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.

**CONTENTS**

**CHAPTER 1—INTRODUCTION, p. 2**

**CHAPTER 2—NOTATION AND DEFINITIONS, p. 2**

2.1—Notation, p. 2  
2.2—Definitions, p. 3

**CHAPTER 3—ANALYSIS CONCEPTS FOR SLENDER CONCRETE WALLS, p. 4**

3.1—Panel design model, p. 4  
3.2—Bending stiffness evaluation, p. 4  
3.3—Iteration method for $P$-$\Delta$ effects, p. 6

**CHAPTER 4—LOADING CONDITIONS, p. 9**

4.1—Lateral loads, p. 9  
4.2—Axial loads, p. 10  
4.3—Panel self-weight, p. 11  
4.4—Load factors and combinations, p. 11

**CHAPTER 5—MINIMUM REINFORCEMENT, p. 11**

5.1—General, p. 11  
5.2—ACI 318 provisions, p. 12

**CHAPTER 6—CONTROL OF DEFLECTIONS, p. 12**

6.1—Creep and initial deflections, p. 13  
6.2—Deflection calculations, p. 13  
6.3—Deflection limits, p. 13

**CHAPTER 7—PANEL DESIGN PROCEDURES, p. 14**

7.1—Solid panels without openings, p. 14  
7.2—Panels with openings, p. 14  
7.3—Concentrated axial loads, p. 14  
7.4—Concentrated lateral loads, p. 15  
7.5—Multiple spans and effects of continuity, p. 15  
7.6—Isolated footings or pier foundations, p. 16  
7.7—Cantilever panels, p. 16

**CHAPTER 8—IN-PLANE SHEAR, p. 17**

8.1—Resistance to panel overturning, p. 18

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8.2—Resistance to sliding, p. 18
8.3—Concrete shear resistance, p. 19
8.4—Seismic ductility, p. 19
8.5—In-plane frame design, p. 19
8.6—Lateral analysis of wall panels linked in-plane, p. 20

CHAPTER 9—CONNECTIONS FOR TILT-UP PANELS, p. 20
9.1—Connection types, p. 20
9.2—Design considerations, p. 22

CHAPTER 10—CONSTRUCTION REQUIREMENTS, p. 25
10.1—Forming and construction tolerances, p. 25
10.2—Concrete for tilt-up panels, p. 25
10.3—Panel reinforcement, p. 26

CHAPTER 11—DESIGN FOR LIFTING STRESSES, p. 26
11.1—General lifting concepts, p. 26
11.2—Steps for performing a lifting design, p. 27
11.3—Lifting considerations: building engineer of record, p. 27
11.4—Lifting design considerations: panel specialty engineer, p. 28

CHAPTER 12—TEMPORARY PANEL BRACING, p. 29
12.1—Brace geometry and number of braces, p. 29
12.2—Knee and lateral bracing, p. 29
12.3—Bracing to slab-on-ground, p. 29
12.4—Deadmen, p. 29
12.5—Base sliding, p. 29
12.6—Alternate bracing methods, p. 30

CHAPTER 13—REFERENCES, p. 30
Authored references, p. 30

APPENDIX A—DERIVATION OF $M_o$ AND $I_{cr}$, p. 30
A.1—Derivation of $M_o$ and $I_{cr}$ based on rectangular stress block, p. 30
A.2—Derivation of $M_o$ and $I_{cr}$ based on triangular stress distribution, p. 31

APPENDIX B—DESIGN EXAMPLES FOR OUT-OF-PLANE FORCES, p. 31
B.1—Panel with no openings design example, p. 33
B.1M—Panel with no openings design example (metric), p. 35
B.2—Panel with a 10 x 15 ft door opening design example, p. 39
B.3—Panel with concentrated axial load design example, p. 44
B.4—Panel with concentrated lateral load design example, p. 48
B.5—Multi-story panel design example, p. 51
B.6—Panel with dock-high condition design example, p. 56
B.7—Plain panel with fixed end design example, p. 61
B.8—Plain panel on isolated footing or pier design example, p. 65
B.9—Panel with stiffening pilasters and header design example, p. 68

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
$A_c$ = gross area of concrete section, in.$^2$ (mm$^2$)
$A_r$ = area of tension reinforcement, in.$^2$ (mm$^2$)
$A_{et}$ = effective area of tension reinforcement, in.$^2$ (mm$^2$)
$A_s$ = area of shear reinforcement, in.$^2$ (mm$^2$)
$h$ = depth of equivalent rectangular stress block, in. (mm)
$b_d$ = design width, in. (mm)
$b_t$ = tributary width, in. (mm)
$b_s$ = 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$e$ = 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)
ecc$ = eccentricity of applied load(s), in. (mm)
$F$ = loads due to weight or pressure of fluids
$F_o$ = factored load
$f'_{cu}$ = specified compressive strength of concrete, psi (MPa)
f$R$ = modulus of rupture, psi (MPa)
$f_y$ = reinforcement yield stress, psi (MPa)
$G_C$ = external pressure coefficient
$G_C^{ps}$ = 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.$^4$ (mm$^4$)
$I_e$ = effective moment of inertia, in.$^2$ (mm$^2$)
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

2.2—Definitions


- **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 otherwise, 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-Δ 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-Δ 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-Δ 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 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

\[ K_s = \frac{M_{\text{max}}}{\Delta_{\text{max}}} \]

Simplify the procedure for calculating deflection and final bending moment by using a constant value for the bending stiffness \( K_s \) 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 \( M_{\text{max}} \) will occur at midheight and the maximum deflection \( \Delta_{\text{max}} \) is given by

\[ \Delta_{\text{max}} = \frac{5wf^4}{384EI_m} = \frac{5M_{\text{max}}}{48EI_m} = \frac{M_{\text{max}}}{9.6EI_m} \]

in which

\[ M_{\text{max}} = \frac{wf^2}{8} \]

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

\[ \Delta_{\text{max}} = \frac{M_{\text{max}}}{8EI_m} \]
Fig. 3.2a—Maximum deflection due to lateral load only.

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

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

\[ \Delta_{\text{aux}} = \frac{5M_{\text{aux}}}{48E_J I_e} = \frac{M_{\text{max}}}{K_b} \]

Bending stiffness, \( K_b \), for a slender wall is therefore defined as

\[ K_b = \frac{48E_J I_e}{5\ell^2} = \frac{9.6E_J I_e}{\ell^2} \]

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, \( P_{cr} \),

\[ P_{cr} = \frac{\pi^2 E_J I_e}{\ell^2} = 9.87 \frac{E_J I_e}{\ell^2} \]

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-curvature deflected shape due to
the effects of a concentric axial load only. The applicable units for \( F_c \) 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 \( E_{eff} \) 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 \( E_{eff} \) equal to the cracked section stiffness \( E_{cr} \) correlates closely with the load-deflection characteristic of the test results. Ultimate load deflections using the preceding \( \Delta_{max} \) equations will likely be overestimated. The cracked section moment of inertia, \( I_{cr} \), can be taken as

\[
I_{cr} = nA_w \left( d - c \right)^2 + \frac{bc^3}{3}
\]

where

\[
c = \alpha/\beta_1
\]

\[
a = \frac{A_s f_y}{0.85 f_b}
\]

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

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

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

\[
n = E_{eff} / E_{cr}
\]

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 \( f_c \) 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 0.10\( f'_c \), this can be accounted for by a simple modification of the area of reinforcement as follows

\[
A_w = A_t + \frac{P_a}{f_y} \left( \frac{h}{2a} \right)
\]

\( A_{cr} \) can also be used to account for the increased bending stiffness when computing \( P-\Delta \) deflections.

The assumption that concrete section stiffness is equal to \( E_{eff} \), 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, 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 \( P-\Delta \) 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 \( P-\Delta \) moment

\[
M_{max} = M_a + P\Delta_{max}
\]

The relation between maximum bending moment and deflection is

\[
\Delta_{max} = \frac{5M_{max}E_{eff}}{48E_{cr}I_{cr}} = \frac{M_{max}}{K_b}
\]

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
- \( e_{cc} = 3 \text{ in. (76 mm)} \)
- \( E_{cr}I_{cr} = 45 \times 10^6 \text{ lb-in.}^2 (129 \times 10^6 \text{ N-mm}^2) \)
- \( K_b = \frac{48E_{cr}I_{cr}}{5\ell^2} = 48 \times \frac{45 \times 10^6}{5 \times 20^2 \times 12^2} = 7500 \text{ in.-lb/in. (33 kN-mm/mm)} \)
- \( M_a = 25 \times \frac{20^3}{8} + 4000 \times \frac{1}{2} \times \frac{1}{12} = 1250 + 500 = 1750 \text{ ft-lb (25.5 kN-m)} \)

Start with:
\[ \Delta_b = \frac{M_s}{K_b} = \frac{21,000}{7500} = 0.233 \text{ ft or 2.80 in. (71 mm)} \]

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

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

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

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

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

\[ \Delta_4 = 0.4595 \text{ ft (152 mm)} \]

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

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

3.4—Moment magnifier method

ACI 318 adopted the magnifier equation for evaluation of the \( P-\Delta \) 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)

\[ M_{\text{max}} = M_a + P\Delta_{\text{max}} \]

where

\[ \Delta_{\text{max}} = \frac{5M_{\text{max}}\delta^2}{48E_s I_{cr}} = \frac{M_{\text{max}}}{K_b} \]

or

\[ K_b = \frac{M_{\text{max}}}{\Delta_{\text{max}}} = \frac{48E_s I_{cr}}{5\delta^2} \]

Maximum moment \( M_{\text{max}} \) can now be written in the following form

\[ M_{\text{max}} = M_a + \frac{PM_{\text{max}}}{K_b} \]

This equation can be rewritten as

\[ M_{\text{max}} = M_a \left( \frac{1}{1 - \frac{P}{K_b}} \right) = M_s \delta_b \]

where

\[ \delta_b = \frac{1}{1 - \frac{P}{K_b}} \]

Using the values from the previous example,

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

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

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

\[ \delta_b = \frac{1}{1 - \frac{4000}{7500}} = 2.143 \]

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

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

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 \( \phi M_u \) for combined flexure and axial loads at the midheight cross section shall be

\[ \phi M_u \geq M_u \]

(14-3)

where

\[ M_u = M_{\text{max}} + P_u \Delta_u \]

(14-4)

and

\[ \Delta_u = \frac{5M_{\text{cr}} \delta^2}{0.75(48E_s I_{cr})} \]

(14-5)

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

\[ M_u = \frac{M_u}{1 - \frac{5P_u \delta^2}{0.75(48E_s I_{cr})}} \]

(14-6)

where

\[ I_{cr} = \frac{E_s}{E_c} \left[ A + \frac{P_u}{f_c} \left( \frac{h}{2d} \right) \right] (d - c)^2 + \frac{\xi_c c^2}{3} \]

(14-7)
The term $b$ will be used in place of $t_w$ for the remainder of this guide to avoid confusion with the term $t_c$ for wall height and the substitution of $A_{se}$ defined by

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

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 $E J_{mv}$, 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 0.06$f_y^2$ for the vertical stress $P/A_y$ at the midheight section.

(b) Using a constant value for panel bending stiffness $K_p$ 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

$$\phi M_n \geq M_{cr}$$

(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

$$M_{cr} = f_s S$$

(9-9)

where

$$S = \frac{bh^2}{6} = \frac{I_y}{y_i} \quad \text{(uncracked section modulus)}$$

$$f_s = 7.5 \lambda \sqrt{f_y}$$

(9-10)

where $\lambda$ 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 $\phi$ 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 when $c/d$ is 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 $P$-$\Delta$ 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.

$$M_n = M_\phi + P_n A_{se} \leq \phi M_u$$

where

$$M_u = \frac{w_y e_f^2}{8} + \frac{P_y e_{se}}{2}$$

Other forces contributing to the primary moment, such as lateral out-of-plane point loads, need to be included when computing $M_u$. The effect of panel self-weight should also be taken into account in the $P$-$\Delta$ 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
where, for normal weight concrete

\[ E_c = 57,000 \sqrt{f_y'} \text{ psi (in.-lb units)} \]

\[ E_c = 4700 \sqrt{f_y'} \text{ MPa (SI units)} \]

and

\[ I_{cr} = nA_o (d - c)^2 + \frac{bc^3}{3} \]

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

\[ A_{o} = \frac{P_v + A_s f_y}{f_y} \]

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 $\phi$ is greater than $M_{cr}$

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

\[ \Delta_s = \frac{f}{150} \]

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 $M_c$ rather than $\phi M_c$.

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

\[ \frac{L_c}{h} \]

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 load—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

\[ q_z = K_d \frac{K_z}{K_{t}} (\frac{z}{h})^{1/2} \]

where

\[ P_v = 0.00256 \times 0.98 \times 1.0 \times 0.85 \times 115^{1/2} \times 1.0 = 28.2 \text{ lb/ft}^2 \]

(1.35 kPa)
where \( GC_p = \pm 0.18 \); \( +P(\text{pressure}) = 28.2(0.7 + 0.18) = 24.8 \text{ lb/ft}^2 \) (1.19 kPa); and \( -P(\text{suction}) = 28.2(0.8 + 0.18) = 27.6 \text{ lb/ft}^2 \) (1.32 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: \( F_p = 0.40 \times I_e \times S_{OS} \times \text{panel weight but not less than 10 percent of the panel weight.} \)

Given: 8 in. (200 mm) thick tilt-up wall panel
- \( S_5 = 0.75 \)
- Site Class D
- \( I_e = 1.0 \)
- \( S_{OS} = F_a \times S_5 = 1.2 \times 0.75 = 0.90 \)
- \( S_{OS} = 2/3 \times S_{OS} = 2/3 \times 0.90 = 0.60 \)
- \( F_p = 0.40 \times 1.0 \times S_{OS} \times \text{panel weight} = 0.40 \times 0.60 \times 100 \text{ lb/ft}^2 \) (4.8 kPa) = 24 lb/ft^2 (1.15 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-A 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 lb/ft^2 (0.48 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 lb/ft^2 (0.24 kPa) is recommended, 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.
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 0.06\( f_c' \).

4.3—Panel self-weight

The effect of panel self-weight should be considered because it represents a significant contribution to \( P-\Delta \) 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

\[
U = 1.2(D + 1.6(L, \text{ or } S \text{ or } R)) + (1.0L \text{ or } 0.5W) \quad (9-1)
\]

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

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

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

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

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 1.6\( W \) and 0.8\( W \) in the previous equations for 1.0\( W \) and 0.5\( W \), 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 0.002\( A_{ce} \) 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.

If a wall resists in-plane shear force, 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-Δ 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 bending 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 symmetrically 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 \( \ell /150 \). The midheight deflection \( \Delta_s \) shall be determined as follows:

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

\[
\Delta_s = \frac{2}{3} \Delta_{cr} + \frac{(M_{cr} - (2/3)M_{cr})}{(M_{cr} - (2/3)M_{cr})} (\Delta_e - (2/3)\Delta_{cr}) \quad (14-8)
\]

If \( M_{cr} \) does not exceed \((2/3)M_{cr} \), \( \Delta_e \) shall be calculated

\[
\Delta_e = \left( \frac{M_{cr}}{M_{cr}} \right) \Delta_{cr} \quad (14-9)
\]

where

\[
\Delta_{cr} = \frac{5M_{cr}}{48EJ_{cr}} \quad (14-10)
\]

\[
\Delta_e = \frac{5M_{cr}}{48EJ_{cr}} \quad (14-11)
\]

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 \( 2/3 M_{cr} \) (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: \( L/15 \) 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-A 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 \( \ell_{/50} \) where a single layer of reinforcement is desired, or \( \ell_{/65} \) 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.

\[ b_\text{d} = \text{design width} \]

\[ b_\text{t} = \text{tributary width} \]

\[ h = \text{panel thickness} \]

\[ D + 0.5L + W_a \]

\[ b_\text{d} = 12h \max \]

\[ b_\text{t} = 12h \max \]

**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, \( b_\text{d} \), in the panel
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

Roof

Header beam over opening

Pilaster at edge of opening

Fig. 7.2b—Stiffening header and pilasters.

Roof

Beam supported on pilaster

Floor

Fig. 7.3—Pilaster supporting beam load.

Where loads are very large, which is greater than 0.10f_e b w 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 accor­
dance 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 consid­
ered 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:

(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

\[ W = \text{canopy load} \]
\[ R_1 = \text{end reaction} = \frac{W}{2} \frac{2b}{h} \]
\[ H = \text{horizontal line load} = \frac{W}{2} b \]

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 reinforce­
ment 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 interme­
tiate 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. 7.4—Suspended canopy on panel.
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-Δ 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-Δ magnifications. Primary moments are calculated by conventional elastic methods. Both positive and negative moments can then be increased proportionally by the P-Δ 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 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.
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, $P$-$\Delta$ 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

$$\Delta = \frac{M_n}{4EI} = \frac{M_n}{K_n}$$

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 $P$-$\Delta$ 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
(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.

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:

\[ M_o = V_{\text{roof}} l_{\text{roof}} + V_{\text{floor}} l_{\text{floor}} + (V_{\text{panel}} l_{\text{panel}}) \]

where the resisting moment is given by

\[ M_R = (W_{\text{roof}} + W_{\text{floor}} + W_{\text{panel}})h/2 + V_{\text{R.main}} l_{\text{main}} \]

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,
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

\[ V_s > 0.5 \phi V_c \]

where \( V_s = 2 \sqrt{f_y b_s d} \) (in.-lb) \[ V_s = 0.17 \sqrt{f_y b_s d} \] (SI).

The minimum area of shear reinforcement is given by

\[ A_{s,\text{min}} = 0.75 \sqrt{f_y b_s d} \] (in.-lb)

\[ A_{s,\text{min}} = 0.62 \sqrt{f_y b_s d} \] (SI)

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

\[ A_s = b_s d \]

\[ A_s = \frac{50}{60,000} A_s = 0.000833 A_s \]

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_s, but the minimum requirements listed in Chapter 5 of this guide should be carefully reviewed for any additional reinforcement needed.

Where factored shear force \( V_s \) exceeds concrete shear strength \( \phi V_c \), minimum shear reinforcement according to ACI 318-11, Chapter 11 should be provided to satisfy

\[ \phi V_c \geq V_s \] (11.1)

\[ V_n = V_c + V_s \] (11.2)

\[ V_s = A_s f_y d \] (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 elements. ACI 318 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 $M_p$ 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 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 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 edge distance is inadequate.
22 DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

Fig. 9.1.2d—Panel on “L” flooring.

Fig. 9.1.2e—Angle seat for steel roof joist.

Fig. 9.1.2f—Edge angle with tie strut.

Powder-driven fasteners or drive pins may be used for light architectural components or for connections to light-gauge steel stud framing. The design for connections using several types of post-installed anchors can be obtained from ACI 318-11, Appendix D, in combination with ICC-ES Evaluation Reports for the specific anchor product being specified.

9.2—Design considerations

Explicit analysis and design of cast-in-place concrete joints is not commonly considered in tilt-up construction, unless special seismic provisions are required for spandrel-to-pier joints, wall-to-foundation joints, wall-to-slab joints, or wall-to-wall joints. Refer to ACI 318-11, Chapter 21 for seismic design requirements. In a low or moderate seismic region where special detailing is not required, use sound concrete detailing practices:

(a) To account for reinforcement geometry in placement and fit for design and constructability, such as concrete covers, bar widths, bend radii, and hierarchy of layers of reinforcement.

(b) To provide adequate cover and spacing of reinforcement to avoid concrete consolidation issues, reduce corrosion, and improve anchorage of reinforcement.

(c) To provide adequate development of reinforcement based on the seismic design category: standard straight and hooked bar development and splice lengths per ACI 318-11, Chapter 12 or seismic straight and hooked bar development and splice lengths per ACI 318-11, Chapter 21.
(d) To confine concrete—for example, closely spaced stirrups and ties—where substantial yielding of reinforcement is expected.

Steel anchorage to concrete is often the source of construction and performance issues. This class of connections is relatively sensitive to proper detailing and construction, where a shortcoming in either can disproportionately reduce the connection capacity. There are several nonductile failure modes in anchorage-to-concrete, and nonductile failure modes are not forgiving to unexpected load actions. An unexpected load action or magnitude of load can suddenly fail the connection without the ability for redistribution, leaving it with little or no residual capacity. Engineers should carefully consider critical connections, particularly where there is little or no redundancy and the consequences of failure are significant. Refer to ACI 318-11, Appendix D, for anchorage-to-concrete design provisions.

In tilt-up concrete construction, there are three issues that most affect anchorage-to-concrete design:

1) **Free edges of concrete**—While all concrete design should deal with anchorages influenced by concrete edges, the precast components of tilt-up necessitate joints that cast-in-place concrete construction would not have. This means that there are generally free edges in proximity to anchorages. Anchors in tilt-up panels should be designed taking into account the edge distance to these joints.

2) **Thin concrete thickness (small embedment depths) for connections located on panel faces**—The thin substrate of tilt-up walls can make a challenging design for even a simple shear connection if the reaction is significant. This issue is exacerbated in thin concrete layers in multi-wythe panels. Also, seismic design may require out-of-plane axial load to be resisted, and to make these types of connections ductile may require a large connection to spread out the load over an adequate area of concrete.

3) **Most field connections of precast components require steel-to-concrete connections**—Numerous field connections increase the possibility of improperly located or missing anchorages, requiring post-installed connections. Additionally, connections between precast components or precast components and cast-in-place concrete tend to concentrate forces at edges and corners of concrete components where the loads may have otherwise been more dispersed in cast-in-place concrete construction. This requires the connections to be more robust due to the magnitude of the load being resisted, as well the consequences of failure.

Unintended load actions can cause anchorages to fail at loads below their intended load-carrying capacity. Unintended load actions that may cause problems include the following:

(a) Forces locked in during panel bracing that are released when the braces are removed
(b) Axial loads due to restraint of temperature and shrinkage effects
(c) Moment due to flexural stiffness in connections assumed to be pinned
(d) Shear connections with eccentricities—for example, shear plates, seats

(e) Moment and magnified axial load due to unaccounted for eccentricities

9.2.1 **Magnifying effect of cantilever action**—The designer should anticipate conditions where an eccentric force could cause an unintended moment, or a moment must be resisted by a small distance between forces in a force couple. Realistic construction eccentricities in the design of anchorages should also be considered. Eccentricities create pyramidal actions that magnify loads on individual anchors, rows of anchors, and reinforcement. A small eccentricity of a force can cause a significant load increase on an anchorage if that force had been considered perfectly aligned in design. These unexpected loads can cause premature failures in anchorages. A minimum eccentricity of 1 in. (25 mm) is recommended based on construction tolerances for tilt-up panels found in most project specifications.

9.2.2 **Reinforcement limitations**—Anchor reinforcement is reinforcement designed to resist the full applied load on an anchorage. Anchor reinforcement should be oriented parallel to the line of action of the applied load, and developed on both sides of the theoretical concrete breakout plane. Anchor reinforcement cannot be considered additive to concrete breakout capacity because the concrete will resist nearly all the applied load, due to its stiffness, up to brittle failure. After that point, the reinforcement is required to resist the entire applied load. Also consider how close and parallel to the applied load that the reinforcement is to the anchorage, to account for the geometry of the breakout prism in the effectiveness of the reinforcement. Anchor reinforcement perpendicular to the applied load is not considered effective because the bar provides little stiffness and capacity perpendicular to its axis. The commentary to ACI 318-11, Appendix D, reviews and illustrates the required geometry of effective anchor reinforcement. Appendix D also requires a 0.75 resistance factor be used in the design of anchor reinforcement to account for uncertainty in assumed load and resistance model, similar to strut-and-tie models.

It is suggested that reinforcement and deformed bar anchors in connections be designed based on the more conservative capacity obtained from the development length equations of ACI 318-11, Chapter 12 or 21, as applicable, which capture splitting and pullout failure modes, and the ACI 318-11, Appendix D concrete breakout provisions, which capture concrete breakout modes. This approach is recommended when either of the following conditions exists at the connection:

(a) In a connection resisting flexure, the compression resultant is more than 1.5 times the embedment depth away from the tension resultant; an example is deep flexural members.
(b) The connection is in direct tension and there is no nearby compression confinement.

These recommendations are important in tilt-up panel-to-panel and panel-to-foundation connections where welded reinforcement and deformed bar anchors are commonly used and the connections are often subject to direct tension. Additionally, these recommendations may be necessary to avoid a brittle failure in critical seismic connections where substantial ductility is required.
9.2.3 Anchor type selection—Anchors in concrete can be cast-in or post-installed. Cast-in anchors are cast into the concrete and typically are headed shear studs, reinforcing bars, deformed bar anchors, or anchor rods. For the same configuration and similar anchor geometry, cast-in anchors generally provide more design capacity than post-installed anchors.

9.2.3.1 Cast-in anchors—With some limitations on diameters, embedment depths, concrete strengths, placement, and uses, ACI 318-11, Appendix D addresses the design of cast-in anchors, and post-installed expansion, undercut, and adhesive anchors. Notably not governed by Appendix D are through-bolts, screw anchors, and grouted anchors. Currently, screw anchors can be designed indirectly with Appendix D in conjunction with ACI 193-12. ACI 318-11, Appendix D is limited to cast-in anchors installed in concrete with a compressive strength up to 10,000 psi (70 MPa), and post-installed anchors installed in concrete up to 8000 psi (55 MPa), or as limited in anchor product reports. Concrete that has a higher strength than a post-installed anchor has been tested and approved for can be detrimental to the performance of the anchor. Adhesive bond can be worse; expansion anchors may not be capable of full expansion, and screw anchors may have excessive thread wear during installation. ACI 318-11, Appendix D design provisions also do not cover anchors subject to high-cycle fatigue and impact, nor anchorages located in the plastic hinge zones of concrete members.

As mentioned for screw anchors, ACI 193-12 can provide a useful mechanism to design products, such as some types of anchors, which fall outside the scope of design codes. In cases where the design of an anchor is not governed by ACI 318-11, Appendix D, use of ACI 193-12 in conjunction with the product's evaluation service report (ESR) is recommended. For example, AC 193-12 and the ESR for a screw anchor product will reference or modify portions of ACI 318-11, Appendix D to provide a satisfactory procedure for the design of screw anchors in concrete.

9.2.3.2 Post-installed anchors—These anchors tend to be more sensitive to installation procedures than cast-in anchors. Inadequate installation of post-installed anchors is often cited for their poor performance. As a result, continuous or periodic special inspections of the installations of post-installed anchors are required in the International Building Code (International Code Council 2012) and many state building codes. Furthermore, ACI 318-11, Appendix D requires that adhesive anchors installed in a horizontal or upwardly inclined orientation be installed and used by personnel certified according to the ACI-CRSI Adhesive Anchor Installer Certification program or equivalent.

Post-installed anchors that produce expansive forces are limited by how closely they can be installed to concrete edges, whereas cast-in anchors not torqued are limited only by concrete clear cover. Expansion, undercut, and screw anchors also have restrictions on embedment depth relative to the concrete thickness in which they are installed. Adhesive anchors have a gel time during which they cannot be disturbed, and a cure time before which they cannot be loaded. Adhesive products vary in how sensitive they are to installation and use in low and high temperatures. Additionally, they cannot be used to support fire-resistant construction unless the loads are transient or nonstructural, or the anchor is protected by a fire-resistive envelope or membrane.

For the design of anchors in concrete, concrete should be assumed to be cracked under service loads unless proven otherwise. For investigation of uncracked concrete, all load actions, including temperature, shrinkage, and secondary effects, should be considered. Construction experiences have shown that cracks tend to propagate toward anchors because anchors create a discontinuity in the concrete. As a result, only conditions similar to precompressed members (for example, prestressed members or a column with substantial compression force that can never experience tension) should be assumed to remain uncracked.

Post-installed anchors are installed in concrete after the concrete has hardened. Holes in concrete are made using a rotary hammer drill or core drill, and anchors are installed with an impact wrench, spud wrench, or by hand. Post-installed anchors transfer load to the concrete via friction, bearing, mechanical interlock, chemical bond, or a combination of these mechanisms. Common mechanical anchors include expansion and undercut and screw types, while bonded anchors are usually threaded rod or reinforcing bars installed in adhesive or grout.

Provisions governing post-installed anchors have changed dramatically in the past decade. To ensure a common threshold level of safety, ACI 318-11, Appendix D requires expansion and undercut anchors to be tested according to ACI 355.2, and adhesive anchors to be tested according to ACI 355.4. Product-specific capacities are often, if not always, limited to product-specific designs and specifications. Not all post-installed anchors of a given type and size are equal, so it should not be assumed that a 3/4 in. (19 mm) expansion anchor from two different manufacturers performs the same. The engineer should list design loads or a specific manufacturer on the design drawings and review anchor submittals proposed by the contractor for conformance with the project requirements. There are a few nuances to specifying post-installed anchors:

(a) Anchor length considerations include overall length, tabulated length, nominal embedment depth, and effective embedment depth.

(b) Effect of attachment thickness and washers, including threaded rod lengths for adhesive anchors and embedment depth of expansion and undercut anchors, critical to effective embedment depth screw anchors.

9.2.3.3 Adhesive anchors—Important developments in adhesive anchors include:

(a) The uniform bond model for adhesives used in ACI 318-11, Appendix D is limited to embedment depths between four anchor diameters and 20 anchor diameters.

(b) Per ACI 318-11, Appendix D, adhesive anchors cannot be installed within 21 days of concrete casting.

