6

REINFORCED CONCRETE

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In this chapter, a 12-story reinforced concrete office building with some retail shops on the first floor is designed for both high and moderate seismic loadings. For the more extreme loading, it is assumed that the structure will be located in Berkeley, California, and for the moderate loading, in Honolulu, Hawaii.

Figure 6-1 shows the basic structural configuration for each location in plan view and Figure 6-2, in section. The building, to be constructed primarily from sand-lightweight (LW) aggregate concrete, has 12 stories above grade and one basement level. The typical bays are 30 ft long in the north-south (N-S) direction and either 40 ft or 20 ft long in the east-west (E-W) direction. The main gravity framing system consists of seven continuous 30-ft spans of pan joists. These joists are spaced 36 in. on center and have an average web thickness of 6 in. and a depth below slab of 16 in. Due to fire code requirements, a 4-in.-thick floor slab is used, giving the joists a total depth of 20 in.

The joists along Gridlines 2 through 7 are supported by variable depth "haunched" girders spanning 40 ft in the exterior bays and 20 ft in the interior bays. The girders are haunched to accommodate mechanical-electrical systems. The girders are not haunched on exterior Gridlines 1 and 8, and the 40-ft spans have been divided into two equal parts forming a total of five spans of 20 ft. The girders along all spans of Gridlines A and D are of constant depth, but along Gridlines B and C, the depth of the end bay girders has been reduced to allow for the passage of mechanical systems.

Normal weight (NW) concrete walls are located around the entire perimeter of the basement level. NW concrete also is used for the first (ground) floor framing and, as described later, for the lower levels of the structural walls in the Berkeley building.

For both locations, the seismic-force-resisting system in the N-S direction consists of four 7-bay moment-resisting frames. The interior frames differ from the exterior frames only in the end bays where the girders are of reduced depth. At the Berkeley location, these frames are detailed as special moment-resisting frames. Due to the lower seismicity and lower demand for system ductility, the frames of the Honolulu building are detailed as intermediate moment-resisting frames.

In the E-W direction, the seismic-force-resisting system for the Berkeley building is a dual system composed of a combination of frames and frame-walls (walls integrated into a moment-resisting frame). Along Gridlines 1 and 8, the frames have five 20-ft bays with constant depth girders. Along Gridlines 2 and 7, the frames consist of two exterior 40-ft bays and one 20-ft interior bay. The girders in each span are of variable depth as described earlier. At Gridlines 3, 4, 5 and 6, the interior bay has been filled with a shear panel and the exterior bays consist of 40-ft-long haunched girders. For the Honolulu building, the structural walls are not necessary so E-W seismic resistance is supplied by the moment frames along Gridlines 1 through 8. The frames on Gridlines 1 and 8 are five-bay frames and those on Gridlines 2 through 7 are three-bay frames with the exterior bays having a 40-ft span and the interior bay having a 20-ft span. Hereafter, frames are referred to by their gridline designation (e.g., Frame 1 is located on

Gridline 1). It is assumed that the structure for both the Berkeley and Honolulu locations is founded on very dense soil (shear wave velocity of approximately 2000 ft/sec).

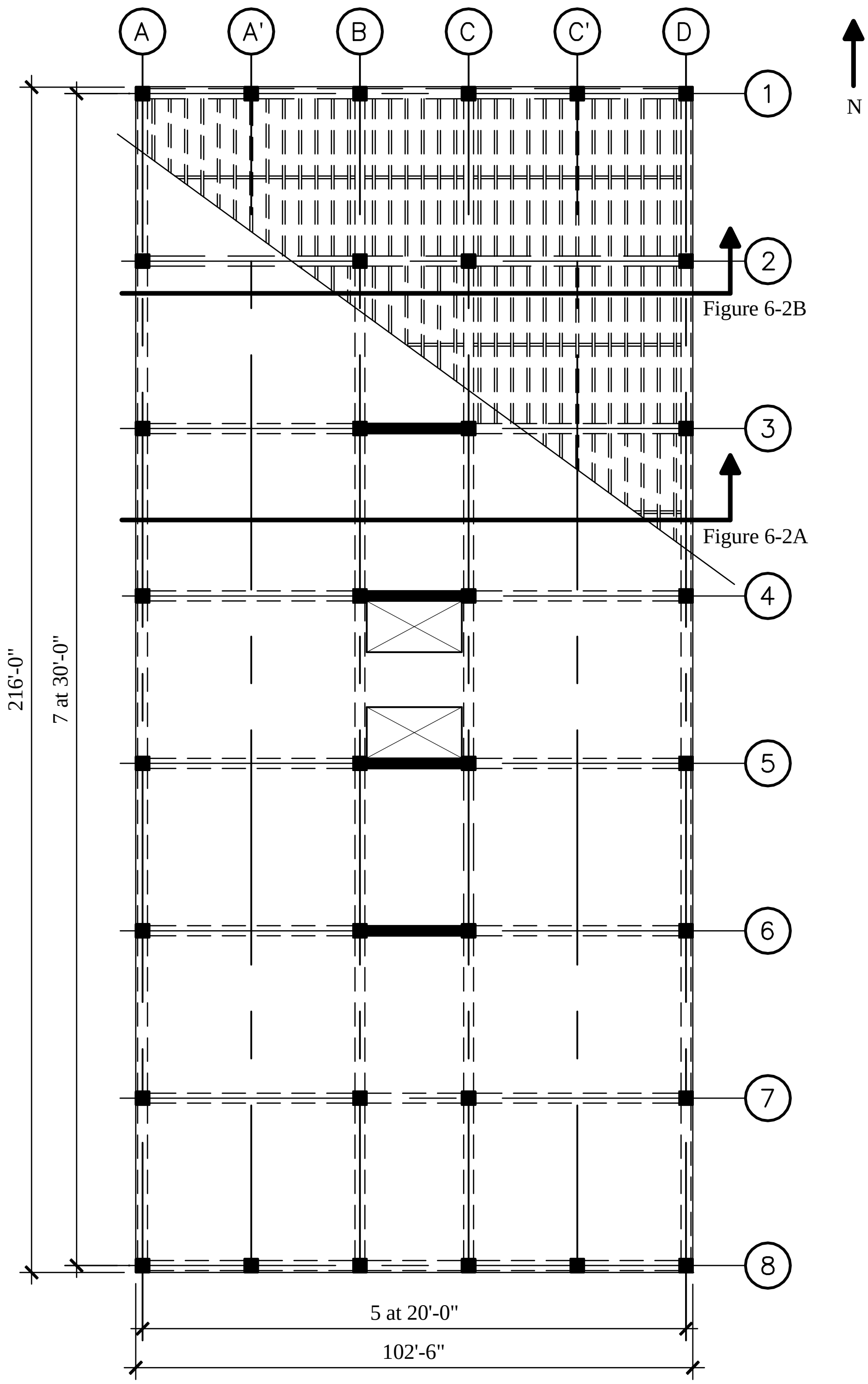
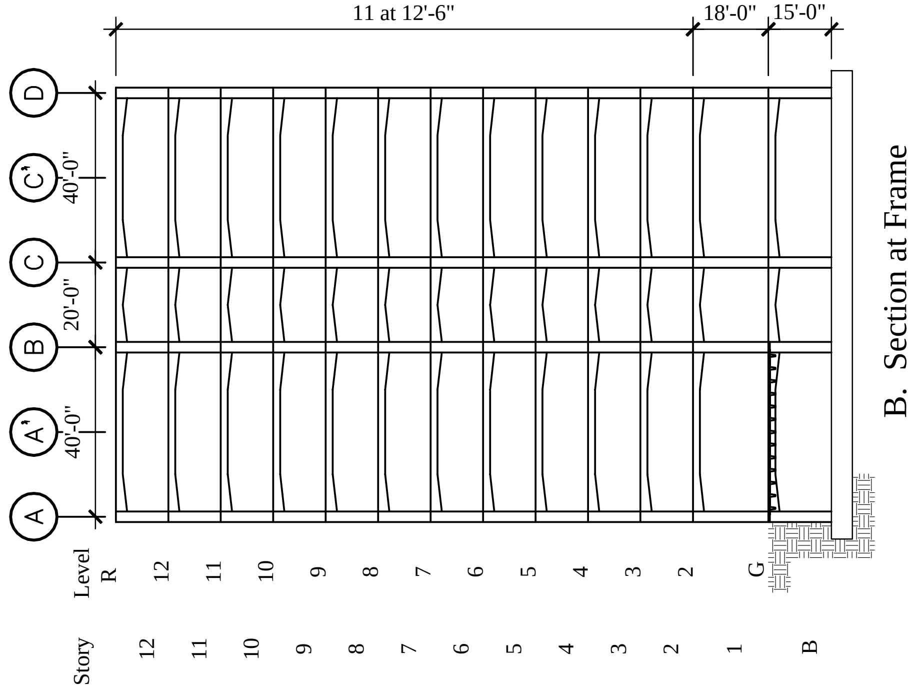


Figure 6-1 Typical floor plan of the Berkeley building. The Honolulu building is similar but without structural walls (1.0 ft = 0.3048 m).



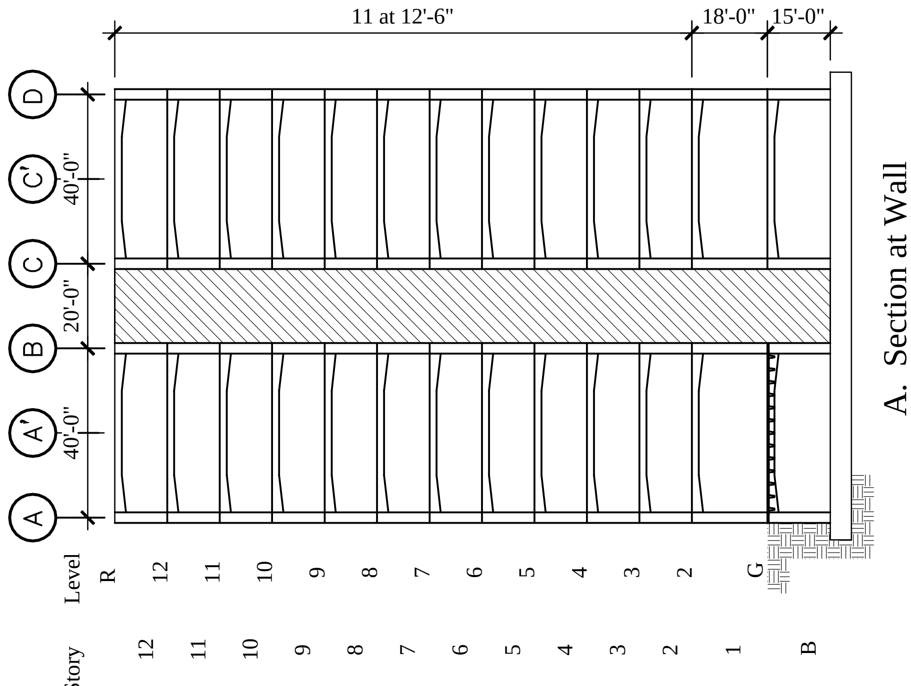


Figure 6-2 Typical elevations of the Berkeley building; the Honolulu building is similar but without structural walls (1.0 ft = 0.3048 m).

The calculations herein are intended to provide a reference for the direct application of the design requirements presented in the 2000 *NEHRP Recommended Provisions* (hereafter, the *Provisions*) and to assist the reader in developing a better understanding of the principles behind the *Provisions*.

Because a single building configuration is designed for both high and moderate levels of seismicity, two different sets of calculations are required. Instead of providing one full set of calculations for the Berkeley building and then another for the Honolulu building, portions of the calculations are presented in parallel. For example, the development of seismic forces for the Berkeley and Honolulu buildings are presented before structural design is considered for either building. The full design then is given for the Berkeley building followed by the design of the Honolulu building. Each major section (development of forces, structural design, etc.) is followed by discussion. In this context, the following portions of the design process are presented in varying amounts of detail for each structure:

1. Development and computation of seismic forces;
2. Structural analysis and interpretation of structural behavior;
3. Design of structural members including typical girder in Frame 1, typical interior column in Frame 1, typical beam-column joint in Frame 1, typical girder in Frame 3, typical exterior column in Frame 3, typical beam-column joint in Frame 3, boundary elements of structural wall (Berkeley building only) and panel of structural wall (Berkeley building only).

The design presented represents the first cycle of an iterative design process based on the equivalent lateral force (ELF) procedure according to *Provisions* Chapter 5. For final design, the *Provisions* may require that a modal response spectrum analysis or time history analysis be used. The decision to use more advanced analysis can not be made a priori because several calculations are required that cannot be completed without a preliminary design. Hence, the preliminary design based on an ELF analysis is a natural place to start. The ELF analysis is useful even if the final design is based on a more sophisticated analysis (e.g., forces from an ELF analysis are used to apply accidental torsion and to scale the results from the more advanced analysis and are useful as a check on a modal response spectrum or time-history analysis).

In addition to the *Provisions*, ACI 318 is the other main reference in this example. Except for very minor exceptions, the seismic-force-resisting system design requirements of ACI 318 have been adopted in their entirety by the *Provisions*. Cases where requirements of the *Provisions* and ACI 318 differ are pointed out as they occur. ASCE 7 is cited when discussions involve live load reduction, wind load, and load combinations.

Other recent works related to earthquake resistant design of reinforced concrete buildings include:

ACI 318 — American Concrete Institute. 1999 [2002]. *Building Code Requirements and Commentary for Structural Concrete*.

ASCE 7 — American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures*.

Fanella — Fanella, D.A., and M. Munshi. 1997. *Design of Low-Rise Concrete Buildings for Earthquake Forces*, 2nd Edition. Portland Cement Association, Skokie, Illinois.

ACI 318 Notes — Fanella, D.A., J. A. Munshi, and B. G. Rabbat, Editors. 1999. *Notes on ACI 318-99 Building Code Requirements for Structural Concrete with Design Applications*. Portland Cement Association, Skokie, Illinois.

ACI SP127	Ghosh, S. K., Editor. 1991. *Earthquake-Resistant Concrete Structures Inelastic Response and Design*, ACI SP127. American Concrete Institute, Detroit, Michigan.
Ghosh	Ghosh, S. K., A. W. Domel, and D. A. Fanella. 1995. *Design of Concrete Buildings for Earthquake and Wind Forces*, 2nd Edition. Portland Cement Association, Skokie, Illinois.
Paulay	Paulay, T., and M. J. N. Priestley. 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons, New York.

The Portland Cement Association's notes on ACI 318 contain an excellent discussion of the principles behind the ACI 318 design requirements and an example of the design and detailing of a frame-wall structure. The notes are based on the requirements of the 1997 *Uniform Building Code* (International Conference of Building Officials) instead of the *Provisions*. The other publications cited above provide additional background for the design of earthquake-resistant reinforced concrete structures.

Most of the large-scale structural analysis for this chapter was carried out using the ETABS Building Analysis Program developed by Computers and Structures, Inc., Berkeley, California. Smaller portions of the structure were modeled using the SAP2000 Finite Element Analysis Program, also developed by Computers and Structures. Column capacity and design curves were computed using Microsoft Excel, with some verification using the PCACOL program created and developed by the Portland Cement Association.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

The changes related to reinforced concrete in Chapter 9 of the 2003 *Provisions* are generally intended to maintaining compatibility between the *Provisions* and the ACI 318-02. Portions of the 2000 *Provisions* have been removed because they were incorporated into ACI 318-02. Other chances to Chapter 9 are related to precast concrete (as discussed in Chapter 7 of this volume of design examples).

Some general technical changes in the 2003 *Provisions* that relate to the calculations and/or design in this chapter include updated seismic hazard maps, revisions to the redundancy requirements, revisions to the minimum base shear equation, and revisions several of the system factors (R, Ω_0, C_d) for dual systems.

Where they affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

Note that these examples illustrate comparisons between seismic and wind loading for illustrative purposes. Wind load calculations are based on ASCE 7-98 as referenced in the 2000 *Provisions*, and there have not been any comparisons or annotations related to ASCE 7-02.

6.1 DEVELOPMENT OF SEISMIC LOADS AND DESIGN REQUIREMENTS

6.1.1 Seismicity

Using *Provisions* Maps 7 and 8 [Figures 3.3-3 and 3.3-4] for Berkeley, California, the short period and one-second period spectral response acceleration parameters S_S and S_1 are 1.65 and 0.68, respectively. [The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package.] For the very dense soil conditions, Site Class C is appropriate as described in *Provisions* Sec. 4.1.2.1 [3.5.1]. Using S_S = 1.65 and Site Class C, *Provisions* Table 4.1.2.4a [3.3-1] lists a short period site coefficient F_a of 1.0. For S_1 > 0.5 and Site Class C, *Provisions* Table 4.1.2.4b [3.3-2] gives a velocity based site coefficient F_v of 1.3. Using *Provisions* Eq. 4.1.2.4-1 and 4.1.2.4-2 [3.3-1 and 3.3-2], the maximum considered spectral response acceleration parameters for the Berkeley building are:

$S_{MS} = F_a S_S = 1.0 \text{ x } 1.65 = 1.65$
$S_{M1} = F_v S_1 = 1.3 \text{ x } 0.68 = 0.884$

The design spectral response acceleration parameters are given by *Provisions* Eq. 4.1.2.5-1 and 4.1.2.5-2 [3.3-3 and 3.3-4]:

$S_{DS} = (2/3)\, S_{MS} = (2/3)\, 1.65 = 1.10$
$S_{D1} = (2/3)\, S_{M1} = (2/3)\, 0.884 = 0.589$

The transition period (T_s) for the Berkeley response spectrum is:

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.589}{1.10} = 0.535 \text{ sec}$$

T_s is the period where the horizontal (constant acceleration) portion of the design response spectrum intersects the descending (constant velocity or acceleration inversely proportional to T) portion of the spectrum. It is used later in this example as a parameter in determining the type of analysis that is required for final design.

For Honolulu, *Provisions* Maps 19 and 20 [Figure 3.3-10] give the short-period and 1-sec period spectral response acceleration parameters of 0.61 and 0.178, respectively. For the very dense soil/firm rock site condition, the site is classified as Site Class C. Interpolating from *Provisions* Table 4.1.4.2a [3.3-1], the short-period site coefficient (F_a) is 1.16 and, from *Provisions* Table 4.1.2.4b [3.3-2], the interpolated long-period site coefficient (F_v) is 1.62. The maximum considered spectral response acceleration parameters for the Honolulu building are:

$S_{MS} = F_a S_S = 1.16 \text{ x } 0.61 = 0.708$
$S_{M1} = F_v S_1 = 1.62 \text{ x } 0.178 = 0.288$

and the design spectral response acceleration parameters are:

$S_{DS} = (2/3)\, S_{MS} = (2/3)\, 0.708 = 0.472$
$S_{D1} = (2/3)\, S_{M1} = (2/3)\, 0.288 = 0.192$

The transition period (T_s) for the Honolulu response spectrum is:

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.192}{0.472} = 0.407 \text{ sec}$$

6.1.2 Structural Design Requirements

According to *Provisions* Sec. 1.3 [1.2], both the Berkeley and the Honolulu buildings are classified as Seismic Use Group I. *Provisions* Table 1.4 [1.3] assigns an occupancy importance factor (I) of 1.0 to all Seismic Use Group I buildings.

According to *Provisions* Tables 4.2.1a and 4.2.1b [Tables 1.4-1 and 1.4-2], the Berkeley building is classified as Seismic Design Category D. The Honolulu building is classified as Seismic Design Category C because of the lower intensity ground motion.

The seismic-force-resisting systems for both the Berkeley and the Honolulu buildings consist of moment-resisting frames in the N-S direction. E-W loading is resisted by a dual frame-wall system in the Berkeley building and by a set of moment-resisting frames in the Honolulu building. For the Berkeley building, assigned to Seismic Design Category D, *Provisions* Sec. 9.1.1.3 [9.2.2.1.3] (which modifies language in the ACI 318 to conform to the *Provisions*) requires that all moment-resisting frames be designed and detailed as special moment frames. Similarly, *Provisions* Sec. 9.1.1.3 [9.2.2.1.3] requires the structural walls to be detailed as special reinforced concrete shear walls. For the Honolulu building assigned to Seismic Design Category C, *Provisions* Sec. 9.1.1.3 [9.2.2.1.3] allows the use of intermediate moment frames. According to *Provisions* Table 5.2.2 [4.3-1], neither of these structures violate height restrictions.

Provisions Table 5.2.2 [4.3-1] provides values for the response modification coefficient (R), the system over strength factor (Ω_0), and the deflection amplification factor (C_d) for each structural system type. The values determined for the Berkeley and Honolulu buildings are summarized in Table 6-1.

Table 6-1 Response Modification, Overstrength, and Deflection Amplification Coefficients for Structural Systems Used

Location	Response Direction	Building Frame Type	R	Ω_0	C_d
Berkeley	N-S	Special moment frame	8	3	5.5
	E-W	Dual system incorporating special moment frame and structural wall	8	2.5	6.5
Honolulu	N-S	Intermediate moment frame	5	3	4.5
	E-W	Intermediate moment frame	5	3	4.5

[For a dual system consisting of a special moment frame and special reinforced concrete shear walls, R = 7, Ω_0 = 2.5, and C_d = 5.5 in 2003 *Provisions* Table 4.3-1.]

For the Berkeley building dual system, the *Provisions* requires that the frame portion of the system be able to carry 25 percent of the total seismic force. As discussed below, this requires that a separate analysis of a frame-only system be carried out for loading in the E-W direction.

With regard to the response modification coefficients for the special and intermediate moment frames, it is important to note that R = 5.0 for the intermediate frame is 0.625 times the value for the special frame. This indicates that intermediate frames can be expected to deliver lower ductility than that supplied by the more stringently detailed special moment frames.

For the Berkeley system, the response modification coefficients are the same (R = 8) for the frame and frame-wall systems but are higher than the coefficient applicable to a special reinforced concrete structural wall system (R = 6). This provides an incentive for the engineer to opt for a frame-wall system under conditions where a frame acting alone may be too flexible or a wall acting alone cannot be proportioned due to excessively high overturning moments.

6.1.3 Structural Configuration

Based on the plan view of the building shown in Figure 6-1, the only possibility of a plan irregularity is a torsional irregularity (*Provisions* Table 5.2.3.2 [4.3-2]) of Type 1a or 1b. While the actual presence of such an irregularity cannot be determined without analysis, it appears unlikely for both the Berkeley and the Honolulu buildings because the lateral-force-resisting elements of both buildings are distributed evenly over the floor. For the purpose of this example, it is assumed (but verified later) that torsional irregularities do not exist.

As for the vertical irregularities listed in *Provisions* Table 5.2.3.3 [4.3-3], the presence of a soft or weak story cannot be determined without calculations based on an existing design. In this case, however, the first story is suspect, because its height of 18 ft is well in excess of the 12.5-ft height of the story above. As with the torsional irregularity, it is assumed (but verified later) that a vertical irregularity does not exist.

6.2 DETERMINATION OF SEISMIC FORCES

The determination of seismic forces requires knowledge of the magnitude and distribution of structural mass, the short period and long period response accelerations, the dynamic properties of the system, and the system response modification factor (R). Using *Provisions* Eq. 5.4.1 [5.2-1], the design base shear for the structure is:

$$V = C_S W$$

where W is the total (seismic) weight of the building and C_S is the seismic response coefficient. The upper limit on C_S is given by *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_S = \frac{S_{DS}}{R/I}$$

For intermediate response periods, Eq. 5.4.1.1-2 [5.2-3] controls:

$$C_S = \frac{S_{D1}}{T(R/I)}$$

However, the response coefficient must not be less than that given by Eq. 5.4.1.1-3 [changed in the 2003 *Provisions*]:

$$C_S = 0.044 S_{DS} I$$

Note that the above limit will apply when the structural period is greater than $S_{D1}/0.044RS_{DS}$. This limit is (0.589)/(0.044 x 8 x 1.1) = 1.52 sec for the Berkeley building and (0.192)/(0.044 x 5 x 0.472) = 1.85 sec for the Honolulu building. [The minimum C_s value is simply 0.01in the 2003 *Provisions*, which would not be applicable to this example as discussed below.]

In each of the above equations, the importance factor (*I*) is taken as 1.0. With the exception of the period of vibration (*T*), all of the other terms in previous equations have been defined and/or computed earlier in this chapter.

6.2.1 Approximate Period of Vibration

Requirements for the computation of building period are given in *Provisions* Sec. 5.4.2 [5.2.2]. For the preliminary design using the ELF procedure, the approximate period (T_a) computed in accordance with *Provisions* Eq. 5.4.2.1-1 [5.2-6] could be used:

$$T_a = C_r h_n^x$$

Because this formula is based on lower bound regression analysis of measured building response in California, it will generally result in periods that are lower (hence, more conservative for use in predicting base shear) than those computed from a more rigorous mathematical model. This is particularly true for buildings located in regions of lower seismicity. If a more rigorous analysis is carried out (using a computer), the resulting period may be too high due to a variety of possible modeling errors. Consequently, the *Provisions* places an upper limit on the period that can be used for design. The upper limit is $T = C_uT_a$ where C_u is provided in *Provisions* Table 5.4.2 [5.2-1].

For the N-S direction of the Berkeley building, the structure is a reinforced concrete moment-resisting frame and the approximate period is calculated according to *Provisions* Eq. 5.4.2.1-1 [5.2-6]. Using *Provisions* Table 5.4.2.1 [5.2-2], C_r = 0.016 and x = 0.9. With h_n = 155.5 ft, T_a = 1.50 sec. With S_{D1} > 0.40 for the Berkeley building, C_u = 1.4 and the upper limit on the analytical period is T = 1.4(1.5) = 2.1 sec.

For E-W seismic activity in Berkeley, the structure is a frame-wall system with C_r = 0.020 and x =0.75. Substituting the appropriate values in *Provisions* Eq. 5.4.2.1-1 [5.2-6], the E-W period T_a = 0.88 sec. The upper limit on the analytical period is (1.4)0.88 = 1.23 sec.

For the Honolulu building, the T_a = 1.5 sec period computed above for concrete moment frames is applicable in both the N-S and E-W direction. For Honolulu, S_{D1} is 0.192g and, from *Provisions* Table 5.4.2 [5.2-1], C_u can be taken as 1.52. The upper limit on the analytical period is T = 1.52(1.5) = 2.28 sec.

The period to be used in the ELF analysis will be in the range of T_a to C_uT_a. If an accurate analysis provides periods greater than C_uT_a, C_uT_a should be used. If the accurate analysis produces periods less than C_uT_a but greater than T_a, the period from the analysis should be used. Finally, if the accurate analysis produces periods less than T_a, T_a may be used.

Later in this chapter, the more accurate periods will be computed using a finite element analysis program. Before this can be done, however, the building mass must be determined.

6.2.2 Building Mass

Before the building mass can be determined, the approximate size of the different members of the seismic-force-resisting system must be established. For special moment frames, limitations on beam-column joint shear and reinforcement development length usually control. This is particularly true when lightweight (LW) concrete is used. An additional consideration is the amount of vertical reinforcement in the columns. ACI 318 Sec. 21.4.3.1 limits the vertical steel reinforcing ratio to 6 percent for special moment frame columns; however, 4 percent vertical steel is a more practical limit.

Based on a series of preliminary calculations (not shown here), it is assumed that all columns and structural wall boundary elements are 30 in. by 30 in., girders are 22.5 in. wide by 32 in. deep, and the panel of the structural wall is 16 in. thick. It has already been established that pan joists are spaced 36 in. o.c., have an average web thickness of 6 in., and, including a 4-in.-thick slab, are 20 in. deep. For the Berkeley building, these member sizes probably are close to the final sizes. For the Honolulu building (which has no structural wall and ultimately ends up with slightly smaller elements), the masses computed from the above member sizes are on the conservative (heavy) side.

In addition to the building structural weight, the following superimposed dead loads (DL) were assumed:

Partition DL (and roofing)	= 10 psf
Ceiling and mechanical DL	= 15 psf
Curtain wall cladding DL	= 10 psf

Based on the member sizes given above and on the other dead load, the individual story weights, masses, and mass moments of inertia are listed in Table 6-2. These masses were used for both the Berkeley and the Honolulu buildings.

As discussed below, the mass and mass moments of inertia are required for the determination of modal properties using the ETABS program. Note from Table 6-2 that the roof and lowest floor have masses slightly different from the typical floors. It is also interesting to note that the average density of this building is 11.2 pcf. A normal weight (NW) concrete building of the same configuration would have a density of approximately 14.0 pcf.

The use of LW instead of NW concrete reduces the total building mass by more than 20 percent and certainly satisfies the minimize mass rule of earthquake-resistant design. However, there are some disadvantages to the use of LW concrete. In general, LW aggregate reinforced concrete has a lower toughness or ductility than NW reinforced concrete and the higher the strength, the larger the reduction in available ductility. For this reason and also the absence of pertinent test results, ACI 318 Sec. 21.2.4.2 allows a maximum compressive strength of 4,000 psi for LW concrete in areas of high seismicity. [Note that in ACI 318-02 Sec. 21.2.4.2, the maximum compressive strength for LW concrete has been increased to 5,000 psi.] A further penalty placed on LW concrete is the reduction of shear strength. This primarily affects the sizing of beam-column joints (ACI 318 Sec. 21.5.3.2) but also has an effect on the amount of shear reinforcement required in the panels of structural walls.[1] For girders, the reduction in shear strength of LW aggregate concrete usually is of no concern because ACI 318 disallows the use of the concrete in determining the shear resistance of members with significant earthquake shear (ACI 318 Sec. 21.4.5.2). Finally, the required tension development lengths for bars embedded in LW concrete are significantly greater than those required for NW concrete.

Table 6-2 Story Weights, Masses, and Moments of Inertia

Story Level	Weight (kips)	Mass (kips-sec^2/in.)	Mass Moment of Inertia (in.-kip-sec^2/rad)

[1]ACI 318 Sec. 21.6.4 [21.7.4] gives equations for the shear strength of the panels of structural walls. In the equations, the term $\sqrt{f_c'}$ appears, but there is no explicit requirement to reduce the shear strength of the concrete when LW aggregate is used. However, ACI 318 Sec. 11.2 states that wherever the term $\sqrt{f_c'}$ appears in association with shear strength, it should be multiplied by 0.75 when all-LW concrete is used and by 0.85 when sand-LW concrete is used. In this example, which utilizes sand-LW concrete, the shear strength of the concrete will be multiplied by 0.85 as specified in ACI 318 Chapter 11.

Roof	2,783	7.202	4,675,000
12	3,051	7.896	5,126,000
11	3,051	7.896	5,126,000
10	3,051	7.896	5,126,000
9	3,051	7.896	5,126,000
8	3,051	7.876	5,126,000
7	3,051	7.896	5,126,000
6	3,051	7.896	5,126,000
5	3,051	7.896	5,126,000
4	3,051	7.896	5,126,000
3	3,051	7.896	5,126,000
2	3,169	8.201	5,324,000
Total	36,462		

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm.

6.2.3 Structural Analysis

Structural analysis is used primarily to determine the forces in the elements for design purposes, compute story drift, and assess the significance of P-delta effects. The structural analysis also provides other useful information (e.g., accurate periods of vibration and computational checks on plan and vertical irregularities). The computed periods of vibration are addressed in this section and the other results are presented and discussed later.

The ETABS program was used for the analysis of both the Berkeley and Honolulu buildings. Those aspects of the model that should be noted are:

1. The structure was modeled with 12 levels above grade and one level below grade. The perimeter basement walls were modeled as shear panels as were the main structural walls. It was assumed that the walls were "fixed" at their base.

2. As automatically provided by the ETABS program, all floor diaphragms were assumed to be infinitely rigid in plane and infinitely flexible out-of-plane.

3. Beams, columns, and structural wall boundary members were represented by two-dimensional frame elements. Each member was assumed to be uncracked, and properties were based on gross area for the columns and boundary elements and on effective T-beam shapes for the girders. (The effect of cracking is considered in a simplified manner.) The width of the flanges for the T-beams is based on the definition of T-beams in ACI 318 Sec. 8.10. Except for the slab portion of the joists which contributed to T-beam stiffness of the girders, the flexural stiffness of the joists was ignored. For the haunched girders, an equivalent depth of stem was used. The equivalent depth was computed to provide a prismatic member with a stiffness under equal end rotation identical to that of the nonprismatic haunched member. Axial, flexural, and shear deformations were included for all members.

4. The structural walls of the Berkeley building are modeled as a combination of boundary elements and shear panels.

5. Beam-column joints are modeled as 50 percent rigid. This provides effective stiffness for beam-column joints halfway between a model with fully rigid joints (clear span analysis) and fully flexible joints (centerline analysis).

6. P-delta effects are ignored. An evaluation of the accuracy of this assumption is provided later in this example.

6.2.4 Accurate Periods from Finite Element Analysis

The computed periods of vibration and a description of the associated modes of vibration are given for the first 11 modes of the Berkeley building in Table 6-3. With 11 modes, the accumulated modal mass in each direction is more than 90 percent of the total mass. *Provisions* Sec. 5.5.2 [5.3.2] requires that a dynamic analysis must include at least 90 percent of the actual mass in each of the two orthogonal directions. Table 6-4 provides the computed modal properties for the Honolulu building. In this case, 90 percent of the total mass was developed in just eight modes.

For the Berkeley building, the computed N-S period of vibration is 1.77 sec. This is between the approximate period, T_a = 1.5 sec, and C_uT_a = 2.1 sec. In the E-W direction, the computed period is 1.40 sec, which is greater than both T_a = 0.88 sec and C_uT_a = 1.23 sec.

If cracked section properties were used, the computed period values for the Berkeley building would be somewhat greater. For preliminary design, it is reasonable to assume that each member has a cracked moment of inertia equal to one-half of the gross uncracked moment of inertia. Based on this assumption, and the assumption that flexural behavior dominates, the cracked periods would be approximately 1.414 (the square root of 2.0) times the uncracked periods. Hence, for Berkeley, the cracked N-S and E-W periods are 1.414(1.77) = 2.50 sec, and 1.414(1.4) = 1.98 sec, respectively. Both of these cracked periods are greater than C_uT_a, so C_uT_a can be used in the ELF analysis.

For the Honolulu building, the uncracked periods in the N-S and E-W directions are 1.78 and 1.87 sec, respectively. The N-S period is virtually the same as for the Berkeley building because there are no walls in the N-S direction of either building. In the E-W direction, the increase in period from 1.4 sec to 1.87 sec indicates a significant reduction in stiffness due to the loss of the walls in the Honolulu building. For both the E-W and the N-S directions, the approximate period (T_a) for the Honolulu building is 1.5 sec, and C_uT_a is 2.28 sec. Both of the computed periods fall within these bounds. However, if cracked section properties were used, the computed periods would be 2.52 sec in the N-S direction and 2.64 sec in the E-W direction. For the purpose of computing ELF forces, therefore, a period of 2.28 sec can be used for both the N-S and E-W directions in Honolulu.

A summary of the approximate and computed periods is given in Table 6-5.

Table 6-3 Periods and Modal Response Characteristics for the Berkeley Building

Mode	Period* (sec)	% of Effective Mass Represented by Mode**		Description
		N-S	E-W	
1	1.77	80.23 (80.2)	00.00 (0.00)	First Mode N-S
2	1.40	0.0 (80.2)	71.48 (71.5)	First Mode E-W
3	1.27	0.0 (80.2)	0.00 (71.5)	First Mode Torsion
4	0.581	8.04 (88.3)	0.00 (71.5)	Second Mode N-S
5	0.394	0.00 (88.3)	0.00 (71.5)	Second Mode Torsion
6	0.365	0.00 (88.3)	14.17 (85.6)	Second Mode E-W
7	0.336	2.24 (90.5)	0.00 (85.6)	Third Mode N-S
8	0.230	0.88 (91.4)	0.00 (85.6)	Fourth Mode N-S
9	0.210	0.00 (91.4)	0.00 (85.6)	Third Mode Torsion
10	0.171	0.40 (91.8)	0.00 (85.6)	Fifth Mode N-S
11	0.135	0.00 (91.8)	4.95 (90.6)	Third Mode E-W

* Based on gross section properties.
** Accumulated mass in parentheses.

Table 6-4 Periods and Modal Response Characteristics for the Honolulu Building

Mode	Period* (sec)	% of Effective Mass Represented by Mode**		Description
		N-S	E-W	
1	1.87	79.7 (79.7)	0.00 (0.00)	First Mode E-W
2	1.78	0.00 (79.7)	80.25 (80.2)	First Mode N-S
3	1.38	0.00 (79.7)	0.00 (80.2)	First Mode Torsion
4	0.610	8.79 (88.5)	0.00 (80.2)	Second Mode E-W
5	0.584	0.00 (88.5)	8.04 (88.3)	Second Mode N-S
6	0.452	0.00 (88.5)	0.00 (88.3)	Second Mode Torsion
7	0.345	2.27 (90.7)	0.00 (88.3)	Third Mode E-W
8	0.337	0.00 (90.7)	2.23 (90.5)	Third Mode N-S
9	0.260	0.00 (90.7)	0.00 (90.5)	Third Mode Torsion
10	0.235	0.89 (91.6)	0.00 (90.5)	Fourth Mode E-W
11	0.231	0.00 (91.6)	0.87 (91.4)	Fourth Mode N-S

* Based on gross section properties.
** Accumulated mass in parentheses.

Table 6-5 Comparison of Approximate and "Exact" Periods (in seconds)

Method of Period Computation	Berkeley		Honolulu	
	N-S	E-W	N-S	E-W
Approximate T_a	1.50	0.88	1.50	1.50
Approximate × C_u	*2.10**	*1.23*	*2.28*	*2.28*
ETABS (gross)	1.77	1.40	1.78	1.87
ETABS (cracked)	2.50	1.98	2.52	2.64

* Values in italics should be used in the ELF analysis.

6.2.5 Seismic Design Base Shear

The seismic design base shear for the Berkeley is computed below.

In the N-S direction with W = 36,462 kips (see Table 6-2), S_{DS} = 1.10, S_{D1} = 0.589, R = 8, I = 1, and T = 2.10 sec:

$$C_{S,max} = \frac{S_{DS}}{R/I} = \frac{1.10}{8/1} = 0.1375$$

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.589}{2.10(8/1)} = 0.0351$$

$$C_{S,min} = 0.044 S_{DS} I = 0.044(1.1)(1) = 0.0484$$

[As noted previously in Sec. 6.2, the minimum C_s value is 0.01 in the 2003 *Provisions*.]

$C_{S,min}$ = 0.0484 controls, and the design base shear in the N-S direction is V = 0.0484 (36,462) = 1,765 kips.

In the stiffer E-W direction, $C_{S,max}$ and $C_{S,min}$ are as before, T = 1.23 sec, and

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.589}{1.23(8/1)} = 0.0598$$

In this case, C_S = 0.0598 controls and V = 0.0598 (36,462) = 2,180 kips

For the Honolulu building, base shears are computed in a similar manner and are the same for the N-S and the E-W directions. With W = 36,462 kips, S_{DS} = 0.474, S_{D1} = 0.192, R = 5, I = 1, and T = 2.28 sec:

$$C_{S,max} = \frac{S_{DS}}{R/I} = \frac{0.472}{5/1} = 0.0944$$

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.192}{2.28(5/1)} = 0.0168$$

$$C_{S,min} = 0.044 S_{DS} I = 0.044(0.472)(1.0) = 0.0207$$

C_S = 0.0207 controls and V = 0.0207 (36,462) = 755 kips

A summary of the Berkeley and Honolulu seismic design parameters are provided in Table 6-6.

Note that *Provisions* Sec. 5.4.6 [5.2.6.1] states that for the purpose of computing drift, a base shear computed according to *Provisions* Eq. 5.4.1.1-2 [5.2-3] (used to compute C_S above) may be used in lieu of the shear computed using *Provisions* Eq. 5.4.1.1-3 [5.2-4] (used to compute $C_{S,min}$ above).

Table 6-6 Comparison of Periods, Seismic Shears Coefficients, and Base Shears for the Berkeley and Honolulu Buildings

Location	Response Direction	Building Frame Type	T (sec)	C_s	V (kips)

Berkeley	N-S	Special moment frame	2.10	0.0485	1,765
	E-W	Dual system incorporating special moment frame and structural wall	1.23	0.0598	2,180
Honolulu	N-S	Intermediate moment frame	2.28	0.0207	755
	E-W	Intermediate moment frame	2.28	0.0207	755

1.0 kip = 4.45 kN.

6.2.6 Development of Equivalent Lateral Forces

The vertical distribution of lateral forces is computed from *Provisions* Eq. 5.4.3-1 and 5.4.3-2 [5.2-10 and 5.2-11]:

$$F_x = C_{vx}V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where

$k = 1.0$ for $T < 0.5$ sec
$k = 2.0$ for $T > 2.5$ sec
$k = 0.75 + 0.5T$ for $1.0 < T < 2.5$ sec

Based on the equations above, the seismic story forces, shears, and overturning moments are easily computed using an Excel spreadsheet. The results of these computations are shown in Tables 6-7a and 6-7b for the Berkeley buildings and in Table 6-8 for the Honolulu building. A note at the bottom of each table gives the calculated vertical force distribution factor (k). The tables are presented with as many significant digits to the left of the decimal as the spreadsheet generates but that should not be interpreted as real accuracy; it is just the simplest approach. Also, some of the sums are not exact due to truncation error.

Table 6-7a Vertical Distribution of N-S Seismic Forces for the Berkeley Building*

Level	Height h (ft)	Weight W (kips)	Wh^k	Wh^k/Σ	Force F_x (kips)	Story Shear V_x (kips)	Overturning Moment M_x (ft-k)
R	155.5	2,783	24,526,067	0.187	330.9	330.9	4,136
12	143.0	3,051	23,123,154	0.177	311.9	642.8	12,170
11	130.5	3,051	19,612,869	0.150	264.6	907.4	23,512
10	118.0	3,051	16,361,753	0.125	220.7	1,128.1	37,613
9	105.5	3,051	13,375,088	0.102	180.4	1,308.5	53,970
8	93.0	3,051	10,658,879	0.081	143.8	1,452.3	72,123
7	80.5	3,051	8,220,056	0.063	110.9	1,563.2	91,663
6	68.0	3,051	6,066,780	0.046	81.8	1,645.0	112,226
5	55.5	3,051	4,208,909	0.032	56.8	1,701.8	133,498
4	43.0	3,051	2,658,799	0.020	35.9	1,737.7	155,219
3	30.5	3,051	1,432,788	0.011	19.3	1,757.0	177,181
2	18.0	3,169	575,987	0.004	7.8	1,764.8	208,947
Total		36,462	130,821,129	0.998	1764.8		

* Table based on T = 2.1 sec and k = 1.8.
1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

Table 6-7b Vertical Distribution of E-W Seismic Forces for the Berkeley Building*

Level	Height h (ft)	Weight W (kips)	Wh^k	Wh^k/Σ	Force F_x (kips)	Story Shear V_x (kips)	Overturning Moment M_x (ft-k)
R	155.5	2,783	2,730,393	0.161	350.6	351	4,382
12	143.0	3,051	2,669,783	0.157	342.8	693	13,049
11	130.5	3,051	2,356,408	0.139	302.5	996	25,497
10	118.0	3,051	2,053,814	0.121	263.7	1,260	41,242
9	105.5	3,051	1,762,714	0.104	226.3	1,486	59,816
8	93.0	3,051	1,483,957	0.087	190.5	1,676	80,771
7	80.5	3,051	1,218,579	0.072	156.5	1,833	103,682
6	68.0	3,051	967,870	0.057	124.3	1,957	128,146
5	55.5	3,051	733,503	0.043	94.2	2,051	153,788
4	43.0	3,051	517,758	0.030	66.5	2,118	180,260
3	30.5	3,051	323,975	0.019	41.6	2,159	207,253
2	18.0	3,169	163,821	0.010	21.0	2,180	246,500
Total		36,462	16,982,575	1.000	2180.5		

* Table based on T = 1.23 sec and k = 1.365.
1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

Table 6-8 Vertical Distribution of N-S and E-W Seismic Forces for the Honolulu Building*

Level	Height h (ft)	Weight W (kips)	Wh^k	Wh^k/Σ	Force F_x (kips)	Story Shear V_x (kips)	Overturning Moment M_x (ft-k)
R	155.5	2,783	38,626,348	0.193	145.6	145.6	1,820
12	143.0	3,051	36,143,260	0.181	136.2	281.9	5,343
11	130.5	3,051	30,405,075	0.152	114.6	396.5	10,299
10	118.0	3,051	25,136,176	0.126	94.8	491.2	16,440
9	105.5	3,051	20,341,799	0.102	76.7	567.9	23,539
8	93.0	3,051	16,027,839	0.080	60.4	628.3	31,393
7	80.5	3,051	12,210,028	0.061	46.0	674.3	39,822
6	68.0	3,051	8,869,192	0.044	33.4	707.8	48,669
5	55.5	3,051	6,041,655	0.030	22.8	730.5	57,801
4	43.0	3,051	3,729,903	0.019	14.1	744.6	67,108
3	30.5	3,051	1,948,807	0.010	7.3	751.9	76,508
2	18.0	3,169	747,115	0.004	2.8	754.8	90,093
Total		36,462	200,218,197	1.002	754.7		

* Table based on T = 2.28 sec and k = 1.89.
1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m

The computed seismic story shears for the Berkeley and Honolulu buildings are shown graphically in Figures 6-3 and 6-4, respectively. Also shown in the figures are the story shears produced by ASCE 7 wind loads. For Berkeley, a 3-sec gust of 85 mph was used and, for Honolulu, a 3-sec gust of 105 mph. In each case, an Exposure B classification was assumed. The wind shears have been factored by a value of 1.36 (load factor of 1.6 times directionality factor 0.85) to bring them up to the ultimate seismic loading limit state represented by the *Provisions*.

As can be seen from the figures, the seismic shears for the Berkeley building are well in excess of the wind shears and will easily control the design of the members of the frames and walls. For the Honolulu building, the N-S seismic shears are significantly greater than the corresponding wind shears, but the E-W seismic and wind shears are closer. In the lower stories of the building, wind controls the strength demands and, in the upper levels, seismic forces control the strength demands. (A somewhat more detailed comparison is given later when the Honolulu building is designed.) With regards to detailing the Honolulu building, *all* of the elements must be detailed for inelastic deformation capacity as required by ACI 318 rules for intermediate moment frames.

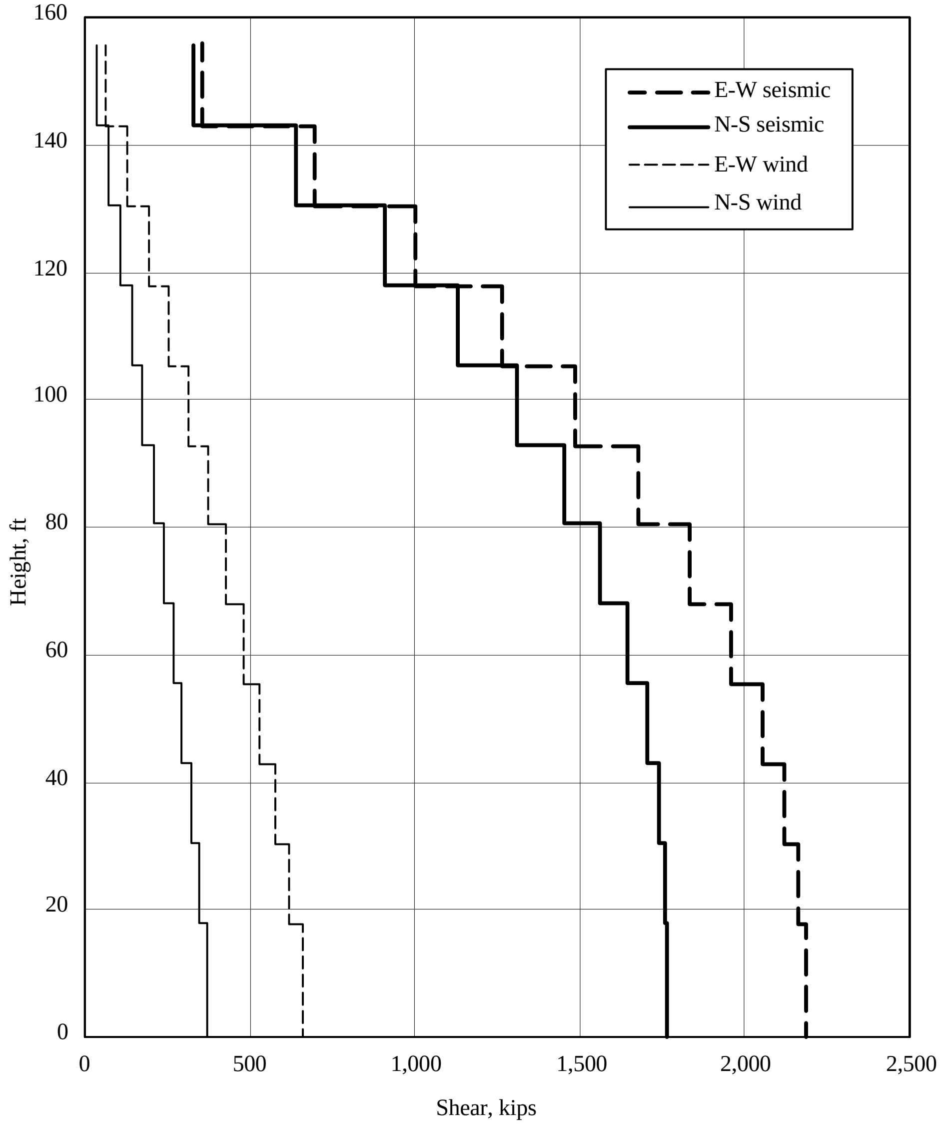


Figure 6-3 Comparison of wind and seismic story shears for the Berkeley building (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

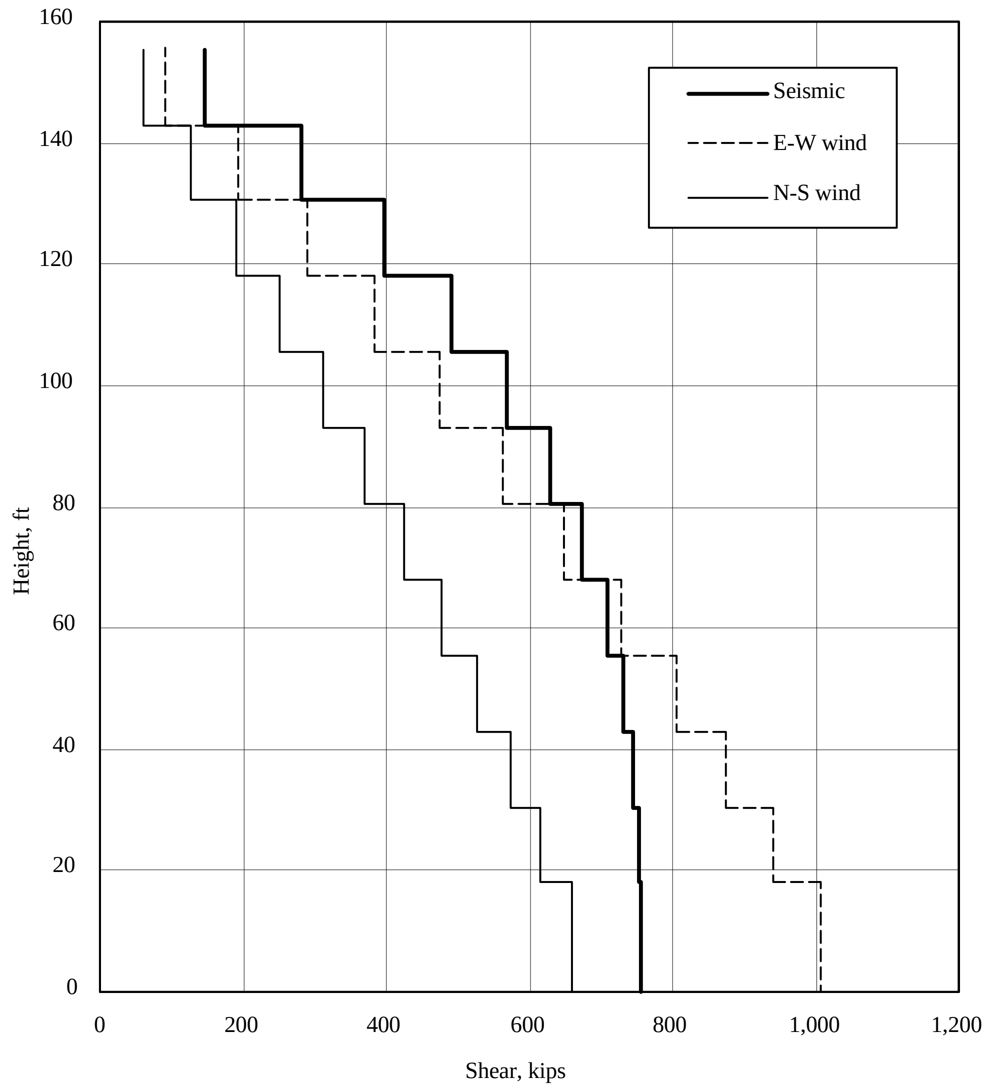


Figure 6-4 Comparison of wind and seismic story shears for the Honolulu building (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

6.3 DRIFT AND P-DELTA EFFECTS

6.3.1 Direct Drift and P-Delta Check for the Berkeley Building

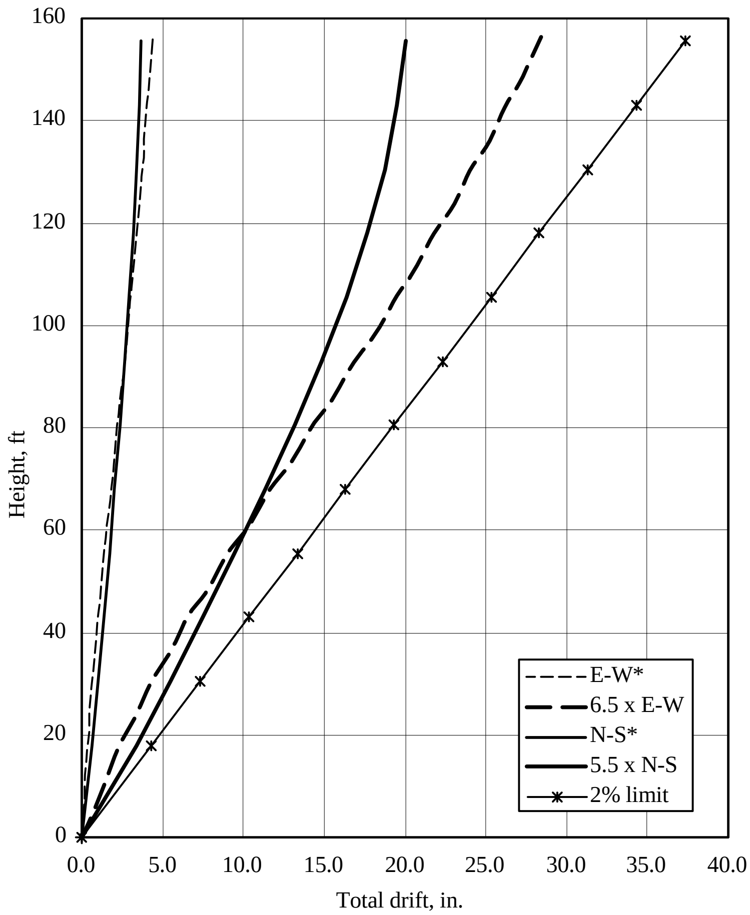
Drift and P-delta effects are checked according to *Provisions* Sec. 5.2.8 [5.2.6.1] and 5.4.6 [5.2.6.2], respectively. According to *Provisions* Table 5.2.8 [4.5-1], the story drift limit for this Seismic Use Group I building is $0.020h_{sx}$ where h_{sx} is the height of story x. This limit may be thought of as 2 percent of the story height. Quantitative results of the drift analysis for the N-S and E-W directions are shown in Tables 6-9a and 6-9b, respectively.

With regards to the values shown in Table 6-9a , it must be noted that cracked section properties were used in the structural analysis and that 0.0351/0.0484=0.725 times the story forces shown in Table 6-7a were applied. This adjusts for the use of *Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*], which governed for base shear, was not used in computing drift. In Table 6-9b, cracked section properties were also used, but the modifying factor does not apply because *Provisions* Eq. 5.4.1.1-2 [5.2-3] controlled in this direction.

In neither case does the computed drift ratio (magnified story drift/h_{sx}) exceed 2 percent of the story height. Therefore, the story drift requirement is satisfied. A plot of the total drift resulting from both the N-S and E-W equivalent lateral seismic forces is shown in Figure 6-5.

An example calculation for drift in Story 5 loaded in the E-W direction is given below. Note that the relevant row is highlighted in Table 6-9b.

Deflection at top of story = δ_{5e} = 1.812 in.
Deflection at bottom of story = δ_{4e} = 1.410 in.
Story drift = $\Delta_{5e} = \delta_{5e} - \delta_{4e}$ = 1.812 - 1.410 = 0.402 in.
Deflection amplification factor, C_d = 6.5
Importance factor, I = 1.0
Magnified story drift = $\Delta_5 = C_d\, \Delta_{5e}/I$ = 6.5(0.402)/1.0 = 2.613 in.
Magnified drift ratio = Δ_5/h_5 = (2.613/150) = 0.01742 = 1.742% < 2.0% OK



* Elasticlly computed under code-prescribed seismic forces

Figure 6-5 Drift profile for Berkeley building (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

Table 6-9a Drift Computations for the Berkeley Building Loaded in the N-S Direction

Story	Total Deflection (in.)	Story Drift (in.)	Story Drift × C_d * (in.)	Drift Ratio (%)
12	3.640	0.087	0.478	0.319
11	3.533	0.145	0.798	0.532
10	3.408	0.203	1.117	0.744
9	3.205	0.232	1.276	0.851
8	2.973	0.276	1.515	1.010
7	2.697	0.305	1.675	1.117
6	2.393	0.334	1.834	1.223
5	2.059	0.348	1.914	1.276
4	1.711	0.348	1.914	1.276
3	1.363	0.364	2.002	1.334
2	0.999	0.381	2.097	1.398
1	0.618	0.618	3.397	1.573

* C_d = 5.5 for loading in this direction; total drift is at top of story, story height = 150 in. for Levels 3 through roof and 216 in. for Level 2.
1.0 in. = 25.4 mm.

Table 6-9b Drift Computations for the Berkeley Building Loaded in the E-W Direction

Story	Total Drift (in.)	Story Drift (in.)	Story Drift × C_d * (in.)	Drift Ratio (%)
12	4.360	0.300	1.950	1.300
11	4.060	0.340	2.210	1.473
10	3.720	0.340	2.210	1.473
9	3.380	0.360	2.340	1.560
8	3.020	0.400	2.600	1.733
7	2.620	0.400	2.600	1.733
6	2.220	0.408	2.652	1.768
5	**1.812**	**0.402**	**2.613**	**1.742**
4	1.410	0.386	2.509	1.673
3	1.024	0.354	2.301	1.534
2	0.670	0.308	2.002	1.335
1	0.362	0.362	2.353	1.089

* C_d = 6.5 for loading in this direction; total drift is at top of story, story height = 150 in. for Levels 3 through roof and 216 in. for Level 2.
1.0 in. = 25.4 mm.

When a soft story exists in a Seismic Design Category D building, *Provision*s Table 5.2.5.1 [4.4-1] requires that a modal analysis be used. However, *Provisions* Sec. 5.2.3.3 [4.3.2.3] lists an exception:

> Structural irregularities of Types 1a, 1b, or 2 in Table 5.2.3.3 [4.3-2] do not apply where no story drift ratio under design lateral load is less than or equal to 130 percent of the story drift ratio of the next story above. . . . The story drift ratios of the top two stories of the structure are not required to be evaluated.

For the building responding in the N-S direction, the ratio of first story to second story drift ratios is 1.573/1.398 = 1.13, which is less than 1.3. For E-W response, the ratio is 1.089/1.335 = 0.82, which also is less than 1.3. Therefore, a modal analysis is not required and the equivalent static forces from Tables 6-7a and 6-7b may be used for design.

The P-delta analysis for each direction of loading is shown in Tables 6-10a and 6-10b. The upper limit on the allowable story stability ratio is given by *Provisions* Eq. 5.4.6.2-2 [changed in the 2003 *Provisions*] as:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.50$$

Taking β as 1.0 (see *Provisions* Sec. 5.4.6.2 [not applicable in the 2003 *Provisions*]), the stability ratio limit for the N-S direction is 0.5/(1.0)5.5 = 0.091, and for the E-W direction the limit is 0.5/(1.0)6.5 = 0.077.

[In the 2003 *Provisions*, the maximum limit on the stability coefficient has been replaced by a requirement that the stability coefficient is permitted to exceed 0.10 if and only "if the resistance to lateral forces is determined to increase in a monotonic nonlinear static (pushover) analysis to the target displacement as determined in Sec. A5.2.3. P-delta effects shall be included in the analysis." Therefore, in this example, the stability coefficient should be evaluated directly using 2003 *Provisions* Eq. 5.2.-16.]

For this P-delta analysis a (reduced) story live load of 20 psf was included in the total story weight calculations. Deflections are based on cracked sections, and story shears are adjusted as necessary for use of *Provisions* Eq. 5.4.1.1-3 [5.2-3]. As can be seen in the last column of each table, the stability ratio (θ) does not exceed the maximum allowable value computed above. Moreover, since the values are less than 0.10 at all levels, P-delta effects can be neglected for both drift and strength computed limits according to *Provisions* Sec. 5.4.6.2 [5.2.6.2].

An example P-delta calculation for the Level 5 under E-W loading is shown below. Note that the relevant row is highlighted in Table 6-10b.

Magnified story drift = Δ_5 = 2.613 in.
Story shear = V_5 = 1,957 kips
Accumulated story weight P_5 = 27,500 kips
Story height = h_{s5}= 150 in.
C_d = 6.5
$\theta = (P_5\ (\Delta_5/C_d))\ /(V_5 h_{s5})$ = 27,500(2.613/6.5)/(1957.1)(150) = 0.0377 < 0.077 OK

[Note that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, I, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because I = 1.0.]

Table 6-10a P-Delta Computations for the Berkeley Building Loaded in the N-S Direction

Level	Story Drift (in.)	Story Shear* (kips)	Story Dead Load (kips)	Story Live Load (kips)	Total Story Load (kips)	Accum. Story Load (kips)	Stability Ratio θ
12	0.478	239.9	2783	420	3203	3203	0.0077
11	0.798	466.0	3051	420	3471	6674	0.0138
10	1.117	657.8	3051	420	3471	10145	0.0209
9	1.276	817.9	3051	420	3471	13616	0.0257
8	1.515	948.7	3051	420	3471	17087	0.0331
7	1.675	1052.9	3051	420	3471	20558	0.0396
6	1.834	1133.3	3051	420	3471	24029	0.0471
5	1.914	1192.6	3051	420	3471	27500	0.0535
4	1.914	1233.8	3051	420	3471	30971	0.0582
3	2.002	1259.8	3051	420	3471	34442	0.0663
2	2.097	1273.8	3051	420	3471	37913	0.0757
1	3.397	1279.5	3169	420	3589	41502	0.0928

* Story shears in Table 6-7a factored by 0.725. See Sec. 6.3.1.
1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

Table 6-10b P-Delta Computations for the Berkeley Building Loaded in the E-W Direction

Level	Story Drift (in.)	Story Shear (kips)	Story Dead Load (kips)	Story Live Load (kips)	Total Story Load (kips)	Accum. Story Load (kips)	Stability Ratio θ
12	1.950	350.6	2783	420	3203	3203	0.0183
11	2.210	693.3	3051	420	3471	6674	0.0218
10	2.210	995.9	3051	420	3471	10145	0.0231
9	2.340	1259.6	3051	420	3471	13616	0.0259
8	2.600	1485.9	3051	420	3471	17087	0.0307
7	2.600	1676.4	3051	420	3471	20558	0.0327
6	2.652	1832.9	3051	420	3471	24029	0.0357
5	**2.613**	**1957.1**	**3051**	**420**	**3471**	**27500**	**0.0377**
4	2.509	2051.3	3051	420	3471	30971	0.0389
3	2.301	2117.8	3051	420	3471	34442	0.0384
2	2.002	2159.4	3051	420	3471	37913	0.0361
1	2.353	2180.4	3169	420	3589	41502	0.0319

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

6.3.2 Test for Torsional Irregularity for Berkeley Building

In Sec. 6.1.3 it was mentioned that torsional irregularities are unlikely for the Berkeley building because the elements of the seismic-force-resisting system were well distributed over the floor area. This will now be verified by applying the story forces of Table 6-3a at an eccentricity equal to 5 percent of the building dimension perpendicular to the direction of force (accidental torsion requirement of *Provisions* Sec. 5.4.4.2 [5.2.4.2]). This test is required per *Provisions* Sec. 5.2.3.2 [4.3.2.2]. Analysis was performed using the ETABS program.

