

CHAPTER 3

Design Loads for Residential Buildings

3.1 General

Loads are a primary consideration in any building design because they define the nature and magnitude of hazards or external forces that a building must resist to provide reasonable performance (i.e., safety and serviceability) throughout the structure's useful life. The anticipated loads are influenced by a building's intended use (occupancy and function), configuration (size and shape), and location (climate and site conditions). Ultimately, the type and magnitude of design loads affect critical decisions such as material selection, construction details, and architectural configuration. Thus, to optimize the value (i.e., performance versus economy) of the finished product, it is essential to apply design loads realistically.

While the buildings considered in this guide are primarily single-family detached and attached dwellings, the principles and concepts related to building loads also apply to other similar types of construction, such as low-rise apartment buildings. In general, the design loads recommended in this guide are based on applicable provisions of the ASCE 7 standard—*Minimum Design Loads for Buildings and Other Structures* (ASCE, 1999). The ASCE 7 standard represents an acceptable practice for building loads in the United States and is recognized in virtually all U.S. building codes. For this reason, the reader is encouraged to become familiar with the provisions, commentary, and technical references contained in the ASCE 7 standard.

In general, the structural design of housing has not been treated as a unique engineering discipline or subjected to a special effort to develop better, more efficient design practices. Therefore, this part of the guide focuses on those aspects of ASCE 7 and other technical resources that are particularly relevant to the determination of design loads for residential structures. The guide provides supplemental design assistance to address aspects of residential construction where current practice is either silent or in need of improvement. The guide's



methods for determining design loads are complete yet tailored to typical residential conditions. As with any design function, the designer must ultimately understand and approve the loads for a given project as well as the overall design methodology, including all its inherent strengths and weaknesses. Since building codes tend to vary in their treatment of design loads the designer should, as a matter of due diligence, identify variances from both local accepted practice and the applicable building code relative to design loads as presented in this guide, even though the variances may be considered technically sound.

Complete design of a home typically requires the evaluation of several different types of materials as in Chapters 4 through 7. Some material specifications use the allowable stress design (ASD) approach while others use load and resistance factor design (LRFD). *Chapter 4 uses the LRFD method for concrete design and the ASD method for masonry design. For wood design, Chapters 5, 6, and 7 use ASD.* Therefore, for a single project, it may be necessary to determine loads in accordance with both design formats. This chapter provides load combinations intended for each method. The determination of individual nominal loads is essentially unaffected. Special loads such as flood loads, ice loads, and rain loads are not addressed herein. The reader is referred to the ASCE 7 standard and applicable building code provisions regarding special loads.

3.2 Load Combinations

The load combinations in Table 3.1 are recommended for use with design specifications based on allowable stress design (ASD) and load and resistance factor design (LRFD). Load combinations provide the basic set of building load conditions that should be considered by the designer. They establish the proportioning of multiple transient loads that may assume point-in-time values when the load of interest attains its extreme design value. Load combinations are intended as a guide to the designer, who should exercise judgment in any particular application. The load combinations in Table 3.1 are appropriate for use with the design loads determined in accordance with this chapter.

The principle used to proportion loads is a recognition that when one load attains its maximum life-time value, the other loads assume arbitrary point-in-time values associated with the structure's normal or sustained loading conditions. The advent of LRFD has drawn greater attention to this principle (Ellingwood et al., 1982; Galambos et al., 1982). The proportioning of loads in this chapter for allowable stress design (ASD) is consistent with and normalized to the proportioning of loads used in newer LRFD load combinations. However, this manner of proportioning ASD loads has seen only limited use in current code-recognized documents (AF&PA, 1996) and has yet to be explicitly recognized in design load specifications such as ASCE 7. ASD load combinations found in building codes have typically included some degree of proportioning (i.e., $D + W + 1/2S$) and have usually made allowance for a special reduction for multiple transient loads. Some earlier codes have also permitted allowable material stress increases for load combinations involving wind and earthquake loads. None of these adjustments for ASD load combinations is recommended for use with Table 3.1 since the load proportioning is considered sufficient.



It should also be noted that the wind load factor of 1.5 in Table 3.1 used for load and resistant factor design is consistent with traditional wind design practice (ASD and LRFD) and has proven adequate in hurricane-prone environments when buildings are properly designed and constructed. The 1.5 factor is equivalent to the earlier use of a 1.3 wind load factor in that the newer wind load provisions of ASCE 7-98 include separate consideration of wind directionality by adjusting wind loads by an explicit wind directionality factor, K_D , of 0.85. Since the wind load factor of 1.3 included this effect, it must be adjusted to 1.5 in compensation for adjusting the design wind load instead (i.e., $1.5/1.3 = 0.85$). The 1.5 factor may be considered conservative relative to traditional design practice in nonhurricane-prone wind regions as indicated in the calibration of the LRFD load factors to historic ASD design practice (Ellingwood et al., 1982; Galambos et al., 1982). In addition, newer design wind speeds for hurricane-prone areas account for variation in the extreme (i.e., long return period) wind probability that occurs in hurricane hazard areas. Thus, the return period of the design wind speeds along the hurricane-prone coast varies from roughly a 70- to 100-year return period on the wind map in the 1998 edition of ASCE 7 (i.e., not a traditional 50-year return period wind speed used for the remainder of the United States). The latest wind design provisions of ASCE 7 include many advances in the state of the art, but the ASCE commentary does not clearly describe the condition mentioned above in support of an increased wind load factor of 1.6 (ASCE, 1999). Given that the new standard will likely be referenced in future building codes, the designer may eventually be required to use a higher wind load factor for LRFD than that shown in Table 3.1. The above discussion is intended to help the designer understand the recent departure from past successful design experience and remain cognizant of its potential future impact to building design.

The load combinations in Table 3.1 are simplified and tailored to specific application in residential construction and the design of typical components and systems in a home. These or similar load combinations are often used in practice as short-cuts to those load combinations that govern the design result. This guide makes effective use of the short-cuts and demonstrates them in the examples provided later in the chapter. The short-cuts are intended only for the design of residential light-frame construction.



TABLE 3.1 *Typical Load Combinations Used for the Design of Components and Systems¹*

Component or System	ASD Load Combinations	LRFD Load Combinations
Foundation wall (gravity and soil lateral loads)	D + H D + H + L ² + 0.3(L _r + S) D + H + (L _r or S) + 0.3L ²	1.2D + 1.6H 1.2D + 1.6H + 1.6L ² + 0.5(L _r + S) 1.2D + 1.6H + 1.6(L _r or S) + 0.5L ²
Headers, girders, joists, interior load-bearing walls and columns, footings (gravity loads)	D + L ² + 0.3 (L _r or S) D + (L _r or S) + 0.3 L ²	1.2D + 1.6L ² + 0.5 (L _r or S) 1.2D + 1.6(L _r or S) + 0.5 L ²
Exterior load-bearing walls and columns (gravity and transverse lateral load) ³	Same as immediately above plus D + W D + 0.7E + 0.5L ² + 0.2S ⁴	Same as immediately above plus 1.2D + 1.5W 1.2D + 1.0E + 0.5L ² + 0.2S ⁴
Roof rafters, trusses, and beams; roof and wall sheathing (gravity and wind loads)	D + (L _r or S) 0.6D + W _u ⁵ D + W	1.2D + 1.6(L _r or S) 0.9D + 1.5W _u ⁵ 1.2D + 1.5W
Floor diaphragms and shear walls (in-plane lateral and overturning loads) ⁶	0.6D + (W or 0.7E)	0.9D + (1.5W or 1.0E)

Notes:

¹The load combinations and factors are intended to apply to nominal design loads defined as follows: D = estimated mean dead weight of the construction; H = design lateral pressure for soil condition/type; L = design floor live load; L_r = maximum roof live load anticipated from construction/maintenance; W = design wind load; S = design roof snow load; and E = design earthquake load. The design or nominal loads should be determined in accordance with this chapter.

²Attic loads may be included in the floor live load, but a 10 psf attic load is typically used only to size ceiling joists adequately for access purposes. However, if the attic is intended for storage, the attic live load (or some portion) should also be considered for the design of other elements in the load path.

³The transverse wind load for stud design is based on a localized component and cladding wind pressure; D + W provides an adequate and simple design check representative of worst-case combined axial and transverse loading. Axial forces from snow loads and roof live loads should usually not be considered simultaneously with an extreme wind load because they are mutually exclusive on residential sloped roofs. Further, in most areas of the United States, design winds are produced by either hurricanes or thunderstorms; therefore, these wind events and snow are mutually exclusive because they occur at different times of the year.

⁴For walls supporting heavy cladding loads (such as brick veneer), an analysis of earthquake lateral loads and combined axial loads should be considered. However, this load combination rarely governs the design of light-frame construction.

⁵W_u is wind uplift load from negative (i.e., suction) pressures on the roof. Wind uplift loads must be resisted by continuous load path connections to the foundation or until offset by 0.6D.

⁶The 0.6 reduction factor on D is intended to apply to the calculation of net overturning stresses and forces. For wind, the analysis of overturning should also consider roof uplift forces unless a separate load path is designed to transfer those forces.

3.3 Dead Loads

Dead loads consist of the permanent construction material loads comprising the roof, floor, wall, and foundation systems, including claddings, finishes, and fixed equipment. The values for dead loads in Table 3.2 are for commonly used materials and constructions in light-frame residential buildings. Table 3.3 provides values for common material densities and may be useful in calculating dead loads more accurately. The design examples in Section 3.10 demonstrate the straight-forward process of calculating dead loads.



TABLE 3.2 *Dead Loads for Common Residential Construction¹*

Roof Construction			
Light-frame wood roof with wood structural panel sheathing and 1/2-inch gypsum board ceiling (2 psf) with asphalt shingle roofing (3 psf)			15 psf
- with conventional clay/tile roofing			27 psf
- with light-weight tile			21 psf
- with metal roofing			14 psf
- with wood shakes			15 psf
- with tar and gravel			18 psf
Floor Construction			
Light-frame 2x12 wood floor with 3/4-inch wood structural panel sheathing and 1/2-inch gypsum board ceiling (without 1/2-inch gypsum board, subtract 2 psf from all values) with carpet, vinyl, or similar floor covering			10 psf ²
- with wood flooring			12 psf
- with ceramic tile			15 psf
- with slate			19 psf
Wall Construction			
Light-frame 2x4 wood wall with 1/2-inch wood structural panel sheathing and 1/2-inch gypsum board finish (for 2x6, add 1 psf to all values)			6 psf
- with vinyl or aluminum siding			7 psf
- with lap wood siding			8 psf
- with 7/8-inch portland cement stucco siding			15 psf
- with thin-coat-stucco on insulation board			9 psf
- with 3-1/2-inch brick veneer			45 psf
Interior partition walls (2x4 with 1/2-inch gypsum board applied to both sides)			6 psf
Foundation Construction	Masonry ³		Concrete
	Hollow	Solid or Full Grout	
6-inch-thick wall	28 psf	60 psf	75 psf
8-inch-thick wall	36 psf	80 psf	100 psf
10-inch-thick wall	44 psf	100 psf	123 psf
12-inch-thick wall	50 psf	125 psf	145 psf
6-inch x 12-inch concrete footing			73 plf
6-inch x 16-inch concrete footing			97 plf
8-inch x 24-inch concrete footing			193 plf

Notes:

¹For unit conversions, see Appendix B.

²Value also used for roof rafter construction (i.e., cathedral ceiling).

³For partially grouted masonry, interpolate between hollow and solid grout in accordance with the fraction of masonry cores that are grouted.

**TABLE 3.3** *Densities for Common Residential Construction Materials¹*

Aluminum	170 pcf
Copper	556 pcf
Steel	492 pcf
Concrete (normal weight with light reinforcement)	145–150 pcf
Masonry, grout	140 pcf
Masonry, brick	100–130 pcf
Masonry, concrete	85–135 pcf
Glass	160 pcf
Wood (approximately 10 percent moisture content) ²	
- spruce-pine-fir (G = 0.42)	29 pcf
- spruce-pine-fir, south (G = 0.36)	25 pcf
- southern yellow pine (G = 0.55)	38 pcf
- Douglas fir–larch (G = 0.5)	34 pcf
- hem-fir (G = 0.43)	30 pcf
- mixed oak (G = 0.68)	47 pcf
Water	62.4 pcf
Structural wood panels	
- plywood	36 pcf
- oriented strand board	36 pcf
Gypsum board	48 pcf
Stone	
- Granite	96 pcf
- Sandstone	82 pcf
Sand, dry	90 pcf
Gravel, dry	105 pcf

Notes:

¹For unit conversions, see Appendix B.²The equilibrium moisture content of lumber is usually not more than 10 percent in protected building construction. The specific gravity, G, is the decimal fraction of dry wood density relative to that of water. Therefore, at a 10 percent moisture content, the density of wood is $1.1(G)(62.4 \text{ lbs/ft}^3)$. The values given are representative of average densities and may easily vary by as much as 15 percent depending on lumber grade and other factors.

3.4 Live Loads

Live loads are produced by the use and occupancy of a building. Loads include those from human occupants, furnishings, nonfixed equipment, storage, and construction and maintenance activities. Table 3.4 provides recommended design live loads for residential buildings. Example 3.1 in Section 3.10 demonstrates use of those loads and the load combinations specified in Table 3.1, along with other factors discussed in this section. As required to adequately define the loading condition, loads are presented in terms of uniform area loads (psf), concentrated loads (lbs), and uniform line loads (plf). The uniform and concentrated live loads should not be applied simultaneously in a structural evaluation. Concentrated loads should be applied to a small area or surface



consistent with the application and should be located or directed to give the maximum load effect possible in end-use conditions. For example, the stair concentrated load of 300 pounds should be applied to the center of the stair tread between supports. The concentrated wheel load of a vehicle on a garage slab or floor should be applied to all areas or members subject to a wheel or jack load, typically using a loaded area of about 20 square inches.

TABLE 3.4 *Live Loads for Residential Construction*¹

Application	Uniform Load	Concentrated Load
Roof ²		
Slope \geq 4:12	15 psf	250 lbs
Flat to 4:12 slope	20 psf	250 lbs
Attic ³		
With limited storage	10 psf	250 lbs
With storage	20 psf	250 lbs
Floors		
Bedroom areas ^{3,4}	30 psf	300 lbs
Other areas	40 psf	300 lbs
Garages	50 psf	2,000 lbs (vans, light trucks) 1,500 lbs (passenger cars)
Decks	40 psf	300 lbs
Balconies	60 psf	300 lbs
Stairs	40 psf	300 lbs
Guards and handrails	20 plf	200 lbs
Grab bars	N/A	250 lbs

Notes:

¹Live load values should be verified relative to the locally applicable building code.

²Roof live loads are intended to provide a minimum load for roof design in consideration of maintenance and construction activities. They should not be considered in combination with other transient loads (i.e., floor live load, wind load, etc.) when designing walls, floors, and foundations. A 15 psf roof live load is recommended for residential roof slopes greater than 4:12; refer to ASCE 7-98 for an alternate approach.

³Loft sleeping and attic storage loads should be considered only in areas with a clear height greater than about 3 feet. The concept of a “clear height” limitation on live loads is logical, but it may not be universally recognized.

⁴Some codes require 40 psf for all floor areas.

The floor live load on any given floor area may be reduced in accordance with Equation 3.4-1 (Harris, Corotis, and Bova, 1980). The equation applies to floor and support members, such as beams or columns, that experience floor loads from a total tributary floor area greater than 200 square feet. This equation is different from that in ASCE 7-98 since it is based on data that applies to residential floor loads rather than commercial buildings.



[Equation 3.4-1]

$$L = L_o \left[0.25 + \frac{10.6}{\sqrt{A_t}} \right] \geq 0.75$$

where,

- L = the adjusted floor live load for tributary areas greater than 200 square feet
- A_t = the tributary from a single-story area assigned to a floor support member (i.e., girder, column, or footing)
- L_o = the unreduced live load associated with a floor area of 200 ft² from Table 3.4

It should also be noted that the nominal design floor live load in Table 3.4 includes both a sustained and transient load component. The sustained component is that load typically present at any given time and includes the load associated with normal human occupancy and furnishings. For residential buildings, the mean sustained live load is about 6 psf but typically varies from 4 to 8 psf (Chalk, Philip, and Corotis, 1978). The mean transient live load for dwellings is also about 6 psf but may be as high as 13 psf. Thus, a total design live load of 30 to 40 psf is fairly conservative.

3.5 Soil Lateral Loads

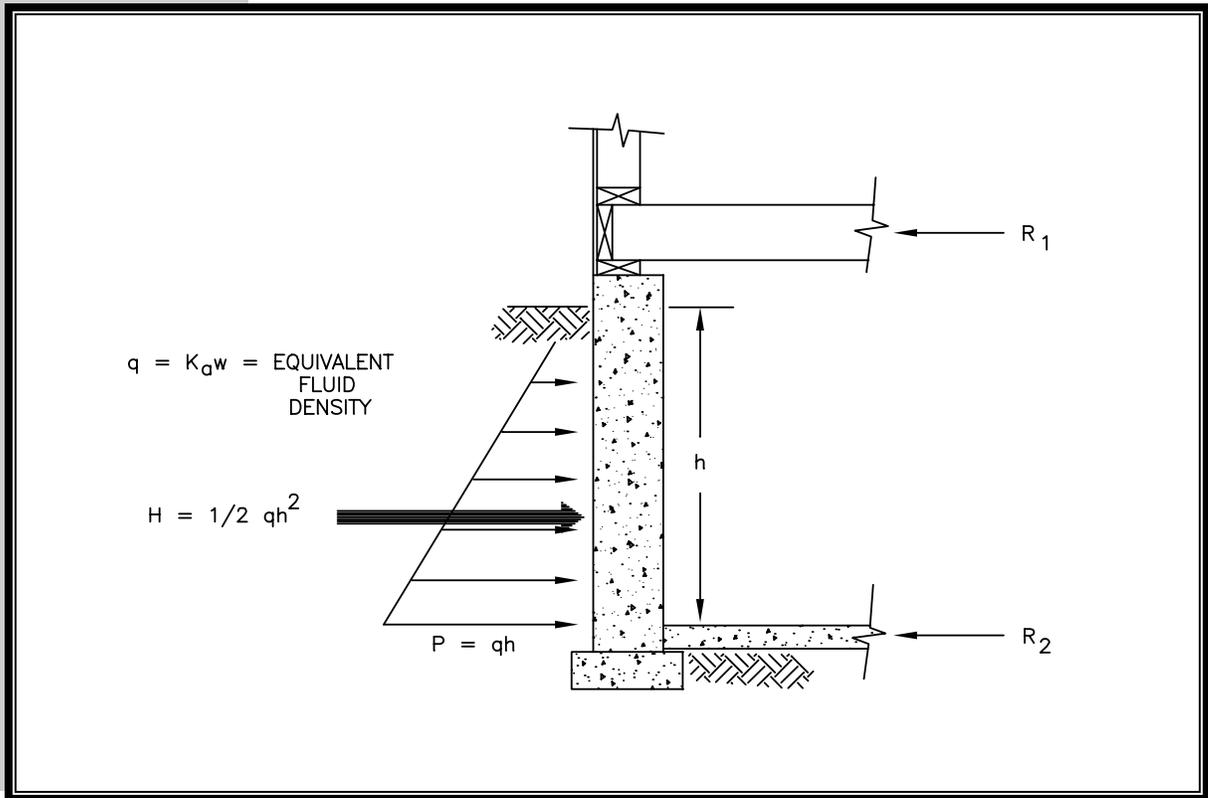
The lateral pressure exerted by earth backfill against a residential foundation wall (basement wall) can be calculated with reasonable accuracy on the basis of theory, but only for conditions that rarely occur in practice (University of Alberta, 1992; Peck, Hanson, and Thornburn, 1974). Theoretical analyses are usually based on homogeneous materials that demonstrate consistent compaction and behavioral properties. Such conditions are rarely experienced in the case of typical residential construction projects.

The most common method of determining lateral soil loads on residential foundations follows Rankine's (1857) theory of earth pressure and uses what is known as the Equivalent Fluid Density (EFD) method. As shown in Figure 3.1, pressure distribution is assumed to be triangular and to increase with depth.

In the EFD method, the soil unit weight w is multiplied by an empirical coefficient K_a to account for the fact that the soil is not actually fluid and that the pressure distribution is not necessarily triangular. The coefficient K_a is known as the active Rankine pressure coefficient. Thus, the equivalent fluid density (EFD) is determined as follows:

[Equation 3.5-1]

$$q = K_a w$$

**FIGURE 3.1*****Triangular Pressure Distribution
on a Basement Foundation Wall***

It follows that for the triangular pressure distribution shown in Figure 3.1, the pressure at depth, h , in feet is

[Equation 3.5-2] $P = qh$

The total active soil force (pounds per lineal foot of wall length) is

[Equation 3.5-3]
$$H = \frac{1}{2}(qh)(h) = \frac{1}{2}qh^2$$

where,

h = the depth of the unbalanced fill on a foundation wall

H = the resultant force (plf) applied at a height of $h/3$ from the base of the unbalanced fill since the pressure distribution is assumed to be triangular

The EFD method is subject to judgment as to the appropriate value of the coefficient K_a . The values of K_a in Table 3.5 are recommended for the determination of lateral pressures on residential foundations for various types of backfill materials placed with light compaction and good drainage. Given the long-time use of a 30 pcf equivalent fluid density in residential foundation wall prescriptive design tables (ICC, 1998), the values in Table 3.5 may be considered somewhat conservative for typical conditions. A relatively conservative safety factor of 3 to 4 is typically applied to the design of unreinforced or nominally reinforced masonry or concrete foundation walls (ACI 1999a and b). Therefore, at



imminent failure of a foundation wall, the 30 psf design EFD would correspond to an active soil lateral pressure determined by using an equivalent fluid density of about 90 to 120 pcf or more. The design examples in Chapter 4 demonstrate the calculation of soil loads.

TABLE 3.5

Values of K_a , Soil Unit Weight, and Equivalent Fluid Density by Soil Type^{1,2,3}

Type of Soil ⁴ (unified soil classification)	Active Pressure Coefficient (K_a)	Soil Unit Weight (pcf)	Equivalent Fluid Density (pcf)
Sand or gravel (GW, GP, GM, SW, SP)	0.26	115	30
Silty sand, silt, and sandy silt (GC, SM)	0.35	100	35
Clay-silt, silty clay (SM-SC, SC, ML, ML-CL)	0.45	100	45
Clay ⁵ (CL, MH, CH)	0.6	100	60

Notes:

¹Values are applicable to well-drained foundations with less than 10 feet of backfill placed with light compaction or natural settlement as is common in residential construction. The values do not apply to foundation walls in flood-prone environments. In such cases, an equivalent fluid density value of 80 to 90 pcf would be more appropriate (HUD, 1977).

²Values are based on the *Standard Handbook for Civil Engineers*, Third Edition, 1983, and on research on soil pressures reported in *Thin Wall Foundation Testing*, Department of Civil Engineering, University of Alberta, Canada, March 1992. It should be noted that the values for soil equivalent fluid density differ from those recommended in ASCE 7-98 but are nonetheless compatible with current residential building codes, design practice, and the stated references.

³These values do not consider the significantly higher loads that can result from expansive clays and the lateral expansion of moist, frozen soil. Such conditions should be avoided by eliminating expansive clays adjacent to the foundation wall and providing for adequate surface and foundation drainage.

⁴Organic silts and clays and expansive clays are unsuitable for backfill material.

⁵Backfill in the form of clay soils (nonexpansive) should be used with caution on foundation walls with unbalanced fill heights greater than 3 to 4 feet and on cantilevered foundation walls with unbalanced fill heights greater than 2 to 3 feet.

Depending on the type and depth of backfill material and the manner of its placement, it is common practice in residential construction to allow the backfill soil to consolidate naturally by providing an additional 3 to 6 inches of fill material. The additional backfill ensures that surface water drainage away from the foundation remains adequate (i.e., the grade slopes away from the building). It also helps avoid heavy compaction that could cause undesirable loads on the foundation wall during and after construction. If soils are heavily compacted at the ground surface or compacted in lifts to standard Proctor densities greater than about 85 percent of optimum (ASTM, 1998), the standard 30 pcf EFD assumption may be inadequate. However, in cases where exterior slabs, patios, stairs, or other items are supported on the backfill, some amount of compaction is advisable unless the structures are supported on a separate foundation bearing on undisturbed ground.



3.6 Wind Loads

3.6.1 General

Wind produces nonstatic loads on a structure at highly variable magnitudes. The variation in pressures at different locations on a building is complex to the point that pressures may become too analytically intensive for precise consideration in design. Therefore, wind load specifications attempt to simplify the design problem by considering basic static pressure zones on a building representative of peak loads that are likely to be experienced. The peak pressures in one zone for a given wind direction may not, however, occur simultaneously with peak pressures in other zones. For some pressure zones, the peak pressure depends on a narrow range of wind direction. Therefore, the wind directionality effect must also be factored into determining risk-consistent wind loads on buildings. In fact, most modern wind load specifications take account of wind directionality and other effects in determining nominal design loads in some simplified form (SBCCI, 1999; ASCE, 1999). This section further simplifies wind load design specifications to provide an easy yet effective approach for designing typical residential buildings.

Because they vary substantially over the surface of a building, wind loads are considered at two different scales. On a large scale, the loads produced on the overall building, or on major structural systems that sustain wind loads from more than one surface of the building, are considered the main wind force-resisting system (MWFRS). The MWFRS of a home includes the shear walls and diaphragms that create the lateral force-resisting system (LFRS) as well as the structural systems such as trusses that experience loads from two surfaces (or pressure regimes) of the building. The wind loads applied to the MWFRS account for the large-area averaging effects of time-varying wind pressures on the surface or surfaces of the building.

On a smaller scale, pressures are somewhat greater on localized surface areas of the building, particularly near abrupt changes in building geometry (e.g., eaves, ridges, and corners). These higher wind pressures occur on smaller areas, particularly affecting the loads borne by components and cladding (e.g., sheathing, windows, doors, purlins, studs). The components and cladding (C&C) transfer localized time-varying loads to the MWFRS, at which point the loads average out both spatially and temporally since, at a given time, some components may be at near peak loads while others are at substantially less than peak.

The next section presents a simplified method for determining both MWFRS and C&C wind loads. Since the loads in Section 3.6.2 are determined for specific applications, the calculation of MWFRS and C&C wind loads is implicit in the values provided. Design Example 3.2 in Section 3.10 demonstrates the calculation of wind loads by applying the simplified method of the following Section 3.6.2 to several design conditions associated with wind loads and the load combinations presented in Table 3.1.



3.6.2 Determination of Wind Loads on Residential Buildings

The following method for the design of residential buildings is based on a simplification of the ASCE 7-98 wind provisions (ASCE, 1999); therefore, the wind loads are not an exact duplicate. Lateral loads and roof uplift loads are determined by using a projected area approach. Other wind loads are determined for specific components or assemblies that comprise the exterior building envelope. Five steps are required to determine design wind loads on a residential building and its components.

Step 1: Determine site design wind speed and basic velocity pressure

From the wind map in Figure 3.2 (refer to ASCE 7-98 for maps with greater detail), select a design wind speed for the site (ASCE, 1999). The wind speed map in ASCE 7-98 (Figure 3.2) includes the most accurate data and analysis available regarding design wind speeds in the United States. The new wind speeds may appear higher than those used in older design wind maps. The difference is due solely to the use of the “peak gust” to define wind speeds rather than an averaged wind speed as represented by the “fastest mile of wind” used in older wind maps. Nominal design peak gust wind speeds are typically 85 to 90 mph in most of the United States; however, along the hurricane-prone Gulf and Atlantic coasts, nominal design wind speeds range from 100 to 150 mph for the peak gust.

If relying on either an older fastest-mile wind speed map or older design provisions based on fastest-mile wind speeds, the designer should convert wind speed in accordance with Table 3.6 for use with this simplified method, which is based on peak gust wind speeds.

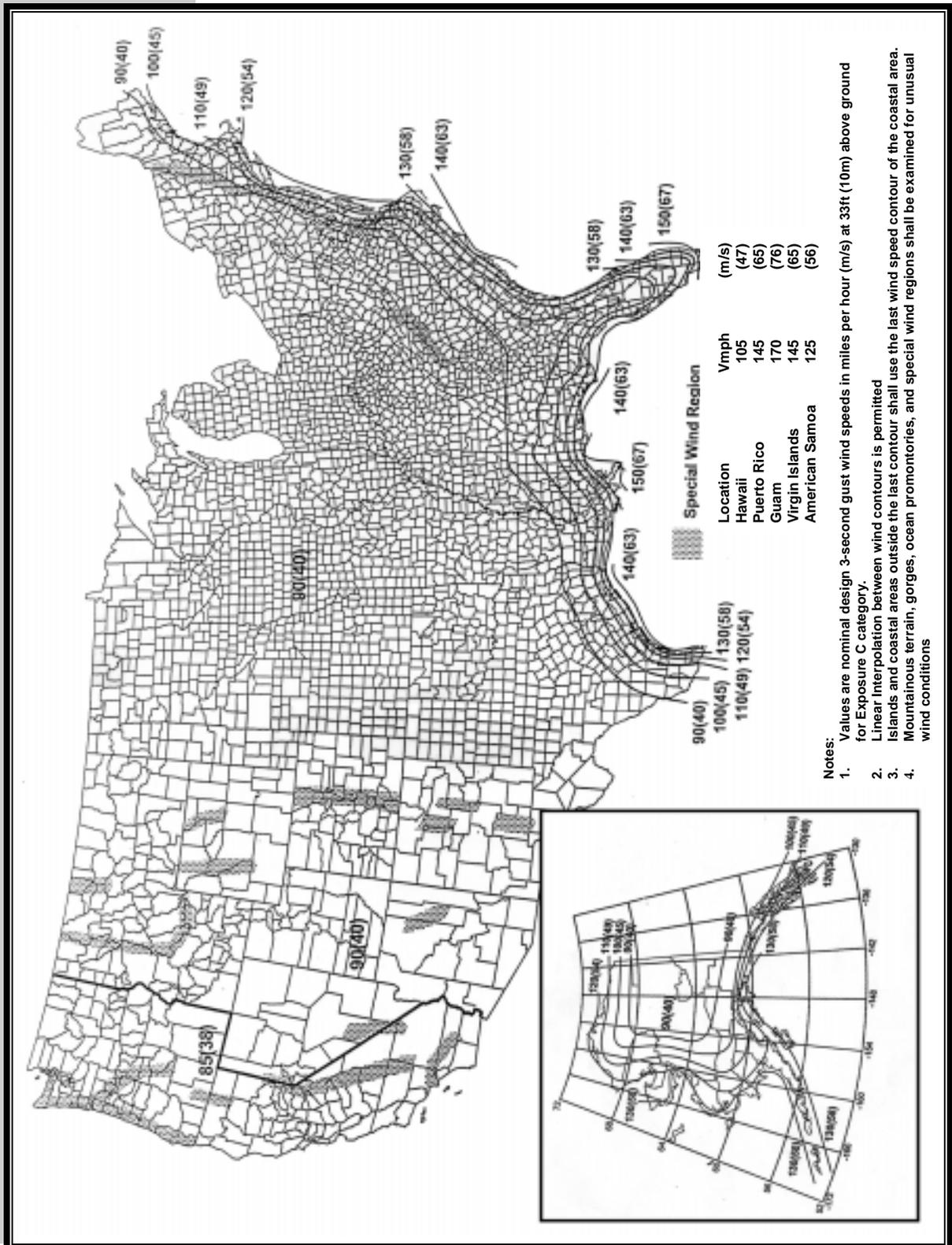
TABLE 3.6 *Wind Speed Conversions*

Fastest mile (mph)	70	75	80	90	100	110	120	130
Peak gust (mph)	85	90	100	110	120	130	140	150

Once the nominal design wind speed in terms of peak gust is determined, the designer can select the basic velocity pressure in accordance with Table 3.7. The basic velocity pressure is a reference wind pressure to which pressure coefficients are applied to determine surface pressures on a building. Velocity pressures in Table 3.7 are based on typical conditions for residential construction, namely, suburban terrain exposure and relatively flat or rolling terrain without topographic wind speed-up effects.



FIGURE 3.2 Basic Design Wind Speed Map from ASCE 7-98



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**TABLE 3.7 Basic Wind Velocity Pressures (psf) for Suburban Terrain¹**

Design Wind Speed, V (mph, peak gust)	One-Story Building ($K_z = 0.6$) ²	Two-Story Building ($K_z = 0.67$) ²	Three-Story Building ($K_z = 0.75$)
85	9.4	10.5	11.8
90	10.6	11.8	13.2
100	13.1	14.6	16.3
110	15.8	17.6	19.7
120	18.8	21.0	23.5
130	22.1	24.6	27.6
140	25.6	28.6	32.0
150	29.4	32.8	36.7

Notes:

¹Velocity pressure (psf) equals $0.00256 K_D K_z V^2$, where K_z is the velocity pressure exposure coefficient associated with the vertical wind speed profile in suburban terrain at the mean roof height of the building. K_D is the wind directionality factor with a default value of 0.85.

²These two K_z factors are adjusted from that in ASCE 7 based on a recent study of the near-ground wind profile (NAHBRC, 1999). To be compliant with ASCE 7-98, a minimum K_z of 0.7 should be applied to determine velocity pressure for one- and two-story buildings in exposure B (suburban terrain) for the design of components and cladding only. For exposure C, the values are consistent with ASCE 7-98 and require no adjustment except that all tabulated values must be multiplied by 1.4 as described in Step 2.

Step 2: Adjustments to the basic velocity pressure

If appropriate, the basic velocity pressure from Step 1 should be adjusted in accordance with the factors below. The adjustments are cumulative.

Open exposure. The wind values in Table 3.7 are based on typical residential exposures to the wind. If a site is located in generally open, flat terrain with few obstructions to the wind in most directions or is exposed to a large body of water (i.e., ocean or lake), the designer should multiply the values in Table 3.7 by a factor of 1.4. The factor may be adjusted for sites that are considered intermediate to open suburban exposures. It may also be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. The wind exposure conditions used in this guide are derived from ASCE 7-98 with some modification applicable to small residential buildings of three stories or less.

- Open terrain. Open areas with widely scattered obstructions, including shoreline exposures along coastal and noncoastal bodies of water.
- Suburban terrain. Suburban areas or other terrain with closely spaced obstructions that are the size of single-family dwellings or larger and extend in the upwind direction a distance no less than ten times the height of the building.

Protected exposure. If a site is generally surrounded by forest or densely wooded terrain with no open areas greater than a few hundred feet, smaller buildings such as homes experience significant wind load reductions from the typical suburban exposure condition assumed in Table 3.7. If such conditions exist and the site's design wind speed does not exceed about 120 mph peak gust, the designer may consider multiplying the values in Table 3.7 by 0.8. The factor may be used to adjust wind loads according to the exposure related to the specific directions of wind approach to the building. Wind load reductions associated with



a protected exposure in a suburban or otherwise open exposure have been shown to approximate 20 percent (Ho, 1992). In densely treed terrain with the height of the building below that of the tree tops, the reduction factor applied to Table 3.7 values can approach 0.6. The effect is known as shielding; however, it is not currently permitted by ASCE 7-98. Two considerations require judgment: Are the sources of shielding likely to exist for the expected life of the structure? Are the sources of shielding able to withstand wind speeds in excess of a design event?

Wind directionality. As noted, the direction of the wind in a given event does not create peak loads (which provide the basis for design pressure coefficients) simultaneously on all building surfaces. In some cases, the pressure zones with the highest design pressures are extremely sensitive to wind direction. In accordance with ASCE 7-98, the velocity pressures in Table 3.7 are based on a directionality adjustment of 0.85 that applies to hurricane wind conditions where winds in a given event are multidirectional but with varying magnitude. However, in “straight” wind climates, a directionality factor of 0.75 has been shown to be appropriate (Ho, 1992). Therefore, if a site is in a nonhurricane-prone wind area (i.e., design wind speed of 110 mph gust or less), the designer may also consider multiplying the values in Table 3.7 by 0.9 (i.e., $0.9 \times 0.85 \cong 0.75$) to adjust for directionality effects in nonhurricane-prone wind environments. ASCE 7-98 currently does not recognize this additional adjustment to account for wind directionality in “straight” wind environments.

Topographic effects. If topographic wind speed-up effects are likely because a structure is located near the crest of a protruding hill or cliff, the designer should consider using the topographic factor provided in ASCE 7-98. Wind loads can be easily doubled for buildings sited in particularly vulnerable locations relative to topographic features that cause localized wind speed-up for specific wind directions (ASCE, 1999).

Step 3: Determine lateral wind pressure coefficients

Lateral pressure coefficients in Table 3.8 are composite pressure coefficients that combine the effect of positive pressures on the windward face of the building and negative (suction) pressures on the leeward faces of the building. When multiplied by the velocity pressure from Steps 1 and 2, the selected pressure coefficient provides a single wind pressure that is applied to the vertical projected area of the roof and wall as indicated in Table 3.8. The resulting load is then used to design the home’s lateral force-resisting system (see Chapter 6). The lateral wind load must be determined for the two orthogonal directions on the building (i.e., parallel to the ridge and perpendicular to the ridge), using the vertical projected area of the building for each direction. Lateral loads are then assigned to various systems (e.g., shear walls, floor diaphragms, and roof diaphragms) by use of tributary areas or other methods described in Chapter 6.