(c) Engineers should pay careful attention to conditions where an adhesive anchor is subjected to sustained tension. Not all conditions are as evident as direct tension from the
support of an overhead concrete panel. Sustained indirect tension describes anchors subjected to sustained tension that do not have a line of action in the same direction as the applied load. Two common examples of sustained indirect tension are horizontal anchors in ledge angles and seats subjected to vertical loads, and shear friction reinforcement that is:

i. Oriented horizontally, as dictated by ACI 318-11, 11.6
ii. Required to be fully developed on both sides of the shear plane. Adhesive anchors subject to sustained tension must be qualified for such use according ACI 355.4, and there is a substantial reduction factor on the tension capacity in this condition. Anchors installed horizontal or upwardly inclined must be installed by certified personnel and continuously inspected during installation.

CHAPTER 10—CONSTRUCTION REQUIREMENTS

This chapter provides guidelines for designers when specifying construction requirements. ACI 551.1R has a more comprehensive discussion of construction aspects.

10.1—Forming and construction tolerances

Required tolerances for tilt-up construction are generally covered in ACI 318 and ACI 117. Quality control of panel forming and steel reinforcement placing for site-cast tilt-up concrete panels is usually better than for conventional cast-in-place concrete walls. The process uses a concrete floor slab or casting bed as the primary surface for forming the panels in a horizontal position, thus minimizing alignment, maintenance, and deflection of traditional vertical-forming systems. Tolerance requirements can, therefore, be between those for cast-in-place and factory precast. Recommended tolerances for use in the tilt-up industry are found in Table 10.1a.

Table 10.1a—Recommended tolerances for use in the tilt-up industry

<table>
<thead>
<tr>
<th>Length and height</th>
<th>Straightness or skewness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 10 ft (3 m)</td>
<td>Up to 10 ft (3 m)</td>
</tr>
<tr>
<td>+ 0 to –1/2 in. (13 mm)</td>
<td>± 1/2 in. (13 mm)</td>
</tr>
<tr>
<td>10 to 20 ft (3 to 6 m)</td>
<td>10 to 20 ft (3 to 6 m)</td>
</tr>
<tr>
<td>+ 0 to –1/2 in. (13 mm)</td>
<td>± 5/8 in. (16 mm)</td>
</tr>
<tr>
<td>Over 20 ft (6 m)</td>
<td>20 to 40 ft (6 to 12 m)</td>
</tr>
<tr>
<td>+ 0 to –5/8 in. (16 mm)</td>
<td>± 3/4 in. (20 mm)</td>
</tr>
<tr>
<td>Thickness: Overall ±1/4 in. (6 mm)</td>
<td></td>
</tr>
</tbody>
</table>

Concrete clear cover to reinforcement provides protection to the steel from corrosion and fire. Increasing cover also improves the bond of reinforcement. Ultimately, reinforcement location can be bound by concrete cover, which in turn affects analysis and design. In slender members, the loads to be resisted by the member are particularly sensitive to member stiffness. A small change in the location of reinforcement can have a substantial impact on stiffness. This effect how loads are distributed within the section and should be considered in the design of the reinforcement.

Load and resistance factors in ACI 318 are in-part calibrated considering variations in the as-built placement of reinforcement and concrete member extents relative to the specified design geometry. However, there appears to be no records of as-built studies of tilt-up construction tolerances. It is likely tilt-up construction meets or improves on the tolerances for cast-in-place concrete, and it could approach the tolerances of plant-manufactured precast concrete.

ACI 318-11, Chapter 7 provides minimum concrete cover requirements to reinforcement for cast-in-place and precast concrete manufactured under plant control conditions. ACI 318-11, R 7.7.3 states, “Structural elements precast at the job site will also qualify under this section if the control of form dimensions, placing of reinforcement, quality control of concrete, and curing procedure are equal to that normally expected in a plant.”

Concrete cover distances are measured from the surface of a reinforcing bar to the nearest edge or surface of concrete considering architectural features such as reveals, recesses, form liners, sandblasting, and chamfers. The minimum cover distances in Table 10.1b are recommended for use with tilt-up concrete where edges of panels at joints are considered exposed to weather.

Table 10.1b—Minimum cover distances recommended for use with tilt-up concrete

<table>
<thead>
<tr>
<th>Exposure</th>
<th>Concrete cover, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Concrete exposed to earth or weather:</td>
<td></td>
</tr>
<tr>
<td>No. 9 (No. 29M) bar and larger No. 8 (No. 25M) bar and smaller</td>
<td>1-1/2 (40 mm) 1 (25 mm)</td>
</tr>
<tr>
<td>No. 14 (No. 43M) and No. 18 (No. 57M) bars No. 11 (No. 36M) bar and smaller</td>
<td>1-1/2 (40 mm) 3/4 (20 mm)</td>
</tr>
</tbody>
</table>

The values indicated in Table 10.1b consider only corrosion and constructibility; concrete cover may require an increase from these values to meet fire rating requirements. Some contractors have reported that concrete covers of less than 1 in. (25 mm) on the top face of the panel in the casting position make it difficult to finish the panel surface. In addition, bars with shallow cover on the face tend to mirror through the concrete surface, creating an objectionable appearance, but meeting structural requirements. Finally, concrete cover may have to be adjusted where reveals, recesses, architectural form liners, or sandblasting are used on the outside face of the panel.

10.2—Concrete for tilt-up panels

Concrete used for tilt-up panels should provide adequate strength for the in-place condition and for panel erection requirements. A minimum 28-day strength of 3000 psi (21 MPa) should be provided for in-place design requirements. Lifting insert manufacturers typically require a minimum compressive strength of 2500 psi (17 MPa) at the time of the lift to ensure that the full load capacity of their product can be achieved. Because this may be needed only 3 to 5 days after casting the panels, higher 28-day strength is often desired. Sometimes, the concrete mixture is proportioned for flexural strength requirements as defined by the modulus of rupture. This property is important for resisting flexural cracking, particularly during the lifting operation.
Supplementary cementitious materials (SCMs), such as fly ash, can slow the rate of early strength gain, delay panel lifting, and affect panel finishing.

Entrained air for resistance to freezing and thawing may not be required because exposed surfaces are primarily vertical and protected with a paint or sealer. If not properly accounted for, air entrainment may reduce strength and, therefore, increase the potential for flexural cracking during lifting. Conversely, where saturated conditions are anticipated at the base or parapet of the panel, panel concrete design mixes with air entrainment should be considered. Local experience in the performance of concrete without air entrainment should be investigated. Recommended concrete specifications are found in Table 10.2.

### Table 10.2—Recommended concrete specifications

<table>
<thead>
<tr>
<th>Concrete property</th>
<th>Value(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28-day compressive strength</td>
<td>3000 psi (21 MPa) minimum</td>
</tr>
<tr>
<td>28-day flexural strength</td>
<td>500 psi (3.5 MPa) minimum</td>
</tr>
<tr>
<td>Maximum size aggregate</td>
<td>1 to 1-1/2 in. (25 to 38 mm) minimum clear distance between bars of 3 in. (76 mm) or more</td>
</tr>
<tr>
<td></td>
<td>3/4 to 1 in. (19 to 25 mm) for minimum clear distance between bars of less than 3 in. (76 mm)</td>
</tr>
</tbody>
</table>

Note: Use of larger aggregate size can reduce shrinkage.

#### 10.3—Panel reinforcement

**Grade 60** (Grade 420) reinforcement is typically specified for tilt-up panels. Grade 40 (Grade 280) should be avoided because it is difficult to obtain in large quantities and may be installed in the wrong locations if confused with other grades of steel used on the job site. The main vertical bars are typically No. 5 or No. 6 (No. 16M or No. 19M), with No. 4 or No. 5 (No. 13M or No. 16M) bars used for horizontal temperature reinforcement, and No. 3 or No. 4 (No. 10M or No. 13M) bars for transverse ties and stirrups. Bars larger than No. 6 (No. 19M) are not recommended for tilt-up panels where the panel thickness is less than 8 in. (203 mm).

**Nominal versus design resistance**—The use of high-strength reinforcing steel—for example, yields stress greater than 60 ksi (420 MPa)—should be carefully considered in the design of tilt-up walls. Since ACI 318-11, 14.8 requires the wall to be tension-controlled, the substitution of high-strength steel for 60 ksi (420 MPa) steel will reduce the area of steel; however, if there is too much tension capacity in the cross section, the section can switch to being compression-controlled or in the transition zone on the resistance factor. So, whereas the nominal resistance of the section may increase with an increase in yield strength for a given area of steel, the design resistance may increase by a lesser amount, or even decrease. ACI ITG-6R demonstrated this when it reviewed ASTM A1035/A1035M reinforcing steel with steel grades of yield strengths of 100 and 120 ksi (690 and 830 MPa).

**Determination of yield stress**—For high-strength reinforcement, how the yield point is determined is important. ACI 318-11, 3.5.3 requires $f_y$ to be measured at 0.35 percent strain, not at the typical 0.2 percent offset, for $f_y > 60$ ksi (420 MPa).

**Nonlinear behavior**—The nonlinear behavior of a slender wall cannot be ignored when estimating the effect of a substitution using high-strength steel. A reduction in steel area generally causes additional $P$-$\Delta$ moment, leading to more required steel, partly offsetting the perceived savings on steel reinforcement.

**Serviceability considerations**—Serviceability and load-deformation of reinforced concrete typically do not depend on the yield strength of the reinforcement. Cross section area and modulus of elasticity are the critical properties. This is not only an issue for $P$-$\Delta$ effects and deflection of the wall, but limiting service cracking as well. Limiting cracking is important in tilt-up construction where the structure is exposed to the elements. Panel reinforcement may need to be increased to satisfy the service level deflection provisions of ACI 318-11, 14.8.4. Regardless, well-distributed horizontal and vertical reinforcement will assist in controlling cracks in the panel.

### CHAPTER 11—DESIGN FOR LIFTING STRESSES

Panel lifting analysis and lift insert design is commonly performed by a specialty engineer, often employed by the hardware manufacturer. The specialty engineer uses either proprietary software developed by the hardware manufacturers or their own specially-developed software to perform the lifting analysis. In every case, the software determines the geometry of the rigging, the forces in the cables, and the moments and flexural stresses in the panel as it rotates from a horizontal to a vertical position. Traditionally, the panel is treated as a continuous beam with elastic supports at the lift points that continually change in stiffness as the beam is rotated about its base. More sophisticated computer-based finite element analysis can be used in place of the traditional beam theory. Lifting and bracing of the tilt-up panels are considered means and methods of construction and not within the scope of work traditionally provided by the engineer of record. For this reason, the construction team will employ specialty engineers, either directly or through product suppliers, to perform these analyses.

**11.1—General lifting concepts**

The location of lift inserts is critical for safe and efficient handling of tilt-up concrete panels. The primary intent in lifting a panel is to lift it as an uncracked section. Because reinforcement steel is often placed at the center, the panel can experience cracking if the concrete tensile stresses are not kept within required design limits. In those instances where it is not practical to keep the concrete from cracking, face reinforcement is added to minimize the crack width during lifting.

Strongbacks are another way to minimize or avoid cracking during the lift operation. These are structural members that are connected intermittently to the panel so that the panel weight is transferred to the strongback. The strongback is designed to resist the full bending moment without relying...
on the panel, although the moments will be shared, so it is important that the strongback has the proper stiffness.

When lifting a panel from horizontal to vertical, it is often subjected to a higher range of stress than it will encounter from in-place loading. The most critical lateral bending stresses, which is bending side-to-side, occurs just as the panel is first raised from the horizontal position. The highest vertical bending stresses, which are bending top-to-bottom, generally occur between 30 and 50 degrees from the horizontal. Lift insert landing is often critical when the panel is almost vertical and the crane is carrying the full weight of the panel; it can also occur when initially breaking the panel loose from its casting bed due to sticking, suction, or dynamic forces. A factor of 1.25 is generally recommended in lift design to account for these conditions.

Lift inserts are usually located in such a manner as to minimize the panel lifting stresses, by balancing the positive and negative bending moments. They must also be arranged so that the panel will rotate freely about its base with little or no twisting, and hang level when free of the casting bed.

11.2—Steps for performing a lifting design

Computer-based analyses have changed the manner in which the lifting design of panels is approached, but the fundamental steps remain the same:

(a) Determine the panel weight, deducting for openings or recesses in the panel, and adding for pilasters and corbels.

(b) Locate the center of gravity.

(c) Determine an initial minimum number of lift points by first dividing twice the panel weight by the shear capacity of the lifting inserts. A minimum load factor of 2.0 is recommended to account for conditions that increase the panel stress at lifting, such as adhesion to the casting surface, suction created by ponded water, or moving a panel a significant distance or over rough terrain. Use judgment to determine the general pattern of lifting, which may be dictated by unusual panel shapes, such as those created by large, off-center openings or panel projections.

(d) Locate the inserts so that the geometric center of lift matches the horizontal center of gravity of the panel and is above the vertical center of gravity of the panel so that the top will come up first, and the panel will rotate about its base.

(e) Check the bending stresses in the panel. Traditionally, the panel is divided into beam strips in doing the analysis. Determination of beam strips is complicated in panels that are not symmetrical about a vertical centerline, as well as when lift inserts are horizontally offset.

(f) Adjust the vertical locations of inserts as necessary so that allowable stresses are not exceeded. The location of the top inserts generally has the greatest effect on bending moments. It is good practice to move inserts to the solid or heavy areas of the panel, away from openings. Judgment on the best way to adjust inserts comes with experience, but the goal should always be to keep bending stresses low enough to eliminate or minimize flexural cracking.

(g) Check lift insert tension capacity. Tension capacity of inserts is generally less than shear capacity, but a portion of the panel weight is still on the ground as it is first rotated, when maximum tension occurs. A 1.25 factor on the computed tension load at zero degrees is usually sufficient to allow for sticking or initial suction.

11.3—Lifting considerations: building engineer of record

The building engineer of record is responsible for the tilt-up panel design for the in-place loading only, but there are several basic lifting considerations for designing panels to maximize efficiency.

(a) The use of unusual panel shapes should be minimized where possible and as allowed by the architectural design. Adjust panel joints so that panels are as symmetrical as possible and minimize the use of, or avoid, L-shaped panels. Consult and coordinate with the project architect as necessary. By minimizing the difference between panels, grouping panels by type, or both, the number of rigging changes that could be required are reduced.

(b) Whenever possible, consult with the contractor regarding available crane capacities to determine maximum panel weight, and then the panel joint spacing. A good rule of thumb is that crane capacity should be three times the weight of the panel.

(c) The simplest rigging configurations are preferred for economy and ease of use. Generally, try to limit panel widths so that a two-wide rigging can be used, as this rigging offers the simplest approach. A good rule of thumb is 20 to 22 ft (6 to 6.7 m) maximum width for a two-wide arrangement. Also, two-high lifting arrangements are good for panels up to approximately 35 to 40 ft (10.7 to 12.2 m) in height. Panel weight for a two-high by two-wide rigging is limited to approximately 30 tons (27 metric tons), which may not be appropriate for all buildings. Panel thickness can also be a limiting factor on the selected rigging configuration.

(d) Usually, panel height is a project requirement over which the designer has little control. For multi-story projects, there is an option to stack panels to control panel heights and weights. Most tall panels can be handled with a four-high lifting arrangement that has a practical limit on panel height of 75 ft (23 m). Three-high rigging arrangements are cumbersome and should be avoided because they require one long, continuous cable to each set of three lift points. These cables are often 100 to 120 ft (30 to 36 m) long, and easily tangle.

(e) If possible, avoid stepping or sloping the bottom of panels; the length of a step notch in the bottom of a panel should be minimized and typically not exceed 30 percent of the panel width. Likewise, if at all possible, avoid shortening one leg at the bottom of a panel to minimize the difficulty involved in erecting the panel.

(f) Consider splitting a panel with a large opening into two panels, or creating a spandrel panel to avoid overly wide panels.

(g) Use of double mats of reinforcement could have advantages in some cases, and can reduce the need for added lifting steel, but should be balanced with other construction requirements and cost.
(h) A minimum concrete strength of 2500 psi (17 MPa) is generally required for lifting to meet manufacturer’s requirements for full insert capacity. Due to project schedules, it may be necessary to use higher-strength concrete to achieve early lifting strength, which the designer can take advantage of in designing the panel reinforcement for in-place loading, or high early-strength concrete mixture designs.

(i) In specifying the concrete mixture design, the designer should give consideration to concrete modulus of rupture. Concrete flexural strength is important when lifting panels, particularly when panels have center reinforcement only. Characteristics of the concrete mixture, such as coarse aggregate particle shape and surface texture, and the ratio of coarse to fine aggregate, can greatly affect concrete tensile strength. River rock or pea stone aggregates will result in a lower modulus of rupture, and should be avoided. Experience has shown that concrete flexural strengths of 400 to 500 psi (2.8 to 3.4 MPa) are usually sufficient to prevent panel cracking where computed bending stresses are less than 250 psi (1.7 MPa), although panels have lifted successfully at higher bending stresses as much as 350 psi (2.4 MPa).

(j) The production of reinforcing bar and lifting drawings are two separate operations and because of this, lifting reinforcement can easily be overlooked. For this reason, consider adding a note in the construction documents stating that the contractor is responsible for providing the additional steel reinforcement needed for lifting to the in-place steel on the reinforcing bar shop drawings.

11.4—Lifting design considerations: panel specialty engineer

As the lift insert designer, there are a two basic lifting considerations when designing panels:

1) When possible, avoid lifting panels off of pilasters, corbels, or recesses or the cable lengths will require adjustments for the panels to hang straight.

2) Locate the geometric center of the lift inserts at least 18 in. (457 mm) above the center of gravity of the panel. The higher above the center of gravity of the inserts, the straighter the panel will hang, making the panel quicker and easier to lift. Maximize the distance above as the panel stresses allow.

11.4.1 Rigging and lifting inserts

11.4.1.1 Insert capacity—Consult manufacturer’s product literature for insert capacities based on panel thickness. Pay attention to minimum distances to panel edges or openings and concrete strength requirements, and consider impact loading. Minimum edge distance for full insert capacity is normally in the 15 to 18 in. (381 to 457 mm) range. Insert capacity is also reduced when using lightweight concrete. Inserts normally have a safety factor of 2.0 against failure.

While insert spacing is usually not an issue with face inserts, it is important for edge lift inserts. If edge lift inserts are used for lifting a panel from the horizontal position, shear bars are often added to keep the inserts from pulling sideways out of the panel.

11.4.1.2 Insert clearances—Provide clearance for all inserts from panel projections that could interfere with the operation of the lifting hardware, per the manufacturer.

11.4.1.3 Adjusting insert horizontal alignment—Adjusting the horizontal alignment of a pair of inserts is sometimes necessary to avoid panel openings while maintaining a balanced lift. The horizontal alignment of insert pairs on a two- or four-high lift can be offset by as much as 4 ft (1.2 m) (for example, horizontal distance between the top and bottom insert anchoring the same cable to the panel), as long as the centroid of all inserts is balanced about the horizontal center of gravity of the panel. The easiest way to adjust horizontal locations is to move each insert in a pair by the same amount in opposite directions. The permissible offset is also a function of the length of the cable used, with the shorter cable producing a smaller offset, as well as the capacity of the insert to resist a side load.

11.4.1.4 Face-lift—Tilt-up panels are typically face-lifted. Refer to the manufacturer’s literature for rigging arrangements. Avoid three-wide riggings because of complications with the cable geometry. The simplest rigging configurations are the best. Minimize the use of different riggings on a project. It is costly and time-consuming to make rigging changes. It is less expensive to pay for a few extra inserts than to constantly change rigging configurations.

11.4.1.5 Edge-lift—Edge lift spandrel and screen wall panels whenever possible. This allows them to hang perfectly plumb when setting. If a panel is too tall for edge lifting and must hang plumb, furnish face-lift inserts for initial lifting, switching to edge-lift inserts for panel placement or the cable clamp systems offered by most manufacturers.

11.4.2 Concrete strength/design of reinforcement—As stated previously, preventing the panel from cracking at all is ideal. Allowable concrete flexural stress used by most engineers and vendors for normalweight concrete is 6’f’’, based on specifying concrete design mixtures that achieve ultimate concrete tensile strengths (moduli of rupture) between 9’f’’ and 11’f’’. This yields 300 psi (2.1 MPa) allowable for 2500 psi (17 MPa) concrete at time of lift. Allowable stress is reduced when using lightweight concrete.

Some vendors will follow other concrete stress limits, even in the presence of reinforcement. These rules are based primarily on their desire to safely control cracking to limit liability. These rules can lead to conservative results and, therefore, are mentioned herein only as a point of reference for control of cracking in critical panels. Examples include:

1) Allowable bending stress with reinforcement only at the tension face is (0.75)(0.45’‘) = 0.3375’‘.

2) Allowable bending stress with reinforcement on both faces is 0.45’‘.

3) Using strongbacks if bending stress is above 0.45’‘.

When it is impractical to prevent cracking, reinforcement is generally used and is added only where required in the tension face, usually 1 in. (25 mm) clear from the face of the panel, making sure the required reinforcement does not exceed that allowed by ACI 318. Reinforcement should extend beyond the point where it is no longer needed for a distance equal to the effective depth of the member or 12 bar...
diameters, which is greater. Reinforcement should be calculated using ultimate strength methods, with a minimum load factor of 1.5 applied to the panel weight to account for the impact effects discussed previously. Reinforcing cages already in the panel that is required for in-place loading should be used for additional reinforcement as applicable. Center mat reinforcement should not be used for lifting design. So, when lifting stresses in a panel or portion of a panel exceed allowable concrete tension stresses, always provide reinforcement at the tension face, which is preferred, or use strongbacks.

There is a reason for not using the center mat for lifting; although the panel might not fail, once cracking begins due to lifting stresses, they will likely occur over a significant portion of the panel. These cracks would be wide because of the distance from the tension face to the reinforcing steel. Horizontal cracks will close up once the panel is resting on its base, but will still be visible; the panel could be rejected by the owner.

11.4.3 Strongbacks—Although strongbacks are used to prevent cracking, they can also be used as an alternative to adding reinforcing steel. Usually it is more economical to add reinforcement.

A strongback should be stiffer than the panel section it carries or the panel will crack before the strongback takes the load. Tilt-up vendors rent strongbacks, or they can be designed using wood timbers or steel or aluminum channels. Strongbacks are usually fastened to the panel using coil anchors and bolts. These connections should be capable of transferring the weight of the panel section to the strongback plus horizontal shear due to bending.

Strongbacks are usually reusable, so they should be strong enough to withstand repeated use. Strongbacks can also be used to stabilize a panel during erection. One example is an L-shaped or one-legged panel. Strongbacks that are used for rotation at the panel base should have a shoe that extends to the downside of the panel and that is the same thickness of the section of panel where the strongback is connected. This shoe can be wood blocking.

CHAPTER 12—TEMPORARY PANEL BRACING
The brace supplier usually provides design requirements for temporary bracing of tilt-up panels. Bracing should comply with the guidelines published by the Tilt-Up Concrete Association (2012). The following short topics related to temporary panel bracing are provided for information purposes to explain the basic design process, and to know what to expect from the bracing designer.

12.1—Brace geometry and number of braces
Typical tilt-up bracing geometry is one row of braces located at two-thirds of the panel height above the attachment point—slab, footing, or deadman. The angle of the brace is typically based on a 3-4-5 triangle; base of 3, height of 4, and brace length of 5, proportionally. This puts the brace at approximately a 53-degree angle. The use of fixed length braces in particular can result in variations to this exact geometry, however, in general, a brace should be no less than 45 degrees and no more than 60 degrees from the horizontal. In addition, brace inserts should not be placed lower than 60 percent of the panel’s height nor less than 5 percent of the panel’s height above the panel’s geometric centroid or mass center of gravity, whichever is greater.

A minimum of two braces per panel is required, but three or four braces per panel is more common. The number of braces is based on allowable load per brace, from manufacturer’s bracing tables, and the anchor capacity into either the panel or base attachment point. Panel braces should be balanced laterally about the panels’ geometric center, and be arranged such that all braces take an equal tributary load, or at least no brace exceeds the allowable brace load. The simplest approximate arrangement is to go a half-space in from the panel edges for laterally symmetric panels.

Typically, temporary brace inserts are offset 1 ft (0.3 m) horizontally from lines of lifting inserts to avoid conflict between rigging and bracing because braces are normally attached to the panel prior to lifting.

12.2—Knee and lateral bracing
Long braces often require intermediate lateral bracing. This can be in the form of knee and lateral bracing. This type of bracing is costly and should be avoided. When knee braces are used, continuous cross-lacing is required because the brace can buckle in any direction. Knee and lateral braces should be positively connected and lateral braces tied off to something providing lateral resistance, usually the ground, at each end. It is preferable to select a brace that requires no bracing, or has its own built-in lateral bracing.

12.3—Bracing to slab-on-ground
Bracing hardware manufacturers suggest a minimum slab thickness of 5 to 6 in. (125 to 150 mm) for bracing of tilt-up to the slab-on-ground, but recommend the actual requirement for each project be checked by an engineer.

In general, a 6 in. (150 mm) slab should be acceptable for most situations, as should a 5 in. (125 mm) slab, as long as the panels are not too tall. The problem arises when bracing panels to a 4 in. (100 mm) slab, such as in a tilt-up office building, particularly when it is multistory. In this case, it is common to thicken the slab at the location of the brace, which creates an integral deadman. This thickened slab area can result in cracking in the slab due to restricted slab movement. In the design of floor slabs for bracing loads, the location of floor joints should also be considered.

12.4—Deadmen
Similar to slab-on-ground design for bracing loads, design of deadmen is not usually provided by the temporary bracing designer. Deadmen are used when there is no building slab to brace to, or when the panels need to be braced to the outside of the building. Consider that footings can be used as deadmen in certain arrangements where there are many orthogonal walls that are close together.

Helical anchors have been used in place of deadmen in certain applications. Coordination with the tilt-up contractor is key to selecting the right solution.
12.5—Base sliding
Under some circumstances, it is possible that the base of the panel could slide under wind loading. This would be the case if panel braces were installed too high on the panel, or if the panel base was not yet grouted. Base sliding should be checked if there is a concern; checking a few typical cases should give a good indication if there is a need to investigate further.

12.6—Alternate bracing methods
When tilt-up panels are in close proximity to one another, such as in an elevator shaft or stairwell, it is impractical and sometimes physically impossible to brace them to the ground. Bracing panels laterally to other parallel walls, and then bracing the parallel wall to the ground on the opposite side is an option. The bracing design for the panel that gets braced to the ground should consider the load from the other panel.

In the corner of buildings, it is often impractical to brace both corner panels inward because the braces tend to interfere with one another. Consider eliminating the brace closest to the corner on one panel by tying the panels together at the approximate elevation of the braces with a temporary bolted clip angle. Another technique is to lower one of the braces 1 to 2 ft (0.3 to 0.6 m) to avoid interference with the intersecting brace.

CHAPTER 13—REFERENCES
ACI committee documents and documents published by other organizations are listed first by document number and year of publication followed by authored documents listed alphabetically.

American Concrete Institute
ACI 117-10—Specification for Tolerances for Concrete Construction and Materials and Commentary
ACI 318-11—Building Code Requirements for Structural Concrete and Commentary
ACI 355.2-07—Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary
ACI 355.4-11—Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary
ACI 551.1R-14—Guide to Tilt-Up Concrete Construction
ACI ITG-6R-10—Design Guide for the Use of ASTM A1035/A1035M Grade 100 (690) Steel Bars for Structural Concrete

American Institute of Steel Construction
AISC 360-10—Specification for Structural Steel Buildings

American Society of Civil Engineers
ASCE/SEI 07-10—Minimum Design Loads for Buildings and Other Structures

ASTM International
ASTM A1035/A1035M-14—Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement

APPENDIX A—DERIVATION OF Mn AND Icr
A.1—Derivation of Mn and Ic, based on rectangular stress block
For one layer of reinforcement and no axial load, the forces depicted in Fig. A.1 can be calculated as follows

\[ C = 0.85 f'c' a b \]
\[ T = A f_y \]
\[ a = \frac{A f_y}{0.85 f'_c} \]
\[ c = \frac{s}{\beta_i} \]
\[ \beta_i = 0.85 \text{ for } f'_c \leq 4000 \text{ psi} \]
\[ 0.85 - 0.00005(f'_c - 4000) \geq 0.65, \text{ for } f'_c > 4000 \text{ psi} \]
\[ M_n = A f_y (d - a/2) \]

\[ EI_c = E_s A_s (d - c)^2 + E_s a b \frac{12}{12} + E_s a b c \left( -d + a \right)^2 \]
\[ = E_s \left[ n A_s (d - c)^2 + bc \frac{\beta_i}{12} + bc^2 \left( 1 - \frac{\beta_i}{2} \right) \right] \]

where \( n = E_s / E_c \)
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

Fig. A.1—Diagram of rectangular stress distribution.

For $f'_c < 4000$ psi, $b_1 = 0.85$

\[ EI' = E_c \left[ nA_s (d - c)^2 + 0.051bc^3 + 0.281bc^3 \right] \]

\[ I' = nA_s (d - c)^2 + 0.332bc^3 \]

which is approximately $I' = nA_s (d - c)^2 + bc^3/3$.