The eccentricity is 0.05(102.5) = 5.125 ft for forces in the N-S direction and 0.05(216) = 10.8 ft in the E-W direction.

For forces acting in the N-S direction:

Total displacement at center of mass = δ_{avg} = 3.640 in. (see Table 6-9a)
Rotation at center of mass = 0.000189 radians
Maximum displacement at corner of floor plate = d_{max} = 3.640 + 0.000189(102.5)(12)/2 = 3.756 in.
Ratio $\delta_{max}/\delta_{avg}$ = 3.756/3.640 = 1.03 < 1.20, so no torsional irregularity exists.

For forces acting in the E-W direction:

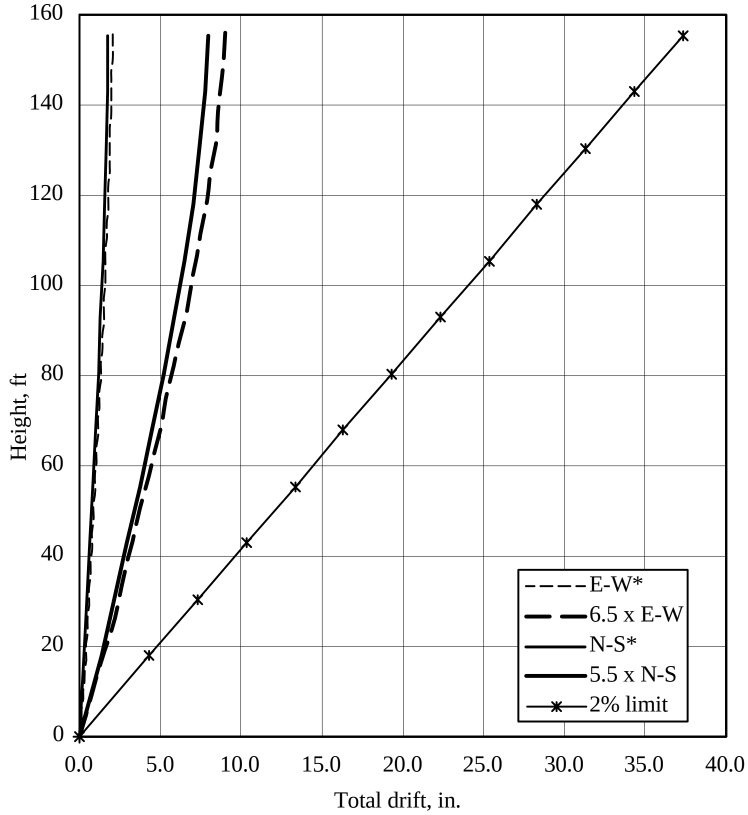
Total displacement at center of mass = δ_{avg} = 4.360 in. (see Table 6-9b)
Rotation at center of mass = 0.000648 radians
Maximum displacement at corner of floor plate = d_{max} = 4.360 + 0.000648(216)(12)/2 = 5.200 in.
Ratio d_{max}/d_{avg} = 5.200/4.360 = 1.19 < 1.20, so no torsional irregularity exists.

It is interesting that this building, when loaded in the E-W direction, is very close to being torsionally irregular (irregularity Type 1a of *Provisions* Table 5.2.3.2 [4.3-2]), even though the building is extremely regular in plan. The torsional flexibility of the building arises from the fact that the walls exist only on interior Gridlines 3, 4, 5, and 6.

6.3.3 Direct Drift and P-Delta Check for the Honolulu Building

The interstory drift computations for the Honolulu building deforming under the N-S and E-W equivalent static forces are shown in Tables 6-11a and 6-11b. As with the Berkeley building, the analysis used cracked section properties. The applied seismic forces, shown previously in Table 6-3b were multiplied by the ratio 0.0168/0.0207 = 0.808 to adjust for the use of *Provisions* Eq. 5.4.1.1-3. [As noted previously in Sec. 6.2, the minimum Cs value has been removed in the 2003 *Provisions*.]

These tables, as well as Figure 6-6, show that the story drift at each level is less than the allowable interstory drift of $0.020h_{sx}$ (*Provisions* Table 5.2.8 [4.5-1]). Even though it is not pertinent for Seismic Design Category C buildings, a soft first story does not exist for the Honolulu building because the ratio of first story to second story drift does not exceed 1.3.



* Elasticlly computed under code-prescribed seismic forces

Figure 6-6 Drift profile for the Honolulu building (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

Table 6-11a Drift Computations for the Honolulu Building Loaded in the N-S Direction

Story	Total Drift (in.)	Story Drift (in.)	Story Drift × C_d * (in.)	Drift Ratio (%)
12	1.766	0.040	0.182	0.121
11	1.726	0.069	0.313	0.208
10	1.656	0.097	0.436	0.291
9	1.559	0.118	0.531	0.354
8	1.441	0.136	0.611	0.407
7	1.306	0.149	0.669	0.446
6	1.157	0.160	0.720	0.480
5	0.997	0.168	0.756	0.504
4	0.829	0.171	0.771	0.514
3	0.658	0.176	0.793	0.528
2	0.482	0.184	0.829	0.553
1	0.297	0.297	1.338	0.619

* C_d = 4.5 for loading in this direction; total drift is at top of story, story height = 150 in. for Levels 3 through roof and 216 in. for Level 2.
1.0 in. = 25.4 mm.

Table 6-11b Drift Computations for the Honolulu Building Loaded in the E-W Direction

Story	Total Drift (in.)	Story Drift (in.)	Story Drift × C_d * (in.)	Drift Ratio (%)
12	2.002	0.061	0.276	0.184
11	1.941	0.090	0.407	0.271
10	1.850	0.116	0.524	0.349
9	1.734	0.137	0.618	0.412
8	1.597	0.157	0.705	0.470
7	1.440	0.171	0.772	0.514
6	1.269	0.179	0.807	0.538
5	**1.089**	**0.186**	**0.836**	**0.558**
4	0.903	0.191	0.858	0.572
3	0.713	0.191	0.858	0.572
2	0.522	0.197	0.887	0.591
1	0.325	0.325	1.462	0.677

* C_d = 4.5 for loading in this direction; total drift is at top of story, story height = 150 in. for Levels 3 through roof and 216 in. for Level 2.
1.0 in. = 25.4 mm.

A sample calculation for Level 5 of Table 6-11b (highlighted in the table) is as follows:

Deflection at top of story = δ_{5e} =1.089 in.
Deflection at bottom of story = δ_{4e} = 0.903 in.
Story drift = $\Delta_{5e} = \delta_{5e} - \delta_{4e}$ = 1.089-0.0903 = 0.186 in.
Deflectiom amplification factor, C_d = 4.5
Importance factor, I = 1.0
Magnified story drift = $\Delta_5 = C_d \Delta_{5e}/I$ = 4.5(0.186)/1.0 = 0.836 in.
Magnified drift ratio = Δ_5 / h_5 = (0.836/150) = 0.00558 = 0.558% < 2.0% OK

Therefore, story drift satisfies the drift requirements.

Calculations for P-delta effects are shown in Tables 6-12a and 6-12b for N-S and E-W loading, respectively. The stability ratio at the 5th story from Table 6-12b is computed:

Magnified story drift = Δ_5 = 0.836 in.
Story shear = V_5 = 571.9 = kips
Accumulated story weight P_5 = 27500 kips
Story height = h_{s5} = 150 in.
C_d = 4.5
$\theta = [P_5\ (\Delta_5/C_d)]/(V_5 h_{s5})$ = 27500(0.836/4.5)/(571.9)(150) = 0.0596

[Note that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, *I*, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because I = 1.0.]

The requirements for maximum stability ratio (0.5/C_d = 0.5/4.5 = 0.111) are satisfied. Because the stability ratio is less than 0.10 at all floors, P-delta effects need not be considered (*Provisions* Sec. 5.4.6.2 [5.2.6.2]). (The value of 0.1023 in the first story for the E-W direction is considered by the author to be close enough to the criterion.)

Table 6-12a P-Delta Computations for the Honolulu Building Loaded in the N-S Direction

Level	Story Drift (in.)	Story Shear * (kips)	Story Dead Load (kips)	Story Live Load (kips)	Total Story Load (kips)	Accum. Story Load (kips)	Stability Ratio θ
12	0.182	117.7	2783	420	3203	3203	0.0073
11	0.313	227.7	3051	420	3471	6674	0.0136
10	0.436	320.4	3051	420	3471	10145	0.0205
9	0.531	396.9	3051	420	3471	13616	0.0270
8	0.611	458.9	3051	420	3471	17087	0.0337
7	0.669	507.7	3051	420	3471	20558	0.0401
6	0.720	544.9	3051	420	3471	24029	0.0470
5	0.756	571.9	3051	420	3471	27500	0.0539
4	0.771	590.3	3051	420	3471	30971	0.0599
3	0.793	601.6	3051	420	3471	34442	0.0672
2	0.829	607.6	3051	420	3471	37913	0.0766
1	1.338	609.8	3169	420	3589	41502	0.0937

* Story shears in Table 6-8 factored by 0.808. See Sec. 6.3.3.
1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

Table 6-12b P-Delta Computations for the Honolulu Building Loaded in the E-W Direction

Level	Story Drift (in.)	Story Shear * (kips)	Story Dead Load (kips)	Story Live Load (kips)	Total Story Load (kips)	Accum. Story Load (kips)	Stability Ratio θ
12	0.276	117.7	2783	420	3203	3203	0.0111
11	0.407	227.7	3051	420	3471	6674	0.0177
10	0.524	320.4	3051	420	3471	10145	0.0246
9	0.618	396.9	3051	420	3471	13616	0.0314
8	0.705	458.9	3051	420	3471	17087	0.0389
7	0.772	507.7	3051	420	3471	20558	0.0463
6	0.807	544.9	3051	420	3471	24029	0.0527
5	**0.836**	**571.9**	**3051**	**420**	**3471**	**27500**	**0.0596**
4	0.858	590.3	3051	420	3471	30971	0.0667
3	0.858	601.6	3051	420	3471	34442	0.0728
2	0.887	607.6	3051	420	3471	37913	0.0820
1	1.462	609.8	3169	420	3589	41502	0.1023

* Story shears in Table 6-8 factored by 0.808. See Sec. 6.3.3.
1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

6.3.4 Test for Torsional Irregularity for the Honolulu Building

A test for torsional irregularity for the Honolulu building can be performed in a manner similar to that for the Berkeley building. However, it is clear that a torsional irregularity will not occur for the Honolulu building if the Berkeley building is not irregular. This will be the case because the walls, which draw the torsional resistance towards the center of the Berkeley building, do not exist in the Honolulu building.

6.4 STRUCTURAL DESIGN OF THE BERKELEY BUILDING

6.4.1 Material Properties

For the Berkeley building, sand-LW aggregate concrete of 4,000 psi strength is used everywhere except for the lower two stories of the structural walls where 6,000 psi NW concrete is used. All reinforcement has a specified yield strength of 60 ksi, except for the panel of the structural walls which contains 40 ksi reinforcement. This reinforcement must conform to ASTM A706. According to ACI 318 Sec. 21.2.5, however, ASTM A615 reinforcement may be used if the actual yield strength of the steel does not exceed the specified strength by more than 18 ksi and the ratio of actual ultimate tensile stress to actual tensile yield stress is greater than 1.25.

6.4.2 Combination of Load Effects

Using the ETABS program, the structure was analyzed for the equivalent lateral loads shown in Tables 6-7a and 6-7b. For strength analysis, the loads were applied at a 5 percent eccentricity as required for accidental torsion by *Provisions* Sec. 5.4.4.2 [5.2.4.2]. Where applicable, orthogonal loading effects were included per *Provisions* Sec. 5.2.5.2.3 [4.4.2.3]. The torsional magnification factor (A_x) given by *Provisions* Eq. 5.4.4.3-1 [5.2-13] was not used because the building has no significant plan irregularities.

Provisions Sec. 5.2.7 [4.2.2.1] and Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2] require combination of load effects be developed on the basis of ASCE 7, except that the earthquake load effect, *E*, be defined as:

$$E = \rho Q_E + 0.2 S_{DS} D$$

when gravity and seismic load effects are additive and

$$E = \rho Q_E - 0.2 S_{DS} D$$

when the effects of seismic load counteract gravity.

The special load combinations given by *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-3 and 4.2-4] do not apply to the Berkeley building because there are no discontinuous elements supporting stiffer elements above them. (See *Provisions* Sec. 9.6.2 [9.4.1].)

The reliability factor (ρ) in Eq. 5.2.7-1 and 5.2.7-2 [not applicable in the 2003 *Provisions*] should be taken as the maximum value of ρ_x defined by *Provisions* Eq. 5.2.4.2:

$$\rho_x = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

where A_x is the area of the floor or roof diaphragm above the story under consideration and r_{max_x} is the largest ratio of the design story shear resisted by a single element divided by the total story shear for a given loading. The computed value for ρ must be greater than or equal to 1.0, but need not exceed 1.5. Special moment frames in Seismic Design Category D are an exception and must be proportioned such that ρ is not greater than 1.25.

For the structure loaded in the N-S direction, the structural system consists of special moment frames, and r_{ix} is taken as the maximum of the shears in any two adjacent columns in the plane of a moment frame divided by the story shear. For interior columns that have girders framing into both sides, only 70 percent of the individual column shear need be included in this sum. In the N-S direction, there are four identical frames. Each of these frames has eight columns. Using the portal frame idealization, the shear in an interior column will be $V_{interior}$ = 0.25 (2/14) V = 0.0357V.

Similarly, the shear in an exterior column will be $V_{exterior}$= 0.25 (1/14) V = 0.0179V.

For two adjacent interior columns:

$$r_{ix} = \frac{0.7(V_{int} + V_{int})}{V} = \frac{0.7(0.0375V + 0.0375V)}{V} = 0.0525$$

For one interior and one exterior column:

$$r_{ix} = \frac{(0.7V_{int} + V_{ext})}{V} = \frac{0.7(0.0375V) + 0.0179V)}{V} = 0.0441$$

The larger of these values will produce the largest value of ρ_x. Hence, for a floor diaphragm area A_x equal to 102.5 × 216 = 22,140 square ft:

$$\rho_x = 2 - \frac{20}{0.0525\sqrt{22{,}140}} = -0.56$$

As this value is less than 1.0, ρ will be taken as 1.0 in the N-S direction.

For seismic forces acting in the E-W direction, the walls carry significant shear, and for the purposes of computing ρ, it will be assumed that they take all the shear. According to the *Provisions*, r_{ix} for walls is taken as the shear in the wall multiplied by $10/l_w$ and divided by the story shear. The term l_w represents the plan length of the wall in feet. Thus, for one wall:

$$r_{max_x} = r_{ix} = \frac{0.25V(10/20)}{V} = 0.125$$

Only 80 percent of the ρ value based on the above computations need be used because the walls are part of a dual system. Hence, in the E-W direction

$$\rho_x = 0.8\left(2 - \frac{20}{0.125\sqrt{22{,}140}}\right) = 0.740$$

and as with the N-S direction, ρ may be taken as 1.0. Note that ρ need not be computed for the columns of the frames in the dual system, as this will clearly not control.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure beam-to-column connections at both ends of a single beam (moment frame system) or failure of a single shear wall with aspect ratio greater than 1.0 (shear wall system) would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Alternatively, if the structure is regular in plan and there are at least 2 bays of perimeter framing on each side of the structure in each orthogonal direction, it is permitted to use, ρ = 1.0. Per 2003 *Provisions* Sec. 4.3.1.4.3 special moment frames in Seismic Design Category D must be configured such that the structure satisfies the criteria for ρ = 1.0. There are no reductions in the redundancy factor for dual systems. Based on the preliminary design, ρ = 1.0 for because the structure has a perimeter moment frame and is regular.]

For the Berkeley structure, the basic ASCE 7 load combinations that must be considered are:

$1.2D + 1.6L$
$1.2D + 0.5L \pm 1.0E$
$0.9D \pm 1.0E$

The ASCE 7 load combination including only 1.4 times dead load will not control for any condition in this building.

Substituting E from the *Provisions*, with ρ taken as 1.0, the following load combinations must be used for earthquake:

$(1.2 + 0.2S_{DS})D + 0.5L + E$
$(1.2 + 0.2S_{DS})D + 0.5L - E$
$(0.9 - 0.2\ S_{DS})D + E$
$(0.9 - 0.2S_{DS})D - E$

Finally, substituting 1.10 for S_{DS}, the following load combinations must be used for earthquake:

$1.42D + 0.5L + E$
$1.42D + 0.5L - E$

$0.68D + E$
$0.68D - E$

It is very important to note that use of the ASCE 7 load combinations in lieu of the combinations given in ACI Chapter 9 requires use of the alternate strength reduction factors given in ACI 318 Appendix C:

Flexure without axial load $\phi = 0.80$
Axial compression, using tied columns $\phi = 0.65$ (transitions to 0.8 at low axial loads)
Shear if shear strength is based on nominal axial-flexural capacity $\phi = 0.75$
Shear if shear strength is <u>not</u> based on nominal axial-flexural capacity $\phi = 0.55$
Shear in beam-column joints $\phi = 0.80$

[The strength reduction factors in ACI 318-02 have been revised to be consistent with the ASCE 7 load combinations. Thus, the factors that were in Appendix C of ACI 318-99 are now in Chapter 9 of ACI 318-02, with some modification. The strength reduction factors relevant to this example as contained in ACI 318-02 Sec. 9.3 are:

Flexure without axial load $\varphi = 0.9$ (tension-controlled sections)
Axial compression, using tied columns $\varphi = 0.65$ (transitions to 0.9 at low axial loads)
Shear if shear strength is based o nominal axial-flexural capacity $\varphi = 0.75$
Shear if shear strength is *not* based o nominal axial-flexural capacity $\varphi = 0.60$
Shear in beam-column joints $\varphi = 0.85$]

6.4.3 Comments on the Structure's Behavior Under E-W Loading

Frame-wall interaction plays an important role in the behavior of the structure loaded in the E-W direction. This behavior is beneficial to the design of the structure because:

1. For frames without walls (Frames 1, 2, 7, and 8), the shears developed in the girders (except for the first story) do not differ greatly from story to story. This allows for a uniformity in the design of the girders.

2. For frames containing structural walls (Frames 3 through 6), the overturning moments in the structural walls are reduced significantly as a result of interaction with the remaining frames (Frames 1, 2, 7, and 8).

3. For the frames containing structural walls, the 40-ft-long girders act as outriggers further reducing the overturning moment resisted by the structural walls.

The actual distribution of story forces developed in the different frames of the structure is shown in Figure 6-7.[2] This figure shows the response of Frames 1, 2, and 3 only. By symmetry, Frame 8 is similar to Frame 1, Frame 7 is similar to Frame 2, and Frame 6 is similar to Frame 3. Frames 4 and 5 have a response that is virtually identical to that of Frames 3 and 6.

As may be observed from Figure 6-7, a large reverse force acts at the top of Frame 3 which contains a structural wall. This happens because the structural wall pulls back on (supports) the top of Frame 1. The deflected shape of the structure loaded in the E-W direction (see Figure 6-5) also shows the effect of frame-wall interaction because the shape is neither a cantilever mode (wall alone) nor a shear mode

[2]The analysis used to create Figures 6-7 and 6-8 did not include the 5 percent torsional eccentricity or the 30 percent orthogonal loading rules specified by the *Provisions*. The eccentricity and orthogonal load were included in the analysis carried out for member design.

(frame alone). It is the "straightening out" of the deflected shape of the structure that causes the story shears in the frames without walls to be relatively equal.

A plot of the story shears in Frames 1, 2, and 3 is shown in Figure 6-8. The distribution of overturning moments is shown in Figure 6-9 and indicates that the relatively stiff Frames 1 and 3 resist the largest portion of the total overturning moment. The reversal of moment at the top of Frame 3 is a typical response characteristic of frame-wall interaction.

6.4.4 Analysis of Frame-Only Structure for 25 Percent of Lateral Load

When designing a dual system, *Provisions* Sec. 5.2.2.1 [4.3.1.1] requires the frames (without walls) to resist at least 25 percent of the total base shear. This provision ensures that the dual system has sufficient redundancy to justify the increase from R = 6 for a special reinforced concrete structural wall to R = 8 for a dual system (see *Provisions* Table 5.2.2 [4.3-1]). [Note that R = 7 per 2003 *Provisions* Table 4.3-1.] The 25 percent analysis was carried out using the ETABS program with the mathematical model of the building being identical to the previous version except that the panels of the structural wall were removed. The boundary elements of the walls were retained in the model so that behavior of the interior frames (Frames 3, 4, 5, and 6) would be analyzed in a rational way.

The results of the analysis are shown in Figures 6-10, 6-11, and 6-12. In these figures, the original analysis (structural wall included) is shown by a solid line and the 25 percent (backup frame) analysis (structural wall removed) is shown by a dashed line. As can be seen, the 25 percent rule controls only at the lower level of the building.

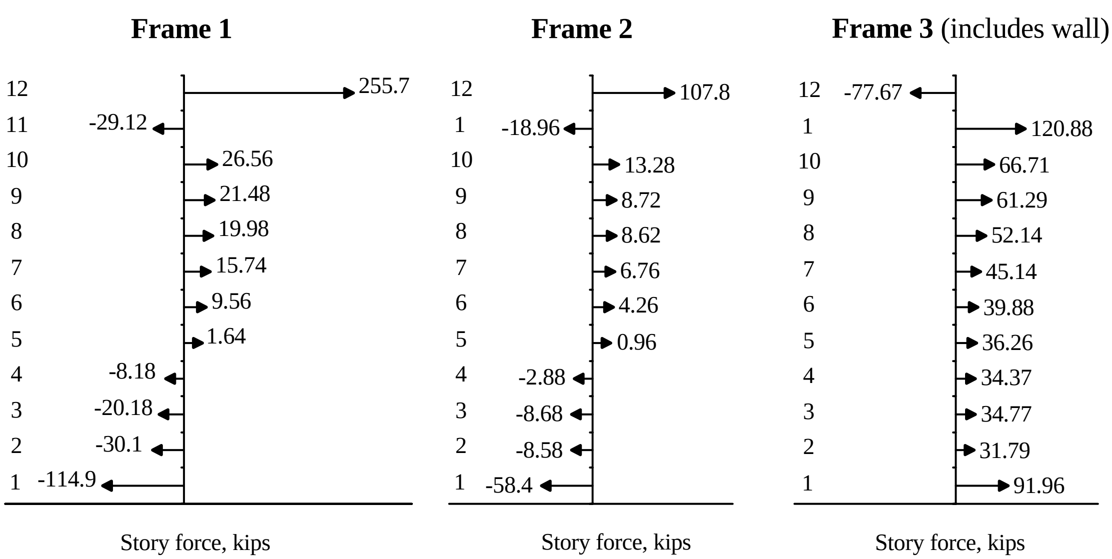


Figure 6-7 Story forces in the E-W direction (1.0 kip = 4.45 kN).

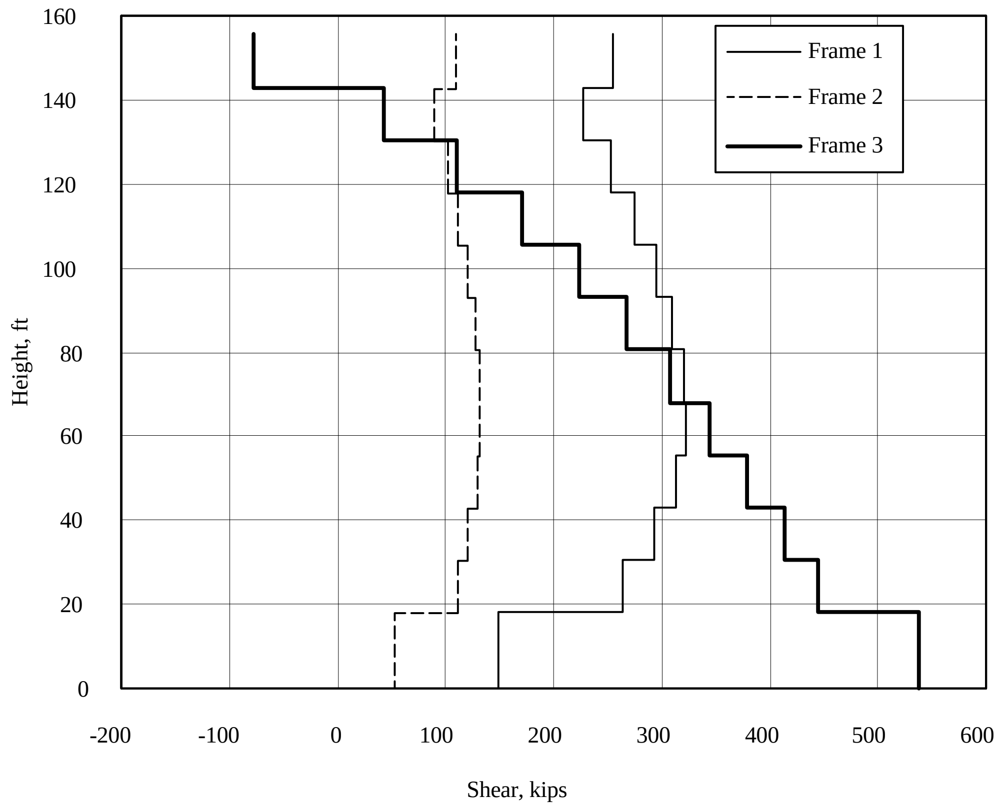


Figure 6-8 Story shears in the E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

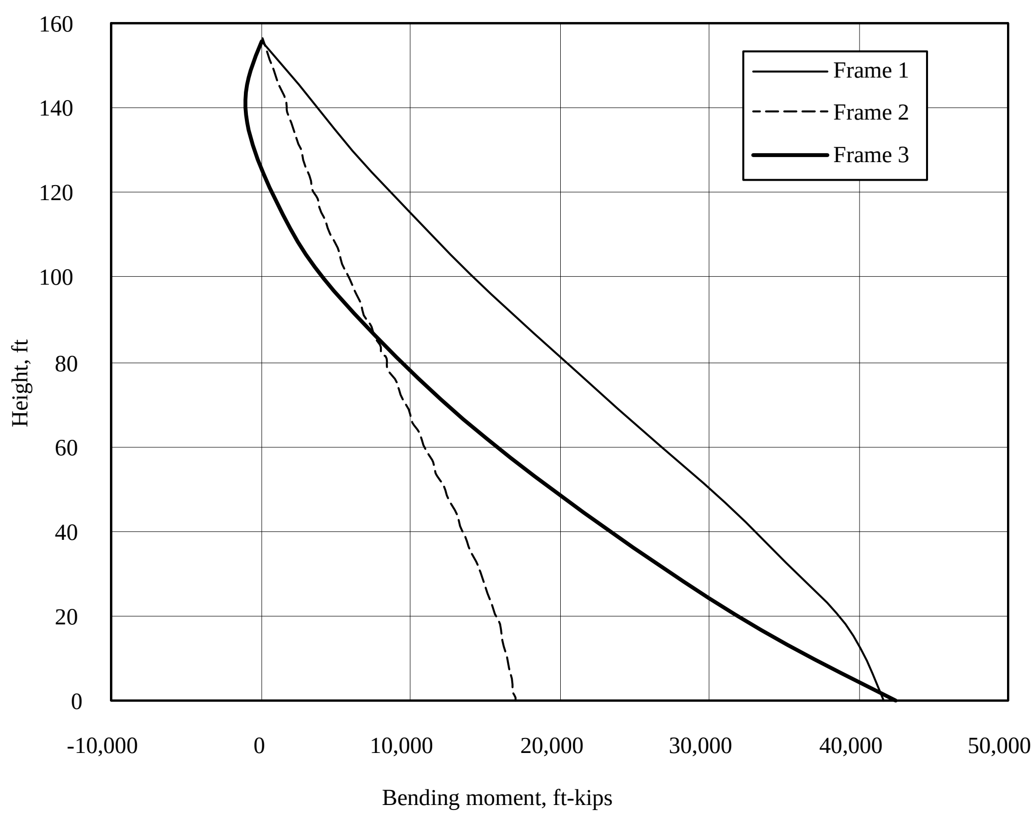


Figure 6-9 Story overturning moments in the E-W direction (1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m).

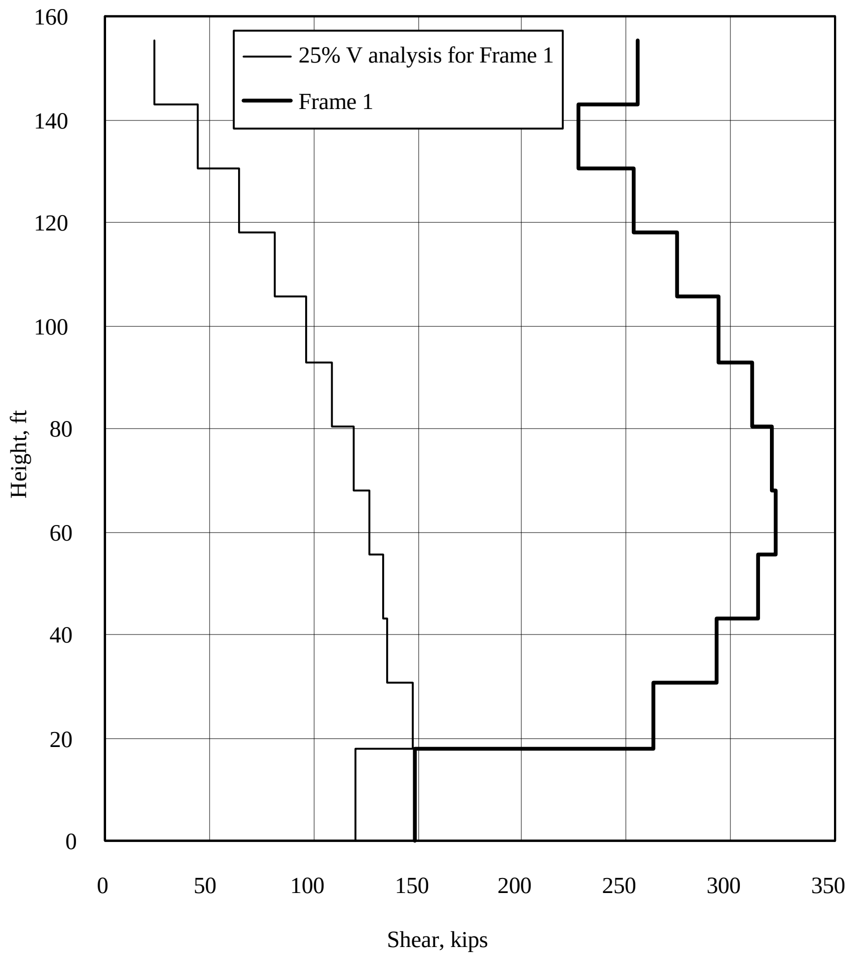


Figure 6-10 25 percent story shears, Frame 1 E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

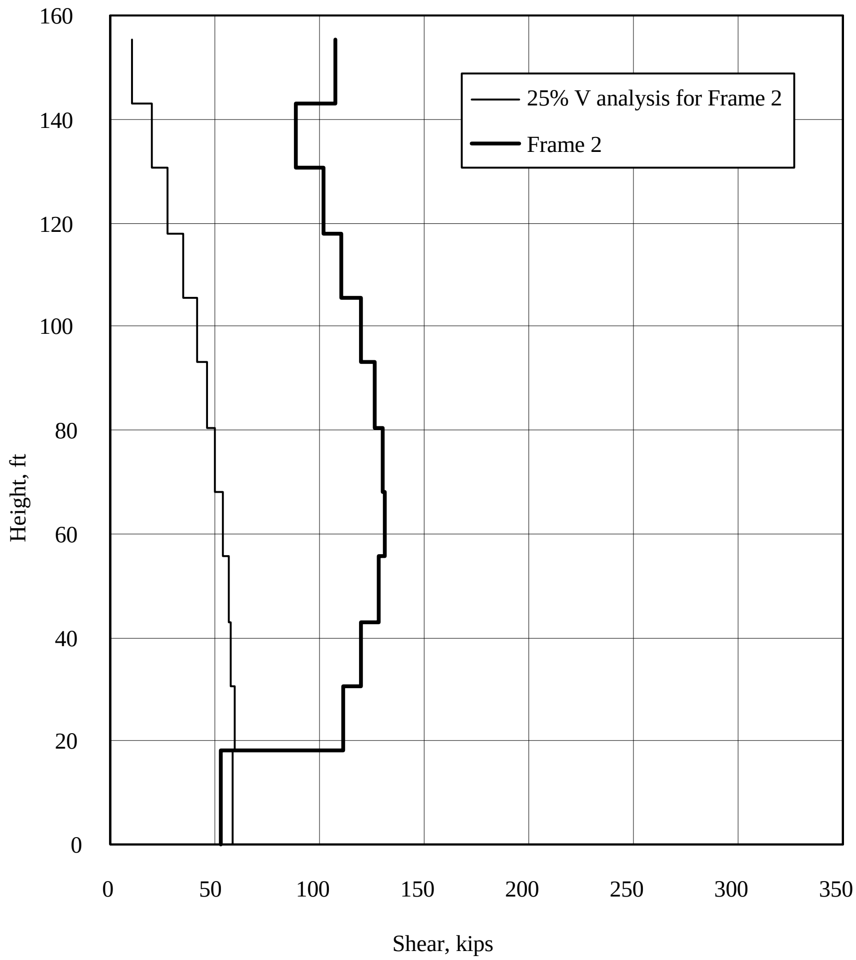


Figure 6-11 25 percent story shears, Frame 2 E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

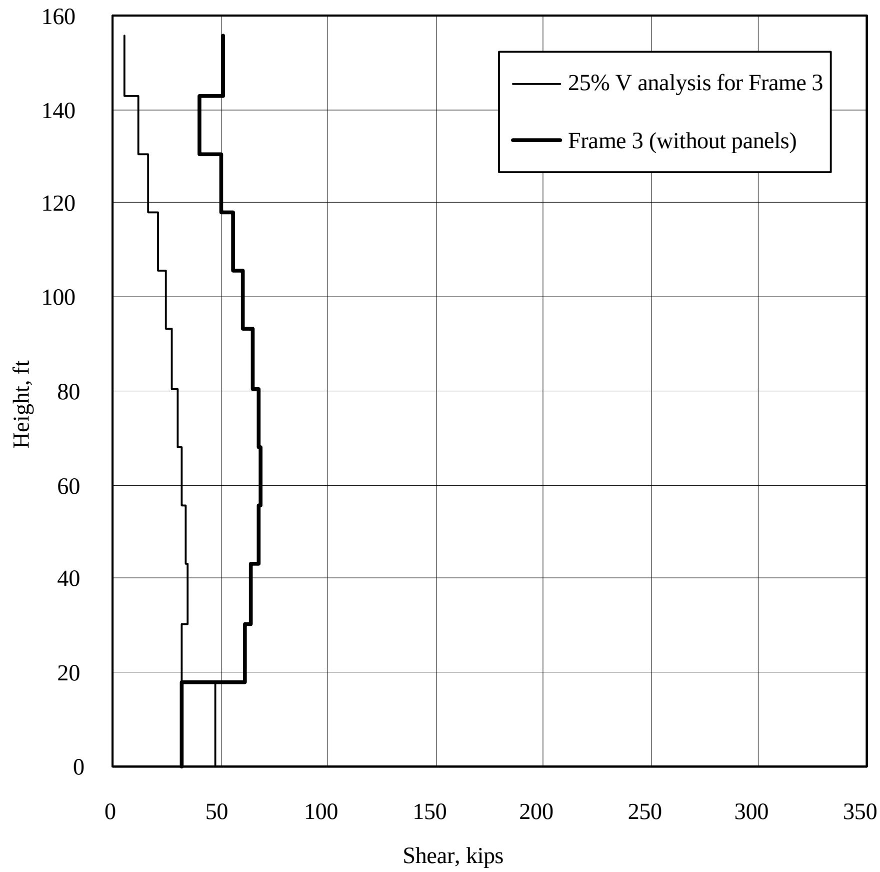


Figure 6-12 25 percent story shear, Frame 3 E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN)..

6.4.5 Design of Frame Members for the Berkeley Building

A sign convention for bending moments is required in flexural design. In this example, when the steel at the top of a beam section is in tension, the moment is designated as a negative moment. When the steel at the bottom is in tension, the moment is designated as a positive moment. All moment diagrams are drawn using the reinforced concrete or tension-side convention. For beams, this means negative moments are plotted on the top and positive moments are plotted on the bottom. For columns, moments are drawn on the tension side of the member.

6.4.5.1 Initial Calculations

Before the quantity and placement of reinforcement is determined, it is useful to establish, in an overall sense, how the reinforcement will be distributed. The preliminary design established that beams would have a maximum depth of 32 in. and columns would be 30 in. by 30 in. In order to consider the beam-column joints "confined" per ACI 318 Sec. 21.5, it was necessary to set the beam width to 22.5 in., which is 75 percent of the column width.

In order to determine the effective depth used for the design of the beams, it is necessary to estimate the size and placement of the reinforcement that will be used. In establishing this depth, it is assumed that #8 bars will be used for longitudinal reinforcement and that hoops and stirrups will be constructed from #3 deformed bars. In all cases, clear cover of 1.5 in. is assumed. Since this structure has beams spanning in two orthogonal directions, it is necessary to layer the flexural reinforcement as shown in Figure 6-13. The reinforcement for the E-W spanning beams was placed in the upper and lower layers because the strength demand for these members is somewhat greater than that for the N-S beams.

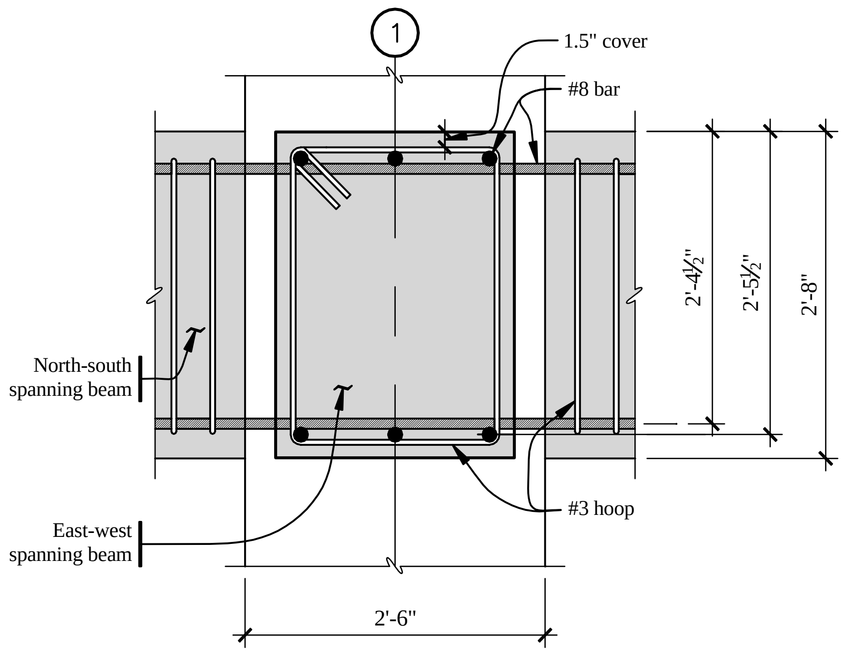


Figure 6-13 Layout for beam reinforcement (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm).

Given Figure 6-13, compute the effective depth for both positive and negative moment as:

Beams spanning in the E-W direction, d = 32 - 1.5 - 0.375 - 1.00/2 = 29.6 in.
Beams spanning in the N-S direction, d = 32 - 1.5 - 0.375 - 1.0 - 1.00/2 = 28.6 in.

For negative moment bending, the effective width is 22.5 in. for all beams. For positive moment, the slab is in compression and the effective T-beam width varies according to ACI 318 Sec. 8.10. The effective widths for positive moment are as follows (with the parameter controlling effective width shown in parentheses):

20-ft beams in Frames 1 and 8	b = 22.5 + 20(12)/12 = 42.5 in. (span length)
Haunched beams	b = 22.5 + 2[8(4)] = 86.5 in. (slab thickness)
30-ft beams in Frames A, B, C, and D	b = 22.5 + [6(4)] = 46.5 in. (slab thickness)

ACI 318 Sec. 21.3.2 controls the longitudinal reinforcement requirements for beams. The minimum reinforcement to be provided at the top and bottom of any section is:

$$A_{s,min} = \frac{200 b_w d}{f_y} = \frac{200(22.5)29.6}{60{,}000} = 2.22 \text{ in.}^2$$

This amount of reinforcement can be supplied by three #8 bars with A_s = 2.37 in.^{2} Since the three #8 bars will be provided continuously top and bottom, reinforcement required for strength will include these #8 bars.

Before getting too far into member design, it is useful to check the required tension development length for hooked bars since the required length may control the dimensions of the columns and the boundary elements of the structural walls.

From Eq. 21-6 of ACI 318 Sec. 21.5.4.1, the required development length is:

$$l_{dh} = \frac{f_y d_b}{65\sqrt{f_c'}}$$

For NW concrete, the computed length should not be less than 6 in. or 8d_b. For LW concrete, the minimum length is the larger of 1.25 times that given by ACI 318 Eq. 21-6, 7.5 in., or 10d_b. For f_c' = 4,000 psi LW concrete, ACI 318 Eq. 21-6 controls for #3 through #11 bars.

For straight "top" bars, $l_d = 3.5 l_{dh}$ and for straight bottom bars, $l_d = 2.5 l_{dh}$. These values are applicable only when the bars are anchored in well confined concrete (e.g., column cores and plastic hinge regions with confining reinforcement). The development length for the portion of the bar extending into unconfined concrete must be increased by a factor of 1.6. Development length requirements for hooked and straight bars are summarized in Table 6-13.

Where hooked bars are used, the hook must be 90 degrees and be located within the confined core of the column or boundary element. For bars hooked into 30-in.-square columns with 1.5 in. of cover and #4 ties, the available development length is 30 - 1.50 - 0.5 = 28.0 in. With this amount of available length, there will be no problem developing hooked bars in the columns. As required by ACI 318 Sec. 12.5, hooked bars have a 12d_b extension beyond the bend. ACI 318 Sec. 7.2 requires that #3 through #8 bars have a 6d_b bend diameter and #9 through #11 bars have a 8d_b diameter.

Table 6-13 is applicable to bars anchored in joint regions only. For development of bars outside of joint regions, ACI 318 Chapter 12 should be used.

Table 6-13 Tension Development Length Requirements for Hooked Bars and Straight Bars in 4,000 psi LW Concrete

Bar Size	d_b (in.)	l_{dh} hook (in.)	l_d top (in.)	l_d bottom (in.)
#4	0.500	9.1	31.9	22.8
#5	0.625	11.4	39.9	28.5
#6	0.750	13.7	48.0	34.3
#7	0.875	16.0	56.0	40.0
#8	1.000	18.2	63.7	45.5
#9	1.128	20.6	72.1	51.5
#10	1.270	23.2	81.2	58.0
#11	1.410	25.7	90.0	64.2

1.0 in. = 25.4 mm.

6.4.5.2 Design of Members of Frame 1 for E-W Loading

For the design of the members of Frame 1, the equivalent lateral forces of Table 6-7b were applied at an eccentricity of 10.5 ft together with 30 percent of the forces of Table 6-7a applied at an eccentricity of 5.0 ft. The eccentricities were applied in such a manner as to maximize torsional response and produce the largest shears in Frame 1.

For this part of the example, the design and detailing of all five beams and one interior column of Level 5 are presented in varying amounts of detail. The beams are designed first because the flexural capacity of the as-designed beams is a factor in the design and detailing of the column and the beam-column joint. The design of a corner column will be presented later.

Before continuing with the example, it should be mentioned that the design of ductile reinforced concrete moment frame members is dominated by the flexural reinforcement in the beams. The percentage and placement of beam flexural reinforcement governs the flexural rebar cutoff locations, the size and spacing of beam shear reinforcement, the cross-sectional characteristics of the column, the column flexural reinforcement, and the column shear reinforcement. The beam reinforcement is critical because the basic concept of ductile frame design is to force most of the energy-absorbing deformation to occur through inelastic rotation in plastic hinges at the ends of the beams.

In carrying out the design calculations, three different flexural strengths were used for the beams. These capacities were based on:

Design strength	$\phi = 0.8$, tensile stress in reinforcement at 1.00 f_y
Nominal strength	$\phi = 1.0$, tensile stress in reinforcement at 1.00 f_y
Probable strength	$\phi = 1.0$, tensile stress in reinforcement at 1.25 f_y

Various aspects of the design of the beams and other members depend on the above capacities as follows:

Beam rebar cutoffs	Design strength
Beam shear reinforcement	Probable strength of beam
Beam-column joint strength	Probable strength of beam
Column flexural strength	6/5 × nominal strength of beam
Column shear strength	Probable strength of column

In addition, beams in ductile frames will always have top and bottom longitudinal reinforcement throughout their length. In computing flexural capacities, only the tension steel will be considered. This is a valid design assumption because reinforcement ratios are quite low, yielding a depth to the neutral axis similar to the depth of the compression reinforcement (d'/d is about 0.08, while the neutral axis depth at ultimate ranges from 0.07 to 0.15 times the depth) .[3]

The preliminary design of the girders of Frame 1 was based on members with a depth of 32 in. and a width of 22.5 in. The effective depth for positive and negative bending is 29.6 in. and the effective widths for positive and negative bending are 42.5 and 22.5 in., respectively. This assumes the stress block in compression is less than the 4.0-inch flange thickness.

The layout of the geometry and gravity loading on the three eastern-most spans of Level 5 of Frame 1 as well as the unfactored gravity and seismic moments are illustrated in Figure 6-14. The seismic moments are taken directly from the ETABS program output and the gravity moments were computed by hand

[3]See Chapter 1 of the 2nd Edition of the *Handbook of Concrete Engineering* edited by Mark Fintel (New York: Van Nostrand Reinhold Company, 1984).

using the coefficient method of ACI 318 Chapter 8. Note that all moments (except for midspan positive moment) are given at the face of the column and that seismic moments are considerably greater than those due to gravity.

Factored bending moment envelopes for all five spans are shown in Figure 6-14. Negative moment at the supports is controlled by the $1.42D + 0.5L + 1.0E$ load combination, and positive moment at the support is controlled by $0.68D - 1.0E$. Midspan positive moments are based on the load combination $1.2D + 1.6L$. The design process is illustrated below starting with Span B-C.

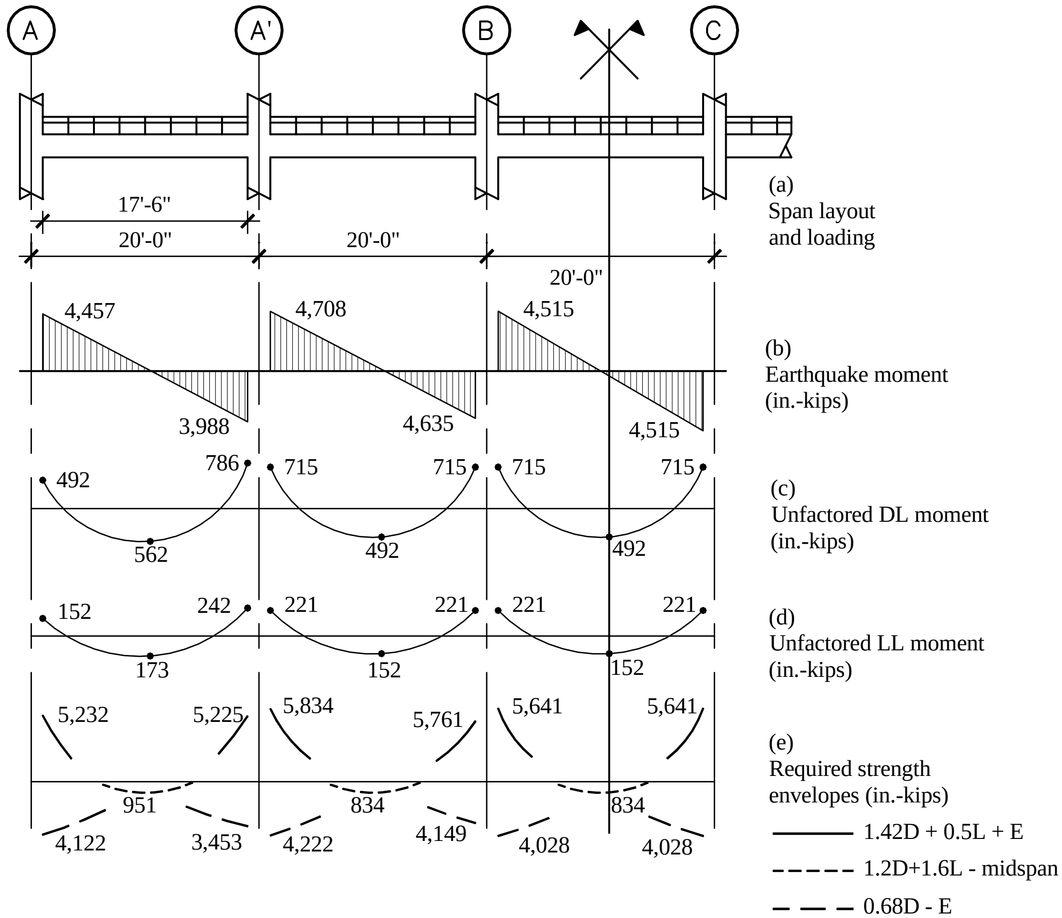


Figure 6-14 Bending moments for Frame 1 (1.0 ft = 0.3048 m, 1.0 in.-kip = 0.113 kN-m).

6.4.5.2.1 Span B-C

1. Design for Negative Moment at the Face of the Support

   $M_u = 1.42(-715) + 0.5(-221) + 1.0(-4515) = -5{,}641$ in.-kips

   Try two #9 bars in addition to the three #8 bars required for minimum steel:

   $A_s = 2(1.0) + 3(0.79) = 4.37$ in.^{2}
   $f_c' = 4{,}000$ psi
   $f_y = 60$ ksi

Width b for negative moment = 22.5 in.
d = 29.6 in.
Depth of compression block, $a = A_s f_y / .85 f_c' b$
a = 4.37 (60)/[0.85 (4) 22.5] = 3.43 in.
Design strength, $\phi M_n = \phi A_s f_y (d - a/2)$
ϕM_n = 0.8(4.37)60(29.6 - 3.43/2) = 5,849 in.-kips > 5,641 in.-kips OK

2. Design for Positive Moment at Face of Support

M_u = [-0.68(715)] + [1.0(4,515)] = 4,028 in.-kips

Try two #7 bars in addition to the three #8 bars already provided as minimum steel:

A_s = [2(0.60)] + [3(0.79)] = 3.57 in.^{2}
Width b for positive moment = 42.5 in.
d = 29.6 in.
a = [3.57(60)]/[0.85(4)42.5] = 1.48 in.
ϕM_n = 0.8(3.57) 60(29.6 - 1.48/2) = 4,945 in.-kips > 4,028 in.-kips OK

3. Positive Moment at Midspan

M_u = [1.2(492)] + [1.6(152)] = 833.6 in.-kips

Minimum reinforcement (three #8 bars) controls by inspection. This positive moment reinforcement will also work for Spans A'-B and A-A'.

6.4.5.2.2 Span A'-B

1. Design for Negative Moment at the Face of Support A'

M_u = [1.42(-715)] + [0.5(-221)] + [1.0(-4,708)] = -5,834 in.-kips

Three #8 bars plus two #9 bars (capacity = 5,849 in.-kips) will work as shown for Span B-C.

2. Design for Negative Moment at the Face of Support B

M_u = [1.42(-715)] + [0.5(-221)] + [1.0(-4,635)] = -5,761 in.-kips

As before, use three #8 bars plus two #9 bars.

3. Design for Positive Moment at Face of Support A'

M_u = [-0.68(715)] + [1.0(4708)] = 4,222 in.-kips

Three #8 bars plus two #7 bars (capacity = 4,945 in.-kips) works as shown for Span B-C.

4. Design for Positive Moment at Face of Support B'

M_u = [-0.68(715)] + [1.0(4,635)] = 4,149 in.-kips

As before, use three #8 bars plus two #7 bars.

6.4.5.2.3 Span A-A'

1. Design for Negative Moment at the Face of Support A

   M_u = [1.42(-492)] + [0.5(-152)] + [1.0(-4,457)] = -5,232 in.-kips

   Try three #8 bars plus two #8 bars:

   A_s = 5 × 0.79 = 3.95 in.^{2}
   Width b for negative moment = 22.5 in.
   d = 29.6 in.
   a = [3.95(60)/[0.85(4)22.5] = 3.10 in.
   ϕM_n =[0.8(3.95)60] (29.6 - 3.10/2) = 5,318 in.-kips > 5,232 in.-kips OK

2. Design for Negative Moment at the Face of Support A'

   M_u = [1.42(-786)] + [0.5(-242)] + [1.0(-3,988)] = -5,225 in.-kips

   Use three #8 bars plus two #9 bars as required for Support B of Span A'-B.

3. Design for Positive Moment at Face of Support A

   M_u = [-0.68(492)] + [1.0(4,457)] = 4,122 in.-kips

   Three #8 bars plus two #7 bars will be sufficient.

4. Design for Positive Moment at Face of Support A'

   M_u = [-0.68(786)] + [1.0(3,988)] = 3,453 in.-kips

   As before, use three #8 bars plus two #7 bars.

6.4.5.2.4 Spans C-C' and C'-D

Reinforcement requirements for Spans C-C' and C'-D are mirror images of those computed for Spans A'-B and A-A', respectively.

In addition to the computed strength requirements and minimum reinforcement ratios cited above, the final layout of reinforcing steel also must satisfy the following from ACI 318 Sec. 21.3.2:

Minimum of two bars continuous top and bottom	OK (three #8 bars continuous top and bottom)
Positive moment strength greater than 50 percent negative moment strength at a joint	OK (at all joints)
Minimum strength along member greater than 0.25 maximum strength	OK (A_s provided = three #8 bars is more than 25 percent of reinforcement provided at joints)

The preliminary layout of reinforcement is shown in Figure 6-15. The arrangement of bars actually provided is based on the above computations with the exception of Span B-C where a total of six #8 top bars were used instead of the three #8 bars plus two #9 bars combination. Similarly, six #8 bars are used at the bottom of Span B-C. The use of six #8 bars is somewhat awkward for placing steel, but it allows for the use of three #8 continuous top and bottom at all spans. An alternate choice would have been to use two #9 continuous across the top of Span B-C instead of the three of the #8 bars. However, the use of two #9 bars (ρ = 0.00303) does not meet the minimum reinforcement requirement ρ_{min} = 0.0033.

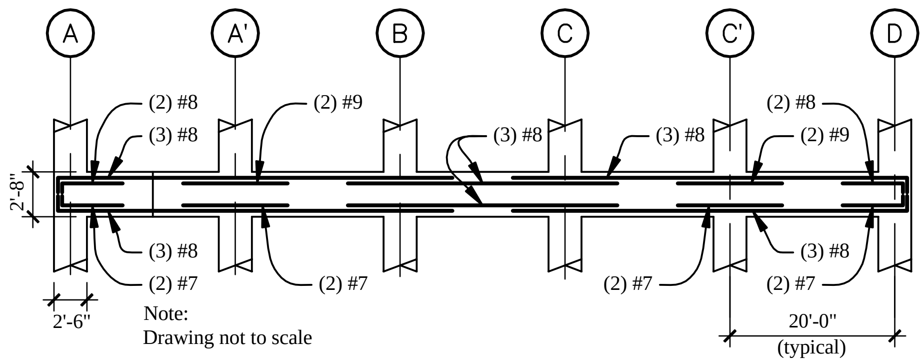


Figure 6-15 Preliminary rebar layout for Frame 1 (1.0 ft = 03.048 m).

As mentioned above, later phases of the frame design will require computation of the design strength and the maximum probable strength at each support. The results of these calculations are shown in Table 6-14.

Table 6-14 Design and Maximum Probable Flexural Strength For Beams in Frame 1

Item		Location					
		A	A'	B	C	C'	D
Negative Moment	Reinforcement	five #8	three #8 + two #9	six #8	six #8	three #8 + two #9	five #8
	Design Strength (in.-kips)	5,318	5,849	6,311	7,100	5,849	5,318
	Probable Strength (in.-kips)	8,195	8,999	9,697	9,697	8,999	8,195
Positive Moment	Reinforcement	three #8 + two #7	three #8 + two #7	six #8	six #8	three #8 + two #7	three #8 + two #7
	Design Strength (in.-kips)	4,945	4,945	6,510	6,510	4,945	4,945
	Probable Strength (in.-kips)	7,677	7,677	10,085	10,085	7,655	7,677

1.0 in.-kip = 0.113 kN-m.

As an example of computation of probable strength, consider the case of six #8 top bars:

A_s = 6(0.79) = 4.74 in.^{2}
Width b for negative moment = 22.5 in.
d = 29.6 in.
Depth of compression block, $a = A_s(1.25f_y)/0.85f_c'b$
a = 4.74(1.25)60/[0.85(4)22.5] = 4.65 in.
$M_{pr} = 1.0A_s(1.25f_y)(d - a/2)$

M_{pr} = 1.0(4.74)1.25(60)(29.6 - 4.65/2) = 9,697 in.-kips

For the case of six #8 bottom bars:

A_s = 6(0.79) = 4.74 in.^{2}
Width b for positive moment = 42.5 in.
d = 29.6 in.
a = 4.74(1.25)60/(0.85 × 4 × 42.5) = 2.46 in.
M_{pr} = 1.0(4.74)1.25(60)(29.6 - 2.46/2) = 10,085 in.-kips

6.4.5.2.5 Adequacy of Flexural Reinforcement in Relation to the Design of the Beam-Column Joint

Prior to this point in the design process, the layout of reinforcement has been considered preliminary because the quantity of reinforcement placed in the girders has a direct bearing on the magnitude of the stresses developed in the beam-column joint. If the computed joint stresses are too high, the only remedies are increasing the concrete strength, increasing the column area, changing the reinforcement layout, or increasing the beam depth. The option of increasing concrete strength is not viable for this example because it is already at the maximum (4,000 psi) allowed for LW concrete. If absolutely necessary, however, NW concrete with a strength greater than 4,000 psi may be used for the columns and beam-column joint region while the LW concrete is used for the joists and beams.

The design of the beam-column joint is based on the requirements of ACI 318 Sec. 21.5.3. The determination of the forces in the joint of the column on Gridline C of Frame 1 is based on Figure 6-16a, which shows how plastic moments are developed in the various spans for equivalent lateral forces acting to the east. An isolated subassemblage from the frame is shown in Figure 6-16b. The beam shears shown in Figure 6-16c are based on the probable moment strengths shown in Table 6-14.

For forces acting from west to east, compute the earthquake shear in Span B-C:

$V_E = (M_{pr}^- + M_{pr}^+)/l_{clear}$ = (9,697 + 10,085)/(240 - 30) = 94.2 kips

For Span C-C':

V_E = (10,085 + 8,999)/(240 - 30) = 90.9 kips

With the earthquake shear of 94.2 and 90.9 kips being developed in the beams, the largest shear that theoretically can be developed in the column above Level 5 is 150.5 kips. This is computed from equilibrium as shown at the bottom of Figure 6-16:

94.2(9.83) + 90.9(10.50) = 2V_c(12.5/2)
V_c = 150.4 kips

With equal spans, gravity loads do not produce significant column shears, except at the end column, where the seismic shear is much less. Therefore, gravity loads are not included in this computation.

The forces in the beam reinforcement for negative moment are based on six #8 bars at 1.25 f_y:

$T = C$ = 1.25(60)[(6(0.79)] = 355.5 kips

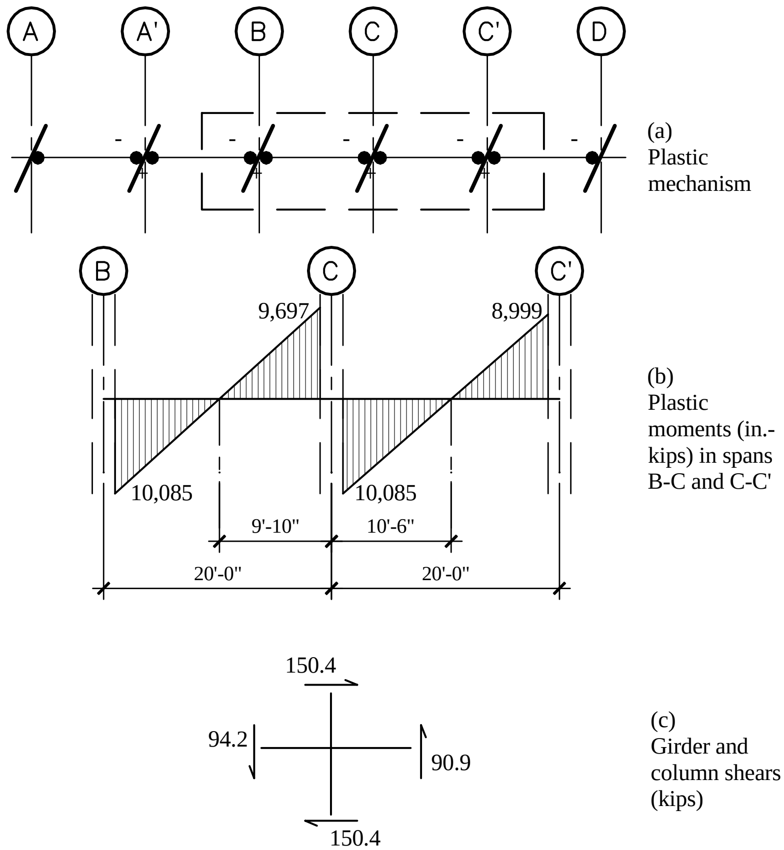


Figure 6-16 Diagram for computing column shears (1.0 ft = 0.3048 m, 1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

For positive moment, six #8 bars also are used, assuming $C = T$, C = 355.5 kips.

