

ACI 551.2R-15

Design Guide for Tilt-Up Concrete Panels

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Design Guide for Tilt-Up Concrete Panels

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Design Guide for Tilt-Up Concrete Panels

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This guide presents information that expands on the provisions of ACI 318 applied to the design of site-cast precast, or tilt-up, concrete panels, and provides a comprehensive procedure for the design of these important structural elements. In addition, this guide provides design recommendations for various support and load conditions not specifically covered in ACI 318, including design guidelines for in-plane shear.

Keywords: panel; panel design; panel lifting; precast; reinforcement design; seismic design of tilt-up; slender wall analysis; tilt-up; tilt-up design, tilt-up detailing.

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CHAPTER 1—INTRODUCTION

Tilt-up concrete buildings have been constructed in North America for over 100 years, but it was not until the late 1990s that ACI 318 specifically addressed the requirements for design of slender concrete walls. ACI 318-11, 14.8, provides a method of analysis and covers only the basic requirements for evaluating the effects of vertical and transverse out-of-plane loads. ACI 318-11, Chapter 10, may also be used to design slender walls, but the requirements are more general and should be applied with discretion.

This guide expands on the provisions of ACI 318-11, Section 14.8, and ASCE/SEI 7 and provides a comprehensive procedure for the design of these structural elements. This guide also provides design recommendations for various support and load conditions not specifically covered in ACI 318, and includes design guidelines for in-plane shear.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

- A_g = gross area of concrete section, in.² (mm²)
- A_s = area of tension reinforcement, in.² (mm²)
- A_{se} = effective area of tension reinforcement, in.² (mm²)
- A_v = area of shear reinforcement, in.² (mm²)
- a = depth of equivalent rectangular stress block, in. (mm)
- b_d = design width, in. (mm)
- b_t = tributary width, in. (mm)
- b_w = width of the concrete section, in. (mm)
- c = distance from the extreme fiber to the neutral axis, in. (mm)
- D = dead load
- d = distance from the extreme concrete compression fiber to the centroid of tension reinforcement, or the effective depth of section, in. (mm)
- d_t = distance from the extreme compression fiber to centroid of extreme layer of longitudinal tension steel, in. (mm)
- E = loads due to seismic force
- E_c = concrete modulus of elasticity, psi (MPa)
- E_s = steel modulus of elasticity, psi (MPa)
- e_{cc} = eccentricity of applied load(s), in. (mm)
- F = loads due to weight or pressure of fluids
- F_p = factored load
- f'_c = specified compressive strength of concrete, psi (MPa)
- f_r = modulus of rupture, psi (MPa)
- f_y = reinforcement yield stress, psi (MPa)
- GC_p = external pressure coefficient
- GC_{pi} = internal pressure coefficient
- H = horizontal line load or soil pressure
- h = panel thickness, in. (mm)
- I_e = importance factor
- I_{cr} = cracked section moment of inertia, in.⁴ (mm⁴)
- I_e = effective moment of inertia, in.⁴ (mm⁴)

K_b	= bending stiffness, in.-lb/in. (N·mm/mm)	W	= wind load
K_d	= wind directionality factor	W_a	= wind load based on serviceability wind speed
K_z	= velocity pressure coefficient at height z	W_c	= panel self-weight
K_{zt}	= topographic factor	W_{floor}	= weight of tributary floor structure
L	= live load	W_{panel}	= weight of panel
L_r	= roof live load	W_{roof}	= weight of tributary roof structure
ℓ	= vertical span of member between support	w	= uniform lateral load
ℓ_c	= cantilever height	w_c	= factored self-weight of concrete wall panel above the base
ℓ_{floor}	= distance from floor diaphragm to the bottom of panel, in. (mm)	w_u	= factored uniform lateral load on element
ℓ_{main}	= distance from main floor to bottom of panel, in. (mm)	z	= mean roof height
ℓ_{panel}	= distance from panel center of gravity to the bottom of panel, in. (mm)	δ_b	= moment magnification factor
ℓ_{roof}	= distance from roof diaphragm to the bottom of panel, in. (mm)	γ_c	= unit weight of concrete, lb/ft ³ (kg/m ³)
ℓ_w	= width of concrete section, in. (mm)	ρ_l	= ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement
M_a	= maximum moment at midheight of wall due to service lateral and eccentric vertical loads, including P - Δ effects, in.-lb (N·mm)	ρ_t	= ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement
M_{cr}	= moment causing flexural cracking of the concrete section, in.-lb (N·mm)	ϕ	= strength reduction factor
M_{max}	= maximum moment occurring over the span of the panel due to uniform lateral loads, in.-lb (N·mm)	Δ_i	= initial deflection at midheight, in. (mm)
M_n	= nominal moment strength at the midheight cross section due to service lateral and eccentric vertical loads only, in.-lb (N·mm)	Δ_{max}	= maximum total deflection at midheight, in. (mm)
M_u	= maximum factored combined bending moment, in.-lb (N·mm)	Δ_n	= maximum potential deflection at midheight, in. (mm)
M_{ua}	= maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including P - Δ effects, in.-lb (N·mm)	Δ_s	= maximum out-of-plane deflection due to service loads, including P - Δ effects, in. (mm)
n	= modular ratio		
P	= applied axial load at top of panel		
P - Δ	= secondary moment caused by axial load P acting on a deflected shape with displacement Δ , in.-lb (N·mm)		
P_{cr}	= critical buckling load		
P_u	= factored axial load		
q_z	= effective velocity pressure at mean roof height z , lb/ft ² (N·m ²)		
R	= rain load in 4.4		
R	= vertical reaction at footing in 8.1		
R	= seismic response modification coefficient in 8.4		
S	= snow load		
S_{DS}	= short-period design spectral response acceleration		
S_{MS}	= maximum considered spectral response acceleration		
S_S	= mapped short-period spectral acceleration		
s	= spacing of transverse shear reinforcement, in. (mm)		
T	= cumulative effects of temperature, creep, shrinkage, settlement		
V_c	= nominal shear strength of normalweight concrete, lb (N)		
V_{floor}	= floor diaphragm shear force		
V_n	= total shear strength of the concrete section, lb (N)		
V_{panel}	= panel shear force (seismic only)		
$V_{R main}$	= resisting shear force at main floor		
V_{roof}	= roof diaphragm shear force		
V_s	= nominal shear strength of the reinforcement, lb (N)		

2.2—Definitions

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

compressive strength—measured maximum resistance of a concrete specimen to axial compressive loading; expressed as force per unit cross-sectional area.

compressive stress—stress directed toward the part on which it acts.

connection—a region that joins two or more members.

modulus of elasticity—ratio of normal stress to corresponding strain for tensile or compressive stress below the proportional limit of the material; also called elastic modulus, Young’s modulus, and Young’s modulus of elasticity; denoted by the symbol E .

moment frame—frame in which members and joints resist forces through flexure, shear, and axial force.

net tensile strain—tensile strain at nominal strength exclusive of strains due to effective prestress, creep, shrinkage, and temperature.

seismic-force-resisting system—portion of the structure designed to resist earthquake design forces required by the legally adopted general building code using the applicable provisions and load combinations.

tensile stress—stress directed away from the part on which it acts.

tension-controlled section—cross section in which the net tensile strain in the extreme tension fiber at nominal strength is greater than or equal to 0.005.

slender wall—wall, structural or nonstructural, whose thickness-to-height ratio makes it susceptible to secondary

moments from eccentric axial loads and self-weight in addition to primary moments from out-of-plane (lateral) forces.

work—entire construction or separately identified parts thereof that are required to be furnished under contract documents.

CHAPTER 3—ANALYSIS CONCEPTS FOR SLENDER CONCRETE WALLS

3.1—Panel design model

Tilt-up concrete wall panels most often serve as load-bearing wall elements spanning vertically from the foundation or slab-on-ground to intermediate floor(s), roof, or both. Bending moments result from out-of-plane loads, eccentric axial loads, or both. Second-order bending effects resulting from axial load acting on a deflected panel shape will cause an increase in these moments, also known as the $P-\Delta$ effect. Ultimate strength failure of a slender wall panel is defined to occur when the maximum factored bending moment at or near midheight exceeds the nominal strength of the concrete section times a strength reduction factor.

The maximum bending moment can be separated into two components: 1) primary moment due to applied loads; and 2) secondary moment due to $P-\Delta$ effects.

Figure 3.1 illustrates the effects of these moments. Primary moments are obtained from applied loads, such as lateral wind or seismic pressures, and eccentric axial loads. In most cases, it is the moment at midheight that is of concern because this is normally where maximum $P-\Delta$ effects and eventual failure will occur. The following contribute to the primary moment in the panel:

- (a) Eccentric axial loads
- (b) Out-of-plane lateral loads (wind or seismic)
- (c) Initial lateral deflections due to panel out-of-straightness

Small horizontal displacements of the top of the panel relative to the bottom have little effect on the bending moments and are typically ignored.

The deflection of a wall panel depends on its bending stiffness. For reinforced concrete, this bending stiffness can be difficult to evaluate because it is influenced by a number of parameters, including:

- (a) Wall thickness
- (b) Concrete compressive strength
- (c) Concrete tensile strength
- (d) Area of steel reinforcement
- (e) Location of steel reinforcement in the wall section
- (f) Applied axial load
- (g) Bending curvature

The flexural properties of a concrete section vary in a nonlinear manner with increasing moment. Both strength and stiffness will vary with changes in axial compression and degree of bending curvature. As curvature increases, bending moment increases until concrete crushing or reinforcement yielding occurs. Bending stiffness of the panel remains relatively constant at small curvatures, but abruptly decreases as the concrete cracks in flexural tension. Following this, stiffness essentially does not deteriorate any further until reinforcement yields in tension. In most cases, however, the

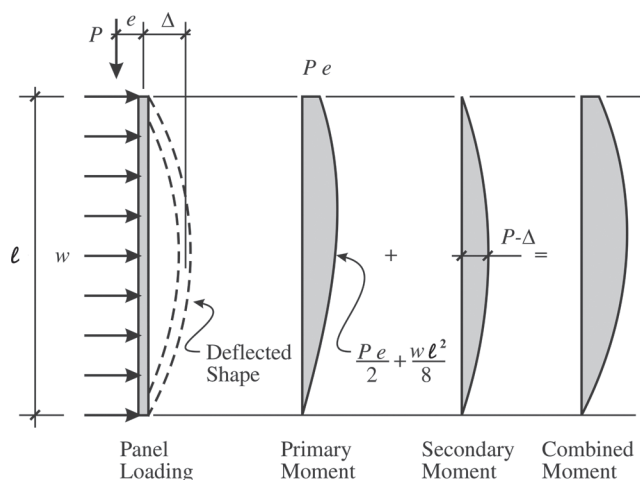


Fig. 3.1—Panel design model—suction force acting with eccentric axial load.

main concern is the failure condition due to factored loads, where the resisting moment and bending stiffness can be determined accurately by simple calculations.

3.2—Bending stiffness evaluation

The relationship between the maximum bending moment in the wall panel and maximum lateral deflection can be expressed by the following ratio

$$K_b = \frac{M_{max}}{\Delta_{max}}$$

Simplify the procedure for calculating deflection and final bending moment by using a constant value for the bending stiffness K_b that will provide reasonable, but conservative, results for the expected range of applied loading.

Where a simply supported slender wall element is subjected to uniform lateral load only (Fig. 3.2a), maximum moment M_{max} will occur at midheight and the maximum deflection Δ_{max} is given by

$$\Delta_{max} = \frac{5w\ell^4}{384E_c I_e} = \frac{5M_{max}\ell^2}{48E_c I_e} = \frac{M_{max}\ell^2}{9.6E_c I_e}$$

in which

$$M_{max} = \frac{w\ell^2}{8}$$

When the same wall element is subjected to a constant moment M along the height due to equal and opposite end moments (Fig. 3.2b), maximum deflection is

$$\Delta_{max} = \frac{M_{max}\ell^2}{8E_c I_e}$$

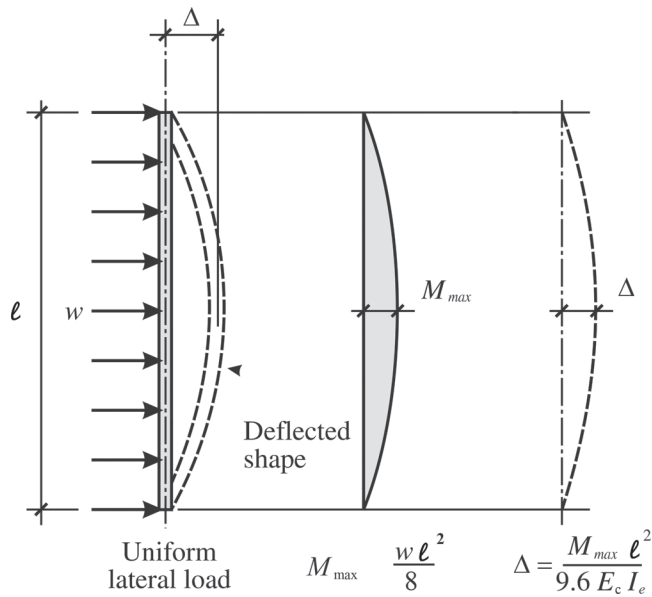


Fig. 3.2a—Maximum deflection due to lateral load only.

