimensions, material strengths, and reinforcement, should be greater than or equal to the required moment strength M_u .

$$\phi M_n \geq M_u \quad (5.11.3)$$

Longitudinal reinforcement should be placed near the member face where flexural moment creates tension (Fig. 5.11.3).

5.11.4 Design moment strength for rectangular sections with tension reinforcement only

5.11.4.1 Design moment strength—Based on the assumption that the steel yields, for a section with tension reinforcement only, the design moment strength should be determined using Eq. (5.11.4.1), with $\phi = 0.90$ (Fig. 5.11.4.1).

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right) \text{ and } a = \frac{A_s f_y}{0.85 f'_c b} \quad (5.11.4.1)$$

In slabs, girders, beams, and joists the design moment strength may be suitably approximated by using Eq. (5.11.4.2), with $\phi = 0.90$.

$$\phi M_n = \phi 0.85 A_s f_y d \quad (5.11.4.2)$$

5.11.4.2 Flexural tension reinforcement area—Flexural tension reinforcement ratio, $\rho = A_s/(bd)$, should be determined using the factored moment M_u as

$$\rho = \frac{A_s}{bd} = \alpha_f - \sqrt{\alpha_f^2 - \left(\frac{M_u}{\phi b d^2} \cdot \frac{2\alpha_f}{f_y} \right)} \quad (5.11.4.3)$$

where $\alpha_f = \frac{f'_c}{1.18 f_y}$

In slabs, girders, beams, and joists the flexural tension reinforcement ratio, $\rho = A_s/(bd)$, can also be estimated from Eq. (5.11.4.4).

$$\rho = \frac{A_s}{bd} \geq \frac{M_u}{\phi 0.85 f_y b d^2} \quad (5.11.4.4)$$

If the computed ρ is less than the minimum, ρ_{min} , as established in Chapters 7 to 14, A_s should be increased to equal or exceed the minimum. When the computed ρ is greater than the maximum, ρ_{max} , member dimensions and the self-weight moment should be appropriately modified. For girders, beams, and joists, when the computed ρ is greater than ρ_{max} (Table 5.11.4.2), use of compression reinforcement should be investigated.

A note on employment of inch-pound units: The factored moments generally are in units of lb-ft and factored loads are determined from concentrated loads in lb, distributed loads

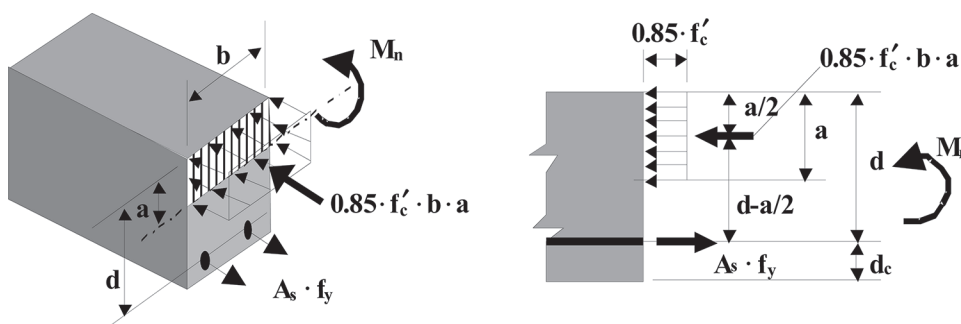


Fig. 5.11.4.1—Flexural nominal moment strength.

Table 5.11.4.2—Maximum flexural reinforcement ratio ρ_{max} for solid slabs

		f_y , psi (MPa)	
		40,000 (280)	60,000 (420)
f'_c , psi (MPa)	3000 (21)	0.0190	0.0100
	3500 (25)	0.0220	0.0125
	4000 (28)	0.0250	0.0140
	4500 (32)	0.0270	0.0160
	5000 (36)	0.0290	0.0170

Note: Different values of f_y and f'_c can be interpolated.

in lb/ft, or loads per unit area in lb/ft², and spans in ft. They should be converted to lb·in. (12 lb·in. = 1 lb·ft) for use with f_y in psi, d and b in inches, and A_s in in.² For slabs in which the units of moment are lb·ft per ft of slab width (lb·ft/ft), the same conversion (12 lb·in./ft = 1 lb·ft/ft) should be used and the A_s determined will be in in.²/ft of slab width.

A note on employment of SI units: The required factored moments generally are in units of kN·m because they are determined from concentrated loads in kN, distributed loads in kN/m, or loads per unit area in kN/m², and spans in m. They should be converted to N·mm (1 kN·m = 10⁶ N·mm) for use with f_y in MPa (1 MPa = 1 N/mm²), d and b in mm, and A_s in mm². For slabs in which the units of moment are kN·m per meter of slab width (kN·m/m), the same conversion (1 kN·m/m = 10⁶ N·mm/m) should be used and the A_s determined will be in mm²/m of slab width.