As illustrated in Figure 6-17, the joint shear force V_j is computed as:

$$
\begin{aligned}
V_j &= T + C - V_E \\
&= 355.5 + 355.5 - 150.4 \\
&= 560.6 \text{ kips}
\end{aligned}
$$

The joint shear stress is:

$$v_j = \frac{V_j}{d_c^2} = \frac{560.5}{30^2} = 623 \text{ psi}$$

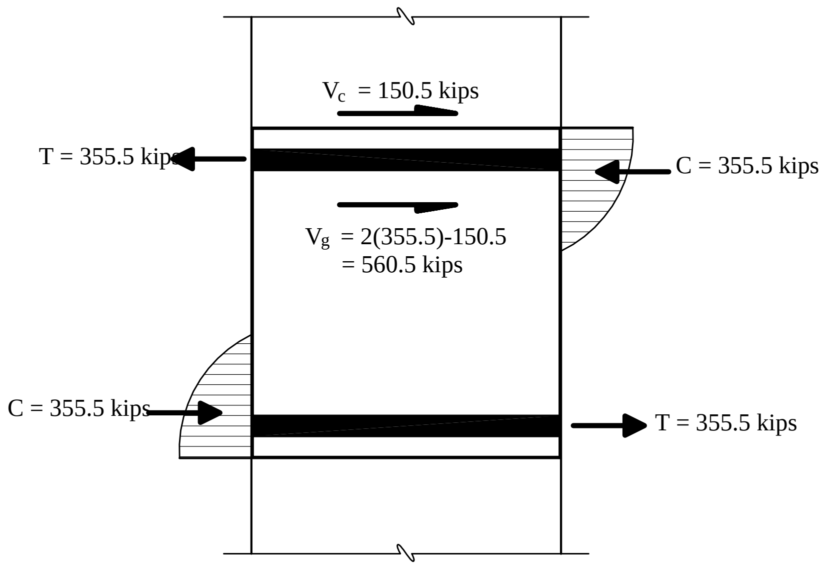


Figure 6-17 Computing joint shear stress (1.0 kip = 4.45kN).

For joints confined on three faces or on two opposite faces, the allowable shear stress for LW concrete is based on ACI 318 Sec. 21.5.3. Using $\phi = 0.80$ for joints (from ACI Appendix C) and a factor of 0.75 as a modifier for LW concrete:

$$v_{j,allowable} = 0.80(0.75)(15\sqrt{4{,}000}) = 569 \text{ psi}$$

[Note that for joints, φ = 0.85 per ACI 318-02 Sec 9.3 as referenced by the 2003 *Provisions*. See Sec 6.4.2 for discussion.]

Since the actual joint stress (623 psi) exceeds the allowable stress (569 psi), the joint is overstressed. One remedy to the situation would be to reduce the quantity of positive moment reinforcement. The six #8 bottom bars at Columns B and C could be reduced to three #8 bars plus two #7 bars. This would require a somewhat different arrangement of bars than shown in Figure 6-15. It is left to the reader to verify that the joint shear stress would be acceptable under these circumstances. Another remedy would be to increase the size of the column. If the column is increased in size to 32 in. by 32 in., the new joint shear stress is:

$$v_j = \frac{V_j}{d_c^2} = \frac{560.5}{32^2} = 547 \text{psi} < 569 \text{ psi}$$

which is also acceptable. For now we will proceed with the larger column, but as discussed later, the final solution will be to rearrange the bars to three #8 plus two #7.

Joint stresses would be checked for the other columns in a similar manner. Because the combined area of top and bottom reinforcement used at Columns A, A', C', and D is less than that for Columns B and C, these joints will not be overstressed.

Given that the joint stress is acceptable, ACI 318 Sec. 21.5.2.3 controls the amount of reinforcement required in the joint. Since the joint is not confined on all four sides by a beam, the total amount of transverse reinforcement required by ACI 318 Sec. 21.4.4 will be placed within the depth of the joint. As shown later, this reinforcement consists of four-leg #4 hoops at 4 in. on center.

Because the arrangement of steel is acceptable from a joint strength perspective, the cutoff locations of the various bars may be determined (see Figure 6-15 for a schematic of the arrangement of reinforcement). The three #8 bars (top and bottom) required for minimum reinforcement are supplied in one length that runs continuously across the two end spans and are cut off in the center span. An additional three #8 bars are placed top and bottom in the center span; these bars are cut off in Spans A'-B and C-C'. At Supports A, A', C' and D, shorter bars are used to make up the additional reinforcement required for strength.

To determine where bars should be cut off in each span, it is assumed that theoretical cutoff locations correspond to the point where the continuous top and bottom bars develop their design flexural strength. Cutoff locations are based on the members developing their design flexural capacities (f_y = 60 ksi and ϕ = 0.8). Using calculations similar to those above, it has been determined that the design flexural strength supplied by a section with only three #8 bars is 3,311 in.-kips for positive moment and 3,261 in.-kips for negative moment.

Sample cutoff calculations are given first for Span B-C. To determine the cutoff location for negative moment, it is assumed that the member is subjected to earthquake plus 0.68 times the dead load forces. For positive moment cutoffs, the loading is taken as earthquake plus 1.42 times dead load plus 0.5 times live load. Loading diagrams for determining cut off locations are shown in Figure 6-18.

For negative moment cutoff locations, refer to Figure 6-19a, which is a free body diagram of the west end of the member. Since the goal is to develop a negative moment capacity of 3,261 in.-kips in the continuous #8 bars summing moments about Point A in Figure 6-19a:

$$6{,}311 + \frac{0.121x^2}{2} - 73.7x = 3{,}261$$

In the above equation, 6,311 (in.-kips) is the negative moment capacity for the section with six #8 bars, 0.121 (kips/in.) is 0.68 times the uniform dead load, 73.3 kips is the end shear, and 3,261 in.-kips is the design strength of the section with three #8 bars. Solving the quadratic equation results in x = 42.9 in. ACI 318 Sec. 12.10.3 requires an additional length equal to the effective depth of the member or 12 bar diameters (whichever is larger). Consequently, the total length of the bar beyond the face of the support is 42.9 + 29.6 = 72.5 in. and a 6 ft-1 in. extension beyond the face of the column could be used.

For positive moment cutoff, see Figure 6-14 and Figure 6-19b. The free body diagram produces an equilibrium equation as:

$$6{,}510 - \frac{0.281x^2}{2} - 31.6x = 3{,}311$$

where the distance x is computed to be 75.7 in. Adding the 29.6 in. effective depth, the required extension beyond the face of the support is 76.0 + 29.6 = 105.3 in, or 8 ft-9 in. Note that this is exactly at the midspan of the member.

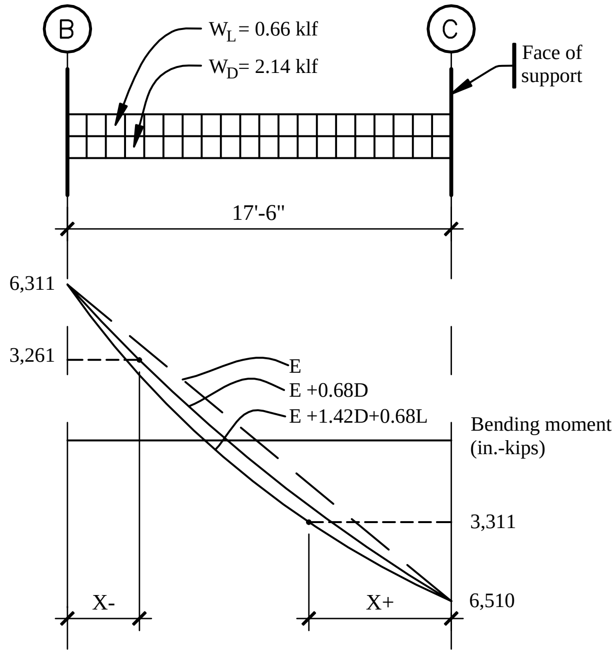


Figure 6-18 Loading for determination of rebar cutoffs (1.0 ft = 0.3048 m, 1.0 klf = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

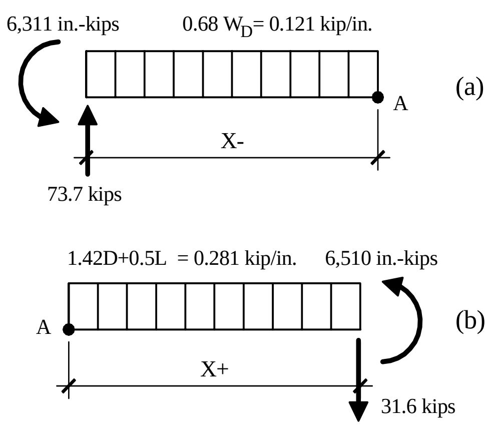


Figure 6-19 Free body diagrams (1.0 kip = 4.45kN, 1.0 klf = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

Clearly, the short bottom bars shown in Figure 6-15 are impractical. Instead, the bottom steel will be rearranged to consist of three #8 plus two #7 bars continuous. Recall that this arrangement of reinforcement will satisfy joint shear requirements, and the columns may remain at 30 in. by 30 in.

As shown in Figure 6-20, another requirement in setting cutoff length is that the bar being cut off must have sufficient length to develop the strength required in the adjacent span. From Table 6-13, the required development length of the #9 top bars in tension is 72.1 in. if the bar is anchored in a confined joint region. The confined length in which the bar is developed is shown in Figure 6-20 and consists of the column depth plus twice the depth of the girder. This length is 30 + 32 + 32 = 94 in., which is greater than the 72.1 in. required. The column and girder are considered confined because of the presence of closed hoop reinforcement as required by ACI 318 Sec. 21.3.3 and 21.4.4.

The bottom bars are spliced at the center of Spans A'-B and C-C' as shown in Figure 6-21. The splice length is taken as the bottom bar Class B splice length for #8 bars. According to ACI 318 Sec. 12.15, the splice length is 1.3 times the development length. From ACI 318 Sec. 12.2.2, the development length (l_d) is computed from:

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c+K_{tr}}{d_b}\right)}$$

using α = 1.0 (bottom bar), β =1.0 (uncoated), γ = 1.0 (#9 bar), λ = 1.3 (LW concrete), taking c as the cover (1.5 in.) plus the tie dimension (0.5 in.) plus 1/2 bar diameter (0.50 in.) = 2.50 in., and using K_{tr} = 0, the development length for one #9 bar is:

$$l_d = \frac{3}{40}\left(\frac{60{,}000}{\sqrt{4{,}000}}\right)\frac{1\times1\times1.0\times1.3}{\left(\dfrac{2.5+0}{1.0}\right)}(1.0) = 37.0 \text{ in.}$$

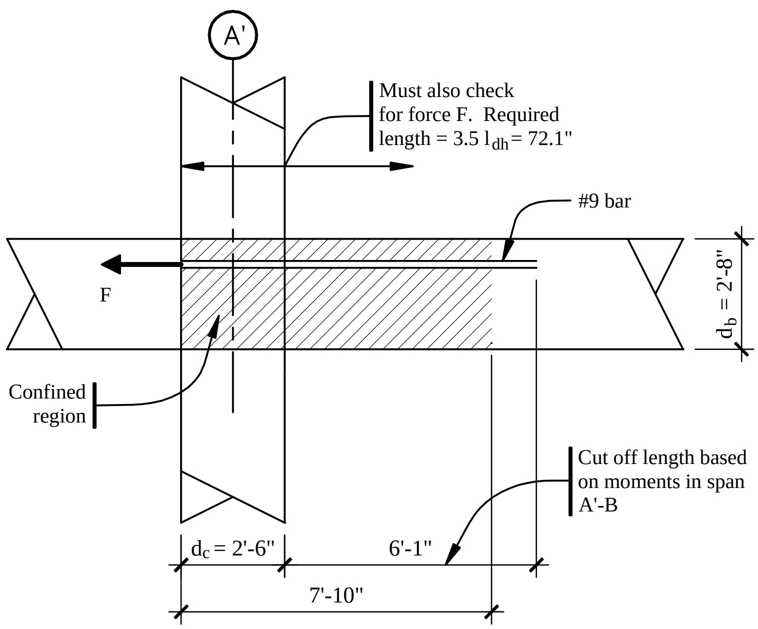


Figure 6-20 Development length for top bars (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm).

The splice length = 1.3 × 37.0 = 48.1 in. Therefore, use a 48-in. contact splice. According to ACI 318 Sec. 21.3.2.3, the entire region of the splice must be confined by closed hoops spaced no closer than d/4 or 4 in.

The final bar placement and cutoff locations for all five spans are shown in Figure 6-21. Due to the different arrangement of bottom steel, the strength at the supports must be recomputed. The results are shown in Table 6-15.

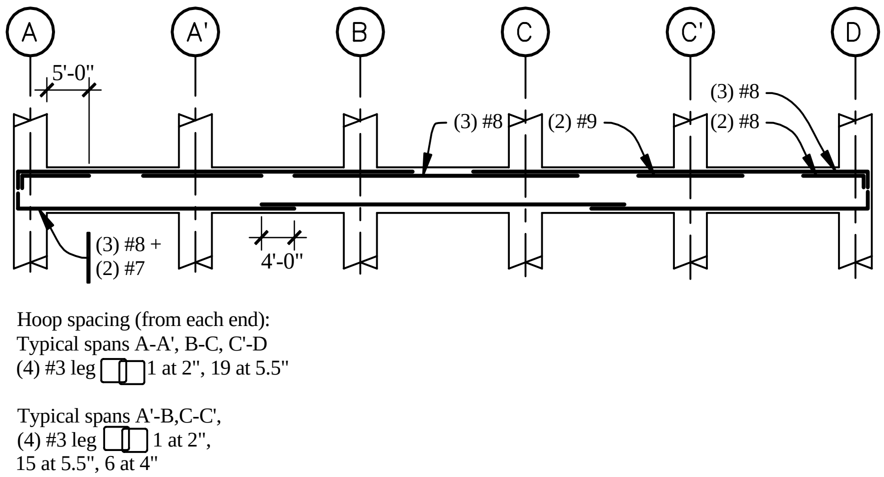


Figure 6-21 Final bar arrangement (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm).

Table 6-15 Design and Maximum Probable Flexural Strength For Beams in Frame 1 (Revised)

Item		Location A	A'	B	C	C'	D
Negative Moment	Reinforcement	five #8	three #8 + two #9	six #8	six #8	three #8 + two #9	five #8
	Design Moment (in.-kips)	5,318	5,849	6,311	6,311	5,849	5,318
	Probable moment (in.-kips)	8,195	8,999	9,696	9,696	8,999	8,195
Positive Moment	Reinforcement	three #8 + two #7	three #8 + two #7	three #8 + two #7	three #8 + two #7	three #8 + two #7	three #8 + two #7
	Design Moment (in.-kips)	4,944	4,944	4,944	4,944	4,944	4,944
	Probable moment (in.-kips)	7,677	7,677	7,677	7,677	7,677	7,677

1.0 in.-kip = 0.113 kN-m.

6.4.5.2.6 Transverse Reinforcement

Transverse reinforcement requirements are covered in ACI 318 Sec. 21.3.3 (minimum reinforcement) and 21.3.4 (shear strength).

To avoid nonductile shear failures, the shear strength demand is computed as the sum of the factored gravity shear plus the maximum probable earthquake shear. The maximum probable earthquake shear is based on the assumption that $\phi = 1.0$ and the flexural reinforcement reaches a tensile stress of $1.25f_y$. The probable moment strength at each support is shown in Table 6-15.

Figure 6-22 illustrates the development of the design shear strength envelopes for Spans A-A', A'-B, and B-C. In Figure 6-22a, the maximum probable earthquake moments are shown for seismic forces acting to the east (solid lines) and to the west (dashed lines). The moments shown occur at the face of the supports.

The earthquake shears produced by the maximum probable moments are shown in Figure 6-22b. For Span A-B, the values shown in the figure are:

$$V_E = \frac{M^-_{pr} + M^+_{pr}}{l_{clear}}$$

where l_{clear} = 17 ft-6 in. = 210 in.

Note that the earthquake shears act in different directions depending on the direction of load.

For forces acting to the east, V_E = (9696 + 7677) / 210 = 82.7 kips.
For forces acting to the west, V_E = (8999 + 7677) / 210 = 79.4 kips.

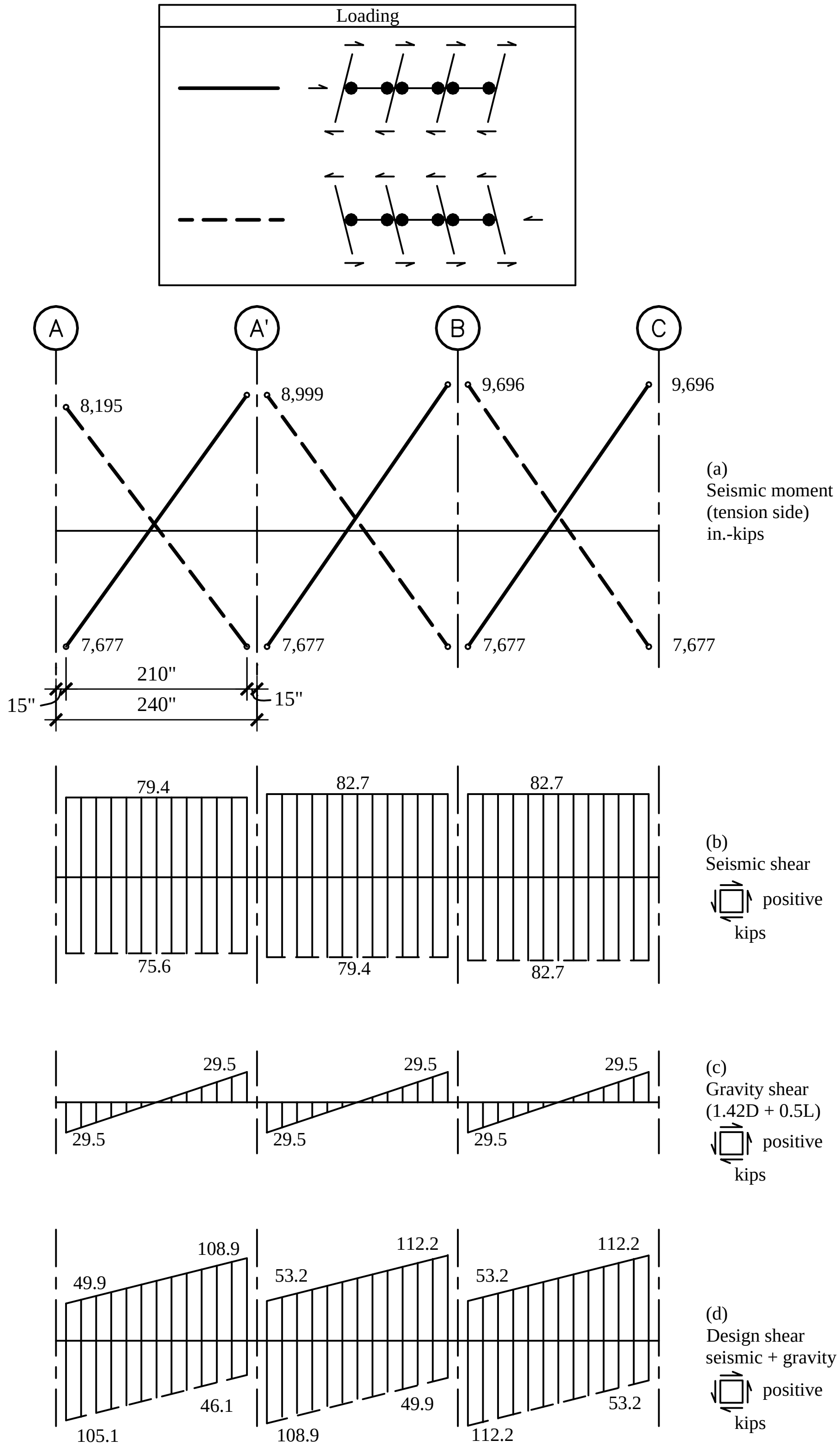


Figure 6-22 Shear forces for transverse reinforcement (1.0 in = 25.4 mm, 1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

The gravity shears shown in Figure 6-22c are:

Factored gravity shear = $V_G = 1.42V_{dead} + 0.5V_{live}$
$V_{dead} = 2.14 \times 17.5/2 = 18.7$ kips
$V_{live} = 0.66 \times 17.5/2 = 5.8$ kips
$V_G = 1.42(18.7) + 0.5(5.8) = 29.5$ kips

Total design shears for each span are shown in Figure 6-22d. The strength envelope for Span B-C is shown in detail in Figure 6-23, which indicates that the maximum design shears is 82.7 + 29.5 = 112.2 kips. While this shear acts at one end, a shear of 82.7 - 29.5 = 53.2 kips acts at the opposite end of the member.

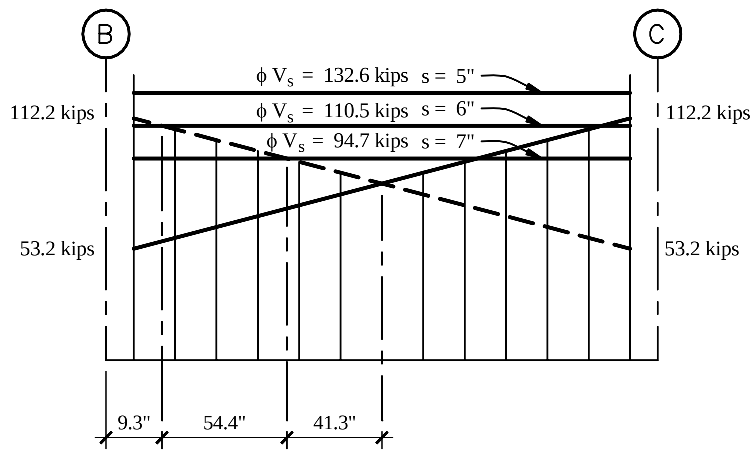


Figure 6-23 Detailed shear force envelope in Span B-C (1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

In designing shear reinforcement, the shear strength can consist of contributions from concrete and from steel hoops or stirrups. However, according to ACI 318 Sec. 21.3.4.2, the design shear strength of the concrete must be taken as zero when the axial force is small ($P_u/A_g f'_c < 0.05$) and the ratio V_E/V_u is greater than 0.5. From Figure 6-22, this ratio is $V_E/V_u = 82.7/112.2 = 0.73$, so concrete shear strength must be taken as zero. Using the ASCE 7 compatible ϕ for shear = 0.75, the spacing of reinforcement required is computed as described below. [Note that this is the basic strength reduction factor for shear per ACI 318-02 Sec 9.3. See Sec 6.4.2 for discussion.]

Compute the shear at d = 29.6 in. from the face of the support:

$V_u = \phi V_s = 112.2 - (29.6/210)(112.2 - 53.2) = 103.9$ kips
$V_s = A_v f_y d/s$

Assuming four #3 vertical legs ($A_v = 0.44$ in.^{2}), $f_v = 60$ ksi and $d = 29.6$ in., compute the required spacing:

$s = \phi A_v f_y d/V_u = 0.75[4(0.11)](60)(29.6/103.9) = 5.65$ in., say 5.5 in.

At midspan, the design shear $V_u = (112.2 + 53.2)/2 = 82.7$ kips. Compute the required spacing:

$s = 0.75[4(0.11)](60)(29.6/82.7) = 7.08$ in., say 7.0 in.

Check maximum spacing per ACI 318 Sec. 21.3.3.2:

$d/4 = 29.6/4 = 7.4$ in.
$8d_b = 8(1.0) = 8.0$ in.
$24d_h = 24(3/8) = 9.0$ in.

The spacing must vary between 5.5 in. at the support and 7.0 in. at midspan. Due to the relatively flat shear force gradient, a spacing of 5.5 in. will be used for the full length of the beam. The first hoop must be placed 2 in. from the face of the support. This arrangement of hoops will be used for Spans A-A', B-C, and C'-D. In Spans A'-B and C-C', the bottom flexural reinforcement is spliced and hoops must be placed over the splice region at $d/4$ or a maximum of 4 in. on center.

ACI 318 Sec. 21.3.3.1 states that closed hoops are required over a distance of twice the member depth from the face of the support. From that point on, stirrups may be used. For the girders of Frame 1, however, stirrups will not be used, and the hoops will be used along the entire member length. This is being done because the earthquake shear is a large portion of the total shear, the girder is relatively short, and the economic premium is negligible.

Where hoops are required (first 64 in. from face of support), longitudinal reinforcing bars should be supported as specified in ACI 318 Sec. 7.10.5.3. Hoops should be arranged such that every corner and alternate longitudinal bar is supported by a corner of the hoop assembly and no bar should be more than 6 in. clear from such a supported bar. Details of the transverse reinforcement layout for all spans of Level 5 of Frame 1 are shown in Figure 6-21.

6.4.5.3 Design of a Typical Interior Column of Frame 1

This section illustrates the design of a typical interior column on Gridline A'. The column, which supports Level 5 of Frame 1, is 30 in. square and is constructed from 4,000 psi LW concrete, 60 ksi longitudinal reinforcement, and 60 ksi transverse reinforcement. An isolated view of the column is shown in Figure 6-24. The flexural reinforcement in the beams framing into the column is shown in Figure 6-21. Using simple tributary area calculations (not shown), the column supports an unfactored axial dead load of 528 kips and an unfactored axial live load of 54 kips. The ETABS analysis indicates that the maximum axial earthquake force is 84 kips, tension or compression. The load combination used to compute this force consists of full earthquake force in the E-W direction, 30 percent of the N-S force, and accidental torsion. Because no beams frame into this column along Gridline A', the column bending moment for N-S forces can be neglected. Hence, the column is designed for axial force plus uniaxial bending.

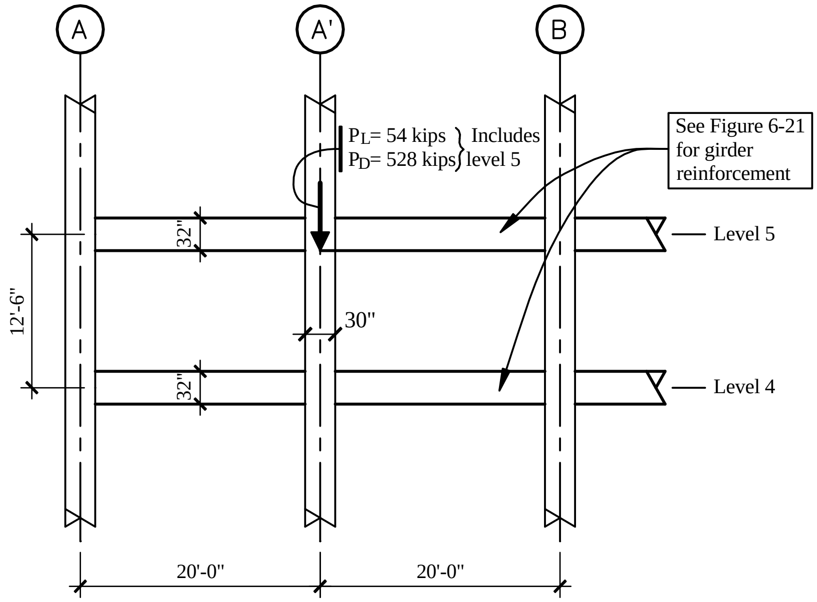


Figure 6-24 Layout and loads on column of Frame A (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

6.4.5.3.1 Longitudinal Reinforcement

To determine the axial design loads, use the basic load combinations:

$1.42D + 0.5L + 1.0\mathrm{E}$
$0.68D - 1.0E$.

The combination that results in maximum compression is:

$P_u = 1.42(528) + 0.5(54) + 1.0(84) = 861$ kips (compression)

The combination for minimum compression (or tension) is:

$P_u = 0.68(528) - 1.0(84) = 275$ kips (compression)

The maximum axial compression force of 861 kips is greater than $0.1f_c'A_g = 0.1(4)(30^2) = 360$ kips. Thus, according to ACI 318 Sec. 21.4.2, the nominal column flexural strength must be at least 6/5 of the nominal flexural strength of the beams framing into the column. Beam moments at the face of the support are used for this computation. These capacities are provided in Table 6-15.

Nominal (negative) moment strength at end A' of Span A-A' = 5,849/0.8 = 7,311 in.-kips
Nominal (positive) moment strength at end A' of Span A' B = 4,945/0.8 = 6,181 in.-kips
Average nominal moment framing into joint = 6,746 in.-kips
Nominal column design moment = 6/5 × 6746 = 8,095 in.-kips.

Knowing the factored axial load and the required design flexural strength, a column with adequate capacity must be selected. Figure 6-25 gives design curves for 30 in. by 30 in. columns of 4,000 psi concrete and reinforcement consisting of 12 #8, #9, or #10 bars. These curves, computed with a

Microsoft Excel spreadsheet, are based on a ϕ factor of 1.0 as required for nominal strength. At axial forces of 275 kips and 861 kips, solid horizontal lines are drawn. The dots on the lines represent the required nominal flexural strength (8095 in.-kips) at each axial load level. These dots must lie to the left of the curve representing the design columns. For both the minimum and maximum axial forces, a column with 12 #8 bars (with A_s = 9.48 in.^{2} and 1.05 percent of steel) is clearly adequate.

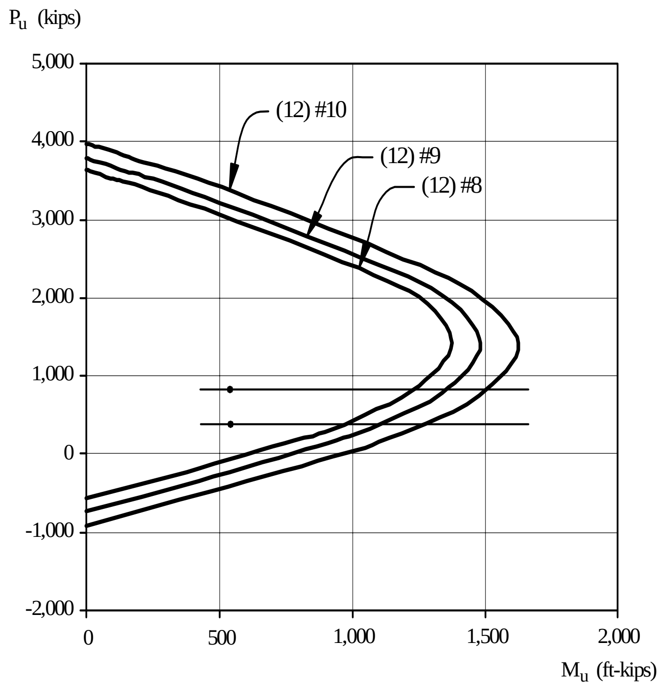


Figure 6-25 Design interaction diagram for column on Gridline A' (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

6.4.5.3.2 Transverse Reinforcement

ACI 318 Sec. 21.4.4 gives the requirements for minimum transverse reinforcement. For rectangular sections with hoops, ACI 318 Eq. 21-3 and 21-4 are applicable:

$$A_{sh} = 0.3\left(\frac{sh_c f'_c}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$

$$A_{sh} = 0.09 sh_c \frac{f'_c}{f_{yh}}$$

The first of these equations controls when A_g/A_{ch} > 1.3. For the 30-in.-by-30-in. columns:

A_{ch} = (30 - 1.5 - 1.5)^2 = 729 in.^{2}
A_g = 30 (30) = 900 in.^{2}
A_g/A_{ch} = 900/729 = 1.24

ACI 318 Eq. 21-4 therefore controls.

For LW concrete, try hoops with four #4 legs and f_c' = 4,000 psi:

h_c = 30 - 1.5 - 1.5 - 0.25 - 0.25 = 26.5 in.
s = [4 (0.2) 60,000]/[0.09 (26.5) 4000] = 5.03 in.

However, the maximum spacing of transverse reinforcement is the lesser of one-fourth the maximum column dimension (30/4=7.5 in.), six bar diameters (6 × 1.0 = 6.0 in.), or the dimension s_x where:

$$s_x = 4 + \frac{14 - h_x}{3}$$

and where h_x is the maximum horizontal spacing of hoops or cross ties. For the column with twelve #8 bars and #4 hoops and cross ties, h_x = 8.833 in. and s_x = 5.72 in. The 5.03-in. spacing required by ACI Eq. 21-4 controls, so a spacing of 5 in. will be used. This transverse reinforcement must be spaced over a distance l_o = 30 in. at each end of the member and, according to ACI 318 Sec. 21.5.2, must extend through the joint at (at most) the same spacing.

ACI 318 Sec. 21.4.4.6 requires a maximum spacing of transverse reinforcement in the region of the column not covered by Sec. 21.4.4.4. The maximum spacing is the smaller of 6.0 in. or $6d_b$, which for #8 bars is also 6 in. ACI 318 requires transverse steel at this spacing, but it does not specify what the details of reinforcement should be. In this example, hoops and crossties with the same details as those placed in the critical regions of the column are used.

6.4.5.3.3 Transverse Reinforcement Required for Shear

The amount of transverse reinforcement computed above is the minimum required. The column also must be checked for shear with the column shears being based on the maximum probable moments in the beams that frame into the column. The average probable moment is roughly 1.25/0.8 (ϕ = 0.8) times the average design moment = (1.25/0.8)(5397) = 8,433 in.-kips. With a clear height of 118 in., the column shear can be estimated at 8433/(0.5x118) = 143 kips. This shear will be compared to the capacity provided by the 4-leg #4 hoops spaced at 6 in. on center. If this capacity is well in excess of the demand, the columns will be acceptable for shear.

For the design of column shear capacity, the concrete contribution to shear strength may be considered because $P_u > A_g f_c'/20$. Using a shear strength reduction factor of 0.85 for sand-LW concrete (ACI 318 Sec. 11.2.1.2) in addition to the capacity reduction factor for shear, the design shear strength contributed by concrete is:

$$\phi V_c = \phi 0.75\sqrt{f_c'}b_c d_c = 0.75(0.85)(\sqrt{4{,}000}(30)(27.5) = 33.2 \text{ kips}$$

$$\phi V_s = \phi A_v f_y d / s = 0.75(4)(0.2)(60)(27.5)/6 = 165 \text{ kips}$$

$$\phi V_n = \phi V_c + \phi V_s = 33.2 + 165 = 198.2 \text{ kips} > 143 \text{ kips} \qquad \text{OK}$$

The column with the minimum transverse steel is therefore adequate for shear. The final column detail with both longitudinal and transverse reinforcement is given in Figure 6-26. The spacing of reinforcement through the joint has been reduced to 4 in. on center. This is done for practical reasons only. Column bar splices, where required, should be located in the center half of the of the column and must be proportioned as (Class B) tension splices.

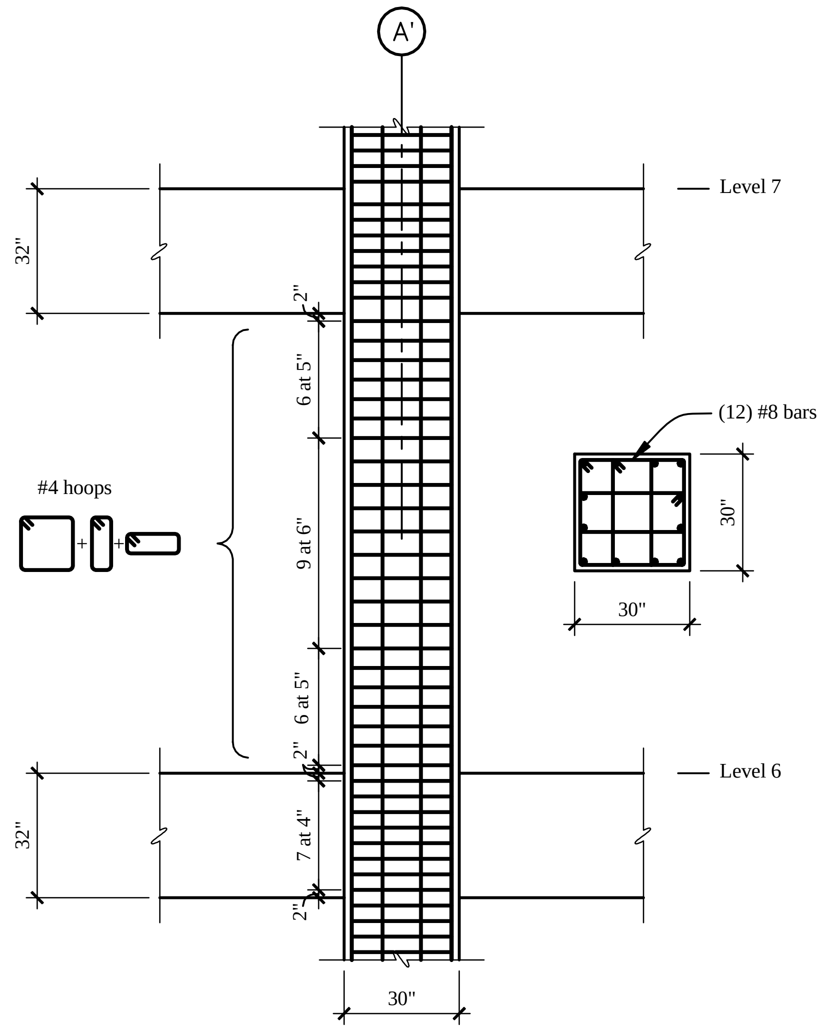


Figure 6-26 Details of reinforcement for column (1.0 in = 25.4 mm).

6.4.5.4 Design of Haunched Girder

The design of a typical haunched girder of Level 5 of Frame 3 is now illustrated. This girder is of variable depth with a maximum depth of 32 in. at the support and a minimum depth of 20 in. for the middle half of the span. The length of the haunch at each end (as measured from the face of the support) is 8 ft-9 in. The width of the web of girder is 22.5 in. throughout.

Based on a tributary gravity load analysis, this girder supports an average of 3.375 kips/ft of dead load and 0.90 kips/ft of reduced live load. For the purpose of estimating gravity moments, a separate analysis of the girder was carried out using the SAP2000 program. End A of the girder was supported with half-height columns pinned at midstory and End B, which is supported by a shear wall, was modeled as fixed. Each haunch was divided into four segments with nonprismatic section properties used for each segment. The loading and geometry of the girder is shown in Figure 6-27a.

For determining earthquake forces, the entire structure was analyzed using the ETABS program. This analysis included 100 percent of the earthquake forces in the E-W direction and 30 percent of the earthquake force in the N-S direction, and accidental torsion. Each of these systems of lateral forces was placed at a 5 percent eccentricity with the direction of the eccentricity set to produce the maximum seismic shear in the member.

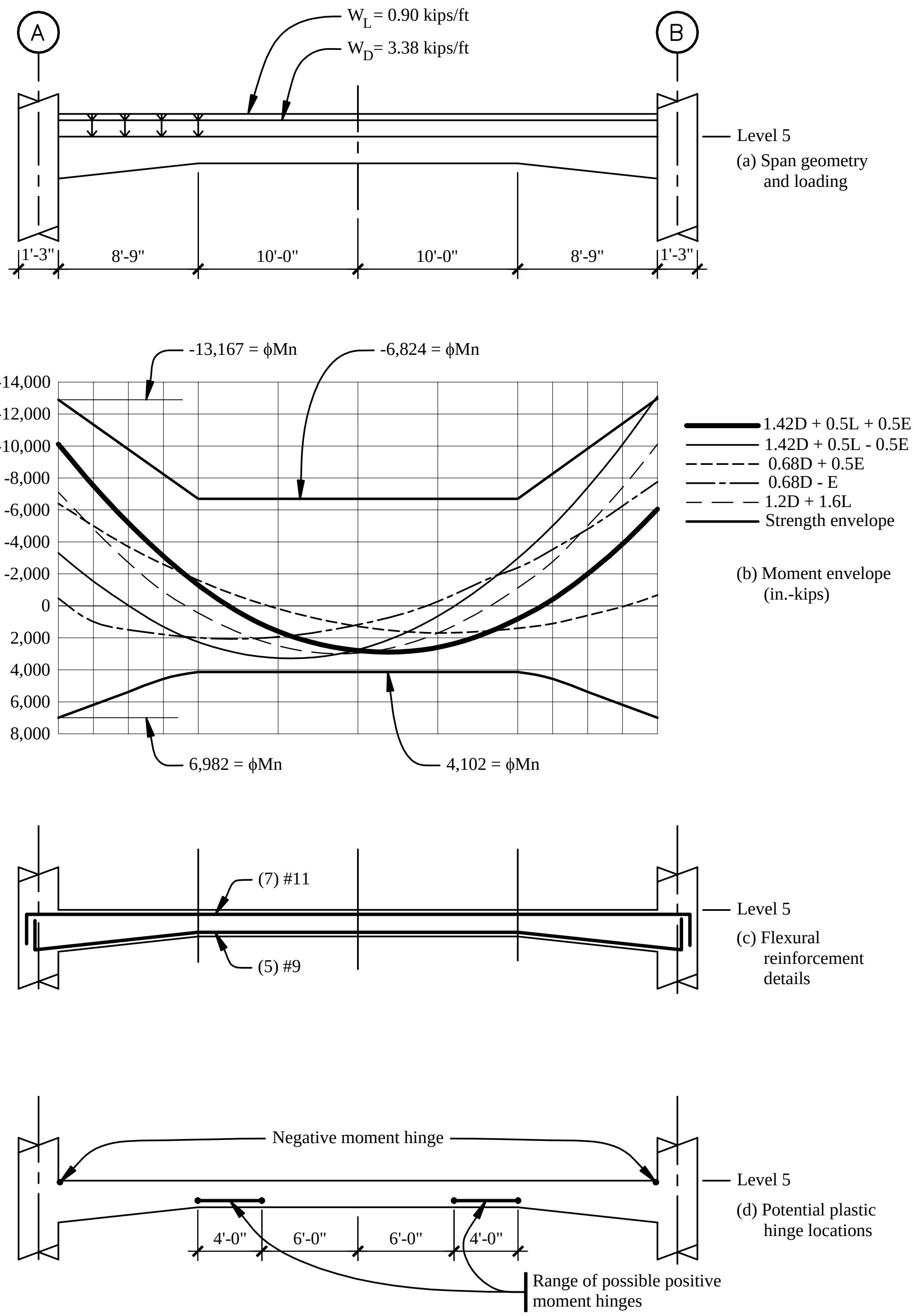


Figure 6-27 Design forces and detailing of haunched girder (1.0 ft = 0.3048 m, 1.0 k/ft = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

6.4.5.4.1 Design and Detailing of Longitudinal Reinforcement

The results of the analysis for five different load combinations are shown in Figure 6-27b. Envelopes of maximum positive and negative moment are shown on the figure indicate that $1.42D + 0.5L \pm E$ controls negative moment at the support, $0.68D \pm E$ controls positive moment at the support, and $1.2D + 1.6L$ controls positive moment at midspan. The maximum positive moment at the support is less than 50 percent of the maximum negative moment and the positive and negative moment at midspan is less than 25 percent of the maximum negative moment; therefore, the design for negative moment controls the amount of reinforcement required at all sections per ACI 318 Sec. 21.3.2.2.

For a factored negative moment of 12,600 in.-kips at Support B, try seven #11 bars, and assuming #3 hoops:

$A_s = 7 \times 1.54 = 10.92$ in.^{2}
$d = 32 - 1.5 - 3/8 - 1.41/2 = 29.4$ in.
$\rho = 10.92/(29.4 \times 22.5) = 0.0165 < 0.025$, O.K.
$b = 22.5$ in.
Depth of compression block, $a = [10.92\ (60)]/[0.85\ (4)\ 22.5] = 8.56$ in.
Design strength, $\phi M_n = [0.8\ (10.92)\ 60](29.4 - 8.56/2) = 13{,}167$ in.-kips > 12,600 in.-kips OK

For positive moment at the support, try five #9 bars, which supplies about half the negative moment reinforcement:

$A_s = 5\ (1.0) = 5.00$ in.^{2}
$d = 32 - 1.5 - 3/8 - 1.128/2 = 29.6$ in.
$\rho = 5.00/(29.6 \times 22.5) = 0.0075 > 0.033$, O.K.
$b = 86.5$ in. (assuming stress block in flange)
$a = [5.00\ (60)]/(0.85\ (4)\ 86.5] = 1.02$ in.
$\phi M_n = [0.8\ (5.00)\ 60]\ (29.6 - 1.02/2) = 6{,}982$ in.-kips.

This moment is larger than the design moment and, as required by ACI 318 Sec. 21.3.2.2, is greater than 50 percent of the negative moment capacity at the face of the support.

For positive moment at midspan the same five #9 bars used for positive moment at the support will be tried:

$A_s = 5\ (1.0) = 5.00$ in.^{2}
$d = 20 - 1.5 - 3/8 - 1.128/2 = 17.6$ in.
$\rho = 5.00/(17.6 \times 22.5) = 0.0126$
$b = 86.5$ in.
$a = [5.00\ (60)]/[0.85\ (4)\ 86.5] = 1.02$ in.
$\phi M_n = [0.8\ (5.00)\ 60]\ (17.6 - 1.02/2) = 4{,}102$ in.-kips > 3,282 in.-kips. OK

The five #9 bottom bars are adequate for strength and satisfy ACI 318 Sec. 21.3.2.2, which requires that the positive moment capacity be not less than 25 percent of the negative moment capacity at the face of the support.

For negative moment in the 20-ft span between the haunches, four #11 bars ($\rho = 0.016$) could be used at the top. These bars provide a strength greater than 25 percent of the negative moment capacity at the support. Using four bars across the top also eliminates the possibility that a negative moment hinge will form at the end of the haunch (8 ft-9 in. from the face of the support) when the $0.68D - E$ load combination is applied. These four top bars are part of the negative moment reinforcement already sized for negative moment at the support. The other three bars extending from the support are not needed for negative moment in the constant depth region and would be cut off approximately 6 ft beyond the haunch; however, this detail results in a possible bar cutoff in a plastic hinge region (see below) that is not desirable. Another alternative would be to extend all seven #11 bars across the top and thereby avoid the bar cutoff in a possible plastic hinge region; however, seven #11 bars in 20-in. deep portion of the girder provide $\rho = 0.028$, which is a violation of ACI 21.3.2.1 ($\rho_{max} = 0.025$). The violation is minor and will be accepted in lieu of cutting off the bars in a potential plastic hinge region. Note that these bars provide a negative design moment capacity of 6,824 in.-kips in the constant depth region of the girder.

The layout of longitudinal reinforcement used for the haunched girder is shown in Figure 6-27c, and the flexural strength envelope provided by the reinforcement is shown in Figure 6-27b. As noted in Table 6-13, the hooked #11 bars can be developed in the confined core of the columns. Finally, where seven #11 top bars are used, the spacing between bars is approximately 1.4 in., which is greater than the diameter of a #11 bar and is therefore acceptable. This spacing should accommodate the vertical column reinforcement.

Under combined gravity and earthquake load, a negative moment plastic hinge will form at the support and, based on the moment envelopes from the loading (Figure 6-27b), the corresponding positive moment hinge will form in the constant depth portion of the girder. As discussed in the following sections, the exact location of plastic hinges must be determined in order to design the transverse reinforcement.

6.4.5.4.2 Design and Detailing of Transverse Reinforcement

The design for shear of the haunched girder is complicated by its variable depth; therefore, a tabular approach is taken for the calculations. Before the table may be set up, however, the maximum probable strength must be determined for negative moment at the support and for positive moment in the constant depth region,

For negative moment at the face of the support and using seven #11 bars:

A_s = 7 (1.56) = 10.92 in.^{2}
d = 32 - 1.5 - 3/8 - 1.41/2 = 29.4 in.
b = 22.5 in.
a = [10.92 (1.25) 60]/[0.85 (4) 22.5] = 10.71 in.
M_{pr} = 1.0(10.92)(1.25)(60)(29.4 - 10.71/2) = 19,693 in.-kips.

For positive moment in the constant depth region and using five #9 bars:

A_s = 5 (1.0) = 5.00 in.^{2}
d = 20 - 1.5 - 3/8 - 1.128/2 = 17.6 in.
b = 86.5 in.
a = [5.00 (1.25) 60]/[0.85 (4) 86.5] = 1.28 in.
M_{pr} = [1.0 (5.00) 1.25 (60)] (17.6 - 1.28/2) = 6,360 in.-kips

Before the earthquake shear may be determined, the location of the positive moment hinge that will form in the constant depth portion of the girder must be identified. To do so, consider the free-body diagram of Figure 6-28a. Summing moments (clockwise positive) about point B gives:

$$M_{pr}^{+} + M_{pr}^{-} + Rx - \frac{wx^2}{2} = 0$$

At the positive moment hinge the shear must be zero, thus $R - wx = 0$

By combining the above equations:

$$x = \sqrt{\frac{2(M_{pr}^{+} + M_{pr}^{-})}{w}}$$

Using the above equation with M_{pr} as computed and w = 1.42(3.38) + 0.5(0.90) = 5.25 k/ft = 0.437 k/in., x = 345 in., which is located exactly at the point where the right haunch begins.[4]

The reaction is computed as R = 345 (0.437) = 150.8 kips.

The earthquake shear is computed as $V_E = R = wL/2$ = 150.8-(0.437)(450)/2 = 52.5 kips

This earthquake shear is *smaller* than would have been determined if the positive moment hinge had formed at the face of support.

The earthquake shear is constant along the span but changes sign with the direction of the earthquake. In Figure 6-28a, this shear is shown for the equivalent lateral seismic forces acting to the west. The factored gravity load shear ($1.42V_D + 0.5V_L$) varies along the length of the span as shown in Figure 6-28b. At Support A, the earthquake shear and factored gravity shear are additive, producing a design ultimate shear of 150.8 kips. At midspan, the shear is equal to the earthquake shear acting alone and, at Support C, the ultimate design shear is -45.8 kips. Earthquake, gravity, and combined shears are shown in Figures 6-28a through 6-28c and are tabulated for the first half of the span in Table 6-16. For earthquake forces acting to the east, the design shears are of the opposite sign of those shown in Figure 6-28.

According to ACI 318 Sec. 21.3.4.2, the contribution of concrete to member shear strength must be taken as zero when V_E/V_U is greater than 0.5 and $P_u/A_g f'_c$ is less than 0.05. As shown in Table 6-16, the V_E/V_U ratio is less than 0.5 within the first three-fourths of the haunch length but is greater than 0.50 beyond this point. In this example, it is assumed that if V_E/V_U is less than 0.5 at the support, the concrete strength can be used along the entire length of the member.

The concrete contribution to the design shear strength is computed as:

$$\phi V_c = \phi(0.85)2\sqrt{f_c'}\,b_w d$$

where the ASCE 7 compatible ϕ = 0.75 for shear, and the 0.85 term is the shear strength reduction factor for sand-LW concrete. [Note that this is the basic strength reduction factor for shear per ACI 318-02 Sec 9.3. See Sec 6.4.2 for discussion.] The remaining shear, $\phi V_s = V_u - \phi V_c$, must be resisted by closed hoops within a distance $2d$ from the face of the support and by stirrups with the larger of $6d_h$ or 3.0 in. hook extensions elsewhere. The $6d_h$ or 3.0 in. "seismic hook" extension is required by ACI 318 Sec. 21.3.3.3.

[4]The equation for the location of the plastic hinge is only applicable if the hinge forms in the constant depth region of the girder. If the computed distance x is greater than 28 ft - 9 in. (345 in.), the result is erroneous and a trial and error approach is required to find the actual hinge location.

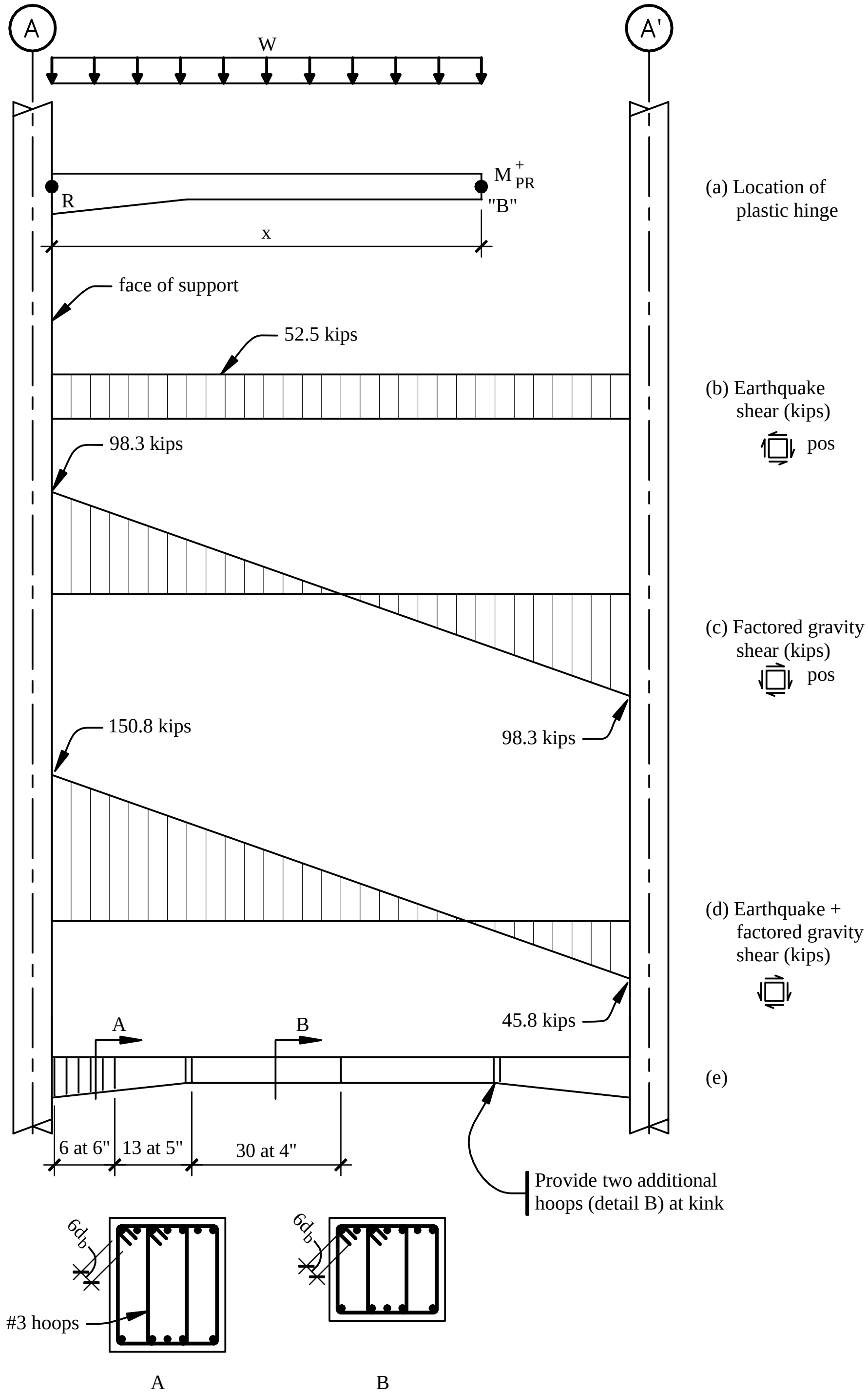


Figure 6-28 Computing shear in haunched girder (1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

Table 6-16 Design of Shear Reinforcement for Haunched Girder

Item	Distance from Center of Support (in.)							Units
	15	42.25	67.5	93.75	120	180	240	
V_e	52.5	52.5	52.5	52.5	52.5	52.5	52.5	kips
$1.42V_D + 0.5V_L$	98.3	86.4	75.4	63.9	52.4	26.2	0.0	kips
V_u	150.8	139.2	127.9	116.6	104.9	78.7	52.5	kips
V_E/V_U	0.35	0.38	0.41	0.45	0.50	0.67	1.00	
d	29.4	26.5	23.5	20.5	17.6	17.6	17.6	in.
ϕV_C	53.3	48.1	42.6	37.2	0.0	0.0	0.0	kips
ϕV_S	97.5	91.2	85.3	79.4	104.9	78.7	52.5	kips
s	5.97	5.78	5.46	5.12	3.32	4.43	6.64	in.
$d/4$	7.35	6.63	5.88	5.13	4.40	4.40	4.40	in.
Spacing	#3 at 6	#3 at 5	#3 at 5	#3 at 5	#3 at 4	#3 at 4	#3 at 4	in.

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

In Table 6-16, spacings are computed for four #3 vertical leg hoops or stirrups. As an example, consider four #3 vertical legs at the section at the face of the support:

$$\phi V_c = \phi(0.85)2\sqrt{f_c'}\,db = 0.75(0.85)2(4000)^{0.5}29.4(22.5) = 53{,}300 \text{ lb} = 53.3 \text{ kips}$$
$$\phi V_s = V_u - \phi V_c = 150.8 - 53.3 = 97.5 \text{ kips}$$
$$\phi V_s = \phi A_v f_y d/s = 97.5 \text{ kips}$$
$$s = [0.75(4)0.11(60)29.4]/97.5 = 5.97 \text{ in.}$$

The maximum spacing allowed by ACI 318 is shown in Table 6-16. These spacings govern only in the center portion of the beam. In the last line of the table, the hoop and stirrup spacing as actually used is shown. This spacing, together with hoop and stirrup details, is illustrated in Figure 6-28d. The double U-shaped stirrups (and cap ties) in the central portion of the beam work well with the #11 top bars and with the #9 bottom bars.

6.4.5.4.3 Design of Beam-Column Joint

The design of the beam-column joint at Support A of the haunched girder is controlled by seismic forces acting to the west, which produces negative moment at Support A. ACI 318 Sec. 21.5 provides requirements for the proportioning and detailing of the joint.

A plastic mechanism of the beam is shown in Figure 6-29a. Plastic hinges have formed at the support and at the location of the far haunch transition. With a total shear at the face of the support of 150.8 kips, the moment at the centerline of the column may be estimated as

$$M_{CL} = M_{pr} + 15(150.6) = 19{,}693 + 15(150.6) = 21{,}955 \text{ in.-kips.}$$

The total shear in the columns above and below the joint is estimated as 21,955/(150) = 146.3 kips.

The stresses in the joint are computed from equilibrium considering the reinforcement in the girder to be stressed at $1.25f_y$. A detail of the joint is shown in Figure 6-30. Compute the joint shear V_j:

Force in the top reinforcement = $1.25A_sf_y$ = 1.25(7)1.56(60) = 819 kips
Joint shear = V_j = 819.0 - 146.3 = 672.7 kips

The joint shear stress $v_j = V_j/d_c^2$ = 672.7/[30 (30)] = 0.819 ksi

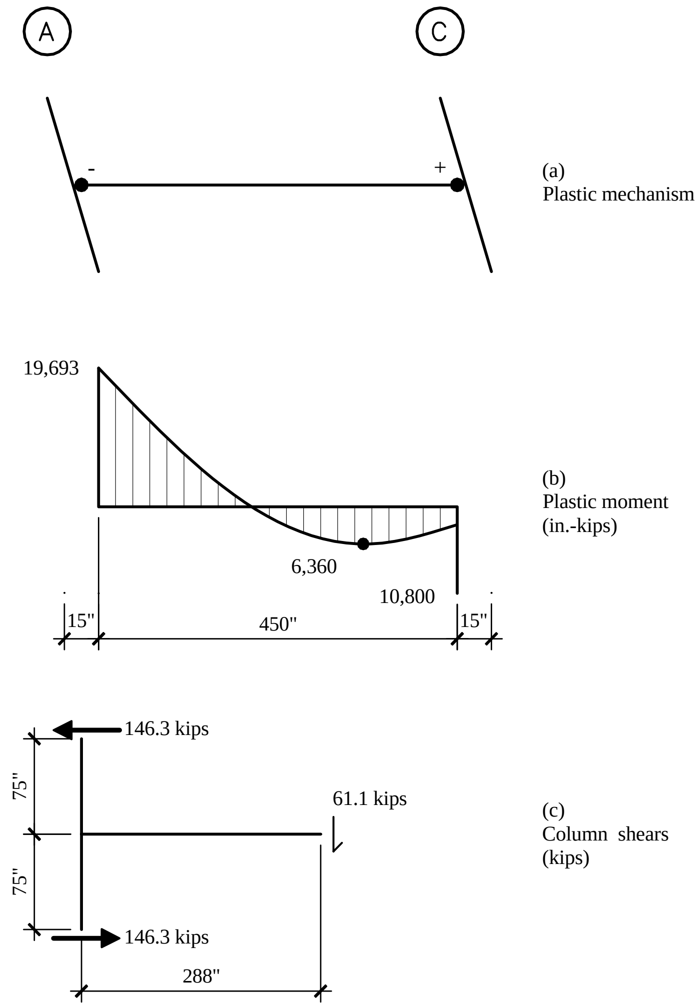


Figure 6-29 Computation of column shears for use in joint design (1.0 in = 25.4 mm, 1.0 kip = 4.45 kN).

In the case being considered, all girders framing into the joint have a width equal to 0.75 times the column dimension so confinement is provided on three faces of the joint. According to ACI 318 Sec. 21.5.3, the allowable joint shear stress = $0.75\phi(15)2\sqrt{f_c'}$. The 0.75 term is the strength reduction factor for LW concrete. Compute the allowable joint shear stress:

$v_{j,allowable}$ = 0.75(0.80)15(4,000)$^{0.5}$
= 569 psi = 0.569 ksi

This allowable stress is significantly less than the applied joint shear stress. There are several ways to remedy the situation:

1. Increase the column size to approximately 35 × 35 (not recommended)

2. Increase the depth of the haunch so that the area of reinforcement is reduced to seven #10 bars. This will reduce the joint shear stress to a value very close to the allowable stress.

2. Use 5000 psi NW concrete for the column. This eliminates the 0.75 reduction factor on allowable joint stress, and raises the allowable stress to 848 psi.

For the remainder of this example, it is assumed that the lower story columns will be constructed from 5000 psi NW concrete.

Because this joint is confined on three faces, the reinforcement within the joint must consist of the same amount and spacing of transverse reinforcement in the critical region of the column below the joint. This reinforcement is detailed in the following section.

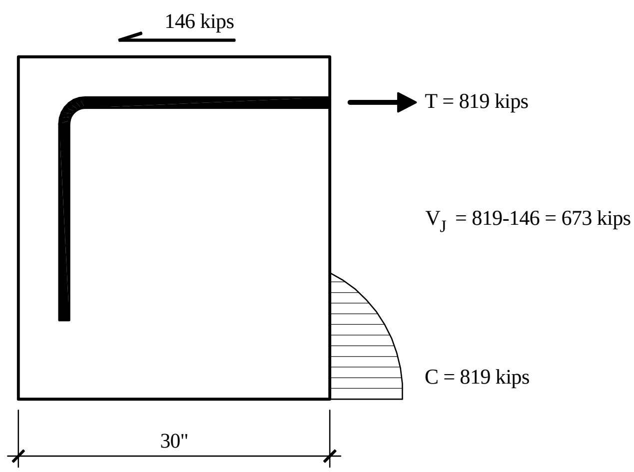


Figure 6-30 Computing joint shear force (1.0 kip = 4.45kN).

6.4.5.5 Design and Detailing of Typical Interior Column of Frame 3

The column supporting the west end of the haunched girder between Gridlines A and B is shown in Figure 6-31. This column supports a total unfactored dead load of 804 kips and a total unfactored live load of 78 kips. From the ETABS analysis, the axial force on the column from seismic forces is ±129 kips. The design axial force and bending moment in the column are based on one or more of the load combinations presented below.