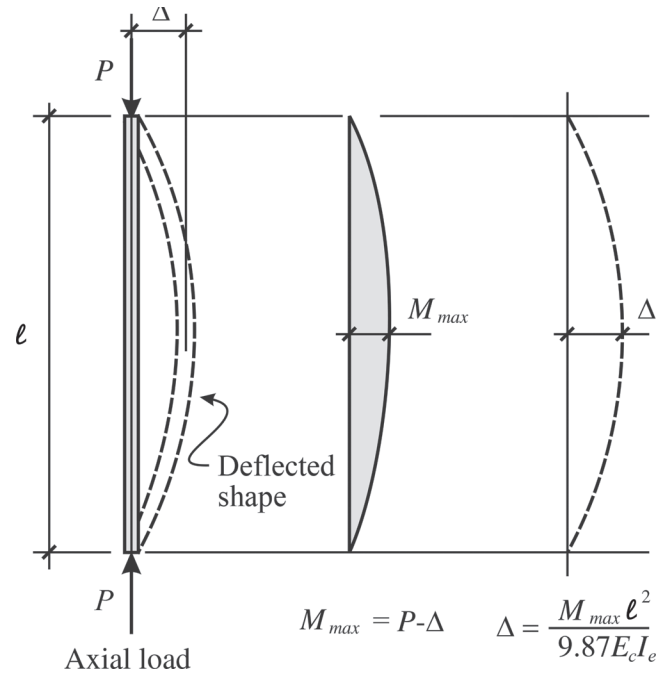


Fig. 3.2c—Deflection due to axial load only.

Maximum moment in a tilt-up panel is usually the result of a combination of these loading conditions. Lateral load effects are often large compared with end moments. Traditional methods for analyzing tilt-up panel walls have adopted the first of the aforementioned relations for deflection calculations

$$\Delta_{max} = \frac{5M_{max}l^2}{48E_cI_e} = \frac{M_{max}}{K_b}$$

Bending stiffness K_b for a slender wall is therefore defined as

$$K_b = \frac{48E_cI_e}{5l^2} = \frac{9.6E_cI_e}{l^2}$$

This will slightly overestimate the deflection and maximum bending moment of a slender wall subjected to the combined effects of lateral and axial load for all axial loads that produce $P-\Delta$ moments larger than the moments produced by lateral loads.

K_b is similar in value to the more familiar term for critical buckling load, P_{cr}

$$P_{cr} = \frac{\pi^2 E_c I_e}{l^2} = 9.87 \frac{E_c I_e}{l^2}$$

Critical buckling capacity is a by-product of the $P-\Delta$ analysis and represents the maximum axial load that can be sustained by a pin-ended slender column or wall in the absence of any other applied loads. The factor $\pi^2 = 9.87$ defines a sinusoidal, single-curve deflected shape due to

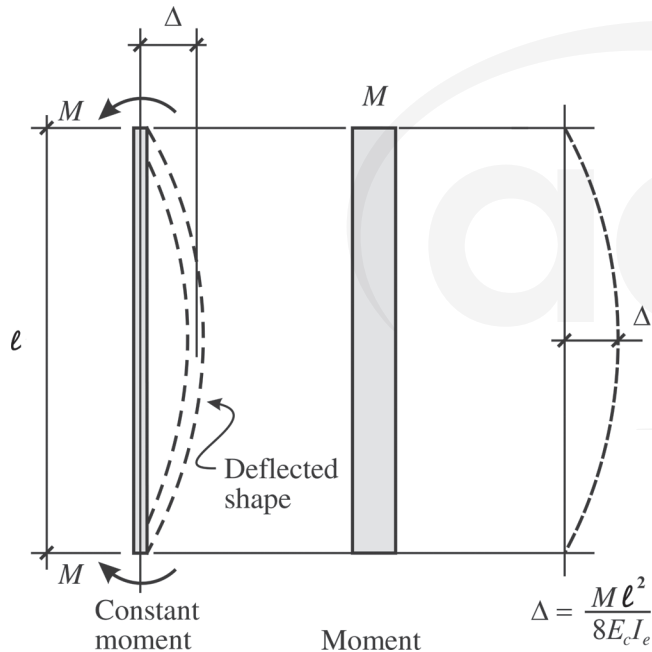


Fig. 3.2b—Maximum deflection due to constant lateral moment.

Where the wall element is subjected to axial load P , only as shown in Fig. 3.2c, plus a small initiating eccentricity, maximum deflection at midheight is given by

$$\Delta_{max} = \frac{M_{max}l^2}{\pi^2 E_c I_e} = \frac{M_{max}l^2}{9.87 E_c I_e}$$

in which

$$M_{max} = P\Delta_{max}$$

the effects of a concentric axial load only. The applicable units for P_{cr} are force (k or kip [N or Newton]) and for K_b , bending moment per unit deflection (ft-kip/ft or in.-kip/in. [N-m/m or N-mm/mm]).

Section stiffness $E_c I_e$ in the preceding equation varies with both axial and lateral loadings, degree of curvature of the panel, and properties of the concrete section. At ultimate load conditions, the concrete section exhibits cracks over most of the panel height. Full-scale testing in California in the early 1980s (SEAOSC 1982) and analytical studies by the SEAOSC Slender Wall Task Group (Lai et al. 2005) verified that a value of $E_c I_e$ equal to the cracked section stiffness $E_c I_{cr}$ correlates closely with the load-deflection characteristic of the test results. Ultimate load deflections using the preceding Δ_{max} equations will likely be overestimated. The cracked section moment of inertia, I_{cr} , can be taken as

$$I_{cr} = n A_{se} (d - c)^2 + \frac{bc^3}{3}$$

where

$$c = a/\beta_1$$

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

$$\beta_1 = 0.85 \text{ for } f'_c \leq 4000 \text{ psi (28 MPa)}$$

$$= 0.85 - 0.05 \left(\frac{f'_c - 4000}{1000} \right) \geq 0.65 \text{ for } f'_c > 4000 \text{ psi (in.-lb units)}$$

$$= 0.85 - 0.05 \left(\frac{f'_c - 28}{7} \right) \geq 0.65 \text{ for } f'_c > 28 \text{ MPa (SI units)}$$

$$n = E_s/E_c$$

Rectangular stress block stiffness has been used because the panel is at the ultimate load state. The development of this relationship, and a comparison to I_{cr} for a triangular concrete stress distribution, is provided in Appendix A.

Applied axial forces will counteract a portion of the flexural tension stresses in the concrete section, resulting in increased bending moment resistance. For small axial stresses less than $0.10f'_c$, this can be accounted for by a simple modification of the area of reinforcement as follows

$$A_{se} = A_s + \frac{P_u}{f_y} \left(\frac{h}{2d} \right)$$

A_{se} can also be used to account for the increased bending stiffness when computing P - Δ deflections.

The assumption that concrete section stiffness is equal to $E_c I_{cr}$ and is constant over the entire height of the panel is considered valid for factored load conditions. The calculation for I_{cr} is based on the value of c for the rectangular concrete stress block that occurs at ultimate loads rather than kd for the triangular stress distribution that occurs at service loads, because the purpose is to compute deflections at ultimate loads. ACI 318 adopted this approach in 1999,

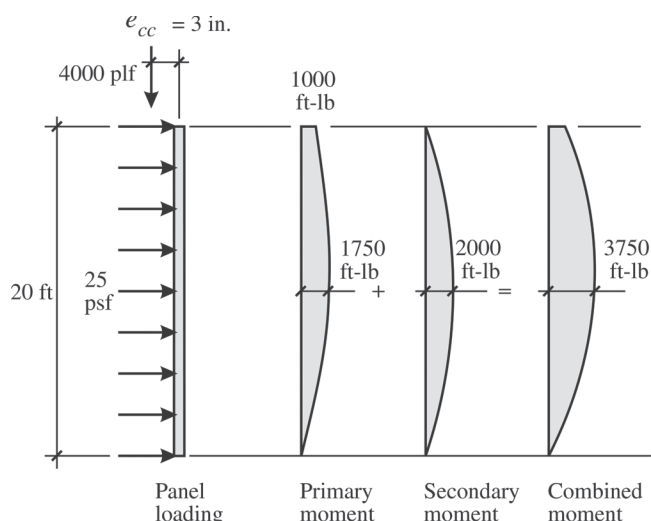


Fig. 3.3—Slender wall example calculation—suction force acting with eccentric axial load. (Note: 1 in. = 24.5 mm; 1 ft = 0.3 m; 1 plf = 1.4 N/m; 1 ft-lb = 15 N-m; 1 lb/ft² = 0.048 kPa.)

which had also been employed in this form by The Uniform Building Code (International Code Council 1997). These assumptions do not introduce significant variations to the final design of a slender tilt-up panel. Appendix A provides a derivation and comparison of the two methods.

3.3—Iteration method for P - Δ effects

As noted in the previous section, the maximum moment M_{max} in a slender wall element typically occurs at or near the midheight section. It is the sum of the applied moment M_a and the P - Δ moment

$$M_{max} = M_a + P\Delta_{max}$$

The relation between maximum bending moment and deflection is

$$\Delta_{max} = \frac{5M_{max}\ell^2}{48E_c I_{cr}} = \frac{M_{max}}{K_b}$$

The solution to the previous two equations can be obtained by a simple iterative procedure. The following example illustrates the method, assuming a 12 in. (300 mm) wide panel strip depicted in Fig. 3.3.

The assumed parameters are:

$$\ell = 20 \text{ ft (6.10 m)}$$

$$w = 25 \text{ lb/ft}^2 (1.2 \text{ kPa})$$

$$P = 4000 \text{ plf (5400 N/m) at top of panel}$$

$$e_{cc} = 3 \text{ in. (76 mm)}$$

$$E_c I_{cr} = 45 \times 10^6 \text{ lb-in.}^2 (129 \times 10^6 \text{ N-mm}^2)$$

$$K_b = \frac{48E_c I_{cr}}{5\ell^2} = \frac{48 \cdot 45 \times 10^6}{5 \cdot 20^2 \times 12^2} = 7500 \text{ in.-lb/in. (33 kN-mm/mm)}$$

$$M_a = 25 \times \frac{20^2}{8} + 4000 \times \frac{3}{2} \times \frac{1}{12} = 1250 + 500 = 1750 \text{ ft-lb (25.5 kN-m)}$$

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$$\Delta_1 = \frac{M_a}{K_b} = \frac{21,000}{7500} = 0.233 \text{ ft or } 2.80 \text{ in. (71 mm)}$$

$$M_2 = 1750 + 4000 \times 0.233 = 2683 \text{ ft-lb (39.2 kN-m)}$$

$$\Delta_2 = 0.358 \text{ ft (109 mm)}$$

$$M_3 = 3181 \text{ ft-lb (46.4 kN-m)}$$

$$\Delta_3 = 0.4241 \text{ ft (129 mm)}$$

$$M_4 = 3447 \text{ ft-lb (50.3 kN-m)}$$

$$\Delta_4 = 0.4595 \text{ ft (140 mm)}$$

$$M_{max} = 3750 \text{ ft-lb (54.7 kN-m)}$$

$$\Delta_{max} = 0.5 \text{ ft (152 mm)}$$

3.4—Moment magnifier method

ACI 318 adopted the magnifier equation for evaluation of the P - Δ effects in compression members. This method is sometimes poorly understood by designers, primarily because of the complex way it has been employed in various codes for the design of slender columns in concrete, steel, and wood. Additionally, many engineers believe that the results obtained by moment magnifier are different than those from an iterative procedure. The moment magnifier equation is obtained by combining the two equations in 3.3. It provides results identical to those using an iteration of moments and deflections, as long as loading conditions and assumptions for material properties are consistent. The derivation of this method is illustrated as follows.