**TABLE 3.8*****Lateral Pressure Coefficients for Application to Vertical Projected Areas***

Application	Lateral Pressure Coefficients
Roof Vertical Projected Area (by slope)	
Flat	0.0
3:12	0.3
6:12	0.5
≥9:12	0.8
Wall Projected Area	1.2

Step 4: Determine wind pressure coefficients for components and assemblies

The pressure coefficients in Table 3.9 are derived from ASCE 7-98 based on the assumption that the building is enclosed and not subject to higher internal pressures that may result from a windward opening in the building. The use of the values in Table 3.9 greatly simplifies the more detailed methodology described in ASCE 7-98; as a result, there is some “rounding” of numbers. With the exception of the roof uplift coefficient, all pressures calculated with the coefficients are intended to be applied to the perpendicular building surface area that is tributary to the element of concern. Thus, the wind load is applied perpendicular to the actual building surface, not to a projected area. The roof uplift pressure coefficient is used to determine a single wind pressure that may be applied to a horizontal projected area of the roof to determine roof tie-down connection forces.

For buildings in hurricane-prone regions subject to wind-borne debris, the GC_p values in Table 3.9 are required to be increased in magnitude by ± 0.35 to account for higher potential internal pressures due to the possibility of a windward wall opening (i.e., broken window). The adjustment is not required by ASCE 7-98 in “wind-borne debris regions” if glazing is protected against likely sources of debris impact as shown by an “approved” test method; refer to Section 3.6.3.

Step 5: Determine design wind pressures

Once the basic velocity pressure is determined in Step 1 and adjusted in Step 2 for exposure and other site-specific considerations, the designer can calculate the design wind pressures by multiplying the adjusted basic velocity pressure by the pressure coefficients selected in Steps 3 and 4. The lateral pressures based on coefficients from Step 3 are applied to the tributary areas of the lateral force-resisting systems such as shear walls and diaphragms. The pressures based on coefficients from Step 4 are applied to tributary areas of members such as studs, rafters, trusses, and sheathing to determine stresses and connection forces.



TABLE 3-9

**Wind Pressure Coefficients for Systems and Components
(enclosed building)¹**

Application	Pressure Coefficients (GC_p) ²
Roof	
Trusses, roof beams, ridge and hip/valley rafters	-0.9, +0.4
Rafters and truss panel members	-1.2, +0.7
Roof sheathing	-2.2, +1.0
Skylights and glazing	-1.2, +1.0
Roof uplift ³	
- hip roof with slope between 3:12 and 6:12	-0.9
- hip roof with slope greater than 6:12	-0.8
- all other roof types and slopes	-1.0
Windward overhang ⁴	+0.8
Wall	
All framing members	-1.2, +1.1
Wall sheathing	-1.3, +1.2
Windows, doors, and glazing	-1.3, +1.2
Garage doors	-1.1, +1.0
Air-permeable claddings ⁵	-0.9, 0.8

Notes:

¹All coefficients include internal pressure in accordance with the assumption of an enclosed building. With the exception of the categories labeled trusses, roof beams, ridge and hip/valley rafters, and roof uplift, which are based on MWFRS loads, all coefficients are based on component with cladding wind loads.

²Positive and negative signs represent pressures acting inwardly and outwardly, respectively, from the building surface. A negative pressure is a suction or vacuum. Both pressure conditions should be considered to determine the controlling design criteria.

³The roof uplift pressure coefficient is used to determine uplift pressures that are applied to the horizontal projected area of the roof for the purpose of determining uplift tie-down forces. Additional uplift force on roof tie-downs due to roof overhangs should also be included. The uplift force must be transferred to the foundation or to a point where it is adequately resisted by the dead load of the building and the capacity of conventional framing connections.

⁴The windward overhang pressure coefficient is applied to the underside of a windward roof overhang and acts upwardly on the bottom surface of the roof overhang. If the bottom surface of the roof overhang is the roof sheathing or the soffit is not covered with a structural material on its underside, then the overhang pressure shall be considered additive to the roof sheathing pressure.

⁵Air-permeable claddings allow for pressure relief such that the cladding experiences about two-thirds of the pressure differential experienced across the wall assembly (FPL, 1999). Products that experience reduced pressure include lap-type sidings such as wood, vinyl, aluminum, and other similar sidings. Since these components are usually considered “nonessential,” it may be practical to multiply the calculated wind load on any nonstructural cladding by 0.75 to adjust for a serviceability wind load (Galambos and Ellingwood, 1986). Such an adjustment would also be applicable to deflection checks, if required, for other components listed in the table. However, a serviceability load criterion is not included or clearly defined in existing design codes.

3.6.3 Special Considerations in Hurricane-Prone Environments

3.6.3.1 Wind-Borne Debris

The wind loads determined in the previous section assume an enclosed building. If glazing in windows and doors is not protected from wind-borne debris or otherwise designed to resist potential impacts during a major hurricane, a building is more susceptible to structural damage owing to higher internal building pressures that may develop with a windward opening. The potential for water damage to building contents also increases. Openings formed in the building envelope during a major hurricane or tornado are often related to unprotected glazing, improperly fastened sheathing, or weak garage doors and their attachment to the building. Section 3.9 briefly discusses tornado design conditions.



Recent years have focused much attention on wind-borne debris but with comparatively little scientific direction and poorly defined goals with respect to safety (i.e., acceptable risk), property protection, missile types, and reasonable impact criteria. Conventional practice in residential construction has called for simple plywood window coverings with attachments to resist the design wind loads. In some cases, homeowners elect to use impact-resistant glazing or shutters. Regardless of the chosen method and its cost, the responsibility for protection against wind-borne debris has traditionally rested with the homeowner. However, wind-borne debris protection has recently been mandated in some local building codes.

Just what defines impact resistance and the level of impact risk during a hurricane has been the subject of much debate. Surveys of damage following major hurricanes have identified several factors that affect the level of debris impact risk, including

- wind climate (design wind speed);
- exposure (e.g., suburban, wooded, height of surrounding buildings);
- development density (i.e., distance between buildings);
- construction characteristics (e.g., type of roofing, degree of wind resistance); and
- debris sources (e.g., roofing, fencing, gravel, etc.).

Current standards for selecting impact criteria for wind-borne debris protection do not explicitly consider all of the above factors. Further, the primary debris source in typical residential developments is asphalt roof shingles, which are not represented in existing impact test methods. These factors can have a dramatic effect on the level of wind-borne debris risk; moreover, existing impact test criteria appear to take a worst-case approach. Table 3.10 presents an example of missile types used for current impact tests. Additional factors to consider include emergency egress or access in the event of fire when impact-resistant glazing or fixed shutter systems are specified, potential injury or misapplication during installation of temporary methods of window protection, and durability of protective devices and connection details (including installation quality) such that they themselves do not become a debris hazard over time.

TABLE 3.10 *Missile Types for Wind-Borne Debris Impact Tests*^{1,2}

Description	Velocity	Energy
2-gram steel balls	130 fps	10 ft-lb
4.5-lb 2x4	40 fps	100 ft-lb
9.0-lb 2x4	50 fps	350 ft-lb

Notes:

¹Consult ASTM E 1886 (ASTM, 1997) or SSTD 12-97 (SBCCI, 1997) for guidance on testing apparatus and methodology.

²These missile types are not necessarily representative of the predominant types or sources of debris at any particular site. Steel balls are intended to represent small gravels that would be commonly used for roof ballast. The 2x4 missiles are intended to represent a direct, end-on blow from construction debris without consideration of the probability of such an impact over the life of a particular structure.



In view of the above discussion, ASCE 7-98 identifies “wind-borne debris regions” as areas within hurricane-prone regions that are located (1) within one mile of the coastal mean high water line where the basic wind speed is equal to or greater than 110 mph or in Hawaii or (2) where the basic wind speed is equal to or greater than 120 mph. As described in Section 3.6.2, ASCE 7-98 requires higher internal pressures to be considered for buildings in wind-borne debris regions unless glazed openings are protected by impact-resistant glazing or protective devices proven as such by an approved test method. Approved test methods include ASTM E1886 and SSTD 12-97 (ASTM, 1997; SBCCI, 1997).

The wind load method described in Section 3.6.2 may be considered acceptable without wind-borne debris protection, provided that the building envelope (i.e., windows, doors, sheathing, and especially garage doors) is carefully designed for the required pressures. Most homes that experience wind-borne debris damage do not appear to exhibit more catastrophic failures, such as a roof blow-off, unless the roof was severely underdesigned in the first place (i.e., inadequate tie-down) or subject to poor workmanship (i.e., missing fasteners at critical locations). Those cases are often the ones cited as evidence of internal pressure in anecdotal field studies. However, garage doors that fail due to wind pressure more frequently precipitate additional damage related to internal pressure. Therefore, in hurricane-prone regions, garage door reinforcement or pressure-rated garage doors should be specified and their attachment to structural framing carefully considered.

3.6.3.2 Building Durability

Roof overhangs increase uplift loads on roof tie-downs and the framing members that support the overhangs. They do, however, provide a reliable means of protection against moisture and the potential decay of wood building materials. The designer should therefore consider the trade-off between wind load and durability, particularly in the moist, humid climate zones associated with hurricanes.

For buildings that are exposed to salt spray or mist from nearby bodies of salt water, the designer should also consider a higher-than-standard level of corrosion resistance for exposed fasteners and hardware. Truss plates near roof vents have also shown accelerated rates of corrosion in severe coastal exposures. The building owner, in turn, should consider a building maintenance plan that includes regular inspections, maintenance, and repair.

3.6.3.3 Tips to Improve Performance

The following design and construction tips are simple options for reducing a building's vulnerability to hurricane damage:

- One-story buildings are much less vulnerable to wind damage than two- or three-story buildings.
- On average, hip roofs have demonstrated better performance than gable-end roofs.



- Moderate roof slopes (i.e., 4:12 to 6:12) tend to optimize the trade-off between lateral loads and roof uplift loads (i.e., more aerodynamically efficient).
- Roof sheathing installation should be inspected for the proper type and spacing of fasteners, particularly at connections to gable-end framing.
- The installation of metal strapping or other tie-down hardware should be inspected as required to ensure the transfer of uplift loads.
- If composition roof shingles are used, high-wind fastening requirements should be followed (i.e., 6 nails per shingle in lieu of the standard 4 nails). A similar concern exists for tile roofing, metal roofing, and other roofing materials.
- Consider some practical means of glazed opening protection in the most severe hurricane-prone areas.

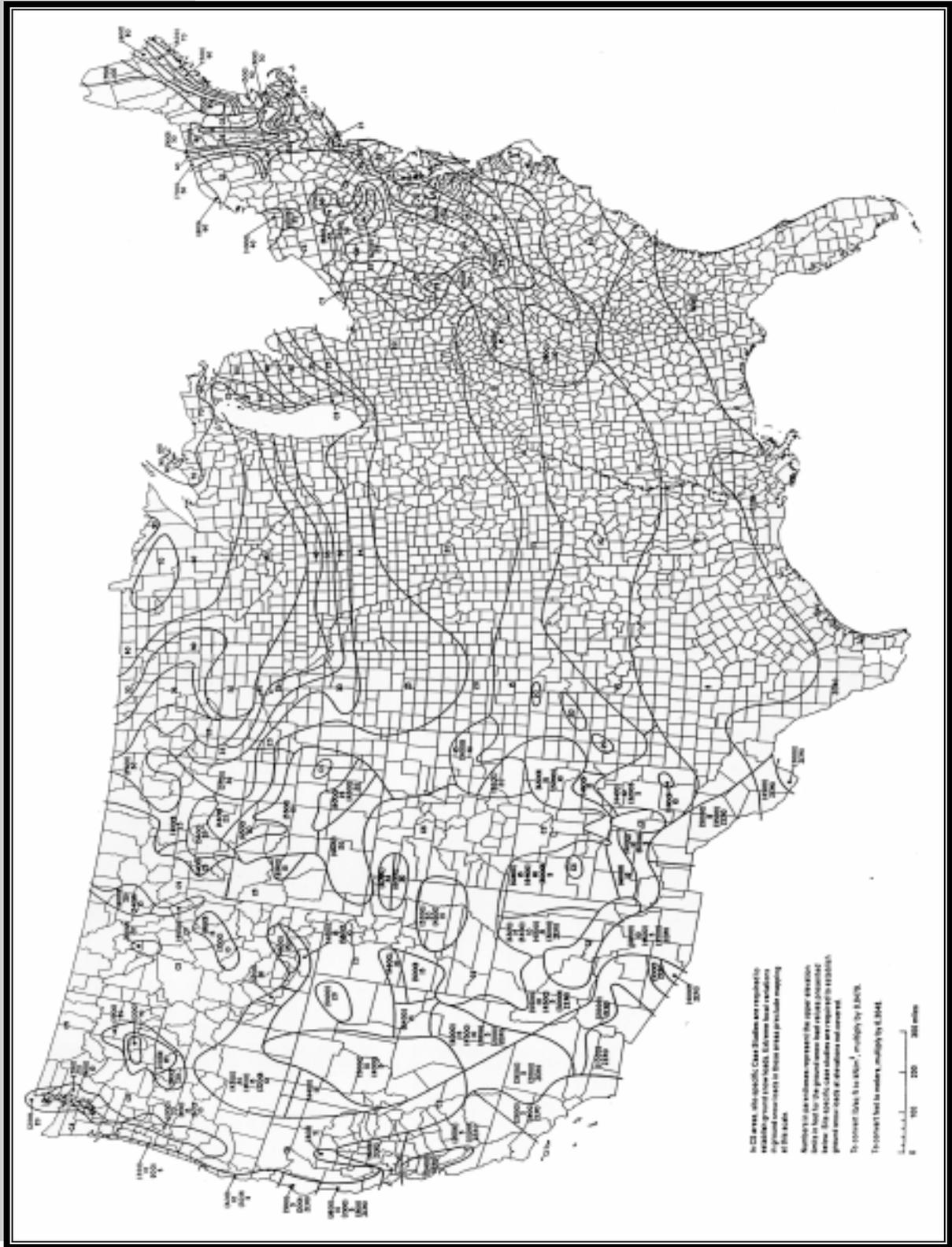
3.7 Snow Loads

For design purposes, snow is typically treated as a simple uniform gravity load on the horizontal projected area of a roof. The uniformly distributed design snow load on residential roofs can be easily determined by using the unadjusted ground snow load. This simple approach also represents standard practice in some regions of the United States; however, it does not account for a reduction in roof snow load that may be associated with steep roof slopes with slippery surfaces (refer to ASCE 7-98). To consider drift loads on sloped gable or hip roofs, the design roof snow load on the windward and leeward roof surfaces may be determined by multiplying the ground snow load by 0.8 and 1.2 respectively. In this case, the drifted side of the roof has 50 percent greater snow load than the non-drifted side of the roof. However, the average roof snow load is still equivalent to the ground snow load.

Design ground snow loads may be obtained from the map in Figure 3.3; however, snow loads are usually defined by the local building department. Typical ground snow loads range from 0 psf in the South to 50 psf in the northern United States. In mountainous areas, the ground snow load can surpass 100 psf such that local snow data should be carefully considered. In areas where the ground snow load is less than 15 psf, the minimum roof live load (refer to Section 3.4) is usually the controlling gravity load in roof design. For a larger map with greater detail, refer to ASCE 7-98.



FIGURE 3.3 *Ground Snow Loads (ASCE 7-98)*



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3.8 Earthquake Loads

3.8.1 General

This section provides a simplified earthquake load analysis procedure appropriate for use in residential light-frame construction of not more than three stories above grade. As described in Chapter 2, the lateral forces associated with seismic ground motion are based on fundamental Newtonian mechanics ($F = ma$) expressed in terms of an equivalent static load. The method provided in this section is a simplification of the most current seismic design provisions (NEHRP, 1997[a and b]). It is also similar to a simplified approach found in more recent building code development (ICC, 1999).

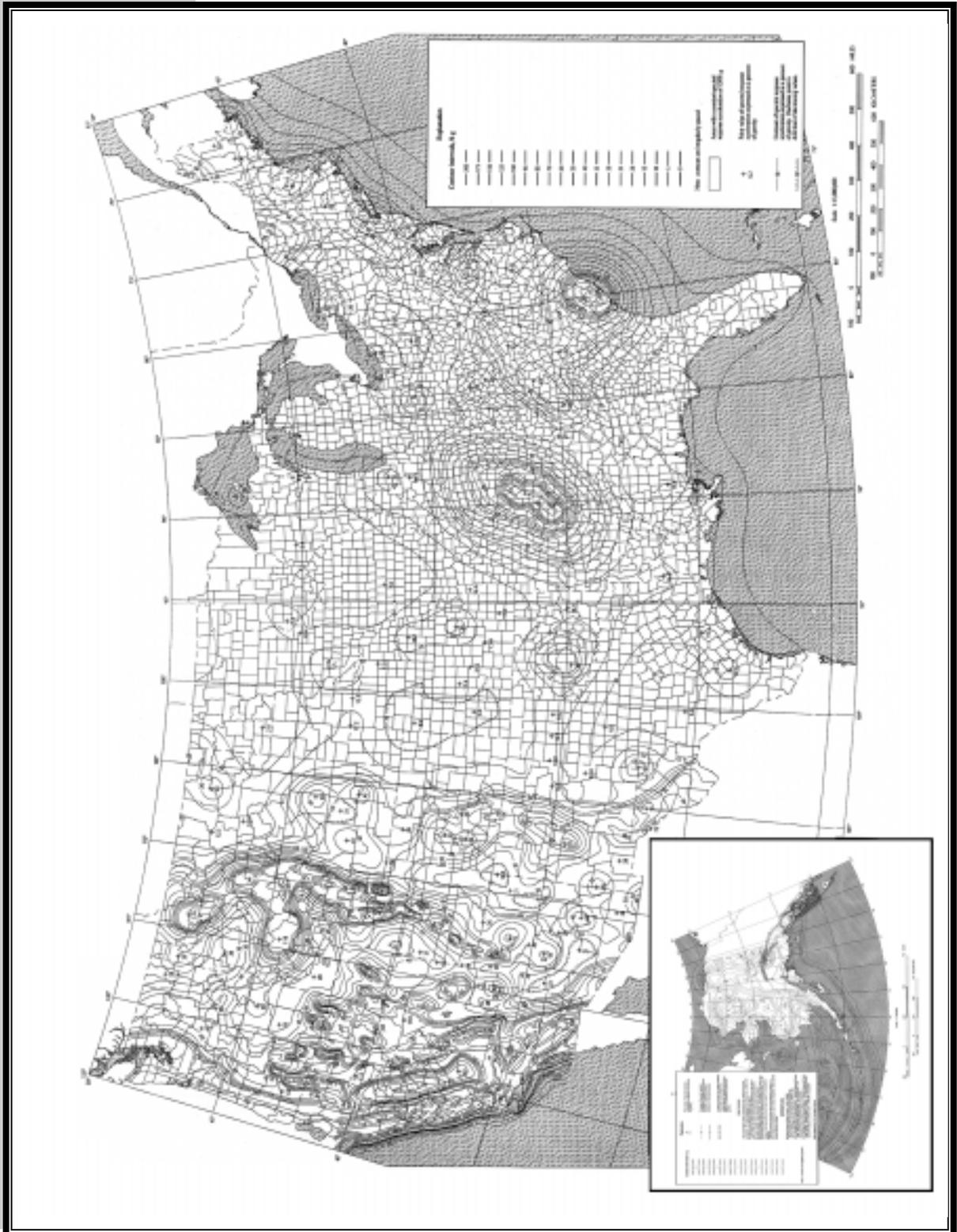
Most residential designers use a simplified approach similar to that in older seismic design codes. The approach outlined in the next section follows the older approach in terms of its simplicity while using the newer seismic risk maps and design format of NEHRP-97 as incorporated into recent building code development efforts (ICC, 1999); refer to Figure 3.4. It should be noted, however, that the newer maps are not without controversy relative to seismic risk predictions, particularly in the eastern United States. For example, the newer maps are believed to overstate significantly the risk of earthquakes in the New Madrid seismic region around St. Louis, MO (Newman et al., 1999). Based on recent research and the manner of deriving the NEHRP-97 maps for the New Madrid seismic region, the design seismic loads may be conservative by a factor of 2 or more. The designer should bear in mind these uncertainties in the design process.

Chapter 1 discussed the performance of conventional residential construction in the Northridge Earthquake. In general, wood-framed homes have performed well in major seismic events, probably because of, among many factors, their light-weight and resilient construction, the strength provided by nonstructural systems such as interior walls, and their load distribution capabilities. Only in the case of gross absence of good judgment or misapplication of design for earthquake forces have severe life-safety consequences become an issue in light-frame, low-rise structures experiencing extreme seismic events.



FIGURE 3.4

Seismic Map of Design Short-Period Spectral Response Acceleration (g) (2 percent chance of exceedance in 50 years or 2,475-year return period)



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3.8.2 Determination of Earthquake Loads on Houses

The total lateral force at the base of a building is called seismic base shear. The lateral force experienced at a particular story level is called the story shear. The story shear is greatest in the ground story and least in the top story. Seismic base shear and story shear (V) are determined in accordance with the following equation:

[Equation 3.8-1]

$$V = \frac{1.2 S_{DS}}{R} W,$$

where,

S_{DS} = the design spectral response acceleration in the short-period range determined by Equation 3.8-2 (g)

R = the response modification factor (dimensionless)

W = the total weight of the building or supported by the story under consideration (lb); 20 percent of the roof snow load is also included where the ground snow load exceeds 30 psf

1.2 = factor to increase the seismic shear load based on the belief that the simplified method may result in greater uncertainty in the estimated seismic load

When determining story shear for a given story, the designer attributes to that story one-half of the dead load of the walls on the story under consideration and the dead load supported by the story. Dead loads used in determining seismic story shear or base shear are found in Section 3.3. For housing, the interior partition wall dead load is reasonably accounted for by the use of a 6 psf load distributed uniformly over the floor area. When applicable, the snow load may be determined in accordance with Section 3.7. The inclusion of any snow load, however, is based on the assumption that the snow is always frozen solid and adhered to the building such that it is part of the building mass during the entire seismic event.

The design spectral response acceleration for short-period ground motion S_{DS} is typically used because light-frame buildings such as houses are believed to have a short period of vibration in response to seismic ground motion (i.e., high natural frequency). In fact, nondestructive tests of existing houses have confirmed the short period of vibration, although once ductile damage has begun to occur in a severe event, the natural period of the building likely increases. Chapter 1 discussed the apparent correlation between housing performance (degree of damage) and long-period (one-second) ground motion characteristics in the Northridge Earthquake (HUD, 1999). As yet, no valid methods are available to determine the natural period of vibration for use in the seismic design of light-frame houses. Therefore, the short-period ground motion is used in the interest of following traditional practice.

Values of S_s are obtained from Figure 3.7. For a larger map with greater detail, refer to ASCE 7-98. The value of S_{DS} should be determined in consideration of the mapped short-period spectral response acceleration S_s and the required soil site amplification factor F_a as follows:



[Equation 3.8-2]

$$S_{DS} = 2/3(S_s)(F_a)$$

The value of S_s ranges from practically zero in low-risk areas to 3g in the highest-risk regions of the United States. A typical value in high seismic areas is 1.5g. In general, wind loads control the design of the lateral force-resisting system of light-frame houses when S_s is less than about 1g. The 2/3 coefficient in Equation 3.8-2 is used to adjust to a design seismic ground motion value from that represented by the mapped S_s values (i.e., the mapped values are based on a “maximum considered earthquake” generally representative of a 2,475-year return period, with the design basis intended to represent a 475-year return period event).

Table 3.11 provides the values of F_a associated with a standard “firm” soil condition used for the design of residential buildings. F_a decreases with increasing ground motion because the soil begins to dampen the ground motion as shaking intensifies. Therefore, the soil can have a moderating effect on the seismic shear loads experienced by buildings in high seismic risk regions. Dampening also occurs between a building foundation and the soil and thus has a moderating effect. However, the soil-structure interaction effects on residential buildings have been the topic of little study; therefore, precise design procedures have yet to be developed. If a site is located on fill soils or “soft” ground, a different value of F_a should be considered. Nonetheless, as noted in the Anchorage Earthquake of 1964 and again 30 years later in the Northridge Earthquake (see Chapter 1), soft soils do not necessarily affect the performance of the above-ground house structure as much as they affect the site and foundations (e.g., settlement, fissuring, liquefaction, etc.).

TABLE 3.11

***Site Soil Amplification Factor Relative to Acceleration
(short period, firm soil)***

S_s	$\leq 0.25g$	0.5g	0.75g	1.0g	$\geq 1.25g$
F_a	1.6	1.4	1.2	1.1	1.0

The seismic response modifier R has a long history in seismic design, but with little in the way of scientific underpinnings. In fact, it can be traced back to expert opinion in the development of seismic design codes during the 1950s (ATC, 1995). In recognition that buildings can effectively dissipate energy from seismic ground motions through ductile damage, the R factor was conceived to adjust the shear forces from that which would be experienced if a building could exhibit perfectly elastic behavior without some form of ductile energy dissipation. The concept has served a major role in standardizing the seismic design of buildings even though it has evolved in the absence of a repeatable and generalized evaluation methodology with a known relationship to actual building performance.

Those structural building systems that are able to withstand greater ductile damage and deformation without substantial loss of strength are assigned a higher value for R . The R factor also incorporates differences in dampening that are believed to occur for various structural systems. Table 3.12 provides some values for R that are relevant to residential construction.

**TABLE 3.12** *Seismic Response Modifiers for Residential Construction*

Structural System	Seismic Response Modifier, R ¹
Light-frame shear walls with wood structural panels used as bearing walls	6.0 ²
Light-frame shear walls with wall board/lath and plaster	2.0
Reinforced concrete shear walls ³	4.5
Reinforced masonry shear walls ³	3.5
Plain concrete shear walls	1.5
Plain masonry shear walls	1.25

Notes:

¹The R-factors may vary for a given structural system type depending on wall configuration, material selection, and connection detailing, but these considerations are necessarily matters of designer judgment.

²The R for light-frame shear walls (steel-framed and wood-framed) with shear panels has been recently revised to 6 but is not yet published (ICC, 1999). Current practice typically uses an R of 5.5 to 6.5 depending on the edition of the local building code.

³The wall is reinforced in accordance with concrete design requirements in ACI-318 or ACI-530. Nominally reinforced concrete or masonry that has conventional amounts of vertical reinforcement such as one #5 rebar at openings and at 4 feet on center may use the value for reinforced walls provided the construction is no more than two stories above grade.

Design Example 3.3 in Section 3.10 demonstrates the calculation of design seismic shear load based on the simplified procedures. The reader is referred to Chapter 6 for additional information on seismic loads and analysis.

3.8.3 Seismic Shear Force Distribution

As described in the previous section, the *vertical distribution* of seismic forces to separate stories on a light-frame building is assumed to be in accordance with the mass supported by each story. However, design codes vary in the requirements related to vertical distribution of seismic shear. Unfortunately, there is apparently no clear body of evidence to confirm any particular method of vertical seismic force distribution for light-frame buildings. Therefore, in keeping with the simplified method given in Section 3.8.2, the approach used in this guide reflects what is considered conventional practice. The *horizontal distribution* of seismic forces to various shear walls on a given story also varies in current practice for light-frame buildings. In Chapter 6, several existing approaches to the design of the lateral force-resisting system of light-frame houses address the issue of horizontal force distribution with varying degrees of sophistication. Until methods of vertical and horizontal seismic force distribution are better understood for application to light-frame buildings, the importance of designer judgment cannot be overemphasized.

3.8.4 Special Seismic Design Considerations

Perhaps the single most important principle in seismic design is to ensure that the structural components and systems are adequately tied together to perform as a structural unit. Underlying this principle are a host of analytic challenges and uncertainties in actually defining what “adequately tied together” means in a repeatable, accurate, and theoretically sound manner.

Recent seismic building code developments have introduced several new factors and provisions that attempt to address various problems or uncertainties in the design process. Unfortunately, these factors appear to introduce as many



uncertainties as they address. Codes have tended to become more complicated to apply or decipher, perhaps detracting from some important basic principles in seismic design that, when understood, would provide guidance in the application of designer judgment. Many of the problems stem from the use of the seismic response modifier R which is a concept first introduced to seismic design codes in the 1950s (see discussion in previous section). Some of the issues and concerns are briefly described below based on a recent critique of seismic design approaches and other sources (ATC, 1995; NEHRP 1997a and b; ICBO, 1997).

Also known as “reserve strength,” the concept of *overstrength* is a realization that a shear resisting system’s ultimate capacity is usually significantly higher than required by a design load as a result of intended safety margins. At the same time, the seismic ground motion (load) is reduced by the R factor to account for ductile response of the building system, among other things. Thus, the actual forces experienced on various components (i.e. connections) during a design level event can be substantially higher, even though the resisting system may be able to effectively dissipate that force. Therefore, overstrength factors have been included in newer seismic codes with recommendations to assist in designing components that may experience higher forces than determined otherwise for the building lateral force resisting system using methods similar to Equation 3.8-1. It should be noted that current overstrength factors should not be considered exact and that actual values of overstrength can vary substantially.

In essence, the overstrength concept is an attempt to address the principle of balanced design. It strives to ensure that critical components, such as connections, have sufficient capacity so that the overall lateral force-resisting system is able to act in its intended ductile manner (i.e., absorbing higher-than-design forces). Thus, a premature failure of a critical component (i.e., a restraining connection failure) is avoided. An exact approach requires near-perfect knowledge about various connections, details, safety margins, and system-component response characteristics that are generally not available. However, the concept is extremely important and, for the most part, experienced designers have exercised this principle through a blend of judgment and rational analysis.

The concept of overstrength is addressed in Chapter 6 relative to the design of restraining connections for light-frame buildings by providing the designer with ultimate capacity values for light-frame shear wall systems. Thus, the designer is able to compare the unfactored shear wall capacity to that of hold-down restraints and other connections to ensure that the ultimate connection capacity is at least as much as that of the shear wall system. Some consideration of the ductility of the connection or component may also imply a response modification factor for a particular connection or framing detail. In summary, overstrength is an area where exact guidance does not exist and the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

The *redundancy factor* was postulated to address the reliability of lateral force-resisting systems by encouraging multiple lines of shear resistance in a building (ATC, 1995). It is now included in some of the latest seismic design provisions (NEHRP, 1997). Since it appears that redundancy factors have little technical basis and insufficient verification relative to light-frame structures (ATC, 1995), they are not explicitly addressed in this guide. In fact, residential buildings are generally recognized for their inherent redundancies that are



systematically overlooked when designating and defining a lateral force resisting system for the purpose of executing a rational design. However, the principle is important to consider. For example, it would not be wise to rely on one or two shear-resisting components to support a building. In typical applications of light-frame construction, even a single shear wall line has several individual segments and numerous connections that resist shear forces. At a minimum, there are two such shear wall lines in either orientation of the building, not to mention interior walls and other nonstructural elements that contribute to the redundancy of typical light-frame homes. In summary, redundancy is an area where exact guidance does not exist and the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

Deflection amplification has been applied in past and current seismic design codes to adjust the deflection or story drift determined by use of the design seismic shear load (as adjusted downward by the R factor) relative to that actually experienced without allowance for modified response (i.e., load not adjusted down by the R factor). For wood-framed shear wall construction, the deflection calculated at the nominal seismic shear load (Equation 3.8-1) is multiplied by a factor of 4 (NEHRP, 1997). Thus, the estimate of deflection or drift of the shear wall (or entire story) based on the design seismic shear load would be increased four-fold. Again, the conditions that lead to this level of deflection amplification and the factors that may affect it in a particular design are not exact (and are not obvious to the designer). As a result, conservative drift amplification values are usually selected for code purposes. Regardless, deflection or drift calculations are rarely applied in a residential (low-rise) wood-framed building design for three reasons. First, a methodology is not generally available to predict the drift behavior of light-frame buildings reliably and accurately. Second, the current design values used for shear wall design are relatively conservative and are usually assumed to provide adequate stiffness (i.e., limit drift). Third, code-required drift limits have not been developed for specific application to light-frame residential construction. Measures to estimate drift, however, are discussed in Chapter 6 in terms of nonlinear approximations of wood-frame shear wall load-drift behavior (up to ultimate capacity). In summary, deformation amplification is an area where exact guidance does not exist and predictive tools are unreliable. Therefore, the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

Another issue that has received greater attention in seismic design provisions is *irregularities*. Irregularities are related to special geometric or structural conditions that affect the seismic performance of a building and either require special design attention or should be altogether avoided. In essence, the presence of limits on structural irregularity speaks indirectly of the inability to predict the performance of a structure in a reliable, self-limiting fashion on the basis of analysis alone. Therefore, many of the irregularity limitations are based on judgment from problems experienced in past seismic events.

Irregularities are generally separated into plan and vertical structural irregularities. Plan structural irregularities include torsional imbalances that result in excessive rotation of the building, re-entrant corners creating “wings” of a building, floor or roof diaphragms with large openings or nonuniform stiffness, out-of-plane offsets in the lateral force resistance path, and nonparallel resisting systems. Vertical structural irregularities include stiffness irregularities (i.e., a



“soft” story), capacity irregularities (i.e., a “weak” story), weight (mass) irregularity (i.e., a “heavy” story), and geometric discontinuities affecting the interaction of lateral resisting systems on adjacent stories.

The concept of irregularities is associated with ensuring an adequate load path and limiting undesirable (i.e., hard to control or predict) building responses in a seismic event. Again, experienced designers generally understand the effect of irregularities and effectively address or avoid them on a case-by-case basis. For typical single-family housing, all but the most serious irregularities (i.e., “soft story”) are generally of limited consequence, particularly given the apparently significant system behavior of light-frame homes (provided the structure is reasonably “tied together as a structural unit”). For larger structures, such as low- and high-rise commercial and residential construction, the issue of irregularity—and loads—becomes more significant. Because structural irregularities raise serious concerns and have been associated with building failures or performance problems in past seismic events, the designer must exercise reasonable care in accordance with or in addition to the applicable building code requirements.

A key issue related to building damage involves *deformation compatibility* of materials and detailing in a constructed system. This issue may be handled through specification of materials that have similar deformation capabilities or by system detailing that improves compatibility. For example, a relatively flexible hold-down device installed near a rigid sill anchor causes greater stress concentration on the more rigid element as evidenced by the splitting of wood sill plates in the Northridge Earthquake. The solution can involve increasing the rigidity of the hold-down device (which can lessen the ductility of the system, increase stiffness, and effectively increase seismic load) or redesigning the sill plate connection to accommodate the hold-down deformation and improve load distribution. As a nonstructural example of deformation compatibility, gypsum board interior finishes crack in a major seismic event well before the structural capability of the wall’s structural sheathing is exhausted. Conversely, wood exterior siding and similar resilient finishes tend to deform compatibly with the wall and limit observable or unacceptable visual damage (HUD, 1994). A gypsum board interior finish may be made more resilient and compatible with structural deformations by using resilient metal channels or similar detailing; however, this enhancement has not yet been proven. Unfortunately, there is little definitive design guidance on deformation compatibility considerations in seismic design of wood-framed buildings and other structures.

As a final issue, it should be understood that the general objective of current and past seismic building code provisions has been to prevent collapse in extreme seismic events such that “protection of life is reasonably provided, but not with complete assurance” as stated in the 1990 *Blue Book* (SEAOC, 1990). It is often believed that damage can be controlled by use of a smaller R factor or, for a similar effect, a larger safety factor. Others have suggested using a higher design event. While either approach may indirectly reduce damage or improve performance, it does not necessarily improve the predictability of building performance and, therefore, may have uncertain benefits, if any, in many cases. However, some practical considerations as discussed above may lead to better-performing buildings, at least from the perspective of controlling damage.



3.9 Other Load Conditions

In addition to the loads covered in Sections 3.3 through 3.8 that are typically considered in the design of a home, other “forces of nature” may create loads on buildings. Some examples include

- frost heave;
- expansive soils;
- temperature effects; and
- tornadoes.

In certain cases, forces from these phenomena can drastically exceed reasonable design loads for homes. For example, frost heave forces can easily exceed 10,000 pounds per square foot (Linell and Lobacz, 1980). Similarly, the force of expanding clay soil can be impressive. In addition, the self-straining stresses induced by temperature-related expansion or contraction of a member or system that is restrained against movement can be very large, although they are not typically a concern in wood-framed housing. Finally, the probability of a direct tornado strike on a given building is much lower than considered practical for engineering and general safety purposes. The unique wind loads produced by an extreme tornado (i.e., F5 on the Fujita scale) may exceed typical design wind loads by almost an order of magnitude in effect. Conversely, most tornadoes have comparatively low wind speeds that can be resisted by attainable design improvements. However, the risk of such an event is still significantly lower than required by minimum accepted safety requirements.

It is common practice to avoid the above loads by using sound design detailing. For example, frost heave can be avoided by placing footings below a “safe” frost depth, building on nonfrost-susceptible materials, or using other frost protection methods (see Chapter 4). Expansive soil loads can be avoided by isolating building foundations from expansive soil, supporting foundations on a system of deep pilings, and designing foundations that provide for differential ground movements. Temperature effects can be eliminated by providing construction joints that allow for expansion and contraction. While such temperature effects on wood materials are practically negligible, some finishes such as ceramic tile can experience cracking when inadvertently restrained against small movements resulting from variations in temperature. Unfortunately, tornadoes cannot be avoided; therefore, it is not uncommon to consider the additional cost and protection of a tornado shelter in tornado-prone areas. A tornado shelter guide is available from the Federal Emergency Management Agency, Washington, DC.

As noted at the beginning of the chapter, this guide does not address loads from flooding, ice, rain, and other exceptional sources. The reader is referred to ASCE 7 and other resources for information regarding special load conditions (ASCE, 1999).



3.10 Design Examples

EXAMPLE 3.1

Design Gravity Load Calculations and Use of ASD Load Combinations



Given

- Three-story conventional wood-framed home
- 28' x 44' plan, clear-span roof, floors supported at mid-span
- Roof dead load = 15 psf (Table 3.2)
- Wall dead load = 8 psf (Table 3.2)
- Floor dead load = 10 psf (Table 3.2)
- Roof snow load = 16 psf (Section 3.7)
- Attic live load = 10 psf (Table 3.4)
- Second- and third-floor live load = 30 psf (Table 3.4)
- First-floor live load = 40 psf (Table 3.4)

Find

1. Gravity load on first-story exterior bearing wall
2. Gravity load on a column supporting loads from two floors

Solution

1.