Earthquake forces acting to the west are:

$$P_u = 1.42(804) + 0.5(78) + 1.0(129)$$
$$= 1310 \text{ kips (compression)}$$

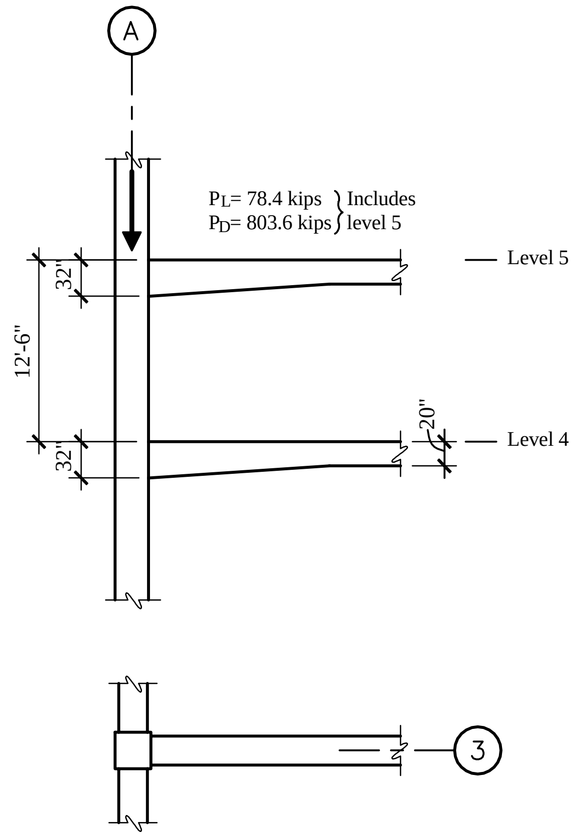


Figure 6-31 Column loading (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

This axial force is greater than $0.1f_c'A_g$ = 360 kips; therefore, according to ACI 318 Sec. 21.4.2.1, the column flexural strength must be at least 6/5 of the nominal strength (using ϕ = 1.0 and 1.0 f_y) of the beam framing into the column. The nominal beam moment capacity at the face of the column is 16,458 in.-kips. The column must be designed for six-fifths of this moment, or 19,750 in-kips. Assuming a midheight inflection point for the column above and below the beam, the column moment at the centerline of the beam is 19,750/2 = 9,875 in.-kips, and the column moment corrected to the face of the beam is 7,768 in.-kips.

Earthquake forces acting to the east are:

P_u = 0.68(804) - 1.0(129) = 424 kips (compression)

This axial force is greater than $0.1f_c'A_g$ = 360 kips. For this loading, the end of the beam supported by the column is under positive moment, with the nominal beam moment at the face of the column being 8,715 in.-kips. Because $P_u > 0.1f_c'A_g$, the column must be designed for 6/5 of this moment, or 10,458 in.-kips. Assuming midheight inflection points in the column, the column moment at the centerline and the face of the beam is 5,229 and 4,113 in.-kips, respectively.

Axial force for gravity alone is:

P_u = 1.6(804) + 1.2(78) = 1,380 kips (compression)

This is approximately the same axial force as designed for earthquake forces to the west, but as can be observed from Figure 6-25, the design moment is significantly less. Hence, this loading will not control.

6.4.5.5.1 Design of Longitudinal Reinforcement

Figure 6-32 shows an axial force-bending moment interaction diagram for a 30 in. by 30 in. column with 12 bars ranging in size from #8 to #10. A horizontal line is drawn at each of the axial load levels computed above, and the required flexural capacity is shown by a solid dot on the appropriate line. The column with twelve #8 bars provides more than enough strength for all loading combinations.

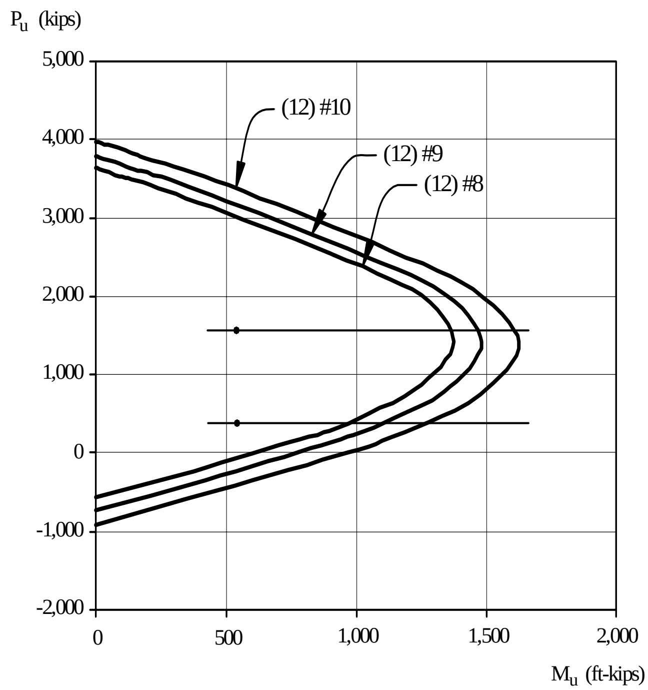


Figure 6-32 Interaction diagram and column design forces (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

6.4.5.5.2 Design of Transverse Reinforcement

In Sec. 6.4.5.3, an interior column supporting Level 5 of Frame 1 was designed. This column has a shear strength of 198.2 kips, which is significantly greater than the imposed seismic plus gravity shear of 146.3 kips. For details on the computation of the required transverse reinforcement for this column, see the "Transverse Reinforcement" and "Transverse Reinforcement Required for Shear" subsections in Sec. 6.4.5.3. A detail of the reinforcement of the column supporting Level 5 of Frame 3 is shown in Figure 6-33. The section of the column through the beams shows that the reinforcement in the beam-column joint region is relatively uncongested.

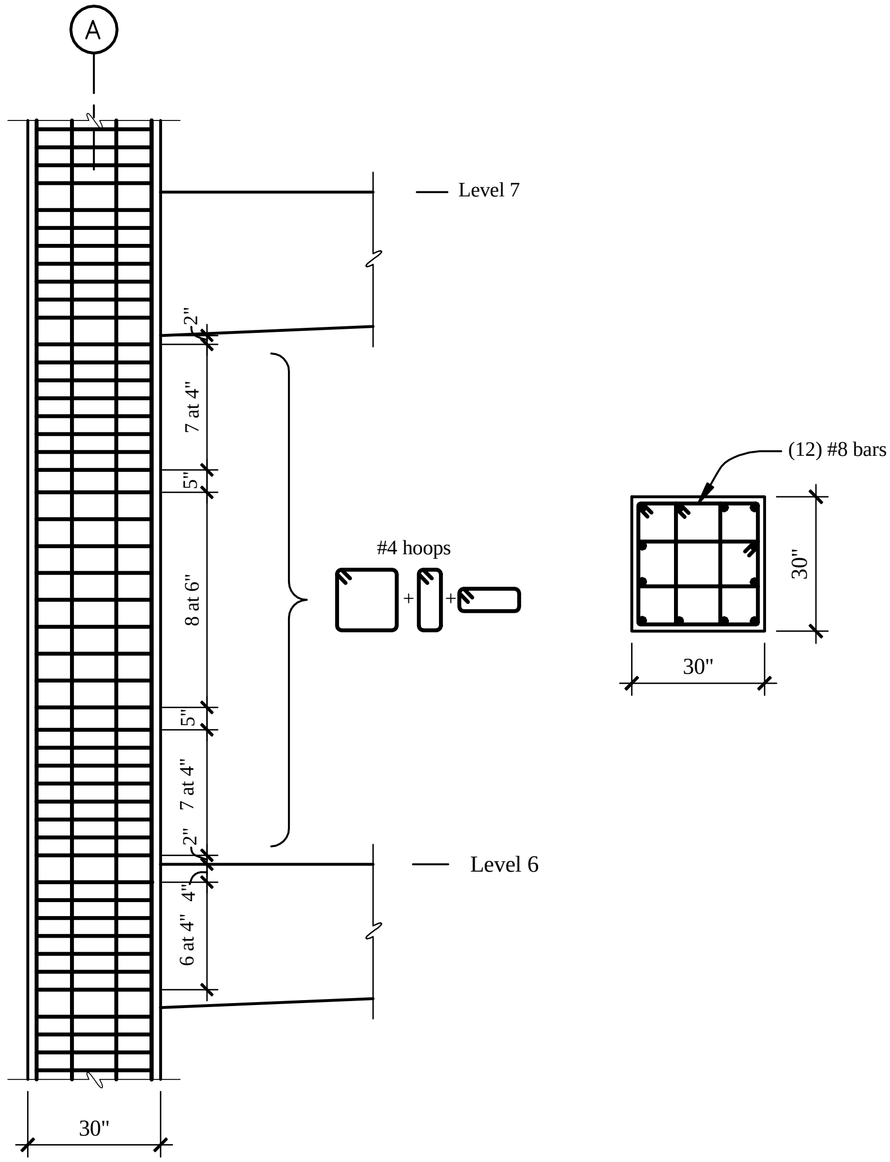


Figure 6-33 Column detail (1.0 in = 25.4 mm).

6.4.5.6 Design of Structural Wall of Frame 3

The factored forces acting on the structural wall of Frame 3 are summarized in Table 6-17. The axial compressive forces are based on a tributary area of 1,800 square ft for the entire wall, an unfactored dead load of 160 psf, and an unfactored (reduced) live load of 20 psf. For the purposes of this example it is assumed that these loads act at each level, including the roof. The total axial force for a typical floor is:

P_u = 1.42D + 0.5L = 1,800((1.42×0.16) + 0.50x0.02)) = 427 kips for maximum compression
P_u = 0.68D = 1,800(0.68×0.16) = 196 kips for minimum compression

The bending moments come from the ETABS analysis. Note the reversal in the moment sign due to the effects of frame-wall interaction. Each moment contains two parts: the moment in the shear panel and the couple resulting from axial forces in the boundary elements. For example, at the base of Level 2:

ETABS panel moment =162,283 in.-kips
ETABS column force = 461.5 kips
Total moment, M_u = 162,283 + 240(461.5) = 273,043 in.-kips

The shears in Table 6-17 also consist of two parts, the shear in the panel and the shear in the column. Using Level 2 as an example:

ETABS panel shear = 527 kips
ETABS column shear = 5.90 kips
Total shear, V_u = 527 + 2(5.90) = 539 kips

As with the moment, note the reversal in wall shear, not only at the top of the wall but also at Level 1 where the first floor slab acts as a support. If there is some in-plane flexibility in the first floor slab, or if some crushing were to occur adjacent to the wall, the shear reversal would be less significant, or might even disappear. For this reason, the shear force of 539 kips at Level 2 will be used for the design of Level 1 as well.

Recall from Sec. 6.2.2 that the structural wall boundary elements are 30 in. by 30 in. in size. The basic philosophy of this design will be to use these elements as "special" boundary elements where a close spacing of transverse reinforcement is used to provide extra confinement. This avoids the need for confining reinforcement in the wall panel. Note, however, that there is no code restriction on extending the special boundary elements into the panel of the wall.

It should also be noted that preliminary calculations (not shown) indicate that a 12-in. thickness of the wall panel is adequate for this structure. This is in lieu of the 18-in. thickness assumed when computing structural mass.

Table 6-17 Design Forces for Structural Wall

Supporting Level	Axial Compressive Force P_u (kips)		Moment M_u (in.-kips)	Shear V_u (kips)
	1.42D + 0.5L	0.68D		
R	427	196	-30,054	-145
12	854	392	-39,725	-4
11	1,281	588	-49,954	62
10	1,708	783	-51,838	118
9	2,135	979	-45,929	163
8	2,562	1,175	-33,817	203
7	2,989	1,371	17,847	240
6	3,416	1,567	45,444	274
5	3,843	1,763	78,419	308
4	4,270	1,958	117,975	348
3	4,697	2,154	165,073	390
2	5,124	2,350	273,043	539
1	5,550	2,546	268,187	-376 (use 539)

1.0 kip = 4.45 kN, 1.0 in.-kip = 0.113 kN-m.

6.4.5.6.1 Design of Panel Shear Reinforcement

First determine the required shear reinforcement in the panel and then design the wall for combined bending and axial force. The nominal shear strength of the wall is given by ACI 318 Eq. 21-7:

$$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_n f_y)$$

where α_c = 2.0 because h_w/l_w = 155.5/22.5 = 6.91 > 2.0. Note that the length of the wall was taken as the length between boundary element centerlines (20 ft) plus one-half the boundary element length (2.5 ft) at each end of the wall.

Using f_c' = 4000 psi, f_y = 40 ksi, A_{cv} = (270)(12) = 3240 in.^{2}, and taking ϕ for shear = 0.55, the ratio of horizontal reinforcement is computed:

$$V_u = \phi V_n$$

$$\rho_n = \frac{\left(\frac{539.000}{0.55}\right) - (0.85\times 2\sqrt{4,000})3,240}{3,240(40,000)} = 0.0049$$

Note that the factor of 0.85 on concrete strength accounts for the use of LW concrete. Reinforcement ratios for the other stories are given in Table 6-18. This table gives requirements using f_c' = 4,000 psi, as well as 6,000 psi NW concrete. As shown later, the higher strength NW concrete is required to manage the size of the boundary elements of the wall. Also shown in the table is the required spacing of horizontal reinforcement assuming that two curtains of #4 bars will be used. If the required steel ratio is less than 0.0025, a ratio of 0.0025 is used to determine bar spacing.

Table 6-18 Design of Structural Wall for Shear

Level	f_c' = 4,000 psi (lightweight)		f_c' = 6,000 psi (normal weight)	
	Reinforcement ratio	Spacing[1] (in.)	Reinforcement ratio	Spacing * (in.)
R	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
12	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
11	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
10	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
9	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
8	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
7	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
6	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
5	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
4	0.00250	13.33 (12.0)	0.00250	13.33 (12.0)
3	0.00278	12.00 (6.0)	0.00250	13.33 (9.0)
2	0.00487	6.84 (6.0)	0.00369	9.03 (9.0)
1	0.00487	6.84 (6.0)	0.00369	9.03 (9.0)

* Values in parentheses are actual spacing used.
1.0 in. = 25.4 mm.

For LW concrete, the required spacing is 6.84 in. at Levels 1 and 2. Minimum reinforcement requirements control all other levels. For the final design, it is recommended to use a 6-in. spacing at

Levels 1, 2, and 3 and a 12-in. spacing at all levels above. The 6-in. spacing is extended one level higher that required because it is anticipated that an axial-flexural plastic hinge could propagate this far.

For the NW concrete, the required spacing is 9.03 in. at Levels 1 and 2 and minimum reinforcement requirements control elsewhere. For the final design, a 9-in. spacing would be used at Levels 1, 2, and 3 with a 12-in. spacing at the remaining levels.

ACI 318 Sec. 21.6.4.3 [21.7.4.3] requires the vertical steel ratio to be greater than or equal to the horizontal steel ratio if $h_w l/_l w$ is less than 2.0. As this is not the case for this wall, the minimum vertical reinforcement ratio of 0.0025 is appropriate. Vertical steel consisting of two curtains of #4 bars at 12 in. on center provides a reinforcement ratio of 0.0028, which ill be used at all levels.

6.4.5.6.2 Design for Flexure and Axial Force

The primary consideration in the axial-flexural design of the wall is determining whether or not special boundary elements are required. ACI 318 provides two methods for this. The first approach, specified in ACI 318 Sec. 21.6.6.2 [21.7.6.2], uses a displacement based procedure. The second approach, described in ACI 318 Sec. 21.6.6.3 [21.7.6.3], is somewhat easier to implement but, due to its empirical nature, is generally more conservative. In the following presentation, only the displacement based method will be used for the design of the wall.

Using the displacement based approach, boundary elements are required if the length of the compression block, c, satisfies ACI 318 Eq. 21-8:

$$c \geq \frac{l_w}{600(\delta_u / h_w)}$$

where δ_u is the total elastic plus inelastic deflection at the top of the wall. From Table 6-9b, the total elastic roof displacement is 4.36 in., and the inelastic drift is C_d times the elastic drift, or 6.5(4.36) = 28.4 in. or 2.37 feet. Recall that this drift is based on cracked section properties assuming $I_{cracked} = 0.5\ I_{gross}$ and assuming that flexure dominates. Using this value together with l_w = 22.5 ft, and h_w = 155.5 ft:

$$\frac{l_w}{600(\delta_u / h_w)} = \frac{22.5}{600(2.37/155.5)} = 2.46 \text{ ft} = 29.52 \text{ in.}$$

To determine if c is greater than this value, a strain compatibility analysis must be performed for the wall. In this analysis, it is assumed that the concrete reaches a maximum compressive strain of 0.003 and the wall reinforcement is elastic-perfectly plastic and yields at the nominal value. A rectangular stress block was used for concrete in compression, and concrete in tension was neglected. A straight line strain distribution was assumed (as allowed by ACI 318 Sec. 21.6.5.1 [21.7.5.1]). Using this straight line distribution, the extreme fiber compressive strain was held constant at 0.003, and the distance c was varied from 100,000 in. (pure compression) to 1 in. (virtually pure tension). For each value of c, a total cross sectional nominal axial force (P_n) and nominal bending moment (M_n) were computed. Using these values, a plot of the axial force (P_n) versus neutral axis location (c) was produced. A design value axial force-bending moment interaction diagram was also produced.

The analysis was performed using an Excel spreadsheet. The concrete was divided into 270 layers, each with a thickness of 1 in. The exact location of the reinforcement was used. When the reinforcement was in compression, an adjustment was made to account for reinforcement and concrete sharing the same physical volume.

Two different sections were analyzed: one with f_c' = 4,000 psi (LW concrete) and the other with f_c' = 6,000 psi (NW concrete). In each case, the boundary elements were assumed to be 30 in. by 30 in. and the panel was assumed to be 12 in. thick. Each analysis also assumed that the reinforcement in the boundary element consisted of twelve #9 bars, producing a reinforcement ratio in the boundary element of 1.33 percent. Panel reinforcement consisted of two curtains of #4 bars spaced at approximately 12 in. on center. For this wall the main boundary reinforcement has a yield strength of 60 ksi, and the vertical panel steel yields at 40 ksi.

The results of the analysis are shown in Figures 6-34 and 6-35. The first of these figures is the nominal interaction diagram multiplied by ϕ = 0.65 for tied sections. Also plotted in the figure are the factored *P-M* combinations from Table 6-17. The section is clearly adequate for both 4,000 psi and 6,000 psi concrete because the interaction curve fully envelopes the design values.

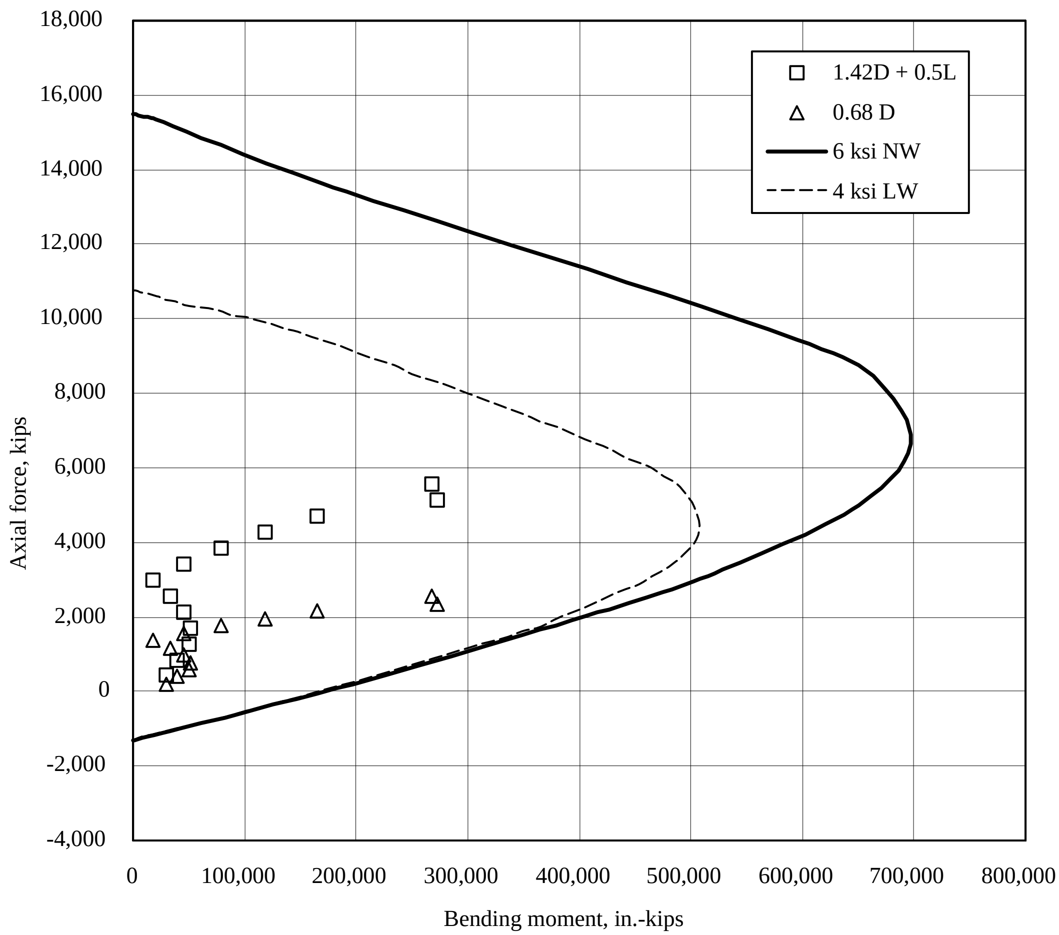


Figure 6-34 Interaction diagram for structural wall (1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

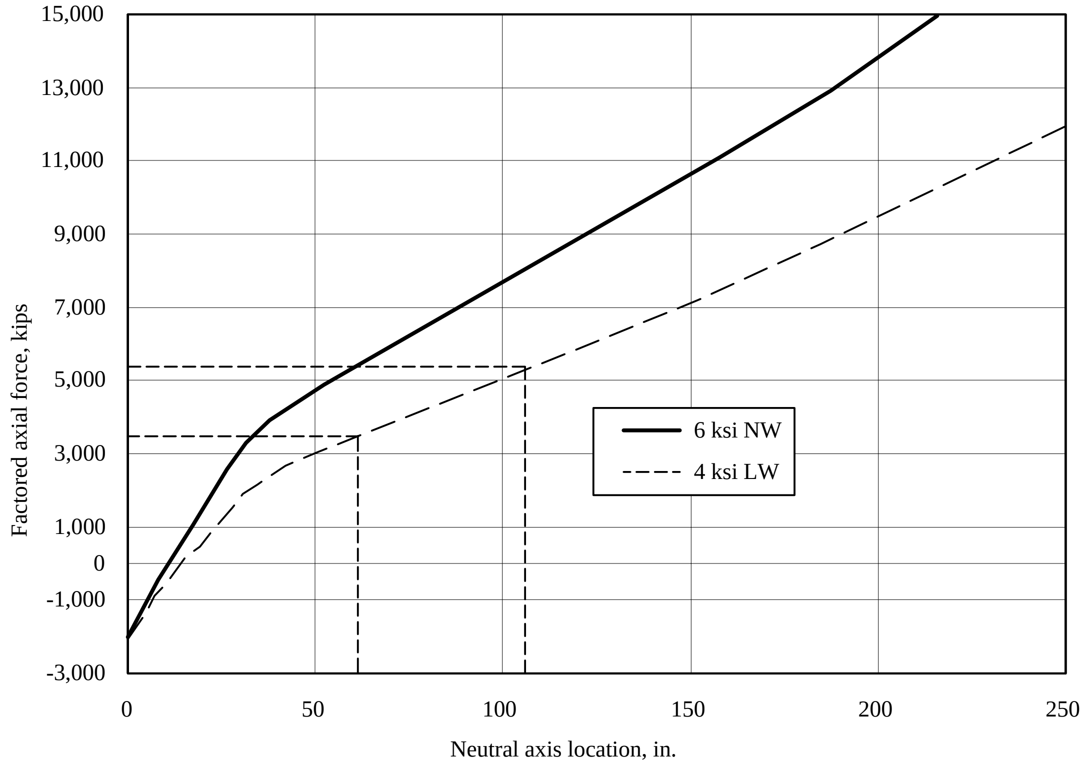


Figure 6-35 Variation of neutral axis depth with compressive force (1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

Figure 6-35 shows the variation in neutral axis depth with axial force. For a factored axial force of 5,550 kips, the distance c is approximately 58 in. for the 6,000 psi NW concrete and c is in excess of 110 in. for the 4,000 psi LW concrete. As both are greater than 29.52 in., special boundary elements are clearly required for the wall.

According to ACI 318 Sec. 21.6.6.4 [21.7.6.4], the special boundary elements must have a plan length of c - 0.1l_w, or 0.5c, whichever is greater. For the 4,000 psi concrete, the first of these values is 110 - 0.1(270) = 83 in., and the second is 0.5(110) = 55 in. Both of these are significantly greater than the 30 in. assumed in the analysis. Hence, the 30-in. boundary element is not adequate for the lower levels of the wall if f_c' = 4,000 psi. For the 6,000 psi concrete, the required length of the boundary element is 58-0.1(270) = 31 in., or 0.5(58) = 29 in. The required value of 31 in. is only marginally greater than the 30 in. provided and will be deemed acceptable for the purpose of this example.

The vertical extent of the special boundary elements must not be less than the larger of l_w or $M_u/4V_u$. The wall length l_w = 22.5 ft and, of the wall at Level 1, $M_u/4V_u$ = 273,043/4(539) =126.6 in., or 10.6 ft. 22.5 ft controls and will be taken as the required length of the boundary element *above the first floor*. The special boundary elements will begin at the basement level, and continue up for the portion of the wall supporting Levels 2 and 3. Above that level, boundary elements will still be present, but they will not be reinforced as special boundary elements.

Another consideration for the boundary elements is at what elevation the concrete may change from 6,000 psi NW to 4,000 LW concrete. Using the requirement that boundary elements have a maximum plan dimension of 30 in., the neutral axis depth (c) must not exceed approximately 57 in. As may be seen from Figure 6-35, this will occur when the factored axial force in the wall falls below 3,000 kips. From Table 6-17, this will occur between Levels 6 and 7. Hence, 6,000 psi concrete will be continued up through Level 7. Above Level 7, 4,000 psi LW concrete may be used.

Where special boundary elements are required, transverse reinforcement must conform to ACI 318 Sec. 21.6.6.4(c) [21.7.6.4(c)], which refers to Sec. 21.4.4.1 through 21.4.4.3. If rectangular hoops are used, the transverse reinforcement must satisfy ACI 318 Eq. 21-4:

$$A_{sh} = 0.09 s h_c \frac{f_c'}{f_{yh}}$$

If #5 hoops are used in association with two crossties in each direction, A_{sh} = 4(0.31) = 1.24 in.^{2}, and h_c = 30 - 2(1.5) - 0.525 = 26.37 in. With f_c' = 6 ksi and f_{yh} = 60 ksi:

$$s = \frac{1.24}{0.09(26.37)\frac{6}{60}} = 5.22$$

If 4,000 psi concrete is used, the required spacing increases to 7.83 in.

Maximum spacing is the lesser of $h/4$, $6d_b$, or s_x where $s_x = 4 + (14-h_x)/3$. With h_x = 8.83 in., the third of these spacings controls at 5.72 in. The 5.22-in. spacing required by ACI 318 Eq. 21-4 is less than this, so a spacing of 5 in. on center will be used wherever the special boundary elements are required.

Details of the panel and boundary element reinforcement are shown in Figures 6-36 and 6-37, respectively. The vertical reinforcement in the boundary elements will be spliced as required using Type 2 mechanical splices at all locations. According to Table 6-13 (prepared for 4,000 psi LW concrete), there should be no difficulty in developing the horizontal panel steel into the 30-in.-by-30-in. boundary elements.

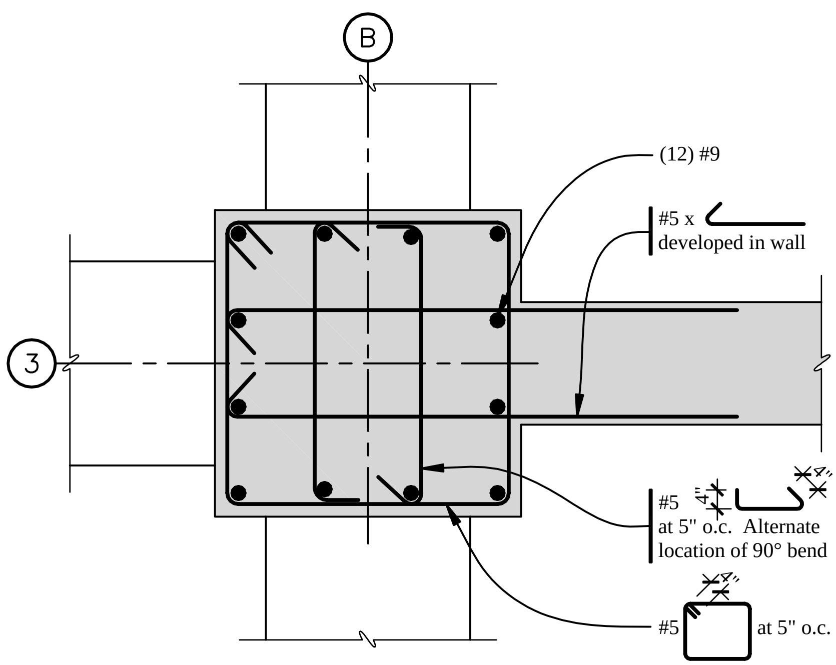


Figure 6-36 Details of structural wall boundary element (1.0 in = 25.4 mm).

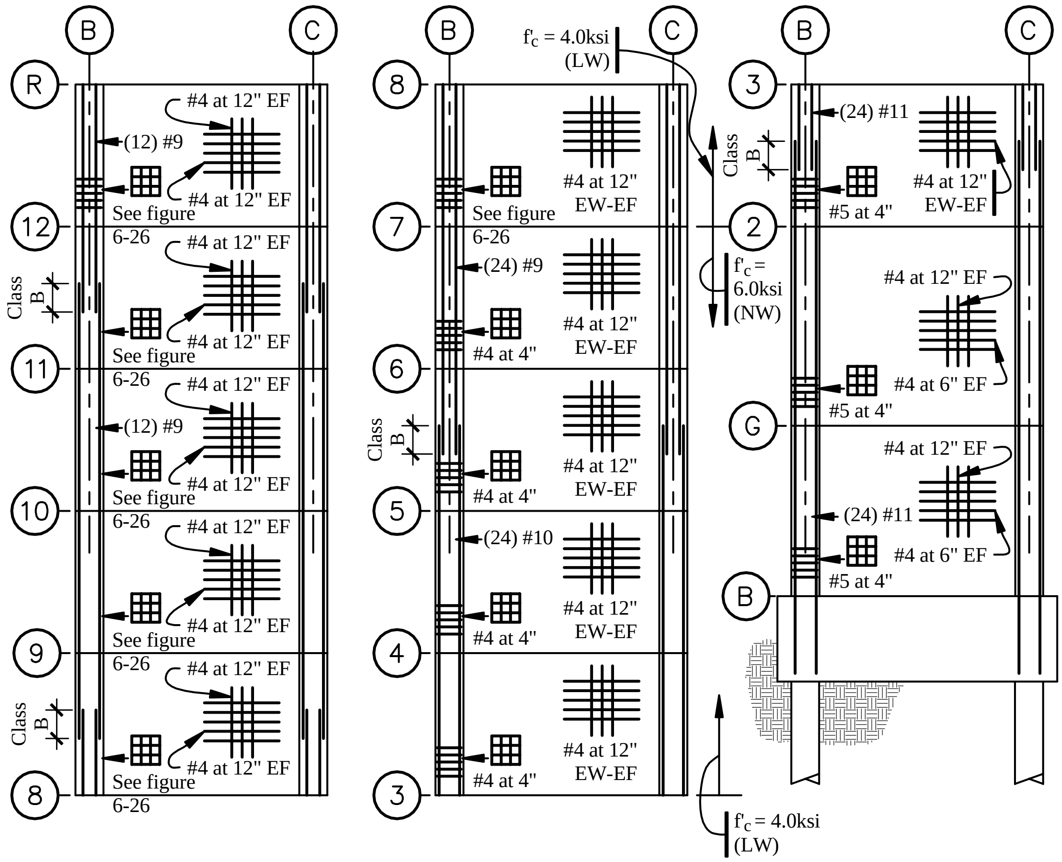


Figure 6-37 Overall details of structural wall (1.0 in = 25.4 mm).

ACI 318 Sec. 21.6.6.4(d) [21.7.6.4(d)] also requires that the boundary element transverse reinforcement be extended into the foundation tie beam a distance equal to the tension development length of the #9 bars used as longitudinal reinforcement in the boundary elements. Assuming the tie beam consists of 6,000 psi NW concrete, the development length for the #9 bar is 2.5 times the value given by ACI 318 Eq. 21-6:

$$l_d = 2.5\left[\frac{f_y d_b}{65\sqrt{f_c'}}\right] = 2.5\frac{60,000(1.128)}{65\sqrt{6,000}} = 33.6 \text{ in.}$$

Hence, the transverse boundary element reinforcement consisting of #5 hoops with two crossties in each direction, spaced at 5 in. on center, will extend approximately 3 ft into the foundation tie beam.

6.5 STRUCTURAL DESIGN OF THE HONOLULU BUILDING

The structure illustrated in Figure 6-1 and 6-2 is now designed and detailed for the Honolulu building. Because of the relatively moderate level of seismicity, the lateral load resisting system will consist of a series of intermediate moment-resisting frames in both the E-W and N-S directions. This is permitted for Seismic Design Category C buildings under *Provisions* Sec. 9.6 [9.4]. Design guidelines for the reinforced concrete framing members are provided in ACI 318 Sec. 21.10 [21.12].

Preliminary design for the Honolulu building indicated that the size of the perimeter frame girders could be reduced to 30 in. deep by 20 in. wide (the Berkeley building has girders that are 32 in. deep by 22.5 in. wide) and that the columns could be decreased to 28 in. square (the Berkeley building uses 30-in.-by-30-in. columns). The haunched girders along Frames 2 through 7 have a maximum depth of 30 in. and a width of 20 in. in the Honolulu building (the Berkeley building had haunches with a maximum depth of 32 in. and a width of 22.5 in.). The Frame 2 through Frame 7 girders in Bays B-C have a constant depth of 30 in. Using these reduced properties, the computed drifts will be increased over those shown in Figure 6-6, but will clearly not exceed the drift limits.

6.5.1 Material Properties

ACI 318 has no specific limitations for materials used in structures designed for moderate seismic risk. For the Honolulu building, 4,000 psi sand-LW concrete is used with ASTM A615 Grade 60 rebar for longitudinal reinforcement and Grade 60 or Grade 40 rebar for transverse reinforcement.

6.5.2 Combination of Load Effects

For the design of the Honolulu building, all masses and superimposed gravity loads generated for the Berkeley building are used. This is conservative because the members for the Honolulu building are slightly smaller than the corresponding members for the Berkeley building. Also, the Honolulu building does not have reinforced concrete walls on Gridlines 3, 4, 5, and 6 (these walls are replaced by infilled, nonstructural masonry designed with gaps to accommodate frame drifts in the Honolulu building).

Provisions Sec. 5.2.7 [4.2.2] and Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2] require a combination of load effects to be developed on the basis of ASCE 7, except that the earthquake load (E) is defined as:

$$E = \rho Q_E + 0.2 S_{DS} D$$

when gravity and seismic load effects are additive and as:

$$E = \rho Q_E - 0.2 S_{DS} D$$

when the effects of seismic load counteract gravity.

For Seismic Design Category C buildings, *Provisions* Sec. 5.2.4.1 [4.3.3.1] permits the reliability factor (ρ) to be taken as 1.0. The special load combinations of *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-3 and 4.2-4] do not apply to the Honolulu building because there are no discontinuous elements supporting stiffer elements above them. (See *Provisions* Sec. 9.6.2 [9.4.1].)

For the Honolulu structure, the basic ASCE 7 load combinations that must be considered are:

$1.2D + 1.6L$
$1.2D + 0.5L \pm 1.0E$
$0.9D \pm 1.0E$

The ASCE 7 load combination including only 1.4 times dead load will not control for any condition in this building.

Substituting E from the *Provisions* and with ρ taken as 1.0, the following load combinations must be used for earthquake:

$(1.2 + 0.2S_{DS})D + 0.5L + E$
$(1.2 + 0.2S_{DS})D + 0.5L - E$
$(0.9 - 0.2S_{DS})D + E$
$(0.9 - 0.2S_{DS})D - E$

Finally, substituting 0.472 for S_{DS} (see Sec. 6.1.1), the following load combinations must be used for earthquake:

$1.30D + 0.5L + E$
$1.30D + 0.5L - E$
$0.80D + E$
$0.80D - E$

Note that the coefficients on dead load have been slightly rounded to simplify subsequent calculations.

As E-W wind loads apparently govern the design at the lower levels of the building (see Sec. 6.2.6 and Figure 6-4), the following load combinations should also be considered:

$1.2D + 0.5L + 1.6W$
$1.2D + 0.5L - 1.6W$
$0.9D - 1.6W$

The wind load (W) from ASCE 7 includes a directionality factor of 0.85.

It is very important to note that use of the ASCE 7 load combinations in lieu of the combinations given in ACI 318 Chapter 9 requires use of the alternate strength reduction factors given in ACI 318 Appendix C:

Flexure without axial load $\phi = 0.80$
Axial compression, using tied columns $\phi = 0.65$ (transitions to 0.8 at low axial loads)
Shear if shear strength is based on nominal axial-flexural capacity $\phi = 0.75$
Shear if shear strength is not based on nominal axial-flexural capacity $\phi = 0.55$
Shear in beam-column joints $\phi = 0.80$

[The strength reduction factors in ACI 318-02 have been revised to be consistent with the ASCE 7 load combinations. Thus, the factors that were in Appendix C of ACI 318-99 are now in Chapter 9 of ACI 318-02, with some modification. The strength reduction factors relevant to this example as contained in ACI 318-02 Sec. 9.3 are:

Flexure without axial load $\varphi = 0.9$ (tension-controlled sections)
Axial compression, using tied columns $\varphi = 0.65$ (transitions to 0.9 at low axial loads)
Shear if shear strength is based o nominal axial-flexural capacity $\varphi = 0.75$
Shear if shear strength is not based o nominal axial-flexural capacity $\varphi = 0.60$
Shear in beam-column joints $\varphi = 0.85$]

6.5.3 Accidental Torsion and Orthogonal Loading (Seismic Versus Wind)

As has been discussed and as illustrated in Figure 6-4, wind forces appear to govern the strength requirements of the structure at the lower floors, and seismic forces control at the upper floors. The seismic and wind shears, however, are so close at the midlevels of the structure that a careful evaluation must be made to determine which load governs for strength. This determination is complicated by the differing (wind versus seismic) rules for applying accidental torsion and for considering orthogonal loading effects.

Because the Honolulu building is in Seismic Design Category C and has no plan irregularities of Type 5 in *Provisions* Table 5.2.3.2 [4.3-2], orthogonal loading effects need not be considered per *Provisions* Sec. 5.2.5.2.2 [4.4.2.2]. However, as required by *Provisions* Sec. 5.4.4.2 [5.2.4.2], seismic story forces must be applied at a 5 percent accidental eccentricity. Torsional amplification is not required per *Provisions* Sec. 5.4.4.3 [5.2.4.3] because the building does not have a Type 1a or 1b torsional irregularity. (See Sec. 6.3.2 and 6.3.4 for supporting calculations and discussion.)

For wind, ASCE 7 requires that buildings over 60 ft in height be checked for four loading cases. The required loads are shown in Figure 6-38, which is reproduced directly from Figure 6-9 of ASCE 7. In Cases 1 and 2, load is applied separately in the two orthogonal directions. Case 2 may be seen to produce torsional effects because 7/8 of the total force is applied at an eccentricity of 3.57% the building width. This is relatively less severe than required for seismic effects, where 100 percent of the story force is applied at a 5 percent eccentricity.

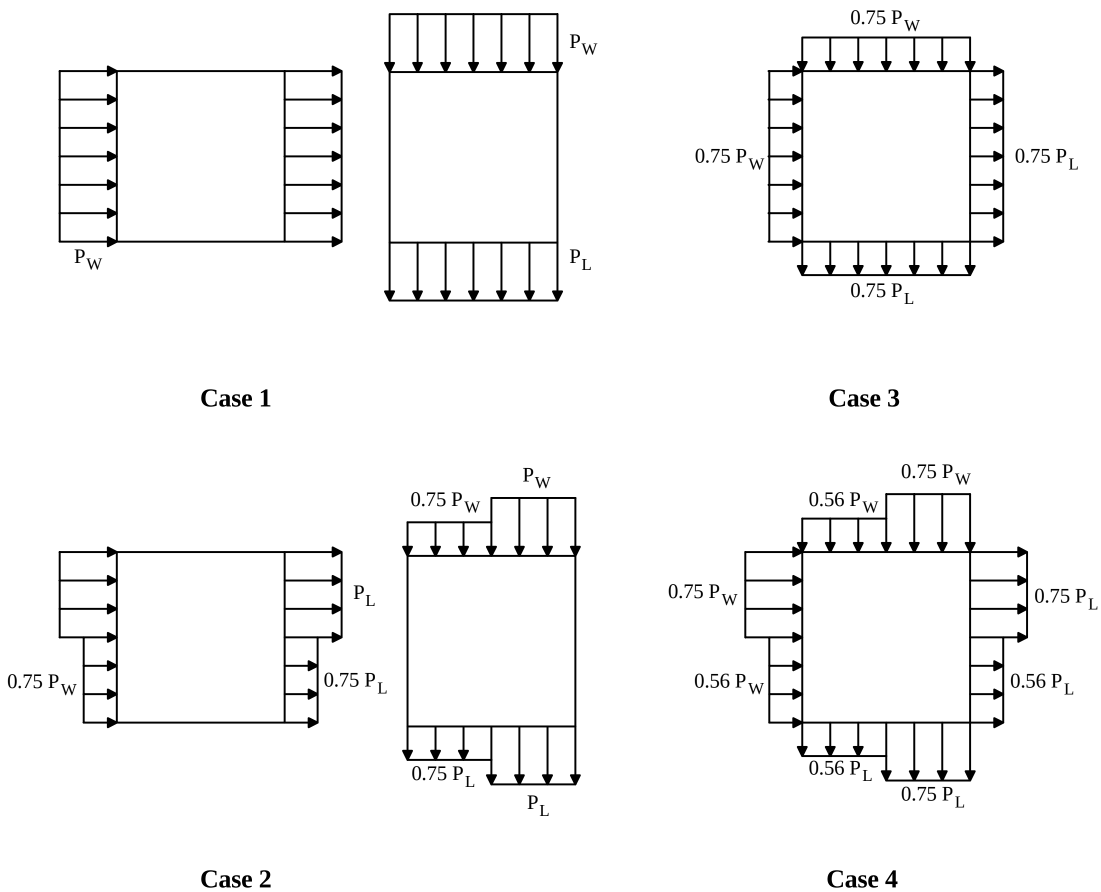


Figure 6-38 Wind loading requirements from ASCE 7.

For wind, Load Cases 3 and 4 require that 75 percent of the wind pressures from the two orthogonal directions be applied simultaneously. Case 4 is similar to Case 2 because of the torsion inducing pressure unbalance. As mentioned earlier, the Honolulu building has no orthogonal seismic loading requirements.

In this example, only loading in the E-W direction is considered. Hence, the following lateral load conditions were applied to the ETABS model:

100% E-W Seismic applied at 5% eccentricity
ASCE 7 Wind Case 1 applied in E-W direction only
ASCE 7 Wind Case 2 applied in E-W direction only
ASCE 7 Wind Case 3
ASCE 7 Wind Case 4

All cases with torsion are applied in such a manner as to maximize the shears in the elements of Frame 1.

6.5.4 Design and Detailing of Members of Frame 1

In this section, the girders and a typical interior column of Level 5 of Frame 1 are designed and detailed. For the five load cases indicated above, the girder shears produced from seismic effects control at the fifth level, with the next largest forces coming from direct E-W wind without torsion. This is shown graphically in Figure 6-39, where the shears in the exterior bay of Frame 1 are plotted vs. story height. Wind controls at the lower three stories and seismic controls for all other stories. This is somewhat different from that shown in Figure 6-4, wherein the total story shears are plotted and where wind controlled for the lower five stories. The basic difference between Figures 6-4 and 6-39 is that Figure 6-39 includes accidental torsion and, hence, Frame 1 sees a relatively larger seismic shear.

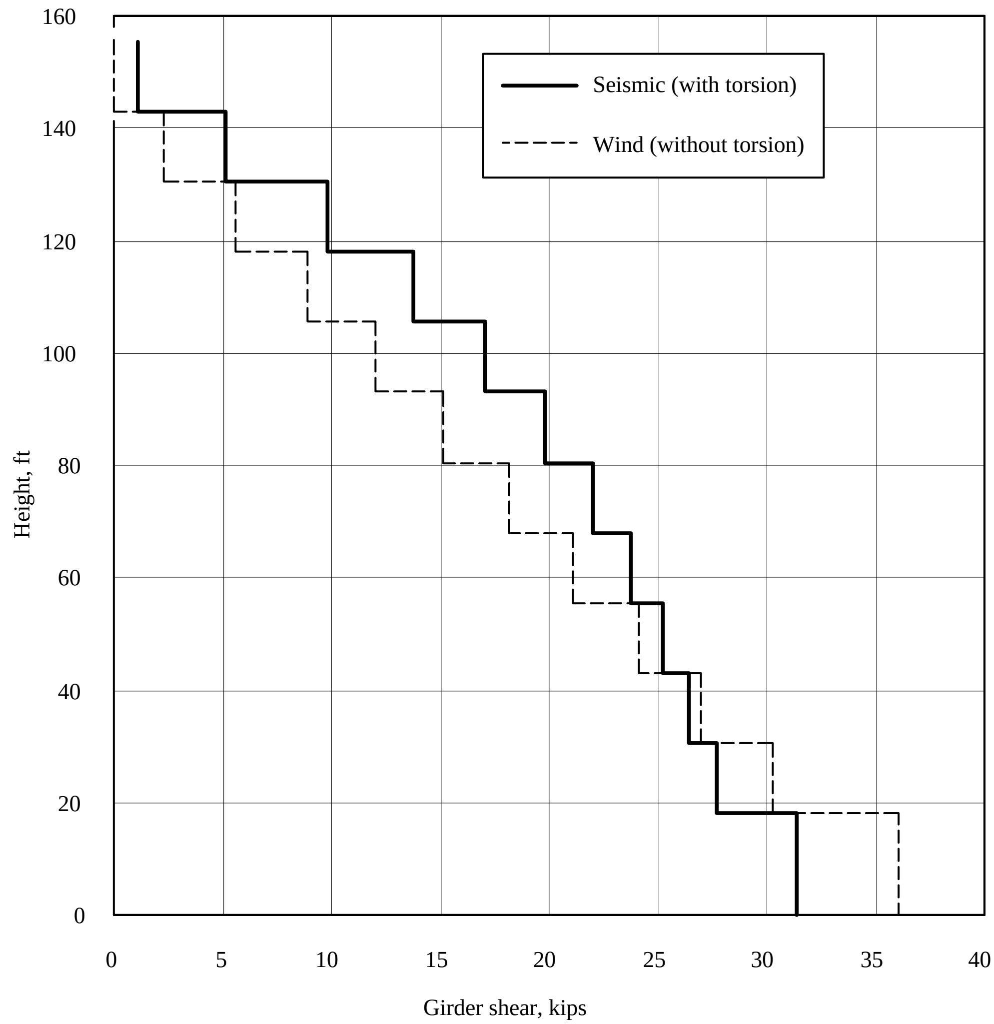


Figure 6-39 Wind vs. seismic shears in exterior bay of Frame 1 (1.0 ft = 0.3048 m, 1.0 kip = 4.45kN).

6.5.4.1 Initial Calculations

The girders of Frame 1 are 30 in. deep and 20 in. wide. For positive moment bending, the effective width of the compression flange is taken as 20 + 20(12)/12 = 40.0 in. Assuming 1.5 in. cover, #3 stirrups and #8 longitudinal reinforcement, the effective depth for computing flexural and shear strength is 27.6 in.

6.5.4.2 Design of Flexural Members

ACI 318 Sec. 21.10.4 [21.12.4] gives the minimum requirements for longitudinal and transverse reinforcement in the beams of intermediate moment frames. The requirements for longitudinal steel are as follows:

1. The positive moment strength at the face of a joint shall be at least one-third of the negative moment strength at the same joint.

2. Neither the positive nor the negative moment strength at any section along the length of the member shall be less than one-fifth of the maximum moment strength supplied at the face of either joint.

The second requirement has the effect of requiring top and bottom reinforcement along the full length of the member. The minimum reinforcement ratio at any section is taken from ACI 318 Sec. 10.5.1 as $200/f_y$ or 0.0033 for f_y = 60 ksi. However, according to ACI 318 Sec. 10.5.3, the minimum reinforcement provided need not exceed 1.3 times the amount of reinforcement required for strength.

The gravity loads and design moments for the first three spans of Frame 1 are shown in Figure 6-40. The seismic moments are taken directly from the ETABS analysis, and the gravity moments were computed by hand using the ACI coefficients. All moments are given at the face of the support. The gravity moments shown in Figures 6-40c and 6-40d are slightly larger than those shown for the Berkeley building (Figure 6-14) because the clear span for the Honolulu building increases due to the reduction in column size from 30 in. to 28 in.

Based on preliminary calculations, the reinforcement layout of Figure 6-41 will be checked. Note that the steel clearly satisfies the detailing requirements of ACI 318 Sec. 21.10.4 [21.12.4].

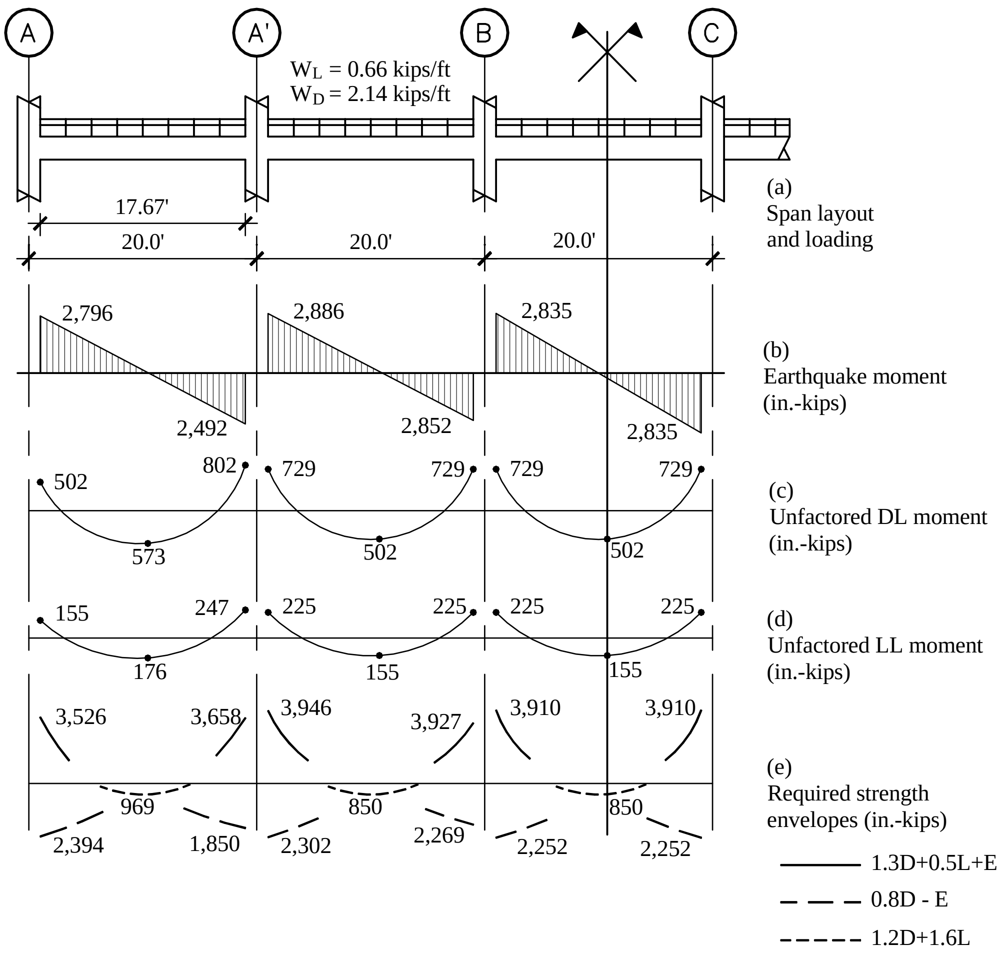


Figure 6-40 Bending moment envelopes at Level 5 of Frame 1 (1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

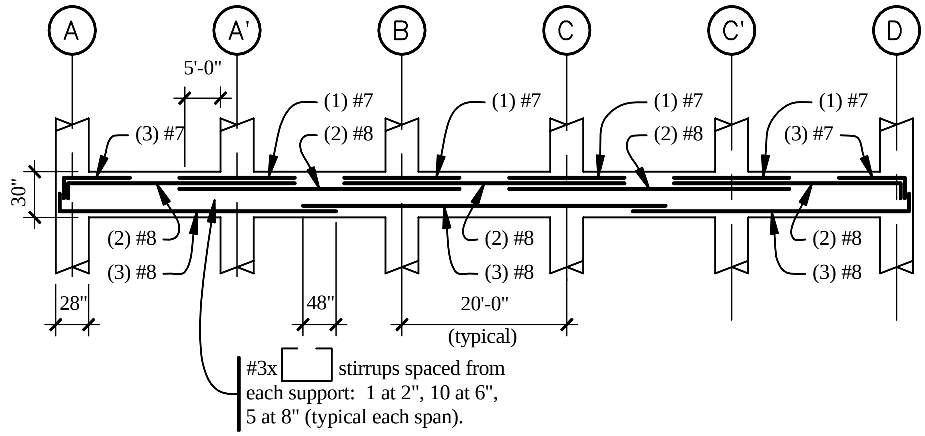


Figure 6-41 Preliminary reinforcement layout for Level 5 of Frame 1 (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

6.5.4.2.1 Design for Negative Moment at Face of Support A

M_u = -1.3 (502) - 0.5 (155) - 1.0 (2,796) = -3,526 in.-kips

Try three #7 short bars and two #8 long bars.

A_s = 3 (0.60) + 2 (0.79) = 3.38 in.^{2}
ρ = 0.0061
Depth of compression block, a = [3.38 (60)]/[0.85 (4) 20] = 2.98 in.
Nominal moment capacity, $M_n = A_s f_y(d - a/2)$ = [3.38 (60.0)] [27.6 - 2.98/2] = 5,295 in.-kips
Design capacity, ϕM_n = 0.8(5,295) = 4,236 in.-kips > 3,526 in.-kips OK

6.5.4.2.2 Design for Positive Moment at Face of Support A

M_u = -0.8 (502) + 1.0 (2,796) = 2,394 in.-kips

Try three #8 long bars.

$A_s f_y$ = 3 (0.79) = 2.37 in.^{2}
ρ = 0.0043
a = 2.37 (60)/[0.85 (4) 40] = 1.05 in.
$M_n = A_s f_y(d - a/2)$ = [2.37 (60.0)][27.6 - 1.05/2] = 3,850 in.-kips
ϕM_n = 0.8(3850) = 3,080 in.-kips > 2,394 in.-kips OK

This reinforcement also will work for positive moment at all other supports.

6.5.4.2.3 Design for Negative Moment at Face of Support A'

M_u = -1.3 (729) - 0.5 (225) - 1.0 (2,886) = 3,946 in.-kips

Try four #8 long bars and one #7 short bar:

A_s = 4 (0.79) + 1 (0.6) = 3.76 in.^{2}
ρ = 0.0068

a = [3.76 (60)]/[0.85 (4) 20] = 3.32 in.
$M_n = A_s f_y(d - a/2)$ = [3.76 (60.0)][27.6 - 3.32/2] = 5,852 in.-kips
ϕM_n = 0.8(5,852) = 4,681 in.-kips > 3,946 in.-kips OK

This reinforcement will also work for negative moment at Supports B and C. Therefore, the flexural reinforcement layout shown in Figure 6-41 is adequate. The top short bars are cut off 5 ft-0 in. from the face of the support. The bottom bars are spliced in Spans A'-B and C-C' with a Class B lap length of 48 in. Unlike special moment frames, there are no requirements that the spliced region of the bars in intermediate moment frames be confined by hoops over the length of the splice.

6.5.4.2.4 Design for Shear Force in Span A'-B:

ACI 318 Sec. 21.10.3 [21.12.3] provides two choices for computing the shear strength demand in a member of an intermediate moment frame:

1. The first option requires that the design shear force for earthquake be based on the nominal moment strength at the ends of the members. Nominal moment strengths are computed with a flexural reinforcement tensile strength of $1.0f_y$ and a flexural ϕ factor of 1.0. The earthquake shears computed from the nominal flexural strength are added to the factored gravity shears to determine the total design shear.

2. The second option requires that the design earthquake shear force be 2.0 times the factored earthquake shear taken from the structural analysis. This shear is used in combination with the factored gravity shears.

For this example, the first option is used. The nominal strengths at the ends of the beam were computed earlier as 3850 in.-kips for positive moment at Support A' and 5,852 in.-kips for negative moment at Support B. Compute the design earthquake shear V_E:

$$V_E = \frac{5,852 + 3,850}{212} = 45.8 \text{ kips}$$

where 212 in. is the clear span of the member. For earthquake forces acting in the other direction, the earthquake shear is 43.1 kips.

The gravity load shears at the face of the supports are:

$$V_D = \frac{2.14(20 - 2.33)}{2} = 18.9 \text{ kips}$$

$$V_L = \frac{0.66(20 - 2.33)}{2} = 5.83 \text{ kips}$$

The factored design shear V_u = 1.3(18.9) + 0.5(5.8) + 1.0(45.8) = 73.3 kips. This shear force applies for earthquake forces coming from either direction as shown in the shear strength design envelope in Figure 6-42.

The design shear force is resisted by a concrete component (V_c) and a steel component (V_s). Note that the concrete component may be used regardless of the ratio of earthquake shear to total shear. The required design strength is:

$V_u \leq \phi V_c + \phi V_s$

where ϕ = 0.75 for shear.

$$V_c = \frac{(0.85)\ (2\sqrt{4,000})20(27.6)}{1,000} = 59.3 \text{ kips}$$

The factor of 0.85 above reflects the reduced shear capacity of sand-LW concrete.

The shear to be resisted by steel, assuming stirrups consist of two #3 legs (A_v = 0.22) and f_y = 40 ksi is:

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{73.3 - 0.75(59.3)}{0.75} = 38.4 \text{ kips}$$

Using $V_S = A_v f_y d/s$:

$$s = \frac{(0.22)(40)(27.6)}{38.4} = 6.32 \text{ in.}$$

Minimum transverse steel requirements are given in ACI 318 Sec. 21.10.4.2 [21.12.4.2]. The first stirrup should be placed 2 in. from the face of the support, and within a distance 2*h* from the face of the support, the spacing should be not greater than *d*/4, eight times the smallest longitudinal bar diameter, 24 times the stirrup diameter, or 12 in. For the beam under consideration *d*/4 controls minimum transverse steel, with the maximum spacing being 27.6/4 = 6.9 in. This is slightly greater, however, than the 6.32 in. required for strength. In the remainder of the span, stirrups should be placed at a maximum of *d*/2 (ACI 318 Sec. 21.10.4.3 [21.12.4.3]).

Because the earthquake shear (at midspan) is greater than 50 percent of the shear strength provided by concrete alone, the minimum requirements of ACI 318 Sec. 11.5.5.3 must be checked:

$$s_{max} = \frac{0.2(40,000)}{50(20)} = 8.0 \text{ in.}$$

This spacing controls over the *d*/2 requirement. The final spacing used for the beam is shown in Figure 6-41. This spacing is used for all other spans as well. The stirrups may be detailed according to ACI 318 Sec. 7.1.3, which requires a 90-degree hook with a $6d_b$ extension. This is in contrast to the details of the Berkeley building where full hoops with 135-degree hooks are required in the critical region (within 2*d* from the face of the support) and stirrups with 135-degree hooks are required elsewhere.

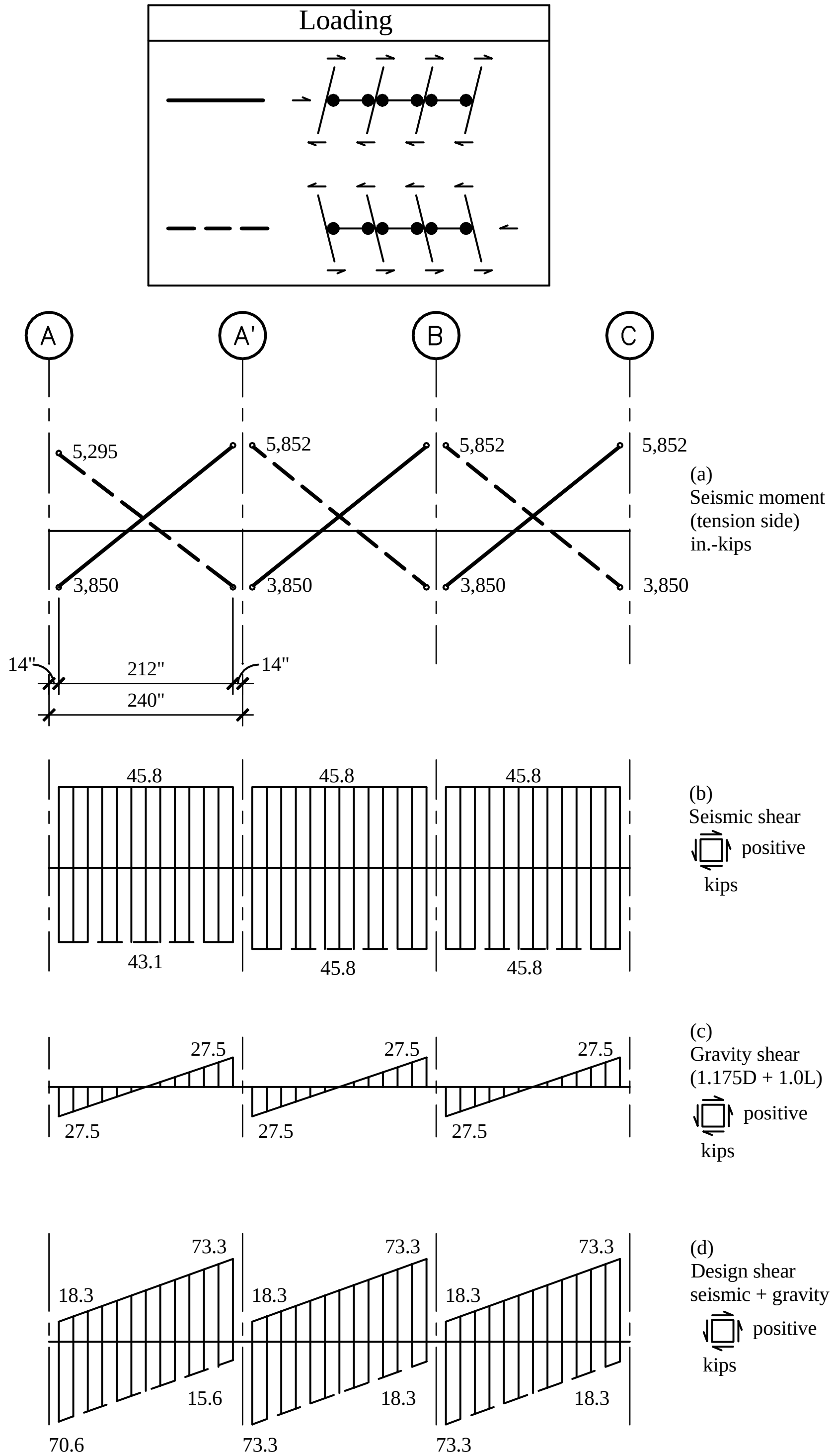


Figure 6-42 Shear strength envelopes for Span A'-B of Frame 1 (1.0 in = 25.4 mm, 1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

6.5.4.3 Design of Typical Interior Column of Frame 1

This section illustrates the design of a typical interior column on Gridline A'. The column, which supports Level 5 of Frame 1, is 28 in. square and is constructed from 4,000 psi LW concrete, 60 ksi longitudinal reinforcement, and 40 ksi transverse reinforcement. An isolated view of the column is shown in Figure 6-43.