From the previous section of this guide (3.3)

$$M_{max} = M_a + P\Delta_{max}$$

where

$$\Delta_{max} = \frac{5M_{max}\ell^2}{48E_c I_{cr}} = \frac{M_{max}}{K_b}$$

or

$$K_b = \frac{M_{max}}{\Delta_{max}} = \frac{48E_c I_{cr}}{5\ell^2}$$

Maximum moment M_{max} can now be written in the following form

$$M_{max} = M_a + \frac{PM_{max}}{K_b}$$

This equation can be rewritten as

$$M_{max} = M_a \left\{ \frac{1}{1 - \frac{P}{K_b}} \right\} = M_a \delta_b$$

where

$$\delta_b = \left\{ \frac{1}{1 - \frac{P}{K_b}} \right\}$$

Using the values from the previous example,

$$M_a = 1750 \text{ ft-lb (25.5 kN-m)}$$

$$K_b = 7500 \text{ ft-lb/ft (33 kN-mm/mm)}$$

$$P = 4000 \text{ plf (5400 N/m)}$$

$$\delta_b = \frac{1}{1 - 4000 / 7500} = 2.143$$

$$M_{max} = 1750 \times 2.143 = 3750 \text{ ft-lb (54.7 kN-m)}$$

$$\Delta_{max} = \frac{3750}{7500} = 0.5 \text{ ft (152 mm)}$$

3.5—ACI 318 provisions

ACI 318 first introduced provisions for the design of slender concrete walls in 1999. These were originally adapted from the Uniform Building Code ([International Code Council 1997](#)), but modified to be consistent with ACI format using the moment magnifier method. Updates to the provisions have been incorporated in subsequent versions of ACI 318, with the strength requirements as follows ([ACI 318-11](#)):

The design moment strength ϕM_n for combined flexure and axial loads at the midheight cross section shall be

$$\phi M_n \geq M_u \quad (14-3)$$

where

$$M_u = M_{ua} + P_u \Delta_u \quad (14-4)$$

and

$$\Delta_u = \frac{5M_u \ell_c^2}{0.75(48E_c I_{cr})} \quad (14-5)$$

M_u shall be obtained by iteration of deflections, or by direct calculation using Eq. (14-6).

$$M_u = \frac{M_{ua}}{1 - \frac{5P_u \ell_c^2}{0.75(48E_c I_{cr})}} \quad (14-6)$$

where

$$I_{cr} = \frac{E_s}{E_c} \left[A_s + \frac{P_u}{f_y} \left(\frac{h}{2d} \right) \right] (d - c)^2 + \frac{\ell_w c^3}{3} \quad (14-7)$$

The term b will be used in place of ℓ_w for the remainder of this guide to avoid confusion with the term ℓ_c for wall height and the substitution of A_{se} defined by

$$A_{se} = A_s + \frac{P_u}{f_y} \left(\frac{h}{2d} \right)$$

A factor of 0.75 is used to reduce the calculated bending stiffness of the concrete section in accordance with ACI 318-11, Chapters 10 and 14. It is intended to account for variations in material properties and workmanship. Test results (SEAOSC 1982) and analytical studies (Lai et al. 2005) have indicated that small variations in the position of reinforcement in the concrete section will have a significant effect on the strength of the wall panel and the bending stiffness properties. A reduction in bending stiffness is not limited to the provisions of ACI 318. It should also be employed where other methods of slender wall analysis are used. Historically, design methods for tilt-up panels have not specifically included this requirement. The 0.75 reduction factor in bending stiffness should be incorporated by all other alternate design methods to comply with the requirements of ACI 318.

Nominal moment strength M_n is obtained in accordance with ACI 318-11, Chapter 10, requirements by using the modified area of reinforcement, A_{se} .

Procedures outlined in ACI 318-11, 14.8.3, provide a simplified method for design of slender concrete walls commonly used in tilt-up construction. The following limitations should be noted:

(a) Section stiffness $E_c I_{cr}$, as obtained from the rectangular stress block derivation using the modified area of reinforcement, A_{se} , as permitted by 14.8 of ACI 318-11, is valid for small axial loads only. Section 14.8.2.6 requires a limit of $0.06f'_c$ for the vertical stress P_u/A_g at the midheight section.

(b) Using a constant value for panel bending stiffness K_b is valid for walls subjected to out-of-plane bending forces due primarily to uniform lateral loads. Where applied moments are primarily due to end moments from eccentric axial loads, a reduction in bending stiffness may be necessary.

(c) The ACI 318 method is intended for simply supported wall elements only. It can, however, be adapted to panels with fixed-end conditions, or for those spanning over multiple supports, as will be demonstrated in Chapter 7 of this guide.

(d) ACI 318-11, 14.8.2.4, provides a minimum strength requirement for the concrete section of

$$\phi M_n \geq M_{cr} \quad (14-2)$$

where

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

as outlined in ACI 318, 10.2.7 (refer to Appendix A for the derivation of this equation using the rectangular stress block) and

$$M_{cr} = f_r S \quad (9-9)$$

where

$$S = \frac{bh^2}{6} = \frac{I_g}{y_t} \text{ (uncracked section modulus)}$$

$$f_r = 7.5\lambda\sqrt{f'_c} \quad (9-10)$$

where λ is taken as 1.0 for normalweight concrete.

The purpose of this provision is to prevent a sudden increase in lateral deflection where panel cracking occurs as a result of a temporary overload condition. An ancillary benefit is that it also contributes to improved behavior of the panel during the lifting operation. In many cases, the reinforcement required to satisfy ACI 318-11, 14.8.2.4, will be greater than minimum reinforcement.

ACI 318-11, 9.3.2.1 and 10.3.4, indicate that the value of ϕ for tension-controlled flexural members is equal to 0.9. This occurs when the steel tension strain is greater than or equal to 0.005 and when the concrete in compression reaches its assumed strain limit of 0.003. This is equivalent to c/d_t being less than 0.375 (refer to the commentary of ACI 318, 9.3.2.2) and is required for tilt-up panels designed in accordance with the provisions of ACI 318-11, 14.8.

3.6—Comparison to 1997 Uniform Building Code

The procedure specified in the Uniform Building Code (International Code Council 1997) was developed specifically for tilt-up concrete panels. Primary factored bending moments are calculated from the applied loading to the panel. Maximum potential deflection is computed for the condition where the panel section is assumed to be at ultimate bending moment over the entire span. The secondary P - Δ moment resulting from axial loads acting over this deflected shape is then added to the primary moment. Where the combined effects are less than or equal to the ultimate resisting moment of the concrete section, strength design requirements are considered satisfied.

$$M_u = M_a + P_u \Delta_u \leq \phi M_n$$

where

$$M_a = \frac{w_u \ell^2}{8} + P_u \frac{e_{cc}}{2}$$

Other forces contributing to the primary moment, such as lateral out-of-plane point loads, need to be included when computing M_a . The effect of panel self-weight should also be taken into account in the P - Δ calculations, although this is not specifically stated in the Uniform Building Code (International Code Council 1997); it is discussed in 4.3.

The potential midheight deflection is given by

$$\Delta_n = \frac{5M_n \ell_c^2}{48E_c I_{cr}}$$

where, for normalweight concrete

$$E_c = 57,000\sqrt{f'_c} \text{ psi (in.-lb units)}$$

$$E_c = 4700\sqrt{f'_c} \text{ MPa (SI units)}$$

and

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$

Modification in the area of reinforcement, as outlined previously, is used to partially account for the increased bending resistance due to applied axial loads.

$$A_{se} = \frac{P_u + A_s f_y}{f_y}$$

This equation is applicable for a single layer of reinforcement at the center of the panel only.

Other applicable requirements specified in the Uniform Building Code (International Code Council 1997) include:

(a) Vertical service load stress at the location of maximum moment does not exceed $0.04f'_c$

(b) Sufficient reinforcement should be provided so that the nominal moment capacity times the factor ϕ is greater than M_{cr}

(c) Midheight deflection Δ_s under service lateral and vertical loads (without load factors) shall be limited by the relation

$$\Delta_s = \frac{\ell_c}{150}$$

This is discussed further in Chapter 6.

The bending moment obtained from the ACI 318 procedure will be greater than that from the Uniform Building Code because ACI requires the application of the 0.75 stiffness reduction factor whereas Uniform Building Code does not. This difference is partially offset in the Uniform Building Code by the method of computing the maximum potential deflection based on moment M_n rather than ϕM_n .

3.7—Limitations on panel slenderness

ACI 318-11, 14.8, does not provide a specific limit for wall panel slenderness ratios. The strength design provisions are self-limiting, and arbitrary limits on panel structural thickness or maximum deflections due to factored loads are not required. There are, however, practical limits in height-to-thickness ratios for slender concrete walls. The following

guidelines taken from CAN/CSA A23.3 may be useful to designers

$$\ell_c/h$$

Single mat of reinforcement

(centered in the panel cross section).....50

Two mats of reinforcement

(1 in. [25 mm] clear of each face) 65

Where panel height-to-thickness ratio exceeds these limits, quantities of reinforcement may not be economical, and a thicker panel should be considered. Section 14.8.2.3 of ACI 318-11 effectively limits the amount of tension reinforcement in a panel by requiring the wall section to be tension-controlled. By meeting this criterion, impending failure of a wall section is observable through large deflections and cracking (refer also to ACI 318-11, R10.3.4). Panel thickness may also be controlled by limitations on service load deflections. Refer to Chapter 6 for more information regarding these limitations.

CHAPTER 4—LOADING CONDITIONS

4.1—Lateral loads

The effect of lateral loads on tilt-up panels is often the largest contribution to the total applied bending moment. Wind pressures, soil pressures, or seismic accelerations are usually applied to the wall panel as a distributed lateral load.

4.1.1 Wind loads—ASCE/SEI 7 specifies wind pressure for low-rise buildings as the algebraic difference between suction or external and internal pressure. Wind exerts loads on walls either as a net inward or a net outward pressure. Applicable building standards should be consulted for the proper determination of these forces, including any modifications or amendments adopted by the authority having jurisdiction.

4.1.1.1 Example of wind load determination based on Chapter 30, Part 1 of ASCE/SEI 7-10

Given: Warehouse structure; Occupancy (Risk) Category II; 30 ft (9 m) high; situated in open, smooth terrain (exposure C) in the central United States.

Basic wind speed = 115 mph (51 m/s) based on wind load determination of ASCE/SEI 7 using strength design. The suction or pressure on the panel is calculated from Equation 30.4-1 in ASCE/SEI 7-10 as

$$p = q_z[(GC_p) - (GC_{pi})]$$

where the velocity pressure q_z is determined by Eq. 30.3-1 in ASCE/SEI 7-10 as

$$q_z = 0.00256K_zK_{zt}K_dV^2$$

where $K_d = 0.85$ for building components and cladding; $K_z = 0.98$ for $z = 30$ ft (9 m); $K_{zt} = 1.0$ for smooth terrain; and $q_z = 0.00256 \times 0.98 \times 1.0 \times 0.85 \times 115^2 \times 1.0 = 28.2$ lb/ft² (1.35 kPa)



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$$GC_p = \begin{cases} +0.7 \text{ pressure} \\ -0.8 \text{ suction} \end{cases} \text{ assuming } 500 \text{ ft}^2 (46 \text{ m}^2) \text{ effective area}$$

where $GC_{pi} = \pm 0.18$; $+P(\text{pressure}) = 28.2(0.7 + 0.18) = 24.8 \text{ lb/ft}^2 (1.19 \text{ kPa})$; and $-P(\text{suction}) = 28.2(0.8 + 0.18) = 27.6 \text{ lb/ft}^2 (1.32 \text{ kPa})$.

Note that multiplying the resultant wind pressure using strength design (for example, based on the wind speeds of ASCE/SEI 7) by 0.6 will be approximately equal to wind pressures calculated by allowable stress design in previous versions of ASCE/SEI 7 for non-hurricane-prone regions. This conversion is not true in hurricane-prone regions, as ASCE/SEI 7 adjusted wind speeds in these areas in addition to the modification to strength design procedures. Therefore, careful review of wind pressures in hurricane-prone regions between newer and older versions of ASCE/SEI 7 is recommended.

