PDHonline Course S223 (8 PDH)

The Design and Construction of Cast-in-Place Concrete Axial Load Carrying Members including Columns and Walls (both Shearwalls and Tilt-Up Walls)

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The Design and Construction of Cast-in-Place Concrete Axial Load Carrying Members including Columns and Walls (both Shearwalls and Tilt-Up Walls)

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COURSE CONTENT

Columns

Simplified Design Approach:

Preliminary Sizing

For overall construction economy, the dimensions of a column should be determined based on the maximum load effects in the structure and should remain constant for the entire height of the building with only the amounts of reinforcement varying over the height of the column. The most economical columns usually have reinforcement ratios in the range of 1 to 2% of the gross cross sectional area of the column (A_g). In general, from a construction standpoint, it is more efficient to increase the column size than to increase the amount of reinforcement. This approach is recommended in order to avoid congestion of column reinforcement, particularly at lap splice locations and at the intersection of the horizontal framing reinforcement such as beam column intersections.

Columns in a frame that is braced by shearwalls (non-sway frame) are designed for gravity loads only. The initial size of non-sway frame columns can be obtained from a number of different design aid resources (CRSI, ACI and PCA) including Figure 1. Figure 1 is based on a reinforcement ratio of 1 to 8% Ag (where 0.08Ag equals the maximum reinforcement allowed by ACI Section 10.9.1), for square columns using the total factored axial load P_u in the lowest story where the load eccentricity does not exceed 0.1h (h = size of the column). Similar charts can also be developed using Equation 10-1 and 10-2 from ACI Section 10.3.6.1.

Interaction Charts

Once an initial column size is established, slenderness effects need to be considered next. When a frame is not braced by shearwalls (i.e. sway frame column), the columns must be designed for the combined effects of gravity and lateral loads. In this case, a preliminary size can still be obtained for a column from Figure 1 assuming that the column carries gravity loads only. The size can be chosen based on the reinforcement ratios in Figure 1 and then increased without having to change the column size to account for the lateral loads. In addition, for sway frame columns, the interaction charts can also be used to determine the required column size and reinforcement for a given combination of factored axial loads and moments for f'_c = 4.0 ksi and f_y = 60 ksi for square, tied, non-slender columns with symmetrical bar arrangements shown in Figure 2.

For simplicity, each column interaction design curve is typically plotted with straight lines connecting a number of points corresponding to certain transition stages. In general, the transition stages are defined as follows (see Figure 3):

Stage 1: Pure compression (no bending moment)
Stage 2: Stress in reinforcement closest to tension face = 0; ($f_s = 0$)
Stage 3: Stress in reinforcement closest to tension face = 0.5 f_y; (fs = 0.5f_y)
Stage 4: Balanced point; stress in reinforcement closest to tension face = f_y; ($f_s = f_y$)
Stage 5: Pure Bending (no axial load)

It should be noted that Stages 2 and 3 are used to determine which type of lap splice is required for a given load combination (ACI Section 12.17). Specifically, for load combinations falling within Zone 1, compression lap splices are allowed, since all of the bars are in compression. In Zone 2, either Class A (half or fewer of the bars spliced at one location) or Class B (more than one-half of the bars spliced at one location) tension lap splices must be used. Class B tension lap splices are required for load combinations falling within Zone 3.

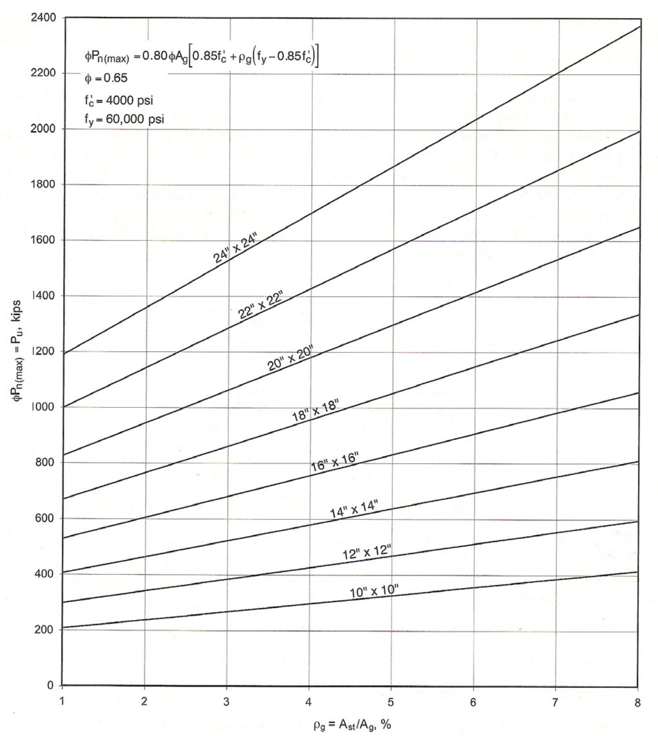


FIGURE 1

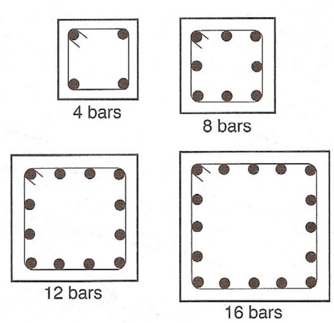


FIGURE 2

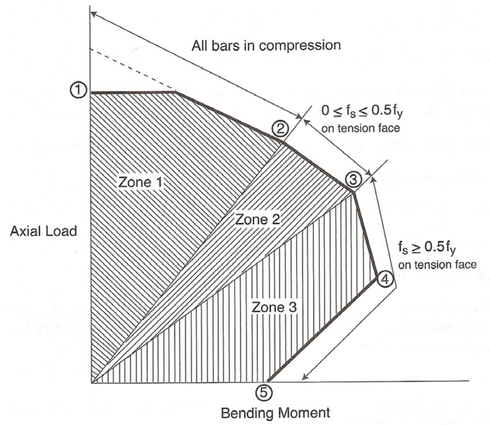


FIGURE 3

Simplified equations based on strain compatibility analysis can be derived to obtain the critical points on design interaction diagram above that correspond to each transition stage. The following equations are valid within the limitations stated above:

Point 1: $\Phi P_{n(max)} = 0.80\Phi((0.85f'_c(A_g - A_{gst})) + f_yA_{st})$; kips

Points 2 thru 4: $\Phi P_n = \Phi((C_1hd_1) + 87(\sum_{i=1}^{n} A_{si}(1 - C_2(di/d_1))))$; kips

$\Phi M_n = \Phi((0.5C_1hd_1)(h - C_3d_1) + 87(\sum_{i=1}^{n} A_{si}(1 - C_2(di/d_1))(h/2 - d_i)))/12$; KF

Point 5: $\Phi M_n = 4A_{s1}d_{1;}$ (columns with 2 or 3 layers of reinforcement); KF

$\Phi M_n = 4(A_{s1} + A_{s2})(d_1 - s/2)$; (columns with 4 or 5 layers of reinforcement); KF

Where: A_g = Gross area of column, in2

A_{st} = Total area of longitudinal reinforcing; in^2

$\Phi = 0.65$ (ACI 318-05 Section 9.3.2.2) for ΦP_n

$\Phi = 0.90$ (ACI 318-05 Section 9.3.2.1) for ΦM_n

h = Column dimension in direction of bending; inches

d_1 = Distance from compression face to centroid of reinforcing steel in layer closet to tension face; inches

d_i = Distance from compression face to centroid of reinforcing steel in layer i; inches

A_{si} = Total steel area in layer i; in^2

n = Total number of layers of reinforcing

C_1, C_2, C_3 = Constants, see Table 1

s = Center-to-center spacing of bars; inches

See Figure 4 for additional information on the variables used above. Also, to assure that the stress in the reinforcing bars is less than or equal to fy, the quantity $(1 - C_2(di/d_1))$ must always be taken less than or equal to 60/87 = 0.69.

The simplified equations for Points 2-4 will produce values of ΦP_n and ΦM_n approximately 3% larger than the exact values. The equations for Point 5 and will produce conservative values of ΦM_n for the majority of the cases. For columns subjected to small axial loads and large bending moments, a more precise investigation into the adequacy of the section should be made because of the approximate shape of the simplified interaction diagram in the tension-controlled region. However, for typical building columns load combinations of this type are rarely encountered, except at the top of buildings where it is common to see reinforcing requirements increase when compared to more heavily loaded columns in the lower levels of the structure.

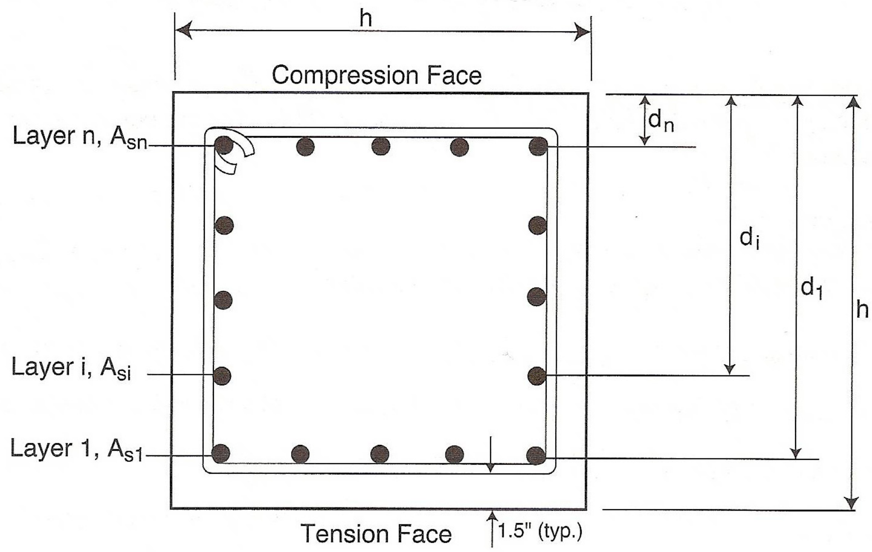


FIGURE 4

Point No.	C_1	C_2	C_3
2	2.89	1.00	0.85
3	2.14	1.35	0.63
4	1.70	1.69	0.50

TABLE 1

For a column with a larger cross-section than required for loads (i.e. oversized for architectural reasons), a reduced minimum reinforcing ratio of not less than one-half percent (.5%) of A_g may be used (see Section 10.8.4). This provision must not be used in regions of high seismic risk, however.

Biaxial Bending

Biaxial being of a column occurs when the loading causes bending simultaneously about both principal axes. This problem is most often encountered in the design of corner columns. A typical biaxial interaction surface is illustrated in Figure 5. To avoid the numerous mathematical complexities associated with the exact surface, several approximate techniques have been developed that relate the response of a column in biaxial bending to is uniaxial resistance about each principal axis. Therefore a conservative estimate of the nominal axial load strength can be obtained from the following:

$$\Phi P_{ni} = 1/((1/\Phi P_{nx}) + (1/\Phi P_{ny}) - (1/\Phi P_o))$$

Where: P_{ni} = Nominal axial load strength of column subjected to P_u at eccentricities e_x and e_y

P_{nx} = Nominal axial load strength of column subjected to P_u at eccentricity e_x only ($e_y = 0$)

P_{ny} = Nominal axial load strength of column subjected to P_u at eccentricity e_y only ($e_x = 0$)

P_o = Nominal axial load strength of column subjected to P_u at eccentricity of 0 (e_x and $e_y = 0$)

For design, $P_u < \Phi P_{ni}$; where P_u is the factored axial load acting at eccentricities e_x and e_y. This method is most suitable when ΦP_{nx} and ΦP_{ny} are greater than the corresponding balanced axial loads, which is usually the case for a typical building column. An iterative design process will be required when using this approximate equation for columns subjected to biaxial loading. A trial section can be obtained from column interaction charts using a factored axial load P_u and a total factored moment $M_u = M_{ux} + M_{uy}$; where $M_{ux} = P_u e_x$ and $M_{uy} = P_u e_y$. The expression for ΦP_{ni} can then be used to check if the section is adequate or not. Usually, only an adjustment in the amount of reinforcement will be required to obtain an adequate or more economical section.

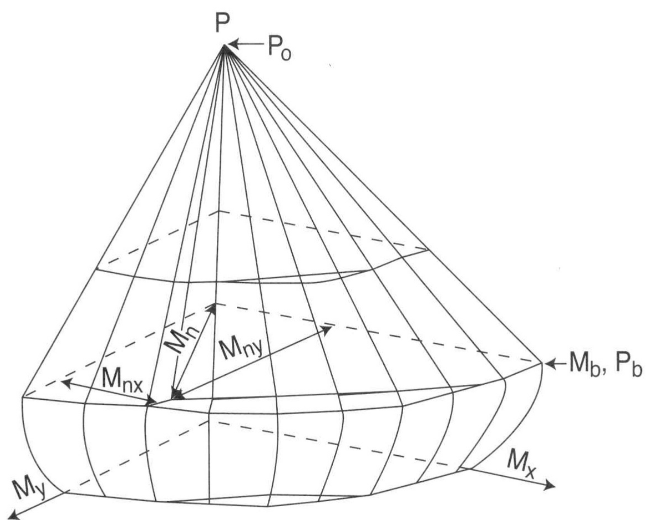


FIGURE 5

Column Ties

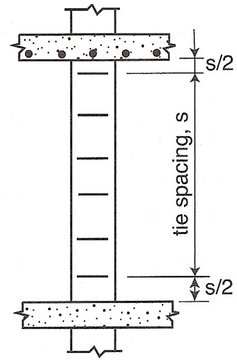
The column tie spacing requirements of Section 7.10.5 are summarized in Figure 6. For #10 column vertical bars and smaller, #3 or larger ties are required. For bars larger than # 10, #4 or larger ties must be used. The maximum tie spacing shall not exceed the lesser of:

1. 16 longitudinal bar diameters.