The column supports an unfactored axial dead load of 528 kips and an unfactored axial live load of 54 kips. The ETABS analysis indicates that the axial earthquake force is ±33.2 kips, the earthquake shear force is ±41.9 kips, and the earthquake moments at the top and the bottom of the column are ±2,137 and ±2,708 in.-kips, respectively. Moments and shears due to gravity loads are assumed to be negligible.

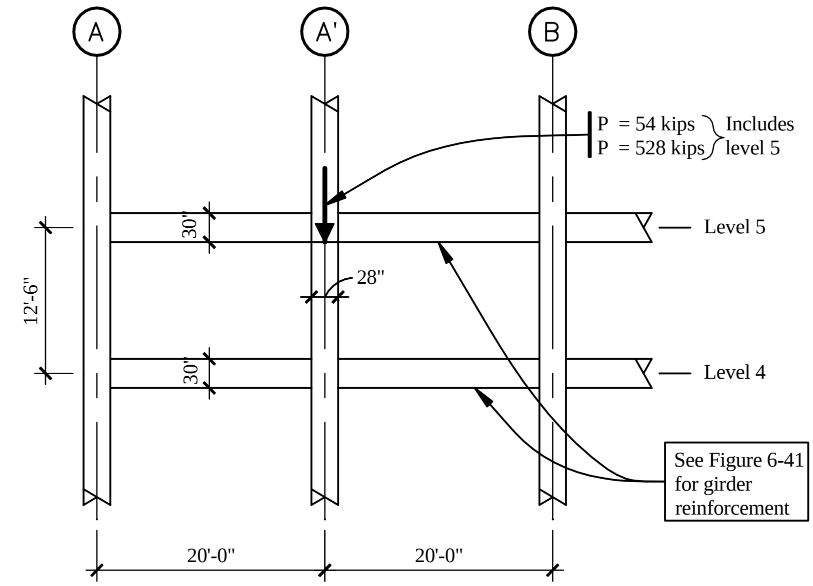


Figure 6-43 Isolated view of column A' (1.0 ft = 0.3048 m, 1.0 kip = 4.45kN).

6.5.4.3.1 Design of Longitudinal Reinforcement

The factored gravity force for maximum compression (without earthquake) is:

$$P_u = 1.2(528) + 1.6(54) = 720 \text{ kips}$$

This force acts with no significant gravity moment.

The factored gravity force for maximum compression (including earthquake) is:

$$P_u = 1.3(528) + 0.5(54) + 33.2 = 746.6 \text{ kips}$$

The factored gravity force for minimum compression (including earthquake) is:

$$P_u = 0.8(528) - 33.2 = 389.2 \text{ kips}$$

Since the frame being designed is unbraced in both the N-S and E-W directions, slenderness effects should be checked. For a 28-in.-by-28-in. column with a clear unbraced length. l_u = 120 in., r = 0.3(28) = 8.4 in. (ACI 318 Sec. 10.11.3) and l_u/r = 120/8.4 = 14.3.

ACI 318 Sec. 10.11.4.2 states that the frame may be considered braced against sidesway if the story stability factor is less than 0.05. This factor is given as:

$$Q = \frac{\sum P_u \delta_0}{V_u l_c}$$

which is basically the same as *Provisions* Eq. 5.4.6.2-1 [5.2-16] except that in the ACI equation, the gravity forces are factored. [Note also that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, *I*, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because I = 1.0.] ACI is silent on whether or not δ_0 should include C_d. In this example, δ_0 does not include C_d, and is therefore consistent with the *Provisions*. As can be seen from earlier calculations shown in Table 6-12b, the ACI story stability factor will be in excess of 0.05 for Level 5 of the building responding in the E-W direction. Hence, the structure must be considered unbraced.

Even though the frame is defined as unbraced, ACI 318 Sec. 10.13.2 allows slenderness effects to be neglected when kl_u/r < 22. This requires that the effective length factor k for this column be less than 1.54. For use with the nomograph for unbraced columns (ACI 318 Figure R10.12.1b):

$$\left(\frac{EI}{L}\right)_{Girder} = \frac{E(45,000)}{240} = 187.5E$$

According to ACI 318 Sec. 10.12.3:

$$\left(\frac{EI}{L}\right)_{Column} = \frac{\left(\frac{0.4EI_{Column}}{(1+\beta_d)}\right)}{150}$$

Using the 1.2 and 1.6 load factors on gravity load:

$$\beta_d = \frac{1.2(528)}{720} = 0.88$$

$$I_{Column} = \frac{28^3(28)}{12} = 51,221 \text{ in.}^4$$

$$\left(\frac{EI}{L}\right)_{Column} = \frac{\frac{0.4(51,221E)}{1+0.88}}{150} = 72.7E$$

Because there is a column above and below as well as a beam on either side:

$$\Psi_{Top} = \Psi_{Bottom} = \frac{72.7}{187.5} = 0.39$$

and the effective length factor k = 1.15 (ACI 318 Figure R10.12.1b). As the computed effective length factor is less than 1.54, slenderness effects need not be checked for this column.[5]

Continuing with the design, an axial-flexural interaction diagram for a 28-in.-by-28-in. column with 12 #8 bars (ρ = 0.0121) is shown in Figure 6-44. The column clearly has the strength to support the applied loads (represented as solid dots in the figure).

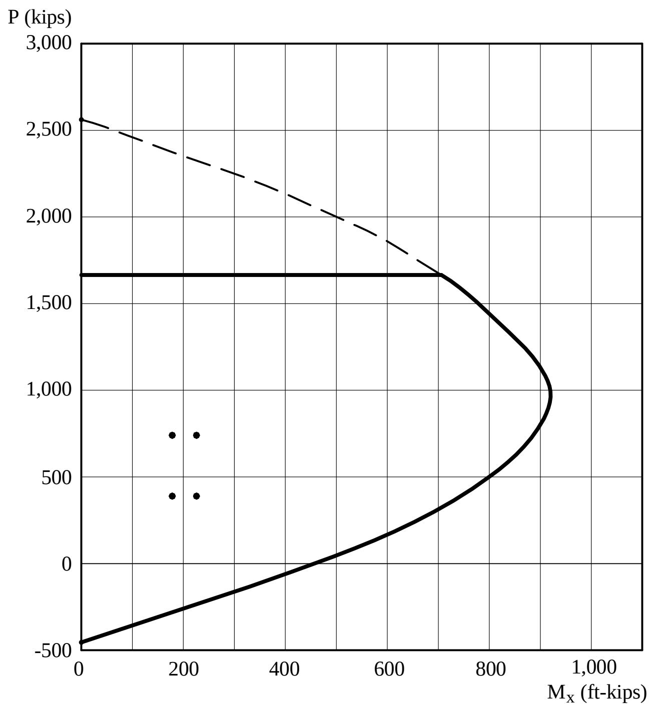


Figure 6-44 Interaction diagram for column (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

6.5.4.3.2 Design and Detailing of Transverse Reinforcement

ACI 318 Sec. 21.10.3 [21.12.3] allows the column to be checked for 2.0 times the factored shear force as derived from the structural analysis. The ETABS analysis indicates that the shear force is 41.9 kips and the design shear is 2.0(41.9) = 83.8 kips.

The concrete supplies a capacity of:

$$V_c = 0.85(2)\sqrt{f_c'}b_w d = 0.85(2)\sqrt{4,000}(28)(25.6) = \text{ 77.1 kips}$$

[5]For loading in the N-S direction, the column under consideration has no beam framing into it in the direction of loading. If the stiffness contributed by the joists and the spandrel beam acting in torsion is ignored, the effective length factor for the column in the N-S direction is effectively infinity. However, this column is only one of four in a story containing a total of 36 columns. Since each of the other 32 columns has a lateral stiffness well in excess of that required for story stability in the N-S direction, the columns on Lines A' and C' can be considered to be laterally supported by the other 32 columns and therefore can be designed using an effective length factor of 1.0. A P-delta analysis carried out per the ACI Commentary would be required to substantiate this.

The requirement for steel reinforcement is:

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{83.8 - 0.75(77.1)}{0.75} = 34.6 \text{ kips}$$

Using ties with four #3 legs, s = [4(0.11)] [40.0 (25.6/34.6)] = 13.02 in.

ACI 318 Sec. 21.10.5 [21.12.5] specifies the minimum reinforcement required. Within a region l_o from the face of the support, the tie spacing should not exceed:

$8.0d_b$ = 8.0 (1.008) = 8.00 in. (using #8 longitudinal bars)
$24d_{tie}$ = 24 (3/8) = 9.0 in. (using #3 ties)
1/2 the smallest dimension of the frame member = 28/2 = 14 in.
12 in.

The 8.0 in. maximum spacing controls. Ties at this spacing are required over a length l_o of:

1/6 clearspan of column = 120/6 = 20 in.
maximum cross section dimension = 28 in.
18.0 in.

Given the above, a four-legged #3 tie spaced at 8 in. over a depth of 28 in. will be used. One tie will be provided at 4 in. below the beam soffit, the next tie is placed 4 in. above the floor slab, and the remaining ties are spaced at 8 in. on center. The final spacing is as shown in Figure 6-45. Note that the tie spacing is not varied beyond l_o.

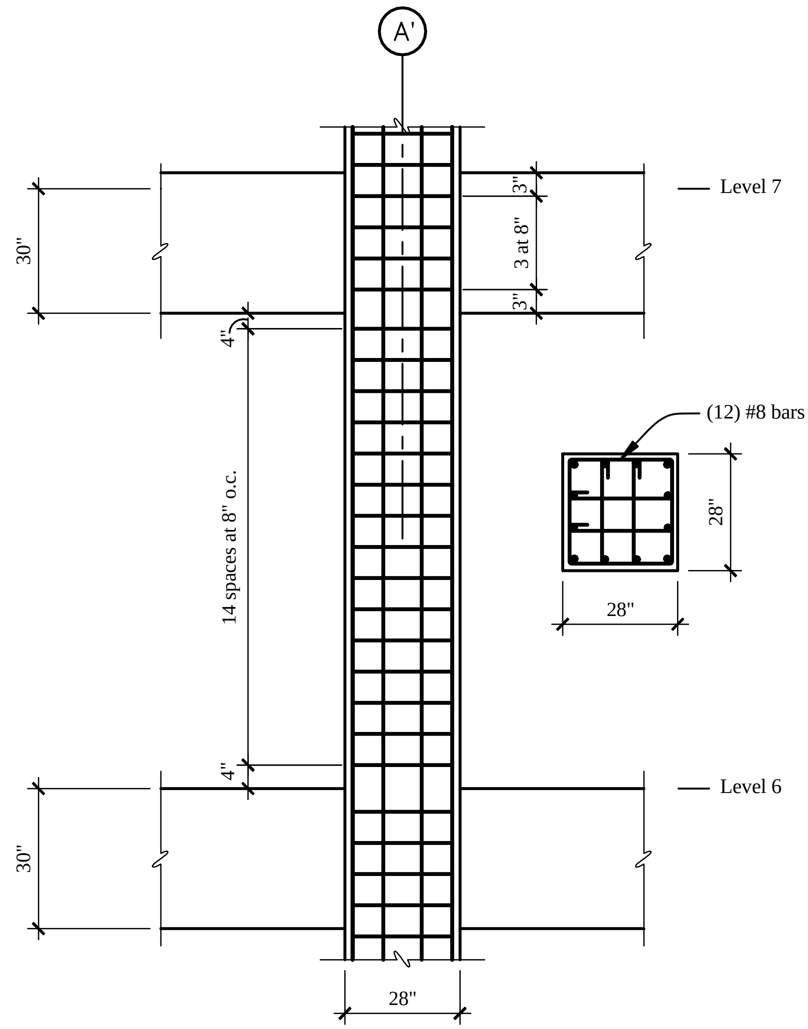


Figure 6-45 Column reinforcement (1.0 in = 25.4 mm).

6.5.4.4 Design of Beam-Column Joint

Joint reinforcement for intermediate moment frames is addressed in ACI 318 Sec. 21.10.5.3 [21.12.5.5], which refers to Sec. 11.11.2. ACI 318 Sec. 11.11.2 requires that all beam-column connections have a minimum amount of transverse reinforcement through the beam-column joints. The only exception is in nonseismic frames where the column is confined on all four sides by beams framing into the column. The amount of reinforcement required is given by ACI 318 Eq. 11-13:

$$A_v = 50\left(\frac{b_w s}{f_y}\right)$$

This is the same equation used to proportion minimum transverse reinforcement in beams. Assuming A_v is supplied by four #3 ties and f_y = 40 ksi:

$$s = \frac{4(0.11)(40{,}000)}{50(28)} = 12.6 \text{ in.}$$

This effectively requires only two ties within the joint. However, the first tie will be placed 3 in. below the top of the beam and then three additional ties will be placed below this hoop at a spacing of 8 in. The final arrangement of ties within the beam-column joint is shown in Figure 6-45.

6.5.5 Design of Members of Frame 3

6.5.5.1 Design of Haunched Girder

A typical haunched girder supporting Level 5 of Frame 3 is now illustrated. This girder, located between Gridlines A and B, has a variable depth with a maximum depth of 30 in. at the support and a minimum depth of 20 in. for the middle half of the span. The length of the haunch at each end (as measured from the face of the support) is 106 in. The width of the girder is 20 in. throughout. The girder frames into 28-in.-by-28-in. columns on Gridlines A and B. As illustrated in Figure 6-46c, the reinforcement at Gridline B is extended into the adjacent span (Span B-C) instead of being hooked into the column.

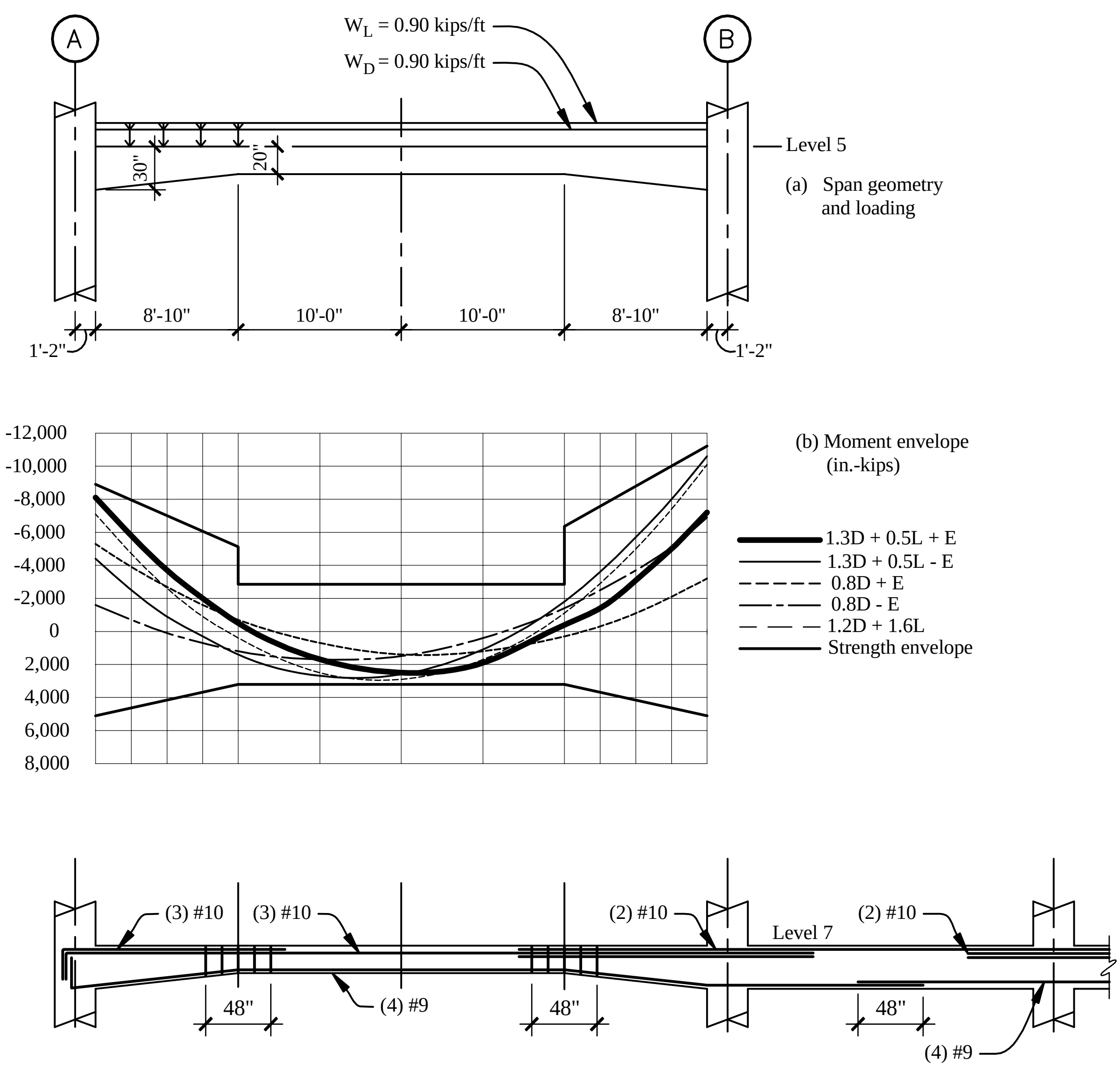


Figure 6-46 Loads, moments, and reinforcement for haunched girder (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

Based on a tributary gravity load analysis, this girder supports an average of 3.38 kips/ft of dead load and 0.90 kips/ft of reduced live load. A gravity load analysis of the girder was carried out in a similar manner similar to that described above for the Berkeley building.

For determining earthquake forces, the entire structure was analyzed using the ETABS program. This analysis included 100 percent of the earthquake forces in the E-W direction placed at a 5 percent eccentricity with the direction of the eccentricity set to produce the maximum seismic shear in the member.

6.5.5.2 Design of Longitudinal Reinforcement

The results of the analysis are shown in Figure 6-46b for five different load combinations. The envelopes of maximum positive and negative moment indicate that $1.2D + 1.6L$ and $1.3D + 0.5L \pm E$ produce approximately equal negative end moments. Positive moment at the support is nearly zero under $0.8D - E$, and gravity controls midspan positive moment. Since positive moment at the support is negligible, a positive moment capacity of at least one-third of the negative moment capacity will be supplied per ACI 318 Sec. 21.10.4.1 [21.12.4.1]. The minimum positive or negative moment strength at any section of the span will not be less than one-fifth of the maximum negative moment strength.

For a factored negative moment of 8,106 in.-kips on Gridline A, try six #10 bars. Three of the bars are short, extending just past the end of the haunch. The other three bars are long and extend into Span B-C.

A_s = 6 (1.27) = 7.62 in.^{2}
d = 30 - 1.5 - 0.375 - 1.27/2 = 27.49 in.
ρ = 7.62/[20 (27.49)] = 0.0139
Depth of compression block, a = [7.62 (60)]/[0.85 (4) 20.0] = 6.72 in.
Nominal capacity, M_n = [7.62 (60)](27.49 - 6.72/2) = 11,031 in-kips
Design capacity, ϕM_n = 0.8(11,031) = 8,824 in.-kips > 8,106 in.-kips OK

The three #10 bars that extend across the top of the span easily supply a minimum of one-fifth of the negative moment strength at the face of the support.

For a factored negative moment of 10,641 in.-kips on Gridline B, try eight #10 bars. Three of the bars extend from Span A-B, three extend from Span B-C, and the remaining two are short bars centered over Support B.

A_s = 8 (1.27) = 10.16 in.^{2}
d = 30 - 1.5 - 0.375 - 1.27/2 = 27.49 in.
ρ = 10.16/[20 (27.49)] = 0.0185
a = [10.16 (60)]/[0.85 (4) 20.0] = 8.96 in.
M_n = [10.16 (60)](27.49 - 8.96/2) = 13,996 in.-kips
ϕM_n = 0.8(13,996) = 11,221 in.-kips > 10,641 in.-kips OK

For the maximum factored positive moment at midspan of 2,964 in-kips., try four #9 bars:

A_s = 4 (1.0) = 4.00 in.2
d = 20 - 1.5 - 0.375 - 1.128/2 = 17.56 in.
ρ = 4.0/[20 (17.56)] = 0.0114
a = [4.00 (60)]/[0.85 (4) 84] = 0.84 in. (effective flange width = 84 in.)
M_n = [4.00 (60)](17.56 - 0.84/2) = 4,113 in.-kips
ϕM_n = 0.8(4,113) = 3,290 in.-kips > 2,964 OK

Even though they provide more than one-third of the negative moment strength at the support, the four #9 bars will be extended into the supports as shown in Figure 6-46. The design positive moment strength for the 30-in.-deep section with four #9 bars is computed as follows:

A_s = 4 (1.00) = 1.00 in.^{2}
d = 30 - 1.5 - 0.375 - 1.128/2 = 27.56 in.
ρ = 4.00/[20 (27.56)] = 0.0073
a = [4.0 (60)]/[0.85 (4) 20.0] = 0.84 in.
M_n = [4.00 (60)] (27.56 - 0.84/2) = 6,514 in.-kips
ϕM_n = 0.8(6,514) = 5,211 in.-kips

The final layout of longitudinal reinforcement used is shown in Figure 6-46. Note that the supplied design strengths at each location exceed the factored moment demands. The hooked #10 bars can easily be developed in the confined core of the columns. Splices shown are Class B and do not need to be confined within hoops.

6.5.5.3 Design of Transverse Reinforcement

For the design for shear, ACI 318 Sec. 21.10.3 [21.12.3] gives the two options discussed above. For the haunched girder, the approach based on the nominal flexural capacity (ϕ = 1.0) of the girder will be used as follows:

For negative moment and six #10 bars, the nominal moment strength = 11,031 in.-kips
For negative moment and eight #10 bars, the nominal strength =13,996 in.-kips
For positive moment and four #9 bars, the nominal moment strength = 6,514 in.-kips

Earthquake shear when Support A is under positive seismic moment is:

V_E = (13,996 + 6,514)/(480 - 28) = 45.4 kips

Earthquake shear when Support B is under positive seismic moment is:

V_E = (11,031 + 6,514)/(480 - 28) = 38.8 kips
$V_G = 1.3V_D + 0.5V_L$ = 1.3 (63.6) + 0.5(16.9) = 91.1 kips

Maximum total shear occurs at Support B:

V_u = 45.4+91.1 = 136.5 kips

The shear at Support A is 38.8 + 91.9 = 130.1 kips. The complete design shear (demand) strength envelope is shown in Figure 6-47a. Due to the small difference in end shears, use the larger shear for designing transverse reinforcement at each end.

Stirrup spacing required for strength is based on two #4 legs with f_y = 60 ksi.

$$V_c = \frac{(0.85)(2)\sqrt{4{,}000}(20)(27.6)}{1{,}000} = 59.3 \text{ kips}$$

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{136.5 - 0.75(59.3)}{0.75} = 122.7 \text{ kips}$$

Using $V_s = A_v f_y d/s$:

$$s = \frac{(0.4)(60)(27.6)}{122.7} = 5.39 \text{ in.}$$

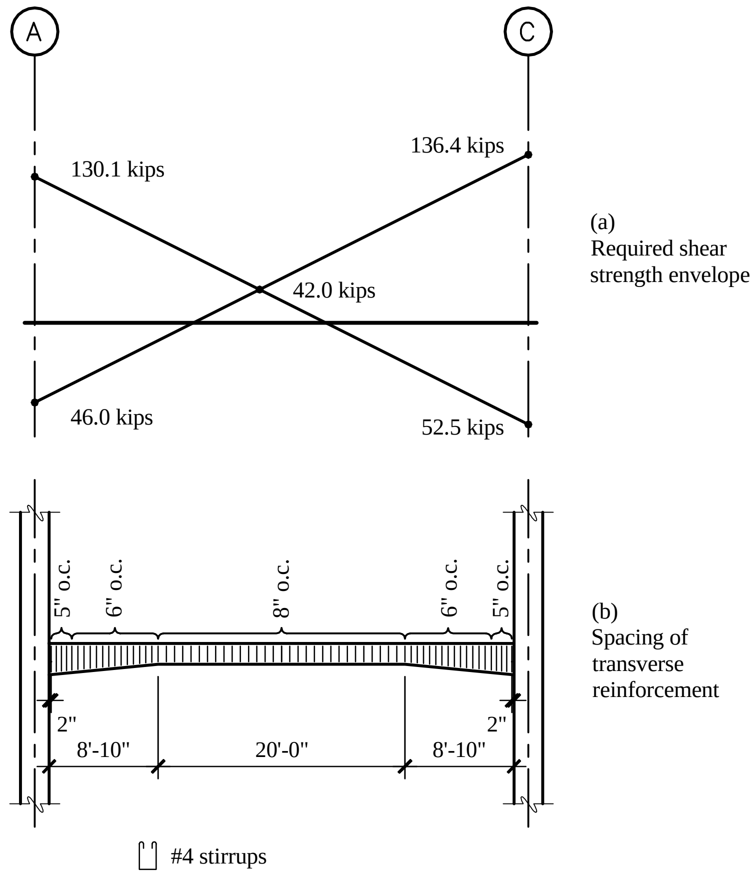


Figure 6-47 Shear force envelope for haunched girder (1.0 ft = 0.3048m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

Following the same procedure as shown above, the spacing required for other stations is:

At support, h = 30 in., V_U = 136.4 kips	s = 5.39 in.
Middle of haunch, h = 25 in., V_U = 114.9 kips	s = 6.67 in.
End of haunch, h =20 in., V_U =93.4 kips	s = 7.61 in.
Quarter point of region of 20-in. depth, V_U = 69.2 kips	s = 12.1 in.
Midspan, V_u = 45.1 kips	s = 29.7 in.

Within a region 2h from the face of the support, the allowable maximum spacing is d/4 = 6.87 in. at the support and approximately 5.60 in. at midhaunch. Outside this region, the maximum spacing is d/2 = 11.2 in. at midhaunch and 8.75 in. at the end of the haunch and in the 20-in. depth region. At the haunched segments at either end of the beam, the first stirrup is placed 2 in. from the face of the support followed by four stirrups at a spacing of 5 in, and then 13 stirrups at 6 in. through the remainder of the haunch. For the constant 20-in.-deep segment of the beam, a constant spacing of 8 in. is used. The final spacing of stirrups used is shown in Figure 6-47b. Three additional stirrups should be placed at each bend or “kink” in the bottom bars. One should be located at the kink and the others approximately 2 in. on either side of the kink.

6.5.5.4 Design of Supporting Column

The column on Gridline A which supports Level 5 of the haunched girder is 28 in. by 28 in. and supports a total unfactored dead load of 803.6 kips and an unfactored reduced live load of 78.4 kips. The layout of the column is shown in Figure 6-48. Under gravity load alone, the unfactored dead load moment is 2,603 in.-kips and the corresponding live load moment is 693.0 in.-kips. The corresponding shears are 43.4 and 11.5 kips, respectively. The factored gravity load combinations for designing the column are as follows:

Bending moment, M = 1.2(2,603) + 1.6(693)
= 4,232 in.-kips

This moment causes tension on the outside face of the top of the column and tension on the inside face of the bottom of the column.

Shear, V = 1.2(43.4) + 0.5(11.5) = 57.8 kips

Axial compression, P = 1.2(803.6) + 1.6(78.4)
= 1,090 kips

For equivalent static earthquake forces acting from west to east, the forces in the column are obtained from the ETABS analysis as follows:

Moment at top of column = 690 in.-kips (tension on inside face subtracts from gravity)
Moment at bottom of column = 874 in.-kips (tension on outside face subtracts from gravity)
Shear in column = 13.3 kips (opposite sign of gravity shear)
Axial force = 63.1 kips tension

The factored forces involving earthquake from west to east are:

Moment at top 0.80(2603) - 690 = 1,392 in.-kips
Moment at bottom = 0.80(2603) - 874 = 1,208 in.-kips
Shear = 0.80(43.4) - 2(13.3) = 8.1 kips (using the second option for computing EQ shear)
Axial force = 0.80(803.6) - 63.1 = 580 kips

For earthquake forces acting from east to west, the forces in the column are obtained from the ETABS analysis as follows:

Moment at top of column = 690 in.-kips (tension on outside face adds to gravity)
Moment at bottom of column = 874 in.-kips (tension on inside face adds to gravity)
Shear in column = 13.3 kips (same sign of gravity shear)
Axial force = 63.1 kips compression

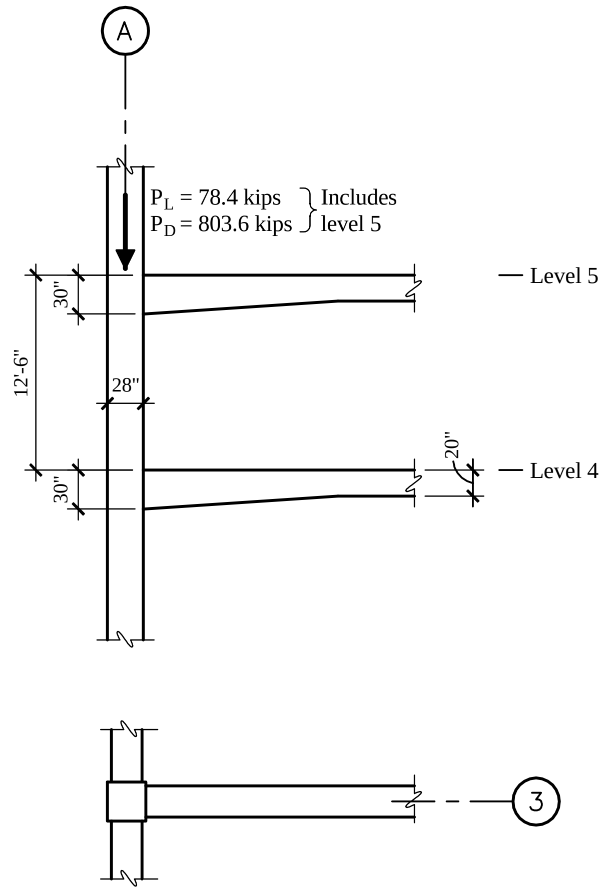


Figure 6-48 Loading for Column A, Frame 3 (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

The factored forces involving earthquake from east to west are:

Moment at top 1.3(2,603) + 0.5(693) + 690 = 4,420 in.-kips
Moment at bottom = 1.3(2,603) + 0.5(693) + 874 = 4,604 in.-kips
Shear = 1.3(43.4) + 0.5(11.5) + 2(13.3) = 94.6 kips (using second option for computing EQ shear)
Axial force = 1.3(803.6) + 0.5(78.4) + 63.1 = 1,147 kips

As may be observed from Figure 6-49, the column with 12 #8 bars is adequate for all loading combinations. Since the maximum design shear is less than that for the column previously designed for Frame 1 and since minimum transverse reinforcement controlled that column, the details for the column currently under consideration are similar to those shown in Figure 6-45. The actual details for the column supporting the haunched girder of Frame 3 are shown in Figure 6-50.

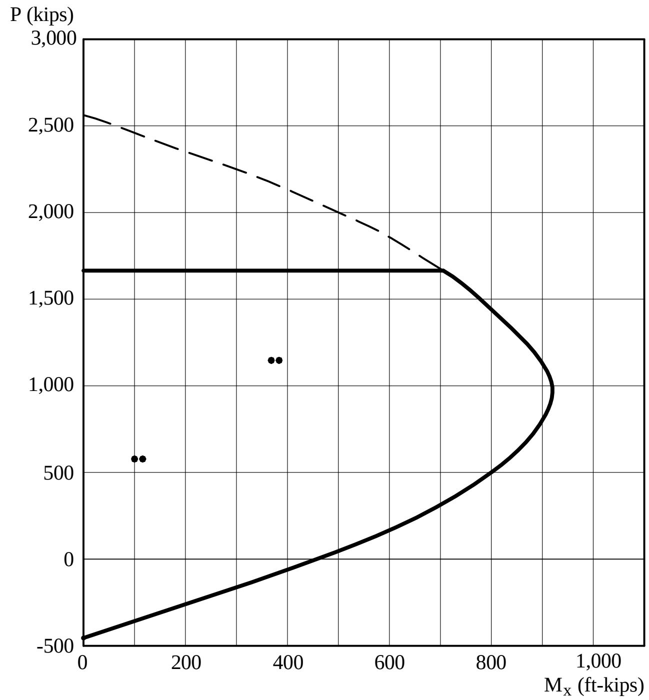


Figure 6-49 Interaction diagram for Column A, Frame 3 (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

6.5.5.5 Design of Beam-Column Joint

The detailing of the joint of the column supporting Level 5 of the haunched girder is the same as that for the column interior column of Frame A. The joint details are shown in Figure 6-50.

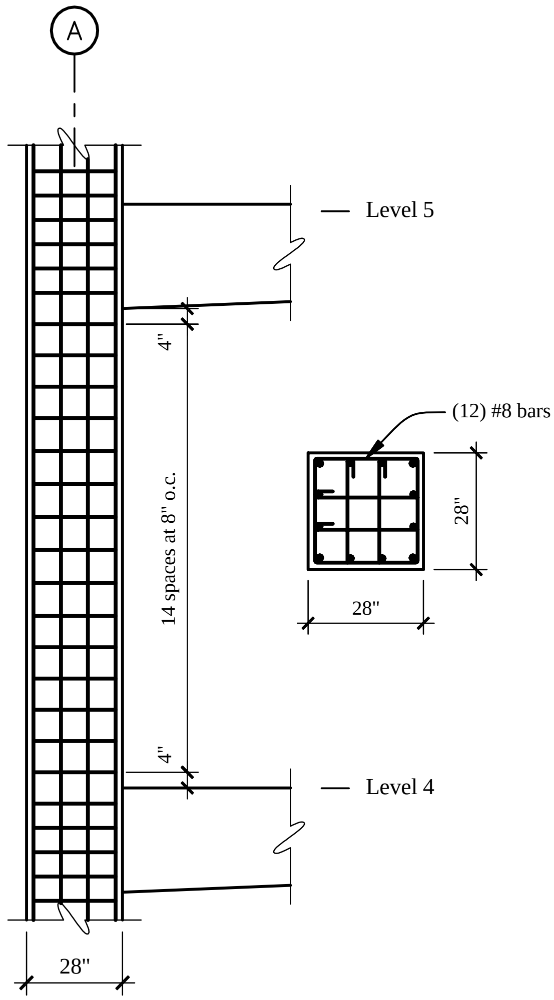


Figure 6-50 Details for Column A, Frame 3 (1.0 in = 25.4 mm).

7

PRECAST CONCRETE DESIGN

Gene R. Stevens, P.E. and James Robert Harris, P.E., Ph.D.

This chapter illustrates the seismic design of precast concrete members using the *NEHRP Recommended Provisions* (referred to herein as the *Provisions*) for buildings in several different seismic design categories. Very briefly, for precast concrete structural systems, the *Provisions*:

1. Requires the system (even if the precast carries only gravity loads) to satisfy one of the following two sets of provisions:

   a. Resist amplified chord forces in diaphragms and, if moment-resisting frames are used as the vertical system, provide a minimum degree of redundancy measured as a fraction of available bays, or
   b. Provide a moment-resisting connection at all beam-to-column joints with positive lateral support for columns and with special considerations for bearing lengths.

   (In the authors' opinion this does not apply to buildings in Seismic Design Category A.)

2. Requires assurance of ductility at connections that resist overturning for ordinary precast concrete shear walls. (Because ordinary shear walls are used in lower Seismic Design Categories, this requirement applies in Seismic Design Categories B and C.)

3. Allows special moment frames and special shear walls of precast concrete to either emulate the behavior of monolithic concrete or behave as jointed precast systems. Some detail is given for special moment frame designs that emulate monolithic concrete. To validate designs that do not emulate monolithic concrete, reference is made to a new ACI testing standard (ACI T1.1-01).

4. Defines that monolithic emulation may be achieved through the use of either:

   a. Ductile connections, in which the nonlinear response occurs at a connection between a precast unit and another structural element, precast or not, or
   b. Strong connections, in which the nonlinear response occurs in reinforced concrete sections (generally precast) away from connections that are strong enough to avoid yield even as the forces at the nonlinear response location increase with strain hardening.

5. Defines both ductile and strong connections can be either:

   a. Wet connections where reinforcement is spliced with mechanical couplers, welds, or lap splices (observing the restrictions regarding the location of splices given for monolithic concrete) and the connection is completed with grout, or

   b. Dry connections, which are defined as any connection that is not a wet connection.

6. Requires that ductile connections be either:

   a. Type Y, with a minimum ductility ratio of 4 and specific anchorage requirements, or
   b. Type Z, with a minimum ductility ratio of 8 and stronger anchorage requirements.

Many of these requirements have been adopted into the 2002 edition of ACI 318, but some differences remain. Where those differences are pertinent to the examples illustrated here, they are explained.

The examples in Sec. 7.1 illustrate the design of untopped and topped precast concrete floor and roof diaphragms of the five-story masonry buildings described in Sec. 9.2 of this volume of design examples. The two untopped precast concrete diaphragms of Sec. 7.1.1 show the requirements for Seismic Design Categories B and C using 8-in.-thick hollow core precast, prestressed concrete planks. Sec. 7.1.2 shows the same precast plank with a 2 ½ in.-thick composite lightweight concrete topping for the five-story masonry building in Seismic Design Category D described in Sec. 9.2. Although untopped diaphragms are commonly used in regions of low seismic hazard, the only place they are addressed in the *Provisions* is the Appendix to Chapter 9. The reader should bear in mind that the appendices of the *Provisions* are prepared for trial use and comment, and future changes should be expected.

The example in Sec. 7.2 illustrates the design of an ordinary precast concrete shear wall building in a region of low or moderate seismicity, which is where most precast concrete seismic-force-resisting systems are constructed. The precast concrete walls in this example resist the seismic forces for a three-story office building, located in southern New England (Seismic Design Category B). There are very few seismic requirements for such walls in the *Provisions*. One such requirement qualifies is that overturning connections qualify as the newly defined Type Y or Z. ACI 318-02 identifies this system as an "intermediate precast concrete shear wall" and does not specifically define the Type Y or Z connections. Given the brief nature of the requirements in both the *Provisions* and ACI 318, the authors offer some interpretation. This example identifies points of yielding for the system and connection features that are required to maintain stable cyclic behavior for yielding.

The example in Sec. 7.3 illustrates the design of a special precast concrete shear wall for a single-story industrial warehouse building in the Los Angeles. For buildings in Seismic Design Category D, *Provisions* Sec. 9.1.1.12 [9.2.2.4] requires that the precast seismic-force-resisting system emulate the behavior of monolithic reinforced concrete construction or that the system's cyclic capacity be demonstrated by testing. The *Provisions* describes methods specifically intended to emulate the behavior of monolithic construction, and dry connections are permitted. Sec. 7.3 presents an interpretation of monolithic emulation of precast shear wall panels with ductile, dry connections. Whether this connection would qualify under ACI 318-02 is a matter of interpretation. The design is computed using the *Provisions* rules for monolithic emulation; however, the system probably would behave more like a jointed precast system. Additional clarity in the definition and application of design provisions of such precast systems is needed.

Tilt-up concrete wall buildings in all seismic zones have long been designed using the precast wall panels as shear walls in the seismic-force-resisting system. Such designs have usually been performed using design force coefficients and strength limits as if the precast walls emulated the performance of cast-in-place reinforced concrete shear walls, which they usually do not. In tilt-up buildings subject to strong ground shaking, the in-plane performance of the precast panels has rarely been a problem, primarily because there has been little demand for post-elastic performance in that direction. Conventional tilt-up buildings may deserve a unique treatment for seismic-resistant design, and they are not the subject of any of the examples in this chapter, although tilt-up panels with large height-to-width ratios could behave in the fashion described in design example 7.3.

In addition to the *Provisions*, the following documents are either referred to directly or are useful design aids for precast concrete construction:

ACI 318-99	American Concrete Institute. 1999. *Building Code Requirements and Commentary for Structural Concrete*.
ACI 318-02	American Concrete Institute. 2002. *Building Code Requirements and Commentary for Structural Concrete*.
AISC LRFD	American Institute of Steel Construction. 2002. *Manual of Steel Construction, Load & Resistance Factor Design,* Third Edition.
ASCE 7	American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures.*
Hawkins	Hawkins, Neil M., and S. K. Ghosh. 2000. "Proposed Revisions to 1997 NEHRP Recommended Provisions for Seismic Regulations for Precast Concrete Structures, Parts 1, 2, and 3." *PCI Journal*, Vol. 45, No. 3 (May-June), No. 5 (Sept.-Oct.), and No. 6 (Nov.-Dec.).
Moustafa	Moustafa, Saad E. 1981 and 1982. "Effectiveness of Shear-Friction Reinforcement in Shear Diaphragm Capacity of Hollow-Core Slabs." *PCI Journal*, Vol. 26, No. 1 (Jan.-Feb. 1981) and the discussion contained in *PCI Journal*, Vol. 27, No. 3 (May-June 1982).
PCI Handbook	Precast/Prestressed Concrete Institute. 1999. *PCI Design Handbook,* Fifth Edition.
PCI Details	Precast/Prestressed Concrete Institute. 1988. *Design and Typical Details of Connections for Precast and Prestressed Concrete,* Second Edition.
SEAA Hollow Core	Structural Engineers Association of Arizona, Central Chapter. *Design and Detailing of Untopped Hollow-Core Slab Systems for Diaphragm Shear*.

The following style is used when referring to a section of ACI 318 for which a change or insertion is proposed by the *Provisions*: *Provisions* Sec. xxx (ACI Sec. yyy) where "xxx" is the section in the *Provisions* and "yyy" is the section proposed for insertion into ACI 318-99.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made for the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformatting of all chapters for the 2003 *Provisions*) and substantive technical changes to the *Provisions* and its primary reference documents. Although the general conepts of the changes are described, the design examples and calculations have not been revised to reflect the changes made for the 2003 *Provisions*.

The most significant change related to precast concrete in the 2003 *Provisions* is that precast shear wall systems are now recognized separately from cast-in-place systems. The 2003 *Provisions* recognizes ordinary and intermediate precast concrete shear walls. The design of ordinary precast shear walls is based on ACI 318-02 excluding Chapter 21 and the design of intermediate shear walls is based on ACI 318-02 Sec. 21.13 (with limited modifications in Chapter 9 of the 2003 *Provisions*). The 2003 *Provisions* does not distinguish between precast and cast-in-place concrete for special shear walls. Special precast shear walls either need to satisfy the design requirements for special cast-in-place concrete shear walls

(ACI 318-02 Sec. 21.7) or most be substantiated using experimental evidence and analysis (2003 *Provisions* Sec. 9.2.2.4 and 9.6). Many of the design provisions for precast shear walls in the 2000 *Provisions* have been removed, and the requirements in ACI 318-02 are in some ways less specific. Where this occurs, the 2000 *Provisions* references in this chapter are simply annotated as "[not applicable in the 2003 *Provisions*]." Commentary on how the specific design provision was incorporated into ACI 318-02 is included where appropriate.

Some general technical changes for the 2003 *Provisions* that relate to the calculations and/or designs in this chapter include updated seismic hazard maps, revisions to the redundancy requirements, and revisions to the minimum base shear equation. Where they affect the design examples in the chapter, other significant changes for the 2003 *Provisions* and primary reference documents are noted. However, some minor changes may not be noted.

7.1 HORIZONTAL DIAPHRAGMS

Structural diaphragms are horizontal or nearly horizontal elements, such as floors and roofs, that transfer seismic inertial forces to the vertical seismic-force-resisting members. Precast concrete diaphragms may be constructed using topped or untopped precast elements depending on the Seismic Design Category of the building. Reinforced concrete diaphragms constructed using untopped precast concrete elements are addressed in the Appendix to Chapter 9 of the *Provisions*. Topped precast concrete elements, which act compositely or noncompositely for gravity loads, are designed using the requirements of ACI 318-99 Sec. 21.7 [ACI 318-02 Sec. 21.9].

7.1.1 Untopped Precast Concrete Units for Five-Story Masonry Buildings Located in Birmingham, Alabama, and New York, New York

This example illustrates floor and roof diaphragm design for the five-story masonry buildings located in Birmingham, Alabama, on soft rock (Seismic Design Category B) and in New York, New York (Seismic Design Category C). The example in Sec. 9.2 provides design parameters used in this example. The floors and roofs of these buildings are to be untopped 8-in.-thick hollow core precast, prestressed concrete plank. Figure 9.2-1 shows the typical floor plan of the diaphragms.

7.1.1.1 General Design Requirements

In accordance with the *Provisions* and ACI 318, untopped precast diaphragms are permitted only in Seismic Design Categories A through C. The Appendix to Chapter 9 provides design provisions for untopped precast concrete diaphragms without limits as to the Seismic Design Category. Diaphragms with untopped precast elements are designed to remain elastic, and connections are designed for limited ductility. No out-of-plane offsets in vertical seismic-force-resisting members (Type 4 plan irregularities) are permitted with untopped diaphragms. Static rational models are used to determine shears and moments on joints as well as shear and tension/compression forces on connections. Dynamic modeling of seismic response is not required.

The design method used here is that proposed by Moustafa. This method makes use of the shear friction provisions of ACI 318 with the friction coefficient, μ, being equal to 1.0. To use $\mu = 1.0$, ACI 318 requires grout or concrete placed against hardened concrete to have clean, laitance free, and intentionally roughened surfaces with a total amplitude of about 1/4 in. (peak to valley). Roughness for formed edges is provided either by sawtooth keys along the length of the plank or by hand roughening with chipping hammers. Details from the SEAA Hollow Core reference are used to develop the connection details.

The terminology used is defined in ACI 318 Chapter 21 and *Provisions* Chapter 9. These two sources occasionally conflict (such as the symbol μ used above), but the source is clear from the context of the discussion. Other definitions (e.g., chord elements) are provided as needed for clarity in this example.

7.1.1.2 General In-Plane Seismic Design Forces for Untopped Diaphragms

The in-plane diaphragm seismic design force (F'_{px}) for untopped precast concrete in *Provisions* Sec. 9A.3.3 [A9.2.2] "shall not be less than the forcee calculated from either of the following two criteria:"

1. $\rho\Omega_0 F_{px}$ but not less than $\rho\Omega_0 C_s w_{px}$ where
   F_{px} is calculated from *Provisions* Eq. 5.2.6.4.4[1] [4.6-3], which also bounds F_{px} to be not less than $0.2S_{DS}Iw_{px}$ and not more than $0.4S_{DS}Iw_{px}$. This equation normally is specified for Seismic Design

[1]Note that this equation is incorrectly numbered as 5.2.5.4 in the first printing of the 2000 *Provisions*.

Categories D and higher; it is intended in the *Provisions* Appendix to Chapter 9 that the same equation be used for untopped diaphragms in Seismic Design Categories B and C.

ρ is the reliability factor, which is 1.0 for Seismic Design Categories A through C per *Provisions* Sec. 5.2.4.1 [4.3.3.1].

Ω_0 is the overstrength factor (*Provisions* Table 5.2.2 [4.3-1])

C_s is the seismic response coefficient (*Provisions* Sec. 5.4.1.1 [5.2.1.1])

w_{px} is the weight tributary to the diaphragm at Level x

S_{DS} is the spectral response acceleration parameter at short periods (*Provisions* Sec. 4.1.2 [3.3.3])

I is the occupancy importance factor (*Provisions* Sec. 1.4 [1.3])

2. 1.25 times the shear force to cause yielding of the vertical seismic-force-resisting system.

   For the five-story masonry buildings of this example, the shear force to cause yielding is first estimated to be that force associated with the development of the nominal bending strength of the shear walls at their base. This approach to yielding uses the first mode force distribution along the height of the building and basic pushover analysis concepts, which can be approximated as:

   $F'_{px} = 1.25KF_{px}^*$ where

   K is the ratio of the yield strength in bending to the demand, M_y/M_u. (Note that $\phi = 1.0$)

   F_{px}^* is the seismic force at each level for the diaphragm as defined above by *Provisions* Eq. 5.2.6.4.4 [4.6-2] and not limited by the minima and maxima for that equation.

This requirement is different from similar requirements elsewhere in the *Provisions*. For components thought likely to behave in a brittle fashion, the designer is required to apply the overstrength factor and then given an option to check the *maximum* force that can be delivered by the remainder of the structural system to the element in question. The maximum force would normally be computed from a plastic mechanism analysis. If the option is exercised, the designer can then use the smaller of the two forces. Here the *Provisions* requires the designer to compute both an overstrength level force and a yield level force and then use the larger. This appears to conflict with the *Commentary*.

For Seismic Design Categories B and C, *Provisions* Sec. 5.2.6.2.6 [4.6.1.9] defines a minimum diaphragm seismic design force that will always be less than the forces computed above.

For Seismic Design Category C, *Provisions* Sec. 5.2.6.3.1 [4.6.2.2] requires that collector elements, collector splices, and collector connections to the vertical seismic-force-resisting members be designed in accordance with *Provisions* Sec. 5.2.7.1 [4.2.2.2], which places the overstrength factor on horizontal seismic forces and combines the horizontal and vertical seismic forces with the effects of gravity forces. Because vertical forces do not normally affect diaphragm collector elements, splices, and connections, the authors believe that *Provisions* Sec. 5.2.7.1 [4.2.2.2] is satisfied by the requirements of *Provisions* Sec. 9A.3.3 [A9.2.2], which requires use of the overstrength factor.

Parameters from the example in Sec. 9.2 used to calculate in-plane seismic design forces for the diaphragms are provided in Table 7.1-1.

Table 7.1-1 Design Parameters from Example 9.2

Design Parameter	Birmingham 1	New York City
ρ	1.0	1.0
Ω_o	2.5	2.5
C_s	0.12	0.156
w_i (roof)	861 kips	869 kips
w_i (floor)	963 kips	978 kips
S_{DS}	0.24	0.39
I	1.0	1.0

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

The *Provisions* Appendix to Chapter 9 does not give the option of using the overstrength factor Ω_0 to estimate the yield of the vertical system, so M_n for the wall is computed from the axial load moment interaction diagram data developed in Sec. 9.2. The shape of the interaction diagram between the balanced point and pure bending is far enough from a straight line (see Figure 9.2-6) in the region of interest that simply interpolating between the points for pure bending and balanced conditions is unacceptably unconservative for this particular check. An intermediate point on the interaction diagram was computed for each wall in Sec. 9.2, and that point is utilized here. Yielding begins before the nominal bending capacity is reached, particularly when the reinforcement is distributed uniformly along the wall rather than being concentrated at the ends of the wall. For lightly reinforced walls with distributed reinforcement and with axial loads about one-third of the balanced load, such as these, the yield moment is on the order of 90 to 95 percent of the nominal capacity. It is feasible to compute the moment at which the extreme bar yields, but that does not appear necessary for design. A simple factor of 0.95 was applied to the nominal capacity here. Thus, Table 7.1-2 shows the load information from Sec. 9.2 (the final numbers in this section may have changed, because this example was completed first).

The factor K is large primarily due to consideration of axial load. The strength for design is controlled by minimum axial load, whereas K is maximum for the maximum axial load, which includes some live load and a vertical acceleration on dead load.

Table 7.1-2 Shear Wall Overstrength

	Birmingham 1	New York City
Pure Bending, M_{n0}	963 ft-kips	1,723 ft-kips
Intermediate Load, M_{nB}	5,355 ft-kips	6,229 ft-kips
Intermediate Load, P_{nB}	335 kips	363 kips
Maximum Design Load, P_u	315 kips	327 kips
Interpolated M_n	5,092 ft-kips	5,782 ft-kips
Approximate M_y	4,837 ft-kips	5,493 ft-kips
Design M_u	2,640 ft-kips	3,483 ft-kips
Factor $K = M_y/M_u$	1.83	1.58

7.1.1.3 Diaphragm Forces for Birmingham Building 1

The weight tributary to the roof and floor diaphragms (w_{px}) is the total story weight (w_i) at Level *i* minus the weight of the walls parallel to the direction of loading.

Compute diaphragm weight (w_{px}) for the roof and floor as follows:

Roof

Total weight	=	861 kips
Walls parallel to force = (45 psf)(277 ft)(8.67 ft/2)	=	-54 kips
w_{px}	=	807 kips

Floors

Total weight	=	963 kips
Walls parallel to force = (45 psf)(277 ft)(8.67 ft)	=	-108 kips
w_{px}	=	855 kips

Compute diaphragm demands in accordance with *Provisions* Eq. 5.2.6.4.4 [4.6.3.4]:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

Calculations for F_{px} are provided in Table 7.1-3.

Table 7.1-3 Birmingham 1 F_{px} Calculations

Level	w_i (kips)	$\sum_{i=x}^{n} w_i$ (kips)	F_i (kips)	$\sum_{i=x}^{n} F_i = V_i$ (kips)	w_{px} (kips)	F_{px} (kips)
Roof	861	861	175	175	807	164
4	963	1,820	156	331	855	155
3	963	2,790	117	448	855	137
2	963	3,750	78	527	855	120
1	963	4,710	39	566	855	103

1.0 kip = 4.45 kN.

The values for F_i and V_i used in Table 7.1-3 are listed in Table 9.2-2.

The minimum value of $F_{px} = 0.2S_{DS}Iw_{px}$ = 0.2(0.24)1.0(807 kips) = 38.7 kips (at the roof)
= 0.2(0.24)1.0(855 kips) = 41.0 kips (at floors)

The maximum value of $F_{px} = 0.4S_{DS}Iw_{px}$ = 2(38.7 kips) = 77.5 kips (at the roof)
= 2(41.0 kips) = 82.1 kips (at floors)

Note that F_{px} by Table 7.1-3 is substantially larger than the maximum F_{px}. This is generally true at upper levels if the R factor is less than 5. The value of F_{px} used for the roof diaphragm is 82.1 kips. Compare this value to C_sw_{px} to determine the minimum diaphragm force for untopped diaphragms as indicated previously.

C_sw_{px} = 0.12(807 kips) = 96.8 kips (at the roof)
C_sw_{px} = 0.12(855 kips) = 103 kips (at the floors)

Since C_sw_{px} is larger than F_{px}, the controlling force is C_sw_{px}. Note that this will always be true when I = 1.0 and R is less than or equal to 2.5. Therefore, the diaphragm seismic design forces are as follows:

$F'_{px} = \rho\Omega_0C_sw_{px}$ = 1.0(2.5)(96.8 kips) = 242 kips (at the roof)
$F'_{px} = \rho\Omega_0C_sw_{px}$ = 1.0(2.5)(103 kips) = 256 kips (at the floors)

The second check on design force is based on yielding of the shear walls:

$F'_{px} = 1.25KF_{px}*$ = 1.25(1.85)164 kips = 379 kips (at the roof)
$F'_{px} = 1.25KF_{px}*$ = 1.25(1.85)155 kips = 358 kips (at the floors)

For this example, the force to yield the walls clearly controls the design. To simplify the design, the diaphragm design force used for all levels will be the maximum force at any level, 379 kips.

7.1.1.4 Diaphragm Forces for New York Building

The weight tributary to the roof and floor diaphragms (w_{px}) is the total story weight (w_i) at Level i minus the weight of the walls parallel to the force.

Compute diaphragm weight (w_{px}) for the roof and floor as follows:

Roof

Total weight	= 870 kips
Walls parallel to force = (48 psf)(277 ft)(8.67 ft/2)	= -58 kips
w_{px}	= 812 kips

Floors

Total weight	= 978 kips
Walls parallel to force = (48 psf)(277 ft)(8.67 ft)	= -115 kips
w_{px}	= 863 kips

Calculations for F_{px} using *Provisions* Eq. 5.2.6.4.4 [4.6.3.4] are not required for the first set of forces because $C_s w_{px}$ will be greater than or equal to the maximum value of $F_{px} = 0.4 S_{DS} I w_{px}$ when $I = 1.0$ and R is less than or equal to 2.5. Compute $C_s w_{px}$ as:

$C_s w_{px}$ = 0.156(812 kips) = 127 kips (at the roof)
$C_s w_{px}$ = 0.156(863 kips) = 135 kips (at the floors)

The diaphragm seismic design forces are:

$F'_{px} = \rho \Omega_0 C_s w_{px}$ = 1.0(2.5)(127 kips) = 318 kips (at the roof)
$F'_{px} = \rho \Omega_0 C_s w_{px}$ = 1.0(2.5)(135 kips) = 337 kips (at the floors)

Calculations for F_{px} using *Provisions* Eq. 5.2.6.4.4 [4.6.3.4] are required for the second check $F'_{px} = 1.25KF_{px}$. Following the same procedure as illustrated in the previous section, the maximum F_{px} is 214 kips at the roof. Thus,

$1.25KF_{px}*$ = 1.25(1.58)214 kips = 423 kips (at the roof)

To simplify the design, the diaphragm design force used for all levels will be the maximum force at any level. The diaphragm seismic design force (423 kips) is controlled by yielding at the base of the walls, just as with the Birmingham 1 building.

7.1.1.5 Static Analysis of Diaphragms

The balance of this example will use the controlling diaphragm seismic design force of 423 kips for the New York building. In the transverse direction, the loads will be distributed as shown in Figure 7.1-1.

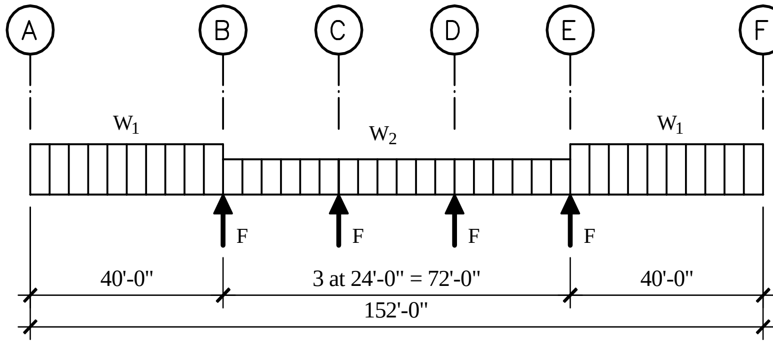


Figure 7.1-1 Diaphragm force distribution and analytical model (1.0 ft = 0.3048 m).

Assuming the four shear walls have the same stiffness and ignoring torsion, the diaphragm reactions at the transverse shear walls (*F* as shown in Figure 7.1-1) are computed as follows:

F = 423 kips/4 = 105.8 kips

The uniform diaphragm demands are proportional to the distributed weights of the diaphragm in different areas (see Figure 7.1-1).

W_1 = [67 psf(72 ft) + 48 psf(8.67 ft)4](423 kips / 863 kips) = 3,180 lb/ft
W_2 = [67 psf(72 ft)](423 kips / 863 kips) = 2,364 lb/ft

Figure 7.1-2 identifies critical regions of the diaphragm to be considered in this design. These regions are:

Joint 1 – maximum transverse shear parallel to the panels at panel-to-panel joints

Joint 2 – maximum transverse shear parallel to the panels at the panel-to-wall joint

Joint 3 – maximum transverse moment and chord force

Joint 4 – maximum longitudinal shear perpendicular to the panels at the panel-to-wall connection (exterior longitudinal walls) and anchorage of exterior masonry wall to the diaphragm for out-of-plane forces

Joint 5 – collector element and shear for the interior longitudinal walls

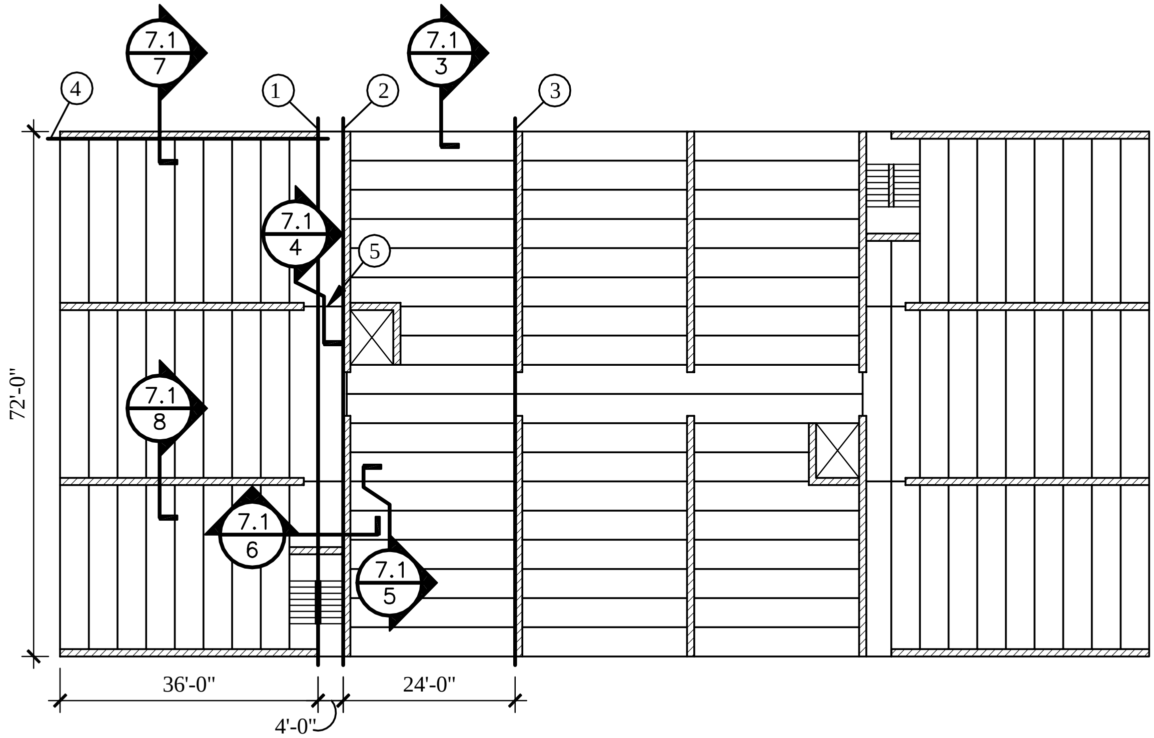


Figure 7.1-2 Diaphragm plan and critical design regions (1.0 ft = 0.3048 m).