4.1.2 Seismic loads—Seismic accelerations create an inertial force that is modeled as a pressure for the design of individual wall panels and based on ASCE/SEI 7. The seismic design force required for parts of a building, such as tilt-up wall panels, is greater than that required for the overall building. This loading will sometimes exceed wind pressure, particularly for thicker panels. Applicable building standards should be consulted for the proper determination of these forces, including any modifications or amendments adopted by the authority having jurisdiction.

4.1.2.1 Example of seismic load determination—ASCE/SEI 7-10, 12.11.1, provides the following relation for factored, lateral seismic forces on tilt-up wall panels: $F_p = 0.40 \times I_e \times S_{DS} \times \text{panel weight}$ but not less than 10 percent of the panel weight.

Given: 8 in. (200 mm) thick tilt-up wall panel

$S_S = 0.75$

Site Class D

$I_e = 1.0$

$S_{MS} = F_a \times S_S = 1.2 \times 0.75 = 0.90$

$S_{DS} = 2/3 \times S_{MS} = 2/3 \times 0.90 = 0.60$

$F_p = 0.40 \times 1.0 \times S_{DS} \times \text{panel weight} = 0.40 \times 0.60 \times 100 \text{ lb/ft}^2 (4.8 \text{ kPa}) = 24 \text{ lb/ft}^2 (1.15 \text{ kPa})$

4.1.3 Lateral earth pressures—Tilt-up panels have also been used to resist lateral pressures due to soils. Active soil pressures plus surcharge effects can be significant, and the required wall section is often much thicker and more heavily reinforced than similar panels aboveground. Panel connections, slabs-on-ground, and footings should be designed to resist these lateral loads. Lateral deflections should be limited to satisfy serviceability requirements. P - Δ effects are usually small, but should be checked.

4.1.4 Minimum lateral loads—Building codes typically require the application of minimum lateral loads due to wind, seismic forces, or both, for the design of exterior walls and cladding plus their associated connections. These are typically in the order of at least $10 \text{ lb/ft}^2 (0.48 \text{ kPa})$ unfactored. Interior tilt-up walls should also be designed for a minimum lateral load. In the absence of a specific building code requirement, an unfactored value of at least $5 \text{ lb/ft}^2 (0.24 \text{ kPa})$ is recom-

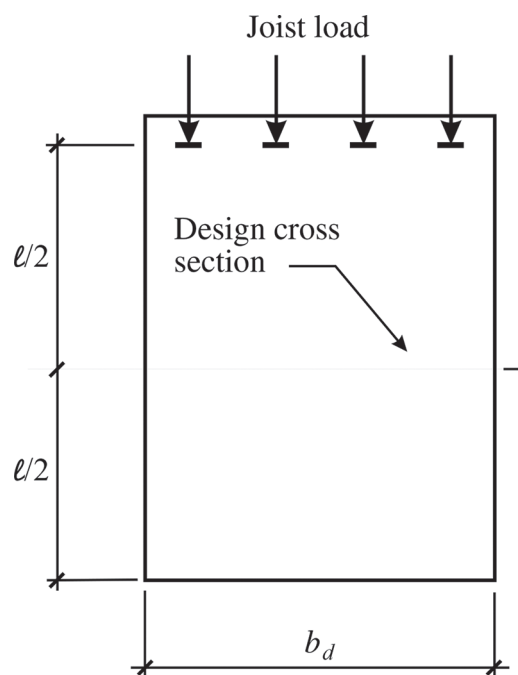


Fig. 4.2a—Distributed axial load.

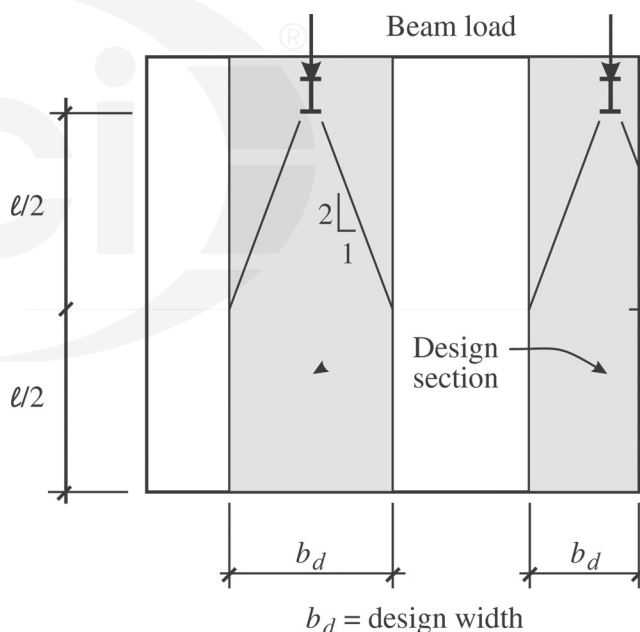


Fig. 4.2b—Concentrated axial load.

mended, provided the wind force on interior partitions in industrial building applications with large exterior overhead door openings and seismic loads have been considered.

4.2—Axial loads

Vertical loads from roof or floor members (Fig. 4.2a) can often be considered as uniformly distributed line loads for wall panel design. Load eccentricity should be based on the assumed bearing conditions and measured from the centroid of the concrete cross section. A minimum axial load eccentricity of one half of the panel thickness is suggested.

Axial load eccentricities should not be used to reduce the bending moment caused by wind or seismic lateral loads.

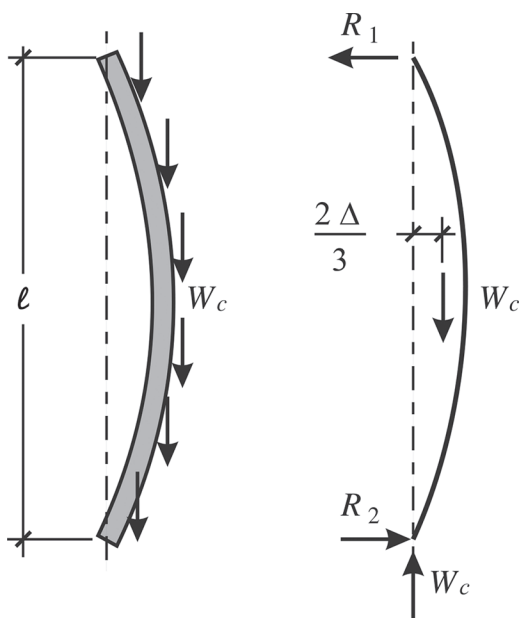


Fig. 4.3—Panel self-weight.

Further, axial load should not be reduced due to wind uplift on roof members.

Where large concentrated loads are supported directly on the panel, the effective width b_d of the design cross section should be limited, as indicated in Fig. 4.2b. Extra reinforcement, where required, should be concentrated in this area of the panel. The maximum factored axial stress on the design width b_d is limited to $0.06f'_c$.

4.3—Panel self-weight

The effect of panel self-weight should be considered because it represents a significant contribution to P - Δ moments in slender walls. It is sufficient to assume that the weight of the panel above the midheight section acts as an additional concentrated axial load with no eccentricity (that is, concentric to the panel centroid) applied at the midheight.

This is illustrated in Fig. 4.3 and by the following derivation

$$R_1 = R_2 = \frac{2W_c \Delta}{3\ell_c}$$

The midheight moment is

$$M = \frac{R_1 \ell_c}{2} + \frac{W_c}{2} \left(\frac{\Delta}{3} \right) = \frac{W_c \Delta}{2}$$

4.4—Load factors and combinations

ACI 318-11, 9.2.1, specifies the following factored load combinations

$$U = 1.4(D + F) \quad (9-1)$$

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (9-2)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.5W) \quad (9-3)$$

$$U = 1.2D + 1.0W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \quad (9-4)$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \quad (9-5)$$

$$U = 0.9D + 1.0W + 1.6H \quad (9-6)$$

$$U = 0.9D + 1.0E + 1.6H \quad (9-7)$$

All of these load combinations should be checked for slender wall design. The reader can observe the following with respect to individual equations:

(a) Equation (9-1) does not generally govern in slender wall design because it predominantly relates to structures resisting fluid pressures.

(b) Equation (9-2) may control the design for walls supporting dead and live loads in combination with lateral soil pressures.

(c) Equation (9-3) could govern the design of walls supporting large gravity loads.

(d) Equation (9-4) often controls the design of slender wall panels in low to moderate seismic locations.

(e) Equation (9-5) could control the design for panels in high seismic areas, but results from Eq. (9-3) and (9-4) should be compared to determine the controlling condition.

(f) Equations (9-6) and (9-7) are intended for situations where higher dead loads reduce the effects of other loads. They do not govern the design for most tilt-up panel applications, except for panel overturning calculations due to in-plane lateral loads.

For the common load case of large bending moments due to lateral forces combined with small axial loads, the critical section for bending will occur near panel midheight. As axial load and top end eccentricity increase, this point shifts upward.

The wind load factor reflects the switch to strength-level (factored) loads in ASCE/SEI 7 as discussed in ACI 318-11, R9.2.1(b). Use of service-level wind loads calculated from earlier versions of ASCE/SEI 7 is permitted by substituting $1.6W$ and $0.8W$ in the previous equations for $1.0W$ and $0.5W$, respectively.

CHAPTER 5—MINIMUM REINFORCEMENT

5.1—General

Due to their segmented nature, experience has shown that there are fewer problems associated with temperature changes and concrete shrinkage in tilt-up panels than with monolithic cast-in-place concrete structures. There are, however, some design techniques that should be considered.

Tilt-up panels are often cast and lifted into place within a period of 1 to 2 weeks, and may not have sufficient time to fully cure. If connections to the panels are made immediately after panel erection, the restraint induced could cause a buildup of stresses in the concrete as it continues to undergo drying shrinkage. Minimum horizontal reinforcement based on $0.002A_g$ may be insufficient to limit cracking. For this

reason, the erector should delay the completion of connections as long as practical. Alternatively, increased reinforcement in the direction of restraint should be considered to counter the stresses that could be caused by making connections early.

Buildings with tilt-up panels have the advantage that each joint can act as an expansion joint. It is possible to have continuous lengths of wall panels without any special provisions for thermal expansion or shrinkage. Some designers, however, may specify connections along the vertical joint of all panels, even if it is not justified by design analysis. This can result in excessive restraint and vertical cracking.

Variations in relative humidity or temperature between the inside and outside panel faces can induce warping. These effects are usually small and can be accounted for in design by including an initial deflection in the calculations (Chapter 6). Panel warping due to temperature differentials can result in splitting of the caulk along the joint at intersecting corners. A simple solution is to routinely connect the panels together at these corners by means of welded embedded metal connectors.

5.2—ACI 318 provisions

If a tilt-up wall spans vertically, the horizontal reinforcement could likely be governed by minimum shrinkage and temperature reinforcement. The designer is permitted to determine shrinkage and temperature requirements by means of a thorough analysis of the structure. The minimum wall reinforcement requirements need not be met if the structural analysis shows that the walls meet the requirements of ACI 318-11, 14.2.7. Designers pursuing this approach are cautioned to consider all load effects and boundary conditions as a function of time. While all the provisions for minimum reinforcement are important, only minimum vertical and horizontal reinforcement provisions are discussed herein.

Tilt-up concrete construction is a unique form of precast concrete (ACI 318-11, R16.1.1). The general structural integrity requirements of ACI 318-11, 7.13.3, reference 16.5 for precast concrete. There are several integrity provisions in ACI 318-11, 16.5 that apply to tilt-up walls and their connections.

If a wall resists in-plane shear force, and factored shear exceeds one-half concrete shear design resistance, horizontal and vertical shear reinforcement should be provided (ACI 318-11, 11.9.9). For relatively short walls with a low height-to-length ratio, the amount of vertical shear reinforcement will exceed the horizontal shear reinforcement. For relatively tall walls or walls with a high height-to-length ratio, the amount of horizontal shear reinforcement will exceed the vertical shear reinforcement. If shear reinforcement is required, ACI 318-11, 11.4.5 and 11.4.6, provide minimum limits on steel area and spacing for both horizontal and vertical shear reinforcement.