Example:

$h = 5.5$ in. (140 mm); $d = 2.75$ in. (70 mm); $b = 12$ in. (305 mm)

$f'_c = 4000$ psi (28 MPa); $b_1 = 0.85$; $A_s = 0.31$ in.\(^2\) (200 mm\(^2\))

$E_c = 60,000$ psi (420 MPa)

\[ n = \frac{29,000}{3605} = 8.04 \]

\[ a = \frac{0.31(60,000)}{0.85(4000)}(12) = 0.456 \text{ in. (12 mm)} \]

\[ c = \frac{0.456}{0.85} = 0.536 \text{ in. (14 mm)} \]

\[ M_s = \frac{0.31(60,000)}{12} (2.75 - 0.456/2) = 3910 \text{ ft-lb (57 kN-m)} \]

\[ k_s = 8.04 \times 0.31(2.75 - 0.536)^2 + 0.281 \times 12 \times 0.536^3 \]

\[ 12.2 + 0.52 = 12.7 \text{ in.}^4 (5.3 \times 10^6 \text{ mm}^4) \]

\[ E_c I' = 3.61 \times 10^6 \times 12.7 = 45.9 \times 10^6 \text{ lb-in.}^2 (132 \times 10^6 \text{ kN-mm}^2) \]

A.2—Derivation of $M_s$ and $I_s'$ based on triangular stress distribution

For one layer of reinforcement and no axial load, the forces depicted in Fig. A.2 can be calculated as follows

\[ C = \frac{f_c b k_s}{2} = T = A_s f_s, \text{ where } k_s = \frac{A_s F_s}{f_c (b/2)} \]

concrete strain $= \varepsilon_c = 0.0005$, maximum for elastic range

concrete stress $= f_c = E_c \varepsilon_c$

steel strain $= \varepsilon_s = \frac{(d - k_s)}{k_s}$

steel stress $= f_s = E_s \varepsilon_s (d - k_s)$

Example:

$f'_c = 4000$ psi (28 MPa); $A_s = 0.31$ in.\(^2\) (200 mm\(^2\)); $f_c = 60,000$ psi (420 MPa)

\[ E_c = 57,000 \sqrt{4000} = 3.61 \times 10^6 \text{ psi (24,900 MPa)} \]

\[ n = \frac{29,000}{3605} = 8.04 \]

\[ nA_s = 8.04 \times 0.31 = 2.49 \text{ in.}^2 (1600 \text{ mm}^2) \]

\[ M_s = \frac{0.31(60,000)}{12} \left(2.75 - \frac{0.881}{2.75} \right) = 3810 \text{ ft-lb (56 kN-m)} \]

\[ k_s = -2.49 + \sqrt{(2.75)^2 + 22.49(12)(2.75)} = 0.881 \text{ in. (22.4 mm)} \]

\[ M_s = \frac{0.31(60,000)}{12} \left(2.75 - \frac{0.881}{3} \right) = 3810 \text{ ft-lb (56 kN-m)} \]

\[ I' = 2.49(2.75 - 0.881)^3 + \frac{12(0.881)^3}{3} = 8.71 + 2.75 = 11.4 \text{ in.}^4 (4.75 \times 10^6 \text{ mm}^4) \]

\[ E_c I' = 3.61 \times 10^6 \times 11.4 = 41.3 \times 10^6 \text{ lb-in.}^2 (119 \times 10^6 \text{ kN-mm}^2) \]

APPENDIX B—DESIGN EXAMPLES FOR OUT-OF-PLANE FORCES

The following examples illustrate the use of the procedure outlined in this guide for the analysis of vertical reinforce-
ment in tilt-up panels using the moment magnifier method as presented in ACI 318-11, 14.8. This method is not an iterative procedure. Instead, the method should be considered a trial-and-error technique for calculating panel moment strength based on an assumed area of tension reinforcement. This assumed area of tension reinforcement should be carefully selected based on the following considerations.

The design moment strength of the panel, \( \phi M_{dr} \), is directly proportional to the area of effective tension reinforcement, \( A_{se} \), by the equation

\[
\phi M_{dr} = \phi A_{se} f_t \left( d - \frac{a}{2} \right)
\]

Because the panel stiffness \( K_b \) is a function of the cracked moment of inertia, \( I_{cr} \), which in turn is a function of \( A_{se} \), the factored moment on the panel, \( M_{fr} \), is inversely proportional to \( A_{se} \), as demonstrated by the following.

This equation defines a hyperbolic curve for the factored moment bounded by the line where

\[
M_a = \frac{M_{ns}}{1 - \frac{P_{ax}}{0.75K_b}}
\]

The reduced panel stiffness is equal to the factored axial load. As Fig. Ba illustrates, a small starting value of \( A_{se} \) can lead to a negative value for the factored moment, with incrementally larger values for \( A_{se} \) yielding even larger negative values for \( M_a \). The panel designer should select an area of tension reinforcement that provides a reduced panel stiffness larger than the factored axial load, thereby making the denominator in the previous equation a positive number.

For a given set of design parameters, the minimum amount of tension reinforcing steel required in the tilt-up panel is the point in Fig. Ba where the \( M_a \) and \( \phi M_{dr} \) curves intersect. Values of \( A_{se} \) to the right of this point will provide excess strength capacity in comparison to the factored moment. The area of tension reinforcement, however, should also be evaluated for other limiting criteria prescribed in ACI 318.

Example calculations are provided in this appendix for tilt-up panels with the following characteristics, including a panel with:

a) No openings
b) A 10 x 15 ft (3 x 4.6 m) door opening offset from the panel centerline
c) A concentrated axial load
d) A concentrated lateral load
e) Multiple continuous spans (multi-story)
f) A dock-high condition
g) A fixed end
h) An isolated footing or pier foundation support
i) Stiffening pilasters and headers

For each example given, two cases of tension reinforcement are investigated within the 6.25 in. (150 mm) wythe: one with reinforcement centered in the panel thickness (singly reinforced), and the other with reinforcement at each face (doubly reinforced), as depicted in Fig. Bb. For the single layer of reinforcement shown, the distance from extreme compression fiber to the centroid of the tension reinforcement (the \( d \) distance) for a No. 6 (19 mm) bar is the chair height (2.25 in. [57 mm]) plus the diameter of a No. 4 (13 mm) bar plus one-half of the No. 6 (10 mm) bar diameter, yielding a \( d \) distance of 3.13 in. (80 mm). Similarly, for the double layer of reinforcement scheme, the \( d \) distance is 5 in. (127 mm).

The concrete and reinforcing steel properties listed in the following are constant in each example: \( f'c = 4000 \) psi (28 MPa); \( f_y = 60,000 \) psi (420 MPa); \( f'_c = 29,000 \) ksi (200 MPa); \( f'_t = 7.5k \sqrt{f'_c} = 474 \) psi (3.3 MPa); \( E_c = 29,000 \) ksi (200 MPa); \( E_c = 57 \sqrt{f'_c} = 3605 \) ksi (24.0 MPa) (normalweight concrete); and \( E/E_c = 8.044 \).

Furthermore, each panel is investigated using three of the load cases specified by ACI 318-11, 9.2.1:

Load Case 1: \( 1.2D + 1.6L_r + 0.5W \) (9-3)
Load Case 2: \( 1.2D + 0.5L_r + 1.0L + 1.0W \) (9-4)
Load Case 3: 0.9D + 1.0W (9-6)

where the strength reduction factor from ACI 318-11, 9.3.2.1 is \( \phi = 0.9 \) for tension-controlled sections.

**B.1—Panel with no openings design example**

Figure B.1 illustrates the geometry of the sample panel with no openings. The panel supports the load from three roof joists bearing in wall pockets (eccentric axial load) in addition to the wind (lateral force). A summary of the applied loading is:

- \( P_{col} = 3 \times 2.4 \text{ kip} = 7.2 \text{ kip} \)
- \( P_{cL} = 3 \times 2.5 \text{ kip} = 7.5 \text{ kip} \)
- Eccentricity, \( e_e = 3 \text{ in} \) (assumed)
- \( w = 27.2 \text{ lb/ft}^2 \)
- \( l_c = \frac{31.0 \text{ ft}}{1.5 \text{ ft}} = 20.5 \text{ ft} \)

The weight of the tilt-up panel above the design section (centerline of the unbraced length) is:

\[
\frac{6.25 \text{ in.}}{12 \text{ in./ft}} \times 150 \text{ lb/ft}^2 \times \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \times \frac{1 \text{ kip}}{1000 \text{ lb}} = 19.0 \text{ kip}
\]

**B.1.1 Reinforcing steel centered in panel thickness—**

Assume 16 No. 6 bars (A_s = 12.0 in.^2).

**B.1.1.1 Load Case 1: 1.2D + 1.6L_e + 0.5W**

- \( P_{sar} = 1.2 \times 7.2 \text{ kip} + 1.6 \times 7.5 \text{ kip} = 20.6 \text{ kip} \)
- \( P_{sar} = 20.6 \text{ kip} + 1.2 \times 19.0 \text{ kip} = 43.4 \text{ kip} \)
- \( w_s = 0.5 \times (15.0 \text{ ft}) \times 27.2 \text{ lb/ft}^2 = 204 \text{ pff} = 0.204 \text{ kif} \)

Check vertical stress at the midheight section of panel per ACI 318-11, 14.8.2.6:

\[
P_{sar} = \frac{43.4 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft})} = 38.6 \text{ psi} < 0.06f'_c = 260 \text{ psi}
\]

Check the design moment strength:

\[
A_s = A + \frac{P_{sar} \left(\frac{h}{2d}\right)}{f_y} = \frac{6.25 \text{ in.}^2}{60 \text{ ksi}} \left(\frac{6.25 \text{ in.}}{2(3.13 \text{ in.})}\right) = 7.72 \text{ in.}^2
\]

\[
a = \frac{A_s f'_c}{0.85 f_y} = \frac{7.72 \text{ in.}^2(60 \text{ ksi})}{0.85(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.757 \text{ in.}
\]

\[
c = \frac{\alpha}{0.85} = \frac{0.757}{0.85} = 0.891 \text{ in.}
\]

where: tension-controlled (refer to the commentary to ACI 318, 9.3.2.2)

\[
f_y = \frac{E}{A_s (d-c)^2 + \frac{\xi_c}{3}} = \frac{E}{A_s (d-c)^2 + \frac{\xi_c}{3}} = 8.04 \times 7.72(3.13 - 0.891)^2 + \frac{(15.0 \text{ ft})(12 \text{ in./ft})(0.891)^2}{3} = 353 \text{ in.}^4
\]

\[
K_s = \frac{48E f_y}{5\xi_c^2} = \frac{48(3605 \text{ ksi})(353 \text{ in.}^4)}{5(29.5 \text{ ft})(12 \text{ in./ft})^2} = 97.4 \text{ kip}
\]

\[
M_{sar} = \frac{w_s f'_c}{8} + \frac{P_{sar} e_{xc}}{2} = \frac{0.204 \text{ klf}(29.5 \text{ ft})^2}{8} + \frac{20.6 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 24.8 \text{ ft-kip}
\]
B.1.1.2 Load Case 2: $1.2D + 0.5L_e + 1.0W$

- $P_{sw} = 12.4$ kip
- $P_{sw} = 35.2$ kip
- $w_u = 0.408$ klf
- $P_{sw}/A_g = 21.0$ psi $< 0.06f'_c = 240$ psi
- $A_{se} = 7.39$ in.$^2$
- $a = 0.725$ in.
- $c = 0.853$ in.
- $c/d = 0.273$ : tension-controlled

- $I_r = 344$ in.$^4$
- $M_{cr} = 46.3$ ft-kip
- $\phi M_a > M_{cr}$
- $K_0 = 95.1$ kip
- $M_{sa} = 45.9$ ft-kip
- $M_a = 89.5$ ft-kip $< \phi M_u$
- $\Delta_a = 14.8$ in.

B.1.1.3 Load Case 3: $0.9D + 1.0W$

- $P_{sw} = 6.48$ kip
- $P_{sw} = 23.6$ kip
- $w_u = 0.408$ klf
- $P_{sw}/A_g = 21.0$ psi $< 0.06f'_c = 240$ psi
- $A_{se} = 7.39$ in.$^2$
- $a = 0.725$ in.
- $c = 0.853$ in.
- $c/d = 0.273$ : tension-controlled

- $I_r = 344$ in.$^4$
- $M_{cr} = 46.3$ ft-kip
- $\phi M_a > M_{cr}$
- $K_0 = 95.1$ kip
- $M_{sa} = 45.2$ ft-kip
- $M_a = 67.5$ ft-kip $< \phi M_u$
- $\Delta_a = 11.4$ in.

Check service load deflection per ACI 318-11, R14.8.4 with $1.0D + 0.5L_e + W_{sw}$ and noting $L = 0$ because only roof load is applied to the panel.

- $\Delta_{allowable} = \frac{L}{150} = 2.36$ in.
- $\Delta_{cr} = \frac{5M_{cr}E^2}{48E_I} = 0.550$ in.
- $M_{sw} = \frac{w_0\ell^2 + P_e w_{sw}}{8} = \frac{0.71(17 \text{ lb/ft}^2)(15.0 \text{ ft})(29.5 \text{ ft})^2}{2(12 \text{ in./ft})} = 20.3$ ft-kip

Using this value as the initial service load moment, the initial deflection is

$$\Delta = \frac{M_{sw}}{0.75K_b} = \frac{24.8 \text{ ft-kip}}{0.75(97.4 \text{ kip})} = 61.2 \text{ ft-kip} < \phi M_a$$

After iteration, maximum service load moment is 20.8 ft-kip, leading to the iterated service load deflection of

$$\Delta = \frac{M_{sw}}{M_{cr}} \Delta_{cr} = \frac{20.8 \text{ ft-kip}}{46.3 \text{ ft-kip}} \Delta_{cr}$$

which is significantly less than the value allowed by ACI 318, so adjustment to the panel stiffness is not needed.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:

- $A_t = 0.002A_g = 0.002(6.25 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 4.60 \text{ in.}^2$

Therefore, 23 No. 4 bars should be used for the horizontal reinforcement. Figure B.1.1.3 details panel reinforcement for this case.

B.1.2 Reinforcing steel at each face of the panel—Assume 15 No. 4 bars per face ($A_t = 3.00 \text{ in.}^2$ per face) to meet minimum reinforcing steel requirements. Because of the larger distance $d$ for the doubly reinforced scheme, a smaller area of steel would be adequate to provide necessary strength, but not sufficiently small enough to benefit from the minimum steel exemption of ACI 318-11, 10.5.3 (which allows the use of four-thirds of the steel area required by analysis instead of meeting the specified minimum reinforcement ratio).
B.1.2.1 Load Case 1: 1.2D + 1.6L + 0.5W

\[ P_{u1} = 1.2(7.2 \text{ kip}) + 1.6(7.5 \text{ kip}) = 20.6 \text{ kip} \]
\[ w_{u1} = 0.5(15.0 \text{ ft})(27.2 \text{ lb/ft}^2) = 204 \text{ pJf} = 0.204 \text{ klf} \]

\[ \frac{P_{u1}}{A_g} = 43.4 \text{ kip}(1000 \text{ lb/kip}) = 38.6 \text{ psi} \leq 0.06f' = 240 \text{ psi} \]

\[ a = 0.338 \text{ in.} \]
\[ c = 0.388 \text{ in.} \]
\[ d = 0.078 \leq \text{ tension-controlled} \]

\[ I_{cr} = 579 \text{ in.}^4 \]
\[ M_{ur} = 46.3 \text{ ft-kip} \]
\[ M_{cr} > M_{ur} \]
\[ \rho = \frac{A_g}{bh} = \frac{3.0 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00267 > \rho_c = 0.0012 \]

\[ K_b = \frac{48 \times 15.0}{5f_c^2} = \frac{48(3605)(592)}{5(29.5 \text{ ft}(12 \text{ in./ft}))^2} = 163 \text{ kip} \]
\[ M_{uw} = \frac{w_{u1}h^2}{8} + \frac{P_{u1}c_{x1}}{2} = \frac{0.204 \text{ klf}(29.5 \text{ ft})^2}{8} + \frac{20.6 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 24.8 \text{ ft-kip} \]

\[ M_{ue} = \frac{M_{uw}}{1 - \frac{P_{u1}}{0.75K_b}} = \frac{24.8 \text{ ft-kip}}{1 - \frac{43.4 \text{ kip}}{0.75(163 \text{ kip})}} = 38.4 \text{ ft-kip} < \phi M_e \]

\[ \Delta_e = \frac{M_{ue}}{0.75K_b} = \frac{38.4 \text{ ft-kip}(12 \text{ in./ft})}{0.75(163 \text{ kip})} = 3.76 \text{ in.} \]

B.1.2.2 Load Case 2: 1.2D + 0.5L + 1.0W

\[ P_{u2} = 12.4 \text{ kip} \]
\[ w_{u2} = 0.408 \text{ klf} \]

\[ \frac{P_{u2}}{A_g} = 31.3 \text{ psi} < 0.06f' = 240 \text{ psi} \]
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"-TRIM BARS AS REO’D BY ANALYSIS (TYP.

Fig. B.1.2.3—Double-layer panel reinforcement.

$P_{DL} = 3 \times 10.7 \text{kN} = 32.1 \text{kN}$

$P_{LL} = 3 \times 11.1 \text{kN} = 33.3 \text{kN}$

$e_{ce} = 75 \text{mm} \text{(assumed)}$

$w = 1304 \text{Pa}$

$E_S = 28 \text{MPa}$

$E_c = 24,900 \text{MPa}$

$E_r = 200,000 \text{MPa}$

The weight of the tilt-up panel above the design section (centerline of the unbraced length) is:

$\text{Weight} = 160 \text{mm} \times (2400 \text{kg/m}^3) \times (4600 \text{mm} - 9050 \text{mm} + 450 \text{mm})$

$= 86.4 \text{kN}$

**B.1.1M Reinforcing steel centered in panel thickness**—

Assume 16 No. 20M bars ($A_s = 4800 \text{mm}^2$)

**B.1.1.1M Load Case 1: 1.2D + 1.6L + 0.5W**

$P_{uw} = 1.2(32.1 \text{kN}) + 1.6(33.3 \text{kN}) = 91.8 \text{kN}$

$P_{w} = 91.8 \text{kN} + 1.2(86.4 \text{kN}) = 196 \text{kN}$

$w_{u} = 0.5(4600 \text{mm})(1304 \text{Pa}) = 3.00 \text{N/mm}$

Check vertical stress at the midheight section of panel per ACI 318-11, 14.8.2.6:

$P_{w} = 196 \text{kN} \times 1000 \text{N/kN}$

$A_s = 160 \text{mm}(4600 \text{mm})$

$= 0.267 \text{MPa} \text{< 0.06} f_{cy} = 1.68 \text{MPa}$

Check the design moment strength:

$A_{as} = A_s + \frac{P_{w} \times h}{f_y \times 2d} = 4800 \text{mm}^2 + \frac{196 \text{kN}}{400 \text{MPa}} \times \frac{160 \text{mm}}{2(79.4 \text{mm})}$

$= 5290 \text{mm}^2$

Fig. B.1M—Plain tilt-up panel.

$a = \frac{A_s f_y}{0.85 f_y' b} = \frac{5290 \text{mm}^2 (400 \text{MPa})}{0.85(28 \text{MPa})(4600 \text{mm})} = 19.3 \text{mm}$

$c = \frac{E_c}{E_r} = \frac{19.3 \text{mm}}{22.7 \text{mm}} = 0.85$

$c = \frac{22.7 \text{mm}}{79.4 \text{mm}} = 0.286 < 0.375$

$I_v = \frac{E_c}{E_r} A_s(d - c)^2 + \frac{f_{cy} c^3}{3}$

$= 8.044(5290)(79.4 - 22.7)^2 + \frac{4600 \text{mm}(22.7)^3}{3} = 155 \times 10^6 \text{mm}^4$

$M_{ee} = f_y f_y' \left[ d - \frac{a}{2} \right]$,

$= 0.9(5290 \text{mm}^2)(400 \text{MPa}) \left[ \frac{19.3 \text{mm}}{2} \right] = 133 \text{kN-m}$

Check minimum reinforcement required by ACI 318-11, 14.8.2.4:

$\phi M_{re} \geq M_{ee}$

Check minimum reinforcement required by ACI 318-11, 14.3.2:
As $4800 \text{ mm}^2$

$p_e = \frac{w_{,e}^2}{8} + \frac{P_{,e}e_e}{2} = \frac{3.00 \text{ N/mm} \times (9050 \text{ mm})^2}{8} + \frac{91.8 \text{ kN} \times (75 \text{ mm})}{2} = 34.2 \text{ kN-m}$

$M_u = M_{sa} = \frac{0.75K_b}{0.75(452 \text{ kN})} \times 8.11 \text{ kN-m} \times 239 \text{ mm}$

B.1.1.2M Load Case 2: $1.2D + 0.5L + 1.0W$

$P_{sa} = 55.2 \text{ kN}$

$P_{um} = 159 \text{ kN}$

$w_u = 6.00 \text{ N/mm}$

$P_{um}/A_g = 0.216 \text{ MPa} < 0.06f_c' = 1.68 \text{ MPa}$

$A_{re} = 5200 \text{ mm}^2$

$a = 19.0 \text{ mm}$

$c = 22.4 \text{ mm}$

$c/d = 0.282$ tension-controlled

$I_{re} = 153 \times 10^6 \text{ mm}^4$

$M_{cr} = 64.2 \text{ kN-m}$

$\phi M_{sa} = 131 \text{ kN-m} > M_{cr}$

$K_b = 447 \text{ kN}$

$M_{sa} = 63.5 \text{ kN-m}$

$M_u = 121 \text{ kN-m} < \phi M_u$

$\Delta_u = 361 \text{ mm}$

B.1.1.3M Load Case 3: $0.9D + 1.0W$

$P_{sa} = 28.9 \text{ kN}$

$P_{um} = 107 \text{ kN}$

$w_u = 6.00 \text{ N/mm}$

$P_{um}/A_g = 0.145 \text{ MPa} < 0.06f_c' = 1.68 \text{ MPa}$

$A_{re} = 5070 \text{ mm}^2$

$a = 18.5 \text{ mm}$

$c = 21.8 \text{ mm}$

$c/d = 0.275$ tension-controlled

$I_{re} = 151 \times 10^6 \text{ mm}^4$

$M_{cr} = 64.2 \text{ kN-m}$

$\phi M_u = 128 \text{ kN-m} > M_{cr}$

$K_b = 447 \text{ kN}$

$M_{sa} = 62.5 \text{ kN-m}$

$M_u = 92.4 \text{ kN-m} < \phi M_u$

$\Delta_u = 279 \text{ mm}$

Check service load deflection per ACI 318M-11, R14.8.4, with $1.0D + 0.5L + W_o$, and noting $L = 0$ because only roof load is applied to the panel:

$A_r = 0.002A_g = 0.002(160 \text{ mm})(9500 \text{ mm}) = 3040 \text{ mm}^2$
Therefore, 31 No. 10M bars should be used for the horizontal reinforcement. Figure B.1.1.3M details panel reinforcement for this case.

**B.1.2M Reinforcing steel at each face of the panel—**
Assume 20 No. 10M bars per face ($A_s = 2000 \text{ mm}^2$ per face) to meet minimum reinforcing steel requirements. Because of the larger $d$ distance for the doubly reinforced scheme, a smaller area of steel would be adequate to provide necessary strength, but not sufficiently small enough to benefit from the minimum steel exemption of ACI 318M-11, 10.5.3 (which allows the use of four-thirds of the steel area required by analysis instead of meeting the specified minimum reinforcement ratio).

**B.1.2.1M Load Case 1: $1.2D + 1.6L_r + 0.5W$**

- $P_{ua} = 1.2(32.1 \text{ kN}) + 1.6(33.3 \text{ kN}) = 91.8 \text{ kN}$
- $w_u = 0.5(4600 \text{ mm})(1304 \text{ Pa}) = 3.00 \text{ N/mm}$

\[
M_u = \frac{w_u f_y^2}{8} + \frac{P_{um} e_{cc}}{2} = \frac{3.00 \text{ N/mm}(9050 \text{ mm})}{8} + \frac{91.8 \text{ kN}(75 \text{ mm})}{2} = 34.2 \text{ kN-m}
\]

\[
M_u = \left( \frac{M_{um}}{1 - \frac{P_{um}}{0.75 K_b}} \right) = \left( \frac{34.2 \text{ kN-m}}{1 - \frac{196 \text{ kN}}{0.75(747 \text{ kN})}} \right) = 52.6 \text{ kN-m} < \phi M_u
\]

\[
\Delta_u = \frac{M_u}{0.75 K_b} = \frac{52.6 \text{ kN-m}}{0.75(747 \text{ kN})} = 93.9 \text{ mm}
\]

**B.1.2.2M Load Case 2: $1.2D + 0.5L_r + 1.0W$**

- $P_{ua} = 55.2 \text{ kN}$
- $w_u = 6.00 \text{ N/mm}$

\[
M_u = \frac{64.2 \text{ kN-m}}{\phi M_u} = \frac{64.2 \text{ kN-m}}{99.5 \text{ kN-m}} > M_c
\]

\[
K_b = 733 \text{ kN}
\]

\[
M_u = 63.5 \text{ kN-m}
\]

\[
M_u = 89.3 \text{ kN-m} < \phi M_u
\]

\[
\Delta_u = 163 \text{ mm}
\]

**B.1.2.3M Load Case 3: $0.9D + 1.0W$**

- $P_{ua} = 28.9 \text{ kN}$
- $w_u = 6.00 \text{ N/mm}$

\[
M_u = \frac{64.2 \text{ kN-m}}{\phi M_u} = \frac{64.2 \text{ kN-m}}{96.1 \text{ kN-m}} > M_c
\]

\[
K_b = 709 \text{ kN}
\]

\[
M_u = 62.5 \text{ kN-m}
\]

\[
M_u = 78.2 \text{ kN-m} < \phi M_u
\]

\[
\Delta_u = 147 \text{ mm}
\]

Because maximum service moment at midheight did not exceed two-thirds of the cracking moment in the single layer analysis, there is no need to repeat the calculation for this analysis. Using the minimum horizontal reinforcing steel requirements previously determined, but noting the maximum spacing between bars is 450 mm, Fig. B.1.2.3M details panel reinforcement for the double layer scheme.

The aforementioned calculations neglected the effect of the steel reinforcement in the opposite face, as is consistent with common practice. Depending on the location of the neutral axis within the panel cross section, this additional steel reinforcement may serve as an additional amount of...
B.1.3M Summary of panel reinforcing steel—Neglecting trim bars and miscellaneous reinforcing steel, the weight of the primary horizontal and vertical reinforcing steel in the single mat option is 464 kg, compared with 451 kg for the double mat option. The designer may consider other project considerations such as ease of placement, number of repetitive panels, or the value of consistent detailing before selecting the most economical reinforcing steel configuration.

B.2—Panel with a 10 x 15 ft door opening design example

Figure B.2 illustrates the geometry of the sample panel with a door opening offset from the panel centerline. The panel supports the load from four roof joists bearing in wall pockets (eccentric axial load) in addition to the wind (lateral force). The door opening is offset from the horizontal centerline of the panel to demonstrate the effect of leg tributary width on the design calculations. A summary of the applied loading is:

\[
P_L = 4(2.4 \text{ kip}) = 9.6 \text{ kip}
\]
\[
P_L = 4(2.5 \text{ kip}) = 10.0 \text{ kip}
\]
\[
e_e = 3 \text{ in. (assumed)}
\]
\[
w = 27.2 \text{ lb/ft}^2
\]
\[
\ell_e = 31.0 \text{ ft} - 1.5 \text{ ft} = 29.5 \text{ ft}
\]

The weight of the tilt-up panel above the design section (centerline of the unbraced length) should be divided as follows:

For the left leg

\[
P_{DL} = 4(2.4 \text{ kip}) + 1.2(4.5 \text{ kip}) + 1.6(4.7 \text{ kip}) = 12.9 \text{ kip}
\]
\[
P_{LV} = 1.2(4.5 \text{ kip}) + 1.6(4.7 \text{ kip}) = 12.9 \text{ kip}
\]
\[
P_{LV} = 12.9 \text{ kip} + 1.2(11.3 \text{ kip}) = 26.5 \text{ kip}
\]
\[
\ell_e = 0.5(4.0 \text{ ft} + 5.0 \text{ ft})(27.2 \text{ lb/ft}^2) = 122 \text{ plf} = 0.122 \text{ klf}
\]

Check vertical stress at the midheight section of panel per ACI 318-11, 14.8.2.6:

\[
P_{LV} = 26.5 \text{ kip}(1000 \text{ lb/kip}) = 26.5 \text{ kip}(1000 \text{ lb/kip}) = 88.3 \text{ psi} < 0.06 f'_c = 240 \text{ psi}
\]

Check the design moment strength:
\[ A_u = A_L + \frac{P_{wx}}{f_y} \left( \frac{b}{2d} \right) = 5.28 \text{ in.}^2 + \frac{26.5 \text{ kip}}{60 \text{ ksi} (2(3.13 \text{ in.})} = 5.72 \text{ in.}^2 \]

\[ a = \frac{A_u f_y}{0.85 f_y b} = \frac{5.72 \text{ in.}^2 (60 \text{ ksi})}{0.85(4 \text{ ksi})(4 \text{ ft})(12 \text{ in./ft})} = 2.10 \text{ in.} \]

\[ c = \frac{a}{0.85} = \frac{2.10}{0.85} = 2.47 \text{ in.} \]

\[ \frac{c}{d} = 0.791 > 0.375 \text{ (refer to commentary of ACI 318-11, 9.3.2.2)} \]

Therefore, the requirement of ACI 318-11, 14.8.2.3, that the section be tension-controlled is not met. At this point, there are two options: provide a double layer of reinforcement (as demonstrated in this example), or increase wall panel thickness and repeat the analysis. To have a tension-controlled section for analysis, select a panel thickness of 8.75 in. The distance (using a 3.5 in. chair) is calculated to be 4.38 in.