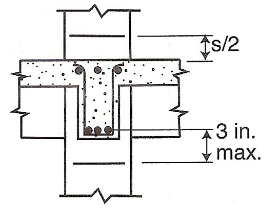
2. 48 tie bar diameters.

3. The least column dimension.

Tie Size	Column Bars	Maximum Spacing* (in.)
#3	# 5	10
	# 6	12
	# 7	14
	# 8	16
	# 9	18
	#10	18
#4	#11	22



Interior column-slab joint**



Interior column-beam joint**

*Maximum spacing not to exceed least column dimension (ACI 7.10.5.2)
**Also valid for joints with beams on less than 4 sides of the column (ACI 7.10.5.4)
***Beams on all 4 sides of the column (ACI 7.10.5.5)

FIGURE 6

Suggested tie details to satisfy Section 7.10.5.3 are shown in Figure 7 for 8, 12 and 16 column bar arrangements. In any square (or rectangular) bar arrangement, the four corner bars must be enclosed by a single one-piece tie (Section 7.10.5.3). The ends of the ties must be anchored by a standard 90 degree or 135 degree hook (Section 7.1.3). It is important to alternate the position of the hooks in placing successive sets of ties. For easy field erection, the intermediate bars in the 8 and 16 bar arrangements can be supported by the separate cross ties shown in Figure 7, however, the position of the 90 degree hooked end at each successive tie location should be alternated. The two-piece tie shown for the 12 bar arrangement should be lap spliced at least 1.3 times the tensile development length (ℓ_d) of the tie bar, but not less than 12 inches. To eliminate the supplementary ties for the 8, 12 and 16 bar arrangements, 2, 3 and 4 bar bundles may be used at each corner of the column, however at least #4 ties are required in this case (Section 7.10.5.1).

Column ties must be located not more than one-half a tie spacing above the top of a footing or slab in any story, and not more than one-half a tie spacing below the lowest reinforcement in the slab (or drop panel) above (Section 7.10.5.4 and Figure 6). Where beams frame into a column from four sides, ties may be terminated 3 inches below the lowest beam reinforcement (Section 7.10.5.5). Note that extra ties are required within 6 inches from points of offset bends at column splices (Section ACI 7.8.1).

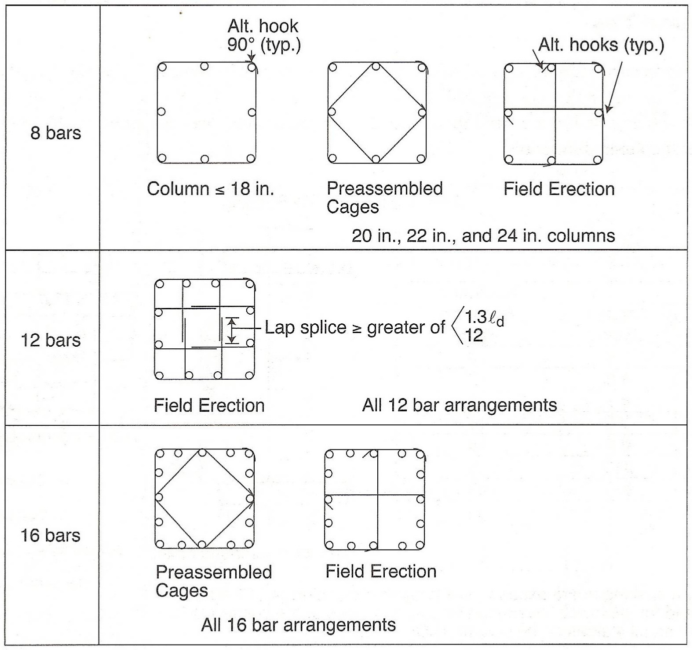


FIGURE 7

Slenderness Considerations

When designing columns, it is important to establish whether or not the building frame is non-sway. A compression member may be assumed non-sway if located in a story in which the bracing elements (i.e. shear walls) have a substantially large enough lateral stiffness to resist lateral moment of the story such that the resulting lateral deflection is not large enough to affect the column strength substantially (see Section R10.11.4). Determining whether or not the building is sway or non-sway cannot be emphasized enough because the design time can be greatly reduced if the building frame is adequately braced by shearwalls, and the columns are sized so that effects of slenderness may be neglected.

The criteria used to determine whether a column is slender (per the requirements of Section 10.10) are summarized in Figure 8, where M_{2b} is the larger factored end moment and M_{1b} is the smaller end moment. Both of these moments, which are determined from elastic analysis, are due to the loads that result in no appreciable side sway. The ratio M_{1b}/M_{2b} is positive for a column bent in single curvature and negative for a column bent in double curvature. For non-sway columns the effective length factor (k) is 1.0 (Section 10.11.12.1). For a sway column with a column-to-beam stiffness ratio (ψ) of 1.0 at both ends, the effects of slenderness may be neglected when values of ℓ_u/h are less than 5, assuming k = 1.3 (see the alignment chart, Section R 10.12). In addition, if the beam stiffness is reduced to one-fifth of the column stiffness at each end, then k = 2.2, and the slenderness effects need not be considered as long as ℓ_u/h is less than 3. Finally, in accordance with ACI Section 10.12.2, effects of slenderness may be neglected when non-sway columns are sized to satisfy the following:

$$\ell_u/h \leq 12$$

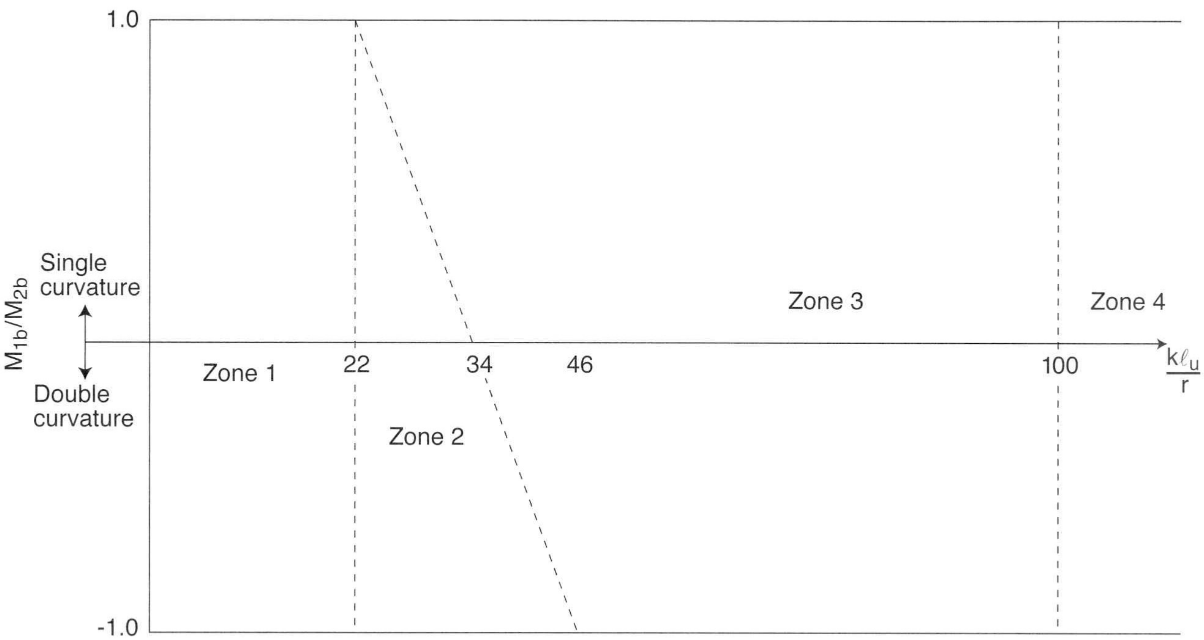
Where: ℓ_u = The clear height between floor members

h = The column size

The above equation is valid for columns that are bent in double curvature with approximately equal end moments. The above equation can be used for the first story columns provided the degree of fixity at the foundation is large enough. Table 2 gives the maximum clear height for a column size that would permit slenderness to be neglected.

Column size h (in.)	Maximum clear height ℓ_u (ft)
10	10
12	12
14	14
16	16
18	18
20	20
22	22
24	24

TABLE 2



Zone 1: Neglect slenderness, braced and unbraced columns
Zone 2: Neglect slenderness, braced columns
Zone 3: Consider slenderness, moment magnifier method (ACI 10.11.5)
Zone 4: Consider slenderness. PΔ analysis (ACI 10.11.4.3)

FIGURE 8

It should also be noted that a minimum moment (i.e. minimum eccentricity requirement) for slender compression members in a braced frame is required by Section 10.12.3.2. If the factored column moments are very small or zero, the design of these columns must be based on a minimum moment of $P_u(0.6 + 0.03h)$

Shear

Columns in sway frames are required to resist the shear forces from lateral loads. For members subjected to axial compression, the concrete shear strength ΦV_c is given in ACI Equation 11.4. Figure 9 can be used to determine this quantity for the square column sizes shown. The largest bar size from column interaction charts provided by the PCA were used to compute ΦV_c in Figure 9. ACI Equation 9-6 should also be used to check column shear strength; where $U = 0.9D + 1.6W$, $N_u = P_u = 0.9D$, and $V_u = 1.6W$. If V_u is greater than ΦV_c, then the spacing of the column ties must be reduced to provide additional shear strength ΦV_s. Values of ΦV_s are given in Table 3.

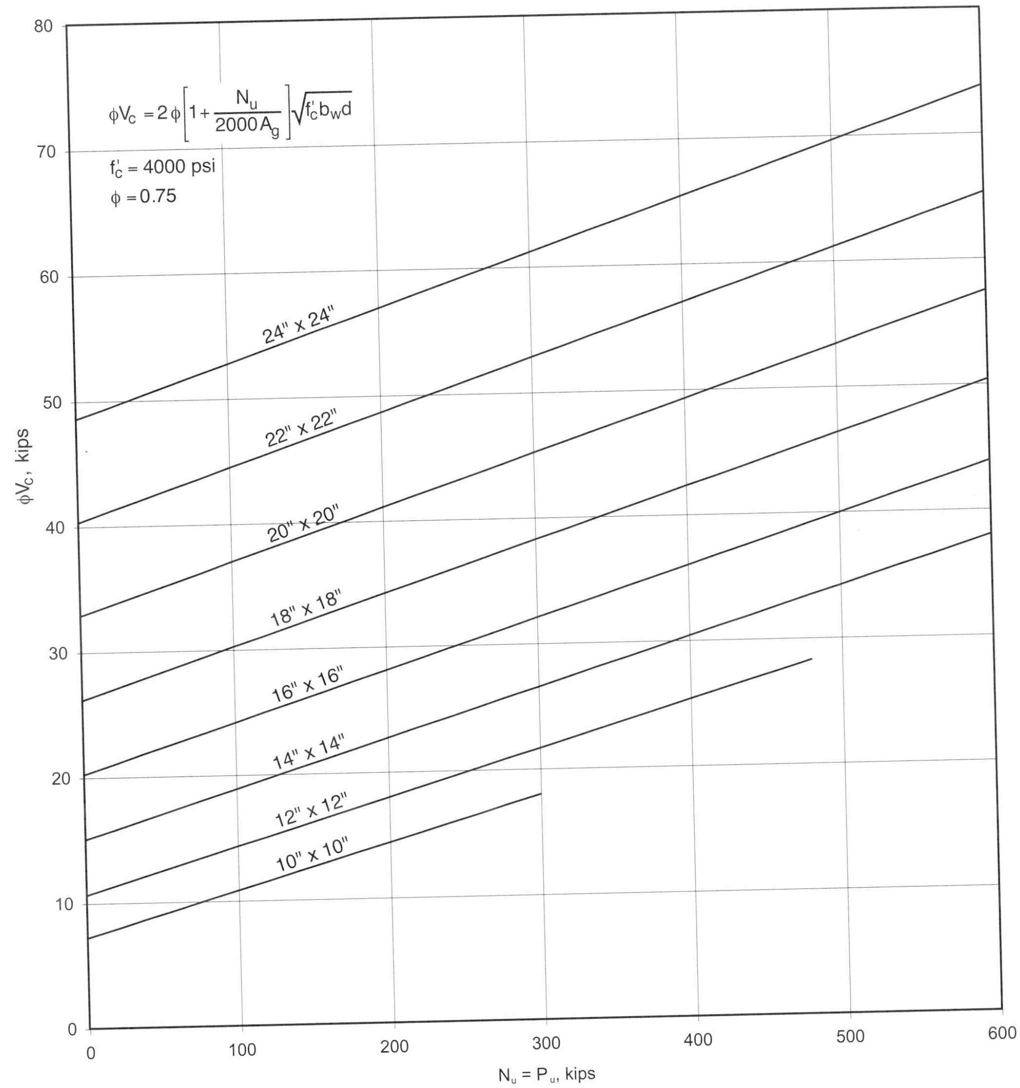


FIGURE 9

Tie Spacing	V_s - #3 ties*	V_s - #4 ties*
d/2	19 kips	35 kips
d/3	29 kips	54 kips
d/4	40 kips	71 kips

*2 legs, Grade 60 bars

TABLE 3

Designing for Slenderness Effects:

Slenderness Effects

Simplified design of columns, including the need to avoid columns associated with sway frames and columns impacted by slenderness effects, was presented first in this lecture to emphasize that the complexities of column design coupled with the time constraints associated with a typical structural consulting firm make it desirable to keep the design of concrete columns as simply as possible. Never the less it is still necessary on some occasions to get involved with a more in depth analysis of a column, therefore we will now discuss slenderness effects in more detail.