Tilt-up walls, therefore, can be subject to the minimum vertical and horizontal reinforcement provisions of ACI 318-11, 14.3. Note that if the wall requires in-plane shear reinforcement, the minimum shear reinforcement provisions

of ACI 318-11, 11.9.9, will govern over the minimum wall reinforcement provisions of 14.3. ACI 318-11, 14.3 also addresses the number of layers of reinforcement required, transverse ties for vertical bars, and special reinforcement required around openings.

The minimum reinforcement for walls in ACI 318-11, 14.3.2 and 14.3.3 addresses shrinkage and temperature reinforcement. Section 14.3 addresses all walls, including continuous cast-in-place walls. It is expected that the temperature and shrinkage requirements could be reduced for walls with frequent joints, such as tilt-up walls that are not linked together in a way that causes restraint.

Crack control in tilt-up panels is deemed to be satisfied when the reinforcement is sufficient to satisfy the deflection limits of ACI 318-11, 14.8.4. Crack control can be particularly important in tilt-up construction where the exterior faces of the panels are exposed to the elements or interior faces to a corrosive environment. Note that the use of high-strength steel to reduce total reinforcement provided could effectively increase cracking.

The smaller, minimum reinforcement indicated in ACI 318-11, 16.4.2, is not recommended for tilt-up panels because tilt-up panels are generally wider than plant-cast, precast panels and subject to more curing restraint.

For seismic design, walls are classified as one of the following seismic-force-resisting systems:

- (a) Ordinary structural walls (ACI 318-11, 11.9.9; no required provisions in Chapter 21)
- (b) Intermediate precast walls (ACI 318-11, 21.4)
- (c) Special structural walls (ACI 318-11, 21.9)
- (d) Special structural walls constructed using precast concrete (ACI 318-11, 21.10)

Intermediate precast structural walls are governed by ACI 318-11, 21.4. This system requires targeted yielding of components of the connections either between the wall panels, or between the wall panel and foundation. Wall piers in this system must be designed per the special structural walls section (ACI 318-11, 21.9) or members not designated as part of the seismic-force-resisting system (ACI 318-11, 21.13).

Special structural walls constructed using precast concrete are governed by ACI 318-11, 21.10, which refers to 21.9 for the design of special structural walls. There are other special reinforcing requirements for wall boundary elements (ACI 318-11, 21.9.6), coupling beams (ACI 318-11, 21.9.7), and piers (ACI 318-11, 21.9.8). A careful review of all of these provisions for ACI 318-11, 21.9 and 21.10, is warranted.

CHAPTER 6—CONTROL OF DEFLECTIONS

Limitations on lateral or out-of-plane deflections for slender walls have traditionally been a concern of building officials and code committees, not only because of the increased bending moments due to $P-\Delta$ effects, but also the potential for long-term bowing of these elements. Experience in actual buildings, however, suggests that long-term deflections have not been a serious problem. This is likely due to the fact that the lateral forces that cause deflection in panels are largely transient, and that the effect of axial

load and self-weight allow the concrete to perform as an uncracked section under design wind forces. In the case of small axial load combined with large cyclic force due to seismic excitation, full-scale tests on slender walls did not show gross instability (SEAOSC 1982).

6.1—Creep and initial deflections

Initial panel deflections, or out-of-straightness, could be the result of uneven casting beds, excessive bending caused by the tilting process, thermal gradients, or uneven shrinkage. Sustained loads can cause additional deflections and P - Δ effects. This is represented by the fact that the axial resistance of a member, based on stability, reduces nonlinearly as initial deformation is increased. For example, initial deformations on a slender wall result in P - Δ moments that can have a significant effect on the remaining axial carrying capacity of the wall. Initial deformations within the tolerance limits of ACI 117 can be ignored for the purposes of design.

6.1.1 Construction tolerances and contributing factors to out-of-plane initial deformations—ACI 117 permits for panel height divided by 360, but not to exceed 1 in. (25 mm), maximum deformation due to initial bow or warping. Any initial panel bow can be exacerbated by unbalanced dead load on the resisting section. Examples include reveals, bump-outs, or eccentrically applied dead loads. The same is true for unbalanced resistance on the wall cross section. Examples include a single layer of reinforcement not placed at midthickness, two layers not symmetrically placed about the centroid, or two layers with unequal bar sizes or spacings. Variations in the panel thickness along the height of the wall, creep and shrinkage, and temperature differential between interior and exterior faces of panels are additional sources of panel bow.

6.2—Deflection calculations

ACI 318-11, 14.8.4, provides a limit for out-of-plane deflections of walls at service-load levels, including P - Δ effects, of $\ell_c/150$. The midheight deflection Δ_s shall be determined as follows:

If M_a , which is the maximum moment at midheight of wall due to service lateral and eccentric vertical loads, including P - Δ effects, exceeds $(2/3)M_{cr}$, Δ_s shall be calculated by

$$\Delta_s = \frac{2}{3}\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}(\Delta_n - (2/3)\Delta_{cr}) \quad (14-8)$$

If M_a does not exceed $(2/3)M_{cr}$, Δ_s shall be calculated

$$\Delta_s = \left(\frac{M_a}{M_{cr}} \right) \Delta_{cr} \quad (14-9)$$

where

$$\Delta_{cr} = \frac{5M_{cr}\ell_c^2}{48E_c I_g} \quad (14-10)$$

$$\Delta_n = \frac{5M_n\ell_c^2}{48E_c I_{cr}} \quad (14-11)$$

I_{cr} shall be calculated by Eq. (14-7), and M_a shall be obtained by iteration of deflections.

As stated in ACI 318-11, R14.8.4, the original test data on slender walls (SEAOSC 1982) was reevaluated, noting that out-of-plane deflections increase rapidly when the service load moment exceeds two-thirds of the cracking moment. To simplify the application to slender wall design, a linear interpolation between the deflection at the cracking moment and deflection at the nominal moment is permitted.

6.3—Deflection limits

One purpose of out-of-plane deflection limits for tilt-up walls is to avoid excessive elastic deformation due to permanent loads and residual deformation due to an inelastic response. Tests of full-scale wall panels indicate that when wall panels are subjected to small axial load and large lateral force, such as wind or seismic, out-of-plane deflection increased rapidly when the induced moment exceeded $2/3M_{cr}$ (SEAOSC 1982). A permanent deflection creates eccentricity for axial force to generate P - Δ moment. Deflection in excess of elastic limits will result in the structure remaining deformed, even when loads are removed. Depending on magnitude, a one-time residual deformation could be a problem. Loads of a similar magnitude repeatedly applied will accumulate deformations, possibly leading to adverse effects including, but not limited to, collapse.

In addition to the effect deflection has on the structure, deflection limits are desirable to avoid damage to nonstructural components such as a brick façade, curtainwall, drywall, and interior non-load-bearing walls. In this case, consider whether deflection occurs before or after nonstructural components are installed. For example, immediate deflection due to eccentric dead load of the structure will not cause a problem for a brick veneer installed after the deflection occurs. The deflection cannot be so substantial that the nonstructural component cannot be installed. Deflection after nonstructural components are installed will usually be the primary concern. These include, but are not limited to, cracking of brick and drywall, water intrusion through the façade, and severe deformation or buckling of interior non-load-bearing walls.

The out-of-plane deflection limits recommended in various sources are listed in the following. Total deflection must include the P - Δ effect.

- (a) Total deflection: $L/100$ (SEAOSC 1982)
- (b) Total deflection with wind: $L/240$ (Griffis 1993)—out-of-plane deflection of an uncovered tilt-up panel only
- (c) Total deflection: $L/150$ (International Code Council 1997; ACI 318)

The following deflection limits are recommended to avoid residual deformations and negative effects on nonstructural components, respectively:

(a) Total deflection with wind: $L/150$ to $L/240$ (designer discretion to increase limit based on type of veneer and sensitive nonstructural components as appropriate)

(b) Total deflection with seismic: $L/150$

The following service load combinations for checking deflection are recommended:

a) Wind effects—Use the **ASCE/SEI 7** wind speed map according to the proper importance category for the structure and the selected mean recurrence interval (MRI). For typical structures, a 50-year MRI is common among practicing engineers, but a 10-year MRI may be warranted for serviceability checks after the engineer and building owner review all the considerations and risks associated with this lower level of wind. The commentary to ASCE/SEI 7, Appendix C provides a good discussion on this topic.

$$D + 0.5L + W_a$$

b) Seismic effects

$$D + 0.5L + 0.7E$$

where E is a strength level force as calculated by ASCE/SEI 7 (refer to ACI 318-11, R14.8.4) and service moments are calculated with $P-\Delta$ effects.

CHAPTER 7—PANEL DESIGN PROCEDURES

This section covers several common design conditions for vertical and transverse loading that could occur in tilt-up panels. Computer spreadsheet programs greatly simplify the design procedure. Design examples in Appendix B provide a breakdown of the analysis for panels, including comparisons with single and double mats of reinforcement.

7.1—Solid panels without openings

The procedure for designing tilt-up panels involves a combination of trial-and-error and experience. The following steps are typically involved:

(a) Determine panel geometry, including height, width, openings, and recesses.

(b) Define applied loading conditions, including axial load and out-of-plane lateral load.

(c) Start with an assumed panel thickness. For plain panels, the suggested minimum thickness should be $\ell_c/50$ where a single layer of reinforcement is desired, or $\ell_c/65$ for a double layer.

(d) Select a starting area of reinforcement and analyze the panel for each load combination.

(e) Adjust the panel thickness or reinforcement until an optimum design is obtained to satisfy all load conditions and code requirements; check service load deflections and adjust panel thickness or reinforcement as required.

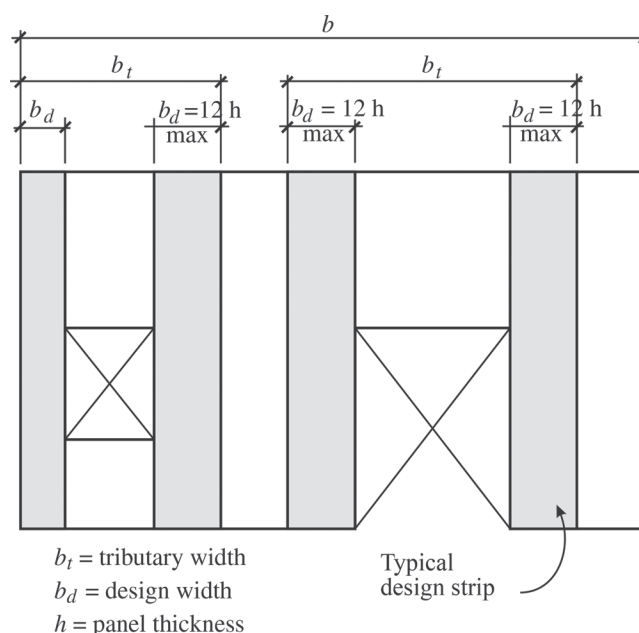


Fig. 7.2a—Design strips at openings.

7.2—Panels with openings

The effect of openings for out-of-plane bending in tilt-up panels can be approximated by a simple, one-dimensional strip analysis that provides accuracy and economy for most designs.

Where openings occur, the entire lateral and axial load, including self-weight above the critical section, is distributed to supporting legs or design strips at each side of the opening (Fig. 7.2a). The effective width of the strip should be limited to approximately 12 times the panel thickness to avoid localized stress concentrations along the edge of the opening. This limit is not mandated by ACI 318, but is included in this document as a practical guideline where the opening width is less than one-half the clear vertical span. In most cases, the tributary width for loads can be taken as the width of the strip plus one-half the width of adjacent openings. The design strip should have constant properties full height and the reinforcement should not be cut off just above or below the opening.

Thickened vertical or horizontal sections can be provided with the panel where openings are large or where there are deep recesses on the exterior face (Fig. 7.2b). Some conditions may require ties around all vertical reinforcement bars in a vertical pilaster for the full height of the panel.