Assume seven No. 6 bars, \( A_s = 3.08 \text{ in.}^2 \). The weight of the tilt-up panel above the design section is 15.9 kip.

\[ P_{wx} = 12.9 \text{ kip} + 1.2(15.9 \text{ kip}) = 32.0 \text{ kip} \]

\[ 
\begin{align*}
A_s &= \frac{32.0 \text{ kip}(1000 \text{ lb/kip})}{8.75 \text{ in.}(40 \text{ ft}/12 \text{ in./ft})} = 76.2 \text{ psi < 0.06} f' &= 240 \text{ psi} \\
\end{align*}
\]

\[ A_u = A_L + \frac{P_{wx}}{f_y} \left( \frac{b}{2d} \right) = 3.08 \text{ in.}^2 + \frac{32.0 \text{ kip}}{60 \text{ ksi} (8.75 \text{ in.} / (2(4.38 \text{ in.}))} = 3.61 \text{ in.}^2 \\
a = \frac{A_u f_y}{0.85 f_y b} = \frac{3.61 \text{ in.}^2 (60 \text{ ksi})}{0.85(4 \text{ ksi})(4 \text{ ft})(12 \text{ in./ft})} = 1.33 \text{ in.} \\
c = \frac{a}{0.85} = \frac{1.33}{0.85} = 1.56 \text{ in.} \\
\frac{c}{d} = 0.356 < 0.375 \text{ : tension-controlled} \\
\]

\[ I_o = \frac{E I_A}{E} = \frac{A_s (d - c)^2 + \frac{f' c^2}{3}}{3} = 8.044(3.61)(4.38 - 1.56)^2 + \frac{(4 \text{ ft})(12 \text{ in./ft})(1.56)^2}{3} = 292 \text{ in.}^4 \]

\[ M_{cr} = \frac{f' I_o}{y} = f' S = f' \left( \frac{1}{6} b h^2 \right) = 0.474 \text{ ksi} \left( \frac{1}{6} \right) (4 \text{ ft})(8.75 \text{ in.})^2 = 24.2 \text{ ft-kip} \]

\[ \phi M_{cr} = \phi A_s f_y \left( d - \frac{a}{2} \right) = 0.9(3.61)(60) \left( 4.38 - \frac{1.33}{2} \right) = 724 \text{ in.-kip = 60.4 ft-kip} \]

\[ \phi M_{cr} > M_{cr} \]

\[ \rho = \frac{A_s}{bh} = \frac{3.08 \text{ in.}^2}{4.0 \text{ ft}(12 \text{ in./ft})(8.75 \text{ in.})} = 0.00733 > \rho_t = 0.0015 \]

Check the applied moment per ACI 318-11, 14.8.3:

\[ K_o = \frac{48E f_y}{5f' c} = \frac{48(3605 \text{ ksi})(292 \text{ in.})^3}{5(29.5 \text{ ft}(12 \text{ in./ft})^2} = 80.6 \text{ kip} \]

\[ M_{wx} = \frac{w_o F^2}{8} + \frac{P_{wx} c_o}{2} = \frac{0.122 \text{ klf}(29.5 \text{ ft})^2}{8} + \frac{12.9 (3 \text{ in.})}{2(12 \text{ in./ft})} = 14.9 \text{ ft-kip} \]

\[ M_o = \frac{M_{wx}}{0.75 K_o} = \frac{14.9 \text{ ft-kip}}{0.75(80.6 \text{ kip})} = 22.4 \text{ ft-kip} < \phi M_{cr} \]

**B.2.1.2 Load Case 2: 1.2D + 0.5L + 1.0W**

\[ P_{wx} = 7.75 \text{ kip} \]

\[ P_{wx} = 26.8 \text{ kip} \]

\[ w_o = 0.245 \text{ klf} \]

\[ P_{wx} / A_s = 63.8 \text{ psi < 0.06} f' = 240 \text{ psi} \]

\[ A_s = 3.53 \text{ in.}^2 \]

\[ a = 1.30 \text{ in.} \]

\[ c = 1.53 \text{ in.} \]

\[ c/d = 0.349 : tension-controlled \]

\[ I_o = 288 \text{ in.}^4 \]

\[ M_o = 24.2 \text{ ft-kip} \]

\[ \phi M_o = 59.3 \text{ ft-kip} > M_{cr} \]

\[ K_o = 79.5 \text{ kip} \]

\[ M_o = 27.6 \text{ ft-kip} \]

\[ M_o = 50.1 \text{ fl-kip < } \phi M_o \]

\[ \Delta_o = 10.1 \text{ in.} \]

**B.2.1.3 Load Case 3: 0.9D + 1.0W**

\[ P_{wx} = 4.05 \text{ kip} \]

\[ P_{wx} = 18.4 \text{ kip} \]

\[ w_o = 0.245 \text{ klf} \]

\[ P_{wx} / A_s = 43.8 \text{ psi < 0.06} f' = 240 \text{ psi} \]

\[ A_s = 3.39 \text{ in.}^2 \]

\[ a = 1.25 \text{ in.} \]

\[ c = 1.47 \text{ in.} \]

\[ c/d = 0.336 : tension-controlled \]

\[ I_o = 282 \text{ in.}^4 \]

\[ M_o = 24.2 \text{ ft-kip} \]

\[ \phi M_o = 57.3 \text{ fl-kip} > M_{cr} \]

\[ K_o = 77.9 \text{ kip} \]

\[ M_o = 27.2 \text{ ft-kip} \]

\[ M_o = 39.7 \text{ fl-kip < } \phi M_o \]

\[ \Delta_o = 8.15 \text{ in.} \]

Check service load deflection per ACI 318-11, R14.8.4, with 1.0D + 0.5L + W, and noting L = 0 because only roof load is applied to the panel:
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15) 41

\[ \Delta_{allowable} = \frac{f_c}{150} = 2.36 \text{ in.} \]
\[ \Delta_{cr} = \frac{5M_{cr}f_c^2}{48E_fI_f} = 0.392 \text{ in.} \]
\[ M_{sw} = \frac{w_e^2}{8} + \frac{P_{w,e}}{2} = 0.7(17 \text{ lb/ft}^2)(9.0 \text{ ft})(29.5 \text{ ft})^2 + 4.5 \text{ kip}(3 \text{ in.}) = 12.2 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is:
\[ \Delta_i = \frac{M_{sw}}{M_{cr}} \Delta_{cr} = \frac{12.2 \text{ ft-kip}}{24.2 \text{ ft-kip}} = 0.392 \text{ in.} = 0.198 \text{ in.} \]
\[ \Delta_i = \frac{M_{sw} + P_{w,e}d_i}{M_{cr}} = 12.2 \text{ ft-kip} + 20.4 \text{ kip}(0.198 \text{ in.}) = 12.5 \text{ ft-kip} < \frac{2}{3}M_{cr} = 16.1 \text{ ft-kip} \]

After iteration, maximum service load moment is 12.5 ft-kip, leading to the iterated service load deflection of:
\[ \Delta_s = \frac{M_{sw}}{M_{cr}} \Delta_{cr} = \frac{12.5 \text{ ft-kip}}{24.2 \text{ ft-kip}} = 0.392 \text{ in.} = 0.202 \text{ in.} \]

which is significantly less than the value allowed by ACI 318.

It is important to consider the impact on the rest of the building structure when making a comparison between selecting a larger panel thickness and implementing a double layer of reinforcement for panel types consistent with this example.

B.2.2 Left leg analysis: Reinforcing steel at each face of the panel—Assume 10 No. 4 bars per face (\( A_s = 2.0 \text{ in.}^2 \) per face) in the original 6.25 in. thickness. Joist loads are divided between the individual legs assuming an equivalent simply supported beam across the top of the panel with the supports at the centerline of each leg.

B.2.2.1 Load Case 1: 1.2D + 1.6L_r + 0.5W
\( P_{w,e} = 12.9 \text{ kip} \)
\( P_{w,m} = 26.5 \text{ kip} \)
\( P_{w,m}/A_s = 88.3 \text{ psi} < 0.06f_{c}' = 240 \text{ psi} \)
\[ A_s = A_w + \frac{P_{w,m}}{f_y} \left( \frac{h}{2d} \right) = 2.00 \text{ in.}^2 + \frac{26.5 \text{ kip}}{60 \text{ ksi}} \left( \frac{6.25 \text{ in.}}{2(5.00 \text{ in.})} \right) = 2.28 \text{ in.}^2 \]
\[ a = \frac{A_s f_y}{0.85f_y b} = \frac{2.28 \text{ in.}^2}{0.85(60 \text{ ksi})(4.0 \text{ ft})(12 \text{ in./ft})} = 0.837 \text{ in.} \]
\[ c = \frac{a}{0.85} = \frac{0.837}{0.85} = 0.984 \text{ in.} \]

\[ \frac{c}{d} = 0.197 < 0.375 \therefore \text{tension-controlled} \]
(refer to commentary of ACI 318-11, 9.3.2.2)
\[ I_s = \frac{E_f A_s(d - c)^2 + f_y c}{3} \]
\[ = 8.044(2.28)(5.0 - 0.984)^2 + \frac{(4.0 \text{ ft})(12 \text{ in./ft})(0.984)}{3} = 310 \text{ in}^4 \]
\[ M_{sw} = 12.3 \text{ ft-kip} \]
\[ \phi M_s = \phi A_s f_y \left( \frac{d - a}{2} \right) = 0.9(2.28)(60) \left( 5.0 - \frac{0.837}{2} \right) = 563 \text{ in.-kip} = 46.9 \text{ ft-kip} \]
\[ \phi M_s > M_{cr} \]
\[ \rho = \frac{A_s}{bh} = \frac{2.0 \text{ in.}^2}{4.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00667 > \rho_t = 0.0012 \]
\[ K_s = \frac{48E_f I_f}{5c^2} = \frac{48(3605 \text{ ksi})(310 \text{ in.}^4)}{(29.5 \text{ ft}(12 \text{ in./ft}))^2} = 85.7 \text{ kip} \]
\[ M_{sa} = \frac{w_e^2}{8} + \frac{P_{w,e}d_i}{2} = \frac{1.22kbf}(29.5 \text{ ft})^2 + 12.9 \text{ kip}(3 \text{ in.}) = 14.9 \text{ ft-kip} \]
\[ M_{sw} = \frac{M_{sw}}{0.75K_s} = \frac{1.29\text{ ft-kip}}{0.75(85.7 \text{ kip})} = 25.4 \text{ ft-kip} < \phi M_s \]
\[ \Delta_s = \frac{M_{sw}}{0.75K_s} = \frac{25.4 \text{ ft-kip}(12 \text{ in./ft})}{85.7 \text{ kip}} = 4.74 \text{ in.} \]

B.2.2.2 Load Case 2: 1.2D + 0.5L_r + 1.0W
\( P_{w,e} = 7.75 \text{ kip} \)
\( P_{w,m} = 21.3 \text{ kip} \)
\( w_w = 0.245 \text{ klf} \)
\( P_{w,m}/A_s = 71 \text{ psi} < 0.06f_{c}' = 240 \text{ psi} \)
\[ A_s = 2.22 \text{ in.}^2 \]
\[ a = 0.817 \text{ in.} \]
\[ c = 0.96 \text{ in.} \]
\[ c/d = 0.192 \therefore \text{tension-controlled} \]
\[ I_s = 306 \text{ in.}^4 \]
\[ M_{sw} = 12.3 \text{ ft-kip} \]
\[ \phi M_s = 45.9 \text{ ft-kip} > M_{cr} \]
\[ K_s = 84.4 \text{ kip} \]
\[ M_{sw} = 27.6 \text{ ft-kip} \]
\[ M_{sw} = 41.6 \text{ ft-kip} < \phi M_s \]
\[ \Delta_s = 7.88 \text{ in.} \]

B.2.2.3 Load Case 3: 0.9D + 1.0W
\( P_{w,e} = 4.05 \text{ kip} \)
\( P_{w,m} = 14.2 \text{ kip} \)
\( w_w = 0.245 \text{ klf} \)
\( P_{w,m}/A_s = 47.3 \text{ psi} < 0.06f_{c}' = 240 \text{ psi} \)
\[ A_s = 2.15 \text{ in.}^2 \]
\[ a = 0.790 \text{ in.} \]
c = 0.929 in.
c/d = 0.186: tension-controlled
Ic = 299 in.4
Mcr = 12.3 ft-kip
φMcr = 44.5 ft-kip ≥ Mcr
Ko = 82.6 kip
Msa = 27.2 ft-kip
Mer = 35.2 ft-kip < φMsa
Δo = 6.82 in.

Checking the service load deflection per ACI 318-11, R14.8.4, with 1.0D + 0.5L + Wc and noting L = 0 because only roof load is applied to the panel, the values for Δallowable and Msa are the same as calculated for the singly reinforced analysis. For the 6.25 in. thickness, though, Msa exceeds \( \frac{8.75 \text{ in.(6.0 ft)(12 in./ft)}}{27.2 \text{ ft-kip}} \). Therefore, the panel cracks and \( M_{cr} \) should be used in iteration of deflections to revise \( M_{cr} \).

After iteration, maximum service load moment is 13.9 ft-kip, leading to
\[
\Delta_{cr} = \frac{5M_{cr}I_f^2}{48E_I_{cr}} = 0.547 \text{ in.}
\]
\[
\Delta_c = \frac{5M_fI_r^2}{48E_I_{cr}} = 7.19 \text{ in.}
\]
\[
\Delta_a = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_a - (2/3)M_{cr})} = 1.31 \text{ in.}
\]
where \( M_a \) and \( I_{cr} \) are taken from Load Case 3. This deflection is less than the value allowed by ACI 318, so adjustment to the panel stiffness is not needed.

B.2.3 Right leg analysis: Reinforcing steel centered in panel thickness—Assume seven No. 6 bars (\( A_s = 3.08 \text{ in.}^2 \)) in the 8.75 in. thickness required for B.2.1.1. Joist loads are divided between the individual legs assuming an equivalent simply supported beam across the top of the panel with the supports at the centerline of each leg.

B.2.3.1 Load Case 1: 1.2D + 1.6Lr + 0.5W
\( P_{iso} = 1.2(5.1 \text{ kip}) + 1.6(5.3 \text{ kip}) = 14.6 \text{ kip} \)
\( P_{im} = 14.6 \text{ kip} + 1.2(19.4 \text{ kip}) = 37.9 \text{ kip} \)
\( w_n = 0.5(6.0 \text{ ft}) + 5.0 \text{ ft}(27.2 \text{ lb/ft}^2) = 150 \text{ plf} = 0.150 \text{ klf} \)
\( P_{int} = 37.9 \text{ kip}(1000 \text{ lb/kip}) = 60.2 \text{ psi} < 0.06f'_c = 240 \text{ psi} \)
\( A_s = 8.75 \text{ in.}(6.0 \text{ ft})(12 \text{ in./ft}) = 60.2 \text{ psi} < 0.06f'_c = 240 \text{ psi} \)
\( A_s = A_s + \frac{P_{int}}{f'_c} \left( \frac{h}{2d} \right) = 3.08 \text{ in.}^2 + \frac{37.9 \text{ kip}}{60 \text{ ksi}} \left( \frac{8.75 \text{ in.}}{2(4.38 \text{ in.})} \right) = 3.71 \text{ in.}^2 \)
\( a = \frac{A_s f'_c}{0.85f'_c b} = \frac{3.71 \text{ in.}^2}{0.85(4 \text{ ksi})(60 \text{ ft})(12 \text{ in./ft})} = 0.910 \text{ in.} \)
\( c = \frac{a}{0.85} = \frac{0.910}{0.85} = 1.07 \text{ in.} \)
\( \frac{c}{d} = 0.245 < 0.375: \) tension-controlled

(refer to commentary of ACI 318-11, 9.3.2.2)
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

\[ M_{aw} = 33.1 \text{ ft-kip} \]
\[ M_r = 48.2 \text{ ft-kip} < \phi M_r \]
\[ \Delta_r = 8.2 \text{ in.} \]

Checking the service load deflection per ACI 318-11, R14.8.4, with \( L = 0 \) because only roof load is applied to the panel (\( \Delta_{allowable} = 2.36 \text{ in.} \)):

\[ \Delta_r = \frac{5M_{aw}r^2}{8EI} = \frac{5(17 \text{ lb/ft})^2(11.0 \text{ ft})(29.5 \text{ ft})^2}{8(1000 \text{ lb/kip})(12 \text{ in./ft})} = 14.9 \text{ ft-kip} \]
\[ \Delta_r = \frac{5M_{aw}r^2}{8EI} = 0.392 \text{ in.} \]

Using this value as the initial service load moment, the initial deflection is:

\[ \Delta = \frac{M_{aw}r}{M_r} \Delta_r = \frac{14.9 \text{ ft-kip}}{36.3 \text{ ft-kip}} \times 0.392 \text{ in.} = 0.161 \text{ in.} \]
\[ \therefore M_s = M_{aw} + P_{aw} \Delta = 14.9 \text{ ft-kip} + 24.5 \text{ kip}(0.161 \text{ in.}) \]
\[ = 15.2 \text{ ft-kip} < \frac{2}{3} M_r = 24.2 \text{ ft-kip} \]

After iteration, maximum service load moment is 15.2 ft-kip, leading to the iterated service load deflection of:

\[ \Delta_r = \frac{5M_{aw}r^2}{8EI} = 0.392 \text{ in.} = 0.164 \text{ in.} \]

which is less than the maximum permitted by ACI 318.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3.

\[ A_s = 0.002 A_g = 0.002(8.75 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 6.6 \text{ in.}^2 \]

Therefore, 33 No. 4 bars should be used for the horizontal reinforcement. Additional reinforcement requirements are outlined in ACI 318-11, 14.3.7, for the header and jambs of openings. Figure B.2.3.3 details panel reinforcement for the single-layer scheme.

B.2.4 Right leg analysis: reinforcing steel at each face of the panel—Assume 11 No. 4 bars per face (\( A_s = 2.2 \text{ in.}^2 \) per face) in the original 6.25 in. thickness. Joist loads are divided between the individual legs assuming an equivalent simply supported beam across the top of the panel with the supports at the centerline of each leg.

B.2.4.1 Load Case 1: 1.2D + 0.5L + 0.5W
\[ P_{aw} = 14.6 \text{ kip} \]
\[ P_{aw} = 31.3 \text{ kip} \]
\[ w_w = 0.150 \text{ klf} \]
\[ P_{aw}/A_g = 69.5 \text{ psi} < 0.06\sigma' = 240 \text{ psi} \]
\[ A_s = 2.53 \text{ in.}^2 \]
\[ a = 0.619 \text{ in.} \]
\[ c = 0.728 \text{ in.} \]

\[ \sigma' = 0.146 \cdot \tau_t \]
\[ I_s = 380 \text{ in.}^4 \]
\[ M_r = 18.5 \text{ ft-kip} \]
\[ \phi M_n = 53.3 \text{ ft-kip} > M_r \]
\[ K_b = 105 \text{ kip} \]
\[ M_u = 30.0 \text{ ft-kip} < \phi M_n \]
\[ \Delta_u = 4.58 \text{ in.} \]

Fig. B.2.3.3—Single-layer panel reinforcement.

\[ c/d = 0.142 \text{ : : tension-controlled} \]
\[ I_s = 373 \text{ in.}^4 \]
\[ M_r = 18.5 \text{ ft-kip} \]
\[ \phi M_n = 52.1 \text{ ft-kip} > M_r \]
\[ K_b = 103 \text{ kip} \]
\[ M_u = 33.6 \text{ ft-kip} \]
\[ M_n = 50.1 \text{ ft-kip} < \phi M_n \]
\[ \Delta_u = 7.78 \text{ in.} \]

B.2.4.2 Load Case 2: 1.2D + 0.5L + 1.0W
\[ P_{aw} = 8.77 \text{ kip} \]
\[ P_{aw} = 25.5 \text{ kip} \]
\[ w_w = 0.299 \text{ klf} \]
\[ P_{aw}/A_g = 56.6 \text{ psi} < 0.06\sigma' = 240 \text{ psi} \]
\[ A_s = 2.47 \text{ in.}^2 \]
\[ a = 0.604 \text{ in.} \]
\[ c = 0.711 \text{ in.} \]
\[ c/d = 0.142 \text{ : : tension-controlled} \]
\[ I_s = 380 \text{ in.}^4 \]
\[ M_r = 18.5 \text{ ft-kip} \]
\[ \phi M_n = 52.1 \text{ ft-kip} > M_r \]
\[ K_b = 103 \text{ kip} \]
\[ M_u = 33.6 \text{ ft-kip} \]
\[ M_n = 50.1 \text{ ft-kip} < \phi M_n \]
\[ \Delta_u = 7.78 \text{ in.} \]

B.2.4.3 Load Case 3: 0.9D + 1.0W
\[ P_{aw} = 4.59 \text{ kip} \]
\[ P_{aw} = 17.1 \text{ kip} \]
\[ w_w = 0.299 \text{ klf} \]
\[ P_{aw}/A_g = 38.0 \text{ psi} < 0.06\sigma' = 240 \text{ psi} \]
\[ A_s = 2.38 \text{ in.}^2 \]
\[ a = 0.583 \text{ in.} \]
\[ c = 0.686 \text{ in.} \]
\[ c/d = 0.137 \text{ : : tension-controlled} \]
\[ I_s = 364 \text{ in.}^4 \]
\[ M_r = 18.5 \text{ ft-kip} \]
\[ \phi M_n = 50.4 \text{ ft-kip} > M_r \]
\[ K_b = 100 \text{ kip} \]
\[ M_u = 33.1 \text{ ft-kip} \]
Fig. B.2.4.3—Double-layer panel reinforcement.

\[ M_s = 42.8 \text{ ft-kip} < \phi M_u \]
\[ \Delta_s = 6.82 \text{ in.} \]

Checking the service load deflection per ACI 318-11, R14.8.4, with \( 1.0D + 0.5L + W_e \), and noting \( L = 0 \) because only roof load is applied to the panel

\[ \Delta_{\text{allowable}} = 2.36 \text{ in.} \]

\[ M_{\text{ue}} = \frac{wL^2}{8} + \frac{P_e c_e}{2} = \frac{0.7(17 \text{ lb/ft}^2)(11.0 \text{ ft})(29.5 \text{ ft}^2)}{8(1000 \text{ lb/kip})} + \frac{5.1 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 14.9 \text{ ft-kip} \]

This initial service moment without \( P-\Delta \) effects exceeds \((2/3)M_u = 12.3 \text{ ft-kip}\), and the initial service load deflection is calculated by:

\[ \Delta_{\text{cr}} = \frac{5M_{\text{ue}}L^2}{48EJ_{\text{cr}}} = 0.551 \text{ in.} \]
\[ \Delta_s = \frac{5M_{\text{ue}}L^2}{48EJ_{\text{cr}}} = 6.74 \text{ in.} \]
\[ \Delta = \frac{2}{3}\Delta_{\text{cr}} + \left( \frac{M_{\text{ue}}}{M_s} - \frac{2}{3}\Delta_{\text{cr}} \right) \left( \frac{M_s}{M_{\text{ue}}} - \frac{2}{3}\Delta_{\text{cr}} \right) = 0.746 \text{ in.} \]

using the values for \( M_s \) and \( J_{\text{cr}} \) from Load Case 3. After iteration, maximum service load moment is 16.4 ft-kip, leading to:

\[ M_{\Delta} = M_{\Delta_{\text{cr}}} + P_{\Delta_{\text{cr}}} \Delta_{\Delta} \]
\[ \Delta_{\Delta} = \frac{2}{3}\Delta_{\text{cr}} + \left( \frac{M_s}{M_{\Delta}} - \frac{2}{3}\Delta_{\text{cr}} \right) \left( \frac{M_{\Delta}}{M_s} - \frac{2}{3}\Delta_{\text{cr}} \right) = 0.970 \text{ in.} \]

Fig. B.3—Plain tilt-up panel with concentrated axial load.

which is less than the maximum permitted by ACI 318, so no adjustment to the panel stiffness is necessary.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:

\[ A = 0.002A_e = 0.002(6.25 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 4.6 \text{ in.}^2 \]

Because maximum bar spacing is 18 in., 22 No. 4 bars should be used on each face for the horizontal reinforcement. Additional reinforcement requirements are outlined in ACI 318-11, 14.3.7, for the headers and jambs of openings.

Figure B.2.4.3 details panel reinforcement for the double-layer scheme.

B.2.5 Summary of panel reinforcing steel—Neglecting trim bars and miscellaneous reinforcing steel, the weight of the primary horizontal and vertical reinforcing steel in the single mat option is 1089 lb, compared with 1426 lb for the double mat option. The designer should note that the single mat option required a panel thickness increase of 2.5 in. This represents approximately 3.6 more cubic yards of concrete, translating to a panel 14.7 kip heavier. Despite the savings in reinforcing steel weight, the impact the thicker panel may have on the remainder of the project should be investigated.

B.3—Panel with concentrated axial load design example

Figure B.3 illustrates the geometry of the sample panel with a concentrated axial load. The panel supports the load from one roof girder bearing in a wall pocket (eccentric axial load) 4.75 ft from the right edge of the panel in addition to the wind (lateral force). A summary of the applied loading is

\[ \begin{align*}
P_{DL} &= 26.0 \text{ kip} \\
P_{LL} &= 25.0 \text{ kip} \\
e_e &= 3 \text{ in. (assumed)} \\
w &= 27.2 \text{ lb/ft}^2
\end{align*} \]
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

The weight of the tilt-up panel above the design section (centerline of the unbraced length) is

\[ \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \times 150 \text{ lb/ft}^3 (12.1 \text{ ft}) \left( \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \right) \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) \]

= 15.4 kip

B.3.1 Reinforcing steel centered in panel thickness—
Assume 29 No. 6 bars \((A_s = 12.8 \text{ in.}^2)\) in the design strip illustrated.

B.3.1.1 Load Case 1: 1.2D + 1.6L + 0.5W

\[ P_{aw} = 1.2(26.0 \text{ kip}) + 1.6(25.0 \text{ kip}) = 71.2 \text{ kip} \]

\[ w_{si} = 0.5(12.1 \text{ ft})(27.2 \text{ lb/ft}^2) = 165 \text{ plf} = 0.165 \text{ klf} \]

Check vertical stress at the midheight section of panel per ACI 318-11, 14.8.2.6:

\[ \frac{P_{aw}}{A_s} = \frac{89.7 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(12.1 \text{ ft})(12 \text{ in./ft})} = 98.6 \text{ psi} < 0.06 f'_c = 240 \text{ psi} \]

Check the design moment strength:

\[ A_w = A_s + P_{aw} \left( \frac{h}{2d} \right) = 12.8 \text{ in.}^2 + \frac{89.7 \text{ kip}}{60 \text{ ksi}} \left( \frac{6.25 \text{ in.}}{2(3.13 \text{ in.})} \right) = 14.3 \text{ in.}^2 \]

\[ a = \frac{A_w f_y}{0.85 f'\text{b}} = \frac{14.3 \text{ in.}^2 (60 \text{ ksi})}{0.85(4 \text{ ksi})(12.1 \text{ ft})(12 \text{ in./ft})} = 1.73 \text{ in.} \]

\[ c = \frac{a}{0.85} = \frac{1.73}{0.85} = 2.04 \text{ in.} \]

\[ \frac{c}{d} = 0.653 > 0.375 \] (refer to commentary of ACI 318-11, 9.3.2.2)

Therefore, the requirements of ACI 318-11, 14.8.2.3, that the section be tension-controlled are not met. At this point, there are two options: provide a double layer of reinforcement (as demonstrated in this example), or increase wall panel thickness and repeat the analysis. To have a tension-controlled section for analysis, select a panel thickness of 7.75 in. The distance \(d\) (using a 3.0 in. chair to the horizontal bar) is calculated to be 3.88 in. Assume 17 No. 6 bars \((A_s = 7.48 \text{ in.}^2)\) in the design strip. The weight of the tilt-up panel above the design section is 19.1 kip.

\[ P_{aw} = 1.2(26.0 \text{ kip}) + 1.0(19.1 \text{ kip}) = 94.1 \text{ kip} \]

Check the applied moment per ACI 318-11, 14.8.3:

\[ K_s = \frac{48E_f I_{cr}}{5f'_c^2} = \frac{48(3605 \text{ ksi})(590 \text{ in.}^4)}{5(29.5 \text{ ft})(12 \text{ in./ft})^2} = 163 \text{ kip} \]

\[ M_{aw} = \frac{w_{si} f_c'^2 + P_{aw} e_{cc}}{8} + 71.2 \text{ kip}(3 \text{ in.}) = 26.8 \text{ ft-kip} \]

\[ M_{aw} = \frac{M_{aw}}{1 - \frac{P_{aw}}{0.75 K_s}} = \frac{26.8 \text{ ft-kip}}{1 - \frac{94.1 \text{ kip}}{0.75(163 \text{ kip})}} = 117 \text{ ft-kip} < \phi M_u \]

\[ \Delta_s = \frac{M_{aw}}{0.75 K_s} = \frac{117 \text{ ft-k}(12 \text{ in./ft})}{0.75(163 \text{ kip})} = 11.4 \text{ in.} \]

B.3.1.2 Load Case 2: 1.2D + 0.5L + 1.0W

\[ P_{aw} = 43.7 \text{ kip} \]

\[ w_{si} = 0.329 \text{ klf} \]

\[ P_{aw} = 43.7 \text{ kip} \]

\[ w_{si} = 0.329 \text{ klf} \]

\[ P_{aw} = 43.7 \text{ kip} \]

\[ w_{si} = 0.329 \text{ klf} \]

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\[ P_{aw} = 43.7 \text{ kip} \]

\[ w_{si} = 0.329 \text{ klf} \]

\[ P_{aw} = 43.7 \text{ kip} \]

\[ w_{si} = 0.329 \text{ klf} \]
Fig. B.3.1.3—Single-layer panel reinforcement.