5.12—Axial loads with or without flexure

5.12.1 General—Calculation of the design strength of columns and reinforced concrete walls subjected to axial loads alone or axial loads accompanied by flexure should be in accordance with 5.12.

5.12.2 Combined axial load and moment—The factored axial load P_u and the factored moment M_u , due to applied factored loads, should be determined for the particular member type from Chapters 7 to 14.

5.12.3 Design axial compression strength

5.12.3.1 Design axial compression strength without flexure—Equation (5.12.3.1) should be used to determine the design axial strength without flexure, ϕP_{on} .

$$\phi P_{on} = \phi [0.85f'_c(A_g - A_{st}) + A_{st}f_y] \quad (5.12.3.1)$$

where $\phi = 0.65$ for columns with ties and reinforced concrete walls, and $\phi = 0.70$ for columns with spiral reinforcement.

5.12.3.2 Maximum design axial compression strength—Design axial strength ϕP_n in columns and reinforced concrete walls subjected to compression, with or without flexure, should not exceed the following:

(a) Columns with ties and reinforced concrete walls

$$\phi P_{n(max)} \leq 0.80\phi P_{on} \quad (\text{with } \phi = 0.65) \quad (5.12.3.2a)$$

(b) Columns with spiral reinforcement

$$\phi P_{n(max)} \leq 0.85\phi P_{on} \quad (\text{with } \phi = 0.70) \quad (5.12.3.2b)$$

5.12.4 Balanced design strength for axial load with flexure

5.12.4.1 Square and rectangular tied columns and reinforced concrete walls—Values for design axial compression strength ϕP_{bn} and design moment strength ϕM_{bn} at the balanced design strength point should be determined using Eq. (5.12.4.1a) and Eq. (5.12.4.1b), respectively, with $\phi = 0.65$.

$$\phi P_{bn} = \phi 0.42f'_c h b \quad (5.12.4.1a)$$

$$\phi M_{bn} = \phi P_{bn} 0.32h + \phi [0.6A_{se} + 0.15A_{ss}] f_y \left(\frac{h}{2} - d' \right) \quad (5.12.4.1b)$$

For Eq. (5.12.4.1b), the total longitudinal reinforcement area A_{st} should be divided into steel at the extreme face, A_{se} , and steel at the side face, A_{ss} , in such a manner that $A_{se} + A_{ss} = A_{st}$ (Fig. 5.12.4.1).

5.12.4.2 Circular columns with spiral reinforcement—Values for design axial compression strength ϕP_{bn} and design moment strength ϕM_{bn} at the balanced design strength point should be determined using Eq. (5.12.4.2a) and Eq. (5.12.4.2b), respectively, with h taken as the column diameter, and $\phi = 0.70$ (Fig. 5.12.4.2).

$$\phi P_{bn} = \phi 0.5f'_c A_{cs} \quad (5.12.4.2a)$$

$$\phi M_{bn} = \phi P_{bn} 0.2h + \phi 0.6A_{st} f_y \left(\frac{h}{2} - d' \right) \quad (5.12.4.2b)$$

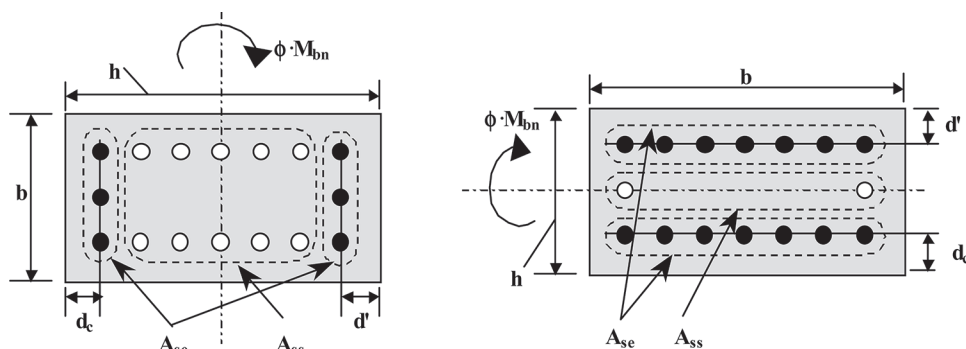


Fig. 5.12.4.1—Dimensions for calculation of balanced design strength for rectangular tied-columns and reinforced concrete walls.