Column slenderness is expressed in terms of its slenderness ratio $k\ell_u/r$ where k is an effective length factor (dependent on rotational and lateral restrains at the ends of the column), ℓ_u is the unsupported column length, and r is the radius of gyration of the column cross-section. In general, a column is slender if its applicable cross-sectional dimension is small in comparison to its length.

For design purposes, the term "short column" is used to denote a column that has a strength equal to that computed for its cross-section, using the forces and moments obtained from an analysis for combined bending and axial load. A "slender column" is defined as a column whose strength is reduced by second-order deformations (secondary moments). By these definitions, a column with a given slenderness ratio may be considered a short column for design under one set of restrains, and a long column under another set.

A short column may fail due to a combination of moment and axial load that exceeds the strength of the cross-section. This type of failure is known as "material failure." In this scenario, as illustrated by Figure 10, the column exhibits a deflection Δ which will cause an additional (secondary) moment in the column. From the free body diagram shown in Figure 10, it can be seen that the maximum moment (M) in the column occurs at section A-A, and is equal to the applied moment plus the moment due to the member deflection, which can be represented as $M = P(e + \Delta)$.

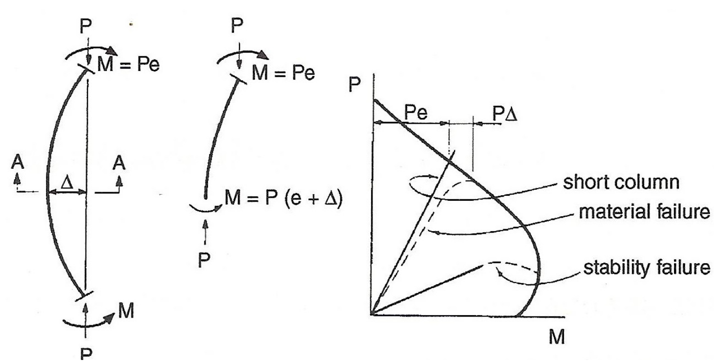


FIGURE 10

If a column is very slender, it may reach a deflection due to axial load P and a moment Pe such that deflections will increase indefinitely with an increase in the load P. This type of failure is known as a "stability failure".

The basic concept of the behavior of straight, concentrically loaded, slender columns was originally developed by Euler more than 200 years ago. It states that a member will fail by buckling at the critical load $P_c = \pi^2 EI/(\ell_e)^2$; where EI is the flexural stiffness of the member cross-section, and ℓ_e is the effective length, which is equal to $k\ell_u$. For a "stocky" short column, the value of the buckling load will exceed the direct crushing strength (corresponding to material failure). In members that are more slender (i.e., members with larger $k\ell_u/r$ values), failure may occur by buckling (stability failure), with the buckling load decreasing with increasing slenderness (see Figure 11).

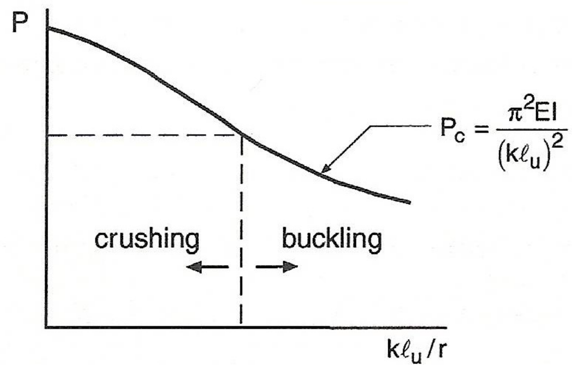


FIGURE 11

Slenderness limits are prescribed for both non-sway and sway frames, including design methods permitted for each slenderness range. At the lower-bound slenderness limits, secondary moments are allowed to be disregard and only axial load and primary moment have to be considered to select a column cross-section and reinforcement (i.e. short column design). For moderate slenderness ratios, an approximate analysis of slenderness effects based on a moment magnifier (see Section 10.12 for non-sway frames and Section 10.13 for sway frames) is permitted. For columns with high slenderness ratios, a more exact second-order analysis is required (see Section 10.11.5) which takes into account material nonlinearity and cracking, as well as the effects of the member's curvature and lateral drift, duration of the loads, shrinkage and creep, and interaction with the supporting foundation. No upper limits for column slenderness are prescribed. The slenderness ratio limits in 10.12.2 for non-sway frames and 10.13.2 for sway frames, and design methods permitted for consideration of column slenderness, are summarized in Figure 12.

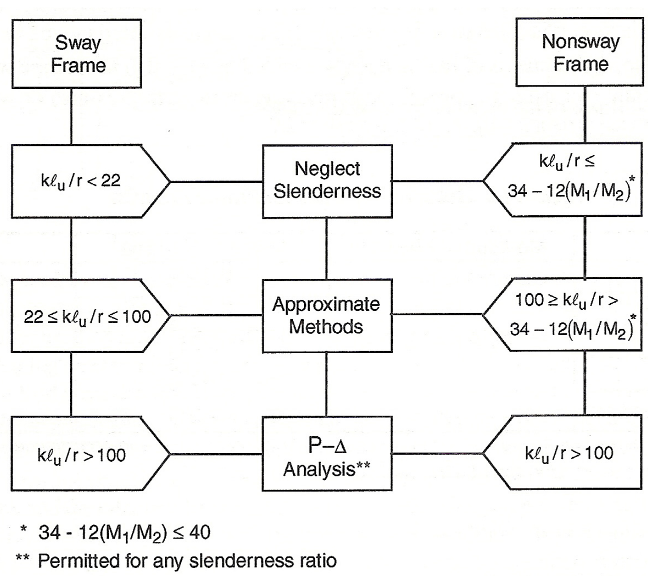


FIGURE 12

The code encourages the use of a second-order frame analysis or P-Δ analysis for consideration of slenderness effects in compression members. Generally, the results of a second-order analysis give more realistic values of the moments than those obtained from an approximate analysis using Sections 10.12 or 10.13. For sway frames, the use of second-order analyses will generally result is a more economical design. However, if a more exact analysis is not feasible or practical, Section 10.10.2 permits an approximate moment magnifier method to account for column slenderness. Note, however, that for all compression members with a column slenderness ratio ($k\ell_u/r$) greater than 100, a more exact analysis as defined in Section 10.10.1 must be used for consideration of slenderness effects.

Approximate Methods of Evaluation

The moment magnification factor δ is used to magnify the primary moments to account for increased moments due to member curvature and lateral drift. The moment magnifier δ is a function of the ratio of the applied axial load to the critical or buckling load of the column, the ratio of the applied moments at the ends of the column, and the deflected shape of the column. In order to use the moment magnification approach it is first necessary to obtain the column ends moments using elastic first order analysis, taking into account the cracked section properties along the length of the member. Since this type of analysis can become very time consuming relative to the calculation of the effective moment of inertia, Section 10.11.1. (see Table 4 for a summary) allows for the use of approximate values for the effective moment of inertia as a percentage of I_g. It is important to note that for service load analysis of the structure, it is satisfactory to multiply the moments of inertia given in Table 4 by 1/0.70 = 1.43 (see Section R10.11.1). Also the moments of inertia must be divided by ($1 + \beta_d$) in a case where sustained lateral loads act on the structure (for example, lateral loads resulting from earth pressure) or when the gravity load stability check made in accordance with Section 10.13.6 is performed.

	Modulus of Elasticity	Moment of Inertia†	Area
Beams	E_c from 8.5.1	$0.35\ I_g$	$1.0A_g$
Columns		$0.70\ I_g$	
Walls - uncracked		$0.70\ I_g$	
Walls - cracked		$0.35\ I_g$	
Flat plates and flat slabs		$0.25\ I_g$	

†Divide by ($1+\beta_d$) when sustained lateral loads act or for stability checks made in accordance with 10.13.6. For service load analyses, multiply by 1/0.70 = 1.43.

TABLE 4

The radius of gyration, r, used to calculate the column slenderness ratio = $(I_g/A_g)^{1/2}$. However, r may be taken as 0.30 times the dimension in the direction of analysis for a rectangular section and 0.25 times the diameter of a circular section. The unsupported length, ℓ_u, used to calculate the column slenderness ratio, as defined by Section 10.11.3, is the clear distance between lateral supports as shown in Figure 13. Note that the length ℓ_u may be different for buckling about each of the principal axes of the column cross-section. The basic equations for the design of slender columns were derived for hinged end members, and therefore, must be modified to account for the effects of end restraint. The effective column length, $k\ell_u$, as contrasted to the actual unbraced length ℓ_u is the term used in estimating slender column strength and considers end restrains as well as non-sway and sway conditions.

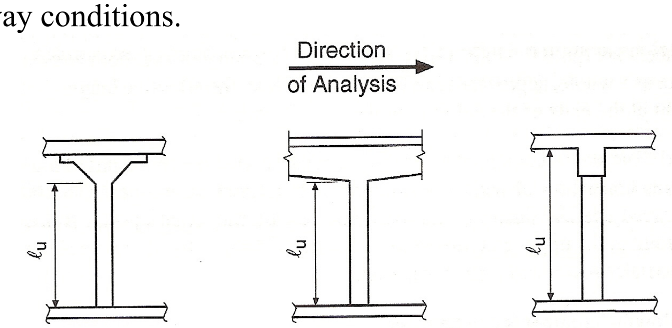


FIGURE 13

At the critical load defined by the Euler equation, an originally straight member buckles into a half-sine wave as shown in Figure 14a. In this configuration, an additional moment P-Δ acts at every section, where Δ is the lateral deflection at the specific location under consideration long the length of the member. This deflection continues to increase until the bending stress caused by the increasing P-Δ moment plus the original compression stress caused by the applied loading, exceeds the compressive strength of the concrete and member fails. The effective length, ℓ_e (= $k\ell_u$), is the length between pinned ends, between zero moments or between inflection points. For the pinned condition illustrated in Figure 14a, the effective length is equal to the unsupported length ℓ_u. If the member is fixed against rotation at both ends, it will buckle in the shape depicted in Figure 14b and the inflection points will occur at the locations shown, and the effective length ℓ_e will be one-half of the unsupported length. The critical buckling load P_c for the fixed-end condition is four times that for a pin-end condition. Rarely are columns in actual structures either hinged or fixed, in reality they are partially restrained against rotation by members framing into the column, therefore the effective length is somewhere between $\ell_u/2$ and ℓ_u as shown in Figure 14c, as long as the lateral displacement of one end of the column with respect to the other end is prevented. The actual value of the effective length depends on the rigidity of the members framing into the top and bottom ends of the column.

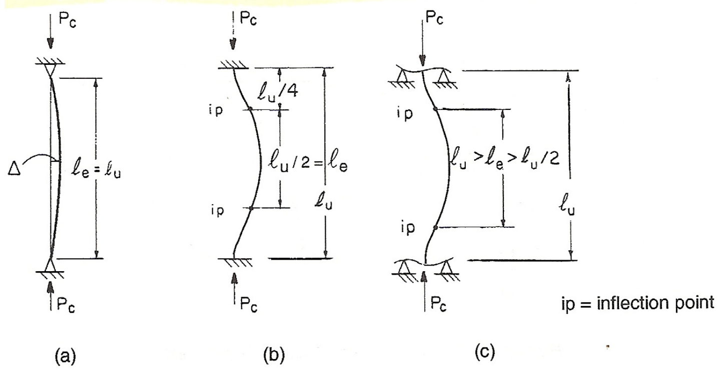


FIGURE 14a, b & c

A column that is fixed at one end and entirely free at the other end (cantilever) will buckle as shown in Figure 15a. The upper end will deflect laterally relative to the lower end. This condition is known as sidesway. The deflected shape of such a member is similar to one-half of the sinusoidal deflected shape of the pin-ended member illustrated in Figure 14a. Therefore the effective length is equal to twice the actual length. If the column is fixed against rotation at both ends but one end can move laterally with respect to the other, it will buckle as shown in Figure 15 b. The effective length ℓ_e will be equal to the actual length ℓ_u with an inflection point (ip) occurring as shown. The buckling load of the column in Figure 15b, where sidesway is not prevented, is one-quarter that of the column shown in Figure 14b (where sideway is prevented). As previously noted, the ends of columns are rarely either completely hinged or completely fixed, but instead are partially restrained against rotation by the members framing into the ends of the columns. Therefore, the effective length will vary between ℓ_u and infinity, as shown in Figure 15c. If the restraining members (beam or slab) are very rigid as compared to the column, the buckling illustrated in Figure 15b is approached. If however, the restraining members are quite flexible, a hinged condition is approached at both ends and the columns and possibly the structure as a whole, approaches instability. In general, the effective length ℓ_e depends on the degree of rotational restraint at the ends of the column, such that $\ell_u < \ell_e < \infty$.