$\phi M_u = 130 \text{ ft-kip} > M_{cr}$

$K_b = 159 \text{ kip}$

$M_{ad} = 41.3 \text{ ft-kip}$

$M_u = 940 \text{ ft-kip} < \phi M_u$

$\Delta_w = 9.48 \text{ in.}$

**B.3.1.3 Load Case 3: 0.9D + 1.0W**

$P_{ua} = 23.4 \text{ kip}$

$P_{wu} = 40.6 \text{ kip}$

$w_u = 0.329 \text{ klf}$

$P_{uw}/A_g = 36.0 \text{ psi} < 0.06f_y' = 240 \text{ psi}$

$A_{wc} = 8.16 \text{ in.}^2$

$a = 0.989 \text{ in.}$

$c = 1.16 \text{ in.}$

$c/d = 0.300 \cdot \text{ tension-controlled}$

$I_{cr} = 559 \text{ in.}^4$

$M_{cr} = 57.5 \text{ ft-kip}$

$\phi M_u = 124 \text{ ft-kip} > M_{cr}$

$K_b = 154 \text{ kip}$

$M_{ad} = 38.8 \text{ ft-kip}$

$M_u = 59.8 \text{ ft-kip} < \phi M_u$

$\Delta_w = 6.20 \text{ in.}$

Check service load deflection per ACI 318-11, R14.8.4, with $1.0D + 0.5L + W_u$ noting $L = 0$ because only roof load is applied to the panel

$\Delta_{allowable} = \frac{1}{150} = 2.36 \text{ in.}$

$\Delta_s = \frac{5M_{cr}f_y^2}{48EI_s} = 0.443 \text{ in.}$

Using this value as the initial service load moment, the initial deflection is:

$\Delta_i = \frac{M_{cr}f_y^2}{K_b} = \frac{18.9 \text{ ft-kip}}{57.5 \text{ ft-kip}} = 0.443 \text{ in.} = 0.146 \text{ in.}$

$\therefore M_u = M_{ad} + P_{uw} \Delta_s = 18.9 \text{ ft-kip} + 45.1 \text{ kip} \cdot (0.146 \text{ in.})$

$= 19.5 \text{ ft-kip} < (2/3)M_{cr} = 38.3 \text{ ft-kip}$

After iteration, maximum service load moment is 19.5 ft-kip, leading to the iterated service load deflection of:

$\Delta_s = \frac{M_{cr}f_y^2}{K_b} = \frac{19.5 \text{ ft-kip}}{57.5 \text{ ft-kip}} = 0.443 \text{ in.} = 0.150 \text{ in.}$

which is less than the maximum permitted by ACI 318, so no adjustment to the panel stiffness is necessary.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:

$A_s = 0.002 A_g = 0.002(7.75 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 5.8 \text{ in.}^2$

Therefore, 29 No. 4 bars should be used for the horizontal reinforcement. Figure B.3.1.3 details panel reinforcement for this case.

**B.3.2 Reinforcing steel at each face of the panel**—Assume 16 No. 4 bars per face ($A_s = 3.2 \text{ in.}^2$ per face) in the design strip in the original 6.25 in. thickness.

**B.3.2.1 Load Case 1: 1.2D + 1.6L + 0.5W**

$P_{ua} = 71.2 \text{ kip}$

$P_{uw} = 89.7 \text{ kip}$

$w_u = 0.165 \text{ klf}$

$P_{uw}/A_g = 98.6 \text{ psi} < 0.06f_y' = 240 \text{ psi}$

$A_{wc} = A_s + P_{uw}/f_y = \frac{h}{2d} = 4.13 \text{ in.}^2$

$a = \frac{A_{wc}f_y}{0.85f_y} = \frac{4.13 \text{ in.}^2}{(60 \text{ ksi})(12 \text{ in./ft})} = 0.501 \text{ in.}$

$c = \frac{0.85}{0.85} = 0.590 \text{ in.}$

$\therefore c/d = 0.118 < 0.375 \cdot \text{ tension-controlled}$

(refer to commentary to ACI 318-11, 9.3.2.2)
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

\[ I_x = \frac{E}{E_c} A_e (d - c)^2 + \frac{t_e d^2}{3} \]
\[ = 8.044(4.13)(5.0 - 0.590)^2 + \frac{(12.1 \text{ ft})(12 \text{ in./ft})(0.590)^3}{3} = 656 \text{ in}^4 \]

\[ M_{cr} = 37.4 \text{ ft-kip} \]

\[ \phi M_e = \phi A_e f_y \left( d - \frac{a}{2} \right) \]
\[ = 0.9(4.13)(60) \left( 5.0 - \frac{0.501}{2} \right) = 1060 \text{ in.-kip} \approx 88.3 \text{ ft-kip} \]

\[ \phi M_e > M_{cr} \]

\[ \rho = \frac{A_e}{bh} = \frac{3.2 \text{ in}^2}{12.1 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00353 > \rho_r = 0.0012 \]

\[ K_s = \frac{48 E I_x}{5 e_c} = \frac{48(3605 \text{ ksi})(657 \text{ in}^4)}{5(29.5 \text{ ft})(12 \text{ in./ft})^3} = 181 \text{ kip} \]

\[ M_s = \frac{w_e f_y^2}{8} + \frac{P e_c}{2} = \frac{0.165 \text{ klf}(29.5 \text{ ft})^2}{8} + \frac{71.2 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 26.8 \text{ ft-kip} \]

\[ M_a = \frac{M_{sa}}{1 - \frac{0.75}{K_s}} = \frac{26.8 \text{ ft-kip}}{1 - \frac{0.75}{181 \text{ kip}}} = 78.8 \text{ ft-kip} < \phi M_e \]

\[ \Delta_a = \frac{M_{sa}}{0.75K_s} = \frac{78.8 \text{ ft-kip}(12 \text{ in./ft})}{0.75(181 \text{ kip})} = 6.95 \text{ in.} \]

**B.3.2.2 Load Case 2: 1.2D + 0.5L + 1.0W**

\[ P_{sa} = 43.7 \text{ kip} \]
\[ P_{sm} = 62.2 \text{ kip} \]
\[ w_u = 0.329 \text{ klf} \]
\[ P_{sa}/A_e = 68.4 \text{ psi} < 0.06 f_y = 240 \text{ psi} \]
\[ A_{re} = 3.85 \text{ in}^2 \]
\[ d = 0.467 \text{ in.} \]
\[ c = 0.549 \text{ in.} \]
\[ c/d = 0.110 \therefore \text{ tension-controlled} \]
\[ I_x = 621 \text{ in}^4 \]

\[ M_{cr} = 37.4 \text{ ft-kip} \]

\[ \phi M_e = 82.5 \text{ ft-kip} > M_{cr} \]

\[ K_s = 162 \text{ kip} \]

\[ M_{sa} = 38.8 \text{ ft-kip} \]

\[ M_{sm} = 55.9 \text{ ft-kip} < \phi M_e \]

\[ \Delta_a = 5.51 \text{ in.} \]

Check service load deflection per ACI 318-11, R14.8.4, with 1.0D + 0.5L + W, noting L = 0 because only roof load is applied to the panel:

\[ \Delta_{allowable} = \frac{\Delta_a}{\kappa} = \frac{2.36}{150} \approx 0.016 \text{ in.} \]

\[ \Delta_s = \frac{5M_{cr}^2}{48E f_y^2} = 0.549 \text{ in.} \]

\[ M_s = \frac{w_e f_y^2}{8} + \frac{P e_c}{2} = \frac{0.7(17 \text{ lb/ft}^2)(12.1 \text{ ft})(29.5 \text{ ft})^2}{8(1000 \text{ lb/kip})} + \frac{26.0 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 18.9 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is:

\[ \Delta_s = \frac{M_{sa}}{M_{cr}} = \frac{18.9 \text{ ft-kip}}{37.4 \text{ ft-kip}} = 0.519 \text{ in.} \]

\[ \Delta_s = \frac{18.9 \text{ ft-kip}}{37.4 \text{ ft-kip}} = 0.292 \text{ in.} \]

**B.3.2.3 Load Case 3: 0.9D + 1.0W**

\[ P_{sa} = 23.4 \text{ kip} \]
\[ P_{sm} = 37.3 \text{ kip} \]
\[ w_u = 0.329 \text{ klf} \]
\[ P_{sa}/A_e = 41.0 \text{ psi} < 0.06 f_y = 240 \text{ psi} \]
\[ A_{re} = 3.59 \text{ in}^2 \]
\[ d = 0.435 \text{ in.} \]
\[ c = 0.512 \text{ in.} \]
\[ c/d = 0.102 \therefore \text{ tension-controlled} \]
\[ I_x = 258 \text{ in}^4 \]

Check service load deflection per ACI 318-11, R14.8.4, with 1.0D + 0.5L + W, noting L = 0 because only roof load is applied to the panel:

\[ \Delta_{allowable} = \frac{\Delta_a}{\kappa} = \frac{2.36}{150} \approx 0.016 \text{ in.} \]

\[ \Delta_s = \frac{5M_{cr}^2}{48E f_y^2} = 0.549 \text{ in.} \]

\[ M_s = \frac{w_e f_y^2}{8} + \frac{P e_c}{2} = \frac{0.7(17 \text{ lb/ft}^2)(12.1 \text{ ft})(29.5 \text{ ft})^2}{8(1000 \text{ lb/kip})} + \frac{26.0 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 18.9 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is:

\[ \Delta_s = \frac{M_{sa}}{M_{cr}} = \frac{18.9 \text{ ft-kip}}{37.4 \text{ ft-kip}} = 0.519 \text{ in.} \]

\[ \Delta_s = \frac{18.9 \text{ ft-kip}}{37.4 \text{ ft-kip}} = 0.292 \text{ in.} \]

After iteration, maximum service load moment is 19.9 ft-kip, leading to the iterated service load deflection of

\[ \Delta_s = \frac{M_{sa}}{M_{cr}} = \frac{19.9 \text{ ft-kip}}{37.4 \text{ ft-kip}} = 0.529 \text{ in.} \]

Load Case 3: 0.9D + 1.0W

\[ P_{sa} = 23.4 \text{ kip} \]
\[ P_{sm} = 37.3 \text{ kip} \]
\[ w_u = 0.329 \text{ klf} \]
\[ P_{sa}/A_e = 41.0 \text{ psi} < 0.06 f_y = 240 \text{ psi} \]
\[ A_{re} = 3.59 \text{ in}^2 \]
\[ d = 0.435 \text{ in.} \]
\[ c = 0.512 \text{ in.} \]
\[ c/d = 0.102 \therefore \text{ tension-controlled} \]
\[ I_x = 258 \text{ in}^4 \]

**B.3.2.3 Load Case 3: 0.9D + 1.0W**

\[ P_{sa} = 23.4 \text{ kip} \]
\[ P_{sm} = 37.3 \text{ kip} \]
\[ w_u = 0.329 \text{ klf} \]
\[ P_{sa}/A_e = 41.0 \text{ psi} < 0.06 f_y = 240 \text{ psi} \]
\[ A_{re} = 3.59 \text{ in}^2 \]
\[ d = 0.435 \text{ in.} \]
\[ c = 0.512 \text{ in.} \]
\[ c/d = 0.102 \therefore \text{ tension-controlled} \]
\[ I_x = 258 \text{ in}^4 \]
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

Fig. B.4—Plain tilt-up panel with canopy.

which is less than the value allowed by ACI 318.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:

\[ A_s = 0.002A_g = 0.002(6.25 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 4.6 \text{ in.}^2 \]

Maximum spacing of the horizontal bars is 18 in.; therefore, 22 No. 4 bars on each face should be used. Figure B.3.2.3 details panel reinforcement for the double layer scheme.

B.3.3 Summary of panel reinforcing steel—Neglecting trim bars and miscellaneous reinforcing steel, the weight of the primary horizontal and vertical reinforcing steel in the single mat option is 1129 lb, compared with 1210 lb for the double mat option. The designer should note that the single mat option required a panel thickness increase of 1.5 in. This represents approximately 2.2 more cubic yards of concrete, translating to a panel 8.7 kip heavier. Despite the savings in reinforcing steel weight, the impact the thicker panel may have on the remainder of the project should be investigated.

B.4—Panel with concentrated lateral load design example

Figure B.4 illustrates the geometry of the sample panel with a concentrated lateral load. In this example, a canopy is added to the plain panel from Fig. B.1. The load diagram illustrates a horizontal force couple on the face of the panel, which replaces the eccentric vertical load from the canopy. A summary of the applied loading is

\[ P_{\text{Ll,conopy}} = 5.4 \text{ kip} \]
\[ \Delta w = 27.2 \text{ lb/ft}^2 \]
\[ \ell_c = 31.0 \text{ ft} - 1.5 \text{ ft} = 29.5 \text{ ft} \]
\[ a = 16.0 \text{ ft} - 1.5 \text{ ft} = 14.5 \text{ ft} \]
\[ b = 6.0 \text{ ft} \]

The weight of the tilt-up panel above the design section (centerline of the unbraced length) is:

\[ \left( \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \right) 150 \text{ lb/ft}(15.0 \text{ ft}) \left[ \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 19.0 \text{ kip} \]

Similarly, only half of the canopy load is applied to the face of the panel above the design section, so only half of the load is used in the calculations that follow.

B.4.1 Reinforcing steel centered in panel thickness—Assume 19 No. 6 bars \( (A_s = 8.36 \text{ in.}^2) \).

B.4.1.1 Load Case 1: 1.2D + 1.6L + 0.5W

\[ P_{\text{Ll}} = 1.2[7.2 \text{ kip} + 0.5(0.9 \text{ kip})] + 1.6[7.5 \text{ kip} + 0.5(5.4 \text{ kip})] = 25.5 \text{ kip} \]
\[ P_{\text{conopy}} = 25.5 \text{ kip} + 1.2(19.0 \text{ kip}) = 48.3 \text{ kip} \]
\[ w_c = 0.5(15.0 \text{ ft})(27.2 \text{ lb/ft}^2) = 204 \text{ plf} = 0.204 \text{ kif} \]

Check vertical stress at the midheight section of panel per ACI 318-11, 14.8.2.6:

\[ \frac{P_{\text{Ll}}}{A_s} = \frac{48.3 \text{ k}(1000 \text{ lb/kip})}{6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft})} = 42.9 \text{ psi} < 0.06f'_c = 240 \text{ psi} \]

Check the design moment strength:

\[ A_s = A_s + \frac{P_{\text{conopy}}}{f_y} \left( \frac{h}{2d} \right) \]
\[ = 8.36 \text{ in.}^2 + \frac{48.3 \text{ kip} \left( \frac{6.25 \text{ in.}}{2(3.13 \text{ in.})} \right)}{60 \text{ ksi}} = 9.17 \text{ in.}^2 \]

\[ a = \frac{A_s f_y}{0.85f_y b} = \frac{9.17 \text{ in.}^2}{0.85(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.899 \text{ in.} \]

\[ c = \frac{a}{d} = \frac{0.899}{0.85} = 1.06 \text{ in.} \]

\[ \frac{c}{d} = 0.339 < 0.375 \therefore \text{ tension-controlled} \]

(Refer to commentary of ACI 318-11, 9.3.2.2)

\[ I_v = \frac{E_c}{E_s} A_s (d - c)^3 + \frac{E_s c^3}{3} \]
\[ = 8.044(9.17)(3.13 - 1.06)^3 + \left( \frac{15.0 \text{ ft}(12 \text{ in./ft})(1.06)}{3} \right) = 386 \text{ in.}^4 \]

\[ M_{\text{conopy}} = \frac{f_s I_v}{h_i} = f_s S = f_s \left( \frac{1}{6} \right) \]
\[ = 0.474 \text{ ksf} \left( \frac{1}{6} \right)(15.0 \text{ ft})(6.25 \text{ in.})^2 = 46.3 \text{ ft-kip} \]
\[ \Phi M_s = \Phi A_s \int_0^d \left( \frac{d - \ell}{2} \right) \]
\[ = 0.9(9.17)(60)(3.13 - 0.899) = 1320 \text{ in.-kip} = 110 \text{ ft-kip} \]

Check minimum reinforcement required by ACI 318-11, 14.8.2.4:
\[ \Phi M_s > M_{cr} \]

Check minimum reinforcement required by ACI 318-11, 14.3.2:
\[ \rho = \frac{A}{bh} = \frac{8.36 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.)}} = 0.00743 > \rho_t = 0.0015 \]

Check applied moment per ACI 318-11, 14.8.3
\[ K_b = \frac{48E}{I_c^2} = \frac{48(3605 \text{ ksi})(386 \text{ in.}^4)}{29.5 \text{ ft}(12 \text{ in./ft})^2} = 107 \text{ kip} \]
\[ M_{sa} = \frac{w L^2}{8} + \frac{P_{sa}}{\ell_c} + \frac{H_{ba}}{\ell_c} \]
\[ = 0.204 \text{ klf}(29.5 \text{ ft})^2 + 206 \text{ kip}(3 \text{ in.)}/2(12 \text{ in./ft}) + 4.9 \text{ kip}(6.0 \text{ ft})(14.5 \text{ ft})/29.5 \text{ ft} \]
\[ = 39.2 \text{ ft-kip} \]
\[ M_s = \frac{M_{sa}}{1 - \frac{P_{sa}}{0.75K_b}} = \frac{39.2 \text{ ft-kip}}{1 - \frac{48.3 \text{ kip}}{0.75(107 \text{ kip})}} = 99.0 \text{ ft-kip} < \Phi M_s \]
\[ \Delta_s = \frac{M_s}{0.75K_b} = \frac{99.0 \text{ ft-kip}(12 \text{ in./ft})}{0.75(107 \text{ kip})} = 14.9 \text{ in.} \]

B.4.1.2 Load Case 2: 1.2D + 0.5L_r + 1.0W
\[ P_{sa} = 14.3 \text{ kip} \]
\[ P_{sa} = 37.1 \text{ kip} \]
\[ w_s = 0.408 \text{ klf} \]
\[ P_{sa}A_g = 33.0 \text{ psi} < 0.06f' = 240 \text{ psi} \]
\[ A_{cr} = 8.98 \text{ in.}^2 \]
\[ a = 0.880 \text{ in.} \]
\[ c = 1.04 \text{ in.} \]
\[ c/d = 0.323 < 1.0 \text{ tension-controlled} \]
\[ I_c = 382 \text{ in.}^4 \]
\[ \Phi M_s = 108 \text{ ft-kip} > M_{cr} \]
\[ K_b = 105 \text{ kip} \]
\[ M_{sa} = 51.5 \text{ ft-kip} \]
\[ M_s = 97.4 \text{ ft-kip} < \Phi M_s \]
\[ \Delta_s = 14.8 \text{ in.} \]

B.4.1.3 Load Case 3: 0.9D + 1.0W
\[ P_{sa} = 6.89 \text{ kip} \]
\[ P_{sa} = 24.0 \text{ kip} \]
\[ w_s = 0.408 \text{ klf} \]
\[ P_{sa}A_g = 21.3 \text{ psi} < 0.06f' = 240 \text{ psi} \]
\[ A_{cr} = 8.76 \text{ in.}^2 \]
\[ a = 0.859 \text{ in.} \]
\[ c = 1.01 \text{ in.} \]
\[ c/d = 0.323 < 1.0 \text{ tension-controlled} \]
\[ I_c = 377 \text{ in.}^4 \]
\[ M_{cr} = 46.3 \text{ ft-kip} \]
\[ \Phi M_s = 106 \text{ ft-kip} > M_{cr} \]
\[ K_b = 104 \text{ kip} \]
\[ M_{sa} = 46.4 \text{ ft-kip} \]
\[ M_s = 67.0 \text{ ft-kip} < \Phi M_s \]
\[ \Delta_s = 10.3 \text{ in.} \]

Check service load deflection per ACI 318-11, R14.8.4, with 1.0D + 0.5L_r + W_{cr} and noting L = 0 because only roof load is applied to the panel
\[ \Delta_{allowable} = \frac{\ell_c}{150} = 2.36 \text{ in.} \]
\[ \Delta_c = \frac{5M_{cr}}{48E/I_c} = 0.549 \text{ in.} \]
\[ M_{sa} = \frac{w_L^2}{8} + \frac{P_{sa}}{\ell_c} + \frac{H_{ba}}{\ell_c} \]
\[ = 0.7(17 \text{ lb/ft}^2)(15.0 \text{ ft})(29.5 \text{ ft})^2 + 7.2 \text{ kip}(3 \text{ in.})/2(12 \text{ in./ft}) \]
\[ + 0.9 \text{ kip}(6.0 \text{ ft})(14.5 \text{ ft})/29.5 \text{ ft} = 23.0 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is
\[ \Delta_i = \frac{M_{sa}}{M_{cr}} \Delta_c = \frac{23.0 \text{ ft-kip}}{46.3 \text{ ft-kip}}(0.549 \text{ in.}) = 0.273 \text{ in.} \]
\[ \Delta_i = M_{sa} + P_{sa} \Delta_i = 23.0 \text{ ft-kip} + 27.1(0.273 \text{ in.}) \]
\[ = 23.6 \text{ ft-kip} < (2/3)M_{cr} = 30.9 \text{ ft-kip} \]

After iteration, maximum service load moment is 23.6 ft-kip, leading to an iterated service load deflection of:
\[ \Delta_i = \frac{M_{sa}}{M_{cr}} \Delta_c = \frac{23.6 \text{ ft-kip}}{46.3 \text{ ft-kip}}(0.549 \text{ in.}) = 0.280 \text{ in.} \]

which is significantly less than the value allowed by ACI 318.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:
\[ A_s = 0.002A_g = 0.002(6.25 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 4.6 \text{ in.}^2 \]

Therefore, 23 No. 4 bars should be used for the horizontal reinforcement. Figure B4.1.3 details panel reinforcement for this case.

B.4.2 Reinforcing steel at each face of the panel—Assume 16 No. 4 bars per face (A_s = 3.20 \text{ in.}^2 per face) to meet minimum reinforcing steel requirements.

B.4.2.1 Load Case 1: 1.2D + 1.6L_r + 0.5W
\[ P_{sa} = 25.5 \text{ kip} \]
\[ P_{sa} = 48.3 \text{ kip} \]
\[ w_s = 0.204 \text{ klf} \]
Fig. B.4.1.3—Single-layer panel reinforcement.

\[ P_{w/m} = 42.9 \text{ psi} \times 0.06f_y = 240 \text{ psi} \]

\[ A_e = A + \frac{P_{w/m}}{f_y} \left( \frac{h}{2d} \right) = 3.20 \text{ in.}^2 + 48.3 \text{ kip} \left( \frac{6.25 \text{ in.}}{2(5.00 \text{ in.})} \right) = 3.70 \text{ in.}^2 \]

\[ a = \frac{A_e f_y}{0.85 f_y b} = \frac{3.70 \text{ in.}^2 (60 \text{ ksi})}{0.85(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.363 \text{ in.} \]

\[ c = \frac{a}{0.85} = 0.427 \text{ in.} \]

\[ \frac{c}{d} = 0.085 < 0.375 \Rightarrow \text{tension-controlled} \]

(Refer to commentary of ACI 318-11, 9.3.2.2)

\[ I_{cr} = \frac{E_e A_e (d - c)^2 + \frac{f_y c^4}{3}}{3} = \frac{8.043(3.70)(5.0 - 0.427)^2 + (15.0 \text{ ft})(12 \text{ in./ft})(0.427)^4}{3} = 628 \text{ in.}^4 \]

\[ M_{cr} = 46.3 \text{ ft-kip} \]

\[ \phi M_{se} = \phi A_e f_y \left( d - \frac{c}{2} \right) \]

\[ = 0.9(3.70)(60) \left( 5.0 - \frac{0.363}{2} \right) = 964 \text{ in.-kip} = 80.3 \text{ ft-kip} \]

\[ \phi M_{se} > M_{cr} \]

\[ \rho = \frac{A_e}{bh} = \frac{3.20 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00284 > \rho_r = 0.0012 \]

\[ K_b = \frac{48E I_{cr}}{5f_y c} = \frac{48(3605 \text{ ksi})(628 \text{ in.}^4)}{5[29.5 \text{ ft}(12 \text{ in./ft})]^2} = 173 \text{ kip} \]

**B.4.2.2 Load Case 2: 1.2D + 0.5L_e + 1.0W**

\[ P_{w/m} = 14.3 \text{ kip} \]

\[ P_{w/m} = 37.1 \text{ kip} \]

\[ w_u = 0.408 \text{ klf} \]

\[ P_{w/m}/A_e = 33.0 \text{ psi} < 0.06f_y = 240 \text{ psi} \]

\[ A_e = 3.59 \text{ in.}^2 \]

\[ a = 0.352 \text{ in.} \]

\[ c = 0.414 \text{ in.} \]

\[ \frac{c}{d} = 0.083 \Rightarrow \text{tension-controlled} \]

\[ I_{cr} = 611 \text{ in.}^4 \]

\[ M_{se} = 46.3 \text{ ft-kip} \]

\[ \phi M_{se} = 77.9 \text{ ft-kip} > M_{cr} \]

\[ K_b = 169 \text{ kip} \]

\[ M_{se} = 51.5 \text{ ft-kip} \]

\[ M_{se} = 72.8 \text{ ft-kip} < \phi M_{se} \]

\[ \Delta_{se} = 6.91 \text{ in.} \]

Because maximum service moment at midheight did not exceed two-thirds of the cracking moment in the aforementioned single-layer analysis, there is no need to repeat the calculation for this analysis.

Using the minimum horizontal reinforcing steel requirements previously determined, but noting the maximum bar spacing is 18 in., Fig. B.4.2.3 details panel reinforcement for the double layer scheme. The designer is encouraged to investigate the impact of load reversal due to wind uplift forces and how it would impact the reinforcing steel design for both the single and double mat options.
B.4.3 Summary of panel reinforcing steel—Neglecting trim bars and miscellaneous reinforcing steel, the weight of the primary horizontal and vertical reinforcing steel in the single mat option is 1101 lb, compared with 1108 lb for the double mat option. The designer may consider other project considerations, such as ease of placement, number of repetitive panels, or the value of consistent detailing before selecting the most economical reinforcing steel configuration.

B.5—Multi-story panel design example

Multi-story tilt-up panel design presents unique challenges not encountered in the previous examples. To be economical, the process for choosing the panel thickness is much different than a typical single-story application, and can often result in a much thinner section. Consequently, stresses on the panel during lifting and temporary construction conditions where the roof, intermediate floors, or both, are not attached, should be investigated for the influence on required vertical reinforcing steel. The designer should carefully select which vertical loads will be present in the temporary condition, and what, if any, reduction can be taken on the applied lateral load.

This example examines the reinforcing steel required for the final in-service condition only. Figure B.5 is a cross section of the sample multi-story panel with no openings. A summary of the applied loading is:

\[
\begin{align*}
P_{DL,\text{roof}} &= 3(2.4 \text{ kip}) = 7.2 \text{ kip} \\
P_{LL,\text{roof}} &= 3(2.5 \text{ kip}) = 7.5 \text{ kip} \\
\varepsilon_{x,\text{roof}} &= \text{assumed} \\
P_{DL,\text{floor}} &= 6(2.95 \text{ kip}) = 17.7 \text{ kip} \\
P_{LL,\text{floor}} &= 6(5.0 \text{ kip}) = 30.0 \text{ kip} \\
w &= 27.2 \text{ lb/ft}^2
\end{align*}
\]

where roof framing members are assumed to bear on seats in wall pockets and the floor framing members are assumed to bear on sufficiently stiff seats such that the eccentric vertical load is at the face of the panel.

Bending moment diagrams for the three-span continuous panel are shown for each ACI load case considered based on the applied loads without considering the P-\(\Delta\) effect. The weight of the tilt-up panel above the design section is calculated individually for each load case because location of the maximum moment differs. This example assumes the moment magnification due to P-\(\Delta\) effects increases the positive and negative moment proportionally by the same amount.

B.5.1 Reinforcing steel centered in panel thickness—Assume 11 No. 6 bars \((A_s = 4.84 \text{ in.}^2)\) to satisfy maximum bar spacing limitations.