Provisions Sec. 9.1.1.4 [not applicable in 2003 *Provisions*] defines a chord amplification factor for diaphragms in structures having precast gravity-load systems. [The chord amplification factor has been dropped in the 2003 *Provisions* and does not occur in ASC 318-02. See the initial section of this chapter for additional discussion on changes for the 2003 *Provisions*.] This amplification factor appears to apply to buildings with vertical seismic-force-resisting members constructed of precast or monolithic concrete. Because these masonry wall buildings are similar to buildings with concrete walls, this amplification factor has been included in calculating the chord forces. The amplification factor is:

$$b_d \frac{\left[1+0.4\left(\frac{L_{eff}}{b_d}\right)^2\right]}{12h_s} \geq 1.0$$

where

L_{eff} = length of the diaphragm between inflection points. Since the diaphragms have no infection points, twice the length of the 40-ft-long cantilevers is used for L_{eff} = 80 ft
h_s = story height = 8.67 ft
b_d = diaphragm width = 72 ft

The amplification factor = $(72)\dfrac{\left[1+0.4\left(\frac{80}{72}\right)^2\right]}{12(8.67)}$ = 1.03

Joint forces are:

Joint 1 – Transverse forces

Shear, V_{u1} = 3.18 kips/ft (36 ft) = 114.5 kips
Moment, M_{u1} = 114.5 kips (36 ft/2) = 2,061 ft-kips
Chord tension force, $T_{u1} = M/d$ = 1.03(2,061 ft-kips/71 ft) = 29.9 kips

Joint 2 – Transverse forces

Shear, V_{u2} = 3.18 kips/ft (40 ft) = 127 kips
Moment, M_{u2} = 127 kips (40 ft/2) = 2,540 ft-kips
Chord tension force, $T_{u2} = M/d$ = 1.03(2,540 ft-kips/71 ft) = 36.9 kips

Joint 3 – Transverse forces

Shear, V_{u3} = 127 kips + 2.36 kips/ft (24 ft) - 105.8 kips = 78.1 kips
Moment, M_{u3} = 127 kips (44 ft) + 56.7 kips (12 ft) - 105.8 kips (24 ft) = 3,738 ft-kips
Chord tension force, $T_{u3} = M/d$ = 1.03(3,738 ft-kips/71 ft) = 54.2 kips

Joint 4 – Longitudinal forces

Wall Force, F = 423 kips/8 = 52.9 kips
Wall shear along wall length, V_{u4} = 52.9 kips (36 ft)/(152 ft /2) = 25.0 kips
Collector force at wall end, $T_{u4} = C_{u4}$ = 52.9 kips - 25.0 kips = 27.9 kips

Joint 4 – Out-of-plane forces

The *Provisions* have several requirements for out-of-plane forces. None are unique to precast diaphragms and all are less than the requirements in ACI 318 for precast construction regardless of seismic considerations. Assuming the planks are similar to beams and comply with the minimum requirements of *Provisions* Sec. 5.2.6.1.1 [4.6.1.1] (Seismic Design Category A and greater) [In the 2003 *Provisions*, all requirements for Seismic Design Category A are in Sec. 1.5 but they generally are the same as those in the 2000 *Provisions*. The design and detailing requirements in 2003 *Provisions* Sec. 4.6 apply to Seismic Design Category B and greater], the required out-of-plane horizontal force is:

$0.05(D + L)_{plank}$ = 0.05(67 psf + 40 psf)(24 ft/2) = 64.2 plf

According to *Provisions* Sec. 5.2.6.1.2 [4.6.1.2] (Seismic Design Category A and greater), the minimum anchorage for masonry walls is:

$F_p = 400(S_{DS})I = 400(0.39)1.0$ = 156 plf

According to *Provisions* Sec. 5.2.6.2.7 [4.6.1.3] (Seismic Design Category B and greater), bearing wall anchorage shall be designed for a force computed as:

$0.4(S_{DS})W_{wall}$ = 0.4(0.39)(48 psf)(8.67 ft) = 64.9 plf

Provisions Sec. 5.2.6.3.2 [4.6.2.1] (Seismic Design Category C and greater) requires masonry wall anchorage to flexible diaphragms to be designed for a larger force. This diaphragm is considered rigid with respect to the walls, and considering that it is designed to avoid yield under the loads that will yield the walls, this is a reasonable assumption.

$F_p = 1.2(S_{DS})Iw_p = 1.2(0.39)1.0[(48\text{ psf})(8.67\text{ ft})]$ = 195 plf

[In the 2003 *Provisions*, Eq. 4.6-1 in Sec. 4.6.2.1 has been changed to $0.85S_{DS}IW_p$.]

The force requirements in ACI 318 Sec. 16.5 will be described later.

Joint 5 – Longitudinal forces

Wall force, F = 423 kips/8 = 52.9 kips
Wall shear along each side of wall, V_{u4} = 52.9 kips [2(36 ft)/152 ft]/2 = 12.5 kips
Collector force at wall end, $T_{u5} = C_{u5}$ = 52.9 kips - 25.0 kips = 27.9 kips

ACI 318 Sec. 16.5 also has minimum connection force requirements for structural integrity of precast concrete bearing wall building construction. For buildings over two stories there are force requirements for horizontal and vertical members. This building has no vertical precast members. However, ACI 318 Sec. 16.5.1 specifies that the strengths ". . . for structural integrity shall apply to all precast concrete structures." This is interpreted to apply to the precast elements of this masonry bearing wall structure. The horizontal tie force requirements are:

1. 1,500 lb/ft parallel and perpendicular to the span of the floor members. The maximum spacing of ties parallel to the span is 10 ft. The maximum spacing of ties perpendicular to the span is the distance between supporting walls or beams.

2. 16,000 lb parallel to the perimeter of a floor or roof located within 4 ft of the edge at all edges.

ACI's tie forces are far greater than the minimum tie forces given in the *Provisions* for beam supports and anchorage for of masonry walls. They do control some of the reinforcement provided, but most of the reinforcement is controlled by the computed connections for diaphragm action.

7.1.1.6 Diaphragm Design and Details

Before beginning the proportioning of reinforcement, a note about ACI's ϕ factors is necessary. The *Provisions* cites ASCE 7 for combination of seismic load effects with the effects of other loads. Both ASCE 7 and the *Provisions* make it clear that the appropriate ϕ factors within ACI 318 are those contained within Appendix C of ACI 318-99. These factors are about 10% less than the comparable factors within the main body of the standard. The 2002 edition of ACI 318 has placed the ASCE 7 load combinations within the main body of the standard and revised the ϕ factors accordingly. This example uses the ϕ factors given in the 2002 edition of ACI 318, which are the same as those given in Appendix C of the 1999 edition with one exception. Thus, the ϕ factors used here are:

Tension control (bending and ties) $\phi = 0.90$
Shear $\phi = 0.75$
Compression control in tied members $\phi = 0.65$.

The minimum tie force requirements given in ACI 318 Sec. 16.5 are specified as nominal values, meaning that $\phi = 1.00$ for those forces.

7.1.1.6.1 Design and Detailing at Joint 3

Joint 3 is designed first to check the requirements of *Provisions* Sec. 9A.3.9 [A9.2.4], which references ACI 318 Sec. 21.7.8.3 [21.9.8.3], which then refers to ACI 318 Sec. 21.7.5.3 [21.9.5.3]. This section provides requirements for transverse reinforcement in the chords of the diaphragm. The compressive stress in the chord is computed using the ultimate moment based on a linear elastic model and gross section properties. To determine the in-plane section modulus (S) of the diaphragm, an equivalent thickness (t) based on the cross sectional area is used for the hollow core precast units as follows.

t = area/width = 215/48 = 4.5 in.
$S = td^2/6$

Chord compressive stress is computed as:

$$M_u/S = 6M_{u3}/td^2 = 6(3{,}738 \times 12)/(4.5)(72 \times 12)^2 = 80.1 \text{ psi}$$

The design 28-day compressive strength of the grout is 4,000 psi. Since the chord compressive stress is less than 0.2 f_c' = 0.2(4,000) = 800 psi, the transverse reinforcement indicated in ACI 318 Sec. 21.4.4.1 through 21.4.4.3 is not required.

Compute the required amount of chord reinforcement as:

Chord reinforcement, $A_{s3} = T_{u3}/\phi f_y$ = (54.2 kips)/[0.9(60 ksi)] = 1.00 in.^{2}

Use two #7 bars, A_s = 2(0.60) = 1.20 in.^{2} along the exterior edges (top and bottom of the plan in Figure 7.1- 2). Require cover for chord bars and spacing between bars at splices and anchorage zones by ACI 318 Sec. 21.7.8.3 [21.9.8.3].

Minimum cover = 2.5(7/8) = 2.19 in., but not less than 2.0 in.
Minimum spacing = 3(7/8) = 2.63 in., but not less than 1-1/2 in.

Figure 7.1-3 shows the chord element at the exterior edges of the diaphragm. The chord bars extend along the length of the exterior longitudinal walls and act as collectors for these walls in the longitudinal direction (see Joint 4 collector reinforcement and Figure 7.1-7).

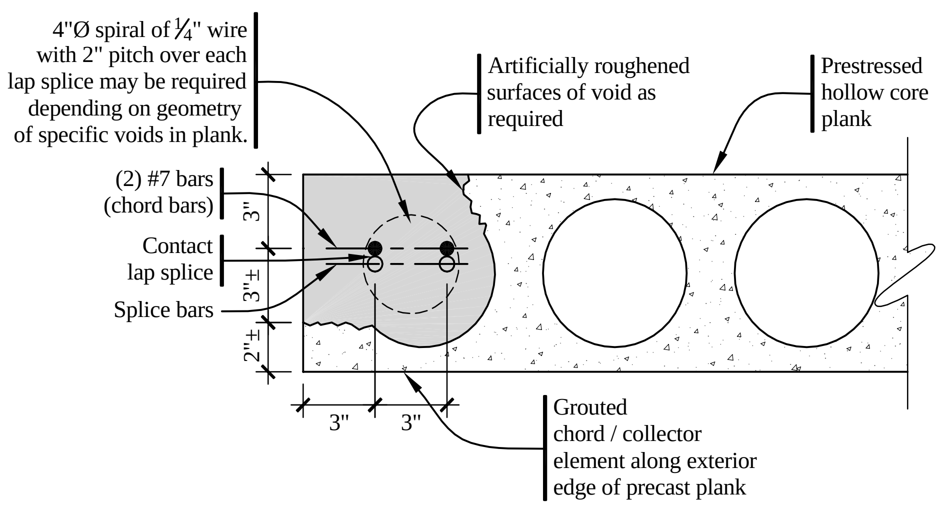


Figure 7.1-3 Joint 3 chord reinforcement at the exterior edge (1.0 in. = 25.4 mm).

Joint 3 must also be checked for the minimum ACI tie forces. The chord reinforcement obviously exceeds the 16 kip perimeter force requirement. The 1.5 kips per foot requirement requires a 6 kip tie at each joint between the planks, which is satisfied with a #3 bar in each joint (0.11 in.^{2} at 60 ksi = 6.6 kips). This bar is required at all bearing walls and is shown in subsequent details.

7.1.1.6.2 Joint 1 Design and Detailing

The design must provide sufficient reinforcement for chord forces as well as shear friction connection forces as follows:

Chord reinforcement, $A_{s1} = T_{u1}/\phi f_y$ = (29.9 kips)/[0.9(60 ksi)] = 0.55 in.^{2} (collector force from Joint 4 calculations at 27.9 kips is not directly additive).

Shear friction reinforcement, $A_{vf1} = V_{u1}/\phi\mu f_y$ = (114.5 kips)/[(0.75)(1.0)(60 ksi)] = 2.54 in.^{2}

Total reinforcement required = 2(0.55 in.^{2}) + 2.54 in.^{2} = 3.65 in.^{2}

ACI tie force = (3 kips/ft)(72 ft) = 216 kips; reinforcement = (216 kips)/(60 ksi) = 3.60 in.^{2}

Provide four #7 bars (two at each of the outside edges) plus four #6 bars (two each at the interior joint at the ends of the plank) for a total area of reinforcement of 4(0.60 in^2) + 4(0.44 in.^{2}) = 4.16 in.^{2}

Because the interior joint reinforcement acts as the collector reinforcement in the longitudinal direction for the interior longitudinal walls, the cover and spacing of the two # 6 bars in the interior joints will be provided to meet the requirements of ACI 318 Sec. 21.7.8.3 [21.9.8.3]:

Minimum cover = 2.5(6/8) = 1.88 in., but not less than 2.0 in.
Minimum spacing = 3(6/8) = 2.25 in., but not less than 1-1/2 in.

Figure 7.1-4 shows the reinforcement in the interior joints at the ends of the plank, which is also the collector reinforcement for the interior longitudinal walls (Joint 5). The two #6 bars extend along the length of the interior longitudinal walls as shown in Figure 7.1-8.

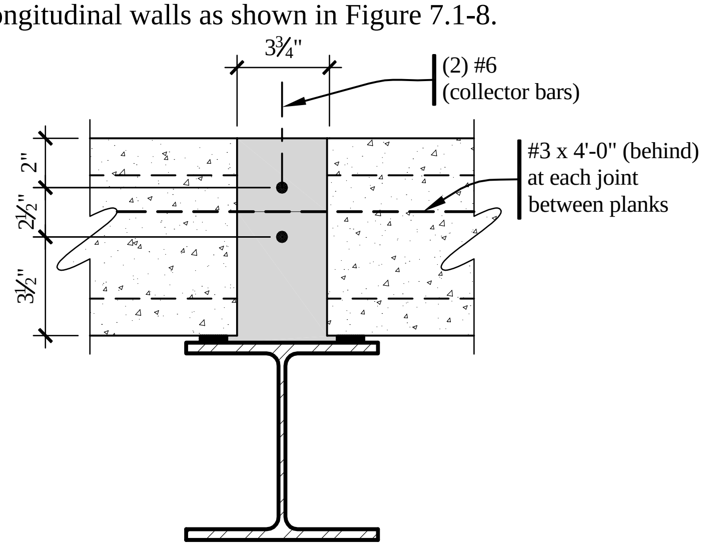


Figure 7.1-4 Interior joint reinforcement at the ends of plank and the collector reinforcement at the end of the interior longitudinal walls - Joints 1 and 5 (1.0 in. = 25.4 mm).

Figure 7.1-5 shows the extension of the two #6 bars of Figure 7.1-4 into the region where the plank is parallel to the bars. The bars will need to be extended the full length of the diaphragm unless supplemental plank reinforcement is provided. This detail makes use of this supplement plank reinforcement (two #6 bars or an equal area of strand per ACI 318-99 Sec. 21.7.5.2 [21.9.5.2]) and shows the bars anchored at each end of the plank. The anchorage length of the #6 bars is calculated using ACI 318-99 Sec. 21.7.5.4 [21.9.5.4] which references ACI 318 Sec. 21.5.4:

$$l_d = 1.6(2.5)\left(\frac{f_y d_b}{65\sqrt{f_c'}}\right) = 1.6(2.5)\left[\frac{60{,}000(d_b)}{65\sqrt{4{,}000}}\right] = 58.2 d_b$$

The 2.5 factor is for the difference between straight and hooked bars, and the 1.6 factor applies when the development length is not within a confined core. Using #6 bars, the required l_d = 58.2(0.75 in.) = 43.7 in. Therefore, use l_d = 4 ft, which is the width of the plank.

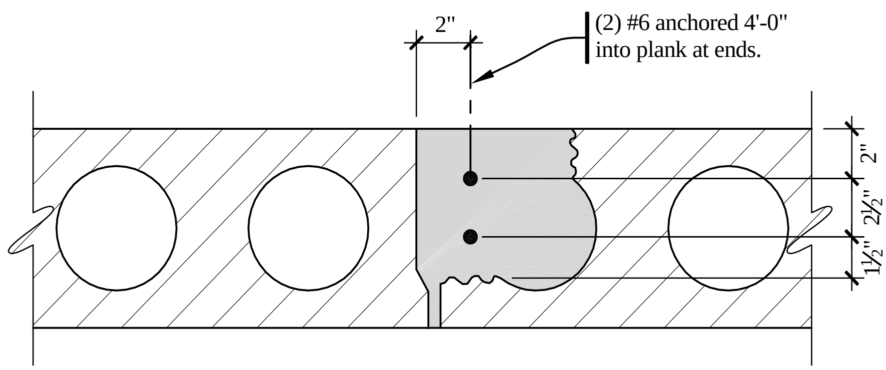


Figure 7.1-5 Anchorage region of shear reinforcement for Joint 1 and collector reinforcement for Joint 5 (1.0 in. = 25.4 mm).

7.1.1.6.3 Joint 2 Design and Detailing

The chord design is similar to the previous calculations:

Chord reinforcement, $A_{s2} = T_{u2}/\phi f_y$ = (36.9 kips)/[0.9(60 ksi)] = 0.68 in.^{2}

The shear force may be reduced along Joint 2 by the shear friction resistance provided by the supplemental chord reinforcement ($2A_{chord} - A_{s2}$) and by the four #6 bars projecting from the interior longitudinal walls across this joint. The supplemental chord bars, which are located at the end of the walls, are conservatively excluded here. The shear force along the outer joint of the wall where the plank is parallel to the wall is modified as:

$$V_{u2}^{Mod} = V_{u2} - \left[\phi f_y \mu\left(A_{4\#6}\right)\right] = 127 - \left[0.75(60)(1.0)(4 \times 0.44)\right] = 47.8 \text{ kips}$$

This force must be transferred from the planks to the wall. Using the arrangement shown in Figure 7.1-6, the required shear friction reinforcement (A_{vf2}) is computed as:

$$A_{vf2} = \frac{V_{u2}^{Mod}}{\phi f_y\left(\mu \sin \alpha_f + \cos \alpha_f\right)} = \frac{47.8}{0.75(60)(1.0 \sin 26.6° + \cos 26.6°)} = 0.79 \text{ in.}^2$$

Use two #3 bars placed at 26.6 degrees (2-to-1 slope) across the joint at 4 ft from the ends of the plank and at 8 ft on center (three sets per plank). The angle (α_f) used above provides development of the #3 bars while limiting the grouting to the outside core of the plank. The total shear reinforcement provided is 9(0.11 in.^{2}) = 0.99 in.^{2}

The shear force between the other face of this wall and the diaphragm is:

$$V_{u2} - F = 127 - 106 = 21 \text{ kips}$$

The shear friction resistance provided by #3 bars in the grout key between each plank (provided for the 1.5 klf requirement of the ACI) is computed as:

$$\phi A_{vf} f_y \mu = (0.75)(10 \text{ bars})(0.11 \text{ in.}^2)(60 \text{ ksi})(1.0) = 49.5 \text{ kips}$$

The development length of the #3 and #4 bars will now be checked. For the 180 degree standard hook use ACI 318 Sec. 12.5, $l_{dh} = l_{hb}$ times the factors of ACI 318 Sec. 12.5.3, but not less than 8d$_b$ or 6 in. Side cover exceeds 2-1/2 in. and cover on the bar extension beyond the hook is provided by the grout and the planks, which is close enough to 2 in. to apply the 0.7 factor of ACI 318 Sec. 12.5.3.2. The continuous #5 provides transverse reinforcement, but it is not arranged to take advantage of ACI 318's 0.8 factor. For the #3 hook:

$$l_{dh} = \frac{0.7(1,200)d_b}{\sqrt{f_c'}} = \frac{0.7(1,200)0.375}{\sqrt{4,000}} = 4.95 \text{ in.} \qquad \text{(6" minimum)}$$

The available distance for the perpendicular hook is about 5-1/2 in. The bar will not be fully developed at the end of the plank because of the 6 in. minimum requirement. The full strength is not required for shear transfer. By inspection, the diagonal #3 hook will be developed in the wall as required for the computed diaphragm-to-shear-wall transfer. The straight end of the #3 will now be checked. The standard development length of ACI 318 Sec. 12.2 is used for l_d.

$$l_d = \frac{f_y d_b}{25\sqrt{f_c'}} = \frac{60,000(0.375)}{25\sqrt{4,000}} = 14.2 \text{ in.}$$

Figure 7.1-6 shows the reinforcement along each side of the wall on Joint 2.

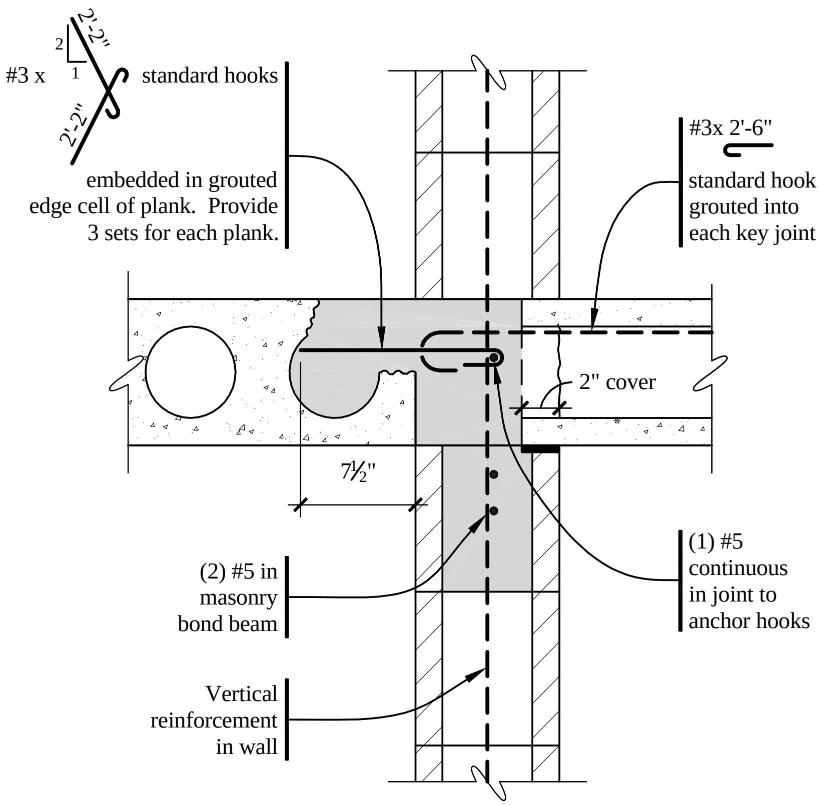


Figure 7.1-6 Joint 2 transverse wall joint reinforcement (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

7.1.1.6.4 Joint 4 Design and Detailing

The required shear friction reinforcement along the wall length is computed as:

$$A_{vf4} = V_{u4}/\phi\mu f_y = (25.0 \text{ kips})/[(0.75)(1.0)(60 \text{ ksi})] = 0.56 \text{ in.}^2$$

Based upon the ACI tie requirement, provide #3 bars at each plank-to-plank joint. For eight bars total, the area of reinforcement is 8(0.11) = 0.88 in.^{2}, which is more than sufficient even considering the marginal development length, which is less favorable at Joint 2. The bars are extended 2 ft into the grout key, which is more than the development length and equal to half the width of the plank.

The required collector reinforcement is computed as:

$$A_{s4} = T_{u4}/\phi f_y = (27.9 \text{ kips})/[0.9(60 \text{ ksi})] = 0.52 \text{ in.}^2$$

The two #7 bars, which are an extension of the transverse chord reinforcement, provide an area of reinforcement of 1.20 in.^{2}

The reinforcement required by the *Provisions* for out-of-plane force is (195 plf) is far less than the ACI 318 requirement.

Figure 7.1-7 shows this joint along the wall.

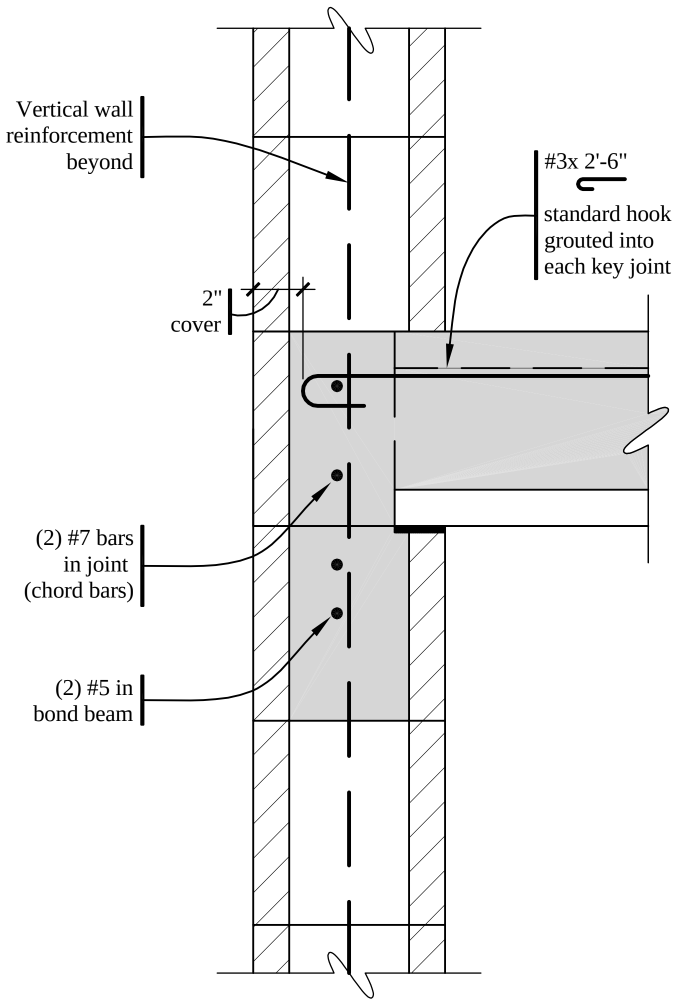


Figure 7.1-7 Joint 4 exterior longitudinal walls to diaphragm reinforcement and out-of-plane anchorage (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

7.1.1.6.5 Joint 5 Design and Detailing

The required shear friction reinforcement along the wall length is computed as:

$$A_{vf5} = V_{u5}/\phi\mu f_y = (12.5 \text{ kips})/[(0.75)(1.0)(60 \text{ ksi})] = 0.28 \text{ in.}^2$$

Provide #3 bars at each plank-to-plank joint for a total of 8 bars.

The required collector reinforcement is computed as:

$$A_{s5} = T_{u5}/\phi f_y = (27.9 \text{ kips})/[0.9(60 \text{ ksi})] = 0.52 \text{ in.}^2$$

Two #6 bars specified for the design of Joint 1 above provide an area of reinforcement of 0.88 in.^{2} Figure 7.1-8 shows this joint along the wall.

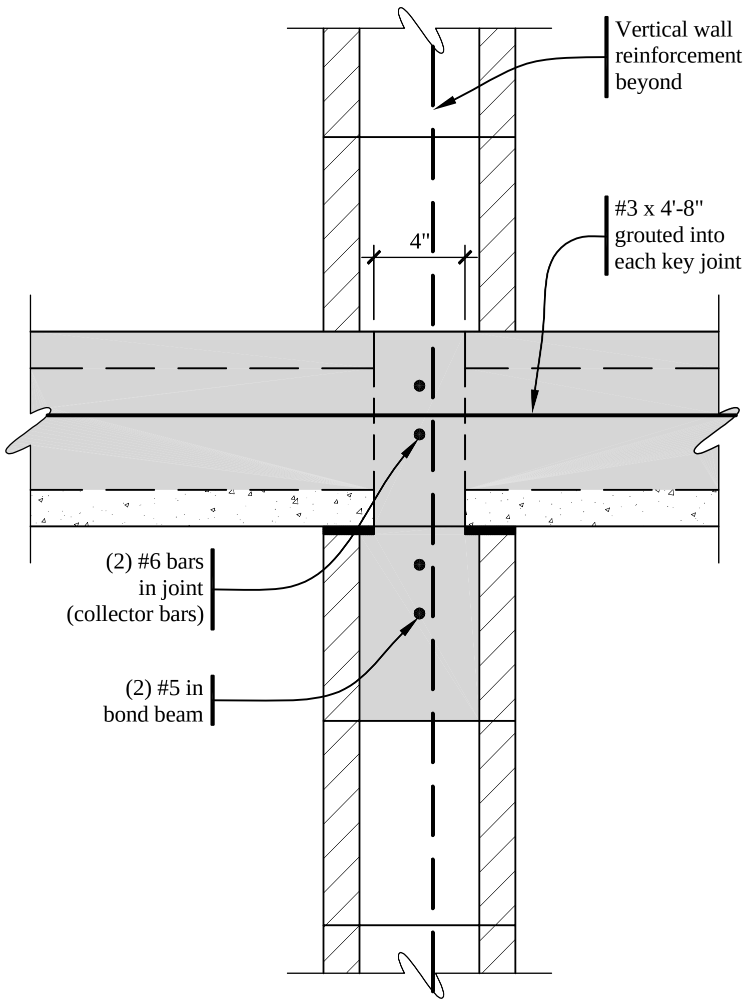


Figure 7.1-8 Wall-to-diaphragm reinforcement along interior longitudinal walls - Joint 5 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

7.1.2 Topped Precast Concrete Units for Five-Story Masonry Building, Los Angeles, California (see Sec. 9.2)

This design shows the floor and roof diaphragms using topped precast units in the five-story masonry building in Los Angeles, California. The topping thickness exceeds the minimum thickness of 2 in. as required for composite topping slabs by ACI 318 Sec. 21.7.4 [21.9.4]. The topping shall be lightweight concrete (weight = 115 pcf) with a 28-day compressive strength (f_c') of 4,000 psi and is to act compositely with the 8-in.-thick hollow-core precast, prestressed concrete plank. Design parameters are provided in Sec. 9.2. Figure 9.2-1 shows the typical floor and roof plan.

7.1.2.1 General Design Requirements

Topped diaphragms may be used in any Seismic Design Category. ACI 318 Sec. 21.7 [21.9]provides design provisions for topped precast concrete diaphragms. *Provisions* Sec. 5.2.6 [4.6] specifies the forces to be used in designing the diaphragms. The amplification factor of *Provisions* Sec. 9.1.1.4 [not applicable in the 2003 *Provisions*] is 1.03, the same as previously computed for the untopped diaphragm.

[As noted above, the chord amplification factor has been dropped for the 2003 *Provisions* and does not occur in ASC 318-02.]

7.1.2.2 General In-Plane Seismic Design Forces for Topped Diaphragms

The in-plane diaphragm seismic design force (F_{px}) is calculated using *Provisions* Eq. 5.2.6.4.4 [4.6-2] but must not be less than $0.2S_{DS}Iw_{px}$ and need not be more than $0.4S_{DS}Iw_{px}$. V_x must be added to F_{px} calculated using Eq. 5.2.6.4.4 [4.6-2] where:

w_{px} = the weight tributary to the diaphragm at Level x
S_{DS} = the spectral response acceleration parameter at short periods (*Provisions* Sec. 4.1.2 [3.3.5])
I = occupancy importance factor (*Provisions* Sec. 1.4 [1.3])
V_x = the portion of the seismic shear force required to be transferred to the components of the vertical seismic-force-resisting system due to offsets or changes in stiffness of the vertical resisting member at the diaphragm being designed.

For Seismic Design Category C and higher, *Provisions* Sec. 5.2.6.3.1 [4.6.2.2] requires that collector elements, collector splices, and collector connections to the vertical seismic-force-resisting members be designed in accordance with *Provisions* Sec. 5.2.7.1 [4.2.2.2], which combines the diaphragm forces times the overstrength factor (Ω_0) and the effects of gravity forces. The parameters from example in Sec. 9.2 used to calculate in-plane seismic design forces for the diaphragms are provided in Table 7.1-4.

Table 7.1-4 Design Parameters from Sec. 9.2

Design Parameter	Value
Ω_o	2.5
w_i (roof)	1,166 kips
w_i (floor)	1,302 kips
S_{DS}	1.0
I	1.0
Seismic Design Category	D

1.0 kip = 4.45 kN.

7.1.2.3 Diaphragm Forces

As indicated previously, the weight tributary to the roof and floor diaphragms (w_{px}) is the total story weight (w_i) at Level i minus the weight of the walls parallel to the force.

Compute diaphragm weight (w_{px}) for the roof and floor as:

Roof

Total weight	= 1,166 kips
Walls parallel to force = (60 psf)(277 ft)(8.67 ft/2)	= -72 kips
w_{px}	= 1,094 kips

Floors

Total weight	= 1,302 kips
Walls parallel to force = (60 psf)(277 ft)(8.67 ft)	= -144 kips
w_{px}	= 1,158 kips

Compute diaphragm demands in accordance with *Provisions* Eq. 5.2.6.4.4 [4.6-2]:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

Calculations for F_{px} are provided in Table 7.1-5. The values for F_i and V_i are listed in Table 9.2-17.

Table 7.1-5 F_{px} Calculations from Sec. 9.2

Level	w_i (kips)	$\sum_{i=x}^{n} w_i$ (kips)	F_i (kips)	$\sum_{i=x}^{n} F_i = V_i$ (kips)	w_{px} (kips)	F_{px} (kips)
Roof	1,166	1,166	564	564	1,094	529
4	1,302	2,468	504	1,068	1,158	501
3	1,302	3,770	378	1,446	1,158	444
2	1,302	5,072	252	1,698	1,158	387
1	1,302	6,384	126	1,824	1,158	331

1.0 kip = 4.45 kN.

The minimum value of $F_{px} = 0.2S_{DS}Iw_{px}$	= 0.2(1.0)1.0(1,094 kips)	= 219 kips (at the roof)
	= 0.2(1.0)1.0(1,158 kips)	= 232 kips (at floors)
The maximum value of $F_{px} = 0.4S_{DS}Iw_{px}$	= 2(219 kips)	= 438 kips (at the roof)
	= 2(232 kips)	= 463 kips (at floors)

The value of F_{px} used for design of the diaphragms is 463 kips, except for collector elements where forces will be computed below.

7.1.2.4 Static Analysis of Diaphragms

The seismic design force of 463 kips is distributed as in Sec. 7.1.1.6 (Figure 7.1-1 shows the distribution). The force is only 9.5 percent higher than that used to design the untopped diaphragm for the New York design due to the intent to prevent yielding in the untopped diaphragm. Figure 7.1-2 shows critical regions of the diaphragm to be considered in this design. Collector elements will be designed for 2.5 times the diaphragm force based on the overstrength factor (Ω_0).

Joint forces taken from Sec. 7.1.1.5 times 1.095 are as:

Joint 1 – Transverse forces

Shear, V_{u1} = 114.5 kips × 1.095	= 125 kips
Moment, M_{u1} = 2,061 ft-kips × 1.095	= 2,250 ft-kips
Chord tension force, $T_u1 = M/d$ = 1.03 × 2,250 ft-kips / 71 ft	= 32.6 kips

Joint 2 – Transverse forces

Shear, V_{u2} = 127 kips × 1.095	= 139 kips
Moment, M_{u2} = 2,540 ft-kips × 1.095	= 2,780 ft-kips
Chord tension force, $T_{u2} = M/d$ = 1.03 × 2,780 ft-kips / 71 ft	= 39.3 kips

Joint 3 – Transverse forces

Shear, V_{u3} = 78.1 kips × 1.095	= 85.5 kips
Moment, M_{u2} = 3,738 ft-kips × 1.095	= 4,090 ft-kips
Chord tension force, $T_{u3} = M/d$ = 1.03 × 4,090 ft-kips/71 ft	= 59.3 kips

Joint 4 – Longitudinal forces

Wall Force, F = 52.9 kips × 1.095	= 57.9 kips
Wall shear along wall length, V_{u4} =25 kips × 1.095	= 27.4 kips
Collector force at wall end, $\Omega_0 T_{u4}$ = 2.5(27.9 kips)(1.095)	= 76.4 kips

Out-of-Plane forces

Just as with the untopped diaphragm, the out-of-plane forces are controlled by ACI 318 Sec. 16.5, which requires horizontal ties of 1.5 kips per foot from floor to walls.

Joint 5 – Longitudinal forces

Wall Force, F = 463 kips / 8 walls	= 57.9 kips
Wall shear along each side of wall, V_{u4} = 12.5 kips × 1.095	= 13.7 kips
Collector force at wall end, $\Omega_0 T_{u4}$ = 2.5(27.9 kips)(1.095)	= 76.4 kips

7.1.2.5 Diaphragm Design and Details

7.1.2.5.1 Minimum Reinforcement for 2.5 in. Topping

ACI 318 Sec. 21.7.5.1 [21.9.5.1] references ACI 318 Sec. 7.12, which requires a minimum $A_s = 0.0018bd$ for welded wire fabric. For a 2.5 in. topping, the required A_s = 0.054 in.^{2}/ft. WWF 10×10 - W4.5×W4.5 provides 0.054 in.^{2}/ft. The minimum spacing of wires is 10 in. and the maximum spacing is 18 in. Note that the ACI 318 Sec. 7.12 limit on spacing of five times thickness is interpreted such that the topping thickness is not the pertinent thickness.

7.1.2.5.2 Boundary Members

Joint 3 has the maximum bending moment and is used to determine the boundary member reinforcement of the chord along the exterior edge. The need for transverse boundary member reinforcement is reviewed using ACI 318 Sec. 21.7.5.3 [21.9.5.3]. Calculate the compressive stress in the chord with the ultimate moment using a linear elastic model and gross section properties of the topping. It is conservative to ignore the precast units, but not necessary. As developed previously, the chord compressive stress is:

$$6M_{u3}/td^2 = 6(4{,}090 \times 12)/(2.5)(72 \times 12)^2 = 158 \text{ psi}$$

The chord compressive stress is less than $0.2f_c' = 0.2(4{,}000) = 800$ psi. Transverse reinforcement in the boundary member is not required.

The required chord reinforcement is:

$$A_{s3} = T_{u3}/\phi f_y = (59.3 \text{ kips})/[0.9(60 \text{ ksi})] = 1.10 \text{ in.}^2$$

7.1.2.5.3 Collectors

The design for Joint 4 collector reinforcement at the end of the exterior longitudinal walls and for Joint 5 at the interior longitudinal walls is the same.

$$A_{s4} = A_{s5} = \Omega_0 T_{u4}/\phi f_y = (76.4 \text{ kips})/[0.9(60 \text{ ksi})] = 1.41 \text{ in.}^2$$

Use two #8 bars ($A_s = 2 \times 0.79 = 1.58$ in.^{2}) along the exterior edges, along the length of the exterior longitudinal walls, and along the length of the interior longitudinal walls. Provide cover for chord and collector bars and spacing between bars per ACI 318 Sec. 21.7.8.3 [21.9.8.3].

Minimum cover = 2.5(8/8) = 2.5 in., but not less than 2.0 in.
Minimum spacing = 3(8/8) = 3.0 in., but not less than 1-1/2 in.

Figure 7.1-9 shows the diaphragm plan and section cuts of the details and Figure 7.1-10, the boundary member and chord/collector reinforcement along the edge. Given the close margin on cover, the transverse reinforcement at lap splices also is shown.

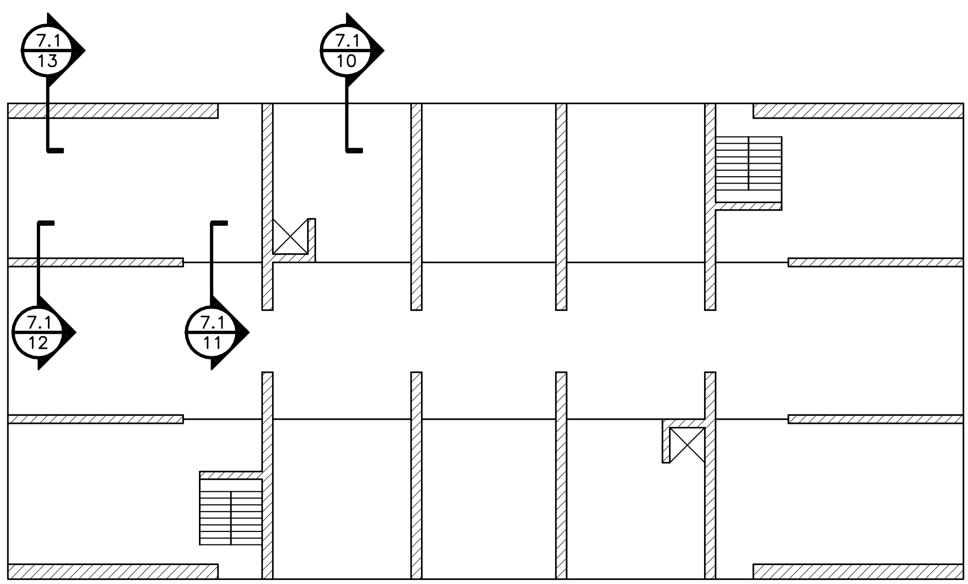


Figure 7.1-9 Diaphragm plan and section cuts.

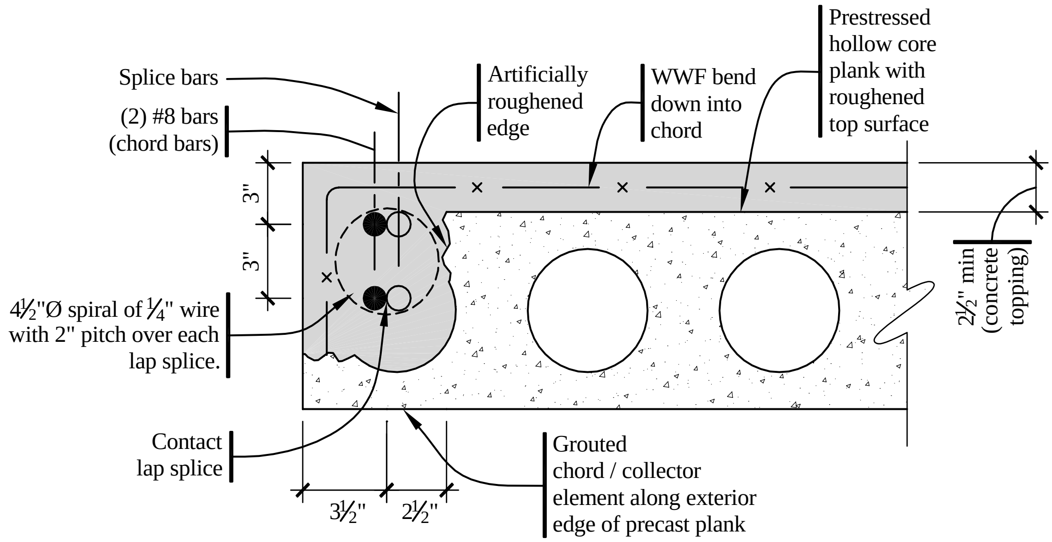


Figure 7.1-10 Boundary member, and chord and collector reinforcement (1.0 in. = 25.4 mm).

Figure 7.1-11 shows the collector reinforcement for the interior longitudinal walls. The side cover of 2-1/2 in. is provided by casting the topping into the cores and by the stems of the plank. A minimum space of 1 in. is provided between the plank stems and the sides of the bars.

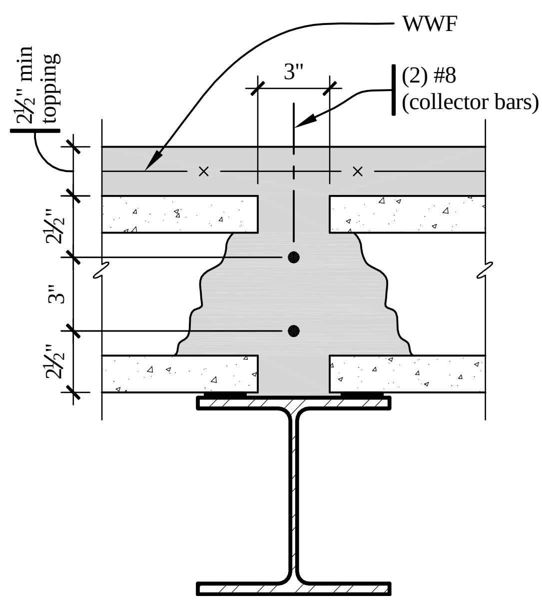


Figure 7.1-11 Collector reinforcement at the end of the interior longitudinal walls - Joint 5 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

7.1.2.5.4 Shear Resistance

Thin composite and noncomposite topping slabs on precast floor and roof members may not have reliable shear strength provided by the concrete. In accordance with ACI 318 Sec. 21.7.7.2 [21.9.7.2], all of the shear resistance must be provided by the reinforcement (that is, $V_c = 0$).

$$\phi V_n = \phi A_{cv}\rho_n f_y = 0.75(0.054 \text{ in.}^2/\text{ft})60 \text{ ksi} = 2.43 \text{ kips/ft}$$

The shear resistance in the transverse direction is:

$$2.43 \text{ kips/ft } (72 \text{ ft}) = 175 \text{ kips}$$

which is greater than the Joint 2 shear (maximum transverse shear) of 139 kips. No. 3 dowels are used to make the welded wire fabric continuous across the masonry walls. The topping is to be cast into the masonry walls as shown in Figure 7.1-12, and the spacing of the No. 3 bars is set to be modular with the CMU.

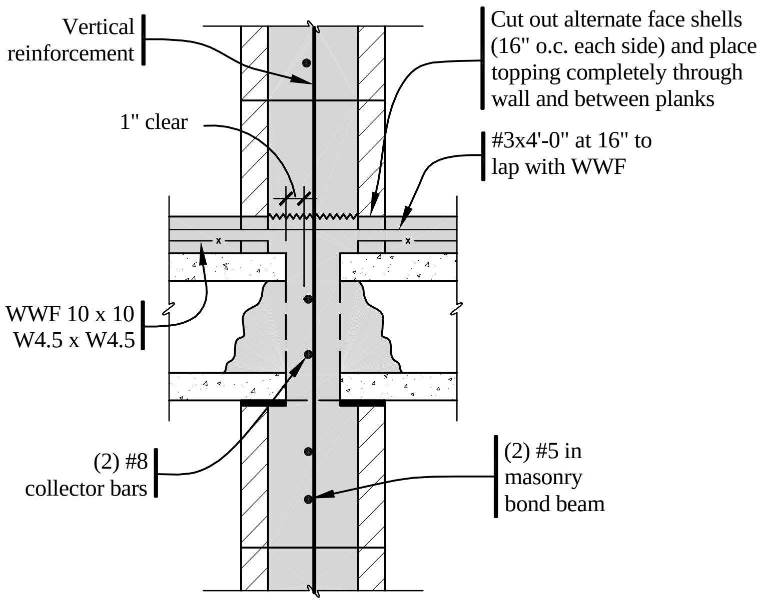


Figure 7.1-12 Wall-to-diaphragm reinforcement along interior longitudinal walls - Joint 5 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The required shear reinforcement along the exterior longitudinal wall (Joint 4) is:

$$A_{vf4} = V_{u4}/\phi\mu f_y = (27.4 \text{ kips})/[(0.75)(1.0)(60 \text{ ksi})] = 0.61 \text{ in.}^2$$

7.1.2.5.5 Check Out-of-Plane Forces

At Joint 4 with bars at 2 ft on center, F_p = 624 plf = 2 ft(624 plf) = 1.25 kips. The required reinforcement, A_s = 1.25/(0.9)(60ksi) = 0.023 in.^{2}. Provide #3 bars at 2 ft on center, which provides a nominal strength of 0.11 x 60 / 2 = 3.3 klf. The detail provides more than required by ACI 318 Sec. 16.5 for the 1.5 klf tie force. The development length was checked in the prior example. Using #3 bars at 2 ft on center will be adequate, and the detail is shown in Figure 7.1-13. The detail at joint 2 is similar.

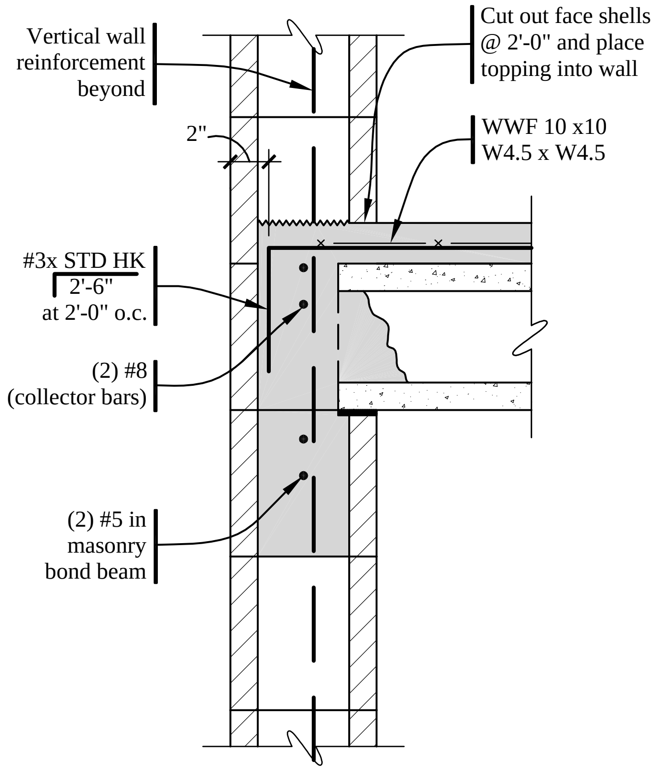


Figure 7.1-13 Exterior longitudinal wall-to-diaphragm reinforcement and out-of-plane anchorage - Joint 4 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

7.2 THREE-STORY OFFICE BUILDING WITH PRECAST CONCRETE SHEAR WALLS

This example illustrates the seismic design of ordinary precast concrete shear walls that may be used in regions of low to moderate seismicity. The *Provisions* has one requirement for detailing such walls: connections that resist overturning shall be Type Y or Z. ACI 318-02 has incorporated a less specific requirement, renamed the system as intermediate precast structural walls, and removed some of the detail. This example shows an interpretation of the intent of the *Provisions* for precast shear wall systems in regions of moderate and low seismicity, which should also meet the cited ACI 318-02 requirements.

[As indicated at the beginning of this chapter, the requirements for precast shear wall systems in the 2003 *Provisions* have been revised – primarily to point to ACI 318-02 by reference. See also Sec. 7.2.2.1 for more discussion of system requirements.]

7.2.1 Building Description

This precast concrete building is a three-story office building (Seismic Use Group I) in southern New England on Site Class D soils. The structure utilizes 10-ft-wide by 18-in.-deep prestressed double tees (DTs) spanning 40 ft to prestressed inverted tee beams for the floors and the roof. The DTs are to be constructed using lightweight concrete. Each of the above-grade floors and the roof are covered with a 2-in.-thick (minimum), normal weight cast-in-place concrete topping. The vertical seismic-force-resisting system is to be constructed entirely of precast concrete walls located around the stairs and elevator/mechanical shafts. The only features illustrated in this example are the rational selection of the seismic design parameters and the design of the reinforcement and connections of the precast concrete shear walls. The diaphragm design is not illustrated.

As shown in Figure 7.2-1, the building has a regular plan. The precast shear walls are continuous from the ground level to 12 ft above the roof. Walls of the elevator/mechanical pits are cast-in-place below grade. The building has no vertical irregularities. The story-to-story height is 12 ft.

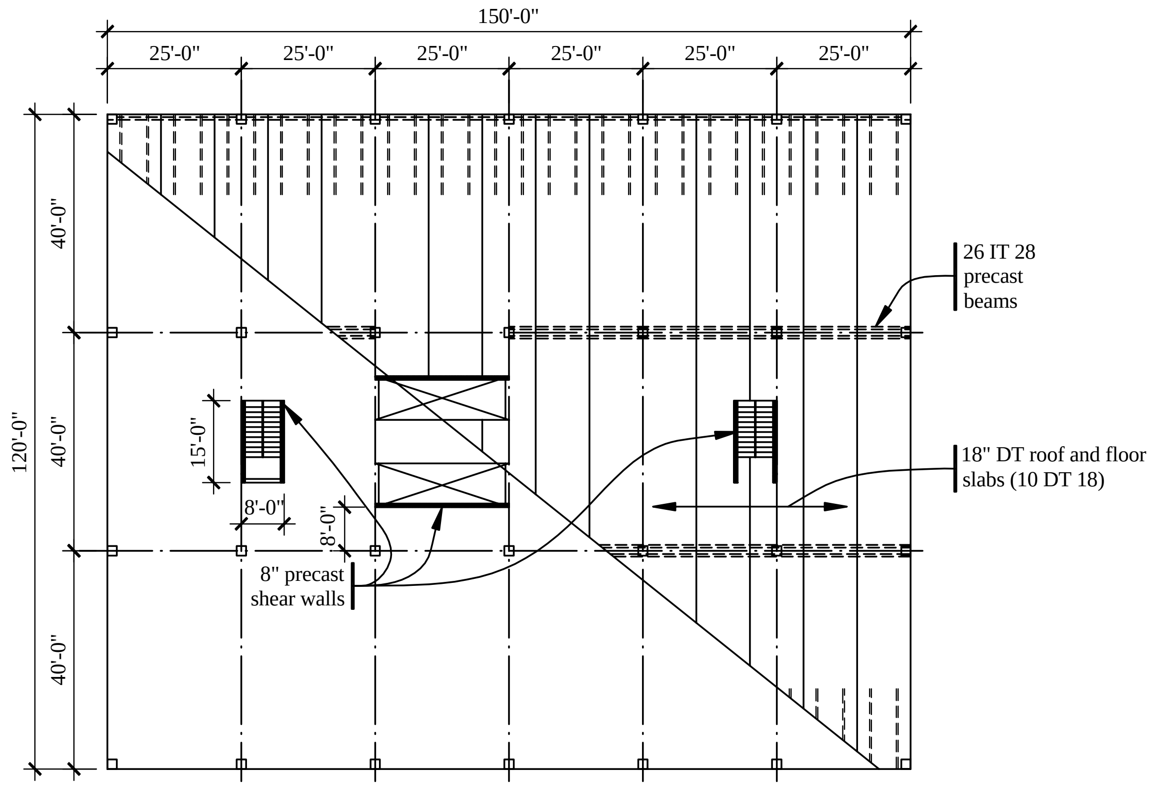


Figure 7.2-1 Three-story building plan (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The precast walls are estimated to be 8 in. thick for building mass calculations. These walls are normal weight concrete with a 28-day compressive strength, f_c' = 5,000 psi. Reinforcing bars used at the ends of the walls and in welded connectors are ASTM A706 (60 ksi yield strength). The concrete for the foundations and below-grade walls has a 28-day compressive strength, f_c' = 4,000 psi.

7.2.2 Design Requirements

7.2.2.1 Seismic Parameters of the *Provisions*

The basic parameters affecting the design and detailing of the building are shown in Table 7.2-1.

Table 7.2-1 Design Parameters

Design Parameter	Value
Seismic Use Group I	I = 1.0
S_S (Map 1 [Figure 3.3-1])	0.266
S_1 (Map 2 [Figure 3.3-2])	0.08
Site Class	D
F_a	1.59
F_v	2.4
$S_{MS} = F_a S_S$	0.425
$S_{M1} = F_v S_1$	0.192
S_{DS} = 2/3 S_{MS}	0.283
S_{D1} = 2/3 S_{M1}	0.128
Seismic Design Category	B
Basic Seismic-Force-Resisting System	Bearing Wall System
Wall Type *	Ordinary Reinforced Concrete Shear Walls
R	4
Ω_0	2.5
C_d	4

* *Provisions* Sec. 9.1.1.3 [9.2.2.1.3] provides for the use of ordinary reinforced concrete shear walls in Seismic Design Category B, which does not require adherence to the special seismic design provisions of ACI 318 Chapter 21.

[The 2003 *Provisions* have adopted the 2002 U.S. Geological Survey probabilistic seismic hazard maps and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3. These figures replace the previously used separate map package.]

[Ordinary precast concrete shear walls is recognized as a system in Table 4.3-1 of the 2003 *Provisions*. Consistent with the philosophy that precast systems are not expected to perform as well as cast-in-place systems, the design factors for the ordinary precast concrete shear walls per 2003 *Provisions* Table 4.3-1 are: R = 3, Ω_0 = 2.5, and C_d = 3. Note that while this system is permitted in Seismic Design Category B, unline ordinary reinforced concrete shear walls, it is not permitted in Seismic Design Category C. Alternatively, as this example indicates conceptually, this building could be designed incorporating intermediate precast concrete shear walls with the following design values per 2003 *Provisions* Table 4.3-1: R = 4, Ω_0 = 2.5, and C_d = 4.]

7.2.2.2 Structural Design Considerations

7.2.2.2.1 Precast Shear Wall System

This system is designed to yield in bending at the base of the precast shear walls without shear slippage at any of the joints. Although not a stated design requirement of the *Provisions* or ACI 318-02 for this Seismic Design Category, shear slip could kink the vertical rebar at the connection and sabotage the intended performance, which counts on an R factor of 4. The flexural connections at the ends of the walls, which are highly stressed by seismic forces, are designed to be the Type Y connection specified in the *Provisions*. See *Provisions* Sec. 9.1.1.2 [9.2.2.1.1] (ACI Sec. 21.1 [21.1]) for the definitions of ordinary precast concrete structural walls and *Provisions* Sec. 9.1.1.12 [not applicable for the 2003 *Provisions*] (ACI Sec. 21.11.6) for the connections. The remainder of the connections (shear connectors) are then made strong enough to ensure that the inelastic straining is forced to the intended location.

[Per 2003 *Provisions* Sec. 9.2.2.1.1 (ACI 318-02 Sec. 21.1), ordinary precast concrete shear walls need only satisfy the requirements of ACI 318-02 Chapters 1-18 (with Chapter 16 superceding Chapter 14). Therefore, the connections are to be designed in accordance with ACI 318-02 Sec. 16.6.]

Although it would be desirable to force yielding to occur in a significant portion of the connections, it frequently is not possible to do so with common configurations of precast elements and connections. The connections are often unavoidable weak links. Careful attention to detail is required to assure adequate ductility in the location of first yield and that no other connections yield prematurely. For this particular example, the vertical bars at the ends of the shear walls act as flexural reinforcement for the walls and are selected as the location of first yield. The yielding will not propagate far into the wall vertically due to the unavoidable increase in flexural strength provided by unspliced reinforcement within the panel. The issue of most significant concern is the performance of the shear connections at the same joint. The connections are designed to provide the necessary shear resistance and avoid slip without unwittingly increasing the flexural capacity of the connection because such an increase would also increase the maximum shear force on the joint. At the base of the panel, welded steel angles are designed to be flexible for uplift but stiff for in-plane shear.

7.2.2.2.2 Building System

No height limitations are imposed (*Provisions* Table 5.2.2 [4.3-1]).

For structural design, the floors are assumed to act as rigid horizontal diaphragms to distribute seismic inertial forces to the walls parallel to the motion. The building is regular both in plan and elevation, for which, according to *Provisions* Table 5.2.5.1 [4.4-4], use of the ELF procedure (*Provisions* Sec. 5.4 [5.2]) is permitted.

Orthogonal load combinations are not required for this building (*Provisions* Sec. 5.2.5.2.1 [4.4.2.1]).

Ties, continuity, and anchorage (*Provisions* Sec. 5.2.6.1 and 5.2.6.2 [4.6.1.1 and 4.6.1.2]) must be explicitly considered when detailing connections between the floors and roof, and the walls and columns.

This example does not include consideration of nonstructural elements.

Collector elements are required due to the short length of shear walls as compared to the diaphragm dimensions, but are not designed in this example.

Diaphragms need to be designed for the required forces (*Provisions* Sec. 5.2.6.2.6 [4.6.1.9]), but that design is not illustrated here.

The bearing walls must be designed for a force perpendicular to their plane (*Provisions* Sec. 5.2.6.2.7 [4.6.1.3]), but this requirement is of no real consequence for this building.

The drift limit is $0.025h_{sx}$ (*Provisions* Table 5.2.8 [4.5-1]), but drift is not computed here.

ACI 318 Sec. 16.5 requires minimum strengths for connections between elements of precast building structures. The horizontal forces were described in Sec. 7.1; the vertical forces will be described in this example.

7.2.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [4.2.2]) require that seismic forces and gravity loads be combined in accordance with the factored load combinations presented in ASCE 7 except that the factors for seismic loads (E) are defined by *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2]:

$$E = \rho Q_E \pm 0.2S_{DS}D = (1.0)Q_E \pm (0.2)(0.283)D = Q_E \pm 0.0567D$$

According to *Provisions* Sec. 5.2.4.1 [4.3.3.1], $\rho = 1.0$ for structures in Seismic Design Categories A, B, and C, even though this seismic resisting system is not particularly redundant.

The relevant load combinations from ASCE 7 are:

$1.2D \pm 1.0E + 0.5L$
$0.9D \pm 1.0E$

Into each of these load combinations, substitute E as determined above:

$1.26D + Q_E + 0.5L$
$1.14D - Q_E + 0.5L$ (will not control)
$0.96D + Q_E$ (will not control)
$0.843D - Q_E$

These load combinations are for loading in the plane of the shear walls.

7.2.4 Seismic Force Analysis

7.2.4.1 Weight Calculations

For the roof and two floors

18 in. double tees (32 psf) + 2 in. topping (24 psf)	= 56.0 psf
Precast beams at 40 ft	= 12.5 psf
16 in. square columns	= 4.5 psf
Ceiling, mechanical, miscellaneous	= 4.0 psf
Exterior cladding (per floor area)	= 5.0 psf
Partitions	= 10.0 psf
Total	= 92.0 psf

The weight of each floor including the precast shear walls is:

$$(120 \text{ ft})(150 \text{ ft})(92 \text{ psf}/1{,}000) + [15 \text{ ft}(4) + 25 \text{ ft}(2)](12 \text{ ft})(0.10 \text{ ksf}) = 1{,}790 \text{ kips}$$

Considering the roof to be the same weight as a floor, the total building weight is W = 3(1,790 kips) = 5,360 kips.

7.2.4.2 Base Shear

The seismic response coefficient (C_s) is computed using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_S = \frac{S_{DS}}{R/I} = \frac{0.283}{4/1} = 0.0708$$

except that it need not exceed the value from *Provisions* Eq. 5.4.1.1-2 [5.2-3] computed as:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.128}{0.29(4/1)} = 0.110$$

where T is the fundamental period of the building computed using the approximate method of *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.02)(36)^{0.75} = 0.29\,\text{sec}$$

Therefore, use C_s = 0.0708, which is larger than the minimum specified in *Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*]:

$C_s = 0.044IS_{DS} = (.044)(1.0)(0.283) = 0\ .0125$

[The minimum C_s has been changed to 0.01 in the 2003 *Provisions*.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4-1 [5.2-1] as:

$V = C_sW = (0.0708)(5{,}370) = 380$ kips

Note that this force is substantially larger than a design wind would be. If a nominal 20 psf were applied to the long face and then amplified by a load factor of 1.6, the result would be less than half this seismic force already reduced by an R factor of 4.

7.2.4.3 Vertical Distribution of Seismic Forces

The seismic lateral force (F_x) at any level is determined in accordance with *Provisions* Sec. 5.4.3 [5.2.3]:

$$F_x = C_{vx}V$$

where

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Since the period, T < 0.5 sec, k = l in both building directions. With equal weights at each floor level, the resulting values of C_{vx} and F_x are as follows:

Roof	C_{vr} = 0.50	F_r = 190 kips
Third Floor	C_{v3} = 0.33	F_3 = 127 kips
Second Floor	C_{v2} = 0.17	F_2 = 63.0 kips

7.2.4.4 Horizontal Shear Distribution and Torsion

7.2.4.4.1 Longitudinal Direction

Design each of the 25-ft-long walls at the elevator/mechanical shafts for half the total shear. Since the longitudinal walls are very close to the center of rigidity, assume that torsion will be resisted by the 15-ft-long stairwell walls in the transverse direction. The forces for each of the longitudinal walls are shown in Figure 7.2-2.

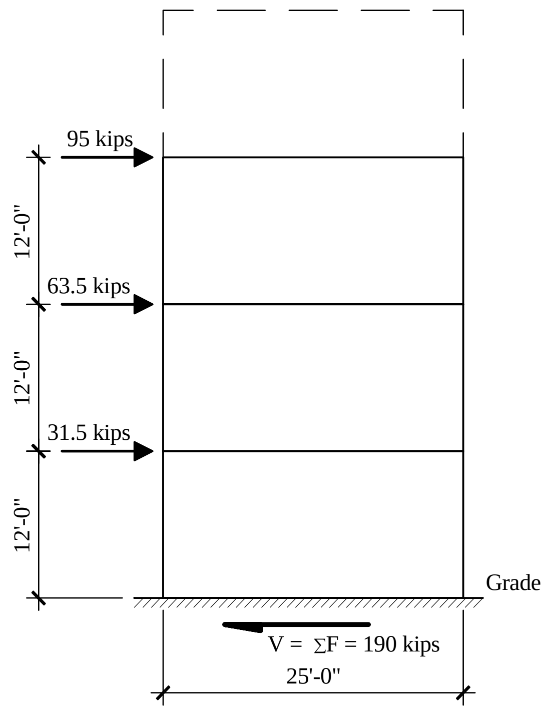


Figure 7.2-2 Forces on the longitudinal walls
(1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m).