7.3—Concentrated axial loads

The effect of a concentrated axial load, such as the reaction from a roof or floor girder connected directly to the panel, was introduced in 4.2. The two most important considerations for design are:

1) To ensure that the connection is capable of distributing the shear and bending forces into the localized area of the panel (Fig. 4.2b)

2) To provide sufficient capacity over a defined vertical design strip, b_d , in the panel

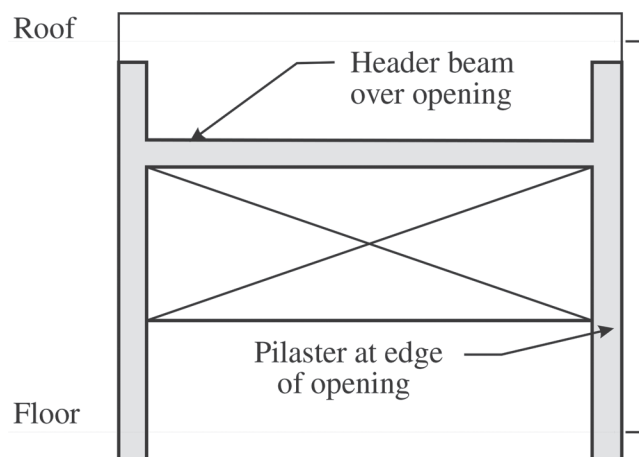


Fig. 7.2b—Stiffening header and pilasters.

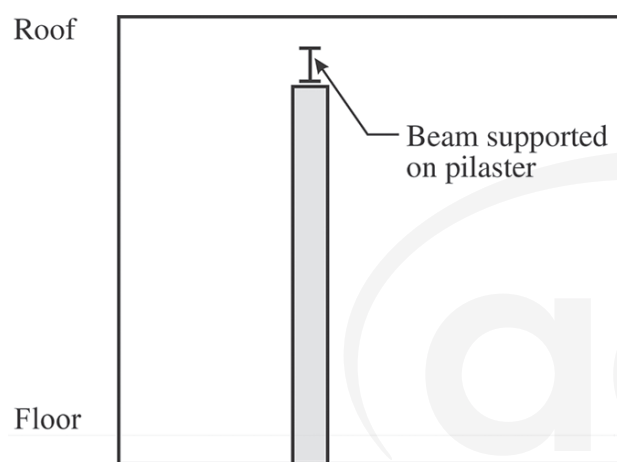


Fig. 7.3—Pilaster supporting beam load.

Where loads are very large, which is greater than $0.10f'_c b_d h$, consider pilasters as shown in Fig. 7.3. These provide greater bearing area at the connection and increase the stiffness for out-of-plane bending. Consider increased local stiffness in the distribution of applied lateral loads.

Provide ties around the vertical reinforcement in accordance with requirements of ACI 318-11, Chapter 7. Axial stress from beams, however, is usually concentrated at the point of bearing and quickly dissipates into the panel such that ties may not be required for the full height. ACI 318 does not mandate how the load should be distributed, so the designer has a choice if a member has to be considered a column and, therefore, subject to the requirement for confinement ties. ACI 318-11, 14.3.6, provides guidance on tie requirements specifically for wall applications.

Often, overall panel design is controlled by flexural tension in vertical reinforcement rather than compression, and ties are not necessary. Ties within 12 in. (305 mm) of the point of bearing are recommended to ensure the axial load is distributed into the panel.

7.4—Concentrated lateral loads

Concentrated lateral loads can occur due to:

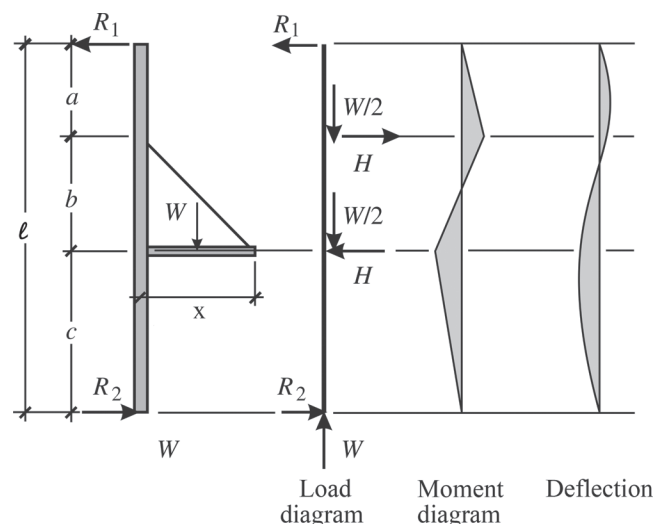


Fig. 7.4—Suspended canopy on panel.

(a) Suspended elements, such as canopies, as shown in Fig. 7.4

(b) End reactions from header beams over wide panel openings

(c) Lateral wind or seismic forces from intermediate roofs or floors where independent lateral-force-resisting systems have not otherwise been provided

The effect of these loads can be included in the analysis by superimposing the moment directly with the other primary bending moments. This is a simplistic approach that may be too conservative, as the algebraic sum of the maximum moments does not consider direction of the applied load(s). The designer may consider a more rigorous analysis of the panel to determine the correct combination of moments to include for reinforcement analysis.

7.4.1 Example: Canopy supported on panel

W = canopy load

R_1 = end reaction = $Wx/2\ell_c = R_2$

H = horizontal line load = $Wx/2b$

where the horizontal load is a point load, the effective panel design width should be limited to no more than 12 times the panel thickness at the application point, and the load should be distributed evenly across this width. Additional reinforcement could be required in this localized area.

7.5—Multiple spans and effects of continuity

Most tilt-up panels are designed as simply supported vertical members spanning between the footing and roof structure. Where a panel is connected to both floor slab and the footing (Fig. 7.5), a degree of panel continuity can be considered. A panel could also be laterally supported by an intermediate floor, resulting in negative bending at intermediate supports and a reduction of positive bending between supports.

It is difficult to properly analyze this condition and, at best, only approximate methods are practical. Some analysis problems and limitations include:

1) Lateral deflection at supports, particularly at flexible roof diaphragms, will affect final results

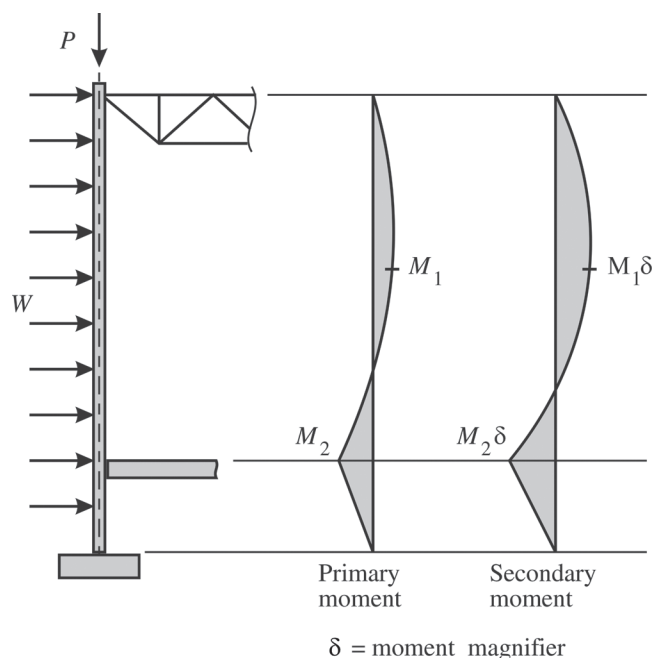


Fig. 7.5—Panel continuity at floor slab.

- 2) Effects of loading due to soil pressure below the floor slab may be significant
- 3) Lateral wind or seismic forces from intermediate floor or roof structures may exist
- 4) Lateral restraints provided by footings are questionable such that full end fixity might not be fully realized
- 5) $P-\Delta$ calculations for statically indeterminate elements should be obtained by an iterative technique practical only with computer analysis

Because of these concerns, it is best to be conservative in the design approach. One technique involves using a reduced effective panel height coefficient k . A value of $k = 0.8$ is suitable for a flexural elastic member fixed at one end and pinned at the other. Conversely, a flexural elastic member pinned at each end has a value of $k = 1.0$. Because concrete stiffness is not uniform and the lower end of a panel is seldom completely fixed, a value of $k = 0.9$ might be appropriate for the panel design with continuity.

An alternative method is based on the assumption that the initial positive midheight moment and negative support moment increase proportionally by the same amount when considering $P-\Delta$ magnifications. Primary moments are calculated by conventional elastic methods. Both positive and negative moments can then be increased proportionally by the $P-\Delta$ moment magnifier. Only this method is illustrated in the design examples of [Appendix B](#).

The designer should be aware that there might be a temporary condition during construction where lateral support at the intermediate floor slab is absent. This will increase the unsupported height of the panel and, therefore, could become the controlling design condition. Do not assume that this will be automatically addressed by those responsible for tilt-up panel lifting and bracing.

In ACI 318-11, 14.8.2.1, states that, "The wall panel shall be designed as...simply supported...at midspan" because

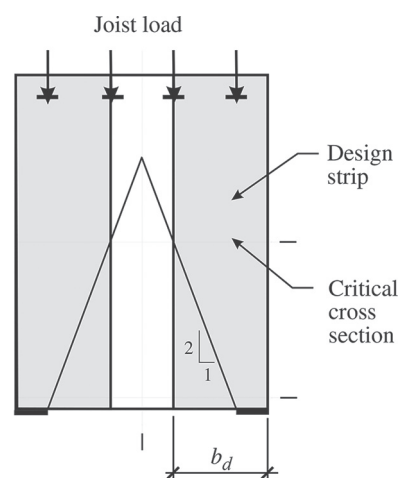


Fig. 7.6—Isolated foundation.

the provisions are generally based on the slender wall tests conducted in the 1980s ([SEAOC 1982](#)). This applies to panels designed in accordance with ACI 318-11, 14.8 only, and does not mean that effects of end fixity or panel continuity cannot be considered when using other accepted design methods, including ACI 318-11, Chapter 10 provisions. Although this lack of correlation is an apparent restriction to the ACI 318-11, 14.8 method, the use of reduced effective length (span) is an appropriate interpretation for design of tilt-up wall panels when used within the recommendations of this guide.

7.6—Isolated footings or pier foundations

In some geographical areas, soil conditions or frost depth requirements dictate the use of pier foundations. For convenience and economy, piers are often located at the panel joints only. This concentrates vertical stresses at the panel edges. Using pier foundations does not negate the requirement to protect the bottom of the panel from ground upheaval due to frost.

Vertical load should effectively be concentrated at support piers or pad footings, as indicated in Fig. 7.6. In conditions where clear panel height is greater than approximately one-and-a-half times the clear distance between footings, the effect of isolated footings can usually be ignored for vertical reinforcement design.

Depending on the length of pier cap and effective bearing area at the bottom of the panel, additional hooked reinforcement or confining ties could be required to prevent localized shear or bearing failure.

In most cases, there is continuous lateral support provided at the top by the roof deck and at the bottom by ties to the floor slab. Where this occurs, lateral loads can therefore be uniformly distributed across the width of the panel.

Where panels contain multiple openings across the width, the exterior legs that are supported on isolated footings should be designed to resist all the axial loads, whereas intermediate legs may be designed to resist tributary lateral loads only. Panels should be designed as deep beams spanning between piers with appropriate horizontal ties along the bottom.

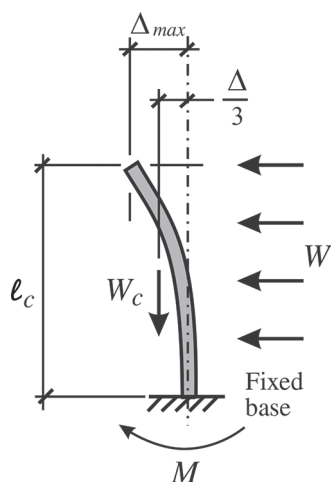


Fig. 7.7a—Cantilever panel.

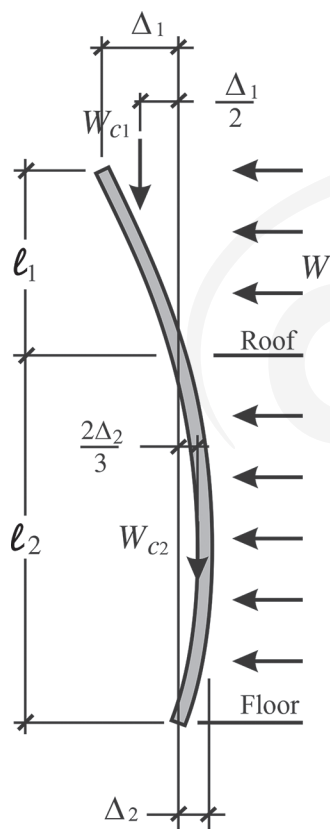


Fig. 7.7b—Panel with parapet.