B.5.1.1 Load Case 1: \(1.2D + 1.6L_s + 0.5W\) (Fig. B.5.1.1)—Check vertical stress at the midheight section of the first-story panel segment per ACI 318-11, 14.8.2.6:

\[
6.25 \text{ in.} \cdot \gamma_s (15.0 \text{ ft}) (45.5 \text{ ft} - 7.9 \text{ ft}) \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 44.1 \text{ kip}
\]

\[
P_{\text{sw}} = 1.2(7.2 \text{ kip} + 2(17.7 \text{ kip}) + 44.1 \text{ kip}) + 1.6(7.5 \text{ kip}) = 116 \text{ kip}
\]
Panel weight at maximum positive moment:

\[
P_{w,m} = \frac{1.16 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft})} = 103 \text{ psi} < 0.06 f_\text{c}' = 240 \text{ psi}
\]

Panel weight at maximum negative moment:

\[
P_{w,m} = \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \cdot 12 \text{ in./ft} \cdot 1000 \text{ lb/kip} = 5.51 \text{ kip}
\]

Check design moment strength to compare to the maximum positive moment:

\[
P_{w,m} = 1.2(7.2 \text{ kip} + 5.51 \text{ kip}) + 1.6(7.5 \text{ kip}) = 27.3 \text{ kip}
\]

\[
A_w = A_c + \frac{P_{w,m}}{f_y} \left( \frac{h}{2d} \right)
\]

\[
a = A_w f_y = \frac{27.3 \text{ kip}}{60 \text{ ksi}} \left( \frac{6.25 \text{ in.}}{2(3.13 \text{ in.})} \right) = 5.29 \text{ in.}^2
\]

\[
c = \frac{0.85 f_y}{0.85} = \frac{0.85}{0.85} = 0.611 \text{ in.}
\]

\[
\frac{c}{d} = 0.195 < 0.375 : \text{ tension-controlled}
\]

(refer to commentary to ACI 318-11, 9.3.2.2)

\[
L_w = E_c A_w (d - c) + \frac{E_c c^2}{3} = 8.044(5.29)(3.13 - 0.611)^2 + \frac{2(5.29)(15.0 \text{ ft})(0.611)^2}{3} = 283 \text{ in.}^4
\]

Check design moment strength to compare to the maximum negative moment:

\[
P_{w,m} = 1.2(7.2 \text{ kip} + 2(17.7 \text{ kip}) + 34.8 \text{ kip}) + 1.6(7.5 \text{ kip}) = 105 \text{ kip}
\]

\[
A_w = A_c + \frac{P_{w,m}}{f_y} \left( \frac{h}{2d} \right)
\]

\[
a = A_w f_y = \frac{4.84 \text{ in.}^2}{60 \text{ ksi}} = \frac{4.84(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})}{2(3.13 \text{ in.})} = 6.59 \text{ in.}^2
\]

\[
c = \frac{0.85 f_y}{0.85} = \frac{0.85}{0.85} = 0.646 \text{ in.}
\]

\[
\frac{c}{d} = 0.243 < 0.375 : \text{ tension-controlled}
\]
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\[ I_c = \frac{E_c}{E_c} A_c (d - c)^2 + f_w c^3 \]

\[ = 8.044(6.59)(3.13 - 0.760)^2 + \frac{(15.0 \text{ ft})(12 \text{ in./ft})(0.760)^3}{3} \]

\[ = 323 \text{ in.}^3 \]

\[ M_{cr} = 46.3 \text{ ft-kip} \]

\[ \phi M_s = \alpha \frac{d - c}{2} \]

\[ = 0.9(6.59)(60)(3.13 - 0.760) \]

\[ = 997 \text{ in.-kip} = 83.1 \text{ ft-kip} \]

\[ \phi M_s > M_{cr} \]

From the above analysis for the positive moment case, the moment magnification term is:

\[ \frac{1}{1 - \frac{P_{sw}}{0.75K_b}} \cdot \frac{27.3 \text{ kip}}{1 - 0.75(332 \text{ kip})} = 1.12 \]

Therefore, the applied negative moment per ACI 318-11, 14.8.3, is:

\[ M_\alpha = 1.12(8.1 \text{ ft-kip}) = 9.07 \text{ ft-kip} < \phi M_s \]

\[ \Delta_\alpha = 0 \text{ in.} \]

**B.5.1.2 Load Case 2: 1.2D + 0.5L, + 1.0L + 1.0W (Fig. B.5.1.2)**—Check vertical stress at the midheight section of the first-story panel segment per ACI 318-11, 14.8.2.6:

\[ 6.25 \text{ in.} = \frac{44.1 \text{ kip}}{12 \text{ in./ft}} \]

\[ P_{sw} = 1.2(7.2 \text{ kip} + 2(17.7 \text{ kip}) + 44.1 \text{ kip}) + 0.5(7.5 \text{ kip}) + 1.0(30.0 \text{ kip}) = 168 \text{ kip} \]

\[ \frac{P_{sw}}{A_g} = \frac{168 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft})} = 149 \text{ psi} < 0.06f'_c = 240 \text{ psi} \]

Panel weight at maximum positive moment:

\[ 6.25 \text{ in.} = \frac{45.5 \text{ ft} - 7.1 \text{ ft}}{12 \text{ in./ft}} \cdot \frac{1 \text{ kip}}{1000 \text{ lb}} = 45.0 \text{ kip} \]

Panel weight at maximum negative moment:

\[ 6.25 \text{ in.} = \frac{45.5 \text{ ft} - 15.8 \text{ ft}}{12 \text{ in./ft}} \cdot \frac{1 \text{ kip}}{1000 \text{ lb}} = 34.8 \text{ kip} \]

Check design moment strength to compare to the maximum positive moment:

\[ P_{sw} = 169 \text{ kip} \]

\[ A_{ge} = 7.66 \text{ in.}^2 \]

\[ a = 0.751 \text{ in.} \]

\[ c = 0.884 \text{ in.} \]

\[ c/d = 0.282 \text{ in.} \cdot \text{tension-controlled} \]

**B.5.1.3 Load Case 3: 0.9D + 1.0W (Fig. B.5.1.3)**—Check vertical stress at the midheight section of the first-story panel segment per ACI 318-11, 14.8.2.6:

\[ 6.25 \text{ in.} = \frac{44.1 \text{ kip}}{12 \text{ in./ft}} \]

\[ P_{sw} = 0.9(7.2 \text{ kip} + 2(17.7 \text{ kip}) + 44.1 \text{ kip}) = 78.0 \text{ kip} \]

\[ \frac{P_{sw}}{A_g} = \frac{78.0 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft})} = 69.3 \text{ psi} < 0.06f'_c = 240 \text{ psi} \]

Panel weight at maximum positive moment:

\[ 6.25 \text{ in.} = \frac{45.5 \text{ ft} - 6.6 \text{ ft}}{12 \text{ in./ft}} \cdot \frac{1 \text{ kip}}{1000 \text{ lb}} = 45.6 \text{ kip} \]
Panel weight at maximum negative moment:
\[ \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \gamma_y (15.0 \text{ ft}) [45.5 \text{ ft} - 15.8 \text{ ft}] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 34.8 \text{ kip} \]

Check design moment strength to compare to the maximum positive moment:

\[ P_{um} = 79.4 \text{ kip} \]
\[ A_{se} = 6.16 \text{ in.}^2 \]
\[ a = 0.604 \text{ in.} \]
\[ c = 0.711 \text{ in.} \]
\[ c/d = 0.227 \]
\[ I_{cr} = 310 \text{ in.}^4 \]
\[ \phi_M = 78.3 \text{ ft-kip} > M_{cr} \]
\[ K_o = 298 \text{ kip} \]
\[ M_{se} = 9.0 \text{ ft-kip} \]
\[ M_n = 14.0 \text{ ft-kip} < \phi M_n \]
\[ \Delta_n = 0.751 \text{ in.} \]

Check design moment strength to compare to the maximum negative moment:

\[ P_{um} = 69.7 \text{ kip} \]
\[ A_{se} = 6.0 \text{ in.}^2 \]
\[ a = 0.588 \text{ in.} \]
\[ c = 0.692 \text{ in.} \]
\[ c/d = 0.221 \]
\[ I_{cr} = 306 \text{ in.}^4 \]
\[ M_{se} = 46.3 \text{ ft-kip} \]
\[ \phi M_n = 76.4 \text{ ft-kip} > M_{cr} \]
\[ M_n = 1.56(12.2 \text{ ft-kip}) = 19.0 \text{ ft-kip} < \phi M_n \]
\[ \Delta_n = 0.0 \text{ in.} \]

All in-service conditions should be considered before determining the final reinforcement. As this load case demonstrates, the governing positive moment effect could be at the third floor level where the effective area of steel due to the panel axial forces is much less.

Service load deflections between each support are limited by ACI 318-11, 14.8.4. Applied service load moments are obtained in the same manner as the factored moments mentioned previously. By inspection, the factored displacements are approximately equal to or less than the values permitted by the code, and it is safe to assume the service load deflections are acceptable.

\[ \Delta_{all} = \frac{f_y}{150} = \frac{15.8 \text{ ft}(12 \text{ in./ft})}{150} = 1.26 \text{ in.} \]
\[ = \frac{13.8 \text{ ft}(12 \text{ in./ft})}{150} = 1.10 \text{ in.} \]
\[ = \frac{14.3 \text{ ft}(12 \text{ in./ft})}{150} = 1.14 \text{ in.} \]

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3.

\[ A_s = 0.002 A_g = 0.002(6.25 \text{ in.})(45.5 \text{ ft})(12 \text{ in./ft}) = 6.8 \text{ in.}^2 \]

Therefore, 34 No. 4 bars should be used for horizontal reinforcement. Panel reinforcement details are not illustrated for this case as a reminder to check the panel for the influence lifting stresses and temporary construction conditions have on the vertical reinforcing steel requirements.

B.5.2 Reinforcing steel at each face of the panel—Assume 15 No. 4 bars per face \((A_s = 3.00 \text{ in.}^2 \text{ per face})\) to meet minimum reinforcing steel requirements.

B.5.2.1 Load Case I: \(1.2D + 1.6L + 0.5W\)

Check vertical stress at the midheight section of the first-story panel segment:

\[ 6.25 \text{ in.} \gamma_y (15.0 \text{ ft}) [45.5 \text{ ft} - 7.9 \text{ ft}] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 44.1 \text{ kip} \]

\[ P_{um} = 116 \text{ kip} \]
\[ P_{um} = 103 \text{ psi} < 0.06 f'_c = 240 \text{ psi} \]

The bending moment diagram from lateral and eccentric vertical loads is depicted in Fig. B.5.11. Panel weight at maximum positive moment is 5.51 kip. Panel weight at maximum negative moment is 34.8 kip.

Check design moment strength to compare to the maximum positive moment:

\[ P_{um} = 1.2(7.2 \text{ kip} + 5.51 \text{ kip}) + 1.6(7.5 \text{ kip}) = 27.3 \text{ kip} \]
\[ A_s = A_e + \frac{P_{um}}{f'_e} \left( \frac{h}{2d} \right) = 3.00 \text{ in.}^2 + \frac{27.3 \text{ kip}}{60 \text{ ksi}} \left( \frac{6.25 \text{ in.}}{2(5.00 \text{ in.})} \right) = 3.28 \text{ in.}^2 \]
\[ a = \frac{A_e f'_e}{0.85 f'_b} = \frac{3.28 \text{ in.}^2(60 \text{ ksi})}{0.85(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.322 \text{ in.} \]
\[
\begin{align*}
c &= \frac{a}{0.85} = \frac{0.322}{0.85} = 0.379 \text{ in.} \\
\frac{c}{d} &= 0.076 < 0.375 \therefore \text{tension-controlled} \\
(\text{refer to the commentary of ACI 318-11, 9.3.2.2}) \\
I_{cr} &= \frac{E_I}{E_s} A_{cr} (d - c)^2 + \frac{f_y c^3}{3} \\
&= 8.044(3.28)(5.0 - 0.379)^2 + (15.0 \text{ ft})(12 \text{ in./ft})(0.379)^3 \\
&= 567 \text{ in.}^4 \\
M_{cr} &= 46.3 \text{ ft-kip} \\
\phi M_s &= A_{cr} f_y \left(\frac{d - c}{2}\right) \\
&= 0.9(3.28)(60) \left(5.0 - \frac{0.322}{2}\right) = 858 \text{ in.-kip} = 71.5 \text{ ft-kip} \\
\phi M_s &\geq M_{cr} \\
\text{Check minimum reinforcement required by ACI 318-11, 14.8.2.4:} \\
\phi M_s &\geq M_{cr}
\end{align*}
\]

Check minimum reinforcement required by ACI 318-11, 14.3.2:

\[
\rho = \frac{A_s}{bh} = \frac{3.0 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00267 > \rho_c = 0.0012
\]

Check applied moment per ACI 318-11, 14.8.3:

\[
K_s = \frac{48 E_s I_{cr}}{5 f_y^2} = \frac{48(3605 \text{ ksi})(567 \text{ in.}^4)}{5[143 \text{ ft}(12 \text{ in./ft})]^2} = 667 \text{ kip} \\
M_{max} = 5.9 \text{ ft-kip} \\
M_s = \frac{M_{max}}{\phi M_s} = \frac{5.9 \text{ ft-kip}}{6.24 \text{ ft-kip} < \phi M_s} = 0.95 \text{ ft-kip} < \phi M_s \\
\Delta_s = \frac{M_s}{0.75 K_s} = \frac{6.24 \text{ ft-kip}(12 \text{ in./ft})}{0.75(667 \text{ kip})} = 0.150 \text{ in.}
\]

Check design moment strength to compare to the maximum negative moment:

\[
P_{\text{prev}} = 1.2[7.2 \text{ kip} + 2(17.7 \text{ kip}) + 348 \text{ kip}] + 1.6(7.5 \text{ kip}) = 105 \text{ kip} \\
A_{cr} = A_s + \frac{P_{\text{prev}} (h)}{f_y (2d)} = 3.00 \text{ in.}^2 + \frac{105 \text{ kip}}{60 \text{ ksi}} \left(\frac{6.25 \text{ in.}^2}{2(5.00 \text{ in.})}\right) = 4.09 \text{ in.}^2 \\
a = \frac{A_{cr} f_y}{0.85 f_y b} = \frac{4.09 \text{ in.}^2(60 \text{ ksi})}{0.85(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.401 \text{ in.}
\]

Therefore, the applied negative moment per ACI 318-11, 14.8.3, is:

\[
M_s = 1.06(8.1 \text{ ft-kip}) = 8.59 \text{ ft-kip} < \phi M_s \\
\Delta_s = 0 \text{ in.}
\]

**B.5.2.2 Load Case 2: 1.2D + 0.5L_x + 1.0L + 1.0W**

\[
P_{\text{prev}} = 168 \text{ kip} \\
A_{cr} = 4.76 \text{ in.}^2 \\
c = 0.467 \text{ in.} \\
c = 0.549 \text{ in.} \\
c/d = 0.110 \therefore \text{tension-controlled} \\
I_{cr} = 768 \text{ in.}^4 \\
M_{cr} = 46.3 \text{ ft-kip} \\
\phi M_s = 102 \text{ ft-kip} > M_{cr} \\
K_s = 737 \text{ kip} \\
M_{max} = 10.2 \text{ ft-kip} \\
M_s = 14.7 \text{ ft-kip} < \phi M_s \\
\]

The bending moment diagram from lateral and eccentric vertical loads is depicted in Fig. B.5.1.2. Panel weight at maximum positive moment is 45.0 kip. Panel weight at maximum negative moment is 34.8 kip.

Check design moment strength to compare to the maximum positive moment:

\[
P_{\text{prev}} = 169 \text{ kip} \\
A_{cr} = 4.76 \text{ in.}^2 \\
c = 0.467 \text{ in.} \\
c = 0.549 \text{ in.} \\
c/d = 0.110 \therefore \text{tension-controlled} \\
I_{cr} = 768 \text{ in.}^4 \\
M_{cr} = 46.3 \text{ ft-kip} \\
\phi M_s = 102 \text{ ft-kip} > M_{cr} \\
K_s = 737 \text{ kip} \\
M_{max} = 10.2 \text{ ft-kip} \\
M_s = 14.7 \text{ ft-kip} < \phi M_s \\
\]
Check design moment strength to compare to the maximum negative moment:

\[
P_{um} = 157 \text{ kip} \\
A_{ce} = 4.63 \text{ in.}^2 \\
a = 0.454 \text{ in.} \\
c = 0.534 \text{ in.} \\
c/d = 0.107 \quad \text{tension-controlled} \\
I_{ce} = 752 \text{ in.}^4 \\
M_{ce} = 46.3 \text{ ft-kip} \\
\phi M_{ce} > M_{cr} \\
M_n = 1.44(18.3 \text{ ft-kip}) = 26.4 \text{ ft-kip} < \phi M_n \\
\Delta_n = 0 \text{ in.} \\
\Delta = 0.319 \text{ in.}
\]

Check design moment strength to compare to the maximum positive moment:

\[
P_{um} = 157 \text{ kip} \\
A_{ce} = 4.63 \text{ in.}^2 \\
a = 0.454 \text{ in.} \\
c = 0.534 \text{ in.} \\
c/d = 0.107 \quad \text{tension-controlled} \\
I_{ce} = 752 \text{ in.}^4 \\
M_{ce} = 46.3 \text{ ft-kip} \\
\phi M_{ce} > M_{cr} \\
M_n = 1.44(18.3 \text{ ft-kip}) = 26.4 \text{ ft-kip} < \phi M_n \\
\Delta_n = 0 \text{ in.}
\]

Panel reinforcement details are not illustrated for this case as a reminder to check the panel for the influence lifting stresses and temporary construction conditions have on vertical reinforcing steel requirements.

**B.6—Panel with dock-high condition design example**

Similar to multistory tilt-up panel design, unique challenges not encountered in the previous examples are associated with panels at a dock-high condition. Stresses on the panel during lifting and temporary construction conditions where the roof, floor slab, or both, are not attached, should be investigated for the influence on required vertical reinforcing steel. The designer should carefully select which vertical loads will be present in the temporary condition, and what, if any, reduction can be taken on the applied lateral load.

The type of analysis conducted should be carefully considered. Commonly, the panel depicted in Fig. B.6 is analyzed with pinned conditions at the support locations (in this case, joist bearing and floor slab), thereby neglecting the continuity of the lower portion. With this set of simplifying assumptions, the analysis would be similar to the example presented in B.1 and is not repeated herein.

A more rigorous analysis would consider the panel continuity, demonstrated in the following example for the final, in-service condition only. Figure B.6 is a cross section of the sample dock-high panel with no openings. A summary of the applied loading is:

\[
P_0 = 3(2.4 \text{ kip}) = 7.2 \text{ kip} \\
P_{dv} = 3(2.5 \text{ kip}) = 7.5 \text{ kip} \\
e_{mov} = 3 \text{ in. (assumed)} \\
w = 27.2 \text{ lb/ft}^2 \\
\gamma_f = 40 \text{ lb/ft}^3 \\
q = 50 \text{ lb/ft}^3
\]

where the roof framing members are assumed to bear in wall pockets, \( \gamma_f \) is the equivalent fluid pressure of the retained soil, and \( q \) is the surcharge on the floor slab. Due to panel continuity, the diaphragm deflection will also contribute to the negative moment. For this example, a 0.63 in. displacement of the diaphragm is considered.

It is important to note that a panel designed for continuity exerts larger reactions at the floor slab and foundation. The reaction at the floor slab is approximately 225 percent of the reaction for a standard pinned-pinned condition with the configuration shown in Fig. B.6. The reaction at the top of

\[
\Delta = \frac{\ell}{150} = \frac{15.8 \text{ ft}(12 \text{ in./ft})}{150} = 1.26 \text{ in.} \\
\Delta = \frac{13.8 \text{ ft}(12 \text{ in./ft})}{150} = 1.10 \text{ in.} \\
\Delta = \frac{14.3 \text{ ft}(12 \text{ in./ft})}{150} = 1.14 \text{ in.}
\]

Horizontal reinforcing steel requirements should consider a maximum bar spacing of 18 in.
Fig. B.6—Dock-high tilt-up panel with no openings.

the footing is approximately 640 percent of the reaction for a standard pinned-pinned condition with the configuration shown in Fig. B.6. If these forces are not properly developed, a full fixity condition for the continuous span would not occur at the floor slab tie-back, and the analysis would not be valid and, possibly, yield less panel reinforcement than required. The design of the slab tie-back and foundation are not presented herein because neither impact the vertical reinforcement design of the tilt-up panel.

Bending moment diagrams for the two-span continuous panel are shown for each load case considered based on the applied loads without considering the P-Δ effect. The weight of the tilt-up panel above the design section is calculated individually for each load case because the location of the maximum moment differs. This example assumes the moment magnification due to P-Δ effects increases the positive and negative moment proportionally by the same amount.

B.6.1 Reinforcing steel centered in panel thickness—Assume 11 No. 6 bars (A_s = 4.84 in.^2) to satisfy maximum bar spacing limitations.

B.6.1.1 Load Case 1: 1.2D + 1.6L + 0.5W (Fig. B.6.1.1)—Check vertical stress at the midheight section of the positive moment panel segment per ACI 318-11, 14.8.2.6.

\[
\frac{6.25 \text{ in}}{12 \text{ in./ft}} \gamma_c (15.0 \text{ ft}) \left[ \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 19.0 \text{ kip}
\]

\[
P_{mn} = 1.2(7.2 \text{ kip} + 19.0 \text{ kip}) + 1.6(7.5 \text{ kip}) = 43.4 \text{ kip}
\]

\[
\rho = \frac{A_s}{bh} = \frac{4.84 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00430 > \rho_t = 0.0015
\]

Panel weight at maximum positive moment:

\[
\frac{6.25 \text{ in}}{12 \text{ in./ft}} \gamma_c (15.0 \text{ ft}) \left[ 36.0 \text{ ft} - 5.0 \text{ ft} - 24.0 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 8.2 \text{ kip}
\]

Panel weight at maximum negative moment:

\[
\frac{6.25 \text{ in}}{12 \text{ in./ft}} \gamma_c (15.0 \text{ ft}) \left[ 36.0 \text{ ft} - 5.0 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 36.3 \text{ kip}
\]

Check design moment strength to compare to the maximum positive moment:

\[
P_{mn} = 1.2(7.2 \text{ kip} + 8.2 \text{ kip}) + 1.6(7.5 \text{ kip}) = 30.5 \text{ kip}
\]

\[
A_s = A_t + \frac{P_{mn}}{f_y} \left( \frac{h}{2d} \right)
\]

\[
= 4.84 \text{ in.}^2 + 30.5 \text{ kip} \left( \frac{6.25 \text{ in.}}{60 \text{ ksf}} \right) = 5.35 \text{ in.}^2
\]

\[
a = \frac{A_s f_y}{0.85 f_y} = \frac{5.35 \text{ in.}^2(60 \text{ ksf})(15.0 \text{ ft})(12 \text{ in./ft})}{0.85(4\text{ ksf})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.525 \text{ in.}
\]

\[
c = 0.197 < 0.375 \therefore \text{tension-controlled}
\]

(Refer to the commentary of ACI 318-11, 9.3.2.2)

\[
I_y = \frac{E_s}{E_c} A_s (d - c)^3 + \frac{f_y c^3}{3} = 8.04(5.35)(3.13 - 0.618)^3 + (15.0 \text{ ft})(12 \text{ in./ft})(0.618)^3 = 285 \text{ in.}^4
\]

\[
M_{cr} = \frac{f_y I_y}{\gamma_c} = f_y S = f_y \left( \frac{1}{6} bh^2 \right) = 0.474 \text{ ksf} \left( \frac{1}{6} \right) (15.0 \text{ ft})(6.25 \text{ in.})^2 = 46.3 \text{ ft-kip}
\]

\[
\phi M_e = \phi A_s f_y \left( d - \frac{a}{2} \right) = 0.9(5.35)(60) \left( 3.13 - \frac{0.525}{2} \right) = 827 \text{ in.-kip} = 68.9 \text{ ft-kip}
\]

Check minimum reinforcement required by ACI 318-11, 14.8.2.4:

\[
\phi M_e \geq M_{cr}
\]

Check minimum reinforcement required by ACI 318-11, 14.3.2:

\[
\rho = \frac{A_s}{bh} = \frac{4.84 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00430 > \rho_t = 0.0015
\]
Check applied moment per ACI 318-11, 14.8.3:

\[ K_p = \frac{48E}{5}\frac{I_{ce}}{c^2} = \frac{48(3605 \text{ ksi})(285 \text{ in.}^4)}{29.5 \text{ ft}(12 \text{ in./ft})^2} = 78.6 \text{ kip} \]

\[ M_{uw} = 16.1 \text{ ft-kip} \]

\[ M_a = \frac{M_{uw}}{1 - \frac{P_{w}}{0.75K_p}} = \frac{16.1 \text{ ft-kip}}{1 - \frac{30.5 \text{ kip}}{0.75(78.6 \text{ kip})}} = 33.3 \text{ ft-kip} < \phi M_o \]

\[ \Delta_a = \frac{M_a}{0.75K_p} = \frac{33.3 \text{ ft-kip}(12 \text{ in./ft})}{0.75(78.6 \text{ kip})} = 6.78 \text{ in.} \]

Check design moment strength to compare to the maximum negative moment:

\[ P_{uw} = 1.2[7.2 \text{ kip} + 36.3 \text{ kip}] + 1.6(7.5 \text{ kip}) = 64.2 \text{ kip} \]

\[ A_a = A + \frac{P_{uw}}{f_y} \left( \frac{h}{2d} \right) = \frac{4.84 \text{ in.}^2 + 64.2 \text{ kip}(60 \text{ ksi})}{2.13 \text{ in.}} = 5.91 \text{ in.}^2 \]

\[ a = \frac{A_f f_y}{0.85f_y b} = \frac{5.91 \text{ in.}^2(60 \text{ ksi})}{0.85(4 \text{ ksi})(15.0 \text{ ft})(12 \text{ in./ft})} = 0.579 \text{ in.} \]

\[ c = \frac{a}{d} = \frac{0.579}{0.85} = 0.681 \text{ in.} \]

\[ \frac{c}{d} = 0.218 < 0.375 \text{; tension-controlled} \]

\[ I_{ce} = E_s A_s (d-c)^2 + bc^3 \]

\[ = 8.04(5.91)(3.13-0.681)^2 + (15.0 \text{ ft})(12 \text{ in./ft})(0.681)^3 \]

\[ = 303 \text{ in.}^4 \]

\[ M_{cr} = 46.3 \text{ ft-kip} \]

Using 0.35\( f_y \) for the moment of inertia of the panel as outlined in ACI 318-11, 10.10.4.1, a 0.63 in. displacement of the diaphragm would result in an additional 5.8 ft-kip of moment. From the previous analysis for the positive moment case, the moment magnification term is

\[ \psi = 2.06(21.2 \text{ ft-kip} + 5.8 \text{ ft-kip}) = 55.6 \text{ ft-kip} < M_o \]

Therefore, the applied negative moment per ACI 318-11, 14.8.3 would be:

\[ M_o = 2.06(21.2 \text{ ft-kip} + 5.8 \text{ ft-kip}) = 55.6 \text{ ft-kip} < \phi M_o \]

\[ \Delta_o = 0 \text{ in.} \]

**B.6.1.2 Load Case 2: 1.2D + 0.5Lw + 1.0W (Fig. B.6.1.2)**

Check vertical stress at the midheight section of the positive moment panel segment per ACI 318-11, 14.8.2.6:

\[ 6.25 \text{ in.}/\text{ft} \gamma_s (15.0 \text{ ft}) \left( \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \right) \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 19.0 \text{ kip} \]

\[ P_{uw} = 1.2[7.2 \text{ kip} + 19.0 \text{ kip}] + 1.6(7.5 \text{ kip}) = 35.2 \text{ kip} \]

\[ P_{uw} = 35.2 \text{ kip}(100 \text{ lb/kip}) \]

\[ A_y = 6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft}) \]

Panel weight at maximum positive moment:

\[ 6.25 \text{ in.} + \gamma_s (15.0 \text{ ft}) [36.0 \text{ ft} - 5.0 \text{ ft} - 23.3 \text{ ft}] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 9.02 \text{ kip} \]

Panel weight at maximum negative moment:
Check design moment strength to compare to the maximum positive moment:

\[ P_{um} = 23.2 \text{ kip} \]
\[ A_{ce} = 5.23 \text{ in.}^2 \]
\[ a = 0.513 \text{ in.} \]
\[ c = 0.604 \text{ in.} \]
\[ c/d = 0.193 \cdot \text{tension-controlled} \]
\[ I_{ce} = 281 \text{ in.}^4 \]
\[ M_{cr} = 46.3 \text{ ft-kip} \]
\[ \phi M_n = 67.5 \text{ ft-kip} > M_{cr} \]
\[ K_b = 77.5 \text{ k} \]
\[ M_{aw} = 28.4 \text{ ft-kip} \]
\[ \phi M_n = 47.3 \text{ ft-kip} < \phi M_n \]
\[ \Delta_n = 9.76 \text{ in.} \]

Check design moment strength to compare to the maximum negative moment:

\[ P_{um} = 56.0 \text{ kip} \]
\[ A_{ce} = 5.77 \text{ in.}^2 \]
\[ a = 0.566 \text{ in.} \]
\[ c = 0.666 \text{ in.} \]
\[ c/d = 0.213 \cdot \text{tension-controlled} \]
\[ I_{ce} = 299 \text{ in.}^4 \]
\[ M_{cr} = 46.3 \text{ ft-kip} \]
\[ \phi M_n = 73.8 \text{ ft-kip} > M_{cr} \]
\[ K_b = 76.1 \text{ k} \]
\[ M_{aw} = 27.5 \text{ ft-kip} \]
\[ \phi M_n = 37.0 \text{ ft-kip} < \phi M_n \]
\[ \Delta_n = 7.78 \text{ in.} \]

### B.6.1.3 Load Case 3: 0.9D + 1.0W + 1.6H (Fig. B.6.1.3)—

Check vertical stress at the midheight section of the positive moment panel segment per ACI 318-11, 14.8.2.6.

\[ \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \times \gamma_c(15.0 \text{ ft}) \left[ \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 19.0 \text{ kip} \]

\[ P_{um} = 0.9(7.2 \text{ kip} + 19.0 \text{ kip}) = 23.6 \text{ kip} \]

\[ \frac{P_{um}}{A_g} = \frac{23.6 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(15.0 \text{ ft})(12 \text{ in./ft})} = 21.0 \text{ psi} < 0.06 f'_{c'} = 240 \text{ psi} \]

Panel weight at maximum positive moment:

\[ \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \times \gamma_c(15.0 \text{ ft}) \left[ 36.0 \text{ ft} - 5.0 \text{ ft} - 23.2 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 9.14 \text{ kip} \]

Panel weight at maximum negative moment:

\[ \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \times \gamma_c(15.0 \text{ ft}) \left[ 36.0 \text{ ft} - 5.0 \text{ ft} \right] \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 36.3 \text{ kip} \]

Check design moment strength to compare to the maximum positive moment:

\[ P_{um} = 14.7 \text{ kip} \]
\[ A_{ce} = 5.09 \text{ in.}^2 \]
\[ a = 0.499 \text{ in.} \]
\[ c = 0.587 \text{ in.} \]
\[ c/d = 0.187 \cdot \text{tension-controlled} \]
\[ I_{ce} = 276 \text{ in.}^4 \]
\[ M_{cr} = 46.3 \text{ ft-kip} \]
\[ \phi M_n = 65.8 \text{ ft-kip} > M_{cr} \]
\[ K_b = 76.1 \text{ k} \]
\[ M_{aw} = 27.5 \text{ ft-kip} \]
\[ \phi M_n = 56.0 \text{ ft-kip} < \phi M_n \]
\[ \Delta_n = 0 \text{ in.} \]

Check design moment strength to compare to the maximum negative moment:

\[ P_{um} = 39.2 \text{ kip} \]
\[ A_{ce} = 5.49 \text{ in.}^2 \]
\[ a = 0.538 \text{ in.} \]
\[ c = 0.633 \text{ in.} \]
\[ c/d = 0.202 \cdot \text{tension-controlled} \]
\[ I_{ce} = 290 \text{ in.}^4 \]
\[ M_{cr} = 46.3 \text{ ft-kip} \]
\[ \phi M_n = 70.6 \text{ ft-kip} > M_{cr} \]
\[ M_{aw} = 1.35(46.0 \text{ ft-kip}) = 62.1 \text{ ft-kip} < \phi M_n \]
\[ \Delta_n = 0 \text{ in.} \]

Service load deflections between each support are limited by ACI 318-11, 14.8.4, and the calculation follows similarly to the previous panel design examples using an effective moment of inertia based on a cracked section of concrete.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3

\[ A_s = 0.002 A_g = 0.002(6.25 \text{ in.})(36.0 \text{ ft})(12 \text{ in./ft}) = 5.4 \text{ in.}^2 \]
Therefore, 27 No. 4 bars should be used for the horizontal reinforcement. Panel reinforcement details are not illustrated for this case as a reminder to check the panel for the influence lifting stresses and temporary construction conditions have on vertical reinforcing steel requirements.