Gravity load on first-story exterior bearing wall

- Determine loads on wall

$$\begin{aligned} \text{Dead load} &= \text{roof DL} + 2 \text{ wall DL} + 2 \text{ floor DL} \\ &= 1/2 (28 \text{ ft})(15 \text{ psf}) + 2(8 \text{ ft})(8 \text{ psf}) + 2(7 \text{ ft})(10 \text{ psf}) \\ &= 478 \text{ plf} \end{aligned}$$

$$\text{Roof snow} = 1/2(28 \text{ ft})(16 \text{ psf}) = 224 \text{ plf}$$

$$\begin{aligned} \text{Live load} &= (30 \text{ psf} + 30 \text{ psf})(7 \text{ ft}) = 420 \text{ plf} \\ &\text{(two floors)} \end{aligned}$$

$$\begin{aligned} \text{Attic live load} &= (10 \text{ psf})(14 \text{ ft} - 5 \text{ ft}^*) = 90 \text{ plf} \\ &\text{*edges of roof span not accessible to roof storage due to} \\ &\text{low clearance} \end{aligned}$$

- Apply applicable ASD load combinations (Table 3.1)

$$(a) \quad D + L + 0.3 (L_r \text{ or } S)$$

$$\begin{aligned} \text{Wall axial gravity load} &= 478 \text{ plf} + 420 \text{ plf} + 0.3 (224 \text{ plf}) \\ &= 965 \text{ plf}^* \end{aligned}$$

*equals 1,055 plf if full attic live load allowance is included with L

$$(b) \quad D + (L_r \text{ or } S) + 0.3L$$

$$\begin{aligned} \text{Wall axial gravity load} &= 478 \text{ plf} + 224 \text{ plf} + 0.3 (420 \text{ plf}) \\ &= 828 \text{ plf} \end{aligned}$$

Load condition (a) controls the gravity load analysis for the bearing wall. The same load applies to the design of headers as well as to the wall studs. Of course, combined lateral (bending) and axial loads on the wall studs also need to be checked (i.e., D+W); refer to Table 3.1 and Example 3.2. For nonload-bearing exterior walls (i.e., gable-end curtain walls), contributions from floor and roof live loads may be negligible (or significantly reduced), and the D+W load combination likely governs the design.

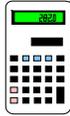


2. Gravity load on a column supporting a center floor girder carrying loads from two floors (first and second stories)
- Assume a column spacing of 16 ft
 - Determine loads on column
- (a) Dead load = Second floor + first floor + bearing wall supporting second floor
- $$\begin{aligned} &= (14 \text{ ft})(16 \text{ ft})(10 \text{ psf}) + (14 \text{ ft})(16 \text{ ft})(10 \text{ psf}) + (8 \text{ ft})(16 \text{ ft})(7 \text{ psf}) \\ &= 5,376 \text{ lbs} \end{aligned}$$
- (b) Live load area reduction (Equation 3.4-1)
- supported floor area = $2(14 \text{ ft})(16 \text{ ft}) = 448 \text{ ft}^2$ per floor
 - reduction = $\left[0.25 + \frac{10.6}{\sqrt{448}} \right] = 0.75 \geq 0.75$ OK
 - first-floor live load = $0.75 (40 \text{ psf}) = 30 \text{ psf}$
 - second-floor live load = $0.75 (30 \text{ psf}) = 22.5 \text{ psf}$
- (c) Live load = $(14 \text{ ft})(16 \text{ ft})[30 \text{ psf} + 22.5 \text{ psf}]$
= 11,760 lbs

- Apply ASD load combinations (Table 3.1)

The controlling load combination is D+L since there are no attic or roof loads supported by the column. The total axial gravity design load on the column is 17,136 lbs (5,376 lbs + 11,760 lbs).

Note. If LRFD material design specifications are used, the various loads would be factored in accordance with Table 3.1. All other considerations and calculations remain unchanged.

**EXAMPLE 3.2****Design Wind Load Calculations and Use of ASD Load Combinations****Given**

- Site wind speed—100 mph, gust
- Site wind exposure—suburban
- Two-story home, 7:12 roof pitch, 28' x 44' plan (rectangular), gable roof, 12-inch overhang

Find

1. Lateral (shear) load on lower-story end wall
2. Net roof uplift at connections to the side wall
3. Roof sheathing pull-off (suction) pressure
4. Wind load on a roof truss
5. Wind load on a rafter
6. Lateral (out-of-plane) wind load on a wall stud

Solution

1. Lateral (shear) load on lower-story end wall

Step 1: Velocity pressure = 14.6 psf (Table 3.7)

Step 2: Adjusted velocity pressure = $0.9 * 14.6 \text{ psf} = 13.1 \text{ psf}$

*adjustment for wind directionality ($V < 110 \text{ mph}$)

Step 3: Lateral roof coefficient = 0.6 (Table 3.8)

Lateral wall coefficient = 1.2 (Table 3.8)

Step 4: Skip

Step 5: Determine design wind pressures

Wall projected area pressure = $(13.1 \text{ psf})(1.2) = 15.7 \text{ psf}$

Roof projected area pressure = $(13.1 \text{ psf})(0.6) = 7.9 \text{ psf}$

Now determine vertical projected areas (VPA) for lower-story end-wall tributary loading (assuming no contribution from interior walls in resisting lateral loads)

$$\begin{aligned} \text{Roof VPA} &= [1/2 (\text{building width})(\text{roof pitch})] \times [1/2 (\text{building length})] \\ &= [1/2 (28 \text{ ft})(7/12)] \times [1/2 (44 \text{ ft})] \\ &= [8.2 \text{ ft}] \times [22 \text{ ft}] \\ &= 180 \text{ ft}^2 \end{aligned}$$

$$\begin{aligned} \text{Wall VPA} &= [(\text{second-story wall height}) + (\text{thickness of floor}) + 1/2 (\text{first-story wall height})] \times [1/2 (\text{building length})] \\ &= [8 \text{ ft} + 1 \text{ ft} + 4 \text{ ft}] \times [1/2 (44 \text{ ft})] \\ &= [13 \text{ ft}] \times [22 \text{ ft}] \\ &= 286 \text{ ft}^2 \end{aligned}$$

Now determine shear load on the first-story end wall

$$\begin{aligned} \text{Shear} &= (\text{roof VPA})(\text{roof projected area pressure}) + (\text{wall VPA})(\text{wall projected area pressure}) \\ &= (180 \text{ ft}^2)(7.9 \text{ psf}) + (286 \text{ ft}^2)(15.7 \text{ psf}) \\ &= 5,912 \text{ lbs} \end{aligned}$$

The first-story end wall must be designed to transfer a shear load of 5,169 lbs. If side-wall loads were determined instead, the vertical projected area would include only the gable-end wall area and the triangular wall area formed by the roof. Use of a hip roof would reduce the shear load for the side and end walls.



2. Roof uplift at connection to the side wall (parallel-to-ridge)

- Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof uplift pressure coefficient = -1.0 (Table 3.9)
Roof overhang pressure coefficient = 0.8 (Table 3.9)
Step 5: Determine design wind pressure
Roof horizontal projected area (HPA) pressure = -1.0 (13.1 psf)
= -13.1 psf
Roof overhang pressure = 0.8 (13.1 psf) = 10.5 psf (upward)

Now determine gross uplift at roof-wall reaction

$$\begin{aligned}\text{Gross uplift} &= 1/2 (\text{roof span})(\text{roof HPA pressure}) + (\text{overhang})(\text{overhang pressure coefficient}) \\ &= 1/2 (30 \text{ ft})(-13.1 \text{ psf}) + (1 \text{ ft})(-10.5 \text{ psf}) \\ &= -207 \text{ plf (upward)}\end{aligned}$$

$$\begin{aligned}\text{Roof dead load reaction} &= 1/2 (\text{roof span})(\text{uniform dead load}) \\ &= 1/2 (30 \text{ ft})(15 \text{ psf}^*) \\ &\quad \text{*Table 3.2} \\ &= 225 \text{ plf (downward)}\end{aligned}$$

Now determine net design uplift load at roof-wall connection

$$\begin{aligned}\text{Net design uplift load} &= 0.6D + W_u \quad (\text{Table 3.1}) \\ &= 0.6 (225 \text{ plf}) + (-207 \text{ plf}) \\ &= -54 \text{ plf (net uplift)}\end{aligned}$$

The roof-wall connection must be capable of resisting a design uplift load of 54 plf. Generally, a toenail connection can be shown to meet the design requirement depending on the nail type, nail size, number of nails, and density of wall framing lumber (see Chapter 7). At appreciably higher design wind speeds or in more open wind exposure conditions, roof tie-down straps, brackets, or other connectors should be considered and may be required.

3. Roof sheathing pull-off (suction) pressure

- Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Roof sheathing pressure coefficient (suction) = -2.2 (Table 3.9)
Step 5: Roof sheathing pressure (suction) = (13.1 psf)(-2.2)
= -28.8 psf

The fastener load depends on the spacing of roof framing and spacing of the fastener. Fasteners in the interior of the roof sheathing panel usually have the largest tributary area and therefore are critical. Assuming 24-inch-on-center roof framing, the fastener withdrawal load for a 12-inch-on-center fastener spacing is as follows:

$$\begin{aligned}\text{Fastener withdrawal load} &= (\text{fastener spacing})(\text{framing spacing}) \\ &\quad (\text{roof sheathing pressure}) \\ &= (1 \text{ ft})(2 \text{ ft})(-28.8 \text{ psf}) \\ &= -57.6 \text{ lbs}\end{aligned}$$



This load exceeds the allowable capacity of minimum conventional roof sheathing connections (i.e., 6d nail). Therefore, a larger nail (i.e., 8d) would be required for the given wind condition. At appreciably higher wind conditions, a closer fastener spacing or higher-capacity fastener (i.e., deformed shank nail) may be required; refer to Chapter 7.

4. Load on a roof truss

- Step 1: Velocity pressure = 14.6 psf (as before)
- Step 2: Adjusted velocity pressure = 13.1 psf (as before)
- Step 3: Skip
- Step 4: Roof truss pressure coefficient = -0.9, +0.4 (Table 3.9)
- Step 5: Determine design wind pressures

- (a) Uplift = $-0.9 (13.1 \text{ psf}) = -11.8 \text{ psf}$
- (b) Inward = $0.4 (13.1 \text{ psf}) = 5.2 \text{ psf}$

Since the inward wind pressure is less than the minimum roof live load (i.e., 15 psf, Table 3.4), the following load combinations would govern the roof truss design while the D+W load combination could be dismissed (refer to Table 3.1):

$$D + (L_r \text{ or } S)$$
$$0.6D + W_u^*$$

*The net uplift load for truss design is relatively small in this case (approximately 3.5 psf) and may be dismissed by an experienced designer.

5. Load on a rafter

- Step 1: Velocity pressure = 14.6 psf (as before)
- Step 2: Adjusted velocity pressure = 13.1 psf (as before)
- Step 3: Skip
- Step 4: Rafter pressure coefficient = -1.2, +0.7 (Table 3.9)
- Step 5: Determine design wind pressures

- (a) Uplift = $(-1.2)(13.1 \text{ psf}) = -15.7 \text{ psf}$
- (b) Inward = $(0.7)(13.1 \text{ psf}) = 9.2 \text{ psf}$

Rafters in cathedral ceilings are sloped, simply supported beams, whereas rafters that are framed with cross-ties (i.e., ceiling joists) constitute a component (i.e., top chord) of a site-built truss system. Assuming the former in this case, the rafter should be designed as a sloped beam by using the span measured along the slope. By inspection, the minimum roof live load (D+L_r) governs the design of the rafter in comparison to the wind load combinations (see Table 3.1). The load combination 0.6 D+W_u can be dismissed in this case for rafter sizing but must be considered when investigating wind uplift for the rafter-to-wall and rafter-to-ridge beam connections.



6. Lateral (out-of-plane) wind load on a wall stud

- Step 1: Velocity pressure = 14.6 psf (as before)
Step 2: Adjusted velocity pressure = 13.1 psf (as before)
Step 3: Skip
Step 4: Wall stud pressure coefficient = -1.2, +1.1 (Table 3.9)
Step 5: Determine design wind pressures

$$(a) \text{ Outward} = (-1.2)(13.1 \text{ psf}) = -15.7 \text{ psf}$$

$$(b) \text{ Inward} = (1.1)(13.1 \text{ psf}) = 14.4 \text{ psf}$$

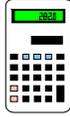
Obviously, the outward pressure of 15.7 psf governs the out-of-plane bending load design of the wall stud. Since the load is a lateral pressure (not uplift), the applicable load combination is D+W (refer to Table 3.1), resulting in a combined axial and bending load. The axial load would include the tributary building dead load from supported assemblies (i.e., walls, floors, and roof). The bending load would be determined by using the wind pressure of 15.7 psf applied to the stud as a uniform line load on a simply supported beam calculated as follows:

$$\begin{aligned} \text{Uniform line load, } w &= (\text{wind pressure})(\text{stud spacing}) \\ &= (15.7 \text{ psf})(1.33 \text{ ft}^*) \\ &\quad \text{*assumes a stud spacing of 16 inches on center} \\ &= 20.9 \text{ plf} \end{aligned}$$

Of course, the following gravity load combinations would also need to be considered in the stud design (refer to Table 3.1):

$$\begin{aligned} D + L + 0.3 (L_r \text{ or } S) \\ D + (L_r \text{ or } S) + 0.3 L \end{aligned}$$

It should be noted that the stud is actually part of a wall system (i.e., sheathing and interior finish) and can add substantially to the calculated bending capacity; refer to Chapter 5.

**EXAMPLE 3.3****Design Earthquake Load Calculation****Given**

- Site ground motion, $S_s = 1g$
- Site soil condition = firm (default)
- Roof snow load < 30 psf
- Two-story home, 28' x 44' plan, typical construction

Find

Design seismic shear on first-story end wall assuming no interior shear walls or contribution from partition walls

Solution

1. Determine tributary mass (weight) of building to first-story seismic shear

Roof dead load = (28 ft)(44 ft)(15 psf) = 18,480 lb

Second-story exterior wall dead load = (144 lf)(8 ft)(8 psf) = 9,216 lb

Second-story partition wall dead load = (28 ft)(44 ft)(6 psf) = 7,392 lb

Second-story floor dead load = (28 ft)(44 ft)(10 psf) = 12,320 lb

First-story exterior walls (1/2 height) = (144 lf)(4 ft)(8 psf) = 4,608 lb

Assume first-story interior partition walls are capable of at least supporting the seismic shear produced by their own weight

Total tributary weight = 52,016 lb

2. Determine total seismic story shear on first story

$$\begin{aligned} S_{DS} &= \frac{2}{3} (S_s)(F_a) && \text{(Equation 3.8-2)} \\ &= \frac{2}{3} (1.0g)(1.1) && (F_a = 1.1 \text{ from Table 3.11}) \\ &= 0.74 g \end{aligned}$$

$$\begin{aligned} V &= \frac{1.2 S_{DS}}{R} W \\ &= \frac{1.2(0.74g)}{5.5} (52,016 \text{ lb}) && (R = 5.5 \text{ from Table 3.12}) \\ &= 8,399 \text{ lb} \end{aligned}$$

3. Determine design shear load on the 28-foot end walls

Assume that the building mass is evenly distributed and that stiffness is also reasonably balanced between the two end walls; refer to Chapter 6 for additional guidance.

With the above assumption, the load is simply distributed to the end walls according to tributary weight (or plan area) of the building. Therefore,

$$\text{End wall shear} = 1/2 (8,399 \text{ lb}) = 4,200 \text{ lb}$$

Note that the design shear load from wind (100 mph gust, exposure B) in Example 3.2 is somewhat greater (5,912 lbs).



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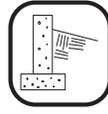
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CHAPTER 4

Design of Foundations

4.1 General

A foundation transfers the load of a structure to the earth and resists loads imposed by the earth. A foundation in residential construction may consist of a footing, wall, slab, pier, pile, or a combination of these elements. This chapter addresses the following foundation types:

- crawl space;
- basement;
- slab-on-grade with stem wall;
- monolithic slab;
- piles;
- piers; and
- alternative methods.

As discussed in Chapter 1, the most common residential foundation materials are concrete masonry (i.e., concrete block) and cast-in-place concrete. Preservative-treated wood, precast concrete, and other methods may also be used. The concrete slab on grade is the most popular foundation type in the Southeast; basements are the most common type in the East and Midwest. Crawl spaces are common in the Northwest and Southeast. Pile foundations are commonly used in coastal flood zones to elevate structures above flood levels, in weak or expansive soils to reach a stable stratum, and on steeply sloped sites. Figure 4.1 depicts different foundation types; a brief description follows.

A *crawl space* is a building foundation that uses a perimeter foundation wall to create an under-floor space that is not habitable; the interior crawl space elevation may or may not be below the exterior finish grade. A *basement* is typically defined as a portion of a building that is partly or completely below the exterior grade and that may be used as habitable or storage space.

A *slab on grade with an independent stem wall* is a concrete floor supported by the soil independently of the rest of the building. The stem wall supports the building loads and in turn is supported directly by the soil or a



footing. A *monolithic or thickened-edge slab* is a ground-supported slab on grade with an integral footing (i.e., thickened edge); it is normally used in warmer regions with little or no frost depth but is also used in colder climates when adequate frost protection is provided (see Section 4.7).

When necessary, *piles* are used to transmit the load to a deeper soil stratum with a higher bearing capacity, to prevent failure due to undercutting of the foundation by scour from flood water flow at high velocities, and to elevate the building above required flood elevations. Piles are also used to isolate the structure from expansive soil movements.

Post-and-pier foundations can provide an economical alternative to crawl space perimeter wall construction. It is common practice to use a brick curtain wall between piers for appearance and bracing purposes.

The design procedures and information in this chapter cover

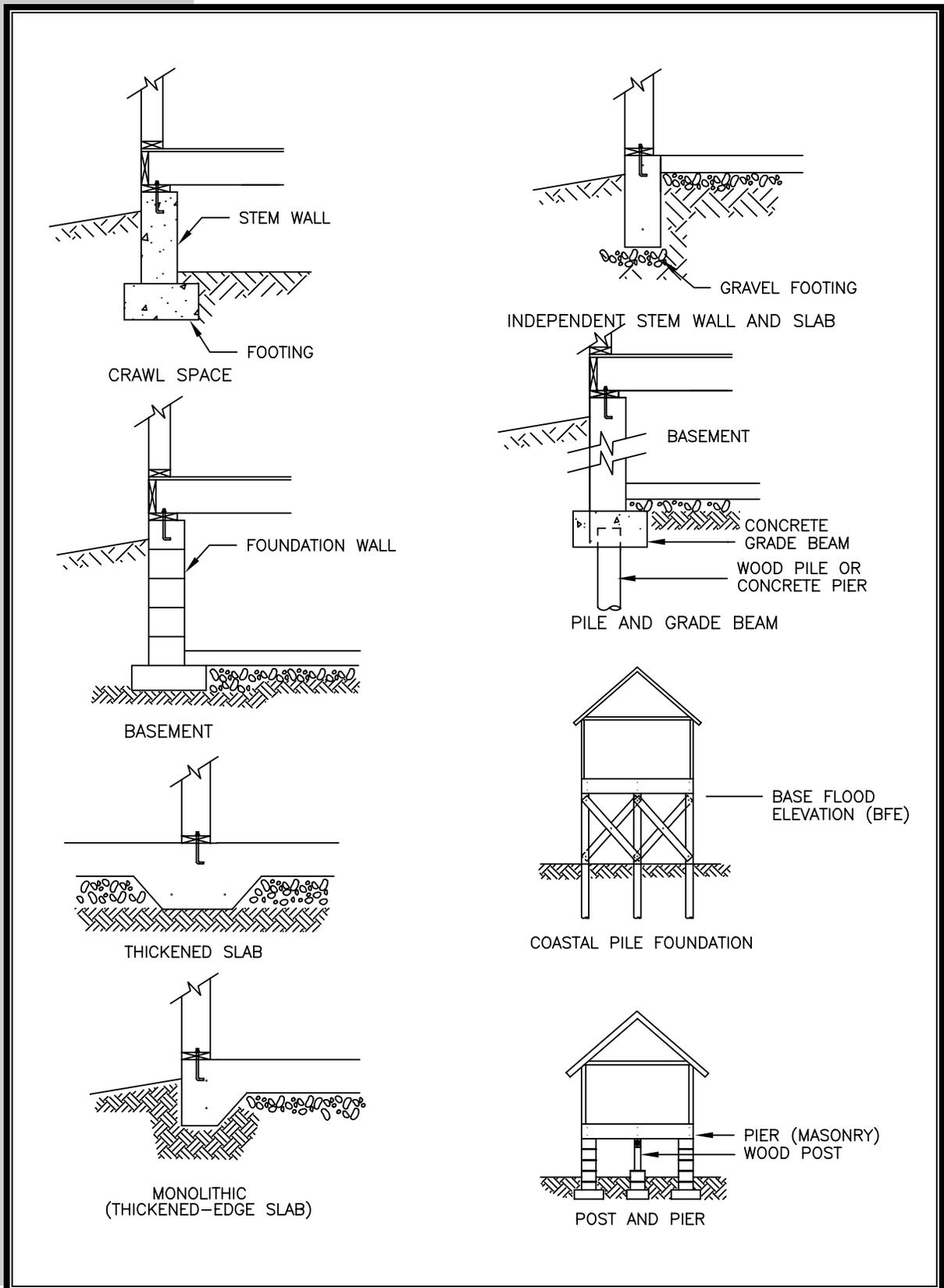
- foundation materials and properties;
- soil bearing capacity and footing size;
- concrete or gravel footings;
- concrete and masonry foundation walls;
- preservative-treated wood walls;
- insulating concrete foundations;
- concrete slabs on grade;
- pile foundations; and
- frost protection.

Concrete design procedures generally follow the strength design method contained in ACI-318 (ACI, 1999), although certain aspects of the procedures may be considered conservative relative to conventional residential foundation applications. For this reason, some supplemental design guidance is provided when practical and technically justified. Masonry design procedures follow the allowable stress design method of ACI-530 (ACI, 1999). Wood design procedures are used to design the connections between the foundation system and the structure above and follow the allowable stress design method for wood construction; refer to Chapter 7 for connection design information. In addition, the designer is referred to the applicable design standards for symbol definitions and additional guidance since the intent of this chapter is to provide supplemental instruction in the efficient design of residential foundations.

As a matter of consistency within the scope of this guide, the LRFD load combinations of Chapter 3 (Table 3.1) are used in lieu of those required in ACI-318 for strength design of concrete. The designer is advised of this variance from what may be considered accepted practice in the local building code. However, the intent is to provide designs that are at least consistent with current residential building code and construction practice. With respect to the design of concrete in residential foundations, it is also intended to provide reasonable safety margins that are at least consistent with the minimums required for other more crucial (i.e., life-safety) elements of a home. If an actual design is performed in accordance with this guide, it is the responsibility of the designer to seek any special approval that may be required for “alternative means and methods” of design and to identify where and when such approval is needed.



FIGURE 4.1 *Types of Foundations*





4.2 Material Properties

A residential designer using concrete and masonry materials must have a basic understanding of such materials as well as an appreciation of variations in the materials' composition and structural properties. In addition, soils are considered a foundation material (Section 4.3 provides information on soil bearing). A brief discussion of the properties of concrete and masonry follows.

4.2.1 Concrete

The concrete compressive strength f_c' used in residential construction is typically either 2,500 or 3,000 psi, although other values may be specified. For example, 3,500 psi concrete may be used for improved weathering resistance in particularly severe climates or unusual applications. The concrete compressive strength may be verified in accordance with ASTM C39 (ASTM, 1996). Given that concrete strength increases at a diminishing rate with time, the specified compressive strength is usually associated with the strength attained after 28 days of curing time. At that time, concrete generally attains about 85 percent of its fully cured compressive strength.

Concrete is a mixture of cement, water, sand, gravel, crushed rock, or other aggregates. Sometimes one or more admixtures are added to change certain characteristics of the concrete, such as workability, durability, and time of hardening. The proportions of the components determine the concrete mix's compressive strength and durability.

Type

Portland cement is classified into several types in accordance with ASTM C150 (ASTM, 1998). Residential foundation walls are typically constructed with *Type I* cement, which is a general-purpose Portland cement used for the vast majority of construction projects. Other types of cement are appropriate in accommodating conditions related to heat of hydration in massive pours and sulfate resistance. In some regions, sulfates in soils have caused durability problems with concrete. The designer should check into local conditions and practices.

Weight

The weight of concrete varies depending on the type of aggregates used in the concrete mix. Concrete is typically referred to as lightweight or normal weight. The density of unreinforced normal weight concrete ranges between 144 and 156 pounds per cubic foot (pcf) and is typically assumed to be 150 pcf. Residential foundations are constructed with normal weight concrete.



Slump

Slump is the measure of concrete consistency; the higher the slump, the wetter the concrete and the easier it flows. Slump is measured in accordance with ASTM C143 (ASTM, 1998) by inverting a standard 12-inch-high metal cone, filling it with concrete, and then removing the cone; the amount the concrete settles in units of inches is the slump. Most foundations, slabs, and walls consolidated by hand methods have a slump between 4 and 6 inches. One problem associated with a high-slump concrete is segregation of the aggregate, which leads to cracking and scaling. Therefore, a slump of greater than 6 should be avoided.

Admixtures

Admixtures are materials added to the concrete mix to improve workability and durability and to retard or accelerate curing. Some of the most common admixtures are described below.

- *Water reducers* improve the workability of concrete without reducing its strength.
- *Retarders* are used in hot weather to allow more time for placing and finishing concrete. Retarders may also reduce the early strength of concrete.
- *Accelerators* reduce the setting time, allowing less time for placing and finishing concrete. Accelerators may also increase the early strength of concrete.
- *Air-entrainers* are used for concrete that will be exposed to freeze-thaw conditions and deicing salts. Less water is needed, and desegregation of aggregate is reduced when air-entrainers are added.

Reinforcement

Concrete has high compressive strength but low tensile strength; therefore, reinforcing steel is often embedded in the concrete to provide additional tensile strength and ductility. In the rare event that the capacity may be exceeded, the reinforcing steel begins to yield, eliminating an abrupt failure that may otherwise occur in plain, unreinforced concrete. For this reason, a larger safety margin is used in the design of plain concrete construction than in reinforced concrete construction.

Steel reinforcement is available in Grade 40 or Grade 60; the grade number refers to the minimum tensile yield strength f_y of the steel (i.e., Grade 40 is minimum 40 ksi steel and Grade 60 is minimum 60 ksi steel). Either grade may be used for residential construction; however, most reinforcement in the U.S. market today is Grade 60. It is also important that the concrete mix or slump is adjusted through the addition of an appropriate amount of water to allow the concrete to flow easily around the reinforcement bars, particularly when the bars are closely spaced or crowded at points of overlap. However, close spacing is rarely required in residential construction and should be avoided in design.



The most common steel reinforcement or rebar sizes in residential construction are No. 3, No. 4, and No. 5, which correspond to diameters of 3/8-inch, 1/2-inch, and 5/8-inch, respectively. These three sizes of rebar are easily handled at the jobsite by using manual bending and cutting devices. Table 4.1 provides useful relationships among the rebar number, diameter, and cross-sectional for reinforced concrete and masonry design.

TABLE 4.1 *Rebar Size, Diameter, and Cross-Sectional Areas*

Size	Diameter (inches)	Area (square inches)
No. 3	3/8	0.11
No. 4	1/2	0.20
No. 5	5/8	0.31
No. 6	3/4	0.44
No. 7	7/8	0.60
No. 8	1	0.79

4.2.2 Concrete Masonry Units

Concrete masonry units (CMU) are commonly referred to as concrete blocks. They are composed of Portland cement, aggregate, and water. Admixtures may also be added in some situations. Low-slump concrete is molded and cured to produce strong blocks or units. Residential foundation walls are typically constructed with units 7-5/8 inches high by 15-5/8 inches long, providing a 3/8-inch allowance for the width of mortar joints.

In residential construction, nominal 8-inch-thick concrete masonry units are readily available. It is generally more economical if the masonry unit compressive strength f'_m ranges between 1,500 and 3,000 psi. The standard block used in residential and light-frame commercial construction is generally rated with a design strength f'_m of 1,900 psi, although other strengths are available.

Grade

Concrete masonry units are described by grades according to their intended use per ASTM C90 (ASTM, 1999) or C129 (ASTM, 1999). Residential foundation walls should be constructed with *Grade N* units. *Grade S* may be used above grade. The grades are described below.

- *Grade N* is typically required for general use such as in interior and backup walls and in above- or below-grade exterior walls that may or may not be exposed to moisture penetration or the weather.
- *Grade S* is typically limited to above-grade use in exterior walls with weather-protective coatings and in walls not exposed to the weather.



Type

Concrete masonry units are classified in accordance with ASTM C90 as *Type I* or *Type II* (ASTM, 1999). *Type I* is a moisture-controlled unit that is typically specified where drying shrinkage of the block due to moisture loss may result in excessive cracking in the walls. *Type II* is a nonmoisture-controlled unit that is suitable for all other uses. Residential foundation walls are typically constructed with *Type II* units.

Weight

Concrete masonry units are available with different densities by altering the type(s) of aggregate used in their manufacture. Concrete masonry units are typically referred to as lightweight, medium weight, or normal weight with respective unit weights or densities less than 105 pcf, between 105 and 125 pcf, and more than 125 pcf. Residential foundation walls are typically constructed with low- to medium-weight units because of the low compressive strength required. However, lower-density units are generally more porous and must be properly protected to resist moisture intrusion. A common practice in residential basement foundation wall construction is to provide a cement-based parge coating and a brush- or spray-applied bituminous coating on the below-ground portions of the wall. This treatment is usually required by code for basement walls of masonry or concrete construction; however, in concrete construction, the parge coating is not necessary.

Hollow or Solid

Concrete masonry units are classified as hollow or solid in accordance with ASTM C90 (ASTM, 1999). The net concrete cross-sectional area of most concrete masonry units ranges from 50 to 70 percent depending on unit width, face-shell and web thicknesses, and core configuration. *Hollow* units are defined as those in which the net concrete cross-sectional area is less than 75 percent of the gross cross-sectional area. *Solid* units are not necessarily solid but are defined as those in which the net concrete cross-sectional area is 75 percent of the gross cross-sectional area or greater.

Mortar

Masonry mortar is used to join concrete masonry units into a structural wall; it also retards air and moisture infiltration. The most common way to lay block is in a running bond pattern where the vertical head joints between blocks are offset by half the block length from one course to the next. Mortar is composed of cement, lime, clean, well-graded sand, and water and is typically classified into *Types M, S, N, O,* and *K* in accordance with ASTM C270 (ASTM, 1999). Residential foundation walls are typically constructed with *Type M* or *Type S* mortar, both of which are generally recommended for load-bearing interior and exterior walls including above- and below-grade applications.



Grout

Grout is a slurry consisting of cementitious material, aggregate, and water. When needed, grout is commonly placed in the hollow cores of concrete masonry units to provide a wall with added strength. In reinforced load-bearing masonry wall construction, grout is usually placed only in those hollow cores containing steel reinforcement. The grout bonds the masonry units and steel so that they act as a composite unit to resist imposed loads. Grout may also be used in unreinforced concrete masonry walls for added strength.

4.3 Soil Bearing Capacity and Footing Size

Soil bearing investigations are rarely required for residential construction except in the case of known risks as evidenced by a history of local problems (e.g., organic deposits, landfills, expansive soils, etc.). Soil bearing tests on stronger-than-average soils can, however, justify smaller footings or eliminate footings entirely if the foundation wall provides sufficient bearing surface. For a conservative relationship between soil type and load-bearing value, refer to Table 4.2. A similar table is typically published in the building codes.

TABLE 4.2 Presumptive Soil Bearing Values by Soil Description

Presumptive Load-Bearing Value (psf)	Soil Description
1,500	Clay, sandy clay, silty clay, clayey silt, silt, and sandy silt
2,000	Sand, silty sand, clayey sand, silty gravel, and clayey gravel
3,000	Gravel and sandy gravel
4,000	Sedimentary rock
12,000	Crystalline bedrock

Source: Naval Facilities Command, 1986.

When a soil bearing investigation is desired to determine more accurate and economical footing requirements, the designer commonly turns to ASTM D1586, *Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils* (ASTM, 1999). This test relies on a 2-inch-diameter device driven into the ground with a 140-pound hammer dropped from a distance of 30 inches. The number of hammer drops or blows needed to create a one-foot penetration (blow count) is recorded. Values can be roughly correlated to soil bearing values as shown in Table 4.3. The instrumentation and cost of conducting the SPT test is usually not warranted for typical residential applications. Nonetheless, the SPT test method provides information on deeper soil strata and thus can offer valuable guidance for foundation design and building location, particularly when subsurface conditions are suspected to be problematic. The values in Table 4.3 are associated



with the blow count from the SPT test method. Many engineers can provide reasonable estimates of soil bearing by using smaller penetrometers at less cost, although such devices and methods may require an independent calibration to determine presumptive soil bearing values and may not be able to detect deep subsurface problems. Calibrations may be provided by the manufacturer or, alternatively, developed by the engineer.

The designer should exercise judgment when selecting the final design value and be prepared to make adjustments (increases or decreases) in interpreting and applying the results to a specific design. The values in Tables 4.2 and 4.3 are generally associated with a safety factor of 3 (Naval Facilities Engineering Command, 1996) and are considered appropriate for noncontinuous or independent spread footings supporting columns or piers (i.e., point loads). Use of a minimum safety factor of 2 (corresponding to a higher presumptive soil bearing value) is recommended for smaller structures with continuous spread footings such as houses. To achieve a safety factor of 2, the designer may multiply the values in Tables 4.2 and 4.3 by 1.5.

Table 4.3

Presumptive Soil Bearing Values (psf) Based on Standard Penetrometer Blow Count

In Situ Consistency, N ¹		Loose ² (5 to 10 blows per foot)	Firm (10 to 25 blows per foot)	Compact (25 to 50 blows per foot)
Noncohesive Soils	Gravel	4,000 (10)	8,000 (25)	11,000 (50)
	Sand	2,500 (6)	5,000 (20)	6,000 (35)
	Fine sand	1,000 (5)	3,000 (12)	5,000 (30)
	Silt	500 (5)	2,000 (15)	4,000 (35)
Insitu Consistency, N ¹ :		Soft ³ (3 to 5 blows per foot)	Medium (about 10 blows per foot)	Stiff (> 20 blows per foot)
Cohesive Soils	Clay, Sand, Gravel Mixtures	2,000 (3)	5,000 (10)	8,000 (20)
	Sandy or Silty Clay	1,000 (4)	3,000 (8)	6,000 (20)
	Clay	500 (5)	2,000 (10)	4,000 (25)

Source: Naval Facilities Command, 1986.

Notes:

¹N denotes the standard penetrometer blow count in blows per foot in accordance with ASTM D1586; shown in parentheses.

²Compaction should be considered in these conditions, particularly when the blow count is five blows per foot or less.

³Pile and grade beam foundations should be considered in these conditions, particularly when the blow count is five blows per foot or less.

The required width or area of a spread footing is determined by dividing the building load on the footing by the soil bearing capacity from Table 4.2 or Table 4.3 as shown below. Building design loads, including dead and live loads, should be determined in accordance with Chapter 3 by using allowable stress design (ASD) load combinations.



$$\text{Area}_{\text{independent spread footing}} = \frac{\text{Load in lbs}}{\text{Soil bearing capacity in psf}}$$

$$\text{Width}_{\text{continuous footing}} = \frac{\text{Load in plf}}{\text{Soil bearing capacity in psf}}$$

4.4 Footings

The objectives of footing design are

- to provide a level surface for construction of the foundation wall;
- to provide adequate transfer and distribution of building loads to the underlying soil;
- to provide adequate strength, in addition to the foundation wall, to prevent differential settlement of the building in weak or uncertain soil conditions;
- to place the building foundation at a sufficient depth to avoid frost heave or thaw weakening in frost-susceptible soils and to avoid organic surface soil layers; and
- to provide adequate anchorage or mass (when needed in addition to the foundation wall) to resist potential uplift and overturning forces resulting from high winds or severe seismic events.

This section presents design methods for concrete and gravel footings. The designer is reminded that the required footing width is first established in accordance with Section 4.3. Further, if soil conditions are stable or the foundation wall can adequately resist potential differential settlement, the footing may be completely eliminated.

By far, the most common footing in residential construction is a continuous concrete spread footing. However concrete and gravel footings are both recognized in prescriptive footing size tables in residential building codes for most typical conditions (ICC, 1998). In contrast, special conditions give rise to some engineering concerns that need to be addressed to ensure the adequacy of any foundation design. Special conditions include

- steeply sloped sites requiring a stepped footing;
- high-wind conditions;
- inland or coastal flooding conditions;
- high-hazard seismic conditions; and
- poor soil conditions.

4.4.1 Simple Gravel and Concrete Footing Design

Building codes for residential construction contain tables that prescribe minimum footing widths for plain concrete footings (ICC, 1998). Alternatively, footing widths may be determined in accordance with Section 4.3 based on a



site's particular loading condition and presumptive soil bearing capacity. The following are general rules of thumb for determining the thickness of plain concrete footings for residential structures once the required bearing width is calculated:

- The minimum footing thickness should not be less than the distance the footing extends outward from the edge of the foundation wall or 6 inches, whichever is greater.
- The footing width should project a minimum of 2 inches from both faces of the wall (to allow for a minimum construction tolerance) but not greater than the footing thickness.