7.7—Cantilever panels

Tilt-up panels are sometimes required to function as vertical cantilevers. Typical examples include freestanding signs and screen walls, or parapets above the roof of a building (refer to Fig. 7.7a and 7.7b). When the cantilever is high, P - Δ effects will increase the bending moments on the panel. A simple but conservative way to analyze a fixed-end cantilever panel is to assume a simply supported panel with a height two times the cantilever height.

The more correct method of analysis for a fixed base cantilever is

$$M_{ua} = \frac{w_u \ell_c^2}{2}$$

$$M_u = M_{ua} + \frac{W_c \Delta}{3} = M_{ua} + \frac{W_c}{3} \frac{M_u \ell_c^2}{4EI}$$

$$\Delta = \frac{M_u \ell_c^2}{4EI} = \frac{M_u}{K_b}$$

$$K_b = \frac{4EI}{\ell_c^2}$$

The maximum moment can now be written as

$$M_u = \frac{M_{ua}}{1 - \frac{W_c}{3(0.75K_b)}}$$

where 0.75 is included as a stiffness reduction factor.

The dynamic effects of wind buffeting or seismic accelerations might temporarily increase cantilever deflection because there may be little structural damping. This should be considered when selecting design forces.

Where the cantilever is a high parapet, a more detailed analysis may be required. As illustrated in Fig. 7.7b, rotation of the panel section at the roof connection can increase deflection and the associated P - Δ effects.

CHAPTER 8—IN-PLANE SHEAR

Design procedures for in-plane shear forces are distinctly different from methods used in design for out-of-plane bending. Forces from the roof or floor diaphragms acting parallel to the plane of the wall induce shear stresses and overturning moments in the panels (Fig. 8). In seismic areas and regions with high wind, in-plane shear requirements may control panel thickness and reinforcement design.

The design considerations for tilt-up panels subjected to in-plane forces include:

- (a) Resistance to panel overturning

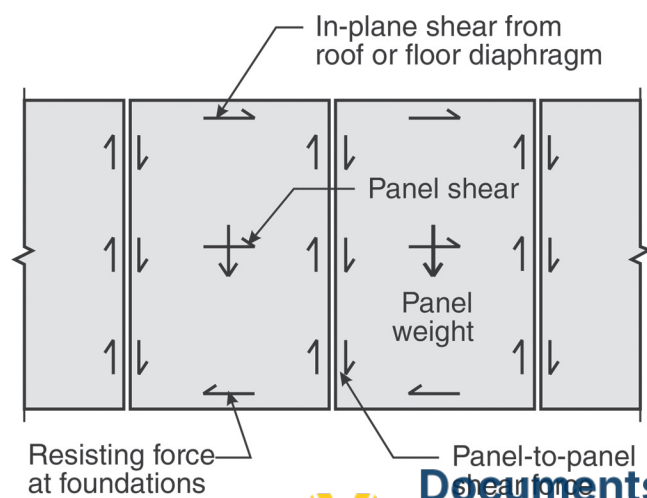


Fig. 8—In-plane shear forces.

- (b) Resistance to sliding
- (c) Concrete shear resistance
- (d) Increased axial forces on portions of the panel
- (e) Load distribution to foundations
- (f) Frame action in panels with openings
- (g) Seismic ductility

In regions of low seismicity (Seismic Design Categories [SDC] A and B), wind most likely controls the lateral analysis, and a target failure mode is not required. In regions of moderate and high seismicity (SDC C, D, E and F), a ductile failure mode is desired with overstrength to guard against brittle failure modes. Energy dissipation can be accomplished through repeated inelastic deformations or rocking. For wall panel in-plane shear, the applicable code sections in **ACI 318-11** are 11.9.9 (SDC A, B), 21.4 (SDC C), or 21.9 and 21.10 (SDC D, E, F).

Other situations where inelastic deformations could occur include extreme events, such as blast design, and in shelters, like those for tornados and hurricanes, which are designed as areas of refuge.

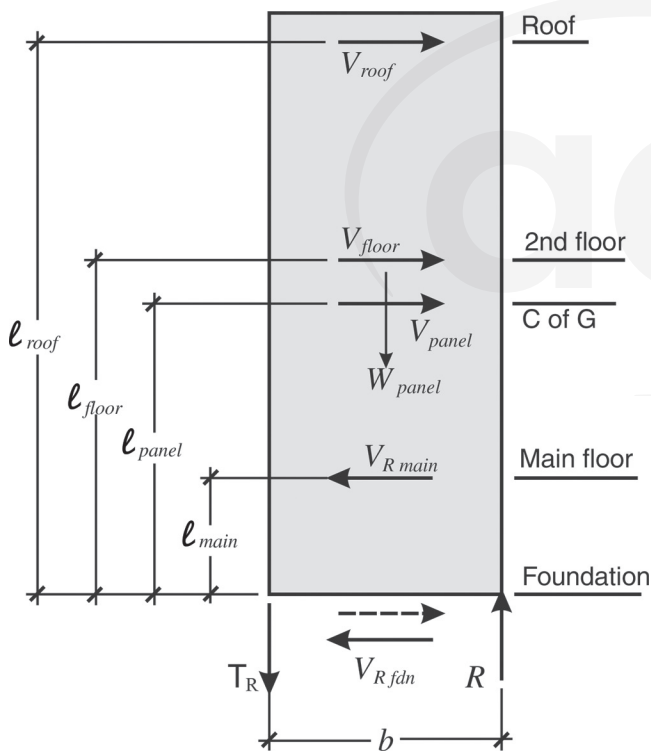


Fig. 8.1a—Panel overturning resistance.

8.1—Resistance to panel overturning

When roof and floor diaphragm forces are applied parallel to the plane of the wall panels, overturning moments and in-plane shears are induced. Overturning moments are usually taken near an outside corner of the panel. Resistance to overturning is obtained from a combination of panel weight, tributary roof or floor loads, panel edge connectors, and tie-down anchors to the foundations. The actual point of rotation will be close to the outside corner of the panel, at the center of the bearing area between the panel and the footing. In most cases, assume that the width of bearing is zero. Footing pressures beneath the footing and the footing design capacity should be checked for this concentration of force.

For the panel shown in Fig. 8.1a, the overturning equation in a seismic event is written as:

Factored overturning moment $M_o \leq$ resisting moment M_R

$$M_o = V_{roof} \ell_{roof} + V_{floor} \ell_{floor} + (V_{panel} \ell_{panel})$$

where the resisting moment is given by

$$M_R = (W_{roof} + W_{floor} + W_{panel})b/2 + V_{R \text{ main}} \ell_{main}$$

All applied shear forces contributing to overturning are factored. Forces and weights that resist overturning should be reduced in accordance with load combination factors in ACI 318-11, 9.2, which is outlined in 4.4 of this guide. No additional safety factor is required. Where there is insufficient overturning capacity, edge connections to an adjacent panel or tie-down anchors to the foundation can be added until the overturning equation is satisfied (Fig. 8.1b).

Depending on the requirements of the seismic-resisting system, connections to the foundation for resisting overturning may need to consider ductility requirements in accordance with ACI 318-11, 21.4 or 21.10.

This additional moment resistance could be limited by the weight of the foundation or adjacent panel. Foundations should be checked to ensure that the footing capacity or the soil-resisting pressure is not exceeded. The geotechnical engineer should be consulted for allowable increases in bearing pressure due to wind and seismic forces.

8.2—Resistance to sliding

Resistance to sliding forces can be obtained by a combination of friction between the bottom of panel and the footing, and connections to the floor slab or foundation (ACI 318-11,

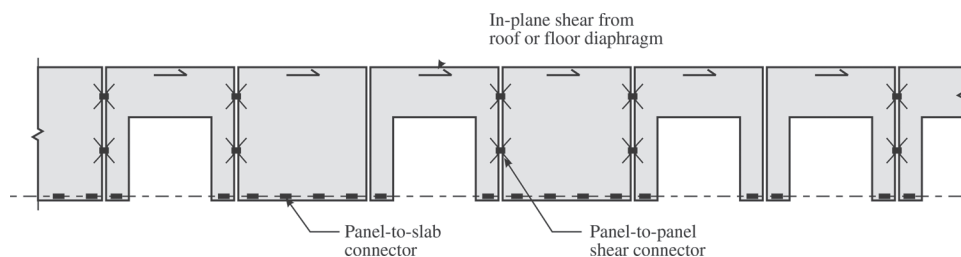


Fig. 8.1b—Typical shear wall with connections to adjacent panels and foundations.

16.5.1.3). The coefficient of friction for factored sliding resistance between the panel bottom and the footing can usually be taken as 0.6. Supplemental positive connections between the panel and footing or floor slab include cast-in-place reinforcement dowels or welded connections.

Where panels are subjected to seismic shear forces, the contribution of friction resistance may not be permitted by some building codes. In addition, connections between the panel and floor slab or footing is a compulsory requirement in many building codes, particularly for seismic forces.

Friction between the footing or floor slab and the soil, or passive soil resistance, should also be checked. The geotechnical engineer should be consulted for assistance.

8.3—Concrete shear resistance

Requirements for shear resistance of the concrete section are covered in [ACI 318-11](#), Chapters 11 and 21. ACI 318-11, 11.4.6.1 states that minimum shear reinforcement must be provided when

$$V_u > 0.5\phi V_c$$

where $V_c = 2\sqrt{f'_c}b_w d$ (in.-lb) [$V_c = 0.17\sqrt{f'_c}b_w d$ (SI)].

The minimum area of shear reinforcement is given by

$$A_{v,min} = 0.75\sqrt{f'_c} \frac{b_w s}{f_{yt}} \quad (\text{in.-lb})$$

$$A_{v,min} = 0.62\sqrt{f'_c} \frac{b_w s}{f_{yt}} \quad (\text{SI})$$

For $f_y = 60,000$ psi (414 MPa),

$$A_g = b_w s$$

$$A_v = \frac{50}{60,000} A_g = 0.000833 A_g$$

This is always less than the minimum requirements for temperature and shrinkage reinforcement specified for walls in ACI 318-11, 14.3.2 and 14.3.3.

ACI 318-11, 21.9.2.1, provides requirements for minimum shear reinforcement in special structural walls subject to seismic forces. Generally, the reinforcement ratio should not be less than $0.0025A_g$, but the minimum requirements listed in [Chapter 5](#) of this guide should be carefully reviewed for any additional reinforcement needed.

Where factored shear force V_u exceeds concrete shear strength ϕV_c , minimum shear reinforcement according to ACI 318-11, Chapter 11 should be provided to satisfy

$$\phi V_n \geq V_u \quad (11-1)$$

$$V_n = V_c + V_s \quad (11-2)$$

$$V_s = \frac{A_v f_{yt} d}{s} \quad (11-15)$$

ACI 318-11, 11.9.4 requires the value of d to be taken as 0.8 times the overall width of the panel when computing the average shear for a typical, rectangular panel.

8.4—Seismic ductility

Lateral design forces specified in various codes for buildings located in seismic areas represent only a portion of the total energy imparted to the structure. Primary lateral-load-resisting elements need to resist overstress and deformation without total structural failure, and they need to contain mechanisms capable of absorbing seismic energy—a property known as seismic ductility.

A large percentage of tilt-up buildings consist of perimeter load-bearing shear walls with horizontal roof and floor diaphragms. Where a wall line consists primarily of rectangular panels with no openings, it will be stiff and may be capable of resisting more than the specified code forces such that yielding of the panel reinforcement is unlikely to occur. Some limited ductility can be achieved by tie-down anchors to the foundation at the base of the panel or connectors along the panel edges if required. The effectiveness of these devices for energy absorption may be limited depending on their detailing. The precast seismic structural systems (PRESSS) testing program ([SESOC 2000](#)) developed ductile connections for traditional style precast structures. Information learned in the PRESSS program may be applied to tilt-up concrete structures at the engineer's judgment.

Where seismic ductility within the panel cannot be provided, concrete shear resistance in these components should be sufficient to resist the full elastic earthquake force to ensure that shear failure does not occur. Base shear equations in [ASCE/SEI 7](#) reflect the inherent ductility provided by various concrete shear wall systems by increasing or decreasing the response modification coefficient R . In areas of higher seismicity, tilt-up shear walls are typically classified as intermediate precast structural walls or special structural walls. Intermediate systems are assumed to behave with less ductility than special systems and, therefore, are subject to higher design forces. In addition, ASCE/SEI 7 limits the maximum height of buildings using intermediate precast concrete structural walls. ACI 318-11, Chapter 21 provides commentary on the use of various concrete structural walls with consideration for their different levels of ductility.