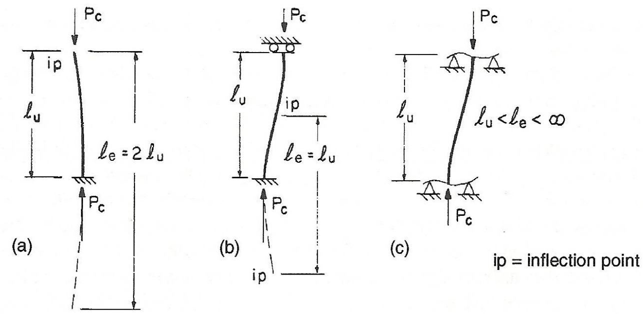


FIGURE 15a, b & c

In typical reinforced concrete structures, the design is rarely concerned with single members and instead focuses on the entire rigid framing system consisting of beam-column and slab-column assemblies. The buckling behavior of a frame that is not braced against sidesway can be illustrated by the simple portal frame shown in Figure 16. Without lateral restraint at the upper end, the entire (unbraced) frame is free to move sideways. The bottom end may be pinned or partially restrained against rotation.

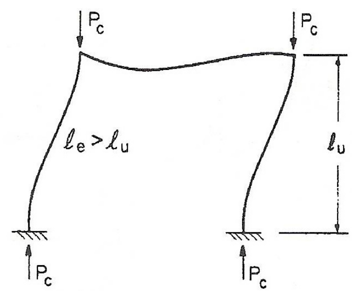


FIGURE 16

In summary:

1. For compression members in a non-sway frame, the effective length ℓ_e falls between $\ell_u/2$ and ℓ_u, where ℓ_u is the actual unsupported length of the column.

2. For compression members in a sway frame, the effective length ℓ_e is always greater than the actual length of the column ℓ_u and may be as great as $2\ell_u$ or higher. In this case, a value of k less than 1.2 normally would not be realistic.

The use of the alignment charts shown in Figures R10.12.1(a) and (b) in ACI 318-05 allows for a graphical determination of the effective length factors for compression members in non-sway and sway frames, respectively. If both ends of a column in a non-sway frame have minimal rotational stiffness, or approach $\psi = \infty$, then k = 1.0. If both ends in a non-sway frame have or approach full fixity $\psi = 0$ and k = 0.5. If both ends of a column in a sway frame have minimal rotational stiffness, or approach $\psi = \infty$, then $k = \infty$. If both ends in a sway frame have or approach full fixity, $\psi = 0$, then k = 1.0.

In actual structures, there is rarely a completely non-sway or sway condition. If it is not readily apparent by inspection whether the structure is sway or non-sway, Sections 10.11.4.1. and 10.11.4.2 provided methods of determining if a frame is non-sway or not. According to Section 10.11.4.1, a column in a structure can be considered non-sway if the column end moments due to second-order effects do not exceed 5% of the first-order end moments. According to Section 10.11.4.2, it is also permitted to assume a story within a structure is non-sway if:

$$Q = (\Sigma P_u \Delta_o)/(V_{us} \ell_c) \leq 0.05$$

Where; Q = The stability index for a story

ΣP_u = The total factored vertical load in the story corresponding to the lateral loading case for which ΣP_u is the greatest (see Section R10.11.4)

V_{us} = The factored horizontal story shear

Δ_o = The first-order relative deflection between the top and bottom of the story

ℓ_c = The column length measured from center-to-center of the joints in the frame.

When biaxial bending occurs in a column, the computed moments about each of the principal exes must be magnified. The magnification factors, δ, are computed considering the buckling load P_c about each axis separately, based on the appropriate effective lengths and the related stiffness ratios of columns to beams in each direction. Thus, different buckling capacities about the two axes are reflected in different magnification factors. The moments about each of the two axes are magnified separately, and the cross-section is then proportioned for an axial load P_u and magnified biaxial moments.

For compression members in a non-sway frame, effects of slenderness may be neglected when $k\ell_u/r$ is less than or equal to $(34 – 12(M_1/M_2))$ where M_2 is the larger end moment and M_1 is the smaller end moment. The ratio is M_1/M_2 is positive if the column is bent in single curvature, negative if bent in double curvature. Note that M_1 may not be taken grater than 40. For compression members in a sway frame, effects of slenderness may be neglected when $k\ell_u/r$ is less than 22 (Section 10.13.2). The moment magnifier method may be used for columns with slenderness ratios exceeding these lower limits.

The upper slenderness limit for columns that may be designed by the approximate moment magnifier method is $k\ell_u/r$ equal to 100 (Section10.11.5). When $k\ell_u/r$ is greater than 100, an analysis as defined in Section 10.10.1 must be used, taking into account the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, the effect of deflections on the moments and forces, and the effects of duration of loading (sustained load effects). Criteria for consideration of column slenderness are summarized in Figure 12 above.

The lower slenderness ratio limits will allow a large number of columns to be exempt from slenderness consideration. Considering the slenderness ratio $k\ell_u/r$ in terms of ℓ_u/h for rectangular columns, the effects of slenderness may be neglected in design when ℓ_u/h is less than 10 for compression members in a non-sway frame with zero moments at both ends. This lower limit increases to 15 for a column in double curvature with equal end moments and a column-to-beam stiffness ratio equal to one at each end. For columns with minimal or zero restraint at both ends, a value of k equal to 1.0 should be used.

For stocky columns restrained by flat slab floors, k values range from about 0.95 to 1.0 and can be conservatively estimated as 1.0. For columns in beam-column frames k values range from about 0.75 to 0.90, and can be conservatively estimated as 0.90. If the initial computation of the slenderness ratio based on estimated values of k indicates that effective slenderness must be considered in the design, a more accurate value of k should be calculated and the slenderness re-evaluated. For a compression member in a sway frame with a column-to beam stiffness ratio equal to 1.0 at both ends, effects of slenderness may be neglected when ℓ_u/h is less than 5. This value reduces to 3 if the beam stiffness is reduced to one-fifth of the column stiffness at each end of the column. Therefore, beam stiffnesses at the top and bottom of a column of a high-rise structure where sidesway is not prevented by structural walls or other means will have a significant effect on the degree of slenderness of the column.

The upper limit on the slenderness ration of $k\ell_u/r$ equal to 100 corresponds to an ℓ_u/h equal to 30 for a compression member in a non-sway frame with zero restraint at both ends. This ℓ_u/h limit increases to 39 for a column-to-beam stiffness ratio of 1.0 at each end.

Moment Magnification

The approximate slender column design equations provided in Section 10.12.3 for non-sway frames are based on the concept of the moment magnifier δ_{ns}, which amplifies the larger factored end moment M_2 on a compression member. The column is then designed for the factored axial load P_u and the amplified moment M_c; where $M_c = \delta_{ns}M_2$

$$\delta_{ns} = C_m/(1 - (P_u/(0.75P_c))) \geq 1.0$$

Where; $P_c = (\pi^2 EI)/(k\ell_u)^2$

$C_m = 0.6 + 0.4(M_1/M_2) \geq 0.4$; for members without transverse loads between supports.

The term C_m is an equivalent moment correction factor. For members with transverse loads between supports, it is possible that the maximum moment will occur at a section away from the ends of a member. In this case, the largest calculated moment occurring anywhere along the length of the members should be magnified by δ_{ns} where C_m is taken as 1.0. Figure 17 illustrates the values of C_m as a function of the end moments.

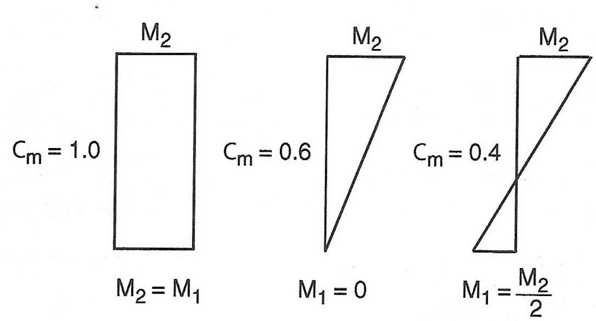


FIGURE 17

The critical load P_c is computed for a non-sway condition using an effective length factor k of 1.0 or less. When k is determined from the alignment charts or the equations in Section R10.12, the values of E and I from Section 10.11.1 must be used in the computations of ψ_A and ψ_B. Note that the 0.75 factor in the equation for δ_{ns} is a stiffness reduction factor (see Section R10.12.3).

In defining the critical column load P_c, the difficult problem is the choice of a stiffness parameter EI which reasonably approximates the stiffness variations due to cracking, creep, and the nonlinearity of the concrete stress-strain curve. In lieu of a more exact analysis, EI can be taken as:

$$EI = ((0.2E_cI_g) + (E_sI_{se}))/(1 + \beta_d); \text{ or } = (0.4E_cI_g)/(1 + \beta_d)$$

The second of the above two equations is a simplified approximation of the first. Both equations approximate the lower limits of EI for practical cross-sections and are therefore conservative values. The first equation for EI given above represents the lower limit of the practical range of stiffness values. This is especially true for heavily reinforced columns. The second equation is simpler to use but greatly underestimates the effect of reinforcement in heavily reinforced columns. Both EI equations were derived for small e/h values and high P_u/P_o values, where the effect of axial load is most pronounced. The term P_o is the nominal axial load strength at zero eccentricity.

For reinforced concrete columns subjected to sustained loads, creep of the concrete transfers some of the load from the concrete to the steel, therefore increasing the steel stresses. For lightly reinforced columns, this load transfer may cause compression steel to yield prematurely, resulting in a loss in the effective value of EI. This is taken into account by dividing EI by $(1 + \beta_d)$. For non-sway frames β_d is defined as the ratio of the maximum factored sustained axial load to the maximum factored axial load associated with the load combination (see Section 10.11.1).

If the computed column moment M_2 in the equation for M_c ($= \delta_{ns}M_2$) is small or zero, the design of a non-sway column must be based on the minimum moment $M_{2,min}$ per Section 10.12.3.2:

$$M_{2,min} = P_u(0.6 + 0.03h)$$

For members where $M_{2,min} > M_2$, the value of C_m shall either be taken equal to 1.0, or shall be computed by the equation; $0.6 + 0.4(M_1/M_2) \geq 0.4$, using the ratio of the actual computed end moments M_1 and M_2.

The design of sway frames for slenderness consists essentially of three steps:

1. The magnified sway moments δ_sM_s are computed in one of two ways:

    a. A second-order elastic frame analysis (see Section 10.13.4.1)

    b. An approximate second-order analysis (see Section 10.13.4.2)

2. The magnified sway moments δ_sM_s are added to the unmagnified non-sway moments M_{ns} at the end of the column, per Section 10.13.3, such that:

    $$M_1 = M_{1ns} + \delta_sM_{1s}$$

    $$M_2 = M_{2ns} + \delta_sM_{2s}$$

    The non-sway moments M_{1ns} and M_{2ns} are computed using a first-order elastic analysis.

3. If the column is slender and subjected to high axial loads, it must be checked to see whether moments at points between the column ends are larger than those at the ends. According to Section 10.13.5, this check is performed using the non-sway magnifier δ_{ns} with P_c computed assuming $k = 1.0$ or less.

As noted above, there are two different ways to compute the magnified sway moments δ_sM_s. If a second-order elastic analysis is used to compute δ_sM_s, the deflections must be representative of the stage immediately prior to the ultimate load. Therefore, the values of EI given in Section 10.11.1. must be used in the second-order analysis. Note that I must be divided by $(1 + \beta_d)$, where β_d is defined (for sway frames) as the ratio of the maximum factored sustained shear within a story to the maximum factored shear in that story (see Section 10.11.1). For wind or earthquake loads, $\beta_d = 0$. An example of a non-zero β_d is when members are subjected to sustained earth pressures.

Section 10.13.4.2. allows an approximate second-order analysis to determine $\delta_s M_s$. In this case, the solution of the infinite series that represents the iterative P-Δ analysis for second-order moments is:

$$\delta_s M_s = M_s/(1 - Q) \geq M_s$$

Note that the above equation closely predicts the second-order moments in a sway frame until δ_s exceed 1.5. For the case then $\delta_s > 1.5$, $\delta_s M_s$ must be computed using Sections 10.13.4.1. or 10.13.4.3 (an approximate moment magnifier method given in earlier ACI Codes). It is also important to note that the moment magnification in the columns farthest from the center of twist in a building subjected to significant torsional displacement may be underestimated by the moment magnifier procedure. A three-dimensional second-order analysis should be considered in such cases.