These rules of thumb generally result in a footing design that differs somewhat from the plain concrete design provisions of Chapter 22 of ACI-318. It should also be understood that footing widths generally follow the width increments of standard excavation equipment (i.e., a backhoe bucket size of 12, 16, or 24 inches). Even though some designers and builders may specify one or two longitudinal No. 4 bars for wall footings, steel reinforcement is not required for residential-scale structures in typical soil conditions. For situations where the rules of thumb or prescriptive code tables do not apply or where a more economical solution is possible, a more detailed footing analysis may be considered (see Section 4.4.2). Refer to Example 4.1 for a plain concrete footing design in accordance with the simple method described herein.

Much like a concrete footing, a gravel footing may be used to distribute foundation loads to a sufficient soil bearing surface area. It also provides a continuous path for water or moisture and thus must be drained in accordance with the foundation drainage provisions of the national building codes. Gravel footings are constructed of crushed stone or gravel that is consolidated by tamping or vibrating. Pea gravel, which is naturally consolidated, does not require compaction and can be screeded to a smooth, level surface much like concrete. Although typically associated with pressure-treated wood foundations (refer to Section 4.5.3), a gravel footing can support cast-in-place or precast concrete foundation walls.

The size of a gravel footing is usually based on a 30- to 45-degree angle of repose for distributing loads; therefore, as with plain concrete footings, the required depth and width of the gravel footing depends on the width of the foundation wall, the foundation load, and soil bearing values. Following a rule of thumb similar to that for a concrete footing, the gravel footing thickness should be no less than 1.5 times its extension beyond the edge of the foundation wall or, in the case of a pressure-treated wood foundation, the mud sill. Just as with a concrete footing, the thickness of a gravel footing may be considered in meeting the required frost depth. In soils that are not naturally well-drained, provision should be made to adequately drain a gravel footing.

4.4.2 Concrete Footing Design

For the vast majority of residential footing designs, it quickly becomes evident that conventional residential footing requirements found in residential building codes are adequate, if not conservative (ICC,1998). However, to improve



performance and economy or to address peculiar conditions, a footing may need to be specially designed.

A footing is designed to resist the upward-acting pressure created by the soil beneath the footing; that pressure tends to make the footing bend upward at its edges. According to ACI-318, the three modes of failure considered in reinforced concrete footing design are one-way shear, two-way shear, and flexure (see Figure 4.2). Bearing (crushing) is also a possible failure mode, but is rarely applicable to residential loading conditions. To simplify calculations for the three failure modes, the following discussion explains the relation of the failure modes to the design of plain and reinforced concrete footings. The designer should refer to ACI-318 for additional commentary and guidance. The design equations used later in this section are based on ACI-318 and principles of engineering mechanics as described below. Moreover, the approach is based on the assumption of uniform soil bearing pressure on the bottom of the footing; therefore, walls and columns should be supported as close as possible to the center of the footings.

One-Way (Beam) Shear

When a footing fails due to *one-way (beam) shear*, the failure occurs at an angle approximately 45 degrees to the wall as shown in Figure 4.2. For plain concrete footings, the soil bearing pressure has a negligible effect on the diagonal shear tension for distance t from the wall edge toward the footing edge; for reinforced concrete footings, the distance used is d , which equals the depth to the footing rebar (see Figure 4.2). As a result, one-way shear is checked by assuming that beam action occurs at a critical failure plane extending across the footing width as shown in Figure 4.2. One-way shear must be considered in similar fashion in both continuous wall and rectangular footings; however, for ease of calculation, continuous wall footing design is typically based on one lineal foot of wall/footing.

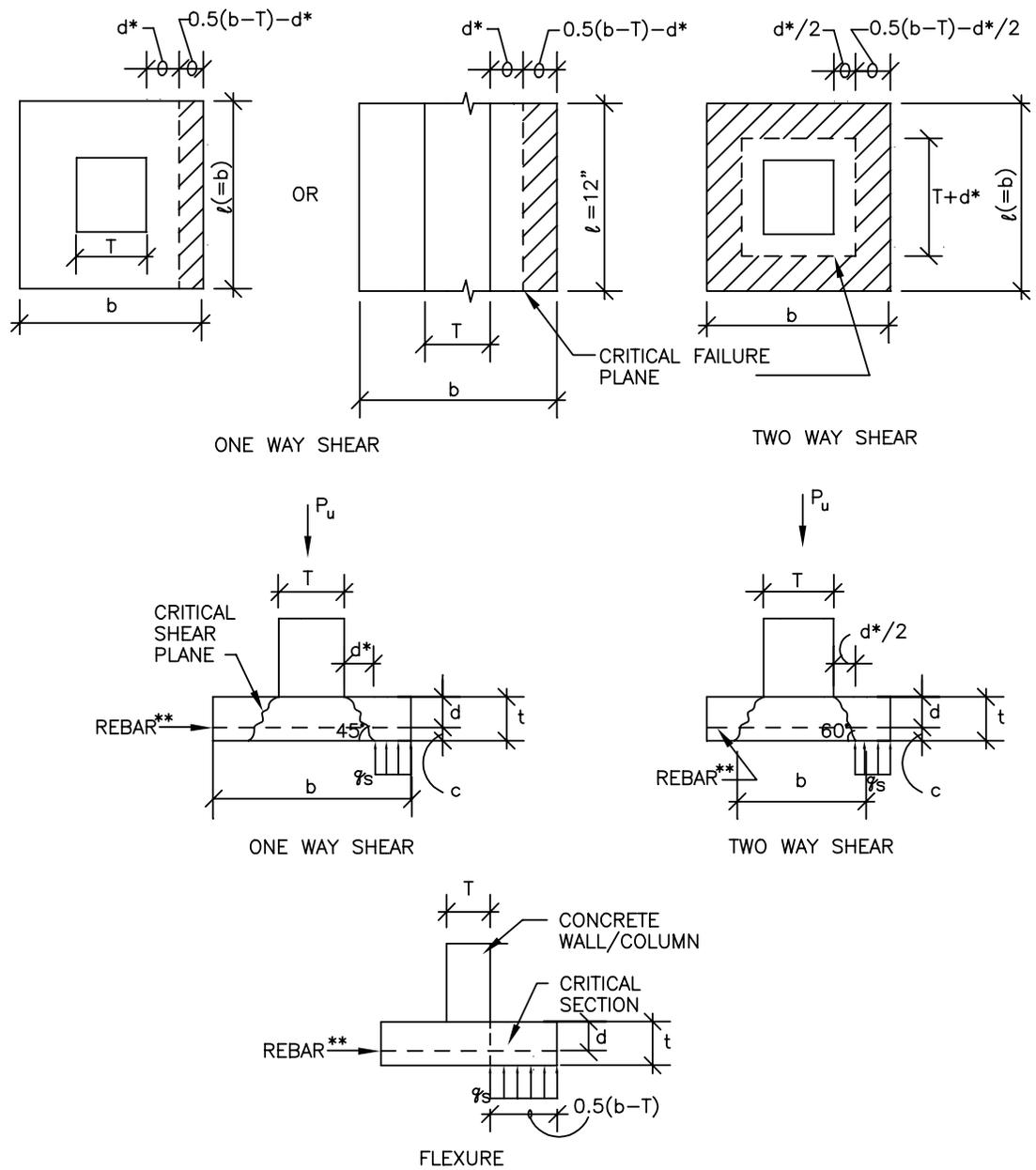
Two-Way (Punching) Shear

When a footing fails by *two-way (punching) shear*, the failure occurs at an angle approximately 30 degrees to the column or pier as shown in Figure 4.2. Punching shear is rarely a concern in the design of continuous wall footings and thus is usually checked only in the case of rectangular or circular footings with a heavily loaded pier or column that creates a large concentrated load on a relatively small area of the footing. For plain concrete footings, the soil bearing pressure has a negligible effect on the diagonal shear tension at distance $t/2$ from the face of a column toward the footing edges; for reinforced concrete footings, the distance from the face of the column is $d/2$ (see Figure 4.2). Therefore, the shear force consists of the net upward-acting pressure on the area of the footing outside the “punched-out” area (hatched area in Figure 4.2). For square, circular, or rectangular footings, shear is checked at the critical section that extends in a plane around a concrete, masonry, wood, or steel column or pier that forms the perimeter b_o of the area described above.



FIGURE 4.2

Critical Failure Planes in Continuous or Square Concrete Spread Footings



NOTES: * SUBSTITUTE t FOR d AS REQUIRED FOR PLAIN CONCRETE FOOTING DESIGN

** REBAR IS REQUIRED ONLY IN REINFORCED CONCRETE FOOTING DESIGN AND IS SHOWN HERE FOR THAT PURPOSE ONLY. IN REINFORCED SQUARE FOOTINGS, THE REBAR MUST BE PLACED IN TWO DIRECTIONS.



Flexure (Bending)

The maximum moment in a footing deformed by the upward-acting soil pressures would logically occur in the middle of the footing; however, the rigidity of the wall or column above resists some of the upward-acting forces and affects the location of maximum moment. As a result, the critical flexure plane for footings supporting a rigid wall or column is assumed to be located at the face of the wall or column. Flexure in a concrete footing is checked by computing the moment created by the soil bearing forces acting over the cantilevered area of the footing that extends from the critical flexure plane to the edge of the footing (hatched area in Figure 4.2). The approach for masonry walls in ACI-318 differs slightly in that the failure plane is assumed to be located one-fourth of the way under a masonry wall or column, creating a slightly longer cantilever. For the purpose of this guide, the difference is considered unnecessary.

Bearing Strength

It is difficult to contemplate conditions where concrete bearing or compressive strength is a concern in typical residential construction; therefore, a design check can usually be dismissed as “OK by inspection.” In rare and peculiar instances where bearing compressive forces on the concrete are extreme and approach or exceed the specified concrete compressive strength, ACI-318•10.17 and ACI-318•12.3 should be consulted for appropriate design guidance.

4.4.2.1 Plain Concrete Footing Design

In this section, the design of plain concrete footings is presented by using the concepts related to shear and bending covered in the previous section. Refer to Example 4.1 in Section 4.9 for a plain concrete footing design example.

Shear

In the equations given below for one- and two-way shear, the dimensions are in accordance with Figure 4.2; units of inches should be used. ACI-318 requires an additional 2 inches of footing thickness to compensate for uneven trench conditions and does not allow a total footing thickness less than 8 inches for plain concrete. These limits may be relaxed for residential footing design, provided that the capacity is shown to be sufficient in accordance with the ACI-318 design equations. Footings in residential construction are often 6 inches thick. The equations below are specifically tailored for footings supporting walls or square columns since such footings are common in residential construction. The equations may be generalized for use with other conditions (i.e., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. In addition, the terms $4/3 \sqrt{f'_c}$ and $4 \sqrt{f'_c}$ are in units of pounds per square inch and represent “lower-bound” estimates of the ultimate shear stress capacity of unreinforced concrete.



[ACI-318•22.5,22.7]

One-Way (Beam) Shear

$\phi V_c \geq V_u$	basic design check for shear
$V_u = (q_s)(0.5(b-T)-t)\ell$	factored shear load (lb)
$q_s = \frac{P_u}{b\ell}$	uniform soil bearing pressure (psi) due to factored foundation load P_u (lb)
$\phi V_c = \phi \frac{4}{3} \sqrt{f'_c} \ell t$	factored shear capacity (lb)
$\phi = 0.65$	resistance factor

Two-Way (Punching) Shear

$\phi V_c \geq V_u$	basic design check for shear
$V_u = (q_s)(b\ell - (T+t)^2)$	shear load (lb) due to factored load P_u (lb)
$q_s = \frac{P_u}{b\ell}$	uniform soil bearing pressure (psi) due to factored foundation load P_u (lb)
$\phi V_c = \phi 4 \sqrt{f'_c} b_o t$	factored shear capacity (lb)
$b_o = 4(T+t)$	perimeter of critical failure plane around a square column or pier
$\phi = 0.65$	resistance factor

Flexure

For a plain concrete footing, flexure (bending) is checked by using the equations below for footings that support walls or square columns (see Figure 4.2). The dimensions in the equations are in accordance with Figure 4.2 and use units of inches. The term $5\sqrt{f'_c}$ is in units of pounds per square inch (psi) and represents a “lower-bound” estimate of the ultimate tensile (rupture) stress of unreinforced concrete in bending.

[ACI-318•22.5,22.7]

$\phi M_n \geq M_u$	basic design check for bending
$M_u = \frac{1}{8} q_s \ell (b-T)^2$	factored moment (in-lb) due to soil pressure q_s (psi) acting on cantilevered portion of footing
$q_s = \frac{P_u}{b\ell}$	uniform soil bearing pressure (psi) due to factored load P_u (lb)
$\phi M_n = \phi 5 \sqrt{f'_c} S$	factored moment capacity (in-lb) for plain concrete
$S = \frac{1}{6} \ell t^2$	section modulus (in ³) for footing
$\phi = 0.65$	resistance factor for plain concrete in bending



4.4.2.2 Reinforced Concrete Footing Design

For infrequent situations in residential construction where a plain concrete footing may not be practical or where it is more economical to reduce the footing thickness, steel reinforcement may be considered. A reinforced concrete footing is designed similar to a plain concrete footing; however, the concrete depth d to the reinforcing bar is used to check shear instead of the entire footing thickness t . The depth of the rebar is equal to the thickness of the footing minus the diameter of the rebar d_b and the concrete cover c . In addition, the moment capacity is determined differently due to the presence of the reinforcement, which resists the tension stresses induced by the bending moment. Finally, a higher resistance factor is used to reflect the more consistent bending strength of reinforced concrete relative to unreinforced concrete.

As specified by ACI-318, a minimum of 3 inches of concrete cover over steel reinforcement is required when concrete is in contact with soil. In addition, ACI-318 does not permit a depth d less than 6 inches for reinforced footings supported by soil. These limits may be relaxed by the designer, provided that adequate capacity is demonstrated in the strength analysis; however, a reinforced footing thickness of significantly less than 6 inches may be considered impractical even though it may calculate acceptably. One exception may be found where a nominal 4-inch-thick slab is reinforced to serve as an integral footing for an interior load-bearing wall (that is not intended to transmit uplift forces from a shear wall overturning restraint anchorage in high-hazard wind or seismic regions). Further, the concrete cover should not be less than 2 inches for residential applications, although this recommendation may be somewhat conservative for interior footings that are generally less exposed to ground moisture and other corrosive agents. Example 4.2 of Section 4.9 illustrates reinforced concrete footing design.

Shear

In the equations given below for one- and two-way shear, the dimensions are in accordance with Figure 4.2; units of inches should be used. Shear reinforcement (i.e., stirrups) is usually considered impractical for residential footing construction; therefore, the concrete is designed to withstand the shear stress as expressed in the equations. The equations are specifically tailored for footings supporting walls or square columns since such footings are common in residential construction. The equations may be generalized for use with other conditions (i.e., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. In addition, the terms $2\sqrt{f'_c}$ and $4\sqrt{f'_c}$ are in units of pounds per square inch and represent “lower-bound” estimates of the ultimate shear stress capacity of reinforced concrete.



[ACI-318•11.12,15.5]

One-Way (Beam) Shear

$$\phi V_c \geq V_u \quad \text{basic design check for shear}$$

$$V_u = (q_s)(0.5(b-T)-d)\ell \quad \text{shear load (lb) due to uniform soil bearing pressure, } q_s \text{ (psi)}$$

$$q_s = \frac{P_u}{b\ell} \quad \text{uniform solid bearing pressure (psi) due to factored foundation load } P_u \text{ (lb)}$$

$$\phi V_c = \phi 2\sqrt{f'_c} \ell d \quad \text{factored shear capacity (lb)}$$

$$d = t - c - 0.5d_b \quad \text{depth of reinforcement}$$

$$\phi = 0.85 \quad \text{resistance factor for reinforced concrete in shear}$$

Two-Way (Punching) Shear

$$\phi V_c \geq V_u \quad \text{basic design check for shear}$$

$$V_u = \left(\frac{P_u}{b\ell} \right) (b\ell - (T+d)^2) \quad \text{shear load (lb) due to factored load } P_u \text{ (lb)}$$

$$\phi V_c = \phi 4\sqrt{f'_c} b_o d \quad \text{factored shear capacity (lb)}$$

$$b_o = 4(T+d) \quad \text{perimeter of punching shear failure plane around a square column or pier}$$

$$\phi = 0.85 \quad \text{resistance factor for reinforced concrete in shear}$$

Flexure

The flexure equations below pertain specifically to reinforced concrete footings that support walls or square columns. The equations may be generalized for use with other conditions (i.e., rectangular footings and rectangular columns, round footings, etc.) by following the same principles. The alternative equation for nominal moment strength M_n is derived from force and moment equilibrium principles by using the provisions of ACI-318. Most designers are familiar with the alternative equation that uses the reinforcement ratio ρ and the nominal strength coefficient of resistance R_n . The coefficient is derived from the design check that ensures that the factored moment (due to factored loads) M_u is less than the factored nominal moment strength ϕM_n of the reinforced concrete. To aid the designer in short-cutting these calculations, design manuals provide design tables that correlate the nominal strength coefficient of resistance R_n to the reinforcement ratio ρ for a specific concrete compressive strength and steel yield strength.



[ACI-318•15.4]

$\phi M_n \geq M_u$	basic design check for bending
$M_u = \frac{1}{8}q_s \ell(b-T)^2$	factored moment (in-lb) due to soil pressure q_s (psi) acting on cantilevered portion of the footing
$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$	factored nominal moment capacity (in-lb)
$a = \frac{A_s f_y}{0.85 f'_c \ell}$	(ℓ is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)
$\phi = 0.9$	resistance factor for reinforced concrete in bending

Alternate method to determine M_n

$\phi M_n = \phi \rho b d f_y \left(d - \frac{0.5 \rho d f_y}{0.85 f'_c} \right)$	
$\rho = \left(\frac{0.85 f'_c}{f_y} \right) \left(\ell - \sqrt{\frac{2 R_n}{0.85 f'_c}} \right)$	reinforcement ratio determined by use of R_n
	nominal strength “coefficient of resistance”
$R_n = \frac{M_u}{\phi \ell d^2}$	(ℓ is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)
	defines reinforcement ratio ρ
$A_s = \rho \ell d$	(ℓ is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)

Minimum Reinforcement

Owing to concerns with shrinkage and temperature cracking, ACI-318 requires a minimum amount of steel reinforcement. The following equations determine minimum reinforcement, although many plain concrete residential footings have performed successfully and are commonly used. Thus, the ACI minimums may be considered arbitrary, and the designer may use discretion in applying the ACI minimums in residential footing design. The minimums certainly should not be considered a strict “pass/fail” criterion.

[ACI-318•7.12, 10.5]

$\rho_{\min} = \frac{200}{f_y}$ or 0.0018	
$A_{s,\min} = \rho_{\min} \ell d$	(ℓ is substituted for the ACI-318 symbol b for the concrete beam width and is consistent with the footing dimensioning in Figure 4.2)

Designers often specify one or two longitudinal No. 4 bars for wall footings as nominal reinforcement in the case of questionable soils or when required to maintain continuity of stepped footings on sloped sites or under conditions resulting in a changed footing depth. However, for most residential foundations, the primary resistance against differential settlement is provided by the deep beam action of the foundation wall; footing reinforcement may provide



limited benefit. In such cases, the footing simply acts as a platform for the wall construction and distributes loads to a larger soil bearing area.

Lap Splices

Where reinforcement cannot be installed in one length to meet reinforcement requirements, as in continuous wall footings, reinforcement bars must be lapped to develop the bars' full tensile capacity across the splice. In accordance with ACI-318, a minimum lap length of 40 times the diameter of the reinforcement bar is required for splices in the reinforcement. In addition, the separation between spliced or lapped bars is not to exceed eight times the diameter of the reinforcement bar or 6 inches, whichever is less.

4.5 Foundation Walls

The objectives of foundation wall design are

- to transfer the load of the building to the footing or directly to the earth;
- to provide adequate strength, in combination with the footing when required, to prevent differential settlement;
- to provide adequate resistance to shear and bending stresses resulting from lateral soil pressure;
- to provide anchorage for the above-grade structure to resist wind or seismic forces;
- to provide a moisture-resistant barrier to below-ground habitable space in accordance with the building code; and
- to isolate nonmoisture-resistant building materials from the ground.

In some cases, masonry or concrete foundation walls incorporate a nominal amount of steel reinforcement to control cracking. Engineering specifications generally require reinforcement of concrete or masonry foundation walls because of somewhat arbitrary limits on minimum steel-to-concrete ratios, even for “plain” concrete walls. However, residential foundation walls are generally constructed of unreinforced or nominally reinforced concrete or masonry or of preservative-treated wood. The nominal reinforcement approach has provided many serviceable structures. This section discusses the issue of reinforcement and presents rational design approach for residential concrete and masonry foundation walls.

In most cases, a design for concrete or concrete masonry walls can be selected from the prescriptive tables in the applicable residential building code or the *International One- and Two-Family Dwelling Code* (ICC, 1998). Sometimes, a specific design applied with reasonable engineering judgment results in a more efficient and economical solution than that prescribed by the codes. The designer may elect to design the wall as either a reinforced or plain concrete wall. The following sections detail design methods for both wall types.



4.5.1 Concrete Foundation Walls

Regardless of the type of concrete foundation wall selected, the designer needs to determine the nominal and factored loads that in turn govern the type of wall (i.e., reinforced or unreinforced) that may be appropriate for a given application. Based on Table 3.1 of Chapter 3, the following LRFD load combinations are suggested for the design of residential concrete foundation walls:

- $1.2 D + 1.6 H$
- $1.2 D + 1.6 H + 1.6 L + 0.5 (L_r \text{ or } S)$
- $1.2 D + 1.6 H + 1.6 (L_r \text{ or } S) + 0.5 L$

In light-frame homes, the first load combination typically governs foundation wall design. Axial load increases moment capacity of concrete walls when they are not appreciably eccentric, as is the case in typical residential construction.

To simplify the calculations further, the designer may conservatively assume that the foundation wall acts as a simple span beam with pinned ends, although such an assumption will tend to overpredict the stresses in the wall. In any event, the simple span model requires the wall to be adequately supported at its top by the connection to the floor framing and at its base by the connection to the footing or bearing against a basement floor slab. Appendix A contains basic load diagrams and beam equations to assist the designer in analyzing typical loading conditions and element-based structural actions encountered in residential design. Once the loads are known, the designer can perform design checks for various stresses by following ACI-318 and the recommendations contained herein.

As a practical consideration, residential designers need to keep in mind that concrete foundation walls are typically 6, 8, or 10 inches thick (nominal). The typical concrete compressive strength used in residential construction is 2,500 or 3,000 psi, although other strengths are available. Typical reinforcement tensile yield strength is 60,000 psi (Grade 60) and is primarily a matter of market supply. Refer to Section 4.2.1 for more information on concrete and steel reinforcement material properties.

4.5.1.1 Plain Concrete Wall Design

ACI-318 allows the design of plain concrete walls with some limits as discussed in ACI-318•22.0. ACI-318 recommends the incorporation of contraction and isolation joints to control cracking; however, this is not a typical practice for residential foundation walls and temperature and shrinkage cracking is practically unavoidable. It is considered to have a negligible impact on the structural integrity of a residential wall. However, cracking may be controlled (i.e., minimize potential crack widening) by reasonable use of horizontal reinforcement.

ACI-318 limits plain concrete wall thickness to a minimum of 7.5 inches; however, the *International One- and Two-Family Dwelling Code* (ICC, 1998)



permits nominal 6-inch-thick foundation walls when the height of unbalanced fill is less than a prescribed maximum. The 7.5-inch-minimum thickness requirement is obviously impractical for a short concrete stem wall as in a crawl space foundation.

Adequate strength needs to be provided and should be demonstrated by analysis in accordance with the ACI-318 design equations and the recommendations of this section. Depending on soil loads, analysis should confirm conventional residential foundation wall practice in typical conditions. Refer to Example 4.3 of Section 4.9 for an illustration of a plain concrete foundation wall design.

The following checks are used to determine if a plain concrete wall has adequate strength.

Shear Capacity

Shear stress is a result of the lateral loads on a structure associated with wind, earthquake, or backfill forces. Lateral loads are, however, either normal to the wall surface (i.e., perpendicular or out of plane) or parallel to the wall surface (i.e., in plane). The designer must consider both perpendicular and parallel shear in the wall.

Perpendicular shear is rarely a controlling factor in the design of residential concrete foundation walls. Parallel shear is also usually not a controlling factor in residential foundation walls.

If greater shear capacity is required in a plain concrete wall, it may be obtained by increasing the wall thickness or increasing the concrete compressive strength. Alternatively, a wall can be reinforced in accordance with Section 4.5.1.2.

The following equations apply to both perpendicular and parallel shear in conjunction with Figure 4.3 for plain concrete walls. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.

[ACI-318•22.5.4]

$$V_u \leq \phi V_n$$

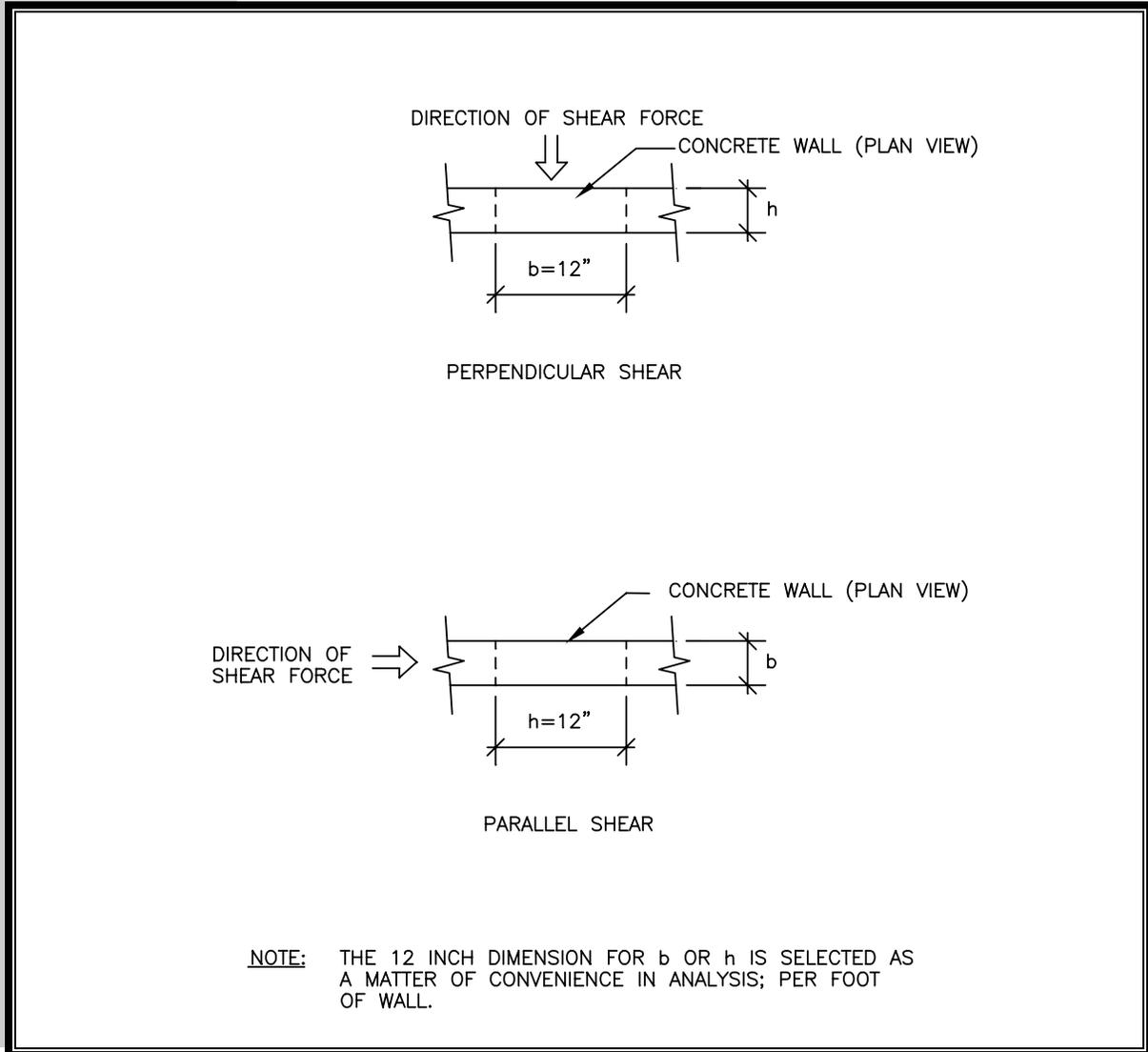
V_u = maximum factored shear load on the wall

$$\phi V_n = \phi \frac{4}{3} \sqrt{f'_c} bh$$

$$\phi = 0.65$$



FIGURE 4.3 *Variables Defined for Shear Calculations in Plain Concrete Walls*



Combined Axial and Bending Capacity

The ACI-318 equations listed below account for the combined effects of axial load and bending moment on a plain concrete wall. The intent is to ensure that the concrete face in compression and the concrete face in tension resulting from factored nominal axial and bending loads do not exceed the factored nominal capacity for concrete. A method of plotting the interaction equation below is shown in Example 4.4 of Section 4.9; refer to Section 4.5.1.3 for information on interaction diagrams.

[ACI-318•22.5.3, 22.6.3]

$$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \leq 1 \text{ on the compression face}$$



$$\frac{M_u}{S} - \frac{P_u}{A_g} \leq 5\phi\sqrt{f'_c}, \text{ on the tension face}$$

$$M_u > M_{u,\min}$$

M_u = maximum factored nominal moment on wall

$$M_{u,\min} = 0.1hP_u$$

$$M_n = 0.85f'_c S$$

$$P_n = 0.6f'_c \left[1 - \left(\frac{l_c}{32h} \right)^2 \right] A_g$$

P_u = factored nominal axial load on the wall at point of maximum moment

$$\phi = 0.65$$

Even though a plain concrete wall often calculates as adequate, the designer may elect to add a nominal amount of reinforcement for crack control or other reasons. Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness or increased concrete compressive strength. Alternatively, the wall may be reinforced in accordance with Section 4.5.1.2. Walls determined to have adequate strength to withstand shear and combined axial load and bending moment may also be checked for deflection, but this is usually not a limiting factor for typical residential foundation walls.

4.5.1.2 Reinforced Concrete Design

ACI-318 allows two approaches to the design of reinforced concrete with some limits on wall thickness and the minimum amount of steel reinforcement; however, ACI-318 also permits these requirements to be waived in the event that structural analysis demonstrates adequate strength and stability in accordance with ACI-318•14.2.7. Refer to Examples 4.5, 4.6, and 4.7 in Section 4.9 for the design of a reinforced concrete foundation wall.

Reinforced concrete walls should be designed in accordance with ACI-318•14.4 by using the strength design method. The following checks for shear and combined flexure and axial load determine if a wall is adequate to resist the applied loads.

Shear Capacity

Shear stress is a result of the lateral loads on a structure associated with wind, earthquake, or lateral soil forces. The loads are, however, either normal to the wall surface (i.e., perpendicular or out of plane) or parallel to the wall surface (i.e., in plane). The designer must check both perpendicular and parallel shear in the wall to determine if the wall can resist the lateral loads present.

Perpendicular shear is rarely a controlling factor in the design of typical residential foundation concrete walls. The level of parallel shear is also usually not a controlling factor in residential foundation walls.

If greater shear capacity is required, it may be obtained by increasing the wall thickness, increasing the concrete compressive strength, adding horizontal



shear reinforcement, or installing vertical reinforcement to resist shear through shear friction. Shear friction is the transfer of shear through friction between two faces of a crack. Shear friction also relies on resistance from protruding portions of concrete on either side of the crack and by dowel action of the reinforcement that crosses the crack. The maximum limit on reinforcement spacing of 12 or 24 inches specified in ACI-318•11.5.4 is considered to be an arbitrary limit. When reinforcement is required, 48 inches as an adequate maximum spacing for residential foundation wall design agrees with practical experience.

The following equations provide checks for both perpendicular and parallel shear in conjunction with Figure 4.4. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.

[ACI-318•11.5,11.7, 11.10]

$$V_u \leq \phi V_n$$

V_u = maximum factored shear load on wall

$$V_n = V_c + V_s$$

$$V_c = 2\sqrt{f'_c} b_w d$$

$$V_s = \frac{A_v f_y d}{s} \leq 8\sqrt{f'_c} b_w d \quad \text{when } V_u > \phi V_c$$

$$\phi = 0.85$$

Shear-Friction Method

$$V_u \leq \phi V_n$$

$$V_n = A_{vf} f_y \mu \leq 0.2f'_c A_c \quad \text{and} \quad \leq 800A_c$$

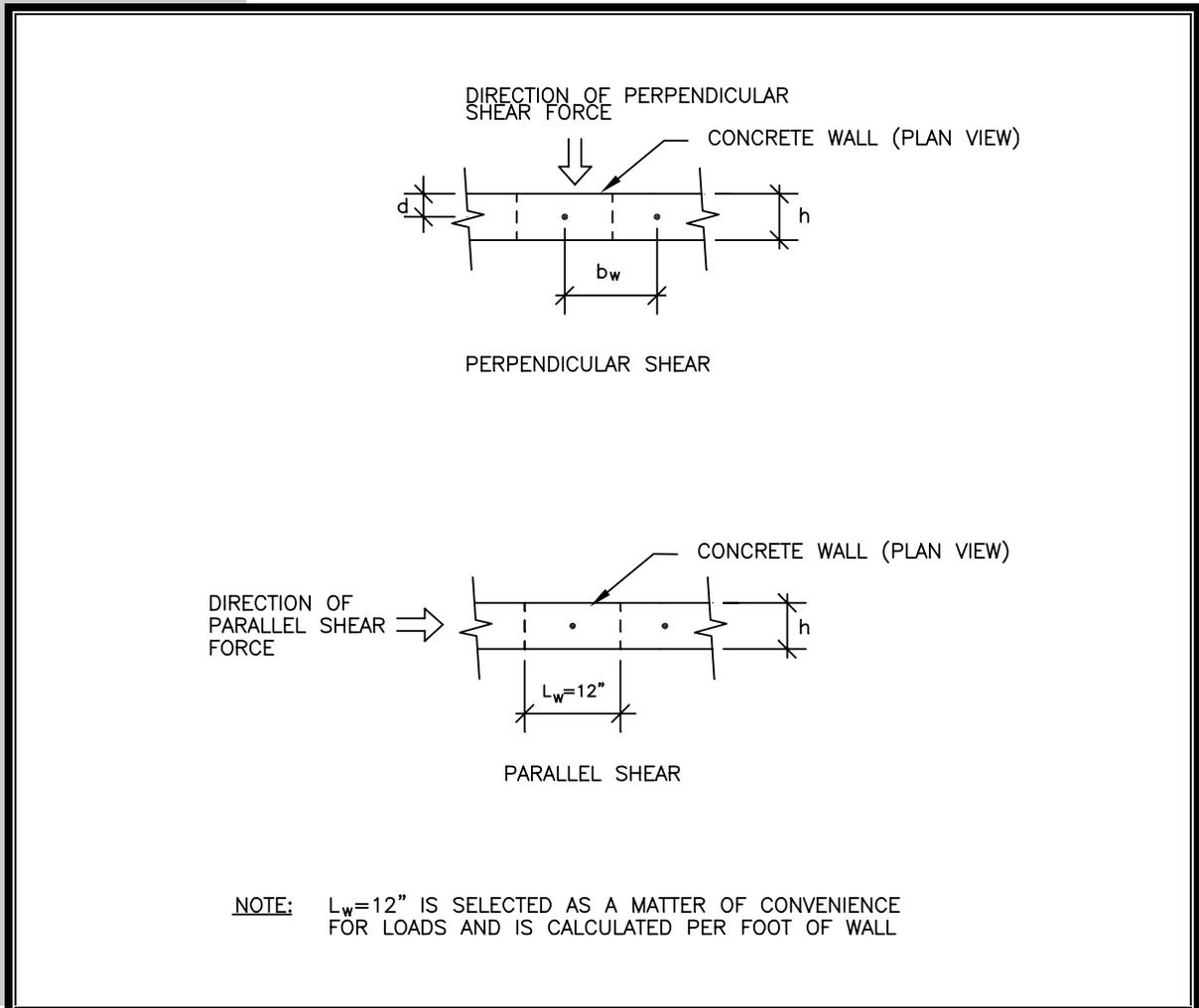
$$A_c = b_w h$$

$$\phi = 0.85$$



FIGURE 4.4

Variables Defined for Shear Calculations in Reinforced Concrete Walls



Combined Flexural and Axial Load Capacity

ACI-318 prescribes reinforcement requirements for concrete walls. Foundation walls commonly resist both an applied axial load from the structure above and an applied lateral soil load from backfill. To ensure that the wall's strength is sufficient, the designer must first determine slenderness effects (i.e., Euler buckling) in the wall. ACI-318•10.10 provides an approximation method to account for slenderness effects in the wall; however, the slenderness ratio must not be greater than 100. The slenderness ratio is defined in the following section as the ratio between unsupported length and the radius of gyration. In residential construction, the approximation method, more commonly known as the moment magnifier method, is usually adequate because slenderness ratios are typically less than 100 in foundation walls.

The moment magnifier method is based on the wall's classification as a "sway frame" or "nonsway frame." In concept, a sway frame is a frame (i.e., columns and beams) as opposed to a concrete bearing wall system. Sway frames are not discussed in detail herein because the soil pressures surrounding a



residential foundation typically provide lateral support to resist any racking and deflections associated with a sway frame. More important, foundation walls generally have few openings and thus do not constitute a framelike system. For more information on sway frames and their design procedure, refer to ACI-318•10.13.