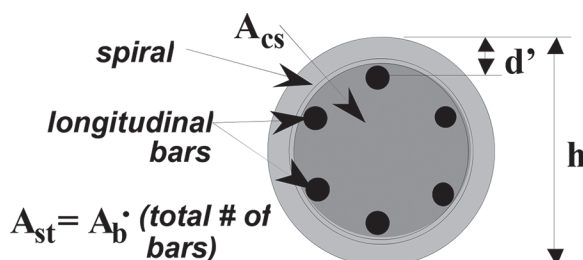


Fig. 5.12.4.2—Dimensions for calculation of balanced design strength for circular section columns with spiral reinforcement.

5.12.5 Design strength for axial tension without flexure—The design axial tension strength without flexure, ϕP_{tn} , should be determined using Eq. (5.12.5) with $\phi = 0.90$.

$$\phi P_{tn} = \phi A_{st} f_y \quad (5.12.5)$$

5.12.6 Combined axial load and moment strength—The design moment strength at the section, ϕM_n , at the level of applied factored axial load, P_u , should be equal to or greater than the maximum factored moment M_u that can accompany the factored axial load P_u , as shown in Eq. (5.12.6a).

$$\phi M_n \geq M_u \quad (5.12.6a)$$

The compliance with Eq. (5.12.6a) should be accomplished by proving that the coordinates of M_u — P_u in a moment versus axial load interaction diagram relating ϕM_n and ϕP_{bn} —are inside the interaction design strength surface, the shaded portion in Fig. 5.12.6.

The following conditions should be met for all couples of M_u and P_u that act on the column section

$$P_u \leq \phi P_{n(max)} \quad (5.12.6b)$$

$$P_u \geq -(\phi P_{tn}) \quad (5.12.6c)$$

For values of $P_u \geq \phi P_{bn}$

$$M_u \leq \phi M_n = \frac{\phi P_{on} - P_u}{\phi P_{on} - \phi P_{bn}} \phi M_{bn} \quad (5.12.6d)$$

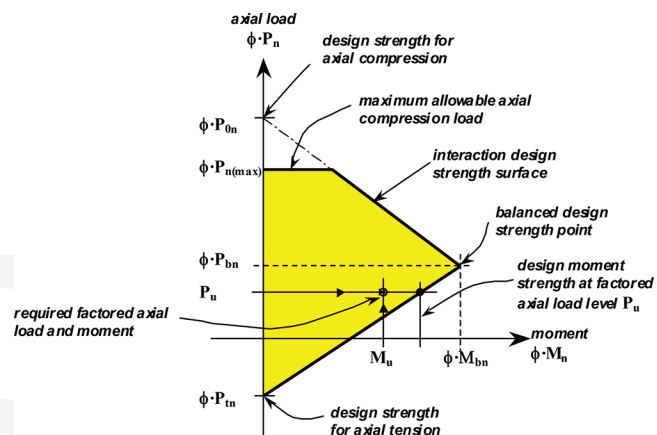


Fig. 5.12.6—Interaction diagram for $(\phi M_n, \phi P_n)$.

For values of $P_u < \phi P_{bn}$

$$M_u \leq \phi M_n = \frac{P_u + \phi P_{tn}}{\phi P_{bn} + \phi P_{tn}} \phi M_{bn} \quad (5.12.6e)$$

5.12.7 Use of interaction diagrams—Interaction diagrams for columns from authoritative sources can be used; however, use the strength reduction factors ϕ as set forth in this guide.

5.12.8 Biaxial moment strength—Corner columns and other columns subjected to moments about each principal section axis simultaneously (Fig. 5.12.8) should comply with Eq. (5.12.8).

$$\frac{(M_u)_x}{(\phi M_n)_x} + \frac{(M_u)_y}{(\phi M_n)_y} \leq 1.0 \quad (5.12.8)$$

where $(M_u)_x$ and $(M_u)_y$ are the factored moments that act about axis x and y (Fig. 5.12.6) simultaneously with the factored axial load P_u . $(\phi M_n)_x$ and $(\phi M_n)_y$ correspond to values of the design moment strength determined from Eq. (5.12.6d) or Eq. (5.12.6e) for the factored axial load P_u and for the appropriate direction x or y .

5.13—Shear

5.13.1 General—Calculation of the design shear strength of sections subjected to shear loads should be made using 5.13. There are two types of shear effects (Fig. 5.13.1):

(a) Beam-action shear that accompanies flexural moments and occurs in girders, beams, joists, solid slabs, and reinforced concrete walls, in the vicinity of supports and concentrated loads; and

(b) Punching-shear or two-way action shear, which occurs in solid slabs and footings, near supports and concentrated loads.