**B.6.2 Reinforcing steel at each face of the panel—**
Assume 15 No. 4 bars per face \((A_s = 3.0\text{ in.}^2\text{ per face})\) to meet minimum reinforcing steel requirements.

**B.6.2.1 Load Case 1: 1.2D + 1.6L + 0.5W—**The vertical stress at the midheight section of the positive moment panel segment was checked in B.6.1.1.

The bending moment diagram from lateral and eccentric vertical loads is depicted in Fig. B.6.1.1. Panel weight at maximum positive moment is 8.2 kip. Panel weight at maximum negative moment is 36.3 kip.

Check design moment strength to compare to the maximum positive moment:

\[
P_{um} = 30.5 \text{ kip} \\
A_{se} = 3.32 \text{ in.}^2 \\
a = 0.325 \text{ in.} \\
c = 0.383 \text{ in.} \\
c/d = 0.077 < 0.375 \\
\Rightarrow \text{ tension-controlled (refer to ACI 318-11, R9.3.2.2)} \\
I_{et} = 572 \text{ in.}^4 \\
M_{cr} = 46.3 \text{ ft-kip} \\
\phi M_u = 72.2 \text{ ft-kip} > M_{cr}
\]

Check minimum reinforcement required by ACI 318-11, 14.8.2.4:

\[
\phi M_u \geq M_{cr}
\]

Check minimum reinforcement required by ACI 318-11, 14.3.2:

\[
\rho = \frac{A_s}{bh} = \frac{3.0 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00267 > \rho_r = 0.0012
\]

Check applied moment per ACI 318-11, 14.8.3:

\[
K_0 = 158 \text{ kip} \\
M_{aw} = 16.1 \text{ ft-kip} \\
M_a = 21.7 \text{ ft-kip} < \phi M_u \\
\Delta_a = 2.19 \text{ in.}
\]

Check design moment strength to compare to the maximum negative moment:

\[
P_{um} = 23.2 \text{ kip} \\
A_{se} = 3.24 \text{ in.}^2 \\
a = 0.318 \text{ in.} \\
c = 0.374 \text{ in.} \\
c/d = 0.075 \Rightarrow \text{ tension-controlled} \\
I_{et} = 561 \text{ in.}^4 \\
M_{cr} = 46.3 \text{ ft-kip} \\
\phi M_u = 70.6 \text{ ft-kip} > M_{cr} \\
K_b = 155 \text{ kip} \\
M_{aw} = 28.4 \text{ ft-kip} \\
M_a = 35.5 \text{ ft-kip} < \phi M_u \\
\Delta_a = 3.66 \text{ in.}
\]

The applied moment is (including the 5.8 ft-kip additional moment due to the 0.63 in. displacement of the diaphragm):

\[
M_a = 1.35(21.2 \text{ ft-kip} + 5.8 \text{ ft-kip}) = 36.5 \text{ ft-kip} < \phi M_u \\
\Delta_a = 0 \text{ in.}
\]

where 1.35 is the moment magnification term from the positive moment analysis.

**B.6.2.2 Load Case 2: 1.2D + 0.5L + 1.0W—**Vertical stress at the midheight section of the positive moment panel segment was checked in B.6.1.2.

The bending moment diagram from lateral and eccentric vertical loads is depicted in Fig. B.6.1.2. Panel weight at maximum positive moment is 9.02 kip. Panel weight at maximum negative moment is 36.3 kip.

Check design moment strength to compare to the maximum positive moment:

\[
P_{um} = 56.0 \text{ kip} \\
A_{se} = 3.58 \text{ in.}^2 \\
a = 0.351 \text{ in.} \\
c = 0.413 \text{ in.} \\
c/d = 0.083 \Rightarrow \text{ tension-controlled} \\
I_{et} = 611 \text{ in.}^4 \\
M_{cr} = 46.3 \text{ ft-kip} \\
\phi M_u = 77.8 \text{ ft-kip} > M_{cr} \\
K_b = 155 \text{ kip} \\
M_{aw} = 35.5 \text{ ft-kip} < \phi M_u \\
\Delta_a = 3.66 \text{ in.}
\]

Check design moment strength to compare to the maximum negative moment:

\[
P_{um} = 56.0 \text{ kip} \\
A_{se} = 3.58 \text{ in.}^2 \\
a = 0.351 \text{ in.} \\
c = 0.413 \text{ in.} \\
c/d = 0.083 \Rightarrow \text{ tension-controlled} \\
I_{et} = 611 \text{ in.}^4 \\
M_{cr} = 46.3 \text{ ft-kip} \\
\phi M_u = 77.8 \text{ ft-kip} > M_{cr} \\
K_b = 155 \text{ kip} \\
M_{aw} = 35.5 \text{ ft-kip} < \phi M_u \\
\Delta_a = 3.66 \text{ in.}
\]

**B.6.2.3 Load Case 3: 0.9D + 1.0W + 1.6H—**Vertical stress at the midheight section of the positive moment panel segment was checked in B.6.1.3.

The bending moment diagram from lateral and eccentric vertical loads is depicted in Fig. B.6.1.3. Panel weight at maximum positive moment is 9.14 kip. Panel weight at maximum negative moment is 36.3 kip.

Check design moment strength to compare to the maximum positive moment:

\[
P_{um} = 14.7 \text{ kip}
\]
Check design moment strength to compare to the maximum negative moment:

\[ P_{\text{sum}} = 39.2 \text{ kip} \]
\[ A_{ce} = 3.41 \text{ in.}^2 \]
\[ a = 0.334 \text{ in.} \]
\[ c = 0.393 \text{ in.} \]
\[ c/d = 0.079 : tension-controlled \]
\[ I_p = 585 \text{ in.}^4 \]
\[ M_{cr} = 46.3 \text{ ft-kip} \]
\[ \phi M_a = 68.8 \text{ ft-kip} > M_{cr} \]
\[ K_b = 191 \text{ kip} \]
\[ M_{na} = 27.5 \text{ ft-kip} \]
\[ M_a = 31.6 \text{ ft-kip} < \phi M_a \]
\[ \Delta_a = 3.34 \text{ in.} \]

Horizontal reinforcing steel requirements should consider a maximum bar spacing of 18 in.

Panel reinforcement details are not illustrated for this case as a reminder to check the panel for the influence that both lifting stresses and temporary construction conditions have on the vertical reinforcing steel requirements.

**B.7—Plain panel with fixed end design example**

Figure B.7 illustrates the geometry of the sample panel. Because the panel is typical of a screen wall or firewall application and is often designed to be free-standing, the base is a fixed support condition. The only external load on the tilt-up panel is from the wind (lateral force). A summary of the applied loading is:

\[ P_{OL} = 0.0 \text{ kip} \]
\[ P_{LT} = 0.0 \text{ kip} \]
\[ e_{ce} = \text{none} \]
\[ w = 16 \text{ lb/ft}^2 \]

The critical section for analysis (point of maximum moment) will obviously occur at the fixed base. Therefore, weight of the tilt-up panel is:

\[
\left( \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \right) \times 150 \text{ lb/ft}^2 (15.0 \text{ ft})(31.0 \text{ ft}) \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 36.3 \text{ kip}
\]

**Fig. B.7—Free-standing tilt-up panel with no openings.**

ACI 318-11, 14.8, can be used to design reinforcement for this panel by recognizing the moment at the support of a cantilevered span of height \( a \) is exactly the same as the midheight moment of a simply supported span of height \( \ell_c \) if the simply supported span is twice that of the cantilevered span, as demonstrated in the following:

\[
K_{bss} = K_b^c \\
M_{ss}^{m} = M_{c}^{c} \\
\frac{w_{s} \ell_{c}^2}{8} = \frac{w_{s} a^2}{2} \\
\ell_{c}^2 = 4a^2 \\
\ell_c = 2a 
\]

where the superscript \( ss \) denotes simply supported, and \( c \) denotes cantilevered.

The deflection calculation, however, would be more conservative than necessary using this approach. Because the deflection is a function of panel stiffness, a similar procedure to the aforementioned can be used to solve for the length of an equivalent simply supported span to match the stiffness value of the cantilevered panel and adhere to the provisions of ACI 318-11, 14.8.2.1.

\[
K_{bss} = K_b^c \\
M_{ss}^{m} = M_{c}^{c} \\
\frac{w_{s} \ell_{c}^2}{8} = \frac{w_{s} a^2}{2} \\
\ell_{c}^2 = 4a^2 \\
\ell_c = 2a 
\]
62 DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

Often, this type of panel does not need to meet the serviceability criteria inherent in the ACI procedure, so the designer may use discretion as to the appropriate modification to panel stiffness based on an equivalent simply supported span model.

B.7.1 Reinforcing steel centered in panel thickness—Assume 53 No. 6 bars (A_6 = 23.3 in.²)

B.7.1.1 Load Case 1: 1.2D + 0.5W

\[ P_{\text{ew}} = 1.2(36.3) = 43.6 \text{ kip} \]

\[ W_1 = 0.5(15.0 \text{ ft})(16 \text{ lb/ft}^2) = 120 \text{ plf} = 0.120 \text{ klf} \]

Check vertical stress at the section of maximum moment in the panel per ACI 318-11, 14.8.2.6:

\[ \sigma = \frac{P_{\text{ew}}(1000 \text{ lb/kip})}{A_s} = \frac{63.8 \text{ psi} < 0.06f'_c = 240 \text{ psi}}{23.3 \text{ in.}^2} \]

Check design moment strength:

\[ A_{\phi} = A_s + \frac{P_{\text{ew}}(h)}{f_\phi} = 23.3 \text{ in.}^2 + \frac{43.6 \text{ kip}}{60 \text{ ksi}} \left( \frac{6.25 \text{ in.}}{2(3.13 \text{ in.})} \right) = 24.0 \text{ in.}^2 \]

\[ a = \frac{A_{\phi}f_\phi}{0.85f_b} = \frac{24.0 \text{ in.}^2(60 \text{ ksi})}{0.85(4 \text{ klf})(15.0 \text{ ft})(12 \text{ in./ft})} = 2.35 \text{ in.} \]

\[ c = \frac{a}{0.85} = \frac{2.35}{0.85} = 2.77 \text{ in.} \]

\[ \frac{c}{d} = 0.887 > 0.375 \] (refer to the commentary of ACI 318-11, 9.3.2.2)

Therefore, the requirement of ACI 318-11, 14.8.2.3, that the section be tension-controlled is not met. At this point, there are two options: 1) provide a double layer of reinforcement as demonstrated in this example; or 2) increase wall panel thickness and repeat the analysis. To have a tension-controlled section for analysis, select a panel thickness of 9.75 in. The distance \( d \) (using a 4.0 in. chair) is calculated to be 4.88 in.

Assume 32 No. 6 bars \((A_6 = 14.1 \text{ in.}^2)\). The weight of the tilt-up panel above the base is 56.7 kip.

\[ P_{\text{ew}} = 1.2(56.7 \text{ kip}) = 68.0 \text{ kip} \]

\[ 0.887 > 0.375 \] (refer to the commentary of ACI 318-11, 9.3.2.2)

Therefore, the requirement of ACI 318-11, 14.8.2.3, that the section be tension-controlled is not met. At this point, there are two options: 1) provide a double layer of reinforcement as demonstrated in this example; or 2) increase wall panel thickness and repeat the analysis. To have a tension-controlled section for analysis, select a panel thickness of 9.75 in. The distance \( d \) (using a 4.0 in. chair) is calculated to be 4.88 in.

Assume 32 No. 6 bars \((A_6 = 14.1 \text{ in.}^2)\). The weight of the tilt-up panel above the base is 56.7 kip.

\[ P_{\text{ew}} = 1.2(56.7 \text{ kip}) = 68.0 \text{ kip} \]
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

\[
P_{u} = 0.0 \text{ kip}
\]
\[
P_{u} = 68.0 \text{ kip}
\]
\[
\omega_{u} = 1.0(15.0 \text{ ft})(16 \text{ lb/ft}^2) = 240 \text{ plf} = 0.240 \text{ klf}
\]
\[
P_{u}/A_g = 38.8 \text{ psi} < 0.06f' = 240 \text{ psi}
\]
\[
A_{ge} = 15.2 \text{ in.}^2
\]
\[
a = 1.49 \text{ in.}
\]
\[
c = 1.76 \text{ in.}
\]
\[
c/d = 0.360; \text{ tension-controlled}
\]
\[
I_{p} = 1518 \text{ in.}^4
\]
\[
M_{cr} = 113 \text{ ft-kip}
\]
\[
\phi M_{u} = 283 \text{ ft-kip} > M_{cr}
\]
\[
K_{s} = 158 \text{ kip}
\]
\[
M_{u} = 115 \text{ ft-kip}
\]
\[
M_{u} = 27.3 \text{ in.}
\]

**B.7.1.3** Load Case 3: 0.9D + 1.0W

\[
P_{u} = 0.0 \text{ kip}
\]
\[
P_{u} = 51.0 \text{ kip}
\]
\[
\omega_{u} = 240 \text{ plf} = 0.240 \text{ klf}
\]
\[
P_{u}/A_g = 29.1 \text{ psi} < 0.06f' = 240 \text{ psi}
\]
\[
A_{ge} = 15.0 \text{ in.}^2
\]
\[
a = 1.47 \text{ in.}
\]
\[
c = 1.72 \text{ in.}
\]
\[
c/d = 0.354; \text{ tension-controlled}
\]
\[
I_{p} = 1501 \text{ in.}^4
\]
\[
M_{cr} = 113 \text{ ft-kip}
\]
\[
\phi M_{u} = 279 \text{ ft-kip} > M_{cr}
\]
\[
K_{s} = 157 \text{ kip}
\]
\[
M_{u} = 115 \text{ ft-kip}
\]
\[
M_{u} = 204 \text{ ft-kip} < \phi M_{u}
\]
\[
\Delta_{u} = 20.8 \text{ in.}
\]

For illustrative purposes, check service load deflection per ACI 318-11, R14.8.4, with 1.0D + 0.5L + \omega_{c}:

\[
\Delta_{\text{allowable}} = \frac{\ell}{150} = \frac{31.0 \text{ ft}(12 \text{ in./ft})}{150} = 2.48 \text{ in.}
\]

\[
\Delta_{u} = \frac{5M_{cr}\ell^2}{48F_{c}I_{p}} = 0.935 \text{ in.}
\]

\[
M_{u} = \frac{w_{u}L_{c}^2}{8} + \frac{P_{u}a_{c}c}{2} = 0.7(10 \text{ lb/ft}^2)(15.0 \text{ ft})(2(31.0 \text{ ft}))^2 + 0 = 50.5 \text{ ft-kip}
\]

\[
M = M_{u} + P_{u}\Delta_{u} = 50.5 \text{ ft-kip} + 56.7 \text{ kip}(0.418 \text{ in.}) = 52.4 \text{ ft-kip} < (2/3)M_{cr} = 75.33 \text{ ft-kip}
\]

After iteration, the maximum service load moment is 52.5 ft-kip, leading to an iterated service load deflection of:

\[
\Delta_{u} = \frac{M_{u} - \Delta_{u}}{113 \text{ ft-kip}} = \frac{52.5 \text{ ft-kip} - 0.935 \text{ in.} = 0.434 \text{ in.}}
\]

which is less than permitted by ACI 318, so no further adjustment to the panel thickness is required.

Unlike previous examples, maximum moment occurs at the panel base. Therefore, the designer should consider how the vertical bars would be fully developed to satisfy strength requirements. For this example, assume the panel is embedded 3.0 ft below the support to fully develop the No. 6 bars and check horizontal reinforcement in the panel by ACI 318-11, 14.3.3.

\[
A_{s} = 0.002A_{g} = 0.002(9.75 \text{ in.})(34.0 \text{ ft})(12 \text{ in./ft}) = 8.0 \text{ in.}^2
\]

Therefore, 40 No. 4 bars should be used for horizontal reinforcement. Figure B.7.1.3 details panel reinforcement for this case.

**B.7.2 Reinforcing steel at each face of the panel**—Assume 48 No. 4 bars per face (A_s = 9.6 in.^2 per face) in the original 6.25 in. thickness.

**B.7.2.1** Load Case 1: 1.2D + 0.5W

\[
P_{u} = 0.0 \text{ kip}
\]
\[
P_{u} = 1.2(36.3) = 43.6 \text{ kip}
\]
\[
\omega_{u} = 0.120 \text{ klf}
\]
\[
P_{u}/A_g = 43.6 \text{ kip}(1000 \text{ lb/kip}) = 38.7 \text{ psi} < 0.06f' = 240 \text{ psi}
\]
\[
A_{w} = A_{s} + P_{u}\left( h \over 2d \right) = 9.60 \text{ in.}^2 + 43.6 \text{ kip} \over 60 \text{ ksi} \left( 6.25 \text{ in.} \over 2(5.00 \text{ in.}) \right) = 10.1 \text{ in.}^2
10.1 in. Z (60 ksi)

\[ c = \frac{a}{0.85} = \frac{0.986}{0.85} = 1.16 \text{ in.} \]

\[ \frac{c}{d} = 0.232 < 0.375 : \text{ tension-controlled} \]

(Refer to commentary of ACI 318-11, 9.3.2.2)

\[ I_{cr} = \frac{E_c}{E} A_c (d - c)^2 + \frac{f_y E_y}{3} (1.16)^3 \]

\[ = 8,044(10)(5.0 - 1.16)^2 + \frac{60(12 \text{ in./ft})(1.16)}{3} \]

\[ = 1290 \text{ in.}^4 \]

\[ \phi M_{cr} = \phi A_c f_y \left( \frac{d - c}{2} \right) \]

\[ = 0.9(10)(60) \left( 5.0 - \frac{0.986}{2} \right) = 2450 \text{ in.-kip} = 204 \text{ ft-kip} \]

\[ \phi M_{cr} > M_{cr} \]

\[ \rho = \frac{A_s}{b h} = \frac{9.6 \text{ in.}^2}{15.0 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.00853 > \rho_c = 0.0012 \]

\[ K_s = \frac{48 E_c I_{cr}}{5\alpha c} = \frac{48(3605 \text{ ksi})(1290 \text{ in.}^4)}{5[1.55(31.0 \text{ ksi})(12 \text{ in./ft})]} = 134 \text{ kip} \]

\[ M_{sn} = \frac{w c^3}{8} + \frac{P_{sn} E_y c}{2} = \frac{0.120 \text{ klf}(231.0 \text{ ft})}{8} + 0 = 57.7 \text{ ft-kip} \]

\[ M_a = \frac{M_{sn}}{1 - \frac{P_{sn} c}{0.75 K_s}} = \frac{57.7 \text{ ft-kip}}{1 - \frac{43.6 \text{ kip}}{0.75(134 \text{ kip})}} = 102 \text{ ft-kip} < \phi M_{cr} \]

\[ \Delta_s = \frac{M_{cr}}{0.75 K_s} = \frac{102 \text{ ft-kip}(12 \text{ in./ft})}{0.75(134 \text{ kip})} = 12.1 \text{ in.} \]

\[ \phi M_{cr} = 204 \text{ ft-kip} > M_{cr} \]

\[ K_s = 134 \text{ kip} \]

\[ M_{cr} = 115 \text{ ft-kip} \]

\[ M_a = 203 \text{ ft-kip} < \phi M_{cr} \]

\[ \Delta_a = 24.2 \text{ in.} \]

**B.7.2.3 Load Case 3: 0.9D + 1.0W**

\[ P_{sn} = 0.0 \text{ kip} \]

\[ P_{om} = 32.7 \text{ kip} \]

w.<sub>n</sub> = 0.240 klf

\[ \frac{P_{om}}{A_c} = 38.7 \text{ psi} < 0.06 f' = 240 \text{ psi} \]

\[ A_{ce} = 10.1 \text{ in.}^2 \]

\[ a = 0.986 \text{ in.} \]

\[ c = 1.16 \text{ in.} \]

\[ \frac{c}{d} = 0.232 : \text{ tension-controlled} \]

\[ I_{cr} = 1290 \text{ in.}^4 \]

\[ M_{cr} = 46.3 \text{ ft-kip} \]

\[ \phi M_{cr} = 204 \text{ ft-kip} > M_{cr} \]

\[ K_s = 134 \text{ kip} \]

\[ M_{cr} = 115 \text{ ft-kip} \]

\[ M_a = 203 \text{ ft-kip} < \phi M_{cr} \]

\[ \Delta_a = 24.2 \text{ in.} \]

\[ \Delta_{all} = \frac{\Delta_c}{150} = \frac{3.10 \text{ ft}(12 \text{ in./ft})}{150} = 0.26 \text{ in.} \]

\[ \Delta_{all} = \frac{5M_{cr} c^2}{48 E_c I_{cr}} = 1.46 \text{ in.} \]

\[ M_a = \frac{0.7(10 \text{ lb/ft}^2)(15.0 \text{ ft})(231.0 \text{ ft})^2}{8(1000 \text{ lb/kip})} = 50.5 \text{ ft-kip} \]

This initial service moment without \( P-\Delta \) effects exceeds \((2/3)M_{cr} = 30.9 \text{ ft-kip}\), and the initial service load deflection is calculated by

\[ \Delta_s = \frac{5M_{cr} c^2}{48 E_c I_{cr}} = 20.2 \text{ in.} \]

\[ \Delta_s = (2/3)\Delta_c + \frac{(M_{cr}^2 - (2/3)M_{cr}^3)}{(4/3)M_{cr}^2}(\Delta_c - (2/3)\Delta_c) = 2.92 \text{ in.} \]

using the values for \( M_a \) and \( I_{cr} \) from Load Case 3. After iteration, maximum service load moment is 63.1 ft-kip, leading to

\[ M_a = M_{sn} + P_{sn} \Delta_s \]

\[ \Delta_s = (2/3)\Delta_c + \frac{(M_{cr}^2 - (2/3)M_{cr}^3)}{(4/3)M_{cr}^2}(\Delta_c - (2/3)\Delta_c) = 4.17 \text{ in.} \]

This service load deflection is greater than the maximum permitted by ACI 318, so an adjustment to the panel stiffness is necessary. From the previous calculation, the maximum...
number of bars that could be used in compliance with ACI 318-11, 14.8.2.3, would be 76 No. 4 bars. However, this reinforcement area would not be sufficient to provide the necessary stiffness for limiting panel deflection. The tilt-up panel should, therefore, be reanalyzed using a larger panel thickness at the discretion of the designer.

8.8—Plain panel on isolated footing or pier design example

Figure B.8 illustrates the geometry of the sample panel on an isolated footing or pier. The design procedure is similar to the concentrated axial load example presented in B.3. The panel supports the load from four roof joists bearing in wall pockets, or eccentric axial load, in addition to the wind, or lateral force. A summary of the applied loading is:

\[ P_{ul} = 4(7.5 \text{kip}) = 30.0 \text{kip} \]
\[ P_{ll} = 4(10.0 \text{kip}) = 40.0 \text{kip} \]
\[ e_{cc} = 3 \text{ in. (assumed)} \]
\[ w = 27.2 \text{ lb/ft}^2 \]
\[ f_{c} = 27.2 \text{ lb/ft}^2 \]
\[ f_{c} = 31.0 \text{ ft} - 1.5 \text{ ft} = 29.5 \text{ ft} \]

Because the panel loading is symmetric, concentrated axial load from the isolated footing or pier is one-half of the total applied loads with no eccentricity. The weight of the tilt-up panel above the design section (centerline of the unbraced length) is:

\[ P_{ul} = 4(10.0 \text{kip}) = 30.0 \text{kip} \]
\[ P_{ll} = 4(10.0 \text{kip}) = 40.0 \text{kip} \]
\[ e_{cc} = 3 \text{ in. (assumed)} \]
\[ w = 27.2 \text{ lb/ft}^2 \]
\[ f_{c} = 27.2 \text{ lb/ft}^2 \]
\[ f_{c} = 31.0 \text{ ft} - 1.5 \text{ ft} = 29.5 \text{ ft} \]

However, vertical reinforcement calculated in the following sections will be placed in the 7 ft 4-1/2 in. design strip illustrated in Fig. B.8.

\[ A = 1.80 \text{ in.} \]
\[ 0.85 \]
\[ 1.31 \text{ in.} \]
\[ 0.85 \]
\[ 0.375 \]
66 DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15)

\[ I_c = \frac{E_c}{E_c} A_h (d - c)^2 + \frac{\ell}{3} c^3 \]

\[ = \frac{8.044(5.57)(4.13 - 1.31)^2}{3} + \left( \frac{7.38}{12} \right)^2 \left( \frac{12}{12} \right)^2 = 422 \text{ in.}^4 \]

\[ M_{cr} = f_{cr} \frac{f_y}{f_y} S = f_c \left( \frac{1}{6} bh^2 \right) \]

\[ = 0.474 \text{ ksi} \left( \frac{1}{6} \right) (7.38 \text{ ft})(8.25 \text{ in.})^2 = 39.7 \text{ ft-kip} \]

\[ \phi M_n = \Phi \frac{A_s}{d} \left( \frac{d - a}{2} \right) \]

\[ = \frac{0.9(5.57)(60)}{2} (4.13 - 1.11) = 1070 \text{ in.-kip} \]

\[ = 89.5 \text{ ft-kip} > M_{cr} \]

\[ \rho = \frac{A_s}{bh} = \frac{4.4 \text{ in.}^2}{7.38 \text{ ft}(12 \text{ in./ft})(8.25 \text{ in.})} = 0.00602 > \rho_c = 0.0015 \]

Check applied moment per ACI 318-11, 14.8.3:

\[ K_b = \frac{48E_c I_{cr}}{5c^4} = \frac{48(3605 \text{ ksi})(422 \text{ in.}^4)}{29.5 \text{ ft}(12 \text{ in./ft})^4} = 116 \text{ kip} \]

\[ M_{w,x} = \frac{w_c}{8} \left[ \frac{P_c e_{cr}}{2} + \frac{50.0 \text{ kip}(0 \text{ in.})}{2(12 \text{ in./ft})} \right] = 14.8 \text{ ft-kip} \]

\[ M_x = \frac{M_{w,x}}{1 - \frac{P_{w,x}}{0.75K_b}} = \frac{14.8 \text{ ft-kip}}{1 - \frac{0.701 \text{ kip}}{0.75(116 \text{ kip})}} = 75.0 \text{ ft-kip} < \phi M_n \]

\[ \Delta_x = \frac{M_x}{0.75K_b} = \frac{75.0 \text{ ft-kip}(12 \text{ in./ft})}{0.75(116 \text{ kip})} = 10.3 \text{ in.} \]

**B.8.1.2 Load Case 2: 1.2D + 0.5L + 1.0W**

- \( P_{w,x} = 28.0 \text{ kip} \)
- \( P_{w,y} = 48.1 \text{ kip} \)
- \( w_x = 0.272 \text{ klf} \)
- \( P_{w,y}/A_g = 65.9 \text{ psi} < 0.06f' = 240 \text{ psi} \)
- \( A_{cr} = 5.2 \text{ in.}^2 \)
- \( a = 1.04 \text{ in.} \)
- \( c = 1.22 \text{ in.} \)
- \( c/d = 0.296 \), tension-controlled
- \( I_{cr} = 407 \text{ in.}^4 \)
- \( M_{cr} = 39.7 \text{ ft-kip} > M_{cr} \)
- \( K_b = 112 \text{ kip} \)
- \( M_{w,x} = 29.6 \text{ ft-kip} \)
- \( M_w = 69.0 \text{ ft-kip} < \phi M_n \)
- \( \Delta_w = 9.83 \text{ in.} \)

**B.8.1.3 Load Case 3: 0.9D + 1.0W**

- \( P_{w,x} = 13.5 \text{ kip} \)
- \( P_{w,y} = 28.6 \text{ kip} \)
- \( w_x = 0.272 \text{ klf} \)
- \( P_{w,y}/A_g = 39.1 \text{ psi} < 0.06f' = 240 \text{ psi} \)
- \( A_{cr} = 4.88 \text{ in.}^2 \)
- \( a = 0.972 \text{ in.} \)
- \( c = 1.14 \text{ in.} \)
- \( c/d = 0.277 \), tension-controlled
- \( I_{cr} = 393 \text{ in.}^4 \)
- \( M_{cr} = 39.7 \text{ ft-kip} \)
- \( \phi M_n = 79.8 \text{ ft-kip} = M_{cr} \)
- \( K_b = 108 \text{ kip} \)
- \( M_{w,x} = 29.6 \text{ ft-kip} \)
- \( M_w = 45.6 \text{ ft-kip} < \phi M_n \)
- \( \Delta_w = 6.73 \text{ in.} \)

Check service load deflection per ACI 318-11, R14.8.4, with \( 1.0D + 0.5L + W_w \), noting \( L = 0 \) because only roof load is applied to the panel:

\[ \Delta_{allowable} = \frac{P_r}{150} = 2.36 \text{ in.} \]

\[ \Delta_w = \frac{5M_{w,x}}{48E_c I_{cr}} = 0.471 \text{ in.} \]

\[ M_{w,x} = \frac{w_{cr}^2 f_y^2}{8} + \frac{M_{w,x}}{2} = \frac{0.7(17 \text{ lb/ft}^2)(10.0 \text{ ft})(29.5 \text{ ft})^2}{80000 \text{ lb/kip}} + \frac{15.0 \text{ kip}(0 \text{ in.})}{2(12 \text{ in./ft})} = 12.9 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is:

\[ \Delta_s = \frac{M_{w,x}}{M_{cr}} = \frac{12.9 \text{ ft-kip}}{39.7 \text{ ft-kip}} = 0.325 \text{ in.} < 0.471 \text{ in.} \]

\[ \Rightarrow M_s = M_{w,x} + P_{w,y} \Delta_s = 12.9 \text{ ft-kip} + 31.8 \text{ kip}(0.154 \text{ in.}) = 13.4 \text{ ft-kip} < (2/3)M_{cr} = 26.5 \text{ ft-kip} \]

After iteration, maximum service load moment is 13.4 ft-kip, leading to an iterated service load deflection of:

\[ \Delta_s = \frac{M_{w,x}}{M_{cr}} = \frac{13.4 \text{ ft-kip}}{39.7 \text{ ft-kip}} = 0.471 \text{ in.} < 0.159 \text{ in.} \]

which is significantly less than the value allowed by ACI 318, so no adjustment to the panel stiffness is necessary.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:

\[ A_{h} = 0.002 A_g = 0.002(8.25 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 6.14 \text{ in.}^2 \]

Therefore, 31 No. 4 bars should be used for the horizontal reinforcement. Figure B.8.1.3 details panel reinforcement for this case.
**Fig. B.8.1.3—Single-layer panel reinforcement.**

**B.8.2 Reinforcing steel at each face of the panel**—Assume 12 No. 4 bars per face \( (A_s = 2.4 \text{ in.}^2/\text{per face}) \) in the design strip of the original 6.25 in. panel thickness.