7.2.4.4.2 Transverse Direction

Design the four 15-ft-long stairwell walls for the total shear including 5 percent accidental torsion (*Provisions* Sec. 5.4.4.2 [5.2.4.2]). A rough approximation is used in place of a more rigorous analysis considering all of the walls. The maximum force on the walls is computed as:

V = 380/4 + 380(0.05)(150)/[(100 ft moment arm) × (2 walls in each set)] = 109 kips

Thus

F_r = 109(0.50) = 54.5 kips
F_3 = 109(0.33) = 36.3 kips
F_2 = 109(0.167) = 18.2 kips

Seismic forces on the transverse walls of the stairwells are shown in Figure 7.2-3.

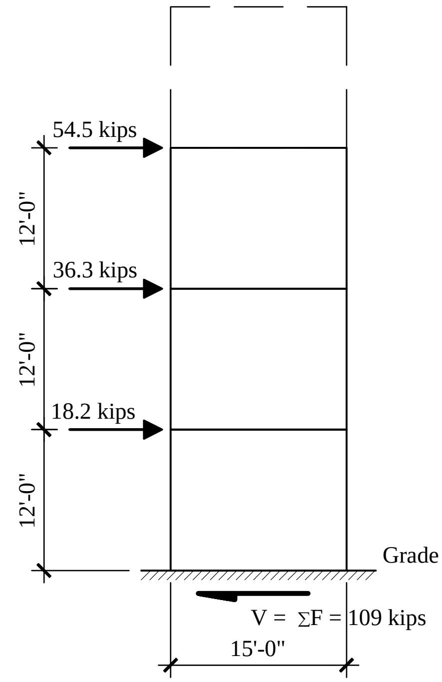


Figure 7.2-3 Forces on the transverse walls
(1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m).

7.2.5 PROPORTIONING AND DETAILING

The strength of members and components is determined using the strengths permitted and required in ACI 318 excluding Chapter 21 (see *Provisions* Sec. 9.1.1.3 [9.2.2.1.3]).

7.2.5.1 Overturning Moment and End Reinforcement

Design shear panels to resist overturning by means of reinforcing bars at each end with a direct tension coupler at the joints. A commonly used alternative is a threaded post-tensioning bar inserted through the stack of panels, but the behavior is different, and the application of the rules for a Type Y connection to such a design is not clear.

7.2.5.1.1 Longitudinal Direction

The free-body diagram for the longitudinal walls is shown in Figure 7.2-4. The tension connection at the base of the precast panel to the below grade wall is governed by the seismic overturning moment and the dead loads of the panel and supported floors and roof. In this example, the weights for an elevator penthouse, with a floor and equipment at 180 psf between the shafts and a roof at 20 psf, are included. The weight for the floors includes double tees, ceiling and partition (total of 70 psf), but not beams and columns. Floor live load is 50 psf, except 100 psf is used in the elevator lobby. Roof snow load is 30 psf. (The elevator penthouse is so small that it was ignored in computing the gross seismic forces on the building, but it is not ignored in the following calculations.)

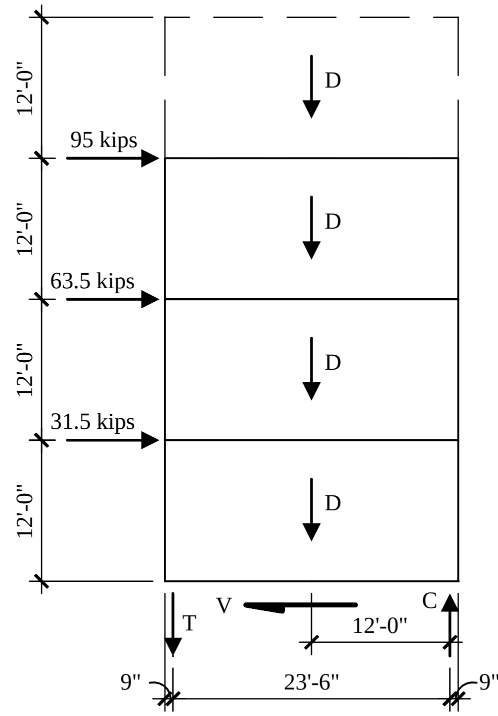


Figure 7.2-4 Free-body diagram for longitudinal walls (1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m).

At the base

M_E = (95 kips)(36 ft) + (63.5 kips)(24 ft) + (31.5 kips)(12 ft) = 5,520 ft-kips

$\sum D$ = wall + exterior floors (& roof) + lobby floors + penthouse floor + penthouse roof
= (25 ft)(48 ft)(0.1 ksf) + (25 ft)(48 ft/2)(0.070 ksf)(3) + (25 ft)(8 ft/2)(0.070 ksf)(2) + (25 ft)(8 ft/2)(0.18 ksf) + (25ft)(24 ft/2)(0.02 ksf)
= 120 + 126 + 14 + 18 + 6 = 284 kips

$\sum L$ = (25 ft)(48 ft/2)(0.05 ksf)(2) + (25 ft)(8 ft/2)(0.1 ksf) = 60 + 10 = 70 kips

$\sum S$ = (25ft)(48 ft + 24 ft)(0.03 ksf)/2 = 27 kips

Using the load combinations described above, the vertical loads for combining with the overturning moment are computed as:

P_{max} = 1.26 D + 0.5 L + 0.2 S = 397 kips
P_{min} = 0.843 D = 239 kips

The axial load is quite small for the wall panel. The average compression P_{max} / A_g = 0.165 ksi (3.3 percent of f'_c). Therefore, the tension reinforcement can easily be found from the simple couple shown on Figure 7.2-4.

The effective moment arm is:

jd = 25 - 1.5 = 23.5 ft

and the net tension on the uplift side is:

$$T_u = \frac{M}{jd} - \frac{P_{\min}}{2} = \frac{5320}{23.5} - \frac{239}{2} = 107 \text{ kips}$$

The required reinforcement is:

$A_s = T_u/\phi f_y$ = (107 kips)/[0.9(60 ksi)] = 1.98 in.^{2}

Use two #9 bars (A_s = 2.0 in.^{2}) at each end with direct tension couplers for each bar at each panel joint. Since the flexural reinforcement must extend a minimum distance d (the flexural depth)beyond where it is no longer required, use both #9 bars at each end of the panel at all three levels for simplicity.

At this point a check of ACI 318 Sec. 16.5 will be made. Bearing walls must have vertical ties with a nominal strength exceeding 3 kips/ft, and there must be at least two ties per panel. With one tie at each end of a 25 ft panel, the demand on the tie is:

T_n = (3 kip/ft)(25 ft)/2 = 37.5 kip

The two #9 bars are more than adequate for the ACI requirement.

Although no check for confinement of the compression boundary is required for ordinary precast concrete shear walls, it is shown here for interest. Using the check from ACI 318-99 Sec. 21.6.6.2 [21.7.6.2], the depth to the neutral axis is:

Total compression force = $A_s f_y + P_{max}$ = (2.0)(60) + 397 = 517 kips
Compression block a = (517 kips)/[(0.85)(5 ksi)(8 in. width)] = 15.2 in.
Neutral axis depth $c = a/(0.80)$ = 19.0 in.

The maximum depth (c) with no boundary member per ACI 318-99 Eq. 21-8 [21-8] is:

$$c \le \frac{l}{600\left(\delta_u / h_w\right)}$$

where the term (δ_u/h_w) shall not be taken less than 0.007. Once the base joint yields, it is unlikely that there will be any flexural cracking in the wall more than a few feet above the base. An analysis of the wall for the design lateral forces using 50% of the gross moment of inertia, ignoring the effect of axial loads, and applying the C_d factor of 4 to the results gives a ratio (δ_u/h_w) far less than 0.007. Therefore, applying the 0.007 in the equation results in a distance c of 71 in., far in excess of the 19 in. required. Thus, ACI 318-99 would not require transverse reinforcement of the boundary even if this wall were designed as a special reinforced concrete shear wall. For those used to checking the compression stress as an index:

$$\sigma = \frac{P}{A} + \frac{M}{S} = \frac{389}{8(25)12} + \frac{6(5{,}520)}{8(25)^2(12)} = 742 \text{ psi}$$

The limiting stress is $0.2f_c'$, which is 1000 psi, so no transverse reinforcement is required at the ends of the longitudinal walls.

7.2.5.1.2 Transverse Direction

The free-body diagram of the transverse walls is shown in Figure 7.2-5. The weight of the precast concrete stairs is 100 psf and the roof over the stairs is 70 psf.

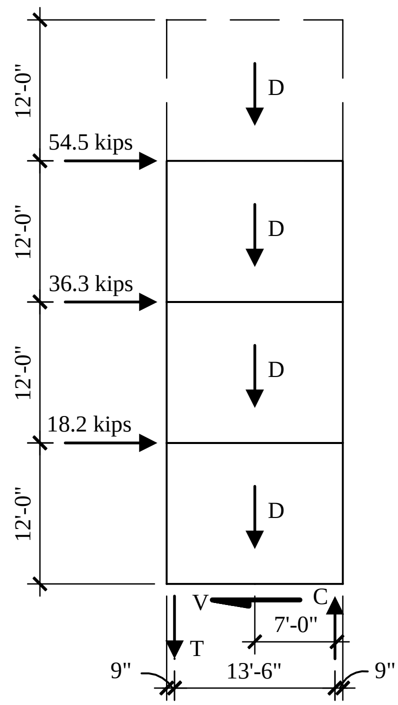


Figure 7.2-5 Free-body diagram of the transverse walls (1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m).

The transverse wall is similar to the longitudinal wall.

At the base

M_E = (54.5 kips)(36 ft) + (36.3 kips)(24 ft) + (18.2 kips)(12 ft) = 3,052 ft-kips

$\sum D$ = (15 ft)(48 ft)(0.1 ksf) + 2(12.5 ft/2)(10 ft/2)(0.07 ksf)(3) + (15 ft)(8 ft/2)[(0.1 ksf)(3) + (0.07 ksf)] = 72 + 13 + 18 + 4 = 107 kips

$\sum L$ = 2(12.5 ft/2)(10 ft/2)(0.05 ksf)(2) + (15 ft)(8 ft/2)(0.1 ksf)(3) = 6 + 18 = 24 kips

$\sum S$ = [2(12.5 ft/2)(10 ft/2) + (15 ft)(8 ft/2)](0.03 ksf) = 3.7 kips

P_{max} = 1.26(107) + 0.5(24) + 0.2(4) = 148 kips

P_{min} = 0.843(107) = 90.5 kips

jd = 15 - 1.5 = 13.5 ft

$T_u = (M_{net}/jd) - P_{min}/2$ = (3,052/13.5) - 90.5/2 = 181 kips

$A_s = T_u/\phi f_y$ = (181 kips)/[0.9(60 ksi)] = 3.35 in.^{2}

Use two #10 and one #9 bars (A_s = 3.54 in.^{2}) at each end of each wall with a direct tension coupler at each bar for each panel joint. All three bars at each end of the panel will also extend up through all three levels for simplicity. Following the same method for boundary member check as on the longitudinal walls:

Total compression force = $A_s f_y + P_{max}$ = (3.54)(60) + 148 = 360 kips
Compression block a = (360 kips)/[(0.85)(5 ksi)(8 in. width)] = 10.6 in.
Neutral axis depth $c = a/(0.80)$ = 13.3 in.

Even though this wall is more flexible and the lateral loads will induce more flexural cracking, the computed deflections are still small and the minimum value of 0.007 is used for the ratio (δ_u/h_w). This yields a maximum value of c = 42.9 in., thus confinement of the boundary would not be required. The check of compression stress as an index gives:

$$\sigma = \frac{P}{A} + \frac{M}{S} = \frac{140}{8(15)12} + \frac{6(2,930)}{8(15)^2(12)} = 951 \text{ psi}$$

Since σ < 1,000 psi, no transverse reinforcement is required at the ends of the transverse walls. Note how much closer to the criterion this transverse wall is by the compression stress check.

The overturning reinforcement and connection are shown in Figures 7.2-6. *Provisions* Sec. 9.1.1.12 [not applicable in the 2003 *Provisions*] (ACI 21.11.6.4) requires that this Type Y connection develop a probable strength of 125% of the nominal strength and that the anchorage on either side of the connection develop 130% of the defined probable strength. [As already noted, the connection requirements for ordianry precast concrete shear walls have been removed in the 2003 *Provisions* and the ACI 318-02 requirements are less specific.] The 125% requirement applies to the grouted mechanical splice, and the requirement that a mechanical coupler develop 125% of specified yield strength of the bar is identical to the Type 1 coupler defined by ACI 318 Sec. 21.2.6.1. Some of the grouted splices on the market can qualify as the Type 2 coupler defined by ACI, which must develop the specified tensile strength of the bar. The development length, l_d, for the spliced bars is multiplied by both the 1.25 and the 1.3 factors to satisfy the *Provisions* requirement. The bar in the panel is made continuous to the roof, therefore no calculation of development length is necessary in the panel. The dowel from the foundation will be hooked, otherwise the depth of the foundation would be more than required for structural reasons. The size of the foundation will provide adequate cover to allow the 0.7 factor on ACI's standard development length for hooked bars. For the # 9 bar:

$$1.3(1.25) l_{dh} = \frac{(1.6.25)0.7(1200)d_b}{\sqrt{f_c'}} = \frac{1365(1.128)}{\sqrt{4000}} = 24.3 \text{ in.}$$

Similarly, for the #10 bar, the length is 27.4 in.

Like many shear wall designs, this design does concentrate a demand for overturning resistance on the foundation. In this instance the resistance may be provided by a large footing (on the order of 20 ft by 28 ft by 3 ft thick) under the entire stairwell, or by deep piers or piles with an appropriate cap for load transfer. Refer to Chapter 4 for examples of design of each type of foundation, although not for this particular example.

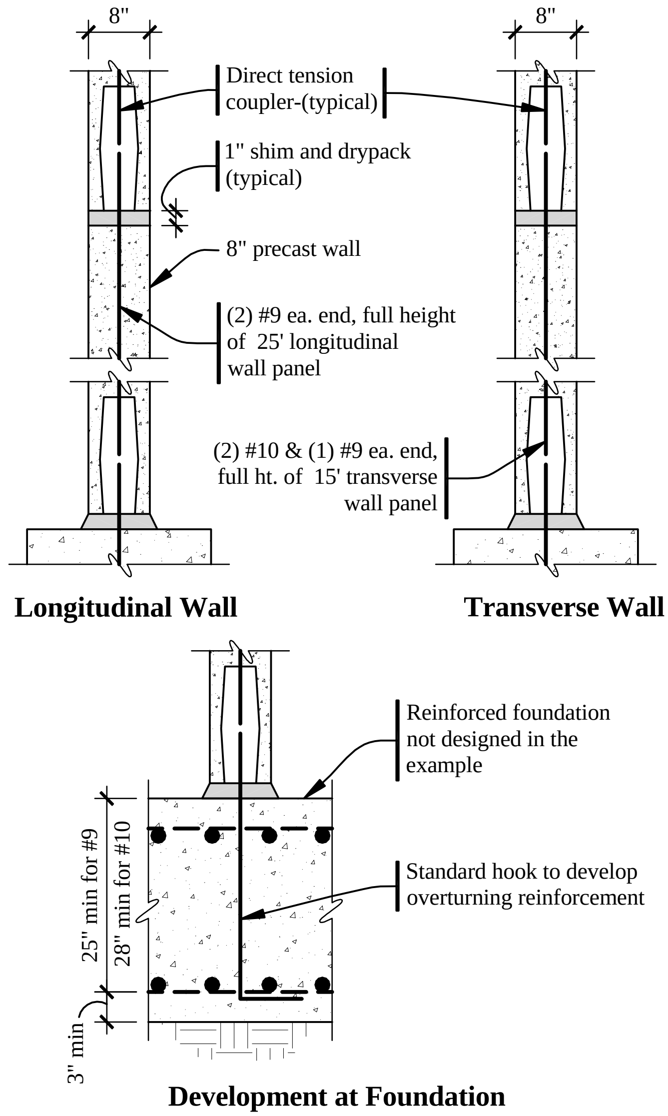


Figure 7.2-6 Overturning connection detail at the base of the walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

7.2.5.2 Shear Connections and Reinforcement

Panel joints often are designed to resist the shear force by means of shear friction but that technique is not used for this example because the joint at the foundation will open due to flexural yielding. This opening would concentrate the shear stress on the small area of the dry-packed joint that remains in compression. This distribution can be affected by the shims used in construction. Tests have shown that this often leads to slip of the joint, which could lead to a kink in the principal tension reinforcement at or near its splice and destroy the integrity of the system. Therefore, the joint will be designed with direct shear connectors that will prevent slip along the joint. This is the authors' interpretation of the *Provisions* text indicating that "Type Y connections shall develop under flexural, shear, and axial load actions, as required, a probable strength. . . ." based upon 125 percent of the specified yield in the connection. It would not be required by the ACI 318-02 rules for intermediate precast walls.

7.2.5.2.1 Longitudinal Direction

The shear amplification factor is determined as:

$$\frac{M_{capacity}}{M_{demand}} = \frac{A_s(1.25)f_y jd + P_{\max} jd/2}{M_u} = \frac{(2.0\text{ in.}^2)(1.25)(60\text{ ksi})(23.5\text{ ft}) + (397\text{ kip})(23.5\text{ ft}/2)}{5320\text{ ft-kip}}$$

$$=1.54$$

Therefore, the design shear (V_u) at the base is 1.54(190 kips) = 292 kips

The base shear connection is shown in Figure 7.2-7 and is to be flexible vertically but stiff horizontally in the plane of the panel. The vertical flexibility is intended to minimize the contribution of these connections to overturning resistance, which would simply increase the shear demand.

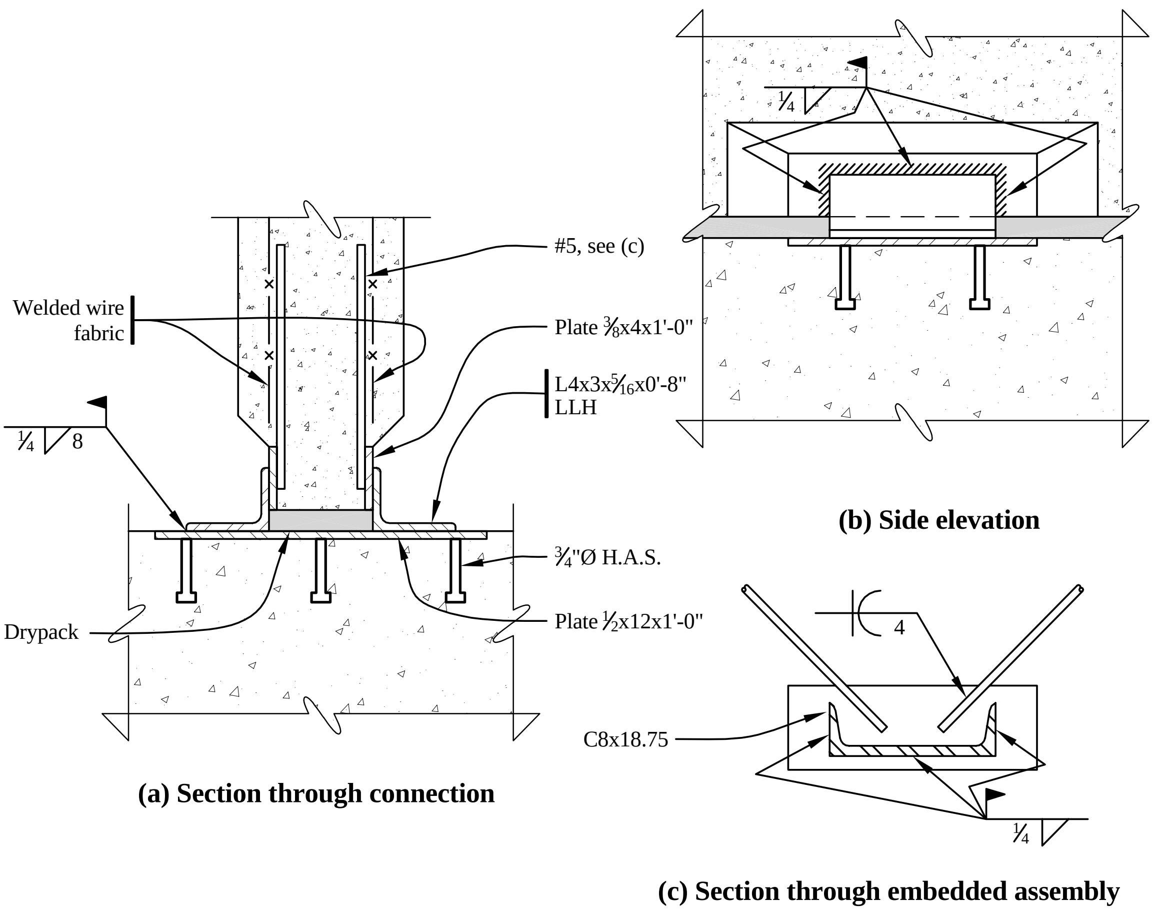


Figure 7.2-7 Shear connection at base (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

In the panel, provide an assembly with two face plates 3/8 in. × 4 in. × 12 in. connected by a C8x18.75 and with diagonal #5 bars as shown in the figure. In the foundation provide an embedded plate 1/2 × 12 × 1'-6" with six 3/4 in. diameter headed anchor studs. In the field, weld an L 4 × 3 × 5/16 × 0'-8", long leg horizontal, on each face. The shear capacity of this connection is checked:

Shear in the two loose angles

$$\phi V_n = \phi(0.6F_u)tl(2) = (0.75)(0.6)(58\text{ ksi})(0.3125\text{ in.})(8\text{ in.})(2) = 130.5\text{ kip}$$

Weld at toe of loose angles

$$\phi V_n = \phi(0.6F_u)t_e l(2) = (0.75)(0.6)(70\text{ ksi})(0.25\text{ in. } / \sqrt{\ } 2)(8\text{ in.})(2) = 89.1\text{ kip}$$

Weld at face plates, using Table 8-9 in AISC Manual (3rd edition; same table is 8-42 in 2nd edition)

$\phi V_n = CC_1Dl(2\text{ sides})$
$C_1 = 1.0$ for E70 electrodes
$l = 8$ in.
$D = 4$ (sixteenths of an inch)
$k = 2$ in. / 8 in. = 0.25
a = eccentricity, summed vectorially: horizontal component is 4 in.; vertical component is 2.67 in.; thus, al = 4.80 in. and a = 4.8 in./8 in. = 0.6 from the table. By interpolation, C = 1.29
$\phi V_n = (1.29)(1.0)(4)(8)(2) = 82.6$ kip

Weld from channel to plate has at least as much capacity, but less demand.

Bearing of concrete at steel channel

$$f_c = \phi(0.85f'_c) = 0.65(0.85)(5\text{ ksi}) = 2.76\text{ ksi}$$

The C8 has the following properties:

$t_w = 0.487$ in.
$b_f = 2.53$ in.
$t_f = 0.39$ in. (average)

The bearing will be controlled by bending in the web (because of the tapered flange, the critical flange thickness is greater than the web thickness). Conservatively ignoring the concrete's resistance to vertical deformation of the flange, compute the width (b) of flange loaded at 2.76 ksi that develops the plastic moment in the web:

$$M_p = \phi F_y t_w^2/4 = (0.9)(50\text{ ksi})(0.487^2\text{ in.}^2)/4 = 2.67\text{ in.-kip/in.}$$
$$M_u = f_c[(b\text{-}t_w)^2/2 - (t_w/2)^2/2] = 2.76[(b - 0.243\text{ in.})^2 - (0.243\text{ in.})^2]/2$$

setting the two equal results in b = 1.65 in.

Therefore bearing on the channel is

$$\phi V_c = f_c(2 - t_w)(l) = (2.76\text{ ksi})[(2(1.65) - 0.487\text{ in.}](6\text{ in.}) = 46.6\text{ kip}$$

To the bearing capacity on the channel is added the 4 - #5 diagonal bars, which are effective in tension and compression; $\phi = 0.75$ for shear is used here:

$$\phi V_s = \phi f_y A_s\cos\alpha = (0.75)(60\text{ ksi})(4)(0.31\text{ in.}^2)(\cos 45^\circ) = 39.5\text{ kip}$$

Thus, the total capacity for transfer to concrete is:

$$\phi V_n = \phi V_c + \phi V_s = 46.6 + 39.6 = 86.1\text{ kip}$$

The capacity of the plate in the foundation is governed by the headed anchor studs. The *Provisions* contain the new anchorage to concrete provisions that are in ACI 318-02 Appendix D. [In the 2003 *Provisions*, the anchorage to concrete provisions have been removed and replaced by the reference to ACI 318-02.] Capacity in shear for anchors located far from an edge of concrete, such as these, and with sufficient embedment to avoid the pryout failure mode is governed by the capacity of the steel:

$$\phi V_s = \phi\, n\, A_{se}\, f_{ut} = (0.65)(6 \text{ studs})(0.44 \text{ in.}^2 \text{ per stud})(60 \text{ ksi}) = 103 \text{ kip}$$

Provisions Sec 9.2.3.3.2 (ACI 318-02 Sec. D.3.3.3) specifies an additional factor of 0.75 to derate anchors in structures assigned to Seismic Design Categories C and higher.

In summary the various shear capacities of the connection are:

Shear in the two loose angles:	130.5 kip
Weld at toe of loose angles:	89.1 kip
Weld at face plates:	82.6 kip
Transfer to concrete:	86.1 kip
Headed anchor studs at foundation:	103 kip

The number of embedded plates (n) required for a panel is:

$$n = 292/82.6 = 3.5$$

Use four connection assemblies, equally spaced along each side (5'-0" on center works well to avoid the end reinforcement). The plates are recessed to position the #5 bars within the thickness of the panel and within the reinforcement of the panel.

It is instructive to consider how much moment capacity is added by the resistance of these connections to vertical lift at the joint. The vertical force at the tip of the angle that will create the plastic moment in the leg of the angle is:

$$T = M_p / x = F_y l t^2/4 / (l\text{-}k) = (36 \text{ ksi})(8 \text{ in})(0.3125^2 \text{ in.}^2)/4]/(4 \text{ in.} - 0.69 \text{ in.}) = 2.12 \text{ kips}$$

There are four assemblies with two loose angles each, giving a total vertical force of 17 kips. The moment resistance is this force times half the length of the panel, which yields 212 ft-kips. The total demand moment, for which the entire system is proportioned, is 5320 ft - kips. Thus, these connections will add about 4% to the resistance and ignoring this contribution is reasonable. If a straight plate 1/4 in. x 8 in., which would be sufficient, were used and if the welds and foundation embedment did not fail first, the tensile capacity would be 72 kips each, a factor of 42 increase over the angles, and the shear connections would have the unintended effect of more than doubling the flexural resistance, which could easily cause failures in the system.

Using ACI 318 Sec. 11.10, check the shear strength of the precast panel at the first floor:

$$\phi V_c = \phi 2 A_{cv}\sqrt{f_c'}\, hd = 0.85(2)\sqrt{5{,}000}(8)(23.5)(12) \quad = 271 \text{ kips}$$

Because $\phi V_c \geq V_u = 190$ kips, the wall is adequate for shear without even considering the reinforcement. Note that the shear strength of wall itself is not governed by the overstrength required for the connection. However, since $V_u \geq \phi V_c/2 = 136$ kips, ACI Sec. 11.10.8 requires minimum wall reinforcement in accordance with ACI 318 Sec. 11.10.9.4 rather than Chapter 14 or 16. For the minimum required ρ_h = 0.0025, the required reinforcement is:

A_v = 0.0025(8)(12) = 0.24 in.^{2}/ft

As before, use two layers of welded wire fabric, WWF 4×4 - W4.0×W4.0, one on each face. Shear reinforcement provided, A_v = 0.12(2) = 0.24 in.^{2}/ft

Next, compute the shear strength at Level 2. Since the end reinforcement at the base extends to the top of the shear wall, bending is not a concern. Yield of the vertical bars will not occur, the second floor joint will not open (unlike at the base) and, therefore, shear friction could rationally be used to design the connections at this level and above. Shear keys in the surface of both panels would be advisable. Also, because of the lack of flexural yield at the joint, it is not necessary to make the shear connection be flexible with respect to vertical movement. To be consistent with the seismic force increase from yielding at the base, the shear at this level will be increased using the same amplification factor as calculated for the first story.

The design shear, V_{u2} = 1.54(95 + 63.5) = 244 kips.

Using the same recessed embedded plate assemblies in the panel as at the base, but welded with a straight plate, the number of plates, n = 244/82.6 = 2.96. Use three plates, equally spaced along each side.

Figure 7.2-8 shows the shear connection at the second and third floors of the longitudinal precast concrete shear wall panels.

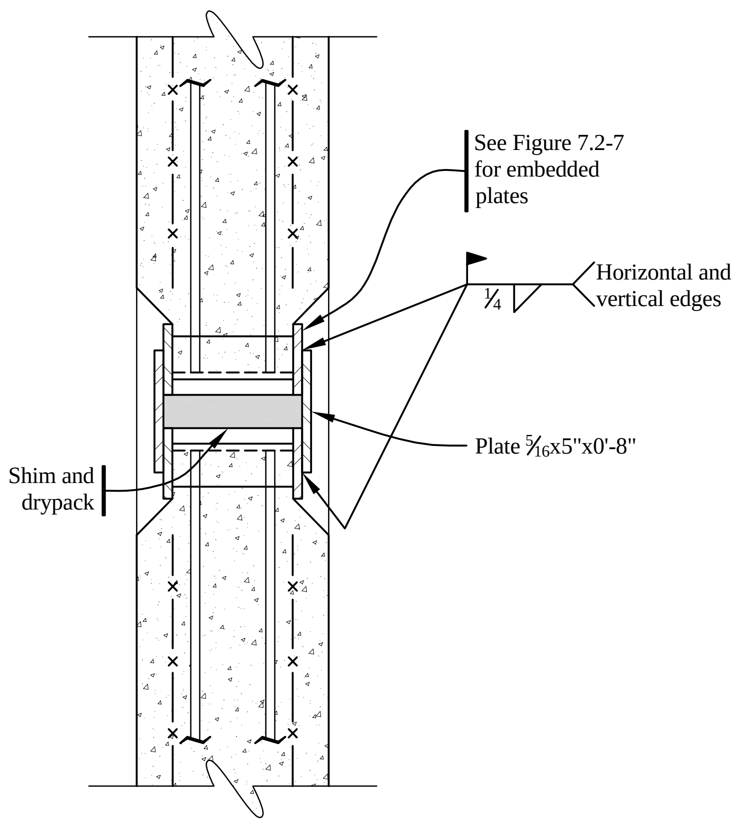


Figure 7.2-8 Shear connections on each side of the wall at the second and third floors (1.0 in = 25.4 mm).

7.2.5.2.2 Transverse Direction

Use the same procedure as for the longitudinal walls:

$$\frac{M_{capacity}}{M_{demand}} = \frac{A_s(1.25)f_y jd + P_{\max} jd/2}{M_u} = \frac{(3.54\text{ in.}^2)(1.25)(60\text{ ksi})(13.5\text{ ft}) + (148\text{ kip})(23.5\text{ ft}/2)}{3052\text{ ft-kip}}$$

$$= 1.50$$

Design shear, V_u at base is 1.50(105 kips) = 157.5 kips.

Use the same shear connections as at the base of the longitudinal walls (Figure 7.2-7). The connection capacity is 82.6 kips and the number of connections required is n = 157.5/82.6 = 1.9. Provide two connections on each panel.

Check the shear strength of the first floor panel as described previously:

$$\phi V_c = \phi 2\sqrt{f_c'}hd = 0.85(2)\sqrt{5{,}000}(8)(13.5)(12) \quad = 156\text{ kips}$$

Similar to the longitudinal direction, $\phi V_c \geq V_u$ = 142 kips, but $V_u \geq \phi V_c/2$ so provide two layers of welded wire fabric, WWF 4×4 - W4.0×W4.0, one on each face as in the longitudinal walls.

Compute the shear demand at the second floor level joint as indicated below.

The design shear, V_u = 1.50(52.3+ 34.9) = 130.8 kips.

Use the same plates as in the longitudinal walls. The number of plates, n = 130.8/82.6 = 1.6. Use two plates, equally spaced. Use the same shear connections for the transverse walls as for the longitudinal walls as shown in Figures 7.2-7 and 7.2-8.

7.3 ONE-STORY PRECAST SHEAR WALL BUILDING

This example illustrates precast shear wall seismic design using monolithic emulation as defined in the *Provisions* Sec. 9.1.1.12 [not applicable in the 2003 *Provisions*] (ACI Sec. 21.11.3) for a single-story building in a region of high seismicity. For buildings in Seismic Design Category D, *Provisions* Sec. 9.1.1.12 [not applicable in the 2003 *Provisions*] (ACI Sec. 21.11.2.1) requires that the precast seismic-force-resisting system emulate the behavior of monolithic reinforced concrete construction or that the system's cyclic capacity be demonstrated by testing. This example presents an interpretation of monolithic emulation design with ductile connections. Here the connections in tension at the base of the wall panels yield by bending steel angles out-of-plane. The same connections at the bottom of the panel are detailed and designed to be very strong in shear and to resist the nominal shear strength of the concrete panel.

[Many of the provisions for precast concrete shear walls in areas of high seismicity have been moved out of the 2003 *Provisions* and into ACI 318-02. For structures assigned to Seismic Design Category D, 2003 *Provisions* Sec. 9.2.2.1.3 (ACI 318-02 Sec. 21.21.1.4) permits special precast concrete shear walls (ACI 318-02 Sec. 21.8) or intermediate precast concrete shear walls (ACI 318-02 Sec. 21.13). The 2003 *Provisions* does not differentiate between precast or cast-in-place concrete for special shear walls. This is because ACI 318-02 Sec. 21.8 essentially requires special precast concrete shear walls to satisfy the same design requirements as special reinforced concrete shear walls (ACI 318-02 Sec. 21.7). Alternatively, special precast concrete shear walls are permitted if they satisfy experimental and analytical requirements contained in 2003 *Provisions* Sec. 9.2.2.4 and 9.6.]

7.3.1 Building Description

The precast concrete building is a single-story industrial warehouse building (Seismic Use Group I) located in the Los Angeles area on Site Class C soils. The structure has 8-ft-wide by 12-1/2-in.-deep prestressed double tee (DT) wall panels. The roof is light gage metal decking spanning to bar joists that are spaced at 4 ft on center to match the location of the DT legs. The center supports for the joists are joist girders spanning 40 ft to steel tube columns. The vertical seismic-force-resisting system is the precast/prestressed DT wall panels located around the perimeter of the building. The average roof height is 20 ft, and there is a 3 ft parapet. The building is located in the Los Angeles area on Site Class C soils. Figure 7.3-1 shows the plan of the building, which is regular.

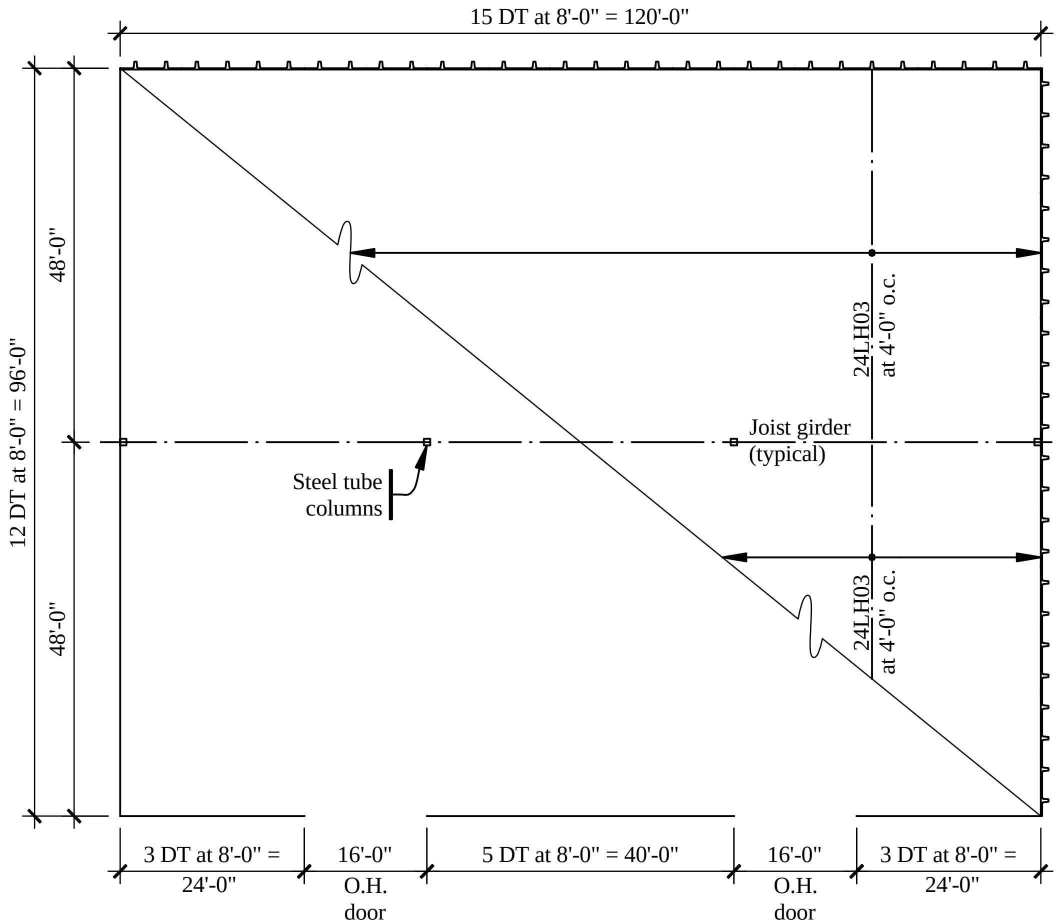


Figure 7.3-1 Single-story industrial warehouse building plan (1.0 ft = 0.3048 m).

The precast wall panels used in this building are typical DT wall panels commonly found in many locations but not normally used in Southern California. For these wall panels, an extra 1/2 in. has been added to the thickness of the deck (flange). This extra thickness is intended to reduce cracking of the flanges and provide cover for the bars used in the deck at the base. The use of thicker flanges is addressed later.

Provisions Sec. 9.1.1.5 [9.2.2.1.5.4] (ACI Sec. 21.2.5.1 [21.2.5.1]) limits the grade and type of reinforcement in boundary elements of shear walls and excludes the use of bonded prestressing tendons (strand) due to seismic loads. ACI 318-99 Sec. 21.7.5.2 [21.9.5.2] permits the use of strand in boundary elements of diaphragms provided the stress is limited to 60,000 psi. This design example uses the strand as the reinforcement based on that analogy. The rationale for this is that the primary reinforcement of the DT, the strand, is not working as the ductile element of the wall panel and is not expected to yield in an earthquake.

The wall panels are normal-weight concrete with a 28-day compressive strength, f_c'= 5,000 psi. Reinforcing bars used in the welded connections of the panels and footings are ASTM A706 (60 ksi). The concrete for the foundations has a 28-day compressive strength, f_c'= 4,000 psi.

7.3.2 Design Requirements

7.3.2.1 Seismic Parameters of the *Provisions*

The basic parameters affecting the design and detailing of the building are shown in Table 7.3-1.

Table 7.3-1 Design Parameters

Design Parameter	Value
Seismic Use Group I	I = 1.0
S_S (Map 1 [Figure 3.3-1])	1.5
S_1 (Map 2 [Figure 3.3-2])	0.60
Site Class	C
F_a	1.0
F_v	1.3
$S_{MS} = F_aS_S$	1.5
$S_{M1} = F_vS_1$	0.78
S_{DS} = 2/3 S_{MS}	1.0
S_{D1} = 2/3 S_{M1}	0.52
Seismic Design Category	D
Basic Seismic-Force-Resisting System	Bearing Walls System
Wall Type *	Special Reinforced Concrete Shear Wall
R	5
Ω_0	2.5
C_d	5

* *Provisions* Sec. 9.7.1.2 [9.2.2.1.3] requires special reinforced concrete shear walls in Seismic Design Category D and requires adherence to the special seismic design provisions of ACI 318 Chapter 21.

[The 2003 *Provisions* have adopted the 2002 U.S. Geological Survey seismic hazard maps and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

7.3.2.2 Structural Design Considerations

7.3.2.2.1 Precast Shear Wall System

The criteria for the design is to provide yielding in a dry connection for bending at the base of each precast shear wall panel while maintaining significant shear resistance in the connection. The flexural connection for a wall panel at the base is located in one DT leg while the connection at the other leg is used for compression. Per *Provisions* Sec. 9.1.1.12 (ACI Sec. 21.11.3.1) [not applicable in the 2003 *Provisions*], these connections resist the shear force equal to the nominal shear strength of the panel and have a nominal strength equal to twice the shear that exists when the actual moment is equal to M_{pr}

(which ACI defines as ϕ = 1.0 and a steel stress equal to 125% of specified yield). Yielding will develop in the dry connection at the base by bending the horizontal leg of the steel angle welded between the embedded plates of the DT and footing. The horizontal leg of this angle is designed in a manner to resist the seismic tension of the shear wall due to overturning and then yield and deform inelastically. The connections on the two legs of the DT are each designed to resist 50 percent of the shear. The anchorage of the connection into the concrete is designed to satisfy the Type Z requirements in *Provisions* Sec. 9.1.1.12 (ACI Sec. 21.11.6.5) [not applicable in the 2003 *Provisions*.]. Careful attention to structural details of these connections is required to ensure tension ductility and resistance to large shear forces that are applied to the embedded plates in the DT and footing.

[Based on the 2003 *Provisions*, unless the design of special precast shear walls is substantiated by experimental evidence and analysis per 2003 *Provisions* Sec. 9.2.2.4 (ACI 318-02 Sec. 21.8.2), the design must satisfy ACI 318-02 Sec. 21.7 requirements for special structural walls as referenced by ACI 318-02 Sec. 21.8.1. The connection requirements are not as clearly defined as in the 2000 *Provisions*.]

7.3.2.2.2 Building System

Height limit is 160 ft (*Provisions* Table 5.2.2 [4.3-1]).

The metal deck roof acts as a flexible horizontal diaphragm to distribute seismic inertia forces to the walls parallel to the earthquake motion (*Provisions* Sec. 5.2.3.1 [4.3.2.1]).

The building is regular both in plan and elevation.

The reliability factor, ρ is computed in accordance with *Provisions* Sec. 5.2.4.2 [4.3.3]. The maximum ρ_x value is given when r_{max_x} is the largest value. r_{max_x} is the ratio of design story shear resisted by the single element carrying the most shear force to the total story shear. All shear wall elements (8-ft-wide panels) have the same stiffness. Therefore, the shear in each element is the total shear along a side divided by the number of elements (wall panels). The largest r_{max_x} value is along the side with the least number of panels. Along the side with 11 panels, r_{max_x} is computed as:

$$r_{max_x} = \frac{1/2 \big/ 11}{1.0} = 0.0455$$

$$A_x = 96 \text{ ft} \times 120 \text{ ft} = 11{,}520 \text{ ft}^2$$

$$\rho_x = 2 - \frac{20}{r_{max_x}\sqrt{A_x}} = 2 - \frac{20}{0.0455\sqrt{11{,}520}} = \text{-}2.10$$

Therefore, use ρ = 1.0.

[The redundancy requirements have been substantially changed for the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a single shear wall with an aspect ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Based on the design procedures for the walls, each individual panel should be considered a separate wall with an aspect ratio greater than 1.0. Alternatively, if the structure is regular in plan and there are at least two bays of perimeter framing on each side of the structure in each orthogonal direction, the exception in 2003 *Provisions* Sec. 4.3.3.2

permits the use of D, ρ = 1.0. This exception could be interpreted as applying to this example, which is regular and has more than two wall panels (bays) in both directions.]

The structural analysis to be used is the ELF procedure (*Provisions* Sec. 5.4 [5.2]) as permitted by *Provisions* Table 5.2.5 [4.4-1].

Orthogonal load combinations are not required for flexible diaphragms in Seismic Design Category D (*Provisions* Sec. 5.2.5.2.3 [4.4.2.3]).

This example does not include design of the foundation system, the metal deck diaphragm, or the nonstructural elements.

Ties, continuity, and anchorage (*Provisions* 5.2.6.1 through 5.2.6.4 [4.6]) must be explicitly considered when detailing connections between the roof and the wall panels. This example does not include the design of these connections, but sketches of details are provided to guide the design engineer.

There are no drift limitations for single-story buildings as long as they are designed to accommodate predicted lateral displacements (*Provisions* Table 5.2.8, footnote b [4.5-1, footnote c]).

7.3.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7) require that seismic forces and gravity loads be combined in accordance with the factored load combinations as presented in ASCE 7, except that the load factor for earthquake effects (E) is defined by *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2]:

$$E = \rho Q_E \pm 0.2S_{DS}D = (1.0)Q_E \pm (0.2)(1.0)D = Q_E \pm 0.2D$$

The relevant load combinations from ASCE 7 are:

$1.2D \pm 1.0E + 0.5L$
$0.9D \pm 1.0E$

Note that roof live load need not be combined with seismic loads, so the live load term, L, can be omitted from the equation.

Into each of these load combinations, substitute E as determined above:

$1.4D + Q_E$
$1.0D - Q_E$ (will not control)
$1.1D + Q_E$ (will not control)
$0.7D - Q_E$

These load combinations are for the in-plane direction of the shear walls.

7.3.4 Seismic Force Analysis

7.3.4.1 Weight Calculations

Compute the weight tributary to the roof diaphragm

Roofing	=	2.0 psf
Metal decking	=	1.8 psf
Insulation	=	1.5 psf
Lights, mechanical, sprinkler system etc.	=	3.2 psf
Bar joists	=	2.7 psf
Joist girder and columns	=	0.8 psf
Total	=	12.0 psf

The total weight of the roof is computed as:

(120 ft × 96 ft)(12 psf/1,000) = 138 kips

The exterior double tee wall weight tributary to the roof is:

(20 ft/2 + 3 ft)[42 psf/1,000](120 ft + 96 ft)2 = 236 kips

Total building weight for seismic lateral load, W = 138 + 236 = 374 kips

7.3.4.2 Base Shear

The seismic response coefficient (C_s) is computed using *Provisions* Eq. 5.4.1.1-1 [5.2-2] as:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.0}{5/1} = 0.20$$

except that it need not exceed the value from *Provisions* Eq. 5.4.1.1-2 [5.2-3] as follows:

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.52}{0.189\left(5/1\right)} = 0.55$$

where T is the fundamental period of the building computed using the approximate method of *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.02)\left(20.0\right)^{0.75} = 0.189 \text{ sec}$$

Therefore, use C_s = 0.20, which is larger than the minimum specified in *Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*]:

$C_s = 0.044IS_{DS}$ = (0.044)(1.0)(1.0) = 0 .044

[The minimum C_s value has been changed to 0.01 in. the 2003 *Provisions*.

The total seismic base shear is then calculated using *Provisions* Eq. 5.4-1 [5.2-1] as:

$V = C_s W = (0.20)(374) = 74.8$ kips

7.3.4.3 Horizontal Shear Distribution and Torsion

Torsion is not considered in the shear distribution in buildings with flexible diaphragms. The shear along each side of the building will be equal, based on a tributary area force distribution.

7.3.4.3.1 Longitudinal Direction

The total shear along each side of the building is $V/2 = 37.4$ kips. The maximum shear on longitudinal panels (at the side with the openings) is:

$V_{lu} = 37.4/11 = 3.4$ kips

On each side, each longitudinal wall panel resists the same shear force as shown in the free-body diagram of Figure 7.3-2, where D_1 represents roof joist reactions and D_2 is the panel weight.

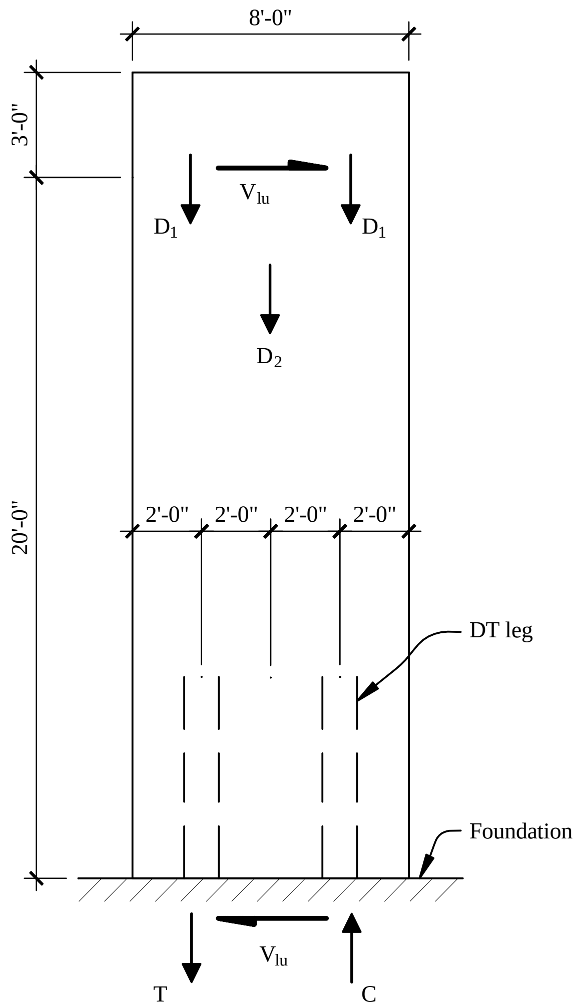


Figure 7.3-2 Free-body diagram of a panel in the longitudinal direction (1.0 ft = 0.3048 m).

7.3.4.3.2 Transverse Direction

Seismic forces on the transverse wall panels are all equal and are:

V_{tu} = 37.4/12 = 3.12 kips

Figure 7.3-3 shows the transverse wall panel free-body diagram.

Note the assumption of uniform distribution to the wall panels in a line requires that the roof diaphragm be provided with a collector element along its edge. The chord designed for diaphragm action in the perpendicular direction will normally be capable of fulfilling this function, but an explicit check should be made in the design.

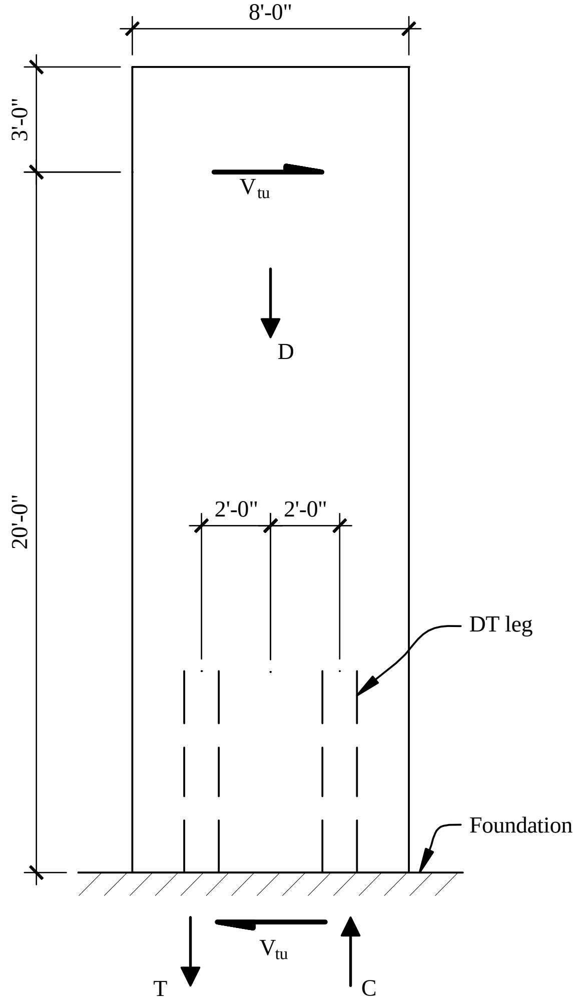


Figure 7.3-3 Free-body diagram of a panel in the transverse direction (1.0 ft = 0.3048 m).

7.3.5 Proportioning and Detailing

The strength of members and components is determined using the strengths permitted and required in ACI 318 including Chapter 21.

7.3.5.1 Tension and Shear Forces at the Panel Base

Design each precast shear panel to resist the seismic overturning moment by means of a ductile tension connector at the base of the panel. A steel angle connector will be provided at the connection of each leg of the DT panel to the concrete footing. The horizontal leg of the angle is designed to yield in bending as needed in an earthquake. *Provisions* Sec. 9.1.1.12 [not applicable in the 2003 *Provisions*] requires that dry connections at locations of nonlinear action comply with applicable requirements of monolithic concrete construction and satisfy the following:

1. Where the moment action on the connection is assumed equal to M_{pr}, the co-existing shear on the connection shall be no greater than $0.5S_{nConnection}$ and

2. The nominal shear strength for the connection shall not be less than the shear strengths of the members immediately adjacent to that connection.

Precisely how ductile dry connections emulate monolithic construction is not clearly explained. The dry connections used here do meet the definition of a yielding steel element at a connection contained in ACI 318-02. For the purposes of this example, these two additional requirements are interpreted as:

1. When tension from the seismic overturning moment causes 1.25 times the yield moment in the angle, the horizontal shear on this connection shall not exceed one-half the nominal shear strength of the connection. For this design, one-half the total shear will be resisted by the angle at the DT leg in tension and the remainder by the angle at the DT leg in compression.

2. The nominal shear strength of the connections at the legs need to be designed to exceed the in-plane shear strength of the DT.

Determine the forces for design of the DT connection at the base.

7.3.5.1.1 Longitudinal Direction

Use the free-body diagram shown in Figure 7.3-2. The maximum tension for the connection at the base of the precast panel to the concrete footing is governed by the seismic overturning moment and the dead loads of the panel and the roof. The weight for the roof is 11.2 psf, which excludes the joist girders and columns.

At the base

M_E = (3.4 kips)(20 ft) = 68.0 ft-kips

Dead loads

$$D_1 = (11.2/1{,}000)\left(\frac{48}{2}\right)4 = 1.08 \text{ kips}$$

D_2 = 0.042(23)(8) = 7.73 kips
ΣD = 2(1.08) + 7.73 = 9.89 kips
1.4D = 13.8 kips
0.7 D = 6.92 kips

Compute the tension force due to net overturning based on an effective moment arm, d = 4.0 ft (distance between the DT legs). The maximum is found when combined with 0.7D:

$T_u = M_E/d - 0.7D/2 = 68.0/4 - 6.92/2 = 13.5$ kips

7.3.5.1.2 Transverse Direction

For the transverse direction, use the free-body diagram of Figure 7.3-3. The maximum tension for connection at the base of the precast panel to the concrete footing is governed by the seismic overturning moment and the dead loads of just the panel. No load from the roof is included, since it is negligible.

At the base

M_E = (3.12 kips)(20 ft) = 62.4 ft-kips

The dead load of the panel (as computed above) is D_2 = 7.73 kips, and 0.7D = 5.41.

The tension force is computed as above for d = 4.0 ft (distance between the DT legs):

$T_u = 62.4/4 - 5.41/2 = 12.9$ kips

This tension force is less than that at the longitudinal wall panels. Use the tension force of the longitudinal wall panels for the design of the angle connections.

7.3.5.2 Panel Reinforcement

Check the maximum compressive stress in the DT leg for the requirement of transverse boundary element reinforcement per ACI 318 Sec. 21.6.6.3 [21.7.6.3]. Figure 7.3-4 shows the cross section used. The section is limited by the area of dry-pack under the DT at the footing.

The reason to limit the area of dry-pack at the footing is to locate the boundary elements in the legs of the DT, at least at the bottom of the panel. The flange between the legs of the DT is not as susceptible to cracking during transportation as are the corners of DT flanges outside the confines of the legs. The compressive stress due to the overturning moment at the top of the footing and dead load is:

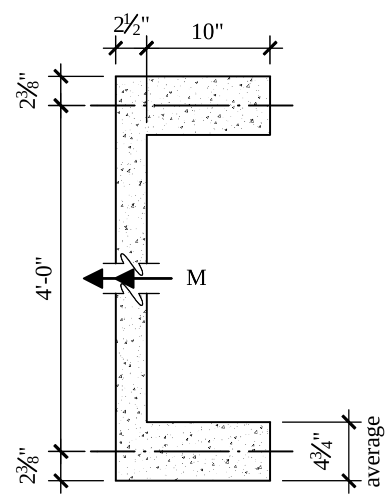


Figure 7.3-4 Cross section of the DT dry-packed at the footing (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

A = 227 in.^{2}
S = 3240 in.^{3}

$$\sigma = \frac{P}{A} + \frac{M_E}{S} = \frac{13{,}800}{227} + \frac{12(68{,}000)}{3{,}240} = 313\,\text{psi}$$

Roof live loads need not be included as a factored axial load in the compressive stress check, but the force from the prestress steel will be added to the compression stress above because the prestress force will be effective a few feet above the base and will add compression to the DT leg. Each leg of the DT will be reinforced with one 1/2-in. diameter and one 3/8-in. diameter strand. Figure 7.3-5 shows the location of these prestressed strands.

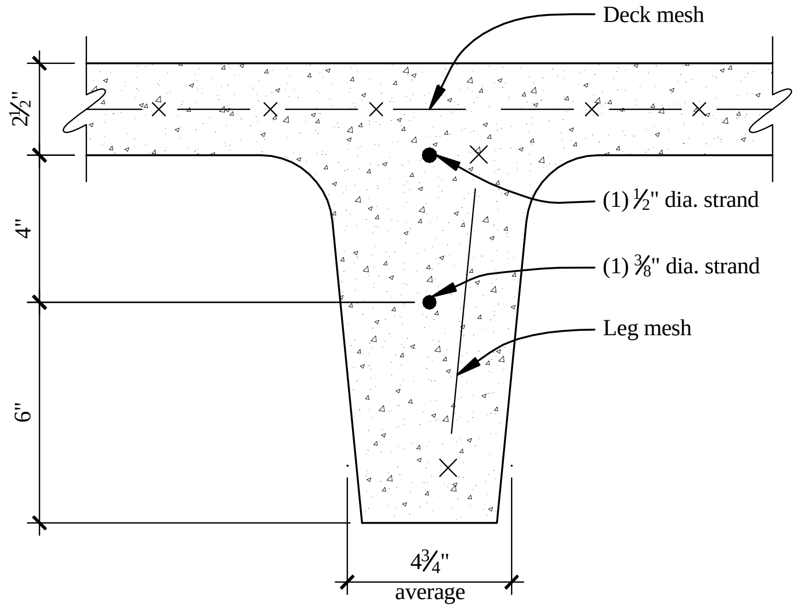


Figure 7.3-5 Cross section of one DT leg showing the location of the bonded prestressing tendons or strand (1.0 in = 25.4 mm).

Next, compute the compressive stress resulting from these strands. Note the moment at the height of strand development above the footing, about 26 in. for the effective stress (f_{se}), is less than at the top of footing. This reduces the compressive stress by:

$$\frac{(3.4)(26)}{3{,}240} \text{ x } 1000 = 27 \text{ psi}$$

In each leg, use

$P = 0.58 f_{pu} A_{ps}$ = 0.58(270 ksi)[0.153 + 0.085] = 37.3 kips
A = 168 in.^{2}
$e = y_b - CG_{Strand}$ = 9.48 - 8.57 = 0.91 in.
S_b = 189 in.^{3}

$$\sigma = \frac{P}{A} + \frac{Pe}{S} = \frac{37{,}300}{168} + \frac{0.91(37{,}300)}{189} = 402 \text{ psi}$$

Therefore, the total compressive stress is approximately 313 + 402 - 27 = 688 psi.

The limiting stress is 0.2 f_c', which is 1000 psi, so no special boundary elements are required in the longitudinal wall panels.

Reinforcement in the DT for tension is checked at 26 in. above the footing. The strand reinforcement of the DT leg resisting tension is limited to 60,000 psi. The rationale for using this stress is discussed at the beginning of this example.

D_2 = (0.042)(20.83)(8) = 7.0 kips

P_{min} = 0.7(7.0 + 2(1.08)) = 6.41 kips
M_E = (3.4)(17.83) = 60.6 ft-kips
$T_u = M_{net}/d - P_{min}/2$= 12.0 kips

The area of tension reinforcement required is:

$A_s = T_u/\phi f_y$ = (12.0 kips)/[0.9(60 ksi)] = 0.22 in.^{2}

The area of one ½ in. diameter and one 3/8 in. diameter strand is 0.153 in.^{2} + 0.085 in.^{2} = 0.236 in.^{2} The mesh in the legs is available for tension resistance, but not required in this check.

To determine the nominal shear strength of the concrete for the connection design, complete the shear calculation for the panel in accordance with ACI Sec. 21.6 [21.7]. The demand on each panel is:

$V_u = V_{lu}$ = 3.4 kips

Only the deck between the DT legs is used to resist the in-plane shear (the legs act like flanges, meaning that the area effective for shear is the deck between the legs). First, determine the minimum required shear reinforcement based on ACI Sec. 21.6.2.1 [21.7.2]. Since

$$A_{cv}\sqrt{f_c'} = 2.5(48)\sqrt{5{,}000} = 8.49 \text{ kips}$$

exceeds V_u = 3.4 kips, the reinforcement of the deck is per ACI 318 Sec. 16.4.2. Using welded wire fabric, the required areas of reinforcement are:

$A_{sh} = A_{sv}$ = (0.001)(2.5)(12) = 0.03 in.^{2}/ft

Provide 6 × 6 - W2.5 × W2.0 welded wire fabric.

A_{sh} = 0.05 in.^{2}/ft
A_{sv} = 0.04 in.^{2}/ft

The nominal shear strength of the wall panel by ACI 318 Sec. 21.6.4.1 is:

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right) = (2.5)(48)\frac{2\sqrt{5{,}000}}{1{,}000} + 0.05(4)(60) = 29.0\,\text{kips}$$

where α_c is 2.0 for h_w/l_w = 23/4 = 5.75, which is greater than 2.0. Given that the connections will be designed for a shear of 29 kips, it is obvious that half the nominal shear strength will exceed the seismic shear demand, which is 3.4 kips.

The prestress force and the area of the DT legs are excluded from the calculation of the nominal shear strength of the DT wall panel. The prestress force is not effective at the base, where the connection is, and the legs are like the flanges of a channel, which are not effective in shear.

7.3.5.3 Size the Yielding Angle

The angle, which is the ductile element of the connection, is welded between the plates embedded in the DT leg and the footing. This angle is a L5 × 3-1/2 × 3/4 × 0 ft-5 in. with the long leg vertical. The steel for the angle and embedded plates will be ASTM A572, Grade 50. The horizontal leg of the angle needs to be long enough to provide significant displacement at the roof, although this is not stated as a requirement in either the *Provisions* or ACI 318. This will be examined briefly here. The angle and its welds are shown in Figure 7.3-6.

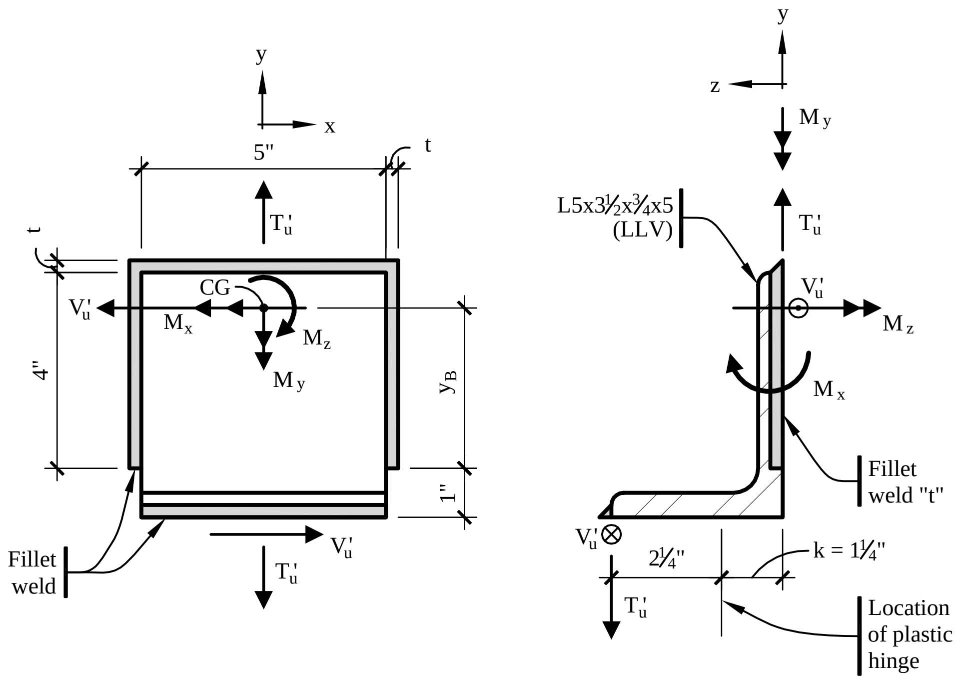


Figure 7.3-6 Free-body of the angle and the fillet weld connecting the embedded plates in the DT and the footing (elevation and section) (1.0 in = 25.4 mm).