The moment magnifier method uses the relationship of the axial load and lateral load in addition to wall thickness and unbraced height to determine a multiplier of 1 or greater, which accounts for slenderness in the wall. The multiplier is termed the moment magnifier. It magnifies the calculated moment in the wall resulting from the lateral soil load and any eccentricity in axial load. Together, the axial load and magnified moment are used to determine whether the foundation wall section is adequate to resist the applied loads. The following steps are required to determine the amount of reinforcement required in a typical residential concrete foundation wall to resist combined flexure and axial loads:

- calculate axial and lateral loads;
- verify that the nonsway condition applies;
- calculate slenderness;
- calculate the moment magnifier; and
- plot the axial load and magnified moment on an interaction diagram.

The following sections discuss the procedure in detail.

Slenderness

Conservatively, assuming that the wall is pinned at the top and bottom, slenderness in the wall can be calculated by using the equation below. The effective length factor k is conservatively assumed to equal 1 in this condition. It should be noted that a value of k much less than 1 (i.e., 0.7) may actually better represent the end conditions (i.e., nonpinned) of residential foundation walls.

[ACI-318•10.10]

$$\frac{kl_u}{r} < 34 \quad \text{slenderness ratio}$$
$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{bd^3/12}{bd}} = \sqrt{\frac{d^2}{12}} \quad \text{radius of gyration}$$

Moment Magnifier Method

The moment magnifier method is an approximation method allowed in ACI-318•10.10 for concrete walls with a slenderness ratio less than or equal to 100. If the slenderness ratio is less than 34, then the moment magnifier is equal to 1 and requires no additional analysis. The design procedure and equations below follow ACI-318•10.12. The equation for EI , as listed in ACI-318, is applicable to walls containing a double layer of steel reinforcement. Residential walls typically contain only one layer of steel reinforcement; therefore, the equation for EI , as listed herein, is based on Section 10.12 (ACI, 1996).



[ACI-318•10.12.3]

$$M_{u,mag} = \delta M_u \leftarrow \text{Magnified Moment}$$

$$\delta = \frac{C_m}{1 - \left(\frac{P_u}{0.75P_c} \right)} \geq 1$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

$$C_m = 0.6$$

or

$C_m = 1$ for members with transverse loads between supports

$$M_{u,min} = P_u (0.6 + 0.03h)$$

$$EI = \frac{0.4E_c I_g}{\beta} \geq \frac{E_c I_g (0.5 - e/h)}{\beta} \geq \frac{0.1E_c I_g}{\beta}$$

$$e = \frac{M_2}{P_u}$$

$$\beta = 0.9 + 0.5\beta_d^2 - 12\rho \geq 1.0$$

$$\rho = \frac{A_s}{A_g}$$

$$\beta_d = \frac{P_{u,dead}}{P_u}$$

$$E_c = 57,000\sqrt{f'_c} \text{ or } w_c^{1.5} 33\sqrt{f'_c}$$

Given that the total factored axial load in residential construction typically falls below 3,000 pounds per linear foot of wall and that concrete compressive strength is typically 3,000 psi, Table 4.4 provides prescriptive moment magnifiers. Interpolation is permitted between wall heights and between factored axial loads. Depending on the reinforcement ratio and the eccentricity present, some economy is lost in using the Table 4.4 values instead of the above calculation method.

TABLE 4.4 *Simplified Moment Magnification Factors, δ_{ns}*

Minimum Wall Thickness (inches)	Maximum Wall Height (feet)	Factored Axial Load (plf)	
		2,000	4,000
5.5	8	1.07	1.15
	10	1.12	1.26
7.5	8	1.03	1.06
	10	1.04	1.09
9.5	8	1.00	1.03
	10	1.00	1.04



Example 4.6 in Section 4.9 presents the complete design of a reinforced concrete foundation wall. The magnified moment and corresponding total factored axial load are plotted on an interaction diagram as shown in Example 4.7. Refer to Section 4.5.1.3 for a description of interaction diagrams and additional resources.

4.5.1.3 Interaction Diagrams

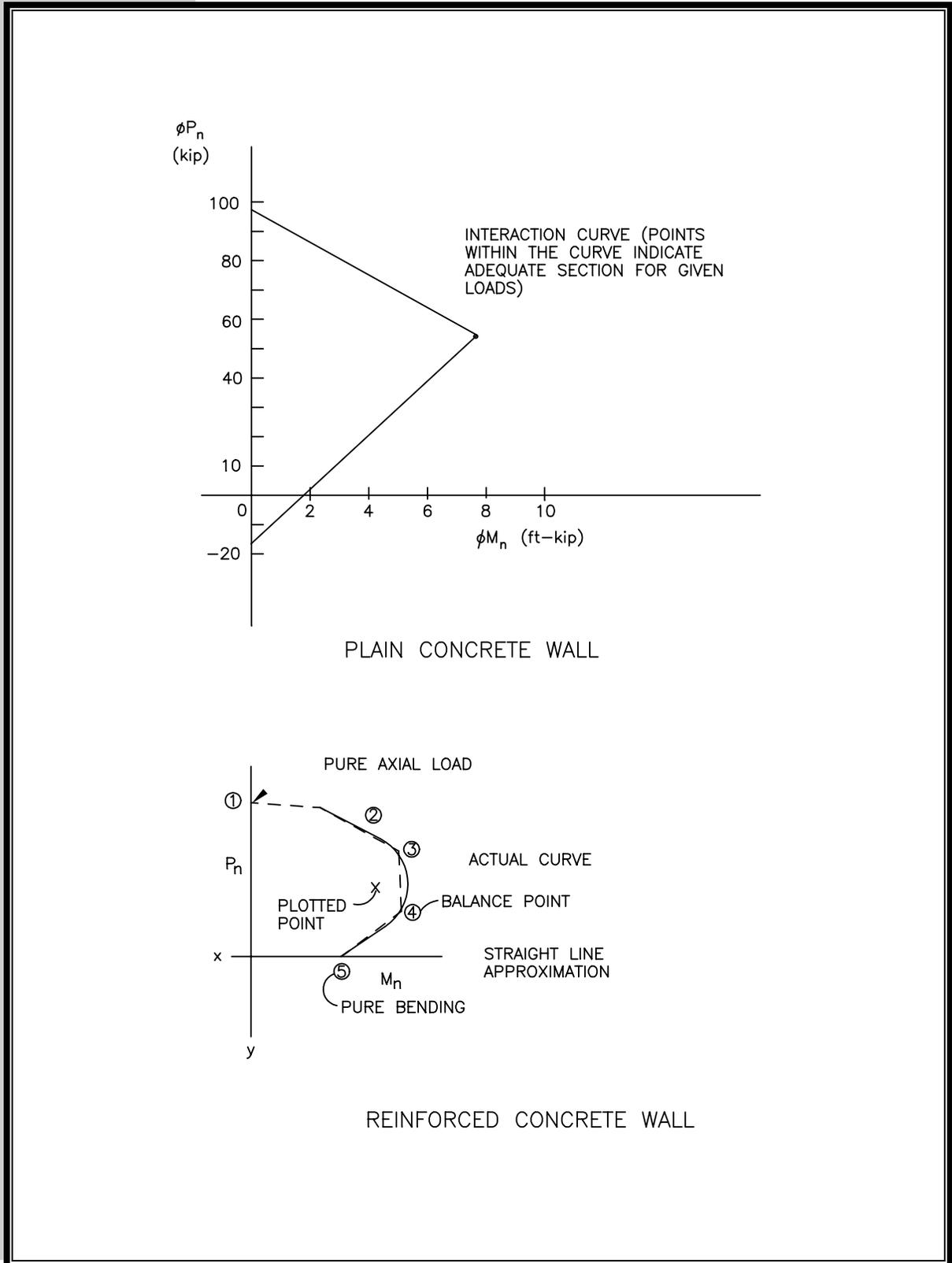
An interaction diagram is a graphic representation of the relationship between the axial load and bending capacity of a reinforced or plain concrete wall. The primary use of interaction diagrams is as a design aid for selecting predetermined concrete wall or column designs for varying loading conditions. Several publications provide interaction diagrams for use with concrete. These publications, however, typically focus on column or wall design that is heavily reinforced in accordance with design loads common in commercial construction. Residential concrete walls are either plain or slightly reinforced with one layer of reinforcement typically placed near the center of the wall. Plain and reinforced concrete interaction diagrams for residential applications and the methods for deriving them may be found in *Structural Design of Insulating Concrete Form Walls in Residential Construction* (PCA, 1998). PCA also offers a computer program that plots interaction diagrams based on user input; the program is entitled *PCA Column* (PCACOL).

An interaction diagram assists the designer in determining the wall's structural adequacy at various loading conditions (i.e., combinations of axial and bending loads). Figure 4.5 illustrates interaction diagrams for plain and reinforced concrete. Both the design points located within the interaction curve for a given wall height and the reference axes represent a combination of axial load and bending moment that the wall can safely support. The most efficient design is close to the interaction diagram curve. For residential applications, the designer, realizing that the overall design process is not exact, usually accepts designs within plus or minus 5 percent of the interaction curve.



FIGURE 4.5

Typical Interaction Diagrams for Plain and Reinforced Concrete Walls





4.5.1.4 Minimum Concrete Wall Reinforcement

Plain concrete foundation walls provide serviceable structures when they are adequately designed (see Section 4.5.1.1). However, when reinforcement is used to provide additional strength in thinner walls or to address more heavily loaded conditions, tests have shown that horizontal and vertical wall reinforcement spacing limited to a maximum of 48 inches on center results in performance that agrees reasonably well with design expectations (Roller, 1996).

ACI-318•22.6.6.5 requires two No. 5 bars around all wall openings. As an alternative more suitable to residential construction, a minimum of one rebar should be placed on each side of openings between 2 and 4 feet wide and two rebars on each side and one on the bottom of openings greater than 4 feet wide. The rebar should be the same size required by the design of the reinforced wall or a minimum No. 4 for plain concrete walls. In addition, a lintel (i.e., concrete beam) is required at the top of wall openings; refer to Section 4.5.1.6 for more detail on lintels.

4.5.1.5 Concrete Wall Deflection

ACI-318 does not specifically limit wall deflection. Therefore, deflection is usually not analyzed in residential foundation wall design. Regardless, a deflection limit of $L/240$ for unfactored soil loads is not unreasonable for below-grade walls.

When using the moment magnifier method, the designer is advised to apply the calculated moment magnification factor to the unfactored load moments used in conducting the deflection calculations. The calculation of wall deflection should also use effective section properties based on $E_c I_g$ for plain concrete walls and $E_c I_e$ for reinforced concrete walls; refer to ACI 318•9.5.2.3 to calculate the effective moment of inertia, I_e .

If unfactored load deflections prove unacceptable, the designer may increase the wall thickness or the amount of vertical wall reinforcement. For most residential loading conditions, however, satisfying reasonable deflection requirements should not be a limiting condition.

4.5.1.6 Concrete Wall Lintels

Openings in concrete walls are constructed with concrete, steel, precast concrete, cast stone, or reinforced masonry wall lintels. Wood headers are also used when not supporting concrete construction above and when continuity at the top of the wall (i.e., bond beam) is not critical, as in high-hazard seismic or hurricane coastal zones, or is maintained sufficiently by a wood sill plate and other construction above.

This section focuses on the design of concrete lintels in accordance with Chapters 10 and 11 of ACI-318. The concrete lintel is often assumed to act as a simple span with each end pinned. However, the assumption implies no top reinforcement to transfer the moment developed at the end of the lintel. Under that condition, the lintel is assumed to be cracked at the ends such that the end

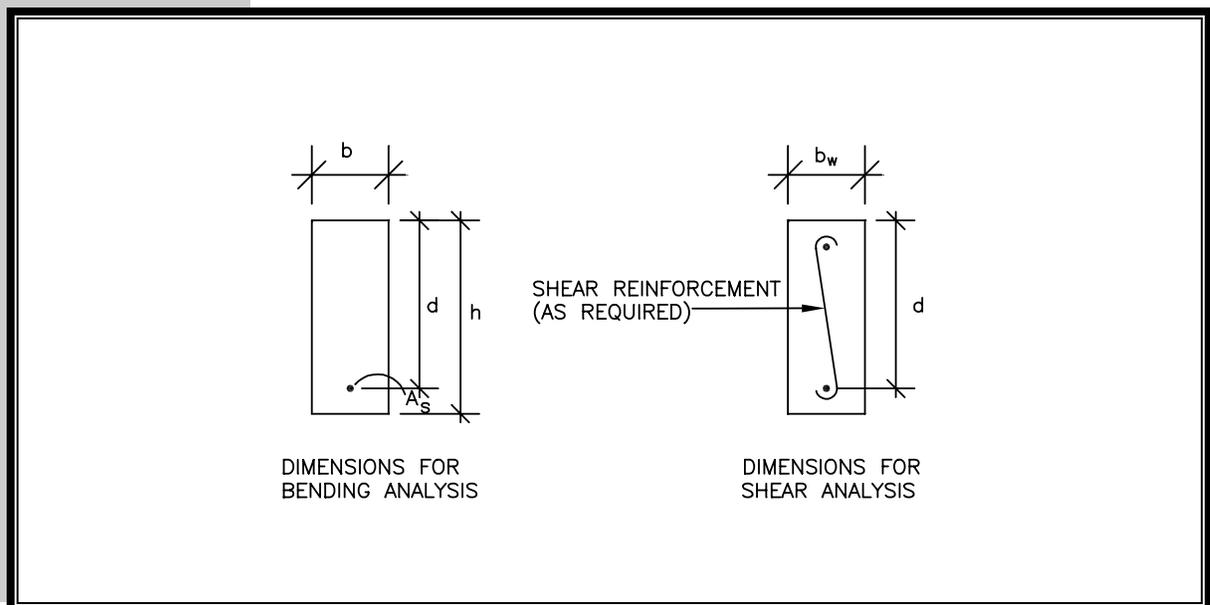


moment is zero and the shear must be transferred from the lintel to the wall through the bottom reinforcement.

If the lintel is assumed to act as a fixed-end beam, sufficient embedment of the top and bottom reinforcement beyond each side of the opening should be provided to fully develop a moment-resisting end in the lintel. Though more complicated to design and construct, a fixed-end beam reduces the maximum bending moment (i.e., $wl^2/12$ instead of $wl^2/8$) on the lintel and allows increased spans. A concrete lintel cast in a concrete wall acts somewhere between a true simple span beam and a fixed-end beam. Thus, a designer may design the bottom bar for a simple span condition and the top bar reinforcement for a fixed-end condition (conservative). Often, a No. 4 bar is placed at the top of each wall story to help tie the walls together (bond beam) which can also serve as the top reinforcement for concrete lintels. Figure 4.6 depicts the cross section and dimensions for analysis of concrete lintels. Example 4.8 demonstrates the design of a concrete lintel; refer to Section 4.9.

For additional information on concrete lintels and their design procedure, refer to the *Structural Design of Insulating Concrete Form Walls in Residential Construction* (PCA, 1998) and to *Testing and Design of Lintels Using Insulating Concrete Forms* (HUD, 2000). The latter, demonstrates through testing that shear reinforcement (i.e., stirrups) of concrete lintels is not necessary for short spans (i.e., 3 feet or less) with lintel depths of 8 inches or more. This research also indicates that the minimum reinforcement requirements in ACI-318 for beam design are conservative when a minimum #4 rebar is used as bottom reinforcement. Further, lintels with small span-to-depth ratios can be accurately designed as deep beams in accordance with ACI-318 when the minimum reinforcement ratios are met; refer to ACI-318•11.4.

FIGURE 4.6 *Design Variables Defined for Lintel Bending and Shear*





Flexural Capacity

The following equations are used to determine the flexural capacity of a reinforced concrete lintel in conjunction with Figure 4.6. An increase in the lintel depth or area of reinforcement is suggested if greater bending capacity is required. As a practical matter, though, lintel thickness is limited to the thickness of the wall in which a lintel is placed. In addition, lintel depth is often limited by the floor-to-floor height and the vertical placement of the opening in the wall. Therefore, in many cases, increasing the amount or size of reinforcement is the most practical and economical solution.

[ACI-318•10]

$$M_u \leq \phi M_n$$

$$M_u = \frac{w\ell^2}{12} \text{ for fixed-end beam model}$$

$$M_u = \frac{w\ell^2}{8} \text{ for simple span beam model}$$

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

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

$$\phi = 0.9$$

Shear Capacity

Concrete lintels are designed for shear resulting from wall, roof, and floor loads in accordance with the equations below and Figure 4.6.

[ACI-318•11]

$$V_u \leq \phi V_n$$

$$V_n = V_c + V_s$$

$$V_c = 2\sqrt{f'_c} b_w d$$

$$V_s = \frac{A_v f_y d}{s} \leq 8\sqrt{f'_c} b_w d \text{ when } V_u > \phi V_c$$

$$A_{v,\min} = \frac{50b_w s}{f_y} \text{ when } V_u > \frac{\phi V_c}{2}$$

$$s \leq \text{minimum of } \left\{ \frac{d}{2} \text{ or } 24 \text{ in} \right\}$$

$$s \leq \text{minimum of } \left\{ \frac{d}{4} \text{ or } 12 \text{ in} \right\} \text{ when } V_s > 4\sqrt{f'_c} b_w d$$

$$\phi = 0.85$$

Check Concrete Lintel Deflection

ACI-318 does not specifically limit lintel deflection. Therefore, a reasonable deflection limit of $L/240$ for unfactored live loads is suggested. The selection of an appropriate deflection limit, however, is subject to designer



discretion. In some applications, a lintel deflection limit of $L/180$ with live and dead loads is adequate. A primary consideration is whether lintel is able to move independently of door and window frames. Calculation of lintel deflection should use unfactored loads and the effective section properties $E_c I_e$ of the assumed concrete section; refer to ACI-318•9.5.2.3 to calculate the effective moment of inertia I_e of the section.

4.5.2 Masonry Foundation Walls

Masonry foundation wall construction is common in residential construction. It is used in a variety of foundation types, including basements, crawl spaces, and slabs on grade. For prescriptive design of masonry foundation walls in typical residential applications, a designer or builder may use the *International One- and Two-Family Dwelling Code* (ICC, 1998) or the local residential building code.

ACI-530 provides for the design of masonry foundation walls by using allowable stress design (ASD). Therefore, design loads may be determined according to load combinations presented in Chapter 3 as follows:

- $D + H$
- $D + H + L + 0.3 (L_r \text{ or } S)$
- $D + H + (L_r \text{ or } S) + 0.3 L$

In light-frame homes, the first load combination typically governs masonry walls for the same reasons stated in Section 4.5.1 for concrete foundation walls. To simplify the calculations, the designer may conservatively assume that the wall story acts as a simple span with pinned ends, although such an assumption may tend to overpredict the stresses in the wall. For a discussion on calculating the loads on a structure, refer to Chapter 3. Appendix A contains basic load diagrams and equations to assist the designer in calculating typical loading conditions and element-based structural actions encountered in residential design. Further, walls that are determined to have adequate strength to withstand shear and combined axial load and bending moment generally satisfy unspecified deflection requirements. Therefore, foundation wall deflection is not discussed in this section. However, if desired, deflection may be considered as discussed in Section 4.5.1.5 for concrete foundation walls.

To follow the design procedure, the designer needs to know the strength properties of various types and grades of masonry, mortar, and grout currently available on the market; Section 4.2.2 discusses the material properties. With the loads and material properties known, the designer can then perform design checks for various stresses by following ACI-530. Residential construction rarely involves detailed masonry specifications but rather makes use of standard materials and methods familiar to local suppliers and trades.

An engineer's inspection of a home is hardly ever required under typical residential construction conditions. Designers should be aware, however, that in jurisdictions covered by the *Uniform Building Code* (ICBO, 1997), lack of inspection on the jobsite requires reductions in the allowable stresses to account for potentially greater variability in material properties and workmanship. Indeed,



a higher level of inspection should be considered when masonry construction is specified in high-hazard seismic or severe hurricane areas. ACI-530 makes no distinction between inspected and noninspected masonry walls and therefore does not require adjustments in allowable stresses based on level of inspection.

As a residential designer, keep in mind that concrete masonry units (i.e., block) are readily available in nominal 6-, 8-, 10-, and 12-inch thicknesses. It is generally more economical if the masonry unit compressive strength f'_m ranges between 1,500 and 3,000 psi. The standard block used in residential and light commercial construction is usually rated at 1,900 psi.

4.5.2.1 Unreinforced Masonry Design

ACI-530 addresses the design of unreinforced masonry to ensure that unit stresses and flexural stresses in the wall do not exceed certain maximum allowable stresses. It provides for two methods of design: an empirical design approach and an allowable stress design approach.

Walls may be designed in accordance with ACI-530•5 by using the empirical design method under the following conditions:

- The building is not located in Seismic Design Category D or E as defined in NEHRP-97 or ASCE 7-98 (i.e., Seismic Zones 3 or 4 in most current and local building codes); refer to Chapter 3.
- Foundation walls do not exceed 8 feet in unsupported height.
- The length of the foundation walls between perpendicular masonry walls or pilasters is a maximum of 3 times the basement wall height. This limit typically does not apply to residential basements as required in the *International One- and Two-Family Dwelling Code* (ICC, 1998) and other similar residential building codes.
- Compressive stresses do not exceed the allowable stresses listed in ACI-530; compressive stresses are determined by dividing the design load by the gross cross-sectional area of the unit per ACI-530•5.4.2.
- Backfill heights do not exceed those listed in Table 4.5.
- Backfill material is nonexpansive and is tamped no more than necessary to prevent excessive settlement.
- Masonry is laid in running bond with Type M or S mortar.
- Lateral support is provided at the top of the foundation wall before backfilling.

Drainage is important when using the empirical table because lack of good drainage may substantially increase the lateral load on the foundation wall if the soil becomes saturated. As required in standard practice, the finish grade around the structure should be adequately sloped to drain surface water away from the foundation walls. The backfill material should also be drained to remove ground water from poorly drained soils.

Wood floor framing typically provides lateral support to the top of masonry foundation walls and therefore should be adequately connected to the masonry in accordance with one of several options. The most common method of



connection calls for a wood sill plate, anchor bolts, and nailing of the floor framing to the sill plate (see Chapter 7).

When the limits of the empirical design method are exceeded, the allowable stress design procedure for unreinforced masonry, as detailed below, provides a more flexible approach by which walls are designed as compression and bending members in accordance with ACI-530•2.2.

TABLE 4.5

Nominal Wall Thickness for 8-Foot-High Masonry Foundation Walls^{1,2}

Nominal Wall Thickness	Maximum Unbalanced Backfill Height		
	Hollow Unit Masonry	Solid Unit Masonry	Fully Grouted Unit Masonry
6 inches	3	5	5
8 inches	5	5	7
10 inches	6	7	8
12 inches	7	7	8

Source: Modified from the ACI-530•9.6 by using the International One-and Two-Family Dwelling Code (ICC, 1998).

Notes:

¹Based on a backfill with an assumed equivalent fluid density of 30 pcf.

²Backfill height is measured from the top of the basement slab to the finished exterior grade; wall height is measured from the top of the basement slab to the top of the wall.

Walls may be designed in accordance with ACI-530•2.2 by using the allowable stress design method. The fundamental assumptions, derivation of formulas, and design procedures are similar to those developed for strength-based design for concrete except that the material properties of masonry are substituted for those of concrete. Allowable masonry stresses used in allowable stress design are expressed in terms of a fraction of the specified compressive strength of the masonry at the age of 28 days, f'_m . A typical fraction of the specified compressive strength is 0.25 or 0.33, which equates to a conservative safety factor between 3 and 4 relative to the minimum specified masonry compressive strength. Design values for flexural tension stress are given in Table 4.6. The following design checks are used to determine if an unreinforced masonry wall is structurally adequate (refer to Example 4.9 for the design of an unreinforced concrete masonry wall).



TABLE 4.6 *Allowable Flexural Tension Stresses F_a for Allowable Stress Design of Unreinforced Masonry*

Type of Masonry Unit Construction	Mortar Type M or S	
	Portland Cement/Lime (psi)	Masonry Cement and Air-Entrained Portland Cement/Lime (psi)
Normal to Bed Joints		
Solid	40	24
Hollow ¹		
UngROUTED	25	15
Fully grouted	68	41
Parallel to Bed Joints in Running Bond		
Solid	80	48
Hollow		
UngROUTED/partially grouted	50	30
Fully grouted	80	48

Source: Table 6.3.1.1 in ACI-530-6.0.

Note:

¹For partially grouted masonry, allowable stresses may be determined on the basis of linear interpolation between fully grouted and ungrouted hollow units based on the amount of grouting.

Shear Capacity

Shear stress is a result of the lateral loads on the structure associated with wind, earthquakes, or backfill forces. Lateral loads are both normal to the wall surface (i.e., perpendicular or out of plane) and parallel to the wall surface (i.e., parallel or in plane). Both perpendicular and parallel shear should be checked; however, neither perpendicular nor parallel shear is usually a controlling factor in residential foundation walls.

If greater perpendicular shear capacity is required, it may be obtained by increasing the wall thickness, increasing the masonry unit compressive strength, or adding vertical reinforcement in grouted cells. If greater parallel shear capacity is required, it may be obtained by increasing the wall thickness, reducing the size or number of wall openings, or adding horizontal joint reinforcement. Horizontal truss-type joint reinforcement can substantially increase parallel shear capacity, provided that it is installed properly in the horizontal mortar bed joints. If not installed properly, it can create a place of weakness in the wall, particularly in out-of-plane bending of an unreinforced masonry wall.

The equations below are used to check perpendicular and parallel shear in masonry walls. The variable N_v is the axial design load acting on the wall at the point of maximum shear. The equations are based on A_n , which is the net cross-sectional area of the masonry. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.



[ACI-530•2.2.5]

$$f_v \leq F_v$$

$$f_v = \frac{3V}{2A_n}$$

$$F_v = \text{minimum of } \begin{cases} 1.5\sqrt{f'_m} & \text{for axial and shear members} \\ 120\text{psi} \\ 37\text{psi} + 0.45 \frac{N_v}{A_n} & \text{for running bond} \end{cases}$$

Axial Compression Capacity

The following equations from ACI-530•2.3 are used to design masonry walls and columns for compressive loads only. They are based on the net cross-sectional area of the masonry, including grouted and mortared areas.

[ACI-530•2.3]

Columns

$$P \leq P_a$$

$$P_a = (0.25f'_m A_n + 0.65A_{st} F_s) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad \text{where } h/r \leq 99$$

$$P_a = (0.25f'_m A_n + 0.65A_{st} F_s) \left(\frac{70r}{h} \right)^2 \quad \text{where } h/r > 99$$

$$P_{a,\text{maximum}} = F_a A_n$$

$$r = \sqrt{\frac{I}{A_n}}$$

Walls

$$f_a \leq F_a$$

$$F_a = (0.25f'_m) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad \text{where } h/r \leq 99$$

$$f_a = P/A$$

$$F_a = (0.25f'_m) \left(\frac{70r}{h} \right)^2 \quad \text{where } h/r > 99$$

$$r = \sqrt{\frac{I}{A_n}} \cong \frac{t}{\sqrt{12}}$$

$$P_e = \frac{\pi^2 E_m I}{h^2} \left(1 - 0.577 \frac{e}{r} \right)^3$$

$$P < 1/4 P_e$$

$$E_m = 900 F'_m$$

**Combined Axial Compression and Flexural Capacity**

The following equations from ACI-530 determine the relationship of the combined effects of axial load and bending moment on a masonry wall.

[ACI-530•2.3]

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$f_a = \frac{P}{A_n}$$

$$P \leq 0.25P_e$$

$$F_a = (0.25f'_m) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \text{ for } h/r \leq 99$$

$$F_a = (0.25f'_m) \left(\frac{70r}{h} \right)^2 \text{ for } h/r > 99$$

$$r = \sqrt{\frac{I}{A_n}}$$

$$f_b = \frac{M}{S}$$

$$F_b = 0.33f'_m$$

$$P_e = \frac{\pi^2 E_m I}{h^2} \left(1 - 0.577 \frac{e}{r} \right)^3$$

$$E_m = 900f'_m$$

$$f_t < F_t$$

$$F_t = \text{ACI-530 Table 2.2.3.2}$$

$$f_t = \frac{-P}{A_n} + \frac{M}{S}$$

Tension Capacity

ACI-530 provides allowable values for flexural tension transverse to the plane of a masonry wall. Standard principles of engineering mechanics determine the tension stress due to the bending moment caused by lateral (i.e., soil) loads and offset by axial loads (i.e., dead loads).

[ACI-530•2.3]

$$f_t < F_t$$

$$F_t = \text{ACI-530 Table 2.2.3.2}$$

$$f_t = \frac{P}{A_n} + \frac{M}{S}$$

Even though an unreinforced masonry wall may calculate as adequate, the designer may consider adding a nominal amount of reinforcement to control cracking (refer to Section 4.5.2.3 for a discussion on nominal reinforcement).

Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness, increased masonry compressive strength, or the addition of steel reinforcement.



Usually the most effective and economical solution for providing greater wall capacity in residential construction is to increase wall thickness, although reinforcement is also common. Section 4.5.2.2 discusses the design procedure for a reinforced masonry wall.

4.5.2.2 Reinforced Masonry Design

When unreinforced concrete masonry wall construction does not satisfy all design criteria (i.e., load, wall thickness limits, etc.), reinforced walls may be designed by following the allowable stress design procedure or the strength-based design procedure of ACI-530. The allowable stress design procedure outlined below describes an approach by which walls are designed in accordance with ACI-530•2.3. Although not discussed in detail herein, walls may also be designed by following the strength-based design method specified in ACI-530.

For walls designed in accordance with ACI-530•2.3 using the allowable stress design method, the fundamental assumptions, derivation of formulas, and design procedures are similar to those for design for concrete except that the material properties of masonry are substituted for those of concrete. Allowable masonry stresses used in allowable stress design are expressed in terms of a fraction of the specified compressive strength of the masonry at the age of 28 days, f'_m . A typical fraction of the specified compressive strength is 0.25, which equates to a conservative safety factor of 4. The following design checks determine if a reinforced masonry wall is structurally adequate (refer to Example 4.10 for the design of a reinforced concrete masonry wall).

Shear Capacity

Shear stress is a result of lateral loads on the structure associated with wind, earthquakes, or backfill forces. Lateral loads are both normal to the wall surface (i.e., perpendicular or out of plane) and parallel to the wall surface (i.e., parallel or in plane). Both perpendicular and parallel shear should be checked, however, perpendicular shear is rarely a controlling factor in the design of masonry walls and parallel shear is not usually a controlling factor unless the foundation is partially or fully above grade (i.e., walk-out basement) with a large number of openings.

The equations below check perpendicular and parallel shear in conjunction with Figure 4.7. Some building codes include a “j” coefficient in these equations. The “j” coefficient defines the distance between the center of the compression area and the center of the tensile steel area; however, it is often dismissed or approximated as 0.9. If greater parallel shear capacity is required, it may be obtained in a manner similar to that recommended in the previous section for unreinforced masonry design. For parallel shear, the equations do not address overturning and bending action that occurs in a direction parallel to the wall, particularly for short segments of walls under significant parallel shear load. For concrete foundation walls, this is generally not a concern. For above-grade wood-frame walls, this is addressed in Chapter 6 in detail.



[ACI-530•7.5]

$$f_v \leq F_v$$

$$f_v = \frac{V}{bd}$$

$$F_v = 1.0\sqrt{f'_m} \leq 50\text{psi} \quad \text{for flexural members}$$

$$F_v = \frac{1}{3} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m} \leq \left(80 - 45 \frac{M}{Vd} \right) \text{psi} \quad \text{for shear walls where } \frac{M}{Vd} < 1$$

$$F_v = 1.0\sqrt{f'_m} \leq 35\text{psi} \quad \text{for shear walls where } \frac{M}{Vd} \geq 1$$

If the shear stress exceeds the above allowables for masonry only, the designer must design shear reinforcing with the shear stress equation changes in accordance with ACI-530•2.3.5. In residential construction, it is generally more economical to increase the wall thickness or to grout additional cores instead of using shear reinforcement. If shear reinforcement is desired, refer to ACI-530. ACI-530 limits vertical reinforcement to a maximum spacing s of 48 inches; however, a maximum of 96 inches on-center is suggested as adequate. Masonry homes built with reinforcement at 96 inches on-center have performed well in hurricane-prone areas such as southern Florida.

Flexural or axial stresses must be accounted for to ensure that a wall is structurally sound. Axial loads increase compressive stresses and reduce tension stresses and may be great enough to keep the masonry in an uncracked state under a simultaneous bending load.

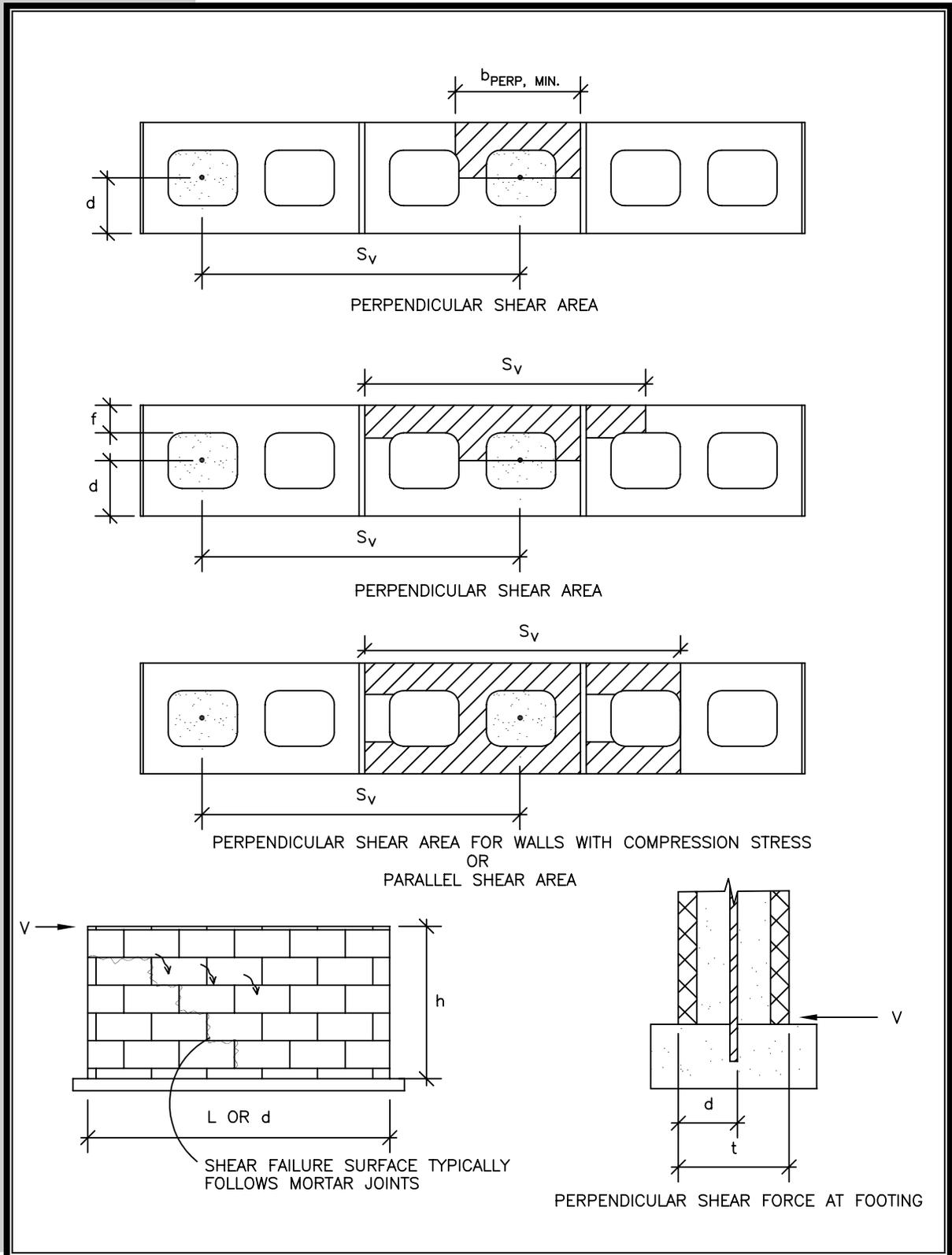
Axial Compression Capacity

The following equations from ACI-530•2.3 are used to determine if a masonry wall can withstand conditions when compressive loads act only on walls and columns (i.e., interior load-bearing wall or floor beam support pier). As with concrete, compressive capacity is usually not an issue in supporting a typical light-frame home. An exception may occur with the bearing points of long-spanning beams. In such a case, the designer should check bearing capacity by using ACI-530•2.1.7.



FIGURE 4.7

Variables Defined for Shear Calculations in Reinforced Concrete Masonry Walls





[ACI-530•2.3]

Columns

$$P \leq P_a$$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad \text{where } h/r \leq 99$$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s) \left(\frac{70r}{h} \right)^2 \quad \text{where } h/r > 99$$

$$P_{a,\text{maximum}} = F_a A_n$$

$$r = \sqrt{\frac{I}{A_e}}$$

Walls

$$f_a \leq F_a$$

$$F_a = (0.25f'_m) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad \text{where } h/r \leq 99$$

$$F_a = (0.25f'_m) \left(\frac{70r}{h} \right)^2 \quad \text{where } h/r > 99$$

$$r = \sqrt{\frac{I}{A_e}}$$

Calculation using the above equations is based on A_e , which is the effective cross-sectional area of the masonry, including grouted and mortared areas substituted for A_n .

Combined Axial Compression and Flexural Capacity

In accordance with ACI-530•2.3.2, the design tensile forces in the reinforcement due to flexure shall not exceed 20,000 psi for Grade 40 or 50 steel, 24,000 psi for Grade 60 steel, or 30,000 psi for wire joint reinforcement. As stated, most reinforcing steel in the U.S. market today is Grade 60. The following equations pertain to walls that are subject to combined axial and flexure stresses.