When the unmagnified non-sway moments at the ends of the column are added to the magnified sway moments at the same points, one of the resulting total end moments is usually the maximum moment in the column. However, for slender columns with high axial loads, the maximum moment may occur between the ends of the column. A simple way of determining if this situation occurs or not is given in Section 10.13.5. If the ℓ_u/r for an individual compression member is greater than…

$$35/(P_u/(f'_c A_g))^{1/2}$$

…the maximum moment will occur at the point between the ends of the column. In this case, M_2, (= M_{2ns} + $\delta_s M_{2s}$) must be magnified by the non-sway moment magnifier, δ_{ns}, given above. The column is then designed for the factor axial load P_u and the moment M_c, where M_c is computed as follows:

$$M_c = \delta_s M_{2s} = \delta_s(M_{2\,ns} + \delta_s M_{2s})$$

For sway frames, the possibility of sideway instability of the structure as a whole under factored gravity loads must also be investigated. This is checked in two different ways, depending on the method that is used in determining $\delta_s M_s$:

1. When $\delta_s M_s$ is computed by a second-order analysis (Section 10.13.4.1) the following expression must be satisfied:

   Second-Order Lateral Deflection/First-Order Lateral Deflections ≤ 2.5

   It should be noted that these deflections are based on the applied loading of $1.2P_D$ and $1.6P_L$ plus the factored lateral load. The frame should be analyzed twice for this set of applied loads. The first analysis should be a first-order analysis and the second should be a second-order analysis. The lateral load may be the actual lateral loads used in design or it may be a single lateral load applied to the top of the frame. In any case, the lateral loads should be large enough to give deflections that can be compared accurately.

2. When $\delta_s M_s$ is computed by the approximate second-order analysis (Section 10.13.4.2), then

$$Q = (\Sigma P_u \Delta_o)/(V_{us}\ell_c) \leq 0.60$$

   Where the value of Q is evaluated using $1.2P_D$ and $1.6P_L$. It should also be noted that the above expression is equivalent to $\delta_s = 2.5$. The values of V_{us} and Δ_o may be determined using the actual or any arbitrary set of lateral loads. The above stability check is satisfied if the value of Q computed in Section 10.11.4.2. is less than or equal to 0.2.

It is important to note that in each of the two cases above, β_d shall be taken as the ratio of the maximum factored sustained axial load to the maximum factored axial load.

The strength of a laterally unbraced frame is governed by the stability of the columns and by the degree of end restraint provided by the beams in the frame. If plastic hinges form in the restraining beams, the structure begins to approach a failure mechanism which drastically reduces its axial load capacity. Section 10.13.7 requires that the restraining flexural members (beams or slabs) have the capacity to resist the magnified column moments. The ability of the moment magnifier method to provide a good approximation of the actual magnified moments at the member ends in sway frames is a significant improvement over the reduction factor method for long columns prescribed in earlier ACI codes to account for member slenderness in design.

Axial Shortening:

Axial shortening of concrete columns due to long-term creep and shrinkage occurs in tall reinforced concrete buildings. Long-term shortening of columns can have detrimental effects on the horizontal framing elements and the attached architectural elements. Therefore it is important for the design engineer to be able to predict the magnitude of column shortening that can occur for a given building.

Walls

Simplified Design Approach:

Lateral Resistance

For buildings in the low to moderate height range, frame action alone is usually sufficient to provide adequate resistance to lateral loads, however, as indicated above the use of concrete shearwalls to avoid a sway-frame in low to mid-rise construction greatly simplifies the design and construction of the building columns. Structural shearwalls are extremely important, however, in high-rise buildings. If unaided by shearwalls, high-rise frames cannot be designed sufficiently to satisfy the strength requirements or acceptable lateral drift limits. Therefore, rigidly framed concrete buildings tend to be uneconomical beyond 10 to 12 stores in regions of high to moderate seismicity and 15 to 20 stories elsewhere.

If possible, shearwalls should be located so that the center of rigidity of the walls coincides with the line of action of the resultant wind loads or center of mass for seismic design. This is important because concrete floor systems act as rigid horizontal diaphragms and distribute the lateral loads to the vertical framing elements in proportion to their rigidities, therefore a shearwall arrangement in a building should be provided that will prevent torsional effects on the structure.

As previously emphasized, the analysis and design of the structural system for a building frame of moderate height can be simplified if structural shearwalls are sized to carry the entire lateral load. Members of the frame (columns and beams or slabs) can then be proportioned to resist the gravity loads only. Thus, for low rise buildings, neglecting the contribution of frame action in resisting lateral loads and assigning the total lateral load resistance to walls is a reasonable design approach. In contrast, frame-wall interaction must be considered in high-rise structures where the walls have a significant effect on the beams and columns in the upper stories where the frame must resist more than 100% of the story shears because of the interaction between the different deflected shapes of a cantilevered shearwall and the shear distortion of a rigid frame.

A structural engineer has to be able to distinguish between sway and non-sway frames when considering the use of a shearwall to simplify the lateral analysis and design of the building columns. This can be done by inspection by comparing the total lateral stiffness of the columns in a story to that of the shearwall elements. A compression member may be considered non-sway if it is located in a story in which the shearwalls have such substantial lateral stiffness to resist the lateral deflection of the story that any resulting deflection is not large enough to affect the column strength substantially. ACI 318-89 contained a simple criterion to establish whether shearwalls provide sufficient lateral bracing to qualify the frame as non-sway. In order to assume a non-sway condition, the shearwalls must have a total stiffness of at least six times the sum of the stiffnesses of all columns in a given direction within the story; $I_{walls} \geq 6I_{columns}$.

Shear Design

The design for horizontal shear forces (in the plane of the wall) can be critical for structural walls with small height-to-length ratios (i.e., walls in low-rise buildings). Special provisions for the design of shear in walls are given in Section 11.10. In addition, to shear, the flexural strength of the wall must also be considered (see discussion below).

Walls with the minimum amounts of vertical and horizontal reinforcement are usually the most economical to construct. If much more than the minimum amount of reinforcement is required to resist the factored shear forces, a change in wall size (length or thickness) should be considered. The amounts of vertical and horizontal reinforcement required for shear depends on the magnitude of the factored shear force, V_u. Table 5 summarizes the amounts of vertical and horizontal reinforcement required for shear in structural walls.

V_u	Horizontal Shear Reinforcement	Vertical Shear Reinforcement
$V_u \leq \phi V_c / 2$	$\rho_h = 0.0020$ for #5 and smaller $\rho_h = 0.0025$ for other bars	$\rho_v = 0.0012$ for #5 and smaller $\rho_v = 0.0015$ for other bars
$\phi V_c / 2 < V_u \leq \phi V_c$	$\rho_h = 0.0025$	$\rho_v = 0.0025$
$V_u > \phi V_c$	$\phi V_s = \phi A_v f_y d / s_2$ $\phi V_s + \phi V_c = V_u$ $\phi V_s + \phi V_c \leq 10\phi\sqrt{f'_c}\,h(0.8\ell_w)$	$\rho_v = 0.0025 + 0.5[2.5 - h_w / \ell_w)](\rho_h - 0.0025)$

TABLE 5

When the factored shear force is less than or equal to one-half the shear strength provided by concrete ($V_u > \Phi V_c/2$), minimum wall reinforcement (see Table 6) according to Section 14.3 must be provided. For walls, subjected to axial compressive forces, ΦV_c may be taken as $\Phi 2(f'_c)^{1/2}hd$, where h is the thickness of the wall, $d = 0.8\ell_w$ (Section 11.10.4), and ℓ_w is the length of the wall (Section 11.10.5). When the design shear force is more than one-half the shear strength provided by concrete ($V_u > \Phi V_c/2$) minimum shear reinforcement according to Section 11.10.9 must be provided. When the design shear force exceeds the concrete shear strength ($V_u > \Phi V_c$), horizontal shear reinforcement must be provided according to Equation 11-31. It should be noted that the vertical and horizontal reinforcement must not be less than that listed in Table 7.

Wall Thickness h (in.)	Vertical		Horizontal	
	Minimum A_s[a] ($in.^2/ft$)	Suggested Reinforcement	Minimum A_s[b] ($in.^2/ft$)	Suggested Reinforcement
6	0.09	#3 @ 15	0.14	#4 @ 16
8	0.12	#3 @ 11	0.19	#4 @ 12
10	0.14	#4 @ 16	0.24	#5 @ 15
12	0.17	#3 @ 15[c]	0.29	#4 @ 16[c]

[a]Minimum A_s/ft of wall = 0.0012(12)h = 0.0144h for #5 bars and less (ACI 14.3.2)
[b]Minimum A_s/ft of wall = 0.0020(12)h = 0.0240h for #5 bars and less (ACI 14.3.3)
[c]Two layers of reinforcement are required (ACI 14.3.4)

TABLE 6

Wall Thickness h (in.)	Vertical and Horizontal	
	Minimum A_s[a] ($in.^2/ft$)	Suggested Reinforcement
6	0.18	#4 @ 13
8	0.24	#4 @ 10
10	0.30	#5 @ 12
12	0.36	#4 @ 13[b]

[a]Minimum A_s/ft of wall = 0.0025(12)h = 0.03h (ACI 11.10.9)
[b]Two layers of reinforcement are required (ACI 14.3.4)

TABLE 7

For situations in which the design shear force exceeds the concrete shear strength ($V_u > \Phi V_c$), the same approach used for the design of beam shear can also be used for the design of the required horizontal shear reinforcement in walls, where:

$$\Phi V_s = \Phi((A_v F_y d)/s_2)$$

Where: A_v = the total area of horizontal reinforcement within a distance s_2, $\Phi = 0.75$, $f_y = 60$ ksi and $d = 0.8\ \ell_w$.

The required amount of vertical shear reinforcement in a wall is given by Equation 11-32:

$$\rho_v = 0.0025 + 0.5(2.5 - (h_w/\ell_w))(\rho_h - 0.0025)$$

Where: h_w = The total height of the wall

$\rho_v = A_{vn}/S_1 h$

$\rho_h = A_{vh}/S_2 h$

When the wall height-to-length ratio h_w/ℓ_w is less than 0.5, the amount of vertical reinforcement is equal to the amount of horizontal reinforcement (Section 11.10.9.4).

Flexural Design

As stated above, for buildings of moderate height, walls with uniform cross-sections and uniformly distributed vertical and horizontal reinforcement are usually the most economical to construct. The concentration of reinforcement at the extreme ends of a wall (or wall segment) is usually not required except in high and moderate seismic zones and high rise buildings where wind or seismic controls. Therefore, the uniform distribution of vertical wall reinforcement required for shear will usually provide adequate moment strength as well.

In general, walls that are subjected to axial loads or combined flexural and axial loads need to be designed as compression members according to the provisions given in ACI Chapter 10. For rectangular shearwalls containing uniformly distributed vertical reinforcement and subjected to an axial load smaller than that producing balanced failure, the following approximate equation can be used to determine the nominal moment capacity of the wall:

$$\Phi M_n = \Phi(0.5A_{st}f_y\ell_w(1 + (P_u/(A_{st}f_y)))(1 - (c/\ell_w)))$$

Where: A_{st} = The total area of vertical reinforcement; in^2

ℓ_w = The horizontal length of wall; inches

P_u = The factored axial compression load; kips

f_y = The yield strength of the vertical reinforcement = 60 ksi

$c/\ell_w = (\omega + \alpha)/(2\omega + 0.85\beta_1)$; where $\omega = (A_{st}/(\ell_w h))(f_y/f'_c)$

$\alpha = P_u/(\ell_w h\ f'_c)$

Empirical and Alternate Design Methods

Section 14.5 of ACI 318-05 contains the Empirical Design Method which applies to walls of solid rectangular cross-section with resultant loads for all applicable load combinations falling within the middle third of the wall thickness at all sections along the height of the wall (i.e. $e \leq h/6$). Minimum thicknesses of walls designed by this method are specified in Section 14.5.3. Walls of nonrectangular cross-section, such as ribbed walls panels, must be designed by the provisions of Section 14.4, or if applicable, 14.8.

The design axial strength of a wall that satisfies the limitations of Section 14.5.1 can be found using the following equation:

$$\Phi P_n = 0.55\Phi f'_c A_g(1 - ((k\ell_c)/(32h))^2)$$

Where: k = 0.8 (wall restrained against rotation at one or both ends; top, bottom or both)

k = 1.0 (wall unrestrained against rotation at each end)

k = 2.0 (wall for braced against lateral translation)

In order to use the above equation, the wall thickness, h, must not be less than 1/25 times the supported length or height, whichever is shorter, or less than 4 inches (Section 14.5.3.1). Exterior basement walls and foundation walls must be at least 7½ inches thick (Section14.5.3.2).

It should be noted that for a wall subjected to a series of point loads, the horizontal length of the wall that is considered effective for each concentrated load is the least of the center-to-center distance between loads and width of bearing plus four times the wall thickness (Section 14.2.4). Columns built integrally with walls shall conform to Sections 10.8.2 and 14.2.5. Walls shall be properly anchored into all intersecting elements, such as floors, columns, other walls, and footings (Section 14.2.6).