**B.8.2.1 Load Case 1: 1.2D + 1.6L, + 0.5W**

\[
P_{u1} = 50.0 \text{ kip}
\]
\[
P_{w1} = 50.0 \text{ kip} + 1.2(12.7 \text{ kip}) = 65.2 \text{ kip}
\]
\[
\nu = 0.136 \text{ klf}
\]
\[
P = \frac{65.2 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(7.38 \text{ ft})(12 \text{ in./ft})} = 118 \text{ psi} < 0.06f' = 240 \text{ psi}
\]
\[
A_s = A_s + \frac{P_{w1}}{f_y} = 2.40 \text{ in.}^2 + \frac{65.2 \text{ kip}}{60 \text{ ksi}} = 3.08 \text{ in.}^2
\]
\[
a = \frac{A_s f_y}{0.85f_y} = \frac{3.08 \text{ in.}^2(60 \text{ ksi})}{0.85(4 \text{ ksi})(7.38 \text{ ft})(12 \text{ in./ft})} = 0.614 \text{ in.}
\]
\[
c = \frac{a}{0.85} = \frac{0.614}{0.85} = 0.722 \text{ in.}
\]

\( \frac{c}{d} = 0.144 < 0.375 \); tension-controlled

(refer to commentary of ACI 318-11, 9.3.2.2)

\[
I_{ct} = \frac{E}{E_c} A_s (d - c)^2 + \frac{\nu c^3}{3} = 8.044(3.08)(5.0 - 0.722)^2 + \frac{(7.38 \text{ ft})(12 \text{ in./ft})(0.722)^3}{3}
\]
\[
= 464 \text{ in.}^4
\]

\[
M_{ct} = \frac{f_y I_{ct}}{y_c} = f_y S = f_y \left( \frac{1}{2} d^2 \right)
\]
\[
= 0.474 \text{ ksi} \left( \frac{1}{2} (7.38 \text{ ft})(6.25 \text{ in.})^2 \right) = 22.8 \text{ ft-kip}
\]

\[
\phi M_{ct} = \phi A_s f_y \left( \frac{d - a}{2} \right)
\]
\[
= 0.9(3.08)(60)(5.0 - 0.614) = 780 \text{ in.-kip} = 65.0 \text{ ft-kip}
\]

\[\phi M_s > M_{ct}\]

\[
\rho = \frac{A_s}{bh} = \frac{2.40 \text{ in.}^2}{7.38 \text{ ft}(12 \text{ in./ft})(6.25 \text{ in.})} = 0.004 \text{ in.} > 0.0012
\]

\[
K_s = \frac{48E f_y}{5\ell_c} = \frac{48(3605 \text{ ksi})(464 \text{ in.}^4)}{29.5 \text{ ft}(12 \text{ in./ft})^2} = 128 \text{ kip}
\]

\[
M_{u1} = \frac{w_{b1} t^2}{8} + \frac{P_{w1} c}{2} = 0.136 \text{ klf}(29.5 \text{ ft}^2) + 50.0 \text{ kip}(0 \text{ in.}) = 14.8 \text{ ft-kip}
\]

\[
M_1 = \frac{M_{u1}}{1 - \frac{K_s}{75K_h}} = \frac{14.8 \text{ ft-kip}}{1 - \frac{128 \text{ kip}}{75(128 \text{ kip})}} = 46.0 \text{ ft-kip} < \phi M_s
\]

**B.8.2.2 Load Case 2: 1.2D + 0.5L, + 1.0W**

\[
P_{u2} = 28.0 \text{ kip}
\]
\[
P_{w2} = 43.2 \text{ kip}
\]
\[
\nu = 0.272 \text{ klf}
\]
\[
P_{w2}/A_s = 78.2 \text{ psi} < 0.06f' = 240 \text{ psi}
\]
\[
A_s = 2.85 \text{ in.}^2
\]
\[
a = 0.568 \text{ in.}
\]
\[
c = 0.669 \text{ in.}
\]

\[
\frac{c}{d} = 0.134; \text{ tension-controlled}
\]

\[
I_{ct} = 439 \text{ in.}^4
\]

\[\phi M_s = 60.5 \text{ ft-kip} > M_{ct}\]

\[
K_s = 121 \text{ kip}
\]

\[
M_{u2} = 29.6 \text{ ft-kip}
\]

\[\phi M_s > M_{ct}\]

\[
\Delta_s = \frac{M_s}{0.75K_h} = \frac{60.5 \text{ ft-kip}}{0.75(128 \text{ kip})} = 5.74 \text{ in.}
\]

**B.8.2.3 Load Case 3: 0.9D + 1.0W**

\[
P_{u3} = 13.5 \text{ kip}
\]
\[
P_{w3} = 24.9 \text{ kip}
\]
\[
\nu = 0.272 \text{ klf}
\]
\[
P_{w3}/A_s = 45.1 \text{ psi} < 0.06f' = 240 \text{ psi}
\]
\[
A_s = 2.66 \text{ in.}^2
\]
\[
a = 0.530 \text{ in.}
\]
\[
c = 0.624 \text{ in.}
\]

\[\frac{c}{d} = 0.125; \text{ tension-controlled}
\]

\[
I_{ct} = 417 \text{ in.}^4
\]

\[\phi M_s = 56.4 \text{ ft-kip} < \phi M_s
\]
Fig. B.8.2.3—Double-layer panel reinforcement.

\[ M_o = 22.8 \text{ ft-kip} \]
\[ \phi M_o = 56.7 \text{ ft-kip} > M_o \]
\[ K_o = 115 \text{ kip} \]
\[ M_{sw} = 29.6 \text{ ft-kip} \]
\[ M_e = 41.6 \text{ ft-kip} < \phi M_o \]
\[ \Delta_o = 5.78 \text{ in.} \]

Check service load deflection per ACI 318-11, R14.8.4, with \( 1.0D + 0.5L + W_o \), noting \( L = 0 \) because only the roof load is applied to the panel:

\[ \Delta_{alterative} = \frac{\ell_f}{150} = 2.36 \text{ in.} \]
\[ \Delta_o = \frac{5M_e\ell_f^2}{48E_I} = 0.550 \text{ in.} \]
\[ M_{sw} = \frac{w\ell_f^2}{8} + \frac{P_e\Delta_o}{2} = \frac{0.7(17 \text{ lb/ft})^2(10.0 \text{ ft})(29.5 \text{ ft})^3}{8(1000 \text{ lb/kip})} + \frac{15.0 \text{ kip}(0 \text{ in.})}{2(12 \text{ in./ft})} = 12.9 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is:

\[ \Delta_s = \frac{M_{sw}}{M_e} \Delta_o = \frac{12.9 \text{ ft-kip}}{22.8 \text{ ft-kip}} \times 0.550 \text{ in.} = 0.311 \text{ in.} \]
\[ M_{sw} = M_s + P_e\Delta_o = 12.9 \text{ ft-kip} + 27.7 \text{ kip}(0.311 \text{ in.}) = 13.7 \text{ ft-kip} < (2/3)M_e = 15.2 \text{ ft-kip} \]

After iteration, maximum service load moment is 13.7 ft-kip, leading to an iterated service load deflection of:

\[ \Delta_s = \frac{M_{sw}}{M_e} \Delta_o = \frac{13.7 \text{ ft-kip}}{22.8 \text{ ft-kip}} \times 0.550 \text{ in.} = 0.330 \text{ in.} \]

which is significantly less than the value allowed by ACI 318.

Check horizontal reinforcement in the panel per ACI 318-11, 14.3.3:

\[ A_s = 0.002A_e = 0.002(6.25 \text{ in.})(31.0 \text{ ft})(12 \text{ in./ft}) = 4.6 \text{ in.}^2 \]

Because maximum bar spacing is 18 in., 22 No. 4 horizontal reinforcing bars are required on each face. Figure B.8.2.3 details panel reinforcement for the double layer scheme.

**B.8.3 Summary of panel reinforcing steel—**Neglecting trim bars and miscellaneous reinforcing steel, the weight of the primary horizontal and vertical reinforcing steel in the single mat option is 1390 lb, compared with 1684 lb for the double mat option. The designer should note that the single mat option required a panel thickness increase of 2 in. This represents approximately 3.83 more cubic yards of concrete, translating to a panel 15.5 kip heavier. The impact the thicker panel may have on the remainder of the project should be investigated.

**B.9 Panel with stiffening pilasters and header design example**

Figure B.9 illustrates the geometry of the sample panel with a large opening. The panel supports the load from five roof joists bearing in wall pockets (eccentric axial load) in addition to the wind (lateral force). A summary of the applied loading is:

\[ P_{rec} = 5(10.0 \text{ kip}) = 50.0 \text{ kip} \]
\[ P_{re2} = 5(15.0 \text{ kip}) = 75.0 \text{ kip} \]
\[ w = 27.2 \text{ lb/ft}^2 \]
DESIGN GUIDE FOR TILT-UP CONCRETE PANELS (ACI 551.2R-15) 69

\[ \ell_c = 31.0 \text{ ft} - 1.5 \text{ ft} = 29.5 \text{ ft} \]

The weight of the tilt-up panel above the design section (centerline of the unbraced length) is:

\[ \left( \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \right) 150 \text{ lb/ft}^3 \left( 4.0 \frac{\text{ft}}{2} \right) + \left( \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \right) \times 150 \text{ lb/ft}^3 (2.5 \text{ ft}) \left( \frac{29.5 \text{ ft}}{2} + 1.5 \text{ ft} \right) \left( \frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 6.30 \text{ kip} \]

From the experience of the previous examples, it is apparent a single layer of reinforcing steel centered in the panel thickness will not produce an acceptable design for the tilt-up panel. To determine if reinforcing steel in each face of the panel will yield acceptable results, perform a quick analysis on Load Case 1.

**B.9.1 Reinforcing steel in each face of the panel**—Before checking vertical reinforcement, check panel stress by applying the eccentric load evenly between the two panel legs.

\[ P_{\text{ua}} = 1.2(25.0 \text{ kip}) + 1.6(37.5 \text{ kip}) = 90.0 \text{ kip} \]
\[ P_{\text{um}} = 90.0 \text{ kip} + 1.2(6.3 \text{ kip}) = 97.6 \text{ kip} \]
\[ w_u = 0.8(2.5 \text{ ft} + 10.0 \text{ ft})(17 \text{ lb/ft}^2) = 170 \text{ plf} = 0.170 \text{ kip/ft} \]
\[ P_{\text{um}} = 97.6 \text{ kip}(1000 \text{ lb/kip}) = 6.25 \text{ in.}(2.5 \text{ ft})(12 \text{ in./ft}) \]

Because vertical stress at midheight exceeds the maximum permitted by ACI 318, the area of the panel leg should be increased. Instead of uniformly thickening the panel to meet the design criteria, which in this case would yield a panel thickness of 13.5 in., a stiffening pilaster 15.0 in. wide on each leg is considered. The revised panel geometry is illustrated in Fig. B.9.1a. It is also anticipated that the loading will further require a stiffening header to be used to span the large opening. Cross sections for both the pilaster and header are depicted in Fig. B.9.1b.

**B.9.2 Vertical reinforcing steel in pilaster**—Assume three No. 4 bars \( (A_s = 0.60 \text{ in.}^2) \) and provide No. 3 ties at sufficient spacing to satisfy shear flow requirements at the cold joint. At the pilaster and header, add an additional 4.5 kip to the panel weight at the centerline of the unbraced length; therefore, total panel weight at the design section is 10.8 kip.

**B.9.2.1 Load Case 1: 1.2D + 1.6L + 0.5W**
\[ P_{\text{ua}} = 1.2(25.0 \text{ kip}) + 1.6(37.5 \text{ kip}) = 90.0 \text{ kip} \]
\[ P_{\text{um}} = 90.0 \text{ kip} + 1.2(10.8 \text{ kip}) = 103 \text{ kip} \]
\[ w_u = 0.5[1.25 \text{ ft} + 1.25 \text{ ft} + 10 \text{ ft}](27.2 \text{ lb/ft}^2) = 170 \text{ plf} = 0.170 \text{ kip/ft} \]

Check vertical stress at the midheight section of panel per ACI 318-11, 14.8.2.6:

\[ \frac{P_{\text{um}}}{A_s} = \frac{103 \text{ kip}(1000 \text{ lb/kip})}{6.25 \text{ in.}(2.5 \text{ ft})(12 \text{ in./ft})} = 252 \text{ psi} > 0.06 f'_c = 240 \text{ psi} \]
For this design example, concrete compressive strength of the tilt-up panel is increased to 4500 psi. The new values for the original set of design parameters provided at the beginning of this appendix are listed as follows:

\[ f'_c = 4500 \text{ psi} \]

\[ P_{ax} = \frac{103 \text{ kip}(1000 \text{ lb/kip})}{\frac{1.25 \text{ ft}(1.23 \text{ ft})(12 \text{ in. ft})^2 + 6.25 \text{ in.}(2.5 \text{ ft})(12 \text{ in. ft})}{252 \text{ psi} < 0.066f'_c = 270 \text{ psi}} = 252 \text{ psi} < 0.06f'_c \]

\[ f_c = 57 \sqrt{f'_c} = 503 \text{ psi} \]

\[ E_c = 57 \sqrt{f'_c} = 3824 \text{ ksi} \]

\[ d_{f_{f_{h,pl}} = 21 \text{ in. - 1 in. - 0.375 in. - 0.5 in.} = 19.4 \text{ in.}} \]

Check design moment strength:

\[ A = A_s + P_{mn} \left( \frac{h}{2d} \right) = 0.60 \text{ in.}^2 + \frac{103 \text{ kip}}{60 \text{ ksi}} \left( \frac{21.0 \text{ in.}}{2(19.4 \text{ in.})} \right) = 1.53 \text{ in.}^2 \]

\[ a = \frac{A_s f_s}{0.85 f'_c} = \frac{1.53 \text{ in.}^2(60 \text{ ksi})}{0.85(4.5 \text{ ksi})(1.25 \text{ ft})(12 \text{ in. ft})} = 1.60 \text{ in.} \]

\[ c = \frac{a}{0.85} = 1.88 \text{ in.} \]

\[ \frac{c}{d} = 0.097 < 0.375 \quad \text{tension-controlled} \]

(Refer to commentary of ACI 318-11, 9.3.2.2)

\[ I_{cr} = \frac{E_c}{E_s} A_s (d - c)^2 \frac{t - c^3}{3} = 7.59(1.53)(19.4 - 1.88)^2 \frac{(1.25 \text{ ft})(12 \text{ in. ft})(1.88)}{3} = 3590 \text{ in.}^4 \]

\[ M_{cr} = \frac{f_c I_{cr}}{y_e} = f_c S = f_c \left( \frac{1}{6} bt^2 \right) = 0.503 \text{ ksi} \left( \frac{1}{6} \right)(1.25 \text{ ft})(21.0 \text{ in.})^2 = 46.2 \text{ ft-kip} \]

\[ \phi M_s = \frac{A_s f_s}{\left( d - \frac{a}{2} \right)} = 0.9(1.53)(60) \left( 19.4 - \frac{1.60}{2} \right) = 1530 \text{ in.-kip} = 128 \text{ ft-kip} \]

Check minimum reinforcement required by ACI 318-11, 14.3.2:

\[ \rho = \frac{A_s}{bh} = \frac{0.6 \text{ in.}^2}{1.25 \text{ ft}(12 \text{ in. ft})(21.0 \text{ in.})} = 0.00190 > \rho_t = 0.0012 \]

Check applied moment per ACI 318-11, 14.8.3:

\[ K_b = \frac{48 E_c f_c}{5 f'_c} = \frac{48(3824 \text{ ksi})(3590 \text{ in.}^4)}{5[29.5 \text{ ft}(12 \text{ in. ft})]^2} = 1050 \text{ kip} \]

\[ M_{cr} = \frac{w f_e^2}{8} + \frac{P_{mn} e_{rc}}{2} = \frac{0.170 \text{ kft}(29.5 \text{ ft})^2 + 90.0 \text{ kip}(3 \text{ in.})}{2(12 \text{ in. ft})} = 29.7 \text{ ft-kip} \]

\[ M_s = \frac{M_{in}}{0.75 K_b} = \frac{29.7 \text{ kip}}{0.75(1050 \text{ kip})} = 34.2 \text{ ft-kip} < \phi M_s \]

\[ \Delta_s = \frac{M_{in}}{0.75 K_b} = \frac{34.2 \text{ ft-kip}(12 \text{ in. ft})}{0.75(1050 \text{ kip})} = 0.521 \text{ in.} \]

\[ B.9.2.2 \text{ Load Case 2: } 1.2D + 0.5L_r + 1.0W \]

\[ P_{mn} = 48.8 \text{ kip} \]

\[ P_{mn} = 61.7 \text{ kip} \]

\[ w_p = 0.34 \text{ klf} \]

\[ P_{mn}/A_s = 151 \text{ psi} < 0.066f'_c = 270 \text{ psi} \]

\[ A_{se} = 1.16 \text{ in.}^2 \]

\[ a = 1.21 \text{ in.} \]

\[ c = 1.42 \text{ in.} \]

\[ (c/d) = 0.073 \quad \text{tension-controlled} \]

\[ I_{cr} = 2850 \text{ in.}^4 \]

\[ M_{cr} = 46.2 \text{ ft-kip} \]

\[ \phi M_s = 97.8 \text{ ft-kip} > M_{cr} \]

\[ K_b = 833 \text{ kip} \]

\[ M_{in} = 43.1 \text{ ft-kip} \]

\[ M_s = 47.8 \text{ ft-kip} < \phi M_s \]

\[ \Lambda_s = 0.918 \text{ in.} \]

\[ B.9.2.3 \text{ Load Case 3: } 0.9D + 1.0W \]

\[ P_{mn} = 22.5 \text{ kip} \]

\[ P_{mn} = 32.2 \text{ kip} \]

\[ w_p = 0.340 \text{ klf} \]

\[ P_{mn}/A_s = 78.7 \text{ psi} < 0.066f'_c = 270 \text{ psi} \]

\[ A_{se} = 0.891 \text{ in.}^2 \]

\[ a = 0.932 \text{ in.} \]

\[ c = 1.1 \text{ in.} \]

\[ (c/d) = 0.057 \quad \text{tension-controlled} \]

\[ I_{cr} = 2270 \text{ in.}^4 \]

\[ M_{cr} = 46.2 \text{ ft-kip} \]

\[ \phi M_s = 75.8 \text{ ft-kip} > M_{cr} \]

\[ K_b = 664 \text{ kip} \]

\[ M_{in} = 39.8 \text{ ft-kip} \]

\[ M_s = 42.6 \text{ ft-kip} < \phi M_s \]
\[ \Delta_w = 1.03 \text{ in.} \]

Check service load deflection per ACI 318-11, R14.8.4, with \( 1.0D + 0.5L = W_w \), noting \( L = 0 \) because only roof load is applied to the panel:

\[ \Delta_{allowable} = \frac{f}{150} = 2.36 \text{ in.} \]

\[ \Delta_s = 5M_w e^2 \frac{f}{48EI} = 0.163 \text{ in.} \]

\[ M_w = \frac{w e^2}{2} + P_a e_w \]

\[ = \frac{0.7(17 \text{ lb/ft}^2)(12.5 \text{ ft})(29.5 \text{ ft})^2}{8(1000 \text{ lb/kip})} + \frac{25.0 \text{ kip}(3 \text{ in.})}{2(12 \text{ in./ft})} = 19.3 \text{ ft-kip} \]

Using this value as the initial service load moment, the initial deflection is:

\[ \Delta_s = \frac{M_w}{M_{cr}} \Delta_e = 19.3 \text{ ft-kip} \cdot \frac{0.163 \text{ in.}}{46.2 \text{ ft-kip}} = 0.068 \text{ in.} \]

After iteration, maximum service load moment is 19.5 ft-kip, leading to an iterated service load deflection of:

\[ \Delta_s = \frac{M_w}{M_{cr}} \Delta_e = 19.5 \text{ ft-kip} \cdot \frac{0.163 \text{ in.}}{46.2 \text{ ft-kip}} = 0.069 \text{ in.} \]

which is significantly less than the maximum permitted by the ACI 318, so no adjustment to the pilaster stiffness is necessary.

**B.9.3 Horizontal reinforcing steel in header**—The header should transfer vertical loads into the tilt-up panel legs and resist the lateral load across the opening width. Assume four No. 5 bars (\( A_s = 1.24 \text{ in.}^2 \)) for vertical loads, one No. 4 bar (\( A_s = 0.20 \text{ in.}^2 \)) for lateral loads, and provide No. 3 ties at sufficient spacing to satisfy shear flow requirements at the cold joint. Refer to Fig. B.9.1b(a). Design the header as a simply supported beam.

The distributed load across the opening due to panel self-weight is:

\[ 150 \text{ lb/ft}^2 \left[ \frac{6.25 \text{ in.}^2}{12 \text{ in.}} \right] (4.0 \text{ ft}) + \left[ \frac{5.75 \text{ in.}^2}{12 \text{ in./ft}} \right] (1.0 \text{ ft}) \left[ \frac{1 \text{ kip}}{1000 \text{ lb}} \right] \]

\[ = 0.384 \text{ klf} \]

**B.9.3.1 Load Case 1: 1.2D + 1.6L + 0.5W**

\[ M_{w,\text{net}} = 1.2(18.8 \text{ kip}) + 1.6(22.5 \text{ kip}) = 58.6 \text{ kip} \]

\[ M_{w,\text{net}} = 1.2(199.2 \text{ ft-kip}) + 1.6(150.0 \text{ ft-kip}) = 383 \text{ ft-kip} \]

\[ w_w = 0.8(4.0 \text{ ft})(17 \text{ lb/ft}^2) = 54.4 \text{ plf} \]

\[ w_w = (1/8)w_w^{\text{net}} = (1/8)(0.0544 \text{ klf})(20.0 \text{ ft})^2 = 2.72 \text{ ft-kip} \]

Check the shear capacity of the 6.25 in. concrete section per ACI 318-11, 11.3.1.1:

\[ \phi V_s = 2\phi f'y b_s d = 2(0.75)\sqrt{4500(6.25 \text{ in.})(45 \text{ in.})} = 28.3 \text{ kip} \]

Because \( \phi V_s < V_w \), the section requires shear stirrups. From ACI 318-11, 11.5.6.2, the shear stirrup area for a 1 in. spacing would be:

\[ V_s = \frac{1}{\phi}(V_w - \phi V_s) \]

\[ A_s = \frac{f_s}{f'_y d} \left( \frac{1}{6} \right) \left( \frac{58.6 \text{ kip} - 28.3 \text{ kip}}{60 \text{ ksi}(45 \text{ in.})} \right)(1 \text{ in.}) = 0.015 \text{ in.}^2/\text{in.} \]

Therefore, No. 3 shear stirrups should be used at 7.25 in. on center in the 6.25 in. thick section.

Check the design moment strength for vertical loads:

\[ a = A_s f_y \frac{0.85 f'_y}{0.85(4.5 \text{ ksf})(6.25 \text{ in.})} = 3.11 \text{ in}. \]

\[ M_w = \frac{f_s I_s}{f_y} = \frac{f_S}{f'_y} \left( \frac{1}{6} b_t^2 \right) \]

\[ = 0.503 \text{ ksi} \left( \frac{1}{6} \right) \left( \frac{6.25 \text{ in.}}{12 \text{ in./ft}} \right) (48.0 \text{ in.})^2 = 101 \text{ ft-kip} \]

\[ \phi M_s = \phi A_s f_y \left( d - \frac{e}{2} \right) = 0.9(1.24)(60) \left( 45.0 - \frac{3.11}{2} \right) = 2910 \text{ in.-kip} = 242 \text{ ft-kip} \]

Because adequate moment strength has not been provided, revise reinforcing steel bars to two No. 7 plus two No. 6 (\( A_s = 2.08 \text{ in.}^2 \)) and recalculate \( \phi M_s \) to be 397 ft-kip. Check minimum reinforcement required by ACI 318-11, 10.5.1:

\[ \rho = \frac{b_t d}{6.25 \text{ in.}(45.0 \text{ in.})} = 0.0074 > \rho_{\text{min}} = \frac{200}{60,000} = 0.00333 \]

Check the design moment strength for lateral loads:

\[ a = A_s f_y \frac{0.20 \text{ in.}^2}{0.85 f'_y} \frac{0.85(4.5 \text{ ksf})(12 \text{ in.})}{12 \text{ in.}} = 0.261 \text{ in.} \]

\[ \phi M_s = \phi A_s f_y \left( d - \frac{e}{2} \right) = 0.9(0.20)(60) \left( 10.4 - \frac{0.261}{2} \right) = 111 \text{ in.-kip} = 9.22 \text{ ft-kip} \]
Fig. B.9.3.3—Revised tilt-up panel header cross section.

**B.9.3.2 Load Case 2: 1.2D + 0.5L + 1.0W**

\[ V_{w,vert} = 1.2(18.8 \text{ kip}) + 0.5(22.5 \text{ kip}) = 33.8 \text{ kip} \]
\[ M_{w,vert} = 1.2(119.2 \text{ ft-kip}) + 0.5(150.0 \text{ ft-kip}) = 218 \text{ ft-kip} \]
\[ w_{u,wind} = 1.6(4.0 \text{ ft})(17 \text{ lb/ft}^2) = 109 \text{ plf} = 0.109 \text{ klf} \]
\[ M_{w,wind} = (1/8)w_{u,wind}r^2 = (1/8)(0.109 \text{ klf})(20.0 \text{ ft})^2 = 5.45 \text{ ft-kip} \]

Because the header is not influenced by \( P-\Delta \) effects, as is the leg in the tilt-up panel, a comparison can be made directly with the calculated strength capacities from B.9.3.1. All reinforcing steel proposed meets the requirements for the factored loads for this load case.

**B.9.3.3 Load Case 3: 0.9D + 1.0W**

\[ V_{w,vert} = 0.9(18.8 \text{ kip}) = 16.9 \text{ kip} \]
\[ M_{w,vert} = 0.9(119.2 \text{ ft-kip}) = 107 \text{ ft-kip} \]
\[ w_{u,wind} = 1.6(4.0 \text{ ft})(17 \text{ lb/ft}^2) = 109 \text{ plf} = 0.109 \text{ klf} \]
\[ M_{w,wind} = (1/8)w_{u,wind}r^2 = (1/8)(0.109 \text{ klf})(20.0 \text{ ft})^2 = 5.45 \text{ ft-kip} \]

Because the header is not influenced by \( P-\Delta \) effects, as is the leg in the tilt-up panel, a comparison can be made directly with the calculated strength capacities from B.9.3.1. All reinforcing steel proposed meets the requirements for the factored loads for this load case.

Therefore, the header cross section originally depicted in Fig. B.9.1b(a) is not valid. The No. 3 shear stirrups should be spaced at 7.25 in. on center in the 6.25 in. section within the region prescribed by ACI 318. The No. 3 ties should be provided at sufficient spacing to satisfy shear flow requirements at the cold joint. Figure B.9.3.3 illustrates the final reinforcing steel details required for the header cross section.
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