[ACI-530•7.3]

$$F_b = 0.33f'_m$$

$$f_b = \frac{M}{S} \leq \left(1 - \frac{f_a}{F_a} \right) F_b$$

Columns

$$\frac{P}{P_a} + \frac{f_b}{F_b} \leq 1$$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad \text{where } h/r \leq 99$$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s) \left(\frac{70r}{h} \right)^2 \quad \text{where } h/r > 99$$



Walls

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$f_a = \frac{P}{A_e} \leq 0.33f'_m \text{ due to flexure only or flexure in combination with axial load}$$

$$F_a = (0.25f'_m) \left[1 - \left(\frac{h}{140r} \right)^2 \right] \text{ for } h/r \leq 99$$

$$F_a = (0.25f'_m) \left(\frac{70r}{h} \right)^2 \text{ for } h/r > 99$$

Walls determined inadequate to withstand combined axial load and bending moment may gain greater capacity through increased wall thickness, increased masonry compressive strength, or added steel reinforcement.

4.5.2.3 Minimum Masonry Wall Reinforcement

Unreinforced concrete masonry walls have proven serviceable in millions of homes. Builders and designers may, however, wish to specify a nominal amount of reinforcement even when such reinforcement is not required by analysis. For example, it is not uncommon to specify horizontal reinforcement to control shrinkage cracking and to improve the bond between intersecting walls. When used, horizontal reinforcement is typically specified as a ladder or truss-type wire reinforcement. It is commonly installed continuously in mortar joints at vertical intervals of 24 inches (i.e., every third course of block).

For reinforced concrete masonry walls, ACI-530 stipulates minimum reinforcement limits as shown below; however, the limits are somewhat arbitrary and have no tangible basis as a minimum standard of care for residential design and construction. The designer should exercise reasonable judgment based on application conditions, experience in local practice, and local building code provisions for prescriptive masonry foundation or above-grade wall design in residential applications.

[ACI-530•2.3.5]

$$A_{s,\text{required}} = \frac{M}{F_s d}$$

$$A_{v,\text{min}} = 0.0013bt$$

$$A_{h,\text{min}} = 0.0007bt$$

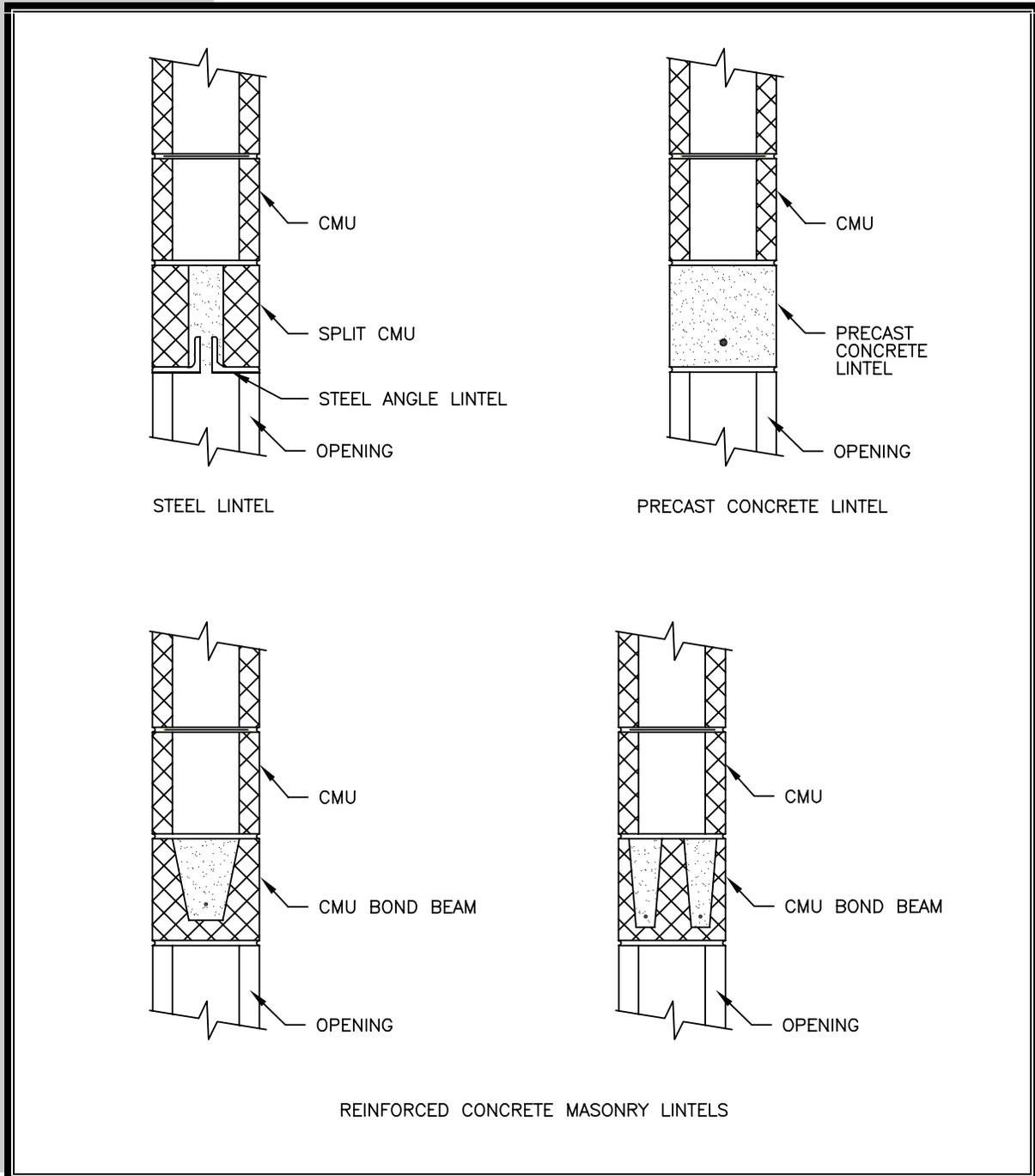
4.5.2.4 Masonry Wall Lintels

Openings in masonry walls are constructed by using steel, precast concrete, or reinforced masonry lintels. Wood headers are also used when they do not support masonry construction above and when continuity at the top of the wall (i.e., bond beam) is not required or is adequately provided within the system of wood-framed construction above. Steel angles are the simplest shapes and are



suitable for openings of moderate width typically found in residential foundation walls. The angle should have a horizontal leg of the same width as the thickness of the concrete masonry that it supports. Openings may require vertical reinforcing bars with a hooked end that is placed on each side of the opening to restrain the lintel against uplift forces in high-hazard wind or earthquake regions. Building codes typically require steel lintels exposed to the exterior to be a minimum 1/4-inch thick. Figure 4.8 illustrates some lintels commonly used in residential masonry construction.

FIGURE 4.8 Concrete Masonry Wall Lintel Types





Many prescriptive design tables are available for lintel design. For more information on lintels, arches, and their design, refer to the NCMA's TEK Notes; refer to contact information in Chapter 1. Information on lintels and arches can also be found in *Masonry Design and Detailing* (Beall, 1997).

4.5.3 Preservative-Treated Wood Foundation Walls

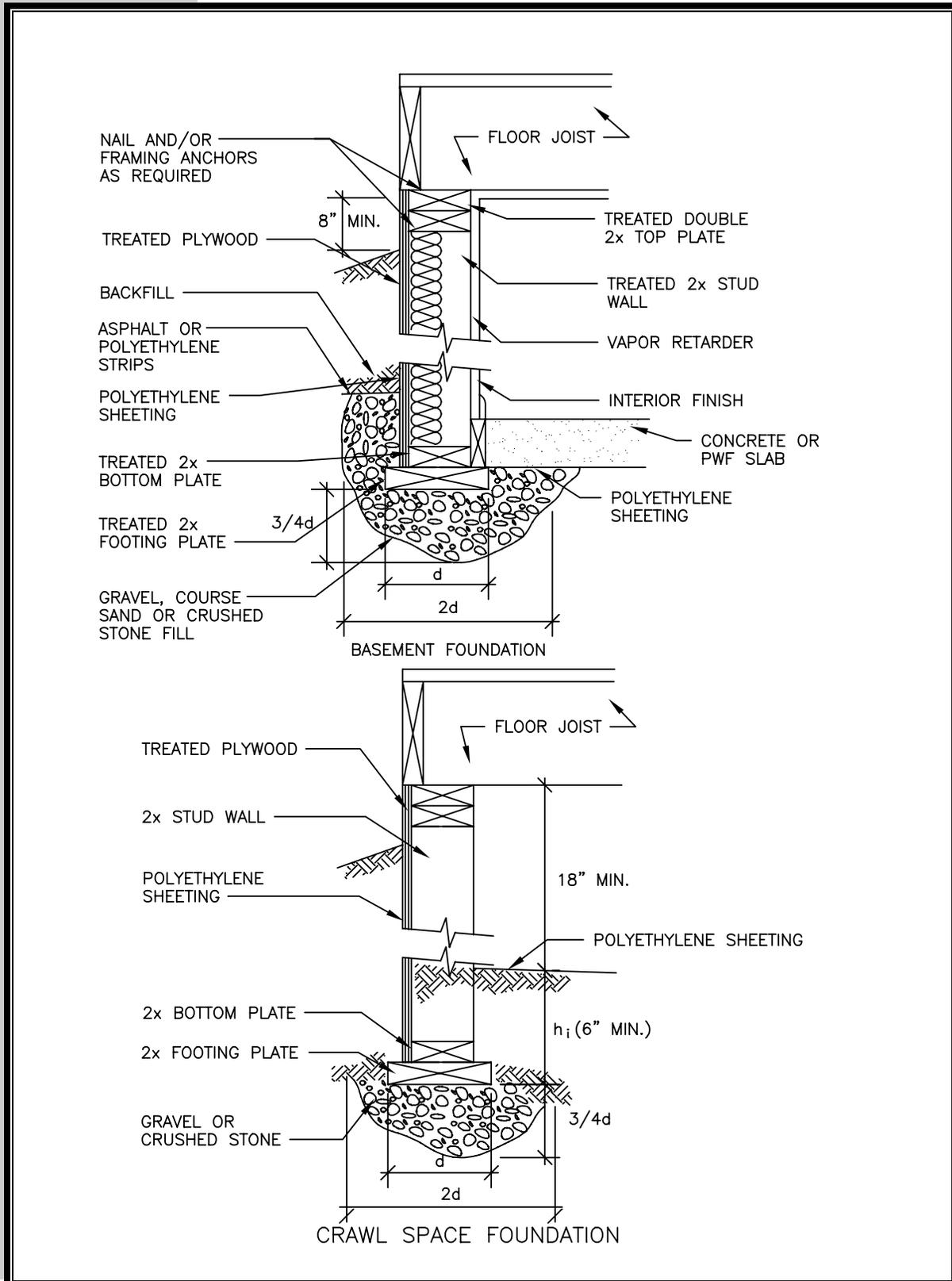
Preservative-treated wood foundations, commonly known as permanent wood foundations (PWF), have been used in over 300,000 homes and other structures throughout the United States. When properly installed, they provide foundation walls at an affordable cost. In some cases, the manufacturer may offer a 50-year material warranty, which exceeds the warranty offered for other common foundation materials.

A PWF is a load-bearing, preservative-treated, wood-framed foundation wall sheathed with preservative-treated plywood; it bears on a gravel spread footing. PWF lumber and plywood used in foundations is pressure treated with calcium chromium arsenate (CCA) to a minimum retention of 0.6 pcf. The walls are supported laterally at the top by the floor system and at the bottom by a cast-in-place concrete slab or pressure-treated lumber floor system or by backfill on the inside of the wall. Proper connection details are essential, along with provisions for drainage and moisture protection. All fasteners and hardware used in a PWF should be stainless steel or hot-dipped galvanized. Figure 4.9 illustrates a PWF.

PWFs may be designed in accordance with the basic provisions provided in the *International One- and Two-Family Dwelling Code* (ICC, 1998). Those provisions, in turn, are based on the Southern Forest Products Association's *Permanent Wood Foundations Design and Construction Guide* (SPC, 1998). The PWF guide offers design flexibility and thorough technical guidance. Table 4.7 summarizes some basic rules of thumb for design. The steps for using the prescriptive tables are outlined below.



FIGURE 4.9 *Preservative-Treated Wood Foundation Walls*



**TABLE 4.7** *Preservative-Treated Wood Foundation Framing¹*

Maximum Unbalanced Backfill Height (feet)	Nominal Stud Size	Stud Center-to-Center Spacing (inches)
5	2x6	16
6	2x6	12
8	2x8	12

- Connect each stud to top plate with framing anchors when the backfill height is 6 feet or greater.
- Provide full-depth blocking in the outer joist space along the foundation wall when floor joists are oriented parallel to the foundation wall.
- The bottom edge of the foundation studs should bear against a minimum of 2 inches of the perimeter screed board or the basement floor to resist shear forces from the backfill.

Note:

¹Connection of studs to plates and plates to floor framing is critical to the performance of PWFs. The building code and the *Permanent Wood Foundation Design and Construction Guide* (SPC, 1998) should be carefully consulted with respect to connections.

- Granular (i.e., gravel or crushed rock) footings are sized in accordance with Section 4.4.1. Permanent wood foundations may also be placed on poured concrete footings.
- Footing plate size is determined by the vertical load from the structure on the foundation wall and the size of the permanent wood foundation studs.
- The size and spacing of the wall framing is selected from tables for buildings up to 36 feet wide that support one or two stories above grade.
- APA-rated plywood is selected from tables based on unbalanced backfill height and stud spacing. The plywood must be preservatively treated and rated for below-ground application.
- Drainage systems are selected in accordance with foundation type (e.g., basement or crawl space) and soil type. Foundation wall moisture-proofing is also required (i.e., polyethylene sheeting).

For more information on preservative-treated wood foundations and their specific design and construction, consult the *Permanent Wood Foundations Design and Construction Guide* (SPC, 1998).

4.5.4 Insulating Concrete Form Foundation Walls

Insulating concrete forms (ICFs) have been used in the United States since the 1970s. They provide durable and thermally efficient foundation and above-grade walls at reasonable cost. Insulating concrete forms are constructed of rigid foam plastic, composites of cement and plastic foam insulation or wood chips, or other suitable insulating materials that have the ability to act as forms for cast-in-place concrete walls. The forms are easily placed by hand and remain in place after the concrete is cured to provide added insulation.

ICF systems are typically categorized with respect to the form of the ICF unit. There are three types of ICF forms: hollow blocks, planks, and panels. The shape of the concrete wall is best visualized with the form stripped away,



For more design information, refer to the *Structural Design of Insulating Concrete Form Walls in Residential Construction* (Lemay and Vrankar, 1998). For a prescriptive construction approach, consult the *Prescriptive Method for Insulating Concrete Forms in Residential Construction* (HUD, 1998). These documents can be obtained from the contacts listed in Chapter 1. Manufacturer data should also be consulted.

4.6 Slabs on Grade

The primary objectives of slab-on-grade design are

- to provide a floor surface with adequate capacity to support all applied loads;
- to provide thickened footings for attachment of the above grade structure and for transfer of the load to the earth where required; and to provide a moisture barrier between the earth and the interior of the building.

Many concrete slabs for homes, driveways, garages, and sidewalks are built according to standard thickness recommendations and do not require a specific design unless poor soil conditions, such as expansive clay soils, exist on the site.

For typical loading and soil conditions, floor slabs, driveways, garage floors, and residential sidewalks are built at a nominal 4 inches thick per ACI-302•2.1. Where interior columns and load-bearing walls bear on the slab, the slab is typically thickened and may be nominally reinforced (refer to Section 4.4 for footing design procedures). Monolithic slabs may also have thickened edges that provide a footing for structural loads from exterior load-bearing walls. The thickened edges may or may not be reinforced in standard residential practice.

Slab-on-grade foundations are often placed on 2 to 3 inches of washed gravel or sand and a 6 mil (0.006 inch) polyethylene vapor barrier. This recommended practice prevents moisture in the soil from wicking through the slab. The sand or gravel layer acts primarily as a capillary break to soil moisture transport through the soil. If tied into the foundation drain system, the gravel layer can also help provide drainage.

A slab on grade greater than 10 feet in any dimension will likely experience cracking due to temperature and shrinkage effects that create internal tensile stresses in the concrete. To prevent the cracks from becoming noticeable, the designer usually specifies some reinforcement, such as welded wire fabric (WWF) or a fiber-reinforced concrete mix. The location of cracking may be controlled by placing construction joints in the slab at regular intervals or at strategic locations hidden under partitions or under certain floor finishes (i.e., carpet).

In poor soils where reinforcement is required to increase the slab's flexural capacity, the designer should follow conventional reinforced concrete design methods. The Portland Cement Association (PCA), Wire Reinforcement



Institute (WRI), and U.S. Army Corps of Engineers (COE) espouse three methods for the design of plain or reinforced concrete slabs on grade.

Presented in chart or tabular format, the PCA method selects a slab thickness in accordance with the applied loads and is based on the concept of one equivalent wheel loading at the center of the slab. Structural reinforcement is typically not required; however, a nominal amount of reinforcement is suggested for crack control, shrinkage, and temperature effects.

The WRI method selects a slab thickness in accordance with a discrete-element computer model for the slab. The WRI approach graphically accounts for the relative stiffness between grade support and the concrete slab to determine moments in the slab. The information is presented in the form of design nomographs.

Presented in charts and tabular format, the COE method is based on Westergaard's formulae for edge stresses in a concrete slab and assumes that the unloaded portions of the slab help support the slab portions under direct loading.

For further information on the design procedures for each design method mentioned above and for unique loading conditions, refer to *ACI-360, Design of Slabs on Grade* (ACI, 1998) or the *Design and Construction of Post-Tensioned Slabs on Ground* (PTI, 1996) for expansive soil conditions.

4.7 Pile Foundations

Piles support buildings under a variety of special conditions that make conventional foundation practices impractical or inadvisable. Such conditions include

- weak soils or nonengineered fills that require the use of piles to transfer foundation loads by skin friction or point bearing;
- inland floodplains and coastal flood hazard zones where buildings must be elevated;
- steep or unstable slopes; and
- expansive soils where buildings must be isolated from soil expansion in the “active” surface layer and anchored to stable soil below.

Piles are available in a variety of materials. Preservative-treated timber piles are typically driven into place by a crane with a mechanical or drop hammer (most common in weak soils and coastal construction). Concrete piles or piers are typically cast in place in drilled holes, sometimes with “belled” bases (most common in expansive soils). Steel H-piles or large-diameter pipes are typically driven or vibrated into place with specialized heavy equipment (uncommon in residential construction).

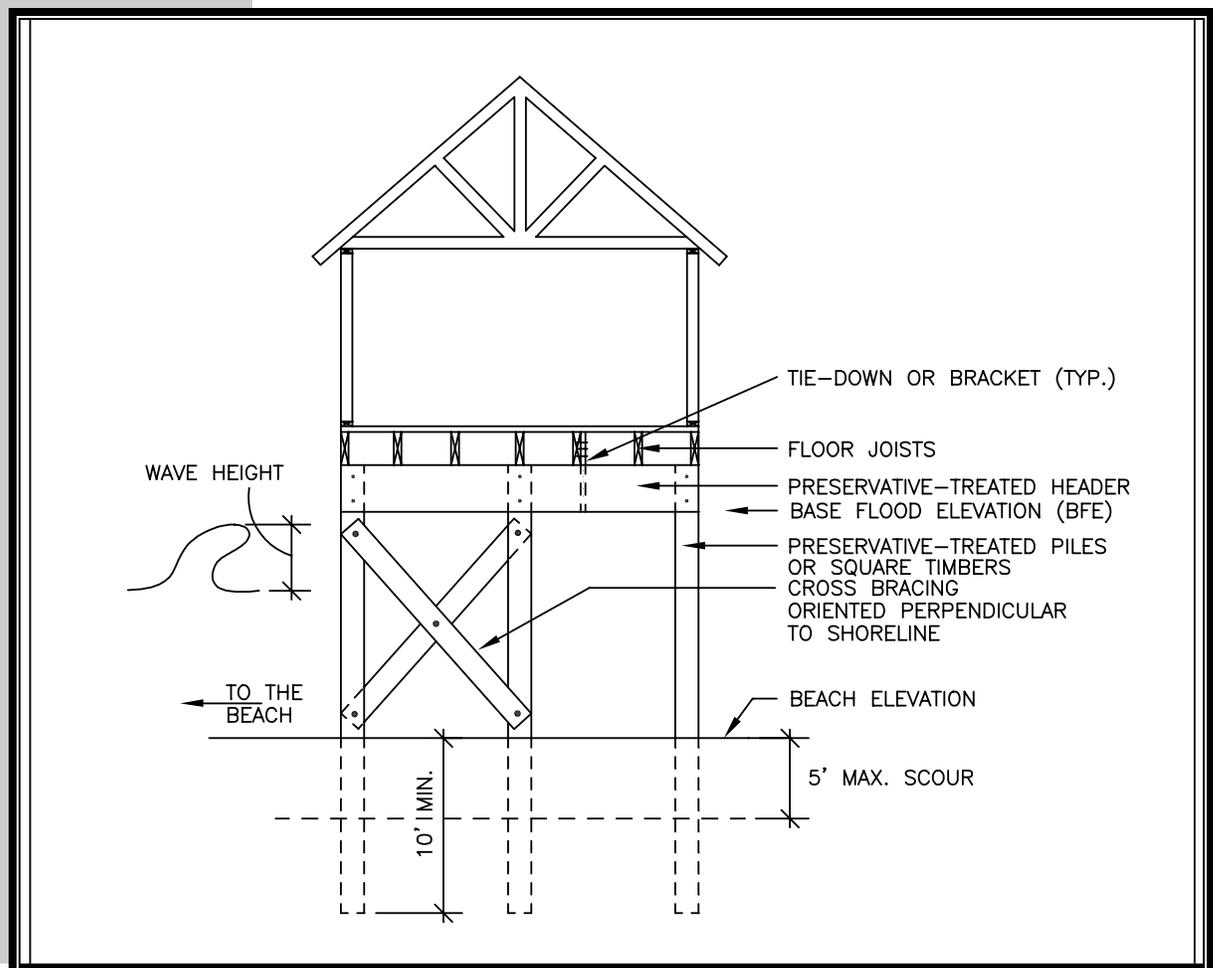
Timber piles are most commonly used in light-frame residential construction. The minimum pile capacity is based on the required foundation loading. Pile capacity is, however, difficult to predict; therefore, only rough estimates of required pile lengths and sizes can be made before installation, particularly when the designer relies only on skin friction to develop capacity in deep, soft soils. For this reason, local successful practice is a primary factor in any pile foundation design such that a pile foundation often can be specified by



experience with little design effort. In other cases, some amount of subsurface exploration (i.e., standard penetrometer test) is advisable to assist in foundation design or, alternatively, to indicate when one or more test piles may be required.

It is rare for pile depth to be greater than 8 or 10 feet except in extremely soft soils, on steeply sloped sites with unstable soils, or in coastal hazard areas (i.e., beachfront property) where significant scour is possible due to storm surge velocity. Under these conditions, depths can easily exceed 10 feet. In coastal high-hazard areas known as “V zones” on flood insurance rating maps (FIRMs), the building must be elevated above the 100-year flood elevation, which is known as the base flood elevation (BFE) and includes an allowance for wave height. As shown in Figure 4.11, treated timber piles are typically used to elevate a structure.

FIGURE 4.11 *Basic Coastal Foundation Construction*



For additional guidance, the designer is referred to the *Coastal Construction Manual* (FEMA, 1986) and *Pile Buck* (Pile Buck, 1990) but should be prepared to make reasonable design modifications and judgments based on personal experience with and knowledge of pile construction and local conditions. National flood Insurance Program (NFIP) requirements should also be carefully considered by the designer since they may affect the availability of insurance and the premium amount. From a life-safety perspective, pile-supported buildings are



often evacuated during a major hurricane, but flood damage can be substantial if the building is not properly elevated and detailed. In these conditions, the designer must consider several factors, including flood loads, wind loads, scour, breakaway wall and slab construction, corrosion, and other factors. The publications of the Federal Emergency Management Agency (FEMA), Washington, DC, offer design guidance. FEMA is also in the process of updating the *Coastal Construction Manual*.

The habitable portion of buildings in coastal “A zones” (nonvelocity flow) and inland floodplains must be elevated above the BFE, particularly if flood insurance is to be obtained. However, piles are not necessarily the most economical solution. Common solutions include fills to build up the site or the use of crawl space foundations.

For driven timber piles, the capacity of a pile can be roughly estimated from the known hammer weight, drop height, and blow count (blows per foot of penetration) associated with the drop-hammer pile-driving process. Several pile-driving formulas are available; while each formula follows a different format, all share the basic relationship among pile capacity, blow count, penetration, hammer drop height, and hammer weight. The following equation is the widely recognized method first reported in *Engineering News Record* (ENR) and is adequate for typical residential and light-frame commercial applications:

$$P_a = \frac{W_r h}{sF}$$

In the above equation, P_a is the net allowable vertical load capacity, W_r is the hammer ram weight, h is the distance the hammer free falls, s is the pile penetration (set) per blow at the end of driving, and F is the safety factory. The units for s and h must be the same. The value of s may be taken as the inverse of the blow count for the last foot of driving. Using the above equation, a “test” pile may be evaluated to determine the required pile length to obtain adequate bearing.

Alternatively, the designer can specify a required minimum penetration and required number of blows per foot to obtain sufficient bearing capacity by friction. The pile size may be specified as a minimum tip diameter, a minimum butt diameter, or both. The minimum pile butt diameter should not be less than 8 inches; 10- to 12-inch diameters are common. The larger pile diameters may be necessary for unbraced conditions with long unsupported heights.

In hard material or densely compacted sand or hard clay, a typical pile meets “refusal” when the blows per foot become excessive. In such a case, it may be necessary to jet or predrill the pile to a specific depth to meet the minimum embedment and then finish with several hammer blows to ensure that the required capacity is met and the pile properly seated in firm soil.

Jetting is the process of using a water pump, hose, and long pipe to “jet” the tip of the pile into hard-driving ground such as firm sand. Jetting may also be used to adjust the pile vertically to maintain a reasonable tolerance with the building layout dimension.

It is also important to connect or anchor the building properly to pile foundations when severe uplift or lateral load conditions are expected. For standard pile and concrete grade beam construction, the pile is usually extended into the concrete “cap” a few inches or more. The connection requirements of the



National Design Specification for Wood Construction (NDS, 1997) should be carefully followed for these “heavy duty” connections. Such connections are not specifically addressed in Chapter 7, although much of the information is applicable.

4.8 Frost Protection

The objective of frost protection in foundation design is to prevent damage to the structure from frost action (i.e., heaving and thaw weakening) in frost-susceptible soils.

4.8.1 Conventional Methods

In northern U.S. climates, builders and designers mitigate the effects of frost heave by constructing homes with perimeter footings that extend below a locally prescribed frost depth. Other construction methods include

- piles or caissons extending below the seasonal frost line;
- mat or reinforced structural slab foundations that resist differential heave;
- nonfrost-susceptible fills and drainage; and
- adjustable foundation supports.

The local building department typically sets required frost depths. Often, the depths are highly conservative in accordance with frost depths experienced in applications not relevant to residential foundations. The local design frost depth can vary significantly from that required by actual climate, soil, and application conditions. One exception occurs in Alaska, where it is common to specify different frost depths for “warm,” “cold,” and “interior” foundations. For homes in the Anchorage, Alaska, area, the perimeter foundation is generally classified as warm, with a required depth of 4 or 5 feet. Interior footings may be required to be 8 inches deep. On the other hand, “cold” foundations, including outside columns, may be required to be as much as 10 feet deep. In the contiguous 48 states, depths for footings range from a minimum 12 inches in the South to as much as 6 feet in some northern localities.

Based on the air-freezing index, Table 4.8 presents minimum “safe” frost depths for residential foundations. Figure 4.12 depicts the air-freezing index, a climate index closely associated with ground freezing depth. The most frost-susceptible soils are silty soils or mixtures that contain a large fraction of silt-sized particles. Generally, soils or fill materials with less than 6 percent fines (as measured by a #200 sieve) are considered nonfrost-susceptible. Proper surface water and foundation drainage are also important factors where frost heave is a concern. The designer should recognize that many soils may not be frost-susceptible in their natural state (i.e., sand, gravel, or other well-drained soils that are typically low in moisture content). However, for those that are frost-susceptible, the consequences can be significant and costly if not properly considered in the foundation design.

**TABLE 4.8** *Minimum Frost Depths for Residential Footings*^{1,2}

Air-Freezing Index (°F-Days)	Footing Depth (inches)
250 or less	12
500	18
1,000	24
2,000	36
3,000	48
4,000	60

Notes:

¹Interpolation is permissible.²The values do not apply to mountainous terrain or to Alaska.

4.8.2 Frost-Protected Shallow Foundations

A frost-protected shallow foundation (FPSF) is a practical alternative to deeper foundations in cold regions characterized by seasonal ground freezing and the potential for frost heave. Figure 4.13 illustrates several FPSF applications. FPSFs are best suited to slab-on-grade homes on relatively flat sites. The FPSF method may, however, be used effectively with walkout basements by insulating the foundation on the downhill side of the house, thus eliminating the need for a stepped footing.

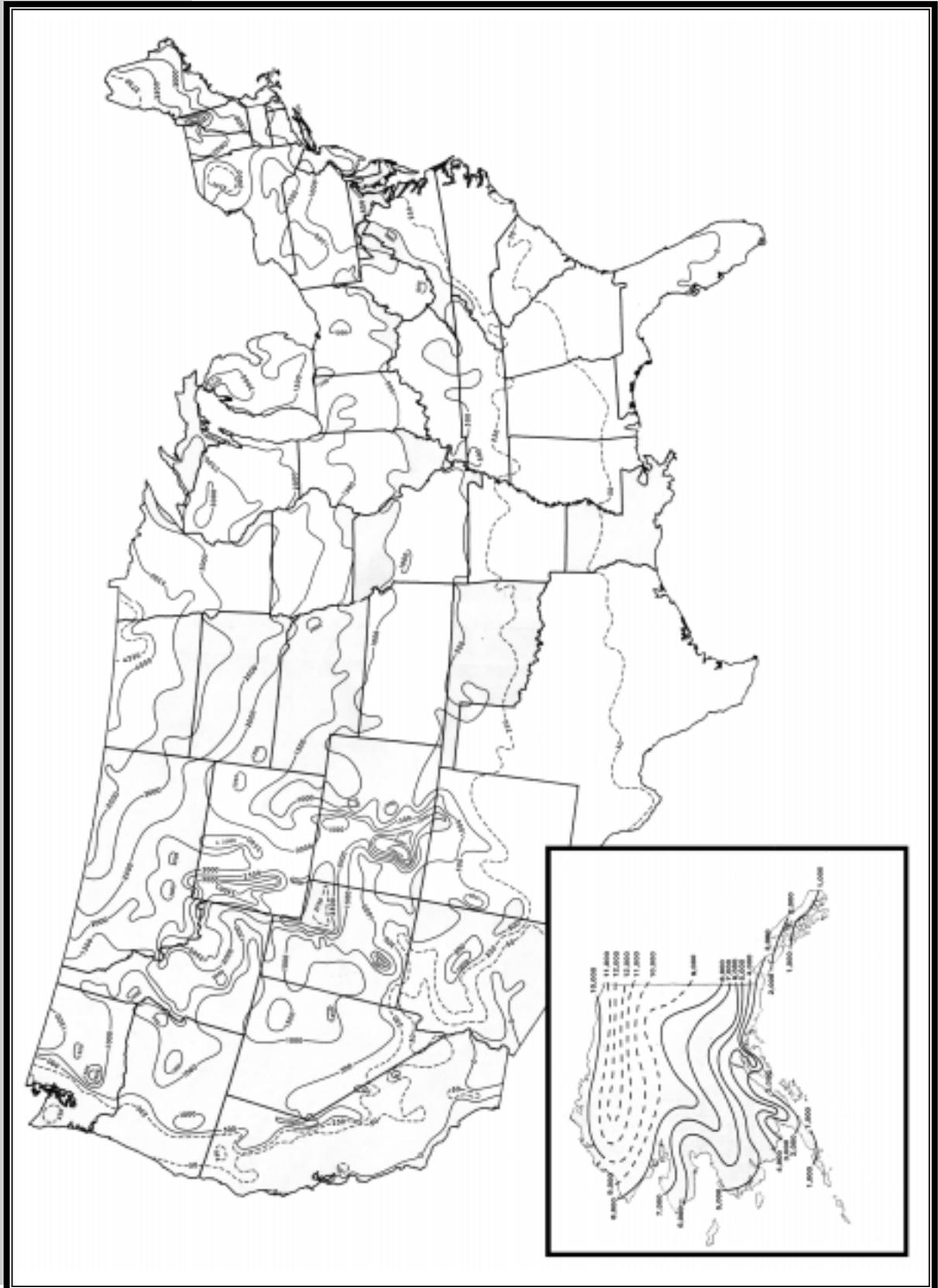
An FPSF is constructed by using strategically placed vertical and horizontal insulation to insulate the footings around the building, thereby allowing foundation depths as shallow as 12 inches in very cold climates. The frost-protected shallow foundation technology recognizes earth as a heat source that repels frost. Heat input to the ground from buildings therefore contributes to the thermal environment around the foundation.

The thickness of the insulation and the horizontal distance that the insulation must extend away from the building depends primarily on the climate. In less severe cold climates, horizontal insulation is not necessary. Other factors such as soil thermal conductivity, soil moisture content, and the internal temperature of a building are also important. Current design and construction guidelines are based on reasonable “worst-case” conditions.

After more than 40 years of use in the Scandinavian countries, FPSFs are now recognized in the prescriptive requirements of the *International One- and Two-Family Dwelling Code* (ICC, 1998) and the 1995 edition. However, the code places limits on the use of foam plastic below grade in areas of noticeably high termite infestation probability. In those areas termite barriers or other details must be incorporated into the design to block “hidden” pathways leading from the soil into the structure between the foam insulation and the foundation wall. The exception to the code limit occurs when termite-resistant materials (i.e., concrete, steel, or preservative-treated wood) are specified for a home’s structural members.



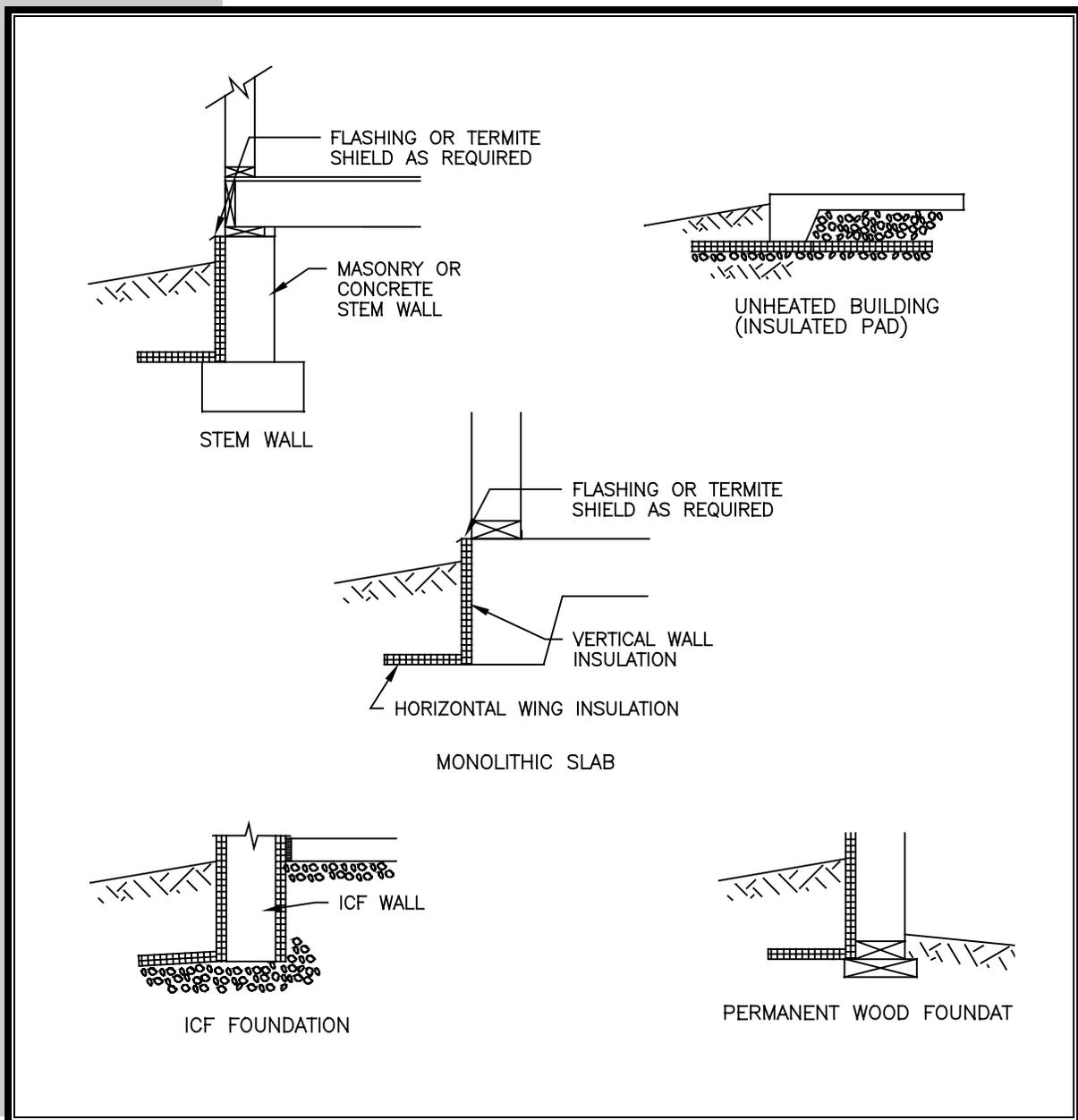
FIGURE 4.12 *Air-Freezing Index Map (100-Year Return Period)*





The complete design procedure for FPSFs is detailed in *Frost Protected Shallow Foundations in Residential Construction, Second Edition* (NAHB Research Center, Inc., 1996). The first edition of this guide is available from the U.S. Department of Housing and Urban Development. Either version provides useful construction details and guidelines for determining the amount (thickness) of insulation required for a given climate or application. Acceptable insulation materials include expanded and extruded polystyrenes, although adjusted insulation values are provided for below-ground use. The American Society of Civil Engineers (ASCE) is currently developing a standard for FPSF design and construction based on the resources mentioned above.