Section 14.8 contains the provisions of the Alternate Design Method, which are applicable to simply supported, axially loaded members subjected to out-of-plane uniform lateral loads, with maximum moments and deflections occurring at mid-height. Also, the wall cross-section must be constant over the height of the panel. No minimum wall thicknesses are prescribed for walls designed by this method.

The Alternate Design Method for walls has appeared in the Uniform Building Code (UBC) since 1988, and was also included in the 2003 International Building Code (IBC) It is important to note that the provisions of Section 14.8 differ from those in the UBC and IBC as listed below. A further discussion of these differences will be provided under Tilt-Up Walls.

1. The nomenclature and wording has been changed to comply with ACI 318 style.

2. The procedure has been limited to out-of-plane, flexural effects on simply supported wall panels with maximum moments and deflections occurring at midspan, and…

3. The procedure has been made as compatible as possible with the provisions of Section 9.5.2.3 for obtaining the cracking moment and the effective moment of inertia.

According to Section 14.8.1, the provisions of Section 14.8 are considered to satisfy Section 10.10 when flexural tension controls the design of a wall. The following limitations apply to the Alternate Design Method (Section 14.8.2):

1. The wall panel shall be simply supported, axially loaded, and subjected to an out-of-plane uniform lateral load. The maximum moments and deflections shall occur at the mid-height of the wall (Section 14.8.2.1).

2. The cross-section is constant over the height of the panel (Section 14.8.2.2).

3. The wall cross sections shall be tension-controlled.

4. Reinforcement shall provide a design moment strength ΦM_n greater than or equal to M_{cr}, where M_{cr} is the moment causing flexural cracking due to the applied lateral and vertical loads. It should be noted that M_{cr} shall be obtained using the modulus of rupture (f_r) given by Equations 9-10 (Section 14.8.2.4).

5. The concentrated gravity loads applied to the wall above the design flexural section shall be distributed over a width equal to the lesser of:

   a. The bearing width plus a width on each side that increases at a slope of 2 (vertical) to 1 (horizontal) down to the design flexural section, or…

   b. The spacing of the concentrated loads. Also, the distribution width shall not extend beyond the edges of the wall panel (Section 14.8.2.5). See Figure 18.

6. The vertical stress (P_u/A_g) at the mid-height section shall not exceed $0.06f'_c$ (Section 14.8.2.6).

When one or more of the above conditions are not satisfied, the wall must be designed per the provisions of Section 14.4.

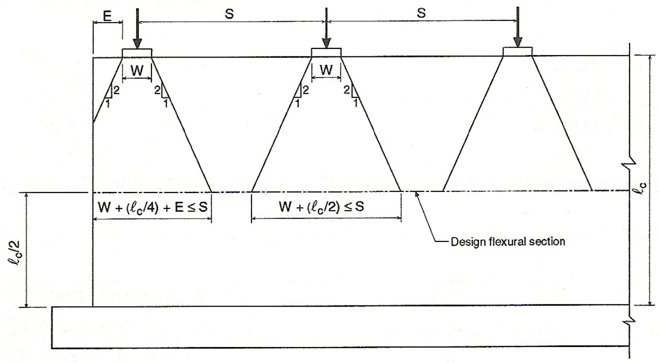


FIGURE 18

According to Section 14.8.3, the design moment strength ΦM_n for combined flexure and axial loads at the mid-height cross-section must be greater than or equal to the total factored moment M_u at this section. The factored moment M_u includes P-Δ effects and is defined as indicated below. It should be note that the equation below includes the effects of the factored axial loads and lateral load (M_{ua}), as well as the P-Δ effects ($P_u\Delta_u$).

$$M_u = M_{ua} + P_u\Delta_u$$

where M_{ua} = factored moment at the mid-height section of the wall due to factored lateral and eccentric vertical loads

P_u = factored axial load

Δ_u = deflection at the mid-height of the wall due to the factored loads

$= 5M_u \ell_c^2/(0.75)48E_cI_{cr}$

ℓ_c = vertical distance between supports

E_c = modulus of elasticity of concrete (8.5)

I_{cr} = moment of inertia of cracked section transformed to concrete

$= nA_{se}(d - c)^2 + (\ell_w c^3/3)$

n = modular ratio of elasticity = $E_s/E_c \geq 6$

E_s = modulus of elasticity of nonprestressed reinforcement

A_{se} = area of effective longitudinal tension reinforcement in the wall segment

$= (P_u + A_s f_y)/f_y$

A_s = area of longitudinal tension reinforcement in the wall segment

f_y = specified yield stress of nonprestressed reinforcement

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

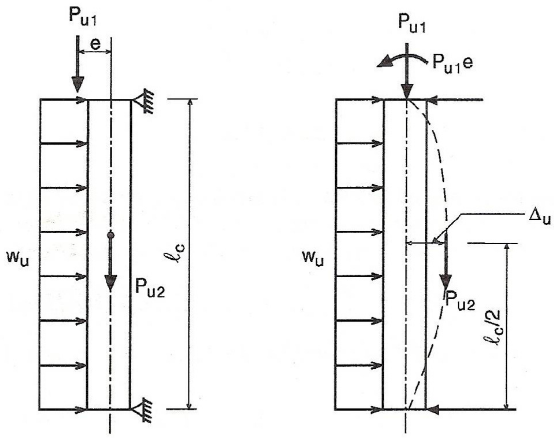
c = distance from extreme compression fiber to neutral axis

ℓ_w = horizontal length of the wall

Substituting the definition for Δ_u into the above equation results in:

$$M_u = M_{ua}/(1- ((5P_u\ell_c^2)/(0.75(48E_cI_{cr}))))$$

Figure 19 shows the analysis of the wall according to the provisions of Section 14.8 for the case of additive lateral and gravity load effects.



P_{u1} = factored applied gravity load
P_{u2} = factored self-weight of the wall (total)
e = eccentricity of applied gravity load
w_u = factored uniform lateral load

$$P_u = P_{u1} + \frac{P_{u2}}{2}$$

$$M_u = M_{ua} + P_u\,\Delta_u = \frac{M_{ua}}{1 - \dfrac{5P_u\ell_c^2}{(0.75)48E_cI_{cr}}}$$

$$M_{ua} = \frac{w_u\ell_c^2}{8} + \frac{P_{u1}e}{2}$$

$$M_u = \frac{w_u\ell_c^2}{8} + \frac{P_{u1}e}{2} + \left(P_{u1} + \frac{P_{u2}}{2}\right)\Delta_u$$

$$\Delta_u = \frac{5M_u\ell_c^2}{(0.75)48E_cI_{cr}}$$

FIGURE 19

The design moment strength of the above wall can be determined from the following equation:

$$\Phi M_n = \Phi A_{se}f_y(d - (a/2))$$

Where: $a = (A_{se}f_y)/(0.85f'_c\ell_w)$, and Φ is determined per Section 9.3.2.

In addition to satisfying the strength requirement of Equation 14-3, the deflection requirements of Section 14.8.4 must also be satisfied. In particular, the maximum deflection Δ_s due to service loads, including P-Δ effects, shall not exceed $\ell_c/150$; where Δ_s is:

$$\Delta_s = (5M\ell_c^2)/(48E_cI_e)$$

Where; M = The maximum unfactored moment due to service loads, including P-Δ effects

$M = M_{sa}/(1 - (5P_s\ell_c^2)/(48E_cI_e))$

M_{sa} = The maximum unfactored applied moment due to service loads, not including P-Δ effects

P_s = Unfactored axial load at the design (mid-height) section including effects of self weight.

I_e = Effective moment of inertia using the procedures required by Section 9.5.2.3, substituting M for M_a

It is important to note that above equation for M does not provide a closed form solution for M, since I_e is a function of M, therefore an iterative process is required to determine Δ_s.

Shearwalls

Two or more shear walls in the same plane (or two wall assemblies) are sometimes connected as floor levels by coupling beams so that the walls act as a unit when resisting lateral loads, as shown in Figure 20. This condition is referred to as coupled shearwalls. The coupling beams frame into the edges of the walls as shown in this same figure. Walls with more than two lines of openings, or walls with coupling beams arranged in an irregular fashion need special attention, especially if the widths and heights of the line or openings are irregular.

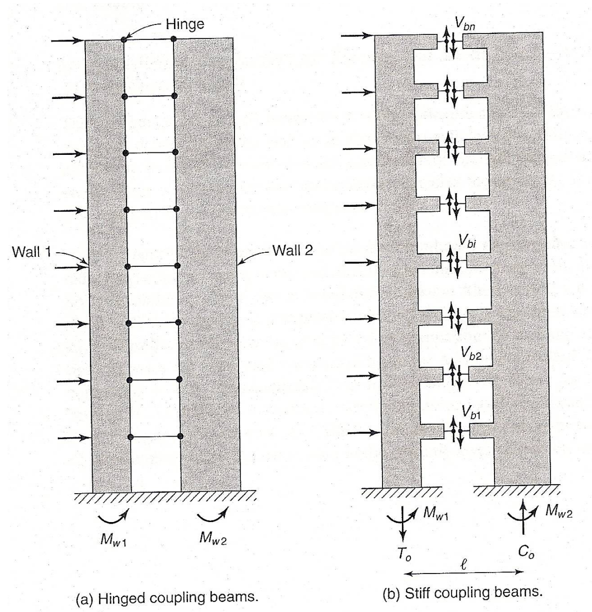


FIGURE 20

The behavior and strength of shearwalls separates them into two groups for design; slender shearwalls and squat shearwalls, as defined below.

1. Walls that extend $2\ell_w$ or more above the point of maximum moment in the wall resist lateral forces by flexural action and beam-like shear action. These are referred to as slender shearwalls. These types of walls should be designed according to ACI Chapters 10 and 11.

2. Walls that extend less than $2\ell_w$ above the point of maximum moment in the wall are assumed to resist lateral forces by strut-and-tie action. These are referred to as squat walls and are designed by using strut-and-tie models according to ACI Appendix A.

Minimum Wall Reinforcement

The minimum wall reinforcement provisions apply to walls designed according to Sections 14.4, 14.5, or 14.8, unless a greater amount is required to resist horizontal shear forces in the plane of the wall according to Section 11.10.9.

Walls must contain both vertical and horizontal reinforcement. The minimum ratio of vertical reinforcement area to gross concrete area is:

1. 0.0012 for deformed bars not larger than # 5 with $f_y \geq 60$ ksi, or for welded wire reinforcement (plain or deformed) not larger than W31 or D31, or…

2. 0.0015 for all other deformed bars (Section 14.3.2).

The minimum ratio of horizontal reinforcement is:

1. 0.0020 for deformed bars not larger than # 5 with $f_y \geq 60$ ksi, or for welded wire reinforcement (plain or deformed) not larger than W31 or D31, or…

2. 0.0025 for all other deformed bars. (Section 14.3.3).

The minimum wall reinforcement required by Section 14.3 is provided for control of cracking due to shrinkage and temperature stresses. Also, the minimum vertical wall reinforcement required by Section 14.3.2 does not substantially increase the strength of a wall above that of a plain concrete wall. It should be noted that the reinforcement and minimum thickness requirements of Sections 14.3 and 14.5.3 may be waived where structural analysis shows adequate strength and wall stability. This required condition may be satisfied by a design using the structural plain concrete provisions in Chapter 22 of the ACI Code.

For walls thicker than 10 inches, except for basement walls, reinforcement in each direction shall be placed in two layers (Section 14.3.4). The spacing of vertical and horizontal reinforcement shall not exceed 18 inches or three times the wall thickness (Section 14.3.5). According to Section 14.3.6, lateral ties for vertical reinforcement are not required as long as the vertical reinforcement is not required as compression reinforcement or the area of vertical reinforcement does not exceed 0.01 times the gross concrete area. A minimum of two #5 bars shall be provided around all window and door openings, with a minimum bar extension beyond the corner of opening equal to the greater of the bar development length or 24 inches (Section 14.3.7).

Walls Designed as Compression Members:

When the limitations of Sections 14.5 or 14.8 are not satisfied, walls must be designed as compression members by the strength design provisions in Chapter 10 for flexure and axial loads. The minimum reinforcement requirements of Section 14.3 apply to walls designed by this method. Vertical wall reinforcement need not be enclosed by lateral ties (as for columns) when the conditions of Sections 14.3.6 are satisfied. All other code provisions for compression members apply to walls designed by Chapter 10.

As with columns, the design of walls is usually difficult without the use of design aids. Wall design is further complicated by the fact that slenderness is a consideration in practically all cases. A second-order analysis, which takes into account variable wall stiffness as well as the effects of member curvature and lateral drift, duration of the loads, shrinkage and creep, and interaction with the supporting foundation, is specified in Section 10.10.1. In lieu of that procedure, the approximate evaluation of slenderness effects prescribed in Section 10.11 may be used (Section 10.10.2).