Other types of shear effects, such as special effects in deep flexural members, shear-friction used when designing brackets and corbels, and strut-and-tie models, are beyond the scope of this guide and ACI 318, ASCE 7, and the International Building Code (International Code Council 2015) should be used for design in those cases.

5.13.2 Required shear strength—The factored shear force V_u (required shear strength) due to the applied factored loads should be determined, for the particular member type, from Chapters 7 to 14.

5.13.3 Design shear strength—The design shear strength at a section, ϕV_n , should be equal to or greater than the required shear strength V_u , as shown in Eq. (5.13.3).

$$\phi V_n \geq V_u \quad (5.13.3)$$

where $\phi = 0.75$.

5.13.4 Design beam-action shear strength—Section 5.13.4 should be used when designing members for beam-action shear. Sections 5.13.4.1, 5.13.4.2, and 5.13.4.3 apply.

5.13.4.1 Location of critical section—Where the support reaction, in the direction of the applied shear, introduces compression into the end regions of the member, and no concentrated load occurs between the support face and a

distance d from the support face for girders, beams, joists, columns, slabs, and footings, the sections in between may be designed for the factored shear force V_u , computed at distance d from the support.

5.13.4.2 Where shear reinforcement is not permitted—Where shear reinforcement is not permitted in Chapters 7 to 14, the design shear strength ϕV_n should be computed using Eq. (5.13.4.2).

$$\phi V_n = \phi V_c \quad (5.13.4.2)$$

In Eq. (5.13.4.2), $\phi = 0.75$, and ϕV_c is the concrete contribution to the design shear strength.

5.13.4.3 Where shear reinforcement is permitted—Where shear reinforcement is permitted in Chapters 7 to 14, the design shear strength ϕV_n should be computed using Eq. (5.13.4.3) with $\phi = 0.75$.

$$\phi V_n = \phi(V_c + V_s) \quad (5.13.4.3)$$

In Eq. (5.13.4.3), ϕV_c is the concrete contribution to the design shear strength, and ϕV_s is the shear reinforcement contribution to the design shear strength.

5.13.5 Two-way action (punching) design shear strength in solid slabs and footings

5.13.5.1 Location of critical section—The design shear strength for punching shear, or two-way action shear, should be investigated in slabs at column edges, concentrated loads, supports, thickness changes such as edges of capitals and drop panels, and where columns transfer load to a footing.

5.13.5.2 Two-way action design shear strength—The design shear strength ϕV_n should be computed using Eq. (5.13.5.2).

$$\phi V_n = \phi V_c \quad (5.13.5.2)$$

where $\phi = 0.75$.

In Eq. (5.13.5.2), ϕV_c is the concrete contribution to the design shear strength. Procedures for design of transverse or shear reinforcement in slabs and footings are beyond the scope of the guide. ACI 318, ASCE 7, and the International Building Code (International Code Council 2015) should be used for design in that case.

5.13.6 Torsion—Design for torsion is beyond the scope of the guide, and torsion effects should be neglected when the calculated required torsion strength T_u is less than the value determined from Eq. (5.13.6)

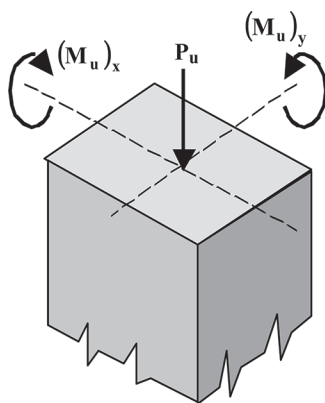


Fig. 5.12.8—Column subjected to biaxial moments.

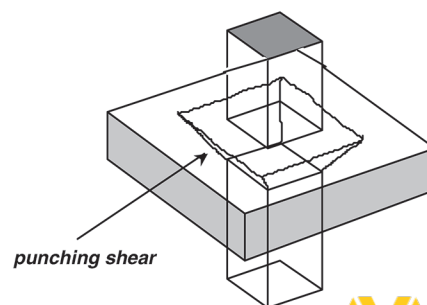
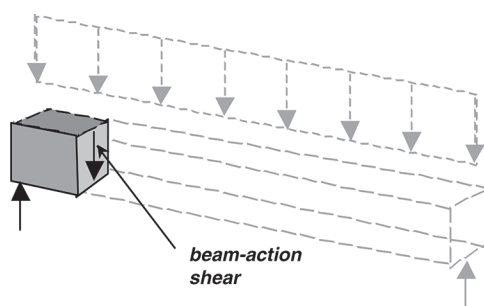


Fig. 5.13.1—Beam-action shear and punching shear.