FIGURE 4.13 *Frost-Protected Shallow Foundation Applications*





4.8.3 Permafrost

Design of residential foundations on permafrost is beyond the scope of this guide. The designer is cautioned that the thawing of permafrost due to a building's thermal effect on a site can quickly undermine a structure. It is critical that the presence of permafrost is properly identified through subsoil exploration. Several effective design approaches are available for building on permafrost. Refer to *Construction in Cold Regions: A Guide for Planners, Engineers, Contractors, and Managers* (McFadden and Bennett, 1991). Permafrost is not a concern in the lower 48 states of the United States.



4.9 Design Examples

EXAMPLE 4.1 Plain Concrete Footing Design



Given Exterior continuous wall footing supporting an 8-inch-wide concrete foundation wall carrying a 12-foot floor tributary width; the wall supports two floor levels each with the same tributary width

Design Loads

$$\text{Live load} = 0.75 [(12 \text{ ft})(40 \text{ psf}) + (12 \text{ ft})(30 \text{ psf})] = 630 \text{ plf} \quad (\text{Table 3.1})$$

$$\text{Dead load} = (12 \text{ ft})(10 \text{ psf})(2 \text{ floors}) = 240 \text{ plf} \quad (\text{Table 3.2})$$

$$\text{Wall dead load} = (8 \text{ ft})(0.66 \text{ ft})(150 \text{ pcf}) = 800 \text{ plf} \quad (\text{Table 3.3})$$

$$\text{Footing dead load allowance} = 200 \text{ plf}$$

Presumptive soil bearing capacity = 1,500 psf (default)

$$f'_c = 2,000 \text{ psi}$$

Find The minimum size of the concrete footing required to support the loads

Solution

- Determine the required soil bearing area

$$\text{Footing width} = \frac{\text{Design load}}{\text{Presumptive soil bearing}} = \frac{(630 \text{ plf} + 240 \text{ plf} + 800 \text{ plf} + 200 \text{ plf})(1 \text{ ft})}{1,500 \text{ psf}} = 1.25 \text{ ft}$$

The required footing width is equal to

$$b = 1.25 \text{ ft} = 15 \text{ in} \cong 16 \text{ in (standard width of excavation equipment)}$$

- Preliminary design (rule of thumb method)

$$\text{Footing projection} = 1/2 (16 \text{ in.} - 8 \text{ in.}) = 4 \text{ in}$$

Required plain concrete footing thickness $\cong 4$ in (i.e., no less than the projection)

\therefore use minimum 6-inch-thick footing

$$\text{Footing weight} = (1.33 \text{ ft})(0.5 \text{ ft})(150 \text{ pcf}) = 100 \text{ lb} < 200 \text{ lb allowance} \quad \text{OK}$$

- Consider design options

- Use 6-inch x 16-inch plain wall concrete footing
- ✓ Design plain concrete footing to check rule of thumb for illustrative purposes only



4. Design a plain concrete footing

- (a) Determine soil pressure based on factored loads

$$q_s = \frac{P_u}{A_{\text{footing}}} = \frac{(1.2)(240 \text{ plf} + 800 \text{ plf} + 200 \text{ plf}) + (1.6)(630 \text{ plf})}{(1.33 \text{ ft})(1 \text{ ft})} = 1,877 \text{ psf}$$

- (b) Determine thickness of footing based on moment at the face of the wall

$$\begin{aligned} M_u &= \frac{q_s \ell}{8} (b - T)^2 \\ &= \frac{(1,877 \text{ psf})(1 \text{ ft})}{8} (1.33 \text{ ft} - 0.66 \text{ ft})^2 = 105 \text{ ft} - \text{lb} / \text{lf} \end{aligned}$$

$$\phi M_n = 5\sqrt{f'_c} S = 5\sqrt{2,000 \text{ psi}} \frac{b t^2}{6}$$

$$\phi M_n \geq M_u$$

$$(105 \text{ ft} - \text{lb} / \text{lf})(12 \text{ in} / \text{ft}) \geq (0.65)(5)(\sqrt{2,000 \text{ psi}}) \left(\frac{(12 \text{ in}) t^2}{6} \right)$$

$$t = 2.1 \text{ in}$$

- (c) Determine footing thickness based on one-way (beam) shear

$$\phi V_c = \phi \frac{4}{3} \sqrt{f'_c} \ell t$$

$$= 0.65 \left(\frac{4}{3} \right) \sqrt{2,000 \text{ psi}} (12 \text{ in})(t)$$

$$V_u = (q_s \ell)(0.5(b - T) - t)$$

$$= (1,849 \text{ psf})(1 \text{ ft})(0.5(1.33 \text{ ft} - 0.66 \text{ ft}) - t)$$

$$\phi V_c \geq V_u$$

$$0.65 \left(\frac{4}{3} \right) \sqrt{2,000 \text{ psi}} (12 \text{ in})(t) = (1,877 \text{ psf})(1 \text{ ft})(0.5(1.33 \text{ ft} - 0.66 \text{ ft}) - t)$$

$$t = 0.27 \text{ ft} = 3.2 \text{ in}$$

Therefore, shear in the footing governs the footing thickness

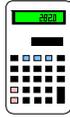


Conclusion

The calculations yield a footing thickness of 3.2 inches. In accordance with ACI-318•22.4.8, two additional inches must be added, resulting in a footing thickness of 5.2 inches. However, in accordance with ACI-318•22.7.4, plain concrete footings may not have a thickness less than 8 inches. A 6-inch-thick plain concrete footing has a history of adequate performance in residential construction and exceeds the calculated thickness requirement. Therefore, use a 6-inch-thick by 16-inch-wide wall footing

In high-hazard seismic areas, a nominal footing reinforcement should be considered (i.e., one No. 4 bar longitudinally). However, longitudinal reinforcement at the top and bottom of the foundation wall provides greater strength against differential soil movement in a severe seismic event, particularly on sites with soft soils.

It is also worthy to note that use of the ACI-318 load combinations in lieu of those provided in Chapter 3 for strength design would have resulted in a calculated footing thickness of 3.2 inches instead of 3.1 inches as governed by flexure. This is a negligible difference for practical purposes.

**EXAMPLE 4.2****Reinforced Footing Design****Given**

Interior footing supporting a steel pipe column (3.5 in x 3.5 in bearing) carrying a 12-ft x 12-ft floor tributary area

Service Loads

$$\text{Live load} = (12 \text{ ft})(12 \text{ ft})(40 \text{ psf}) = 5,760 \text{ lb}$$

$$\text{Dead load} = (12 \text{ ft})(12 \text{ ft})(10 \text{ psf}) = 1,440 \text{ lb}$$

$$\text{Footing and column dead load} = 300 \text{ lb (allowance)}$$

$$\text{Presumptive soil bearing capacity} = 1,500 \text{ psf (default)}$$

$$f'_c = 2,500 \text{ psi}, f_y = 60,000 \text{ psi}$$

Find

The minimum size of the concrete footing required to support the loads.

Solution

1. Determine the required soil bearing area

$$\text{Area reqd} = \frac{\text{Service load}}{\text{Presumptive soil bearing}} = \frac{(5,760 \text{ lb} + 1,440 \text{ lb} + 300 \text{ lb})}{1,500 \text{ psf}} = 5 \text{ ft}^2$$

Assume a square footing

$$b = \sqrt{5 \text{ ft}^2} = 2.2 \text{ ft} = 26 \text{ in}$$

2. Preliminary design (rule of thumb method)

$$\text{Footing projection} = 1/2 (26 \text{ in} - 3.5 \text{ in}) = 11.25 \text{ in}$$

$$\therefore \text{Required plain concrete footing thickness} \cong 12 \text{ in}$$

$$\text{Footing weight} = (5 \text{ ft}^2)(1 \text{ ft})(150 \text{ pcf}) = 750 \text{ lb} > 300 \text{ lb allowance}$$

$$\therefore \text{Recalculation yields a 28-in x 28-in footing.}$$

3. Consider design options

- use 12-in x 28-in x 28-in plain concrete footing (5 ft³ of concrete per footing \$);
- reduce floor column spacing (more but smaller footings, perhaps smaller floor beams, more labor)
- test soil bearing to see if higher bearing value is feasible (uncertain benefits, but potentially large, i.e., one-half reduction in plain concrete footing size);
- design a plain concrete footing to determine if a thinner footing is feasible; or
- ✓ design thinner, reinforced concrete footing (trade-off among concrete, rebar, and labor)



4. Design a reinforced concrete footing

Given Square footing, 28 in x 28 in
 $f'_c = 2,500$ psi concrete; 60,000 psi steel

Find Footing thickness and reinforcement

(a) Select trial footing thickness, rebar size, and placement

$$\begin{aligned}t &= 6 \text{ in} \\c &= 3 \text{ in} \\d_b &= 0.5 \text{ in (No. 4 rebar)}\end{aligned}$$

(b) Calculate the distance from extreme compression fiber to centroid of reinforcement d

$$\begin{aligned}d &= t - c - 0.5d_b \\&= 6 \text{ in} - 3 \text{ in} - 0.5(0.5 \text{ in}) \\&= 2.75 \text{ in}\end{aligned}$$

(c) Determine soil pressure based on factored loads

$$q_s = \frac{P_u}{A_{\text{footing}}} = \frac{(1.2)(1,440 \text{ lb} + 300 \text{ lb}) + (1.6)(5,760 \text{ lb})}{5 \text{ ft}^2} = 2,261 \text{ psf}$$

(d) Check one-way (beam) shear in footing for trial footing thickness

$$\begin{aligned}\phi V_c &= \phi 2\sqrt{f'_c}bd \\&= 0.85(2)\sqrt{2,500 \text{ psi}}(28 \text{ in})(2.75 \text{ in}) = 6,545 \text{ lbs} \\V_u &= \left(\frac{P_u}{b}\right)(0.5(b - T) - d) = \\&= \left(\frac{11,304 \text{ lbs}}{28 \text{ in}}\right)(0.5(28 \text{ in} - 3.5 \text{ in}) - 2.75 \text{ in}) = 3,835 \text{ lbs} \\ \phi V_c &\gg V_u \quad \text{OK}\end{aligned}$$

(e) Check two-way (punching) shear in trial footing

$$\begin{aligned}\phi V_c &= \phi 4\sqrt{f'_c}b_o d \\&= (0.85)(4)\sqrt{2,500 \text{ psi}}(4(3.5 \text{ in} + 2.75 \text{ in}))(2.75 \text{ in}) = 11,688 \text{ lbs} \\V_u &= \left(\frac{P_u}{b^2}\right)(b^2 - (T + d)^2) \\&= \frac{11,304 \text{ lbs}}{(28 \text{ in})^2}((28 \text{ in})^2 - (3.5 \text{ in} + 2.75 \text{ in})^2) = 10,741 \text{ lbs} \\ \phi V_c &> V_u \quad \text{OK}\end{aligned}$$



- (f) Determine reinforcement required for footing based on critical moment at edge of column

$$M_u = q_s b(0.5)(0.5(1 - T))^2$$
$$= (2,261 \text{ psf}) \left(\frac{28 \text{ in}}{12 \text{ in/ft}} \right) (0.5) \left(0.5 \left(\frac{28 \text{ in}}{12 \text{ in/ft}} - \frac{3.5 \text{ in}}{12 \text{ in/ft}} \right) \right)^2 = 2,749 \text{ ft-lbs}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{(2,749 \text{ ft-lbs})(12 \text{ in/ft})}{(0.9)(28 \text{ in})(2.75 \text{ in})^2} = 173 \text{ psi}$$

$$\rho = \left(\frac{0.85f'_c}{f_y} \right) \left(1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right)$$
$$= \left(\frac{0.85(2,500 \text{ psi})}{60,000 \text{ psi}} \right) \left(1 - \sqrt{1 - \frac{(2)(146 \text{ psi})}{0.85(2,500 \text{ psi})}} \right) = 0.022$$

$$\rho_{(\text{gross})} = \frac{d}{t} \rho = \left(\frac{2.75 \text{ in}}{6 \text{ in}} \right) (0.022) = 0.010$$

$$\rho_{\text{gross}} \geq \rho_{\text{min}} = 0.0018 \quad \text{OK}$$

$$A_s = \rho b d = 0.010 (28 \text{ in})(2.75 \text{ in}) = 0.77 \text{ in}^2$$

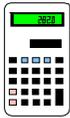
$$\text{Use four No. 4 bars where } A_s = 4(0.2 \text{ in}^2) = 0.8 \text{ in}^2 \geq 0.77 \text{ in}^2 \quad \text{OK}$$

Conclusion

Use minimum 28-in x 28-in x 6-in footing with four No. 4 bars or three No. 5 bars each way in footing.

$f'_c = 2,500 \text{ psi}$ minimum (concrete)

$f_y = 60,000 \text{ psi}$ minimum (steel reinforcing bar)

**EXAMPLE 4.3****Plain Concrete Foundation Wall Design****Given**

Design loads

Snow load (S)	=	280 plf
Live load (L)	=	650 plf
Dead load (D)	=	450 plf
Moment at top	=	0
Concrete weight	=	150 pcf
Backfill material	=	45 pcf
f'_c	=	3,000 psi

Wall thickness	=	8 in
Wall height	=	8 ft
Unbalanced backfill height	=	7 ft

Assume axial load is in middle one-third of wall

Find

Verify that an 8-inch-thick plain concrete wall is adequate for the following load combinations from Chapter 3 (Table 3.1)

- $1.2D + 1.6H$
- $1.2D + 1.6H + 1.6L + 0.5(L_r + S)$
- $1.2D + 1.6H = 1.6(L_r + S) + 0.5L$

Only the first load combination will be evaluated since it can be shown to govern the wall design.

Solution

- Determine loads

Equivalent fluid density of backfill soil

Silty clay: $w = 100$ pcf, $K_a = 0.45$ (see Section 3.5)

$$q = K_a w = (0.45)(100 \text{ pcf}) = 45 \text{ pcf}$$

Total lateral earth load

$$H = \frac{1}{2} q l^2 = \frac{1}{2} (45 \text{ pcf})(7 \text{ ft})^2 = 1,103 \text{ plf}$$

$$X_1 = \frac{1}{3} l = \frac{1}{3} (7 \text{ ft}) = 2.33 \text{ ft}$$

Maximum shear occurs at bottom of wall (see Figure A.1 of Appendix A)

$$V_{\text{bottom}} = V_1 = \frac{1}{2} q h^2 \left(1 - \frac{h}{3L} \right) = \frac{1}{2} (45 \text{ pcf})(7 \text{ ft})^2 \left(1 - \frac{7 \text{ ft}}{3(8 \text{ ft})} \right) = 781 \text{ plf}$$



Maximum moment and its location

$$\begin{aligned} x &= h - \sqrt{h^2 - \frac{2V_1}{q}} \\ &= 7 \text{ ft} - \sqrt{(7 \text{ ft})^2 - \frac{2(781 \text{ plf})}{45 \text{ pcf}}} \\ &= 3.2 \text{ ft from base of wall or } 4.8 \text{ ft from top of wall} \end{aligned}$$

$$\begin{aligned} M_{\max} (\text{at } x = 3.2 \text{ ft}) &= V_1 x - \frac{1}{2} q h x^2 + \frac{1}{6} q x^3 \\ &= (781 \text{ plf})(3.2 \text{ ft}) - \frac{1}{2} (45 \text{ pcf})(7 \text{ ft})(3.2 \text{ ft})^2 + \frac{1}{6} (45 \text{ pcf})(3.2 \text{ ft})^3 \\ &= 1,132 \text{ ft-lb/lf} \end{aligned}$$

2. Check shear capacity

(a) Factored shear load

$$\begin{aligned} V_u &= 1.6 V_{\text{bottom}} \\ &= 1.6 (781 \text{ plf}) = 1,250 \text{ plf} \end{aligned}$$

(b) Factored shear resistance

$$\begin{aligned} \phi V_n &= \phi \frac{4}{3} \sqrt{f'_c} b h \\ &= (0.65) \left(\frac{4}{3} \right) \sqrt{3,000 \text{ psi}} (8 \text{ in})(12 \text{ in}) = 4,557 \text{ plf} \end{aligned}$$

(c) Check $\phi V_n \geq V_u$

$$4,557 \text{ plf} \gg 1,250 \text{ plf} \quad \text{OK}$$

Shear is definitely not a factor in this case. Future designs of a similar nature may be based on this experience as “OK by inspection.”

3. Check combined bending and axial load capacity

(a) Factored loads

$$\begin{aligned} M_u &= 1.6 M_{\max} = 1.6 (1,132 \text{ ft-lb/lf}) = 1,811 \text{ ft-lb/lf} \\ P_u &= 1.2 D \\ D_{\text{structure}} &= 450 \text{ plf (given)} \\ D_{\text{concrete@x}} &= (150 \text{ plf}) \left(\frac{8 \text{ in}}{12 \text{ in/ft}} \right) (8 \text{ ft} - 3.23 \text{ ft}) = 480 \text{ plf} \end{aligned}$$

$$\begin{aligned} D &= 450 \text{ plf} + 480 \text{ plf} = 930 \text{ plf} \\ P_u &= 1.2 (930 \text{ plf}) = 1,116 \text{ plf} \end{aligned}$$



(b) Determine M_n , M_{min} , P_u

$$M_n = 0.85 f'_c S$$

$$S = \frac{1}{6} b d^2 = \left(\frac{1}{6}\right) (12 \text{ in})(8 \text{ in})^2 = 128 \text{ in}^3 / \text{lf}$$

$$M_n = 0.85 (3,000 \text{ psi})(128 \text{ in}^3 / \text{lf}) = 326,400 \text{ in-lb/lf} = 27,200 \text{ ft-lb/lf}$$

$$M_{min} = 0.1 h P_u = 0.1 \left(\frac{8 \text{ in}}{12 \text{ in} / \text{lf}} \right) (1,112 \text{ plf}) = 74 \text{ ft-lb/lf}$$

$$M_u > M_{min} \quad \text{OK}$$

$$P_n = 0.6 f'_c \left[1 - \left(\frac{L}{32h} \right)^2 \right] A_g$$

$$= 0.6 (3,000 \text{ psi}) \left[1 - \left(\frac{(8 \text{ ft}) \left(\frac{12 \text{ in}}{\text{ft}} \right)}{32 (8 \text{ in})} \right)^2 \right] (8 \text{ in})(12 \text{ in}) = 148,500 \text{ plf}$$

(c) Check combined bending and axial stress equations

$$\text{Compression} \quad \frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \leq 1$$

$$\frac{1,116 \text{ plf}}{(0.65)(148,500 \text{ plf})} + \frac{1,811 \text{ ft-lb/lf}}{(0.65)(27,200 \text{ ft-lb/lf})} \leq 1$$

$$0.11 \leq 1 \quad \text{OK}$$

$$\text{Tension} \quad \frac{M_u}{S} - \frac{P_u}{A_g} \leq \phi 5 \sqrt{f'_c}$$

$$\frac{1,811 \text{ ft-lb/lf} (12 \text{ in/ft})}{128 \text{ in}^3 / \text{lf}} - \frac{1,116 \text{ plf}}{(8 \text{ in})(12 \text{ in})} \leq (0.65) (5) \sqrt{3,000 \text{ psi}}$$

$$158 \leq 178 \quad \text{OK}$$

\therefore No reinforcement required



4. Check deflection at mid-span (see Figure A.1 in Appendix A)

$$\begin{aligned}\rho_{\max} &\cong \frac{qL^3}{E_c I_g} \left[\frac{hL}{128} - \frac{L^2}{960} - \frac{h^2}{48} + \frac{h^3}{144L} \right] \\ &= \frac{(45 \text{ pcf})(8 \text{ ft})^3}{(3,122,019 \text{ psi}) \left(\frac{12 \text{ in} (8 \text{ in})^3}{12} \right)} \left[\frac{(7 \text{ ft})(8 \text{ ft})}{128} - \frac{(8 \text{ ft})^2}{960} - \frac{(7 \text{ ft})^2}{48} + \frac{(7 \text{ ft})^3}{144(8 \text{ ft})} \right] \left(\frac{1,728 \text{ in}^3}{\text{ft}^3} \right) \\ &= 0.009 \text{ in/ft} \\ \rho_{\text{all}} &= \frac{L}{240} = \frac{(8 \text{ ft})(12 \text{ in/ft})}{240} = 0.4 \text{ in/ft} \\ \rho_{\max} &\ll \rho_{\text{all}} \quad \text{OK}\end{aligned}$$

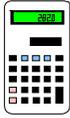
Conclusion

An 8-inch-thick plain concrete wall is adequate under the given conditions.

The above analysis was performed for a given wall thickness. The same equations can be used to solve for the minimum wall thickness h that satisfies the requirements for shear, combined bending and axial stress, and deflection. With this approach to the problem, the minimum thickness would be 7.6 inches (controlled by tensile stress under combined bending and axial load).

In the strength-based design approach, the safety margin is related to the use of load and resistance factors. In this problem, the load factor was 1.6 (for a soil load, H) and the resistance factor 0.65 (for tensile bending stress). In terms of a traditional safety factor, an equivalent safety margin is found by $1.6/0.65 = 2.5$. It is a fairly conservative safety margin for residential structures and would allow for an equivalent soil fluid density of as much as 113 pcf ($45 \text{ pcf} \times 2.5$) at the point the concrete tensile capacity based on the minimum concrete compressive strength (as estimated by $5\sqrt{f'_c}$) is realized. This capacity would exceed loads that might be expected should the soil become saturated as would occur under severe flooding on a site that is not well drained.

The use of reinforcement varies widely as an optional enhancement in residential construction to control cracking and provide some nominal strength benefits. If reinforcement is used as a matter of good practice, one No. 4 bar may be placed as much as 8 feet on-center. One horizontal bar may also be placed horizontally at the top of the wall and at mid-height.

**EXAMPLE 4.4****Plain Concrete Wall Interaction Diagram**

Given Construct an interaction diagram for the wall in Design Example 4.3

$$\begin{aligned} \text{Wall height} &= 8 \text{ ft} \\ \text{Wall thickness} &= 8 \text{ in} \\ f'_c &= 3,000 \text{ psi} \end{aligned}$$

Solution

1.

Determine compression boundary

$$\begin{aligned} P_n &= 0.6 f'_c \left[1 - \left(\frac{L}{32h} \right)^2 \right] A_g \\ &= 0.6 (3,000 \text{ psi}) \left[1 - \left(\frac{(8 \text{ ft})(12 \text{ in / lf})}{32 (8 \text{ in})} \right)^2 \right] (8 \text{ in})(12 \text{ in}) = 148,500 \text{ plf} \end{aligned}$$

$$\begin{aligned} M_n &= 0.85 f'_c S \\ &= (0.85)(3,000 \text{ psi}) \frac{(12 \text{ in})(8 \text{ in})^2}{6} \\ &= 326,400 \text{ in-lb/lf} = 27,200 \text{ ft-lb/lf} \end{aligned}$$

$$A_g = (8 \text{ in})(12 \text{ in}) = 96 \text{ in}^2$$

$$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \leq 1$$

$$\frac{P_u}{0.65 (148,500 \text{ plf})} + \frac{M_u}{0.65 (27,200 \text{ ft-lb / lf})} \leq 1$$

$$\frac{P_u}{(96,525 \text{ plf})} + \frac{M_u}{17,680 \text{ ft-lb / lf}} \leq 1$$

$$M_u = \left(1 - \frac{P_u}{96,525 \text{ plf}} \right) 17,680 \text{ ft-lb / lf}$$

$$M_u = 17,680 \text{ ft-lb / lf} - 0.18316 P_u$$

$$P_u = \left(1 - \frac{M_u}{17,680 \text{ ft-lb / lf}} \right) 96,525 \text{ plf}$$

$$P_u = 96,525 \text{ plf} - 5.46 M_u$$

$$\text{When } P_u = 0, M_u = 17,680 \text{ ft-lb/lf}$$

$$\text{When } M_u = 0, P_u = 96,525 \text{ plf} \quad (0, 96.5 \text{ klf})$$

2. Determine tension boundary

$$\frac{M_u}{S} - \frac{P_u}{A_g} \leq 5\phi \sqrt{f'_c}$$

$$\frac{M_u}{128 \text{ in}^3} - \frac{P_u}{96 \text{ in}^2} \leq 5(0.65) \sqrt{3,000 \text{ psi}}$$

$$\frac{M_u}{128 \text{ in}^3} - \frac{P_u}{96 \text{ in}^2} \leq 178 \text{ psi}$$

$$P_u = 96 \text{ in}^2 \left(\frac{M_u}{128 \text{ in}^3} - 178 \text{ psi} \right)$$

$$P_u = 0.75 M_u - 17,088 \text{ plf}$$

$$\text{When } M_u = 0; P_u = -17,088 \text{ plf} = -17.09 \text{ klf} \quad (-17.09, 0)$$



3. Determine point of intersection of the tensile and compression boundaries

$$P_u = \frac{\phi M_n - 5\phi \sqrt{f'_c} S}{\frac{S}{A_g} + \frac{\phi M_n}{\phi P_n}}$$

$$= \frac{(0.65)(27,200 \text{ ft-lb/lf})(12 \text{ in/ft}) - 5(0.65)\sqrt{3,000 \text{ psi}}(128 \text{ in}^3)}{\frac{128 \text{ in}^3}{96 \text{ in}^2} + \frac{0.65(27,200 \text{ ft-lb/lf})(12 \text{ in/ft})}{96,525 \text{ plf}}} = 53,627 \text{ plf}$$

$$= 53.63 \text{ klf}$$

$$M_u = \phi M_n \left(1 - \frac{(1,000 \text{ lb/kip}) P_u}{\phi P_n} \right)$$

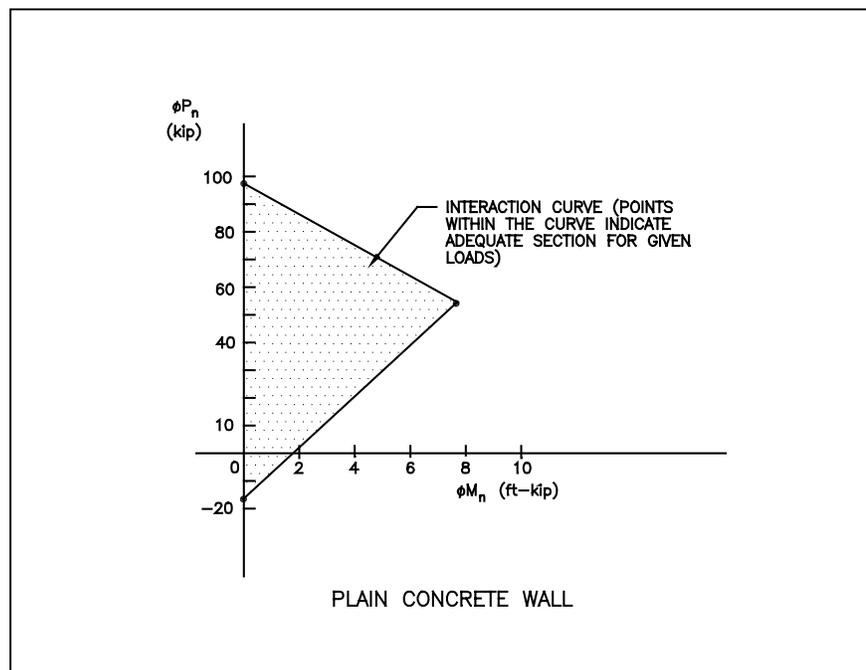
$$= (0.65)(12 \text{ in/ft})(27,200 \text{ ft-lb/lf}) \left(1 - \frac{(1,000 \text{ lb/kip})(53.63)}{96,525 \text{ plf}} \right)$$

$$= 94,282 \text{ in-lb/lf} = 7.9 \text{ ft-kip/lf}$$

Conclusion

Shown below is the interaction diagram for an 8-foot-high, 8-inch-thick plain concrete wall where the concrete compressive strength is 3,000 psi. The interaction diagram uses the points determined in the above steps.

- (0, 96.5) from step (1)
- (-17.09, 0) from step (2)
- (7.9, 53.63) from step (3)



Interaction Diagram

**EXAMPLE 4.5****Moment Magnifier****Given**

Service loads

Live load (L)	= 1,000 plf
Dead load (D)	= 750 plf
Moment at top, (M_{top})	= 0
M_u	= 2,434 ft-lb/lf
Concrete weight	= 150 pcf
Backfill material	= 45 pcf (equivalent fluid density)
f'_c	= 3,000 psi
One No. 6 bar at 12 inches on-center ($A_s=0.44 \text{ in}^2$)	
Nonsway frame	
Wall thickness	= 8 in
Wall height	= 10 ft

Assume axial load is in middle one-third of wall

Find The moment magnifier for load combination $U = 1.2D + 1.6L$ (Chapter 3, Table 3.1)

Solution

- Determine total axial load on wall

$$P_u = 1.2D + 1.6L$$

$$1.2(750 \text{ plf}) + 1.6(1,000 \text{ plf}) = 2,500 \text{ plf}$$

- Determine approximate moment magnifier by using the table in Section 4.3.1.3, assuming the axial load is 2,500 plf

		P_u	
		2,000 lbs	4,000 lbs
7.5-in-thick wall	10 ft height	1.04	1.09
9.5-in-thick wall	10 ft height	1.00	1.04

For an 8-in-thick wall, 10-ft-high with approximately 3,000 plf factored axial load acting on the wall, the magnifier through interpolation is

$$\delta_{ns} \cong 1.04$$

The objective has been met; however, the detailed calculations to determine the moment magnifier are shown below for comparison purposes.

- Calculate the moment magnifier

$$E_c = 57,000\sqrt{f'_c} = 57,000\sqrt{3,000} \text{ psi} = 3,122,019 \text{ psi}$$

$$\beta_d = \frac{P_{u,\text{dead}}}{P_u} = \frac{(1.2)(750 \text{ plf})}{1.2(750 \text{ plf}) + 1.6(1,000 \text{ plf})} = 0.36$$

$$\rho = \frac{A_s}{A_g} = \left(\frac{0.44 \text{ in}^2}{(8 \text{ in})(12 \text{ in})} \right) = 0.0046 \text{ [one No. 6 at 12 inches; } A_s = 0.44 \text{ in}^2 \text{ OK]}$$

$$\beta = 0.9 + 0.5 \beta_d^2 - 12\rho \geq 1$$

$$= 0.9 + 0.5 (0.36)^2 - 12(0.0046) = 0.91 < 1$$

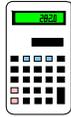
$$= 1 \text{ (governs)}$$



$$\begin{aligned}M_{2,\min} &= P_u (0.6 + 0.03h) \\ &= (2,500 \text{ plf})(0.6 + 0.03 (8 \text{ in})) = 2,100 \text{ in-lb/lf} \\ M_u &= 2,434 \text{ ft-lb/lf} \leftarrow \text{Governs} \\ e &= \frac{M_2}{P_u} = \frac{2,434 \text{ ft-lb/lf}}{(2,500 \text{ plf})} = 0.89 \text{ ft} = 10.7 \text{ in} \\ EI &= \frac{0.4E_c I_g}{\beta} \geq \frac{E_c I_g (0.5 - \frac{e}{h})}{\beta} \geq \frac{0.1E_c I_g}{\beta} \\ EI &= \frac{(3,122,019 \text{ psi}) \left(\frac{(12 \text{ in})(8 \text{ in})^3}{12} \right) \left(0.5 - \frac{10.7 \text{ in}}{8 \text{ in}} \right)}{1} = -1.3 \times 10^9 \text{ lb-in}^2 / \text{lf} \\ EI_{\max} &= \frac{0.4(3,122,019 \text{ psi}) \left(\frac{(12 \text{ in})(8 \text{ in})^3}{12} \right)}{1} = 6.4 \times 10^8 \text{ lb-in}^2 / \text{lf} \\ EI_{\min} &= \frac{0.1(3,122,019 \text{ psi}) \left(\frac{(12 \text{ in})(8 \text{ in})^3}{12} \right)}{1} = 1.6 \times 10^8 \text{ lb-in}^2 / \text{lf} \text{ (governs)} \\ C_m &= 1 \\ P_c &= \frac{\pi^2 EI}{(kl_u)^2} = \frac{\pi^2 (1.6 \times 10^8 \text{ lb-in}^2 / \text{lf})}{(1 (10 \text{ ft})(12 \text{ in/ft}))^2} = 109,662 \text{ plf} \\ \delta_{ns} &= \frac{C_m}{1 - \left(\frac{P_u}{0.75 P_c} \right)} \geq 1.0 \\ &= \frac{1}{1 - \left(\frac{2,500 \text{ plf}}{0.75 (109,662 \text{ plf})} \right)} = 1.03 \geq 1 \\ \delta_{ns} &= 1.03\end{aligned}$$

Conclusion

The moment magnifier by the approximation method is 1.04. It is slightly conservative but saves time in calculation. Through calculation, a slight efficiency is achieved and the calculated moment magnifier is 1.03.

**EXAMPLE 4.6****Reinforced Concrete Foundation Wall Design****Given**

Service loads

Live load (L)	= 1000 plf
Dead load (D)	= 750 plf
Moment at top	= 0
Concrete weight	= 150 pcf
Backfill material	= 60 pcf (equivalent fluid density)
Wall thickness	= 8 in
Wall height	= 10 ft
Unbalanced backfill height	= 8 ft
$f'_c=3,000$ psi, $f_y=60,000$ psi	
Assume axial load is in middle one-third of wall	

Find

If one No. 5 bar at 24 inches on-center vertically is adequate for the load combination, $U = 1.2D + 1.6H + 1.6L$ (Chapter 3, Table 3.1) when rebar is placed 3 inches from the outer face of wall ($d=5$ in)

Solution

- Determine loads

Total lateral earth load

$$H = \frac{1}{2}ql^2 = \frac{1}{2}(60 \text{ pcf})(8 \text{ ft})^2 = 1,920 \text{ plf}$$

$$X = \frac{1}{3}l = \frac{1}{3}(8 \text{ ft}) = 2.67 \text{ ft}$$

Maximum shear occurs at bottom of wall

$$\begin{aligned} \sum M_{\text{top}} &= 0 \\ V_{\text{bottom}} &= \frac{H(L-x)}{L} = \frac{(1,920 \text{ plf})(10 \text{ ft} - 2.67 \text{ ft})}{10 \text{ ft}} = 1,408 \text{ plf} \end{aligned}$$

Maximum moment and its location

$$\begin{aligned} X_{\text{max}} &= \frac{ql - \sqrt{q^2l^2 - 2qV_{\text{bottom}}}}{q} \\ &= \frac{(60 \text{ pcf})(8 \text{ ft}) - \sqrt{(60 \text{ pcf})^2(8 \text{ ft})^2 - 2(60 \text{ pcf})(1,408 \text{ plf})}}{60 \text{ pcf}} \\ X_{\text{max}} &= 3.87 \text{ ft from base of wall or } 6.13 \text{ ft from top of wall} \\ M_{\text{max}} &= \frac{-qlx_{\text{max}}^2}{2} + \frac{qx_{\text{max}}^3}{6} + V_{\text{bottom}}(x_{\text{max}}) \\ &= \frac{-(60 \text{ pcf})(8 \text{ ft})(3.87 \text{ ft})^2}{2} + \frac{(60 \text{ pcf})(3.87 \text{ ft})^3}{6} + (1,408 \text{ plf})(3.87 \text{ ft}) \\ &= 2,434 \text{ ft-lb/lf} \end{aligned}$$



2. Check shear capacity assuming no shear reinforcement is required ($V_s=0$)

(a) Factored shear load

$$\begin{aligned} V_u &= 1.6 V_{\text{bottom}} \\ &= 1.6 (1,408 \text{ plf}) = 2,253 \text{ plf} \end{aligned}$$

(b) Factored shear resistance

$$\begin{aligned} \phi V_n &= \phi (V_c + V_s) \\ &= \phi (2) \sqrt{f'_c} b_w d \\ &= (0.85) (2) \sqrt{3,000 \text{ psi}} (12 \text{ in}) (5 \text{ in}) = 5,587 \text{ plf} \end{aligned}$$

(c) Check $\phi V_n \geq V_u$

$$5,587 \text{ plf} \gg 2,253 \text{ plf} \quad \text{OK}$$

Shear is definitely not a factor in this case. Future designs of a similar nature may be based on this experience as “OK by inspection”

3. Determine slenderness

All four foundation walls are concrete with few openings; therefore, the system is a nonsway frame. This is a standard assumption for residential concrete foundation walls.

$$\begin{aligned} \text{Slenderness} \quad r &= \sqrt{\frac{I_{gg}}{A_{gg}}} = \sqrt{\frac{\left(\frac{1}{12}\right) (12 \text{ in}) (8 \text{ in})^3}{(8 \text{ in})(12 \text{ in})}} = 2.31 \\ \frac{kl_u}{r} &< 34 \\ \frac{(1)(8 \text{ in})(12 \text{ in})}{2.31} &= 41.6 \geq 34 \quad \therefore \text{Use moment magnifier method} \end{aligned}$$

4. Determine the magnified moment using the moment magnifier method

$$P_u = 1.2D + 1.6L = 1.2 (750 \text{ plf}) + 1.6 (1,000 \text{ plf}) = 2,500 \text{ plf}$$

Using the approximated moment magnifiers in Table 4.4, the moment magnifier from the table for a 7.5-inch-thick wall, 10-foot-high is between 1.04 and 1.09. For a 9.5-inch-thick wall, the values are between 1 and 1.04.