It is important to note that Equations 10-11 and 10-12 for EI in the approximate slenderness method were not originally derived for members with a single layer of reinforcement. For members with a single layer of reinforcement, the following expression for EI is recommended:

$$EI = \frac{E_c I_g}{\beta}\left(0.5 - \frac{e}{h}\right) \geq 0.1\,\frac{E_c I_g}{\beta}$$

$$\leq 0.4\,\frac{E_c I_g}{\beta}$$

where

E_c = modulus of elasticity of concrete
I_g = moment of inertia of gross concrete section about the centroidal axis, neglecting reinforcement
e = eccentricity of the axial loads and lateral forces for all applicable load combinations
h = overall thickness of wall
$\beta = 0.9 + 0.5\beta_d^2 - 12\rho \geq 1.0$
β_d = ratio of dead load to total load
ρ = ratio of area of vertical reinforcement to gross concrete area

The definition of β_d included in Equations 10-11 and 10-12 for EI, depends on whether the frame is non-sway or sway. According to Section10.0, β_d for non-sway frames is the ratio of the maximum factored axial sustained load to the maximum factored axial load associated with the same load combination. For consistency, the same definition of β_d seems appropriate for the EI expressions for walls in the equation above. It should be noted that if it is determined by the provisions of Section 10.11.4 that a sway condition exists, $\beta_d = 0$ for the case of lateral loads that are not sustained (Section 10.0).

For walls designed by Chapter 10 with the slenderness evaluated per Section 10.11, the above equation for EI is recommended in lieu of Equation 10-12 for determining wall stiffness. Example 21.1 (see Handout #6) illustrates this method for a tilt-up wall panel.

When the wall slenderness exceeds the limit for application of the approximate slenderness evaluation method of Section 10.11 (i.e. $k\ell_u/r > 100$; $k\ell_u/h > 30$) Section 10.10.1 must be used to determine the slenderness effects (Section 10.11.5). The wall panels currently used in some building systems, particularly tilt-up wall construction, usually fall in this high slenderness category. In this case the slenderness analysis must account for the influence of variable wall stiffness, the effects of deflections on the moments and forces and the effects of load duration.

Tilt-Up Wall Panels

Historical and Recent Developments:

Tilt-up concrete construction can be classified as site-cast precast. As stated in Chapter 1 of the ACI Tilt-Up Construction Guide (ACI 551), several features make the tilt-up construction method a unique form of precast concrete:

Tilt-up panels are generally handled only once. They are lifted or tilted from the casting slab and erected in their final position in one, continuous operations. Tilt-up panels are generally of such large size and weight that they can only be constructed on site and in close proximity to their final location in the structure. Panel gravity loads are supported directly by the foundation instead of being supported by a structural frame. Typically, tilt-up panels are erected before the structural frame. Tilt-up panels are usually load-bearing for gravity loads and lateral loads. Tilt-up concrete construction is a unique form of site-cast precast construction and, as such, has is own specialized set of design parameters and construction techniques.

The current popularity of tilt-up panels is even more remarkable considering that it wasn't until the 1980's that the building codes began to recognize the unique design of slender tilt-up wall panels. In fact, it wasn't until ACI 318-99 was published that the design of slender wall panels was codified on a national level. Prior to the development of slender wall code provisions, concrete wall thickness was controlled by limiting the height/thickness ratio. In the 1960's and l970's, the ACI 318 height/thickness ratio limit of 25 for bearing walls created much thicker walls that those typically seen today. For example, a common 20 feet high bearing wall was limited to a minimum thickness of 10 inches.

Engineers began experimenting with new analysis techniques that included second-order effects, or P-Δ moments, to avoid the height/thickness ratio limits prescribed by ACI. Many engineers used the moment magnification method in ACI 318-71 to account for these second-order effects, but this method was not applicable to flexural members with only a central layer of reinforcement. In response to the explosive growth of tilt-up construction being based on potentially misapplied code provisions, the Structural Engineers Association of Southern California (SEAOSC) published "Recommended Tilt-Up Wall Design", also known as the Yellow Book, in 1979. This publication provided detailed design examples to appropriately consider second-order effects in slender concrete walls. In addition, height/thickness ratio limits of 36 and 42 for unstiffened and stiffened bearing walls, respectively, allowed the use of unstiffened 7 inch thick walls and stiffened 6 inch thick walls that could reach 21 feet in height.

The Yellow Book was quickly followed by the Green Book, entitled "Test Report on Slender Walls", in 1982. Based on the work of SEAOSC and the Southern California Chapter of ACI (SCCACI), this publication contained the results of 30 full-scale slender wall tests subjected to out-of-plane loading. The tests were a dramatic success and showed that despite the high height/thickness ratios, the wall panels were quite capable of undergoing severe deflections while continuing to resist increasing lateral loads before yielding. One specimen didn't yield until deflecting 13 inches and reached its ultimate capacity after deflecting over 19 inches.

The very large deflections, however, raised serviceability concerns with the SEAOSC/SCCACI task committee. Slender walls designed to meet strength requirements alone independent of height/thickness ratio limits could be overly flexible, possibly resulting in permanent deformations, however, the tests did demonstrate that there was no validity for fixed height-to-thickness ratios. At the same time, however, it was determined that there was a need for deflection limits to control potential residual distortions in the panels after the application of service loads. Based on a limited rebound study and after much discussion, the SEAOSC/SCCACI task committee proposed a deflection limit of 1/100 of the height of the panel.

The UBC, however, decreased the deflection limit at service loads established by the Green Book to 1/150 of the height of the panel. Another important aspect of both the Green Book and UBC equations was defining M_{cr} based on a modulus of rupture; $f_r = 5(f'_c)^{1/2}$. This lesser value was only 2/3 of the traditional ACI 318 value of $f_r = 7.5(f'_c)^{1/2}$, however, this limit matched the empirical test data.

The UBC slender wall provisions were incorporated into ACI 318-99 to eliminate conflict with the 2000 IBC. Whereas the equations for determining the design moment were essentially the same, the service level deflection equations were significantly altered. These equations remain in ACI 318-05, Section 14.8.4, and are given as:

$$\Delta_s = (5M\ell_c^2)/(48E_cI_e); \text{ and } M = M_{sa}/(1 - (5P_s\ell_c^2)/(48E_cI_e))$$

The most significant difference between the UBC provisions and ACI 318-05 is the ACI Code's use of Branson's equation for I_e to account for the effect of a cracked moment of inertia instead of using the UBC bilinear load-deflection. In addition, the value for M_{cr} used in Branson's equation was set at the traditional ACI value. In light of these differences and in consideration of the fact that slender walls don't behave in accordance with the long-standing ACI deflection equations it appears that the current ACI 318-05 Code will likely be revised. It should also be noted that neither the Yellow Book, the Green Book, nor the SEAOSC Slender Wall Task Group Report discuss the lower cracking moment M_{cr} used by the ACI or the bilinear moment-deflection equation provided above.

Recent research has also identified significant limitations with Branson's equation for I_e when applied to thin concrete members with a central layer of steel. Branson's equation, first published in 1965, was based on large test beams with a ratio of gross inertia to cracked moment of inertia (I_g/I_{cr}) of 2.2. When this ratio exceeds a value of about 3 the use of Branson's equations leads to poor predictions of deflection. Slender concrete walls are far above this limit, with common ratios ranging from 15 to 25 for single layer reinforced walls and 6 to 12 for double layer reinforced walls, therefore the actual deflection is under predicted. The main culprit for this under prediction is the lack of proper consideration of tension stiffening in Branson's equation. Recommendations to replace Branson's equation with a more accurate equation incorporating tension stiffening effects have recently been proposed.

Comments obtained during the late 2007 development of ACI 318-08 concluded that the 1997 UBC equations match the test data well, but the ACI 318-02 equations (that were unchanged in ACI 318-05) did not correlate well with the test data and typically underestimate service load deflections. In fact a comparison between the test date and the current ACI 318-05 indicates that the cracking moment is overestimated by 26% on average. The revised equations proposed, and assumably included in the ACI 318-08, produce a moment-deflection curve that is identical to the UBC prediction and more closely matches the original test data but still conservatively underestimates M_{cr} by 16% on average.

Seismic Design Considerations:

A tilt-up building structural system is generally classified as a bearing wall system as defined by ASCE 7. ASCE 7, Table 12.2-1 (Design Coefficients and Factors for Seismic Force-Resisting Systems), prescribes three categories under the heading of bearing wall systems that potentially apply to tilt-up buildings as a form of precast concrete construction in seismic regions. These categories are; Ordinary Precast Shear Walls, Intermediate Precast Shear Walls and Special Reinforced Concrete Shear Walls. Per ACI 318-05 tilt-up panels can be considered either Intermediate or Special Reinforced Concrete Structural Walls.

As indicated above, tilt-up walls in seismic regions must follow Intermediate and Special Precast Structural Walls provisions of ACI 318-05 Section 21.13.2 for the connections between wall panels, or between wall panels and the foundation, where yielding shall be restricted to steel elements or reinforcement. In addition, Section 21.13.3 states that elements of the connection that are designed to yield shall develop at least $1.5S_y$, where S_y is the yield strength of the connection.

Some designers interpret Section 21.13.2 to mean that tilt-up panels used as Intermediate or Special Precast Structural Walls must be connected to the foundation. In addition, the Commentary for Section 21.13 states that connections between precast wall panels or between wall panels and the foundation are required to resist forces induced by earthquake motions and to provide for yielding in the vicinity of connections. However, most tilt-up panels, due to their large size and relatively low force levels, are stable as individual elements and do not require panel-to-panel connections or panel-to-footing connections to resist overturning (tension) forces due to earthquake forces. Therefore, tie down connections to the footings are typically not required since there is seldom if every any tension force calculated at the base of a panel. It is common, however, to connect the panel into the adjacent slab on grade to assist in dissipating the horizontal, out-of-plane lateral load reactions.

Another reason for the above practice is recognized by Section 16.5.1.3(c) which states that when design forces result in no tension at the base, the ties required by 16.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab on grade. In other words, the seismic shear forces can be transferred between the tilt-up panels and the floor slab on grade, just as they are transmitted between a suspended structural concrete floor slab and a concrete structural wall.

General Guidelines:

The following items can be used as guidelines for the design and construction of tilt-up panels:

- Limit the heaviest panel on a project to about 40 tons (80,000 pounds). As a rule of thumb, the crane capacity should be about double the weight of the heaviest panel, hence the reason for the 40 ton limit as 80 ton capacity truck cranes are commonly used for tilt-up erection.

- The initial estimate of the required panel thickness (in inches) can be based on 1/4 of the unsupported height in feet.

- Leave at least 18 inches of solid panel between openings, and between an opening and the edge of a panel, preferably 2 feet if possible. This limitation is based on the fact that vertical and transverse loads applied to and above and below large openings in wall panels are transferred to the remaining vertical elements or jambs of the panel on each side of the opening. This rational and conservative design approach is based on the assumption that the vertical supports are made up of narrow column like strips of uniform width for the full height of the panel. These strips are then designed to transfer the additional loads to the footing or roof diaphragm above.

- Insulated, sandwich wall panels are commonly used. These types of tilt-up panels are cast in layers with rigid insulation between the two wythes of concrete. These types of panels are classified either as “composite” or “non-composite” depending on how the two faces or wythes of concrete are attached to each other through the insulation. Each type of system is described below:

    - For composite construction, the two layers of concrete are connected by steel ties, expanded metal strips, steel studs, or other shear transferring mechanism. These shear ties allow the three-layer panel to act structurally in unison, in other words compositely. This type of panel construction has caused problems because the exposed exterior surface is restrained by the shear ties and is therefore not able to move freely as a result of temperature changes, thereby causing cracking and bowing. As a result this type of sandwich panel is not as commonly used as non-composite construction.

    - As indicated above, non-composite construction is generally preferred. In this method the exterior wythe of concrete is thinner (usually 2 ½ inches) and serves only as a weather barrier. The sandwiched insulation is generally 2 inches thick, and usually polystyrene. The interior wythe of the panel serves as the structural layer, and is typically 4 to 7 inches thick depending on the structural requirements of the wall. The only tie between the interior and exterior concrete wythes is a series of thin carbon-fiber or glass-fiber sleeve anchors. These types of connectors allow free movement of the exterior wythe of concrete. For lifting (and in-place) stresses, the thicker interior wythe, which is cast face up, must be designed to carry all of the applied gravity (i.e. framing reactions and self weight of the interior wythe only for in-place loads, and self-weight of the entire panel for lifting) and lateral loads.

- The floor slab of a tilt-up building is especially important because it serve as a casting base for the panels and platform for the support of ready-mix trucks, and in some cases the truck cranes lifting the panels. As a result, the heaviest loads that the floor will be exposed to occur during the construction.

- The foundation system for a tilt-up building requires a few extra considerations not encountered in a conventional masonry, steel framed or wood framed building. The foundation system for a typical tilt-up building consists of continuous perimeter footings to support the wall panels. Wall panels can also be supported on isolated spread footings placed under joints between panels, so that each footing supports one-half of each adjacent panel and the panel spans between the footing pads, or in some cases drilled piers or pile caps. In either case, because the building slab must be cast first to provide a casting bed for the panels, and access to the footing must still be provided for erection, it is common to see the perimeter of the building slab held back approximately 4 feet from the inside face of the erected panel (and footing) to allow for the required construction sequence to occur unimpeded. The 4 feet strip of slab is later cast back as a “sidewalk” pour or “closure strip” (see Figure 21 below).