Design for in-plane shear in wall panels is further complicated by establishing the design base shear; a large flexible roof diaphragm and stiff walls affect the fundamental building period that determines the base shear of the building. Apply the provisions of ACI 318-11, 11.9 and 21.4 for in-plane forces in tilt-up panels.

8.5—In-plane frame design

The trend in tilt-up buildings is for an increasingly higher percentage of openings in the panels. This is particularly true for retail, office, and other commercial buildings. Relative stiffness of these panels may be much smaller than for solid panels, and it is becoming necessary to design these as frames rather than as solid shear wall panels. This gives little prescriptive guidance in classifying whether

narrow wall segments are better judged as frames or shear walls. The aspect ratio of wall piers should be close to the aspect ratios prescribed in **ACI 318-11**, 21.5.1 and 21.6.1 in the design of wall frames. For special structural walls, the International Building Code (**International Code Council 2012**) and **ASCE/SEI 7** provide guidance on classifying narrow wall segments as either frame-like columns or wall piers. Wall piers can be viewed as a transitional shear wall element between a traditional shear wall and a frame-column. Wall segments created between openings and panel joints are judged considering their height-to-length ratio and their length-to-thickness ratio. As wall segments become more square in cross section or taller and more slender in profile, they behave more as frame elements instead of stiff shear walls. As wall segments behave more frame-like, flexural ductility becomes more important, and the added provisions of the International Building Code (International Code Council 2012); ASCE/SEI 7; and ACI 318-11, Chapter 21 address this issue.

Wall piers within special structural walls are designed and detailed to encourage flexural failure before shear failure. Horizontal shear reinforcement within the wall piers are required to be hooked, closely spaced, and in sufficient quantity to fully develop the maximum probable moment M_{pr} at the top and bottom of the wall segment. The intent is to provide a more ductile failure mode by forcing the flexural reinforcement to elongate and yield before a brittle concrete shear failure mode occurs. Shear wall lines where wall pier elements are resisting less than one-sixth of the wall line force may neglect the wall pier detailing requirements.

Frame-columns within special structural walls are primarily flexural elements and are subjected to the detailing requirements of ACI 318-11, 21.5 or 21.6, depending on the level of axial load. These sections contain special confinement reinforcement provisions to increase flexural ductility.

Often, tilt-up panel configurations contain isolated wall segments that could be classified as frame-columns due to their dimensional characteristics, yet are contributing little to the overall seismic resistance of the wall line. In these situations, any contribution that a frame-column provides can be ignored at the licensed design professional's discretion, if the column is not necessary for gravity support of the wall, or the design complies with ACI 318-11, 21.13.

8.6—Lateral analysis of wall panels linked in-plane

Assumed panel-to-panel transfer mechanisms affect how connections are designed and the extent of deformation required to activate the assumed mechanism. There are several ways in which load is assumed to transfer between linked panels, four models of which are:

- 1) Traditional elastic theory
- 2) Shear-only
- 3) General inelastic
- 4) Strut-and-tie

In each case, equilibrium and ductility are the most important factors to satisfy in the connection design. A sample of the calculations associated with achieving equilibrium is shown in Fig. 8.6. Ductility permits the redistribution

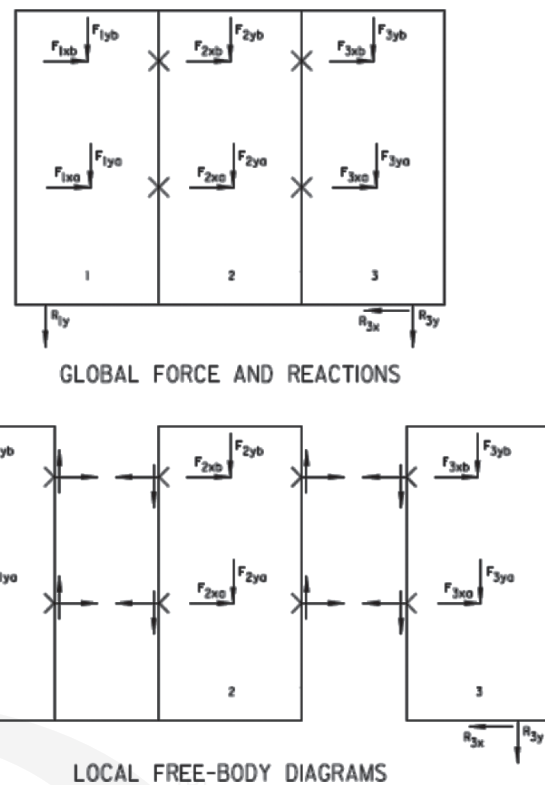


Fig. 8.6—Free-body diagrams for traditional elastic theory connections.

of internal stresses and forces by avoiding brittle failure modes. Common brittle failures in steel-to-concrete connections include concrete breakout in tension and shear, pullout, pryout, and side-face blowout. Brittle steel failure modes include tension and shear rupture, buckling, and weld fracture.

CHAPTER 9—CONNECTIONS FOR TILT-UP PANELS

Connections should be designed to resist forces equal to or greater than the maximum load imposed on the panel component and designed in accordance with the provisions of ACI 318-11, Appendix D for concrete anchorage and **AISC 360** for steel component design. Connections for panels designed to resist seismic forces may have more stringent ductility requirements required by building codes; all load combinations should be checked for the controlling forces in connection design.

9.1—Connection types

There are three main types of connections used for tilt-up panels:

- 1) Cast-in-place
- 2) Welded embedded metal
- 3) Post-installed anchors

9.1.1 Cast-in-place concrete—These connections involve casting concrete around steel reinforcement projecting from the panel to tie into an adjacent panel or another building component. These are often very strong and can be used to distribute loads over a considerable length. Good ductility can be achieved if the overlapping bars are confined by

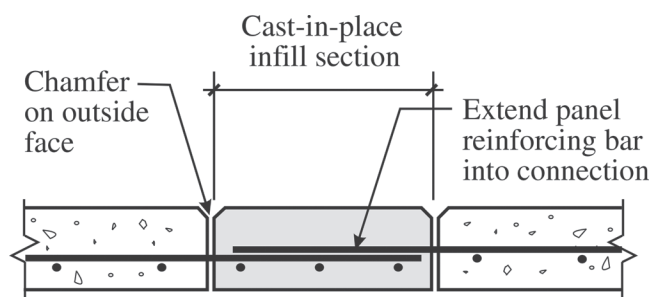


Fig. 9.1.1a—Cast-in-place infill panel.

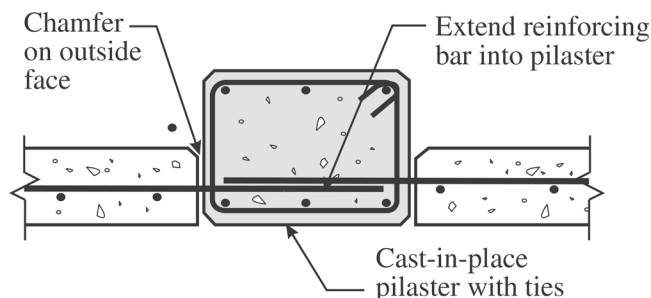


Fig. 9.1.1b—Cast-in-place pilaster.

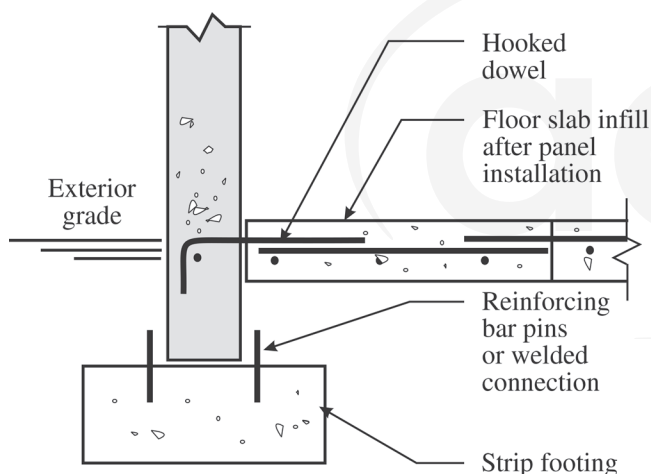


Fig. 9.1.1c—Floor slab infill.

closed ties. Cast-in-place connections are used infrequently because they are usually more expensive than other connections. They could also cause problems, such as panel cracking resulting from concrete shrinkage and excessive restraint. Figures 9.1.1a through 9.1.1c show common cast-in-place connections used in tilt-up structures.

9.1.2 Welded embedded metal—Welded embedded metal is the connection preferred by most designers and builders due to relative cost and construction flexibility in various applications. Strength and ductility vary considerably, depending on embedment length and anchor configuration. Steel plates with studs are suitable for shear and tension forces as long as they are located well away from panel edges, which is generally 12 in. (305 mm) or more. Where there is insufficient embedment, connectors will fail in a brittle (nonductile) manner. Steel angles or plates with

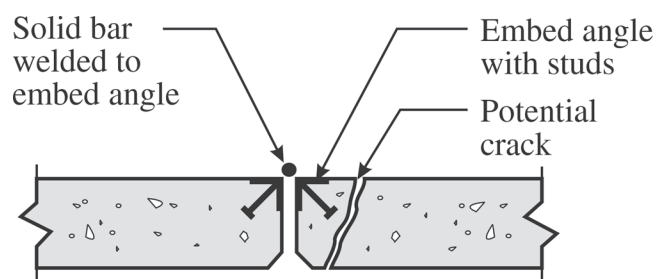


Fig. 9.1.2a—Edge connector with studs.

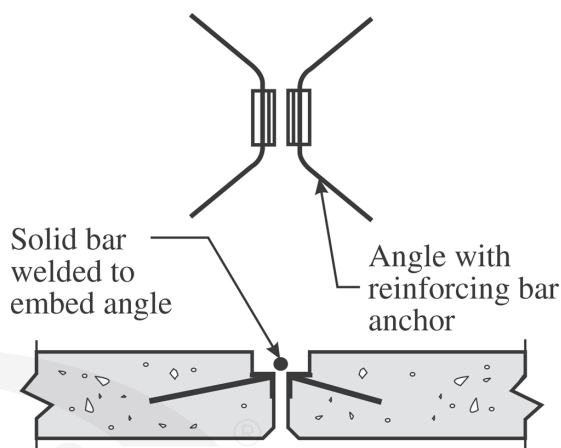


Fig. 9.1.2b—Edge connector with reinforcing bar.

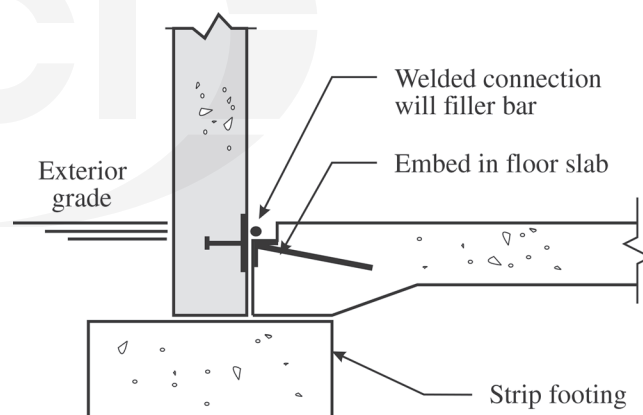


Fig. 9.1.2c—Welded connection to floor slab.

short-length headed studs located at the panel edge should be avoided unless supplemented with reinforcing steel. Sometimes, the concrete shrinkage or thermal effects that occur in the panels after they are welded result in a buildup of stresses in and around the connection. Brittle connections, such as shear plates with short studs, can fail completely, as illustrated in Fig. 9.1.2a. Figures 9.1.2a through 9.1.2h illustrate typical examples of welded connections used for tilt-up panels.

9.1.3 Post-installed anchors—Expansion and adhesive anchors are used extensively in tilt-up construction. Post-installed anchors are often used where the cast-in anchor may have been omitted or misaligned. Expansion anchors can have problems with premature failure in thin panel sections, particularly where the distance is inadequate.