Through interpolation, $\delta = 1.04$ for a 2,500 plf axial load.



5. Check pure bending

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.155 \text{ in}^2)(60,000 \text{ psi})}{0.85 (3,000 \text{ psi})(12 \text{ in})} = 0.304$$

$$\begin{aligned} \phi M_n &= \phi A_s f_y \left(d - \frac{a}{2}\right) \\ &= 0.9 (0.155 \text{ in}^2)(60,000 \text{ psi}) \left(5 \text{ in} - \frac{0.304 \text{ in}}{2}\right) \\ &= 40,577 \text{ in-lb/lf} = 3,381 \text{ ft-lb/lf} \end{aligned}$$

$$\phi P_n = 0$$

$$M_u = 2,434 \text{ ft-lb/lf from step (1)}$$

$$\delta M_u = 1.04 (2,434 \text{ ft-lb/lf}) = 2,531 \text{ ft-lb/lf}$$

By inspection of the interaction diagram in Example 4.6, one No. 5 at 24 inches on center is OK since $\delta M_u P_u$ is contained within the interaction curve. See Example 4.6 to construct an interaction diagram.

6. Check deflection

$$\begin{aligned} \rho_{\max} &= \left[-\frac{q(x-L+1)^5}{120} + \frac{ql^3 x^3}{36L} + \frac{ql^5 x}{120L} - \frac{ql^3 Lx}{36} \right] / E_c I_g \\ &= \frac{(1728 \text{ in}^3)}{\text{ft}^3} \left[\frac{(60 \text{ pcf})(6.13 \text{ ft} - 10 \text{ ft} + 8 \text{ ft})^5}{120} + \frac{(60 \text{ pcf})(8 \text{ ft})^3 (6.13 \text{ ft})^3}{36(10 \text{ ft})} \right. \\ &\quad \left. + \frac{(60 \text{ pcf})(8 \text{ ft})^5 (6.13 \text{ ft})}{120(10 \text{ ft})} - \frac{(60 \text{ pcf})(8 \text{ ft})^3 (10 \text{ ft})(6.13 \text{ ft})}{36} \right] / (3,122,019 \text{ psi}) \left(\frac{(12 \text{ in})(8 \text{ in})^3}{12} \right) \\ &= 0.025 \text{ in/lf} \end{aligned}$$

$$\rho_{\text{all}} = \frac{L}{240} = \frac{(10 \text{ ft})(12 \text{ in} / \text{ft})}{240} = 0.5 \text{ in} / \text{lf}$$

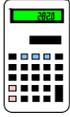
$$\rho_{\max} \ll \rho_{\text{all}} \quad \text{OK}$$

Conclusion

An 8-inch-thick reinforced concrete wall with one vertical No. 5 bar at 24 inches on-center is adequate for the given loading conditions.

This analysis was performed for a given wall thickness and reinforcement spacing. The same equations can be used to solve for the minimum reinforcement that satisfies the requirements for shear, combined bending and axial stress, and deflection. This approach would be suitable for a computer spreadsheet design aid. A packaged computer software program can also be purchased to perform this function; however, certain limitations may prohibit the designer from using design recommendations given in this guide.

The use of horizontal reinforcement varies widely as an optional enhancement. If horizontal reinforcement is used as a matter of preferred practice to control potential cracking, one No. 4 bar placed at the top of the wall and at mid-height is typically sufficient.

**EXAMPLE 4.7****Reinforced Concrete Interaction Diagram**

Given Determine interaction diagram for the 8-inch-thick concrete foundation wall in Example 4.5

Wall height = 10 ft
 Wall thickness = 8 in
 f'_c = 3,000 psi
 f_y = 60,000 psi
 One No. 5 bar at 24 inches on center ($A_s = 0.155 \text{ in}^2/\text{lf}$)

Solution

- $$C_s = A_s f_y = (0.155 \text{ in}^2/\text{lf})(60,000 \text{ psi}) = 9,300 \text{ plf}$$

$$C_c = 0.85 f'_c (A_g - A_s) = 0.85 (3,000 \text{ psi})(8 \text{ in})(12 \text{ in}/\text{lf}) - 0.155 \text{ in}^2/\text{lf} = 244,405 \text{ plf}$$

$$\phi M_n = 0$$

$$\phi P_n = \phi (C_c + C_s) = 0.7 (9,300 \text{ plf} + 244,405 \text{ plf}) = 177,594 \text{ plf} \quad (0, 178)$$

$$\phi P_{n,\max} = 0.8 \phi P_n = 0.8 (177,594 \text{ plf}) = 142,080 \text{ plf} \quad (0, 142)$$
- $$c = d = 5 \text{ in}$$

$$a = \beta c = 0.85 (5 \text{ in}) = 4.25 \text{ in}$$

$$C_c = 0.85 a b f'_c = 0.85 (4.25 \text{ in})(12 \text{ in})(3,000 \text{ psi}) = 130,050 \text{ plf}$$

$$\phi M_n = \phi C_c (d - 0.5a) = 0.7 (130,050 \text{ plf})(5 - 0.5(4.25 \text{ in})) = 261,725 \text{ in-lb}/\text{lf} = 21.8 \text{ ft-kip}/\text{lf}$$

$$\phi P_n = \phi C_c = 0.7 (130,050 \text{ plf}) = 91,035 \text{ plf} \quad (21.8, 91)$$
- $$\epsilon_c = 0.003$$

$$\epsilon_y = \frac{f_y}{E_s} = \frac{60,000 \text{ psi}}{29 \times 10^6 \text{ psi}} = 2.07 \times 10^{-3} = 0.002$$

$$c = \left(\frac{\epsilon_c}{\epsilon_c + 0.5 \epsilon_y} \right) d = \left(\frac{0.003}{0.003 + 0.5(0.002)} \right) (5 \text{ in}) = 3.72 \text{ in}$$

$$a = \beta c = 0.85 (3.72 \text{ in}) = 3.16 \text{ in}$$

$$T_s = A_s (0.5 f_y) = (0.155 \text{ in}^2)(0.5)(60,000 \text{ psi}) = 4,650 \text{ plf}$$

$$C_c = 0.85 a b f'_c = 0.85 (3.16 \text{ in})(12 \text{ in})(3,000 \text{ psi}) = 96,696 \text{ plf}$$

$$\phi M_n = \phi C_c (d - 0.5a) = 0.7 (96,696 \text{ plf})(5 \text{ in} - 0.5(3.16 \text{ in})) = 231,490 \text{ in-lb}/\text{lf} = 19.3 \text{ ft-kip}/\text{lf}$$

$$\phi P_n = \phi (C_c - T_s) = 0.7 (96,696 \text{ plf} - 4,650 \text{ plf}) = 64,432 \text{ plf} \quad (19.3, 64)$$
- $$\epsilon_c = 0.003$$

$$\epsilon_y = \frac{f_y}{E_s} = \frac{60,000 \text{ psi}}{29 \times 10^6 \text{ psi}} = 2.07 \times 10^{-3}$$

$$c = \left(\frac{\epsilon_c}{\epsilon_c + \epsilon_y} \right) d = \left(\frac{0.003}{0.003 + 2.07 \times 10^{-3}} \right) (5 \text{ in}) = 2.96 \text{ in}$$

$$a = \beta c = 0.85 (2.96 \text{ in}) = 2.5 \text{ in}$$

$$C_c = 0.85 a b f'_c = 0.85 (2.5 \text{ in})(12 \text{ in})(3,000 \text{ psi}) = 76,500 \text{ plf}$$

$$T_s = A_s f_y = (0.155 \text{ in}^2)(60,000 \text{ psi}) = 9,300 \text{ plf}$$

$$\phi M_n = \phi C_c (d - 0.5a) = 0.7 (76,500 \text{ plf})(5 \text{ in} - 0.5(2.5 \text{ in})) = 200,810 \text{ in-lb}/\text{lf} = 16.7 \text{ ft-kip}/\text{lf}$$

$$\phi P_n = \phi (C_c - T_s) = 0.7 (76,500 \text{ plf} - 9,300 \text{ plf}) = 47,040 \text{ plf} \quad (16.7, 47)$$



5.

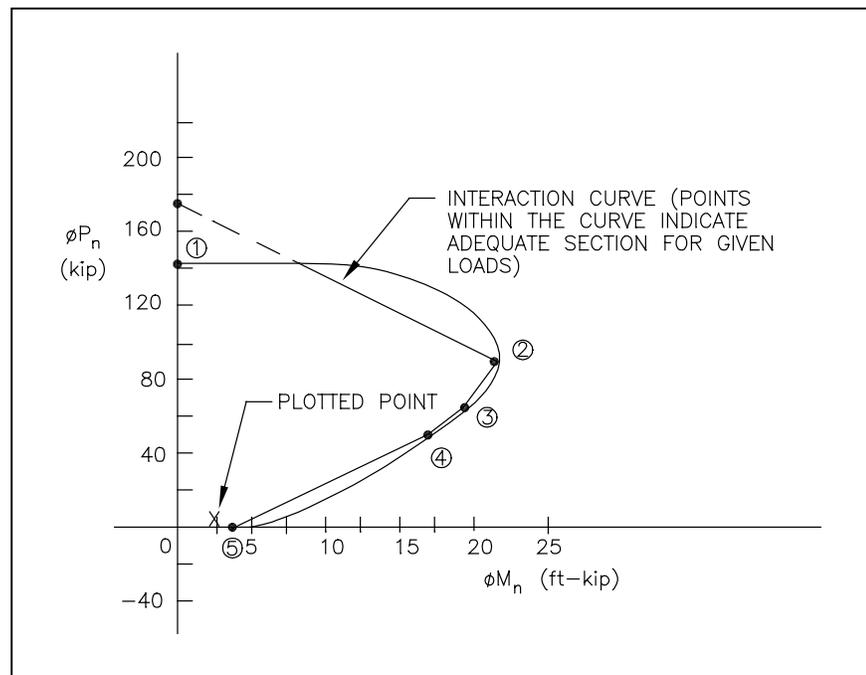
$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.155 \text{ in}^2)(60,000 \text{ psi})}{0.85 (3,000 \text{ psi})(12 \text{ in})} = 0.304 \text{ in}$$
$$\phi M_n = \phi A_s f_y (d - 0.5a)$$
$$= 0.9 (0.155 \text{ in}^2)(60,000 \text{ psi})(5 \text{ in} - 0.5(0.304 \text{ in})) = 40,578 \text{ in-lb/lf} = 3.4 \text{ ft-kip/lf}$$
$$\phi P_n = 0 \quad (3.4, 0)$$

6. Plot the previously calculated points on a graph to determine the interaction diagram boundary for one No. 5 bar at 24 inches on-center vertically in the given wall

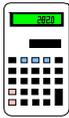
- PT 1: (0,142)
- PT 2: (21.8,91)
- PT 3: (19.3,64)
- PT 4: (16.7,47)
- PT 5: (3.4,0)
- PT X: (2.5,2.5)

Conclusion

The point in question lies within the interaction diagram and the references axes; therefore, one No. 5 bar at 24 inches on-center vertically is adequate for the given loading conditions and wall geometry.



Interaction Diagram

**EXAMPLE 4.8****Concrete Lintel****Given**

$$\begin{aligned}
 f'_c &= 3,000 \text{ psi} \\
 f_y &= 60,000 \text{ psi} \\
 \text{Dead load} &= 250 \text{ plf} \\
 \text{Live load} &= 735 \text{ plf} \\
 \text{Span} &= 6.5 \text{ ft} \\
 \text{Lintel width} &= 8 \text{ in} \\
 \text{Lintel depth} &= 12 \text{ in}
 \end{aligned}$$

Find Minimum reinforcement required**Solution**

- Determine reinforcement required for flexure

$$\phi M_n \geq M_u$$

$$M_u = \frac{wl^2}{12} = \frac{1.2(250 \text{ plf}) + 1.6(735 \text{ plf})}{12} (6.5 \text{ ft})^2 = 5,197 \text{ ft-lb}$$

$$\phi M_n = \phi A_s f_y (d - 0.5a)$$

$$d = 12\text{-in depth} - 1.5\text{-in cover} - 0.375\text{-in stirrup} = 10.125 \text{ in}$$

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

$$\text{set } M_u = \phi M_n \text{ to solve for } A_s$$

$$M_u = \phi A_s f_y \left(d - \frac{1}{2} \left(\frac{A_s f_y}{0.85 f'_c b} \right) \right)$$

$$62,364 \text{ in-lb} = (0.9) A_s (60,000 \text{ psi}) \left(10.125 \text{ in} - 0.5 \left(\frac{A_s (60,000 \text{ psi})}{0.85 (3,000 \text{ psi})(12 \text{ in})} \right) \right)$$

$$0 = 546,750 A_s - 52,941 A_s^2 - 62,364$$

$$A_{s,\text{required}} = 0.115 \text{ in}^2$$

$$\therefore \text{Use one No. 4 bar } (A_s = 0.20 \text{ in}^2)$$

Check reinforcement ratio

$$\rho = \frac{A_s}{bd} = \frac{0.2 \text{ in}^2}{(10.125 \text{ in})(8 \text{ in})} = 0.0025$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \left(\frac{87,000}{f_y + 87,000} \right) = \frac{0.85(3,000 \text{ psi})(0.85)}{60,000 \text{ psi}} \left(\frac{87,000}{60,000 \text{ psi} + 87,000} \right) = 0.021$$

$$\rho_{\text{max}} = 0.75 \rho_b = 0.75(0.021) = 0.016$$

$$\rho_{\text{min}} = 0.0012$$

Since $\rho_{\text{max}} \geq \rho \geq \rho_{\text{min}}$ OK



2. Determine shear reinforcement

$$\phi V_n \geq V_u$$

$$V_u = \frac{wl}{2} = \frac{1.2(250 \text{ plf}) + 1.6(735 \text{ plf})}{2} = (6.5 \text{ ft}) = 4,797 \text{ lb}$$

$$\text{Span-to-depth ratio, } \frac{l}{h} = \frac{(6.5 \text{ ft})(12 \text{ in/ft})}{12 \text{ in}} = 6.5 > 5 \quad \therefore \text{Regular beam}$$

$$\phi V_n = \phi V_c + 0 = \phi 2\sqrt{f'_c} b_w d = (0.85)(2)\sqrt{3,000 \text{ psi}}(8 \text{ in})(10.125 \text{ in}) = 7,542 \text{ lb}$$

$$V_u \leq \frac{\phi V_c}{2} = \frac{7,542 \text{ lb}}{2} = 3,771 \text{ lb} < 4,797 \text{ lb}$$

\therefore Stirrups are required

Since $\phi V_c > V_u > \frac{\phi V_c}{2}$ only the minimum shear reinforcement must be provided.

$$A_{v,\min} = \frac{50 b_w s}{f_y} = \frac{(50)(8 \text{ in}) \left(\frac{10.125 \text{ in}}{2}\right)}{60,000 \text{ psi}}$$

$$= 0.034 \text{ in}^2$$

\therefore Use No. 3 bars

Shear reinforcement is not needed when $\frac{\phi V_c}{2} > V_u$

$$3,771 \text{ lb} = 4,797 \text{ lb} - [1.2(250 \text{ plf}) + 1.6(735 \text{ plf})]x$$

$$x = 0.70 \text{ ft}$$

Supply No. 3 shear reinforcement spaced 5 in on-center for a distance 0.7 ft from the supports.

3. Check deflection

Find x for transformed area

$$h x \left(\frac{x}{2}\right) = n A_s (d - x)$$

$$0.5(8 \text{ in})(x)^2 = \left(\frac{29,000,000 \text{ psi}}{3,122,019 \text{ psi}}\right)(0.2 \text{ in}^2)(10.125 \text{ in} - x)$$

$$0 = 4x^2 + 1.86x - 18.8$$

$$x = 1.95 \text{ in}$$

Calculate moment of inertia for cracked section and gross section

$$I_{CR} = \frac{1}{3} h x^3 + n A_s (d - x)^2$$

$$= \frac{1}{3} (8 \text{ in})(1.95 \text{ in})^3 + (9.29)(0.2 \text{ in}^2)(10.125 \text{ in} - 1.95 \text{ in})^2 = 144 \text{ in}^4$$

$$I_g = \frac{1}{12} b h^3 = \frac{1}{12} (8 \text{ in})(12 \text{ in})^3 = 1,152 \text{ in}^4$$

Calculate modulus of rupture

$$f_r = 7.5 \sqrt{f'_c} = 7.5 \sqrt{3,000 \text{ psi}} = 411 \text{ psi}$$



Calculate cracking moment

$$M_{cr} = \frac{f_r I_g}{Y_t} = \frac{(411 \text{ psi})(1,152 \text{ in}^4)}{(0.5)(12 \text{ in})} = 78,912 \text{ in-lb/lf} = 6.6 \text{ kip-ft/lf}$$

Calculate effective moment of inertia

Since the cracking moment M_{cr} is larger than the actual moment M_u the section is not cracked; thus, $I_e = I_g$.

Calculate deflection

$$\rho_{allow} = \frac{l}{240} = \frac{(6.5 \text{ ft})(12 \text{ in/ft})}{240} = 0.33 \text{ in}$$

$$\rho_{actual} = \frac{5 w l^4}{384 E_c I_e}$$

$$\rho_{i(LL)} = \frac{5(735 \text{ plf})(6.5 \text{ ft})^4}{384(3,122,019 \text{ psi})(1,152 \text{ in}^4)(\text{ft}^3 / 1,728 \text{ in}^3)} = 0.008 \text{ in}$$

$$\rho_{i(DL+20\%LL)} = \frac{5(250 \text{ plf} + (0.20)735 \text{ plf} + (150 \text{ pcf})(0.66 \text{ ft})(1 \text{ ft}))(6.5 \text{ ft})^4}{384(3,122,019 \text{ psi})(1,152 \text{ in}^4)(\text{ft}^3 / 1,728 \text{ in}^3)} = 0.006 \text{ in}$$

$$\begin{aligned} \Delta_{LT} &= \Delta_{i(LL)} + \lambda \Delta_{i(DL+20\%LL)} \\ &= 0.008 \text{ in} + 2(0.0055 \text{ in}) = 0.02 \text{ in} \end{aligned}$$

$$\rho_{LT} \ll \rho_{allow} \quad \text{OK}$$

Conclusion

The minimum reinforcement bar required for an 8-inch x 12-inch concrete lintel spanning 6.5 feet is one No. 4 bar.

**EXAMPLE 4.9****Unreinforced Masonry Wall Design****Given**

Live load	= 1,300 plf
Dead load	= 900 plf
Weight of wall	= 52.5 psf
Moment at top	= 0
Masonry weight	= 120 pcf
Backfill material	= 30 pcf
f'_m	= 1,900 psi
Face shell mortar bedding	

Assume axial load is in middle one-third of wall

Find

Verify if a 10-in-thick unreinforced masonry wall is adequate for the ACI-530 load combination

$$U = D+H$$

Solution

- Determine loads

Equivalent fluid density of backfill soil (Chapter 3)

$$q_s = K_a w = (0.30)(100 \text{ pcf}) = 30 \text{ pcf}$$

Total lateral earth load

$$R = \frac{1}{2} q_s l^2 = \frac{1}{2} (30 \text{ pcf})(4 \text{ ft})^2 = 240 \text{ plf}$$

$$x = \frac{1}{3} \ell = \frac{1}{3} (4 \text{ ft}) = 1.33 \text{ ft}$$

Maximum shear occurs at bottom of wall

$$\Sigma M_{\text{top}} = 0$$

$$V_{\text{bottom}} = \frac{q l^2}{2} - \frac{q l^3}{6L} = \frac{30 \text{ pcf} (4 \text{ ft})^2}{2} - \frac{30 \text{ pcf} (4 \text{ ft})^3}{6 (8 \text{ ft})} = 200 \text{ plf}$$

Maximum moment and its location

$$x_m = \frac{q l - \sqrt{q^2 l^2 - 2q V_{\text{bottom}}}}{q}$$

$$x_m = \frac{30 \text{ pcf} (4 \text{ ft}) - \sqrt{(30 \text{ pcf})^2 (4 \text{ ft})^2 - 2 (30 \text{ pcf}) (200 \text{ plf})}}{(30 \text{ pcf})}$$

$$= 2.37 \text{ ft from base of wall}$$

$$M_{\text{max}} = -\frac{q l x_m}{2} + \frac{q x_m^3}{6} + V_{\text{bottom}} (x_m)$$

$$= -\frac{30 \text{ pcf} (4 \text{ ft})(2.37 \text{ ft})^2}{2} + \frac{(30 \text{ pcf})(2.37 \text{ ft})^3}{6} + 200 \text{ plf} (2.37 \text{ ft})$$

$$= 204 \text{ ft-lb/lf}$$



2. Check perpendicular shear

$$\frac{M}{Vd} = \frac{204 \text{ ft-lb / lf } (12 \text{ in / ft})}{200 \text{ plf } (9.625 \text{ in})} = 1.27 > 1$$

$$F_v = \begin{cases} 1.5\sqrt{f'_m} = 1.5\sqrt{1,900 \text{ psi}} = 65.4 \text{ psi} \\ 120 \text{ psi} \\ 37 \text{ psi} + 0.45 \frac{N_v}{A_n} = 37 \text{ psi} + 0.45 \frac{(900 \text{ plf} + 52.5 \text{ psf } (8 \text{ ft} - 2.37 \text{ ft}))}{33 \text{ in}^2} = 53.3 \text{ psi} \end{cases}$$

$$F_v = 53.3 \text{ psi}$$

$$f_v = \frac{3}{2} \left(\frac{V}{A_n} \right) = 1.5 \left(\frac{200 \text{ plf}}{(2 \text{ face shells})(1.375 \text{ in})(12 \text{ in})} \right) = 9.1 \text{ psi}$$

The shear is assumed to be resisted by 2 face shells since the wall is unreinforced and uncracked.

$$f_v < F_v \quad \text{OK}$$

3. Check axial compression

$$A_n = \ell(2b) = (12 \text{ in})(2)(1.375 \text{ in}) = 33 \text{ in}^2$$

$$I = \frac{1}{12} bh^3 + Ad^2$$

$$= 2 \left[\frac{1}{12} (12 \text{ in})(1.375 \text{ in})^3 + (12 \text{ in})(1.375 \text{ in}) \left(\frac{9.625 \text{ in}}{2} - \frac{1.375 \text{ in}}{2} \right)^2 \right]$$

$$= 567 \text{ in}^4$$

$$r = \sqrt{\frac{I}{A_n}} = \sqrt{\frac{567 \text{ in}^4}{33 \text{ in}^2}} = 4.14 \text{ in}$$

$$S = \frac{I}{c} = \frac{567 \text{ in}^4}{\frac{1}{2}(9.625 \text{ in})} = 118 \text{ in}^3$$

$$\frac{h}{r} = \frac{8 \text{ ft}(12 \text{ in / ft})}{4.14 \text{ in}} = 23.2 < 99$$

$$F_a = (0.25 f'_m) \left[1 - \left(\frac{h}{140r} \right)^2 \right] = (0.25)(1,900 \text{ psi}) \left[1 - \left(\frac{8 \text{ ft } (12 \text{ in / ft})}{140(4.14 \text{ in})} \right)^2 \right] =$$

$$= 462 \text{ psi}$$

$$P_{\max} = F_a A_n = (462 \text{ psi})(33 \text{ in}^2) = 15,246 \text{ plf}$$

$$P = 900 \text{ plf (given for } U=D+H)$$

$$900 \text{ plf} < 15,246 \text{ plf} \quad \text{OK}$$



Check Euler buckling load

$$E_m = 900f'_m = 900 (1,900 \text{ psi}) = 1.71 \times 10^6 \text{ psi}$$

$$e_k = \frac{S}{A_n} = \frac{118 \text{ in}^3}{33 \text{ in}^2} = 3.57 \text{ in} \quad (\text{kern eccentricity})$$

$$P_e = \frac{\pi^2 E_m I}{h^2} \left(1 - 0.577 \frac{e}{r} \right)^3$$

$$= \frac{\pi^2 (1.71 \times 10^6 \text{ psi})(567 \text{ in}^4)}{(8 \text{ ft})^2 (12 \text{ in / ft})^2} \left(1 - 0.577 \left(\frac{3.57 \text{ in}}{4.14 \text{ in}} \right) \right)^3$$

$$= 131,703 \text{ plf}$$

$$P \leq 0.25P_e \quad \text{OK}$$

Euler buckling loads are calculated by using actual eccentricities from gravity loads without including effects of lateral loads.

4. Check combined axial compression and flexural capacity

$$M = 204 \text{ ft-lb/lf}$$

$$P = 900 \text{ plf}$$

$$\text{virtual eccentricity} \quad e = \frac{M}{P} = \frac{204 \text{ ft-lb / lf} (12 \text{ in / ft})}{900 \text{ plf}} = 2.72 \text{ in}$$

$$\text{kern eccentricity} \quad e_k = \frac{S}{A_n} = \frac{118 \text{ in}^3}{33 \text{ in}^2} = 3.57 \text{ in} \quad \text{! GOVERNS}$$

$e < e_k \quad \therefore$ Assume section is uncracked

$$P_e = \frac{\pi^2 E_m I}{h^2} \left(1 - 0.577 \frac{e}{r} \right)^3$$

$$= \frac{\pi^2 (900 \text{ plf})(1,900 \text{ psi})(567 \text{ in}^4)}{(8 \text{ ft} (12 \text{ in / ft}))^2} \left(1 - 0.577 \left(\frac{3.57}{4.14} \right) \right)^3$$

$$P_e = 131,703 \text{ plf}$$

$$P < 0.25 (131,703 \text{ plf}) = 32,926 \text{ plf} \quad \text{OK}$$

$$f_a = \frac{P}{A_n} = \frac{900 \text{ plf}}{33 \text{ in}^2} = 27 \text{ psi}$$

$$f_b = \frac{M}{S} = \frac{(900 \text{ plf})(3.57 \text{ in}) \left(\frac{2.37 \text{ ft}}{8 \text{ ft}} \right) + (204 \text{ ft-lb / lf})(12 \text{ in / ft})}{118 \text{ in}^3}$$

$$= 29 \text{ psi}$$

$$F_a = 462 \text{ psi for } h/r \leq 99$$

$$F_b = 0.33 f'_m = 0.33 (1,900 \text{ psi}) = 627 \text{ psi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{27 \text{ psi}}{462 \text{ psi}} + \frac{29 \text{ psi}}{627 \text{ psi}} = 0.10 \leq 1 \quad \text{OK}$$



5. Check tension capacity from Table 2.2.3.2 for normal to bed joints, hollow, ungrouted (Type M or S mortar)

$$F_t \leq 25 \text{ psi}$$

$$f_t = -\frac{P}{A_n} + \frac{M}{S} = -\frac{900 \text{ plf}}{33 \text{ in}^2} + \frac{3,400 \text{ ft-lb/lf}}{118 \text{ in}^3} = 1.54 \text{ psi}$$

$$f_t < F_t \quad \text{OK}$$

6. Minimum reinforcement

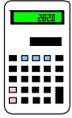
Horizontal reinforcement at 24 inches on-center vertically.

Conclusion

An unreinforced masonry wall is adequate for the ACI-530 load combination evaluated; however, horizontal reinforcement at 24 inches on-center may be optionally provided to control potential shrinkage cracking, particularly in long walls (i.e., greater than 20 to 30 feet long).

If openings are present, use lintels and reinforcement as suggested in Sections 4.5.2.3 and 4.5.2.4.

Note that the calculations have already been completed and that the maximum backfill height calculated for an 8-inch-thick unreinforced masonry wall using hollow concrete masonry is about 5 feet with a safety factor of 4.

**EXAMPLE 4.10****Reinforced Masonry Foundation Wall Design****Given**

Live load = 1,300 plf
 Dead load = 900 plf
 Moment at top = 0
 Masonry weight = 120 pcf
 Wall weight = 52.5 psf
 Backfill material = 45 pcf
 $f'_m = 2,000$ psi
 Face shell mortar bedding
 Type M or S mortar
 Wall is partially grouted, one core is grouted at 24 inches on-center
 Assume axial load is in middle one-third of wall

Find

Verify if one vertical No. 5 bar at 24 inches on-center is adequate for a reinforced concrete masonry foundation wall that is 8 feet high with 7 feet of unbalanced backfill for the ACI-530 load combination

$$U = D + H$$

Solution

- Determine loads

Equivalent fluid density of backfill soil (refer to Chapter 3)

$$q = K_a W = (0.45)(100) = 45 \text{ pcf}$$

Total lateral earth load

$$R = \frac{1}{2} q l^2 = \frac{1}{2} (45 \text{ pcf})(7 \text{ ft})^2 = 1,103 \text{ lb}$$

$$X = \frac{1}{3} \ell = \frac{1}{3} (7 \text{ ft}) = 2.33 \text{ ft}$$

Maximum shear occurs at bottom of wall

$$\sum M_{\text{top}} = 0$$

$$\begin{aligned}
 V_{\text{bottom}} &= \frac{q l^2}{2} - \frac{q l^3}{6L} = \frac{45 \text{ pcf} (7 \text{ ft})^2}{2} - \frac{(45 \text{ pcf}) (7 \text{ ft})^3}{6 (8 \text{ ft})} \\
 &= 781 \text{ plf}
 \end{aligned}$$

Maximum moment and its location

$$\begin{aligned}
 x_m &= \frac{q l - \sqrt{q^2 l^2 - 2q V_{\text{bottom}}}}{q} \\
 &= \frac{(45 \text{ pcf}) (7 \text{ ft}) - \sqrt{(45 \text{ pcf})^2 (7 \text{ ft})^2 - 2(45 \text{ pcf})(781 \text{ plf})}}{45 \text{ pcf}} \\
 &= 3.2 \text{ ft from base of wall}
 \end{aligned}$$



$$\begin{aligned}
 M_{\max} &= \frac{q_l x_m^2}{2} + \frac{q x_m^3}{6} + V_{\text{bottom}}(x_m) \\
 &= \frac{-45 \text{ pcf} (7 \text{ ft})(3.2 \text{ ft})^2}{2} + \frac{(45 \text{ pcf})(3.2 \text{ ft})^3}{6} + (781 \text{ plf})(3.2 \text{ ft}) \\
 &= 1,132 \text{ ft-lb/lf}
 \end{aligned}$$

2. Check perpendicular shear

$$\frac{M}{Vd} = \frac{1,132 \text{ ft-lb/lf} (12 \text{ in/ft})}{(781 \text{ plf})(9.625 \text{ in})} = 1.8 > 1$$

$$\begin{aligned}
 F_v &= 1 \sqrt{f'_m} \leq 50 \text{ psi} \\
 &= 1 \sqrt{2,000 \text{ psi}} = 44.7 \text{ psi} < 50 \text{ psi}
 \end{aligned}$$

$$F_v = (44.7 \text{ psi})(2\text{-ft grouted core spacing}) = 89 \text{ psi}$$

$$\begin{aligned}
 A_e &= A_{\text{CMU faceshells}} + A_{\text{core}} \\
 &= (24 \text{ in} - 8.375 \text{ in})(2)(1.375 \text{ in}) + (1.125 \text{ in} + 1.375 \text{ in} + 5.875 \text{ in})(9.625 \text{ in}) \\
 &= 124 \text{ in}^2
 \end{aligned}$$

$$f_v = \frac{V}{bd} = \frac{V}{A_e} = \frac{(781 \text{ plf})(2\text{ft rebar spacing})}{(124 \text{ in}^2)} = 13 \text{ psi}$$

$$f_v < F_v \text{ OK}$$

This assumes that both mortared face shells are in compression.

3. Check parallel shear

Foundation walls are constrained against lateral loads by the passive pressure of the soil and soil-wall friction. Parallel shear on the foundation wall can be neglected by design inspection.

4. Check axial compression

$$\begin{aligned}
 A_e &= 124 \text{ in}^2 \\
 I &= \frac{1}{12} b h^3 + A d^2 \\
 &= \frac{1}{12} (8.375 \text{ in})(9.625 \text{ in} - 2(1.375 \text{ in})) \\
 &\quad + 2 \left[\left(\frac{1}{12} \right) (24 \text{ in})(1.375 \text{ in})^3 + (24 \text{ in})(1.375 \text{ in}) \left(\frac{9.625 \text{ in}}{2} - \frac{1.375 \text{ in}}{2} \right)^2 \right] \\
 &= 1,138 \text{ in}^4
 \end{aligned}$$

$$r = \sqrt{\frac{I}{A_e}} = \sqrt{\frac{1,138 \text{ in}^4}{124 \text{ in}^2}} = 3.03 \text{ in}$$

$$\frac{h}{r} = \frac{8 \text{ ft} (12 \text{ in/ft})}{3.03 \text{ in}} = 32 < 99$$

$$\begin{aligned}
 \therefore F_a &= (0.25 f'_m) \left(1 - \left(\frac{h}{140r} \right)^2 \right) \\
 &= 0.25 (2,000 \text{ psi}) \left(1 - \left(\frac{(8 \text{ ft})(12 \text{ in/ft})}{140(3.03 \text{ in})} \right)^2 \right) = 474 \text{ psi}
 \end{aligned}$$

$$P_{\max} = F_a A_e = (474 \text{ psi})(124 \text{ in}^2) = 58,776 \text{ lb}$$

$$P = 900 \text{ lb}$$

$$P < P_{\max} \text{ OK}$$



5. Check combined axial compression and flexural capacity

$$M = 1,132 \text{ ft-lb/lf}$$

$$P = 900 \text{ plf}$$

$$\begin{aligned} \text{virtual eccentricity} = e &= \frac{M}{P} \\ &= \frac{1,132 \text{ ft-lb/lf}(12 \text{ in / ft})}{900 \text{ plf}} = 15 \text{ in} \quad \text{!Governs} \end{aligned}$$

$$\begin{aligned} \text{kern eccentricity} = e_k &= \frac{S}{A_e} \\ &= \frac{1,138 \text{ in}^4 / 0.5(9.625 \text{ in})}{124 \text{ in}^2} = 1.9 \text{ in} \end{aligned}$$

$e > e_k$ \therefore Tension on section, assume cracked

$$f_a = \frac{P}{A_e} = \frac{900 \text{ plf}(2 \text{ ft})}{124 \text{ in}^2} = 14.5 \text{ psi}$$

$$f_b = \frac{M}{S} = \frac{1,132 \text{ ft-lb/lf}(12 \text{ in / ft})}{236.5 \text{ in}^3} = 57 \text{ psi}$$

$$f_b > f_a$$

\therefore Assume section is cracked

$$\begin{aligned} F_a &= 0.25 f'_m \left[1 - \left(\frac{h}{140r} \right)^2 \right] \\ &= 0.25 (2,000 \text{ psi}) \left[1 - \left(\frac{8 \text{ ft}(12 \text{ in / ft})}{140(3.03 \text{ in})} \right)^2 \right] \\ &= 474 \text{ psi} \end{aligned}$$

$$F_b = 0.33 f'_m = 0.33 (2,000 \text{ psi}) = 660 \text{ psi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{14.5 \text{ psi}}{474 \text{ psi}} + \frac{57 \text{ psi}}{660 \text{ psi}} = 0.12 \leq 1 \quad \text{OK}$$

6. Minimum steel requirement

$$\begin{aligned} A_{s,\text{req'd}} &= \frac{M}{F_s d} \\ &= \frac{(1,132 \text{ ft-lb/lf})(12 \text{ in / ft})}{(24,000 \text{ psi})(0.5)(9.625 \text{ in})} \\ &= 0.12 \text{ in}^2/\text{lf} \end{aligned}$$

Minimum vertical reinforcement

$$\begin{aligned} A_{s,\text{min}} &= 0.0013 \text{ bt} \\ &= (0.0013 \text{ in}^2/\text{lf})(12 \text{ in})(9.625 \text{ in}) = 0.15 \text{ in}^2/\text{lf} \quad \text{!Governs} \\ \text{No. 5 at 24 inches on-center } (A_s &= 0.3 \text{ in}^2(12 \text{ in}/24 \text{ in}) = 0.155 \text{ in}^2) \\ A_{s,\text{actual}} &> A_{s,\text{required}} \quad \text{OK} \end{aligned}$$



Minimum horizontal reinforcement

$$\begin{aligned}A_{v,hor} &= 0.0007 bt \\ &= 0.0007 (12 \text{ in})(9.625 \text{ in}) = 0.081 \text{ in}^2/\text{lf}\end{aligned}$$

Use truss-type reinforcement at 24 inches on-center or one No. 5 bar at 48 inches on center ($A_s = 0.08 \text{ in}^2/\text{lf}$)

7. Check tension

$$\begin{aligned}M_t &= A_s d F_s \\ &= (0.155 \text{ in}^2)(0.5)(9.625 \text{ in})(24,000 \text{ psi}) \\ &= 17,903 \text{ in-lb/lf} \\ M &= (1,132 \text{ ft-lb/lf})(12 \text{ in/ft}) \\ &= 13,584 \text{ in-lb/lf}\end{aligned}$$

$$M < M_t \quad \text{OK}$$

Conclusion

One vertical No. 5 bar at 24 inches on-center is adequate for the given loading combination. In addition, horizontal truss type reinforcement is recommended at 24 inches (i.e., every third course of block).

Load combination D+H controls design. Therefore, a check of D+L+H is not shown.

Table 4.5 would allow a 10-inch-thick solid unit masonry wall without rebar in soil with 30 pcf equivalent fluid density. This practice has succeeded in residential construction except as reported in places with “heavy” clay soils. Therefore, a design as shown in this example may be replaced by a design in accordance with the applicable residential codes’ prescriptive requirements. The reasons for the apparent inconsistency may be attributed to a conservative soil pressure assumption or a conservative safety factor in ACI-530 relative to typical residential conditions.



4.10 References

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