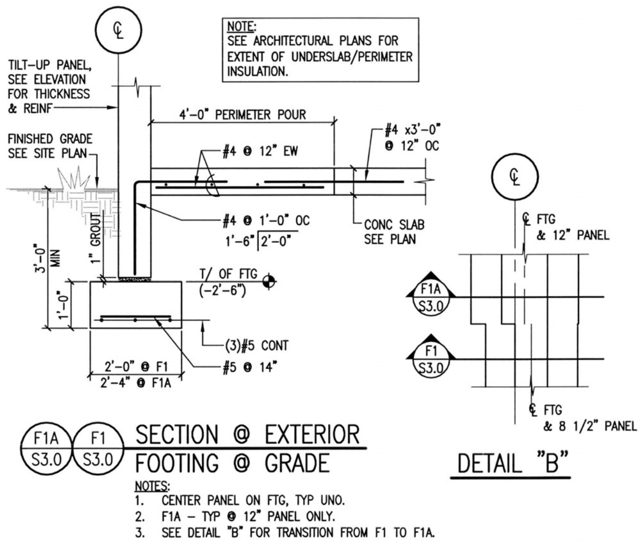


FIGURE 21

- Panel corners can be mitered or overlapped. See Figure 22 for an example of a mitered corner. It is common to connect the abutting panels at a building corner joint in order to prevent any differential bowing that might occur during erection or over the life of the building.

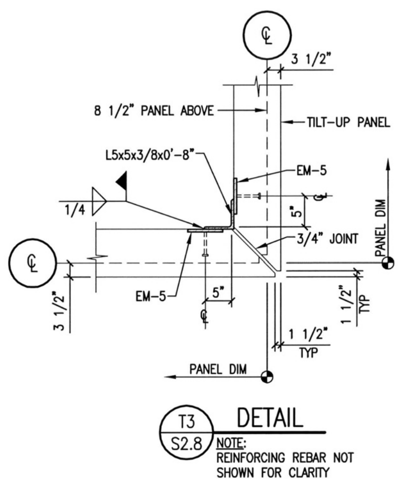


FIGURE 22

- It is common to use a panel thickness that is the same as the actual height of common 2x lumber, which is used as the side rail or form of the panel. Therefore using a 2x8 side rail will result in a panel thickness of 7¼ inches. At the same time it is also common to see 2x lumber built up on the top by nailing a strip of plywood so that a 2x8 and a ¾ inch strip of plywood can form a 8 inch thick panel.

- Although it is more common to cast panels on the building slab, in some instances it is necessary to construction temporary casting beds for the production of the panels. In addition, it is also common to cast the panels individually across the entire building slab, however, in some instances it is necessary to stack cast panels one on top of the other due to space restrictions of the project.

- ¾ inch minimum clear cover is the norm for tilt-up panel construction. This cover must account for any reveals in the panel, however, such that an 8½ inch panel for instance with a ¾ inch reveal will be designed as if it is only a 7¾ inch structural wall section.

- Typically the EOR is responsible for the in-place, dead, live and lateral load design of a tilt-up panel. A completely separate analysis, however, is also required for the lifting and temporary bracing of the panels during the stripping and erection process. This analysis is typically provided by the in-house engineers associated with the manufacturer of the specialty lifting inserts. This area of design and construction of tilt-up panels has been finally tuned over the years to allow for very efficient and economical production and erection of tilt-up panels for any given project. Meadow Burke is an excellent source of information on the erection and bracing of tilt-up panels. The following is a link to the Meadow Burke Tilt-Up Manual: http://www.meadowburke.com/products/tiltup.aspx

- Diaphragm deflections in a typical flexible metal roof deck for a tilt-up building will produce out-of-plane movement at the top of a panel. The amount of panel drift should be limited to 0.005 times the height of the building. For example, for a 26 feet high panel the maximum drift should be held to 1.5 inches. If the panels are fixed at the bottom (i.e. a resisting force couple provided by a slab and footing connection), to avoid cracking at the floor line the deflection or drift of the panel at the top should be limited to:

$$\Delta_{max} = H^2/(70t)$$

  Where: H = Building height in feet; t = Panel thickness in inches

- Other typical tilt-up panel details are provided below.

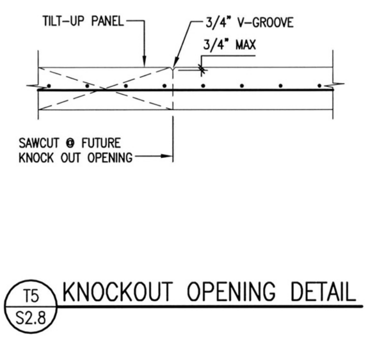


FIGURE 23

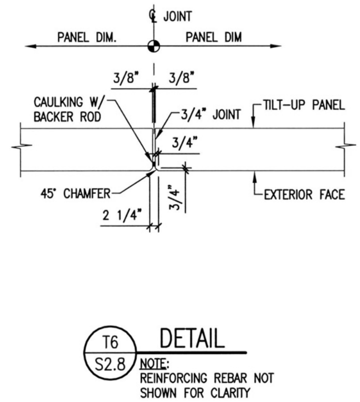


FIGURE 24

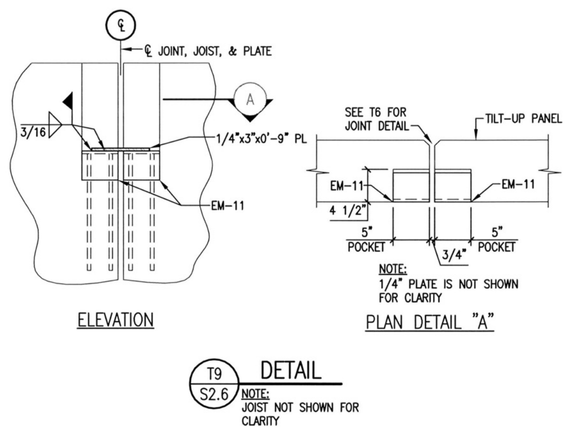


FIGURE 25

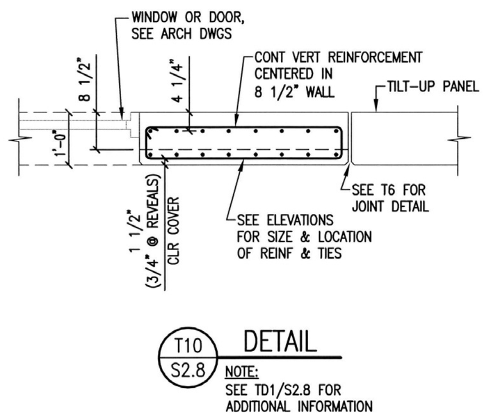


FIGURE 26

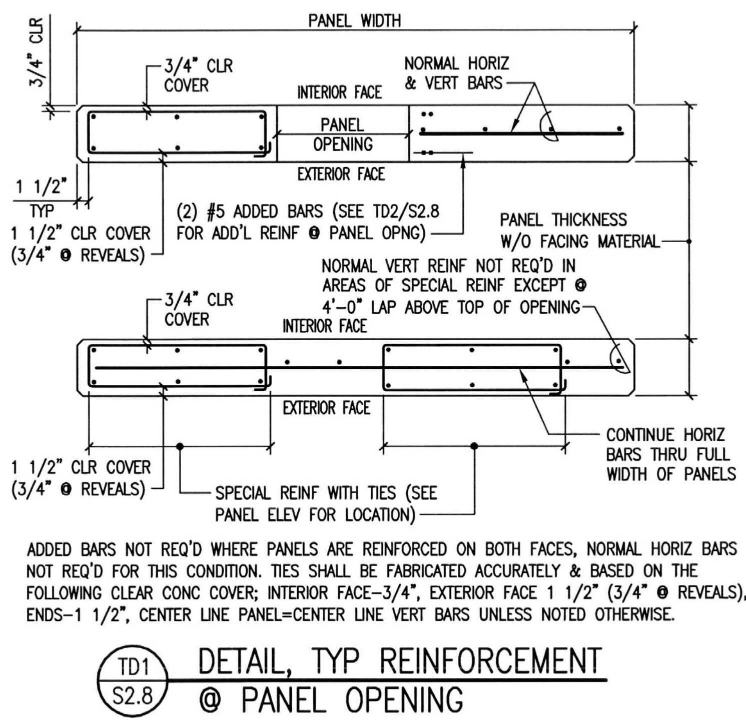
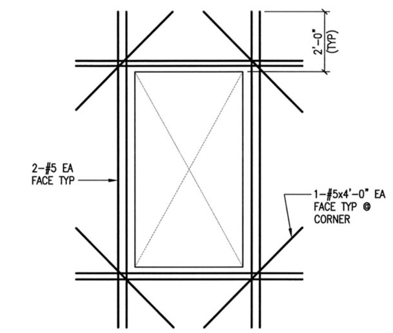
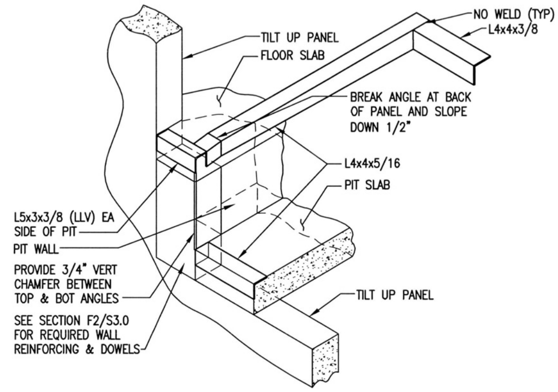


FIGURE 27



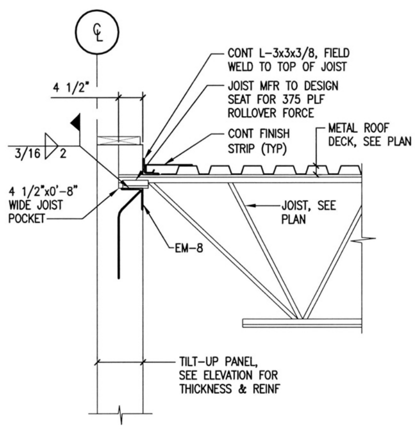
TD2 S2.8 DETAIL, TYP REINFORCEMENT @ PANEL OPENING

FIGURE 28



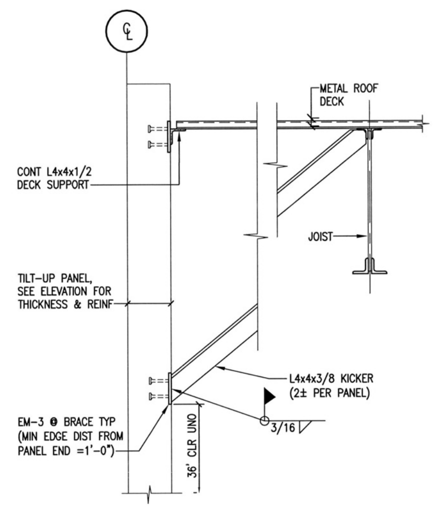
ISOMETRIC DETAIL "C" @ DOCK PIT

FIGURE 29



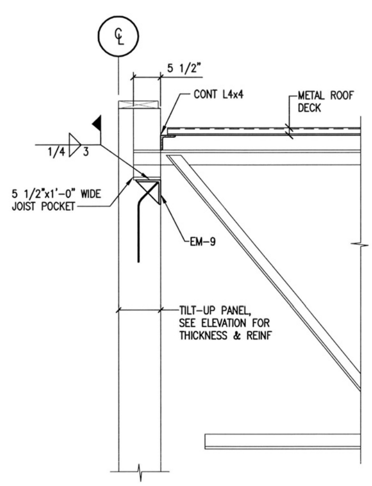
R10 S3.1 SECTION @ ROOF JOIST CONNECTION TO TILT-UP PANEL

FIGURE 30



R12 S3.1 SECTION @ ANGLE KICKER CONNECTION TO TILT-UP PANEL

FIGURE 31



R13 S3.1 SECTION @ JOIST GIRDER CONNECTION TO TILT-UP PANEL

FIGURE 32

Design:

The primary concern for the design of a tall, slender tilt-up panels is lateral instability and buckling due to lateral (earthquake and wind) out-of-plane loading plus the simultaneous effects of vertical loads. The resulting deflection of the panel must also be considered since any excessive deflection of the panel can be undesirable even if not restricted by code. In additional to the out-of-plane forces, the vertical gravity loads can act eccentrically to further increase bending. This phenomenon is referred to as the P-Δ effect, which has already been discussed in great detail for walls previously in this lecture.

In-plane shear forces (that is, parallel to the wall) from roof and floor diaphragms must also be accounted for. Typically these forces are transferred into the panel via connections at the roof and floor and are transmitted down through the panel to the ground via shearwall action. The design of tilt-up panels as shearwalls involves checking shear stresses, overturning, uplift resistance (if required), drag strut connections, collector connections, and other related considerations. Typically each panel is designed as a stand alone shearwall, however, if the shear forces become too large and exceed 1.5 times the uplift resistance of the panel (which would require tension uplift anchors) it is common to join two or more adjacent panels together via connections at the abutting panel joints.

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