The bending moment at a distance *k* from the heel of the angle (location of the plastic hinge in the angle) is:

$$M_u = T_u(3.5 - k) = 13.5(3.5 - 1.25) = 30.4 \text{ in.-kips}$$

$$\phi_b M_n = 0.9 F_y Z = 0.9(50)\left[\frac{5(0.75)^2}{4}\right] = 31.6 \text{ in.-kips}$$

Providing a stronger angle (e.g., a shorter horizontal leg) will simply increase the demands on the remainder of the assembly. Using *Provisions* Sec. 9.1.1.12 (ACI Sec. 21.11.6.5) [not applicable in the 2003 *Provisions*], the tension force for the remainder of this connection other than the angle is based upon a probable strength equal to 140% of the nominal strength. Thus

$$T_u' = \frac{M_n(1.4)}{3.5 - k} = \frac{(50)(5)(0.75)^2/4}{3.5 - 1.25} \times 1.4 = 21.9 \text{ kips}$$

Check the welds for the tension force of 21.9 kips and a shear force (V_u')of 29.0/2 = 14.5 kips, or the shear associated with T_u', whichever is greater. The bearing panel, with its larger vertical load, will give a larger shear.

$1.4D$ = 13.8 kips, and $V = [T_u'(4) + 1.4D(2)]/20 = [21.9/4 + 13.8(2)]/20 = 5.76$ kips. V_n for the panel obviously controls.

But before checking the welds, consider the deformability of the system as controlled by the yielding angle. Ignore all sources of deformation except the angle. (This is not a bad assumption regarding the double tee itself, but other aspects of the connections, particularly the plate and reinforcement embedded in the DT, will contribute to the overall deformation. Also, the diaphragm deformation will overwhelm all other aspects of deformation, but this is not the place to address flexible diaphragm issues.) The angle deformation will be idealized as a cantilever with a length from the tip to the center of the corner, then upward to the level of the bottom of the DT, which amounts to:

L = 3.5 in. - t/2 + 1 in. - t/2 = 3.75 in.

Using an elastic-plastic idealization, the vertical deformation at the design moment in the leg is

$\delta_v = TL^3/3EI$ = (13.5 kips)(3.75 in.)^3/[3(29000 ksi)(5 in.)(0.75 in.)^3/12] = 0.047 in.

This translates into a horizontal motion at the roof of 0.24 in. (20 ft to the roof, divided by the 4 ft from leg to leg at the base of the DT.) With C_d of 4, the predicted total displacement is 0.96 in. These displacements are not very large, but now compare with the expectations of the *Provisions*. The approximate period predicted for a 20-ft-tall shear wall building is 0.19 sec. Given a weight of 374 kips, as computed previously, this would imply a stiffness from the fundamental equation of dynamics:

$$T = 2\pi\sqrt{\frac{W/g}{K}} \Rightarrow K = 4\pi^2 W/(gT) = 4\pi^2 374/(386 \times 0.19) = 201 \text{ kip/in.}$$

Now, given the design seismic base shear of 74.8 kips, this would imply an elastic displacement of

δ_h = 74.8 kip / (201 kip/in.) = 0.37 in.

This is about 50% larger than the simplistic calculation considering only the angle. The bending of angle legs about their weak axis has a long history of providing ductility and, thus, it appears that this dry connection will provide enough deformability to be in the range of expectation of the *Provisions*.

7.3.5.4 Welds to Connection Angle

Welds will be fillet welds using E70 electrodes.

For the base metal, $\phi R_n = \phi(F_y)A_{BM}$.

For which the limiting stress is ϕF_y = 0.9(50) = 45.0 ksi.

For the weld metal, $\phi R_n = \phi(F_y)A_w = 0.75(0.6)70(0.707)A_w$.

For which limiting stress is 22.3 ksi.
Size a fillet weld, 5 in. long at the angle to embedded plate in the footing:

Using an elastic approach

Resultant force = $\sqrt{V^2 + T^2} = \sqrt{14.5^2 + 21.9^2} = 26.3$ kips

A_w = 26.3/22.3 = 1.18 in.^{2}

$t = A_w/l$ =1.18 in.^{2}/5 in. = 0.24 in.

For a 3/4 in. angle leg, use a 5/16 in. fillet weld. Given the importance of this weld, increasing the size to 3/8 in. would be a reasonable step. With ordinary quality control to avoid flaws, increasing the strength of this weld by such an amount should not have a detrimental effect elsewhere in the connection.

Now size the weld to the plate in the DT. Continue to use the conservative elastic method to calculate weld stresses. Try a fillet weld 5 in. long across the top and 4 in. long on each vertical leg of the angle. Using the free-body diagram of Figure 7.3-6 for tension and Figure 7.3-7 for shear, the weld moments and stresses are:

$M_x = T_u'(3.5)$ = 21.9(3.5) = 76.7 in.-kips
$M_y = V_u'(3.5)$ = (14.5)(3.5) = 50.8 in.-kips
$M_z = V_u'(y_b + 1.0)$
= 14.5(2.77 + 1.0) = 54.7 in.-kips

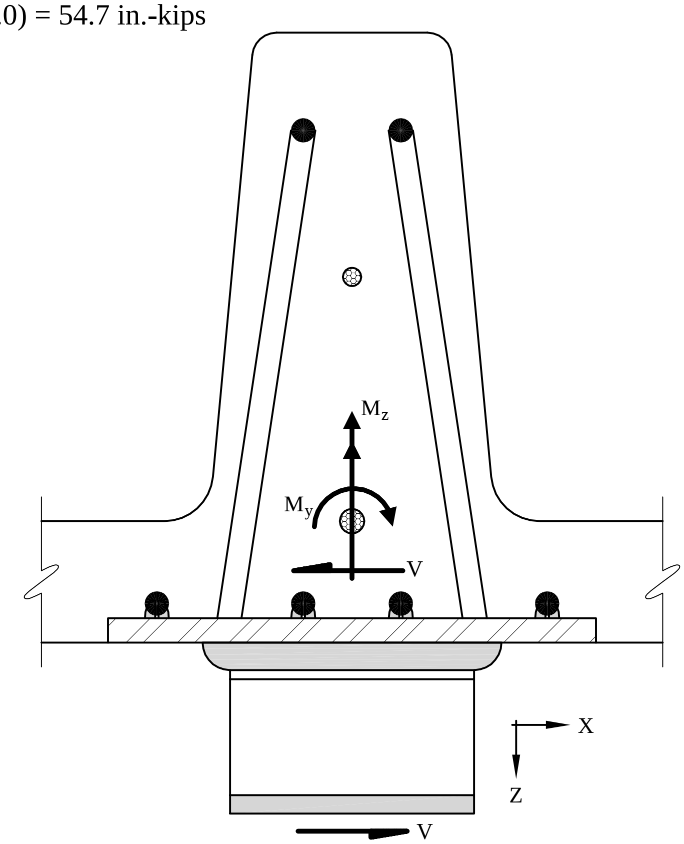


Figure 7.3-7 Free-body of angle with welds, top view, showing only shear forces and resisting moments.

For the weld between the angle and the embedded plate in the DT as shown in Figure 7.3-7 the section properties for a weld leg (t) are:

$A = 13t$ in.^{2}
$I_x = 23.0t$ in.^{4}
$I_y = 60.4t$ in.^{4}

$I_p = I_x + I_y = 83.4t$ in.^{4}
$y_b = 2.77$ in.
$x_L = 2.5$ in.

To check the weld, stresses are computed at all four ends (and corners). The maximum stress is at the lower right end of the inverted U shown in Figure 7.3-6.

$$\sigma_x = \frac{V_u'}{A} + \frac{M_z y_b}{I_p} = \frac{14.5}{13t} + \frac{(54)(2.77)}{83.4t} = \left(\frac{2.93}{t}\right)\text{ksi}$$

$$\sigma_y = -\frac{T_u'}{A} + \frac{M_z x_L}{I_p} = -\frac{21.9}{13t} + \frac{(54.7)(2.5)}{83.4t} = \left(\frac{0.045}{t}\right)\text{ksi}$$

$$\sigma_z = -\frac{M_y x_L}{I_y} - \frac{M_x y_b}{I_x} = -\frac{(50.8)(2.5)}{60.4t} - \frac{(76.7)(2.77)}{23.0\text{t}} = \left(-\frac{11.3}{t}\right)\text{ ksi}$$

$$\sigma_R = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2} = \frac{1}{t}\sqrt{(2.93)^2 + (0.045)^2 + (-11.3)^2} = \left(\frac{11.67}{t}\right)\text{ksi}$$

Thus, t = 11.67/22.3 = 0.52 in., say 9/16 in. Field welds are conservatively sized with the elastic method for simplicity and to minimize construction issues.

7.3.5.5 Tension and Shear at the Footing Embedment

Reinforcement to anchor the embedded plates is sized for the same tension and shear, and the development lengths are lengthened by an additional 30%, per *Provisions* Sec. 9.1.1.12 (ACI Sec. 21.11.6.5) [not applicable in the 2003 *Provisions*]. Reinforcement in the DT leg and in the footing will be welded to embedded plates as shown in Figure 7.3-8.

The welded reinforcement is sloped to provide concrete cover and to embed the bars in the central region of the DT leg and footing. The tension reinforcement area required in the footing is:

$$A_{s,Sloped} = \frac{T_u'}{\phi f_y \cos\theta} = \frac{21.9}{0.9(60)\left(\cos 26.5^\circ\right)} = 0.45\text{ in.}^2$$

Use two #5 bars (A_s = 0.62 in.^{2}) at each embedded plate in the footing.

The shear bars in the footing will be two #4 placed on an angle of two (plus)-to-one. The resultant shear resistance is:

ϕV_n = 0.75(0.2)(2)(60)(cos 26.5°) = 16.1 kips

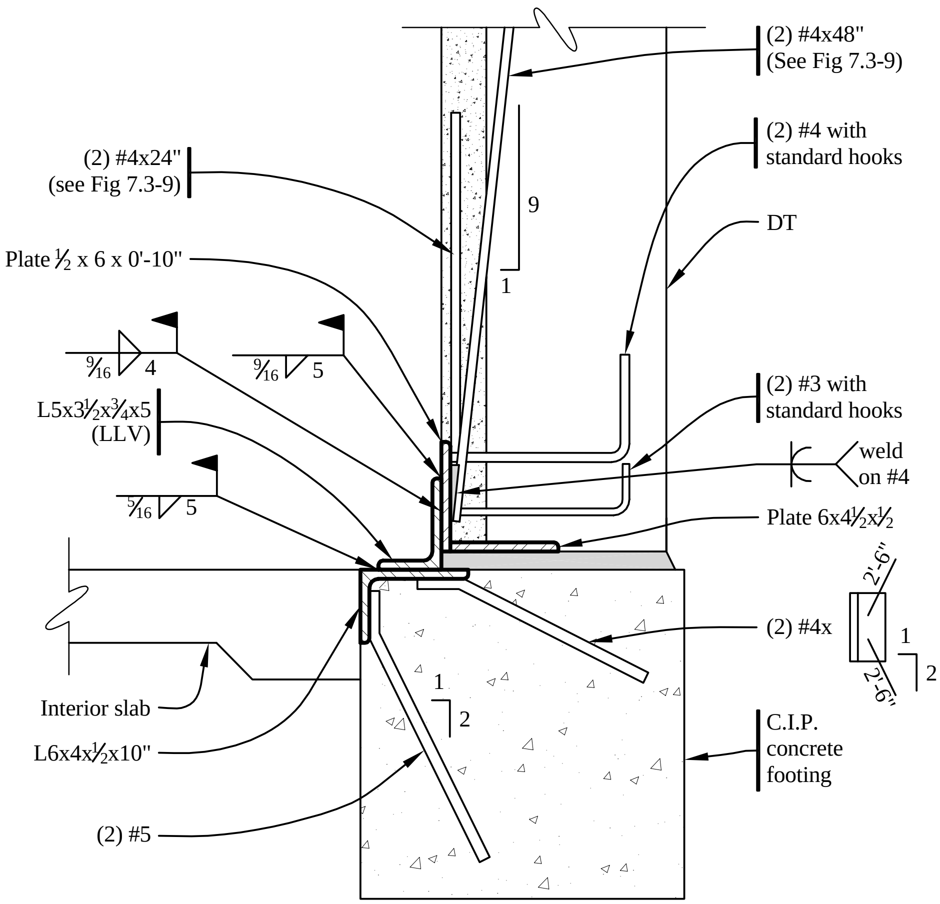


Figure 7.3-8 Section at the connection of the precast/prestressed shear wall panel and the footing (1.0 in = 25.4 mm).

7.3.5.6 Tension and Shear at the DT Embedment

The area of reinforcement for the welded bars of the embedded plate in the DT, which develop tension as the angle bends through cycles is:

$$A_s = \frac{T'_u}{\phi f_y \cos\theta} = \frac{21.9}{0.9(60)\cos 6.3^\circ} = 0.408 \text{ in.}^2$$

Two #4 bars are adequate. Note that the bars in the DT leg are required to extend upward 1.3 times the development length, which would be 22 in. In this case they will be extended 22 in. past the point of development of the effective stress in the strand, which totals about 48 in.

The same embedded plate used for tension will also be used to resist one-half the nominal shear. This shear force is 14.5 kips. The transfer of direct shear to the concrete is easily accomplished with bearing on the sides of the reinforcing bars welded to the plate. Two #5 and two #4 bars (explained later) are welded to the plate. The available bearing area is approximately A_{br} = 4(0.5 in.)(5 in.(available)) = 10 in.^{2} and the bearing capacity of the concrete is ϕV_n = (0.65)(0.85)(5 ksi)(10 in.^{2}) = 27.6 kips > 14.5 kip demand.

The weld of these bars to the plate must develop both the tensile demand and this shear force. The weld is a flare bevel weld, with an effective throat of 0.2 times the bar diameter along each side of the bar. (Refer to the PCI Handbook.) For the #4 bar, the weld capacity is

$$\phi V_n = (0.75)(0.6)(70 \text{ ksi})(0.2)(0.5 \text{ in.})(2) = 6.3 \text{ kips/in.}$$

The shear demand is prorated among the four bars as (14.5 kip)/4 = 3.5 kip. The tension demand is the larger of 1.25 f_y on the bar (15 kip) or T_u/2 (11.0 kip). The vectorial sum of shear and tension demand is 15.4 kip. Thus, the minimum length of weld is 15.4 / 6.3 = 2.4 in.

7.3.5.7 Resolution of Eccentricities at the DT Embedment

Check the twisting of the embedded plate in the DT for M_z.

Use M_z = 54.7 in.-kips.

$$A_s = \frac{M_z}{\phi f_y (jd)} = \frac{54.7}{0.9(60)(9.0)} = 0.11 \text{ in.}^2$$

Use one #4 bar on each side of the vertical embedded plate in the DT as shown in Figure 7.3-9. This is the same bar used to transfer direct shear in bearing.

Check the DT embedded plate for M_y (50.8 in.-kips) and M_x (76.7 in.-kips) using the two #4 bars welded to the back side of the plate near the corners of the weld on the loose angle and the two #3 bars welded to the back side of the plate near the bottom of the DT leg (as shown in Figure 7.3-9). It is relatively straightforward to compute the resultant moment magnitude and direction, assume a triangular shaped compression block in the concrete, and then compute the resisting moment. It is quicker to make a reasonable assumption as to the bars that are effective and then compute resisting moments about the X and Y axes. This approximate method is demonstrated here. The #4 bars are effective in resisting M_x, and one each of the #3 and #4 bars are effective in resisting M_y. For M_y assume that the effective depth extends 1 in. beyond the edge of the angle (equal to twice the thickness of the plate). Begin by assigning one-half of the "corner" #4 to each component.

With A_{sx} = 0.20 + 0.20/2 = 0.30 in.^{2} ,
$\phi M_{nx} = \phi A_s f_y jd$ = (0.9)(0.3 in.^{2})(60 ksi)(0.95)(5 in.) = 77 in.-kips (>76.7).

With A_{sy} = 0.11 + 0.20/2 = 0.21 in.^{2},
$\phi M_{ny} = \phi A_s f_y jd$ = (0.9)(0.21 in.^{2})(60 ksi)(0.95)(5 in.) = 54 in.-kips (>50.8).

Each component is strong enough, so the proposed bars are satisfactory.

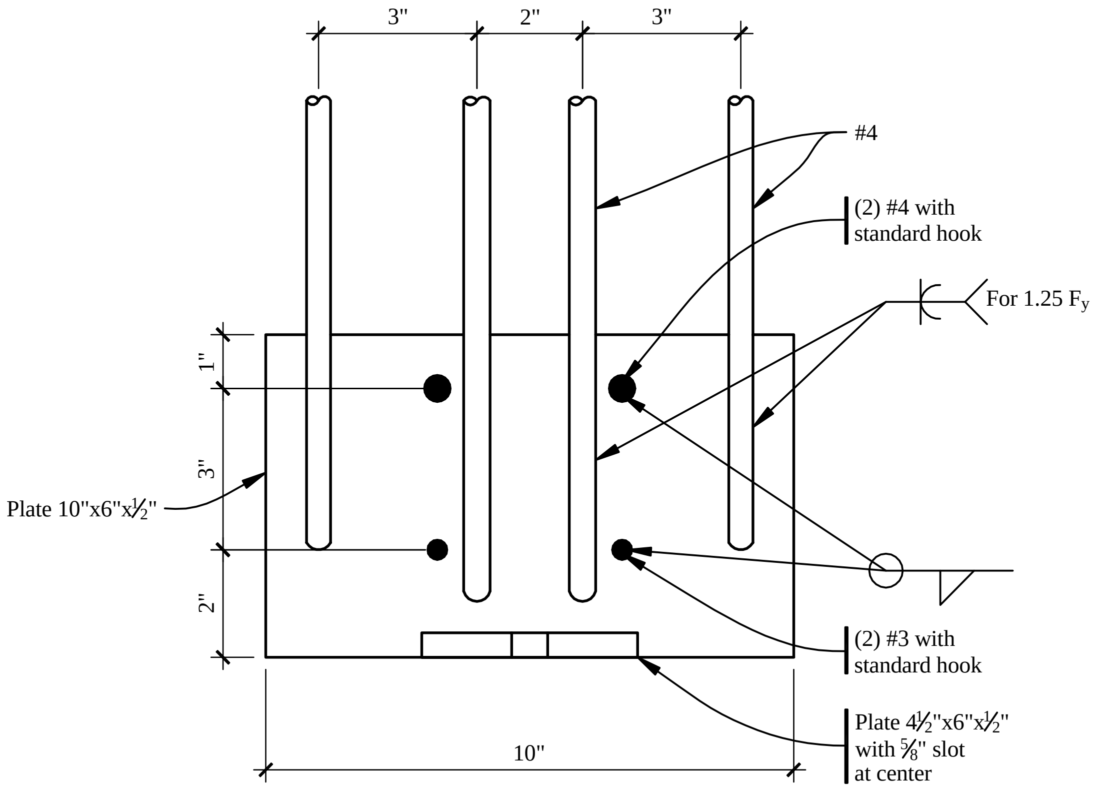


Figure 7.3-9 Details of the embedded plate in the DT at the base (1.0 in = 25.4 mm).

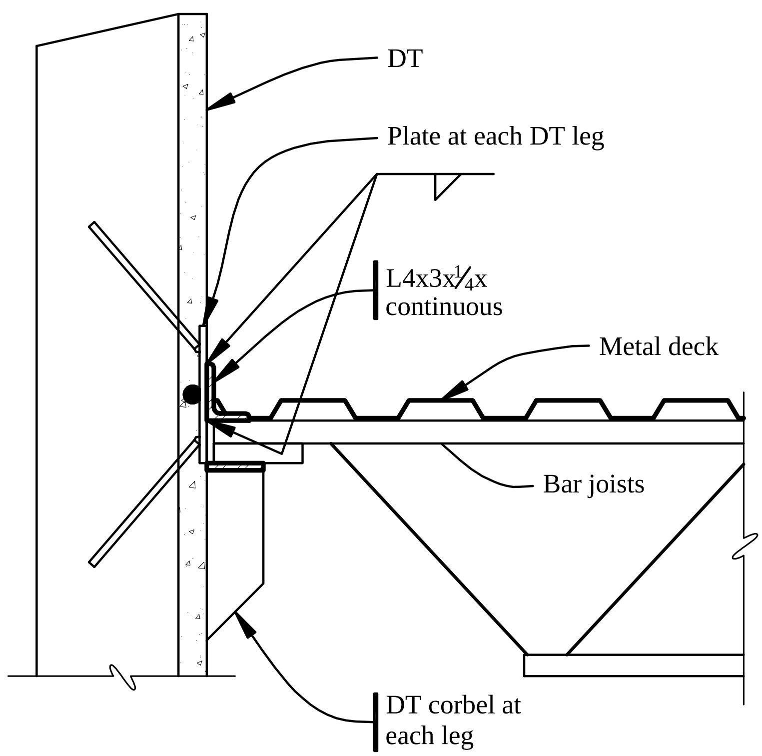


Figure 7.3-11 Sketch of connection of load-bearing DT wall panel at the roof (1.0 in = 25.4 mm).

7.3.5.8 Other Connections

This design assumes that there is no in-plane shear transmitted from panel to panel. Therefore, if connections are installed along the vertical joints between DT panels to control the out-of-plane alignment, they should not constrain relative movement in-plane. In a practical sense, this means the chord for the roof diaphragm should not be a part of the panels. Figures 7.3-10 and 7.3-11 show the connections at the roof and DT wall panels. These connections are not designed here. Note that the continuous steel angle would be expected to undergo vertical deformations as the panels deform laterally.

Because the diaphragm supports concrete walls out of their plane, *Provisions* Sec. 5.2.6.3.2 [4.6.2.1] requires specific force minimums for the connection and requires continuous ties across the diaphragm. Also, it specifically prohibits use of the metal deck as the ties in the direction perpendicular to the deck span. In that direction, the designer may wish to use the top chord of the bar joists, with an appropriate connection at the joist girder, as the continuous cross ties. In the direction parallel to the deck span, the deck may be used but the laps should be detailed appropriately.

In precast double tee shear wall panels with flanges thicker than 2-1/2 in., consideration may be given to using vertical connections between the wall panels to transfer vertical forces resulting from overturning moments and thereby reduce the overturning moment demand. These types of connections are not considered here, since the uplift force is small relative to the shear force and cyclic loading of bars in thin concrete flanges is not always reliable in earthquakes.

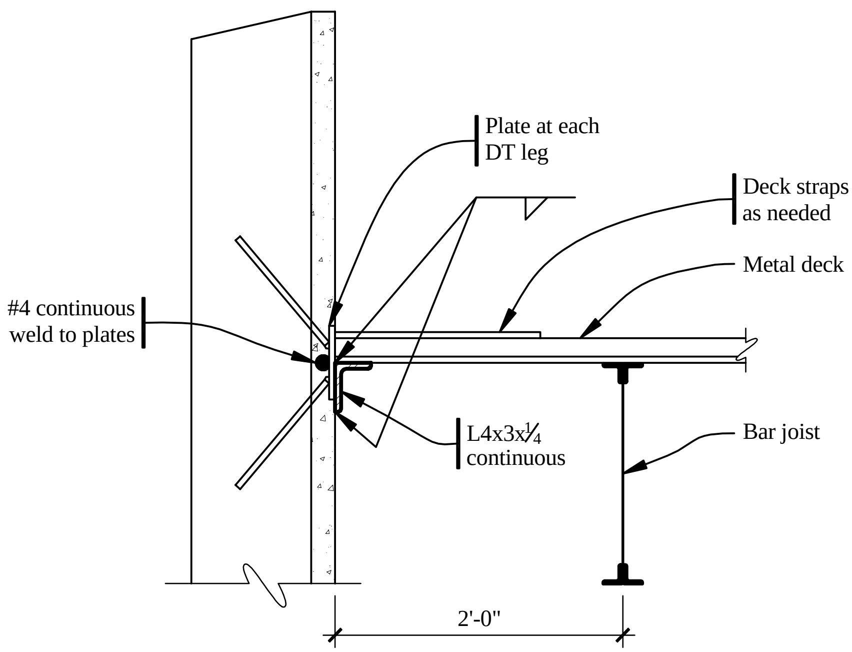


Figure 7.3-10 Sketch of connection of non-load-bearing DT wall panel at the roof (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

8

COMPOSITE STEEL AND CONCRETE

James Robert Harris, P.E., Ph.D. and
Frederick R. Rutz, P.E., Ph.D.

This chapter illustrates application of the 2000 *NEHRP Recommended Provisions* to the design of composite steel and concrete framed buildings using partially restrained composite connections. This system is referred to as a "Composite Partially Restrained Moment Frame (C-PRMF)" in the *Provisions*. An example of a multistory medical office building in Denver, Colorado, is presented. The *Provisions* set forth a wealth of opportunities for designing composite steel and concrete systems, but this is the only one illustrated in this set of design examples.

The design of partially restrained composite (PRC) connections and their effect on the analysis of frame stiffness are the aspects that differ most significantly from a non-composite design. Some types of PRC connections have been studied in laboratory tests and a design method has been developed for one in particular, which is illustrated in this example. In addition, a method is presented by which a designer using readily available frame analysis programs can account for the effect of the connection stiffness on the overall frame.

The example covers only design for seismic forces in combination with gravity, although a check on drift from wind load is included.

The structure is analyzed using three-dimensional static methods. The RISA 3D analysis program, v.4.5 (Risa Technologies, Foothill Ranch, California) is used in the example.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

Chapter 10 in the 2003 *Provisions* has been expanded to include modifications to the basic reference document, AISC Seismic, Part II. These modifications are generally related to maintaining compatibility between the *Provisions* and the most recent editions of the ACI and AISC reference documents and to incorporate additional updated requirements. Updates to the reference documents, in particular AISC Seismic, have some affect on the calculations illustrated herein.

There are not any general technical changes to other chapters of the 2003 *Provisions* that have a significant effect on the calculations and/or design example in this chapter of the *Guide* with the possible exception of the updated seismic hazard maps.

Where they affect the design examples in this chapter, significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the 2000 *NEHRP Recommended Provisions* (referred to herein as the *Provisions*), the following documents are referenced:

ACI 318 — American Concrete Institute. 1999. *Building Code Requirements for Structural Concrete*, Standard ACI 318-99. Detroit: ACI.

AISC LRFD — American Institute of Steel Construction. 1999. *Load and Resistance Factor Design Specification for Structural Steel Buildings*. Chicago: AISC.

AISC Manual — American Institute of Steel Construction. 1998. *Manual of Steel Construction, Load and Resistance Factor Design*, Volumes 1 and 2, 2nd Edition. Chicago: AISC.

AISC Seismic — American Institute of Steel Construction. 1997. *Seismic Provisions for Structural Steel Buildings*, including Supplement No. 2 (2000). Chicago:

AISC SDGS-8 — American Institute of Steel Construction. 1996. *Partially Restrained Composite Connections*, Steel Design Guide Series 8. Chicago: AISC.

ASCE TC — American Society of Civil Engineers Task Committee on Design Criteria for Composite Structures in Steel and Concrete. October 1998. "Design Guide for Partially Restrained Composite Connections," *Journal of Structural Engineering* 124(10)..

ASCE 7 — American Society of Civil Engineers. 1998. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98. Reston: ASCE.

The short-form designations presented above for each citation are used throughout.

The symbols used in this chapter are from Chapter 2 of the *Provisions*, the above referenced documents, or are as defined in the text. Customary U.S. units are used.

8.1 BUILDING DESCRIPTION

This four-story medical office building has a structural steel framework (see Figures 8-1 through 8-3). The floors and roof are supported by open web steel joists. The floor slab is composite with the floor girders and the spandrel beams and the composite action at the columns is used to create moment resisting connections. Figure 8-4 shows the typical connection. This connection has been studied in several research projects over the past 15 years and is the key to the building's performance under lateral loads. The structure is free of irregularities both in plan and elevation. This is considered a Composite Partially Restrained Moment Frame (C-PRMF) per *Provisions* Table 5.2.2 and in AISC Seismic, and it is an appropriate choice for buildings with low-to-moderate seismic demands, which depend on the building as well as the ground shaking hazard.

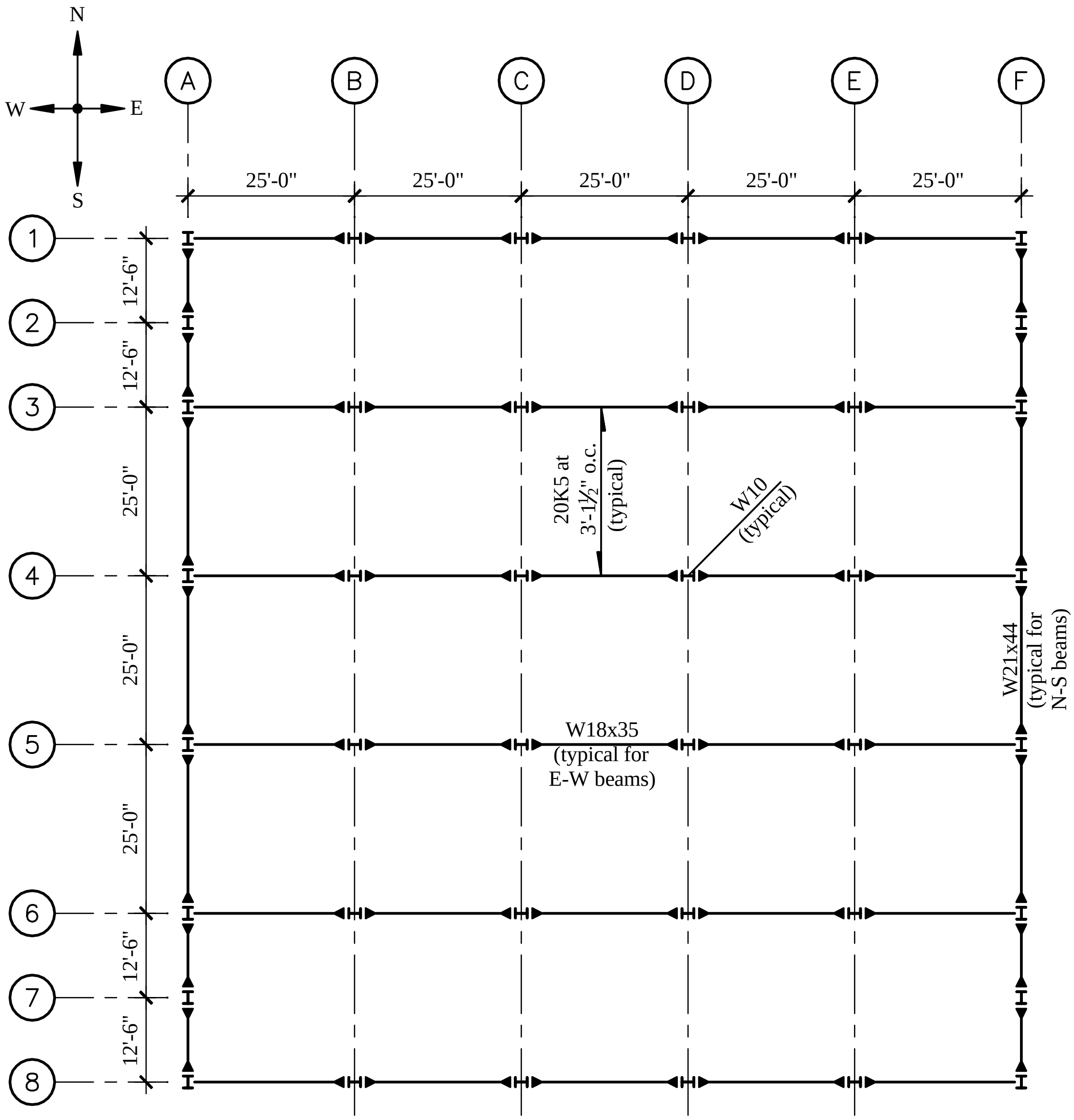


Figure 8-1 Typical floor plan (1.0 ft = 0.3048 m).

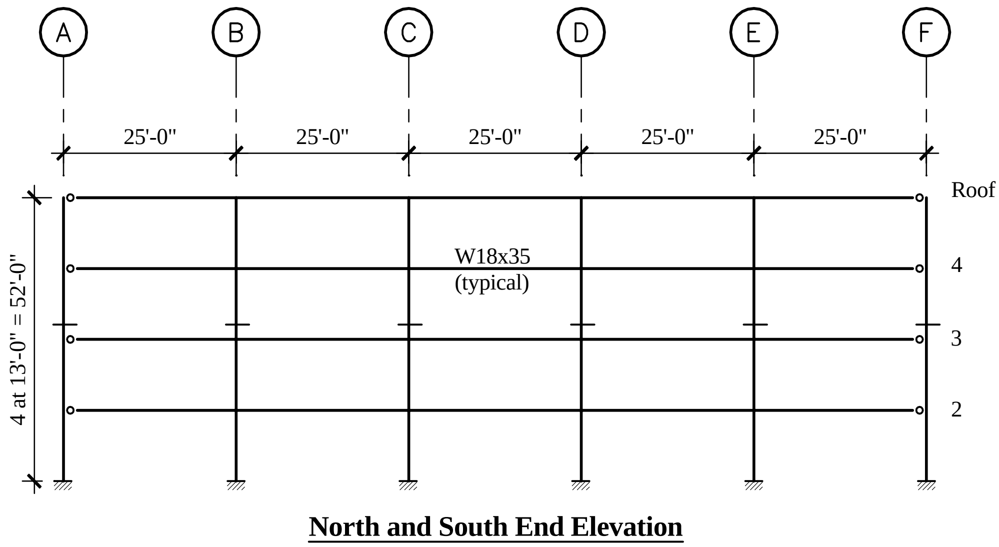


Figure 8-2 Building end elevation (1.0 ft = 0.3048 m).

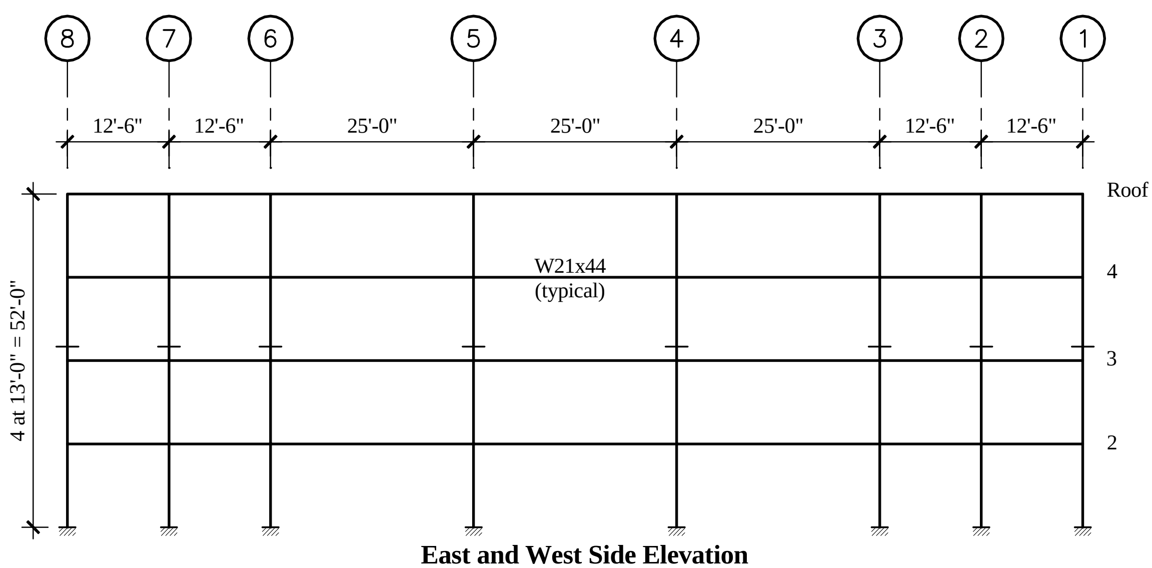


Figure 8-3 Building side elevation (1.0 ft = 0.3048 m).

The building is located in a relatively low hazard region (Denver, Colorado), but some internal storage loading and Site Class E are used in this example to provide somewhat higher seismic design forces for purposes of illustration, and to push the example into Seismic Design Category C.

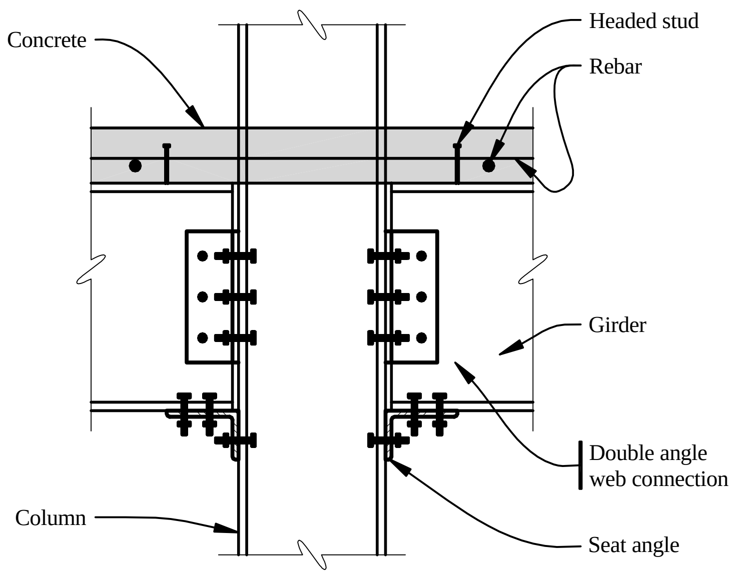


Figure 8-4 Typical composite connection.

There are no foundations designed in this example. For this location and system, the typical foundation would be a drilled pier and voided grade beam system, which would provide flexural restraint for the strong axis of the columns at their base (very similar to the foundation for a conventional steel moment frame). The main purpose here is to illustrate the procedures for the partially restrained composite connections. The floor slabs serve as horizontal diaphragms distributing the seismic forces, and by inspection they are stiff enough to be considered as rigid.

The typical bay spacing is 25 feet. Architectural considerations allowed an extra column at the end bay of each side in the north-south direction, which is useful in what is the naturally weaker direction. The exterior frames in the north-south direction have moment-resisting connections at all columns. The frames in each bay in the east-west direction have moment-resisting connections at all except the end columns. Composite connections to the weak axis of the column are feasible, but they are not required for this design. This arrangement is illustrated in the figures.

Material properties in this example are as follows:

1. Structural steel beams and columns (ASTM A992): F_y = 50 ksi
2. Structural steel connection angles and plates (ASTM A36): F_y = 36 ksi
3. Concrete slab (4.5 inches thick on form deck, normal weight): f_c' = 3000 psi
4. Steel reinforcing bars (ASTM A615): F_y = 60 ksi

The floor live load is 50 psf, except in 3 internal bays on each floor where medical records storage imposes 200 psf, and the roof snow load is taken as 30 psf. Wind loads per ASCE 7 are also checked, and the stiffness for serviceability in wind is a factor in the design. Dead loads are relatively high for a steel building due to the 4.5" normal weight concrete slab used to control footfall vibration response of the open web joist system and the precast concrete panels on the exterior walls.

This example covers the following aspects of seismic design that are influenced by partially restrained composite frame systems:

1. Load combinations for composite design
2. Assessing the flexibility of the connections
3. Incorporating the connection flexibility into the analytical model of the building

4. Design of the connections

8.2 SUMMARY OF DESIGN PROCEDURE FOR COMPOSITE PARTIALLY RESTRAINED MOMENT FRAME SYSTEM

For buildings with low to moderate seismic demands, the partially restrained composite frame system affords an opportunity to create a seismic-force-resisting system in which many of the members are the same size as would already be provided for gravity loads. A reasonable preliminary design procedure to develop member sizes for a first analysis is as follows:

1. Proportion composite beams with heavy noncomposite loads based upon the demand for the unshored construction load condition. For this example, this resulted in W18x35 beams to support the open web steel joists.

2. Proportion other composite beams, such as the spandrel beams in this example, based upon judgment. For this example, the first trial was made using the same W18x35 beam.

3. Select a connection such that the negative moment strength is about 75 percent of the plastic moment capacity of the bare steel beam.

4. Proportion columns based upon a simple portal analogy for either stiffness or strength. If stiffness is selected, keep the column's contribution to story drift to no more than one-third of the target. If strength is selected, an approximate effective column length factor of $K = 1.5$ is suggested for preliminary design. Also check that the moment capacity of the column (after adjusting for axial loads) is at least as large as that for the beam.

Those final design checks that are peculiar to the system are explained in detail as the example is described. The key difference is that the flexibility of the connection must be taken into account in the analysis. There are multiple ways to accomplish this. Some analytical software allows the explicit inclusion of linear, or even nonlinear, springs at each end of the beams. Even for software that does not, a dummy member can be inserted at each end of each beam that mimics the connection behavior. For this example another method is illustrated, which is consistent with the overall requirements of the *Provisions* for linear analysis. The member properties of the composite beam are altered to become an equivalent prismatic beam that gives approximately the same flexural stiffness in the sway mode to the entire frame as the actual composite beams combined with the actual connections. Prudence in the use of this simplification does suggest checking the behavior of the connections under gravity loads to assure that significant yielding is confined to the seismic event.

Once an analytic model is constructed, the member and connection properties are adjusted to satisfy the overall drift limits and the individual strength limits. This is much like seismic design for any other frame system. Column stability does need to account for the flexibility of the connection, but the AISC LRFD and the *Provisions* approaches considering second order moments from the translation of gravity loads are essentially the same. The further checks on details, such as the strong column rule, are also generally familiar. Given the nature of the connection, it is also a good idea to examine behavior at service loads, but there are not truly standard criteria for this.

8.3 DESIGN REQUIREMENTS

8.3.1 *Provisions* Parameters

The basic parameters affecting the design and detailing of the buildings are shown in Table 8.1 below.

Table 8-1 Design Parameters

Parameter	Value
S_s (Map 1)	0.20
S_1 (Map 2)	0.06
Site Class	E
F_a	2.5
F_v	3.5
$S_{MS} = F_aS_s$	0.50
$S_{M1} = F_vS_1$	0.21
$S_{DS} = 2/3S_{MS}$	0.33
$S_{D1} = 2/3S_{M1}$	0.14
Seismic Design Category	C
Frame Type per *Provisions* Table 5.2.2	Composite Partially Restrained Moment Frame
R	6
Ω_0	3
C_d	5.5

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

The frames are designed in accordance with AISC Seismic, Part II, Sec. 8 (*Provisions* Table 5.2.2). AISC SDGS-8 and ASCE TC describe this particular system in detail. Given the need to determine the flexibility of the connections, it would be difficult to design such structures without reference to at least one of these two documents.

8.3.2 Structural Design Considerations Per the *Provisions*

The building is regular both in plan and elevation. *Provisions* Table 5.2.5.1 indicates that use of the Equivalent Lateral Force procedure in accordance with *Provisions* Sec. 5.4 is permitted.

Nonstructural elements (*Provisions* Chapter 14) are not considered in this example.

Diaphragms must be designed for the required forces (*Provisions* Sec. 5.2.6.2.6), however this is not unique to this system and therefore is not explained in this example.

The story drift limit (*Provisions* Table 5.2.8) is 0.025 times the story height. Although the C_d factor is large, 5.5, the seismic forces are low enough that conventional stiffness rules for wind design actually control the stiffness.

Orthogonal effects need not be considered for Seismic Design Category C, provided the structure does not have a plan structural irregularity (*Provisions* Sec. 5.2.5.2.2).

8.3.3 Building Weight and Base Shear Summary

The unit weights are as follows:

Non-composite dead load:	
4.5 in. slab on 0.6 in. form deck, plus sag	58 psf
Joist and beam framing	6 psf
Columns	2 psf
	66 psf
Composite dead load:	
Fire insulation	4 psf
Mechanical and electrical	6 psf
Ceiling	2 psf
Partitions	20 psf
	32 psf
Exterior wall:	
Precast concrete panels:	0.80 klf
Records storage on 3 bays per floor	120 psf

(50 percent is used for seismic weight; minimum per the *Provisions* is 25 percent)

The building weight, W, is found to be 8,080 kips. The treatment of the dead loads for analysis is described in more detail subsequently.

The Seismic Response Coefficient, C_s, is equal to 0.021:

$$C_s = \frac{S_{D1}}{T\dfrac{R}{I}} = \frac{0.14}{1.12\left(\dfrac{6}{1}\right)} = 0.021$$

The methods used to determine W and C_s are similar to those used elsewhere in this volume of design examples. The building is somewhat heavy and flexible. The computed periods of vibration in the first modes are 2.12 and 2.01 seconds in the north-south and east-west directions, respectively. These are much higher than the customary 0.1 second per story rule of thumb, but low-rise frames with small seismic force demands typically do have periods substantially in excess of the rule of thumb. The approximate period per the *Provisions* is 0.66 seconds, and the upper bound for this level of ground motion is 1.12 seconds.

The total seismic force or base shear is then calculated as follows:

$$V = C_sW = (0.021)(8{,}080) = 170 \text{ kips} \qquad (\textit{Provisions} \text{ Eq. 5.3.2})$$

The distribution of the base shear to each floor (again, by methods similar to those used elsewhere in this volume of design examples) is found to be:

Roof	(Level 4):	70 kips
Story 4	(Level 3):	57 kips
Story 3	(Level 2):	34 kips
Story 2	(Level 1):	8 kips
Story 1	(Level 0):	0 kips
	Σ:	169 kips (difference is rounding; total is 170)

Without illustrating the techniques, the gross service level wind force following ASCE 7 is 123 kips. When including the directionality effect and the strength load factor, the design wind force is somewhat less than the design seismic base shear. The wind force is not distributed in the same fashion as the seismic force, thus the story shears and the overturning moments for wind are considerably less than for seismic.

8.4 DETAILS OF THE PRC CONNECTION AND SYSTEM

8.4.1 Connection M-θ Relationships

The composite connections must resist both a negative moment and a positive moment. The negative moment connection has the slab rebar in tension and the leg of the seat angle in compression. The positive moment connection has the slab concrete in compression (at least the "a" dimension down from the top of the slab) and the seat angle in tension (which results in flexing of the seat angle vertical leg). At larger rotations the web angles contribute a tension force that increases the resistance for both negative and positive bending.

Each of these conditions has a moment-rotation relationship available in AISC SDGS-8 and ASCE TC. (Unfortunately there are typographical errors in ASCE TC: A "+" should be replaced by "=" and the symbol for the area of the seat angle is used where the symbol should be that for the area of the web angle.) An M-θ curve can be developed from these equations:

Negative moment connection:

$$M_n^- = C_1(1 - e^{-C_2\theta}) + C_3\theta \qquad \text{(AISC SDGS-8, Eq. 1)}$$

where:

$C_1 = 0.18(4 \times A_s F_{yrb} + 0.857 A_L F_y)(d + Y_3)$
$C_2 = 0.775$
$C_3 = 0.007(A_L + A_{wL})F_y\ (d + Y_3)$
θ = girder end rotation, milliradians (radians/1000)
d = girder depth, in.
Y_3 = distance from top flange of the girder to the centroid of the reinforcement, in.
A_s = steel reinforcing area, in.^{2}
A_L = area of seat angle leg, in.^{2}
A_{wL} = gross area of double web angles for shear calculations, in.^{2} (For use in these equations A_{wL} is limited to 150 percent of A_L).
F_{yrb} = yield stress of reinforcing, ksi
F_y = yield stress of seat and web angles, ksi

Positive moment connection:

$$M_n^+ = C_1(1 - e^{-C_2\theta}) + (C_3 + C_4)\theta \qquad \text{(AISC SDGS-8, Eq. 2)}$$

where:

$C_1 = 0.2400[(0.48A_{wL}) + A_L](d + Y_3)F_y$
$C_2 = 0.0210(d + Y_3/2)$
$C_3 = 0.0100(A_{wL} + A_L)(d + Y_3)F_y$
$C_4 = 0.0065\ A_{wL}\ (d + Y_3)F_y$

From these equations, curves for M-θ can be developed for a particular connection. Figures 8-5 and 8-6 are M-θ curves for the connections associated with the W18x35 girder and the W21x44 spandrel beam respectively, which are used in this example. The selection of the reinforcing steel, connection angles, and bolts are described in the subsequent section, as are the bilinear approximations shown in the figures. Among the important features of the connections demonstrated by these curves are:

1. The substantial ductility in both negative and positive bending,
2. The differing stiffnesses for negative and positive bending, and
3. The substantial post-yield stiffness for both negative and positive bending.

It should be recognized that these curves, and the equations from which they were plotted, do not reproduce the line from a single test. They are averages fit to real test data by numerical methods. They smear out the slip of bolts into bearing. (There are several articles in the *AISC Engineering Journal* that describe actual test results. They are in Vol. 24, No.2; Vol. 24, No.4; Vol. 27, No.1; Vol. 27, No. 2; and Vol 31, No. 2. The typical tests clearly demonstrate the ability of the connection to meet the rotation capabilities of AISC Seismic, Section 8.4 - inelastic rotation of 0.015 radians and total rotation capacity of 0.030 radians.)

[Based on the modifications to AISC Seismic, Part II, Sec. 8.4 in 2003 *Provisions* Sec. 10.5.16, the required rotation capabilities are inelastic rotation of 0.025 radians and total rotation of 0.040 radians.]

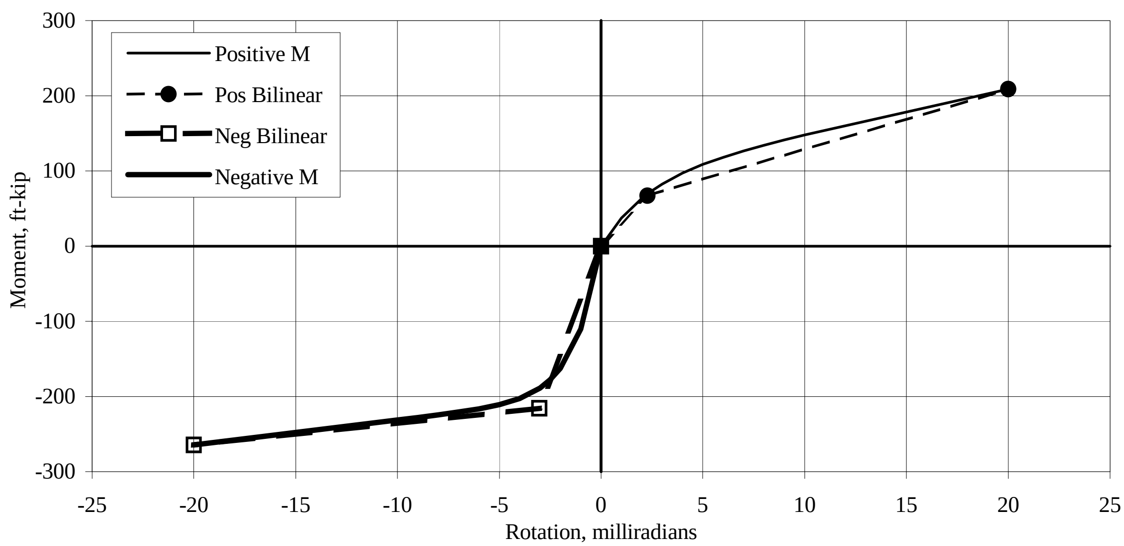


Figure 8-5 M-θ Curve for W18x35 connection with 6-#5 (1.0 ft-kip = 1.36 kN-m)

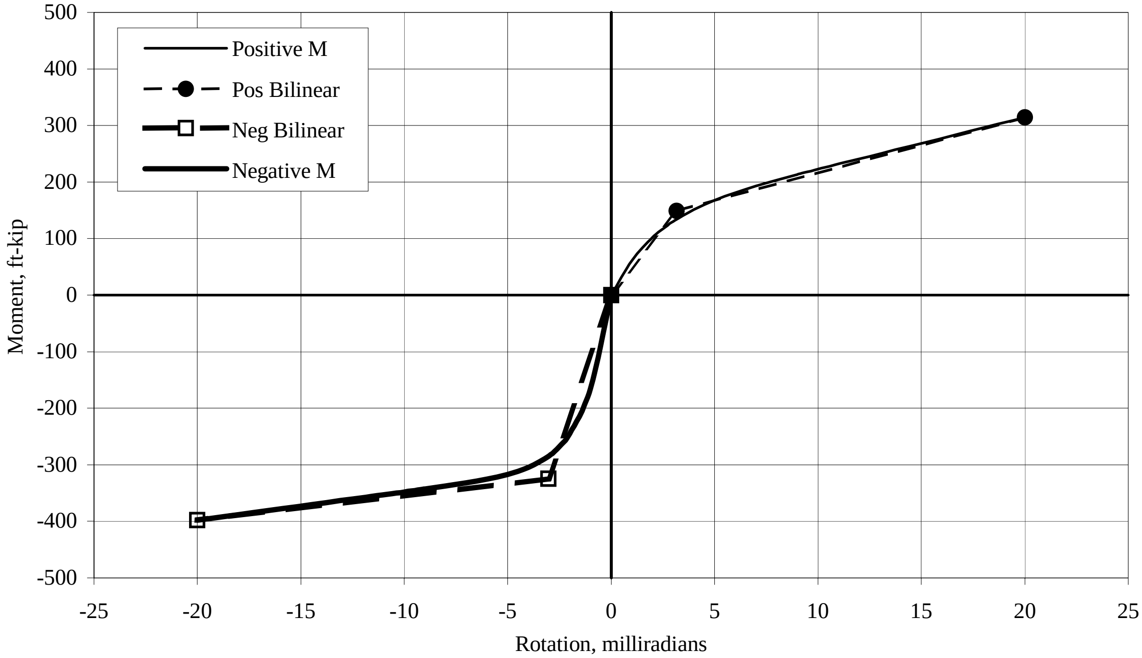


Figure 8-6 *M-θ* Curve for W21x44 connection with 8-#5 (1.0 ft-kip = 1.36 kN-m).

8.4.2 Connection Design and Connection Stiffness Analysis

Table 8-2 is taken from a spreadsheet used to compute various elements of the connections for this design example. It shows the typical W18x35 girder and the W21x44 spandrel beam with the connections used in the final analysis, as well as a W18x35 spandrel beam for the short exterior spans, where a W21x44 was used in the end. Each major step in the table is described in a line-by-line description following the table. [Based on the modifications to AISC Seismic, Part II, Sec. in 2003 *Provisions* Sec. 10.5.16, the nominal strength of the connection must be exceed R_yM_p for the bare steel beam, where R_y is the ratio of expected yield strength to nominal yield strength per AISC Seismic, Part I, Table I-6-1.]

Table 8-2 Partially Restrained Composite Connection Design

Line		Girder	Spandrels	
	Basic Data			
2	Beam size	W18x35	W21x44	W18x35
3	Span, ft	25	25	12.5
4	Area of beam, in.^{2}	10.3	13	10.3
5	I, of beam alone, in.^{4}	510	843	510
6	Z, plastic modulus of beam, in.^{3}	66.5	95.4	66.5
7	Beam depth, in.	17.7	20.7	17.7
8	Slab thickness, in.	7.0	7.0	7.0
9	Y_3 to rebar, in.	5.5	5.5	5.5
10	Column	W10x77	W10x88	W10x77
11	Flange width, in.	10.2	10.3	10.2
12	Flange thickness, in.	0.87	0.99	0.87
13	Flange fillet, k_1, in.	0.88	0.94	0.88
	Basic Negative Moment Capacity			
15	Reinforcing bars	6-#5	8-#5	6-#5
16	A_s, rebar area, in.^{2}	1.86	2.48	1.86
17	T_r, rebar tension, kips	111.6	148.8	111.6
18	M_n^-, nominal negative moment, ft-kips	215.8	324.9	215.8
19	% M_p (M_n^-/beam M_p)	78%	82%	78%
20	Check: > 50%? (75% per ASCE TC)	OK	OK	OK
	Seat Demands for Negative Moment			
22	Seat angle	L7x4x$^1/_2$x8	L7x4x$^5/_8$x8.5	L7x4x$^1/_2$x8
23	Seat F_y, ksi	36	36	36
24	Seat thickness, in.	0.5	0.625	0.5
25	Seat length, in.	8.0	8.5	8.0
26	Leg area, in.^{2}	4.0	5.3125	4.0
27	Minimum area = 1.25 T_r/F_y, in.^{2}	3.875	5.167	3.875
28	Check	OK	OK	OK
29	Leg yield force, kips	144	191.25	144
30	Bolts to beam	(4) 1"-325X	(4) 1$^1/_8$"-490X	(4) 1"-325X
31	Diameter, in.	1.0	0.875	1.0
32	Bolt design shear capacity, kips ($\phi = 0.75$)	141.2	223.6	141.2
33	Check	Close enough	OK	Close enough
	Nominal Positive Moment Capacity			
35	Seat k, fillet length, in.	1.000	1.125	1.000
36	M_p, vertical leg, in.-kips	18.0	29.9	18.0
37	b' (see Figure 8-7), in.	1.00	0.81	1.00
38	Seat tension from bending, kips	31.5	63.8	31.5
39	Seat tension from shear, kips	86.4	114.75	86.4
40	Tension to bottom flange, kips	31.5	63.8	31.5
41	Nominal Positive Moment, M_n^+, ft-kips	67.4	149.9	67.4
42	Percent of Beam M_p	24%	38%	24%
	Demand on Tension Bolts at Nominal Capacity			
44	a' (see Figure 8-7), in.	2.0	2.1	2.0
45	Q (prying), kips	6.8	10.7	6.8
46	Bolt tension, kips	38.3	74.5	38.3
47	Bolts to column	(2) 1"-325X	(2) 1$^1/_8$"-490X	(2) 1"-325X
48	Bolt design tension, kips ($\phi = 0.75$)	106	168.4	106
49	Check	OK	OK	OK

Line		Girder	Spandrels	
	Compute Total Joint Moment to Column based on Nominal Capacities			
51	Connection nominal $M_n^- + M_n^+$, ft-kips	283	475	283
52	Minimum column M_p (125% of sum)	177	297	177
53	Average as percentage of beam	51%	60%	51%
54	Check	OK	OK	OK
	Concrete Compression Transfer to Column			
56	Rebar T_y + bottom seat T_y, kips	143.10	212.62	143.10
57	0.85 f'_c on two flanges, kips	364.14	367.71	364.14
58	Projection for flange M_p, in.	2.72	3.10	2.72
59	Force from flange M_p, kips	225.92	254.88	225.92
60	Ratio, demand / minimum capacity	0.63	0.83	0.63
	Web Shear Connection (needed for effective stiffness)			
62	Seismic shear demand, kips	11.5	19.9	23.1
63	Web angles	L4x4x$^{1}/_{4}$x8.5	L4x4x$^{1}/_{4}$x11.5	L4x4x$^{1}/_{4}$x8.5
64	A_w, area of two legs, in.^{2}	4.25	5.75	4.25
65	A_w, limit based on area of rebar, in.^{2}	2.79	3.72	2.79
66	A_w, used in M-θ calculation, in.^{2}	2.79	3.72	2.79
	Moment Rotation Values for Analysis of Effective Stiffness			
68	M_{neg} at service level (0.0025 rad), ft-kip	-178.0	-267.8	-178.0
69	M_{neg} at maximum capacity(0.020 rad), ft-kip	-264.5	-397.7	-264.5
70	Secant stiffness for M_{neg} at 0.0025 radian	71.2	107.1	71.2
71	M_{pos} at service level (0.0025 rad), ft-kip	73.7	117.3	73.7
72	M_{pos} at maximum capacity(0.020 rad), ft-kip	208.9	313.9	208.9
73	Secant stiffness for M_{pos} at 0.0025 radian	29.5	46.9	29.5
74	Rotation at nominal M_{neg}	3.03	3.03	3.03
75	Rotation at nominal M_{pos}	2.29	3.70	2.29
	Beam Moments of Inertia			
77	Full composite action force, beam AF_y, kips	515.0	650.0	515.0
78	Y_2, to plastic centroid in concrete, in.	5.65	5.30	4.31
79	Composite beam inertia for pos. bending, in.^{4}	1,593	2,435	1,402
80	Centroid of all steel for negative bending, in.	6.66	7.81	6.66
81	Composite beam inertia for neg. bending, in.^{4}	834	1366	834
82	Equivalent beam for positive and negative, in.^{4}	1,290	2,008	1,175
83	Weighted connection stiffness, ft-kips/radian	61,263	88,105	61,263
84	Eff. prismatic inertia, beam and PRCC, in.^{4}	639	955	412
85	Ratio of eff. prismatic I / I of beam alone	1.25	1.13	0.81
	Check Bottom Bolt Tension at Maximum Deformation			
87	Rotation at $\phi \times$ (rotation at nominal M_{pos}) $\times C_d$	10.7	14.9	10.7
88	Moment at $\phi \times$ (rot. at nom. M_{pos}) $\times C_d$, ft-kips	152.3	268.2	152.3
89	Tension demand, kips	80.5	125.1	80.5
90	Nominal capacity of bolts, kips	141.3	224.5	141.3
	Check Positive Moment Capacity as a Percentage of Beam M_p (50% criterion)			
92	M_{pos} (at 0.020 radians) / M_p beam	75%	79%	75%

Detailed explanation of the computations in Table 8-2:

Step 1: Establish nominal negative moment capacity: (This is a step created in this design example; is not actually an explicit step in the procedures recommended in the references. It appears to be necessary to satisfy the basic *Provisions* strength requirement. See *Provisions* Sec. 5.2.1, Sec. 5.2.7, and ASCE 7 Sec. 2.3.

Lines 15-18: M_n is taken as a simple couple of rebar in slab and force at connection of bottom flange of beam; the true maximum moment is larger due to strain hardening in rebar and the bottom connection and due to tension force in the web connection, so long as the bottom connection can handle the additional demand. The nominal capacity is plotted in Figures 8-5 and 8-6 as the break of the bilinear relation. The design capacity, using a resistance factor of 0.85, has two requirements:

1. ϕM_n exceeds demand from seismic load combination: basic *Provisions* requirement

2. ϕM_n exceeds demand from total service gravity loads - simply a good idea to maintain reasonable initial stiffness for lateral loads; by "codes" the factored gravity demand can be checked using plastic analysis

Lines 19-20: M_n exceeds 50 percent (by AISC Seismic, Part II, 8.4) of M_p of the bare steel beam. In this example, the more stringent recommendation of 75 percent contained within the ASCE TC is followed. Note that this check is on nominal strength, not design strength. A larger M_n gives a larger stiffness, thus some drift problems can be addressed by increasing connection capacity.

Step 2: Design bottom seat angle connection for negative moment:

Lines 22-28: Provide nominal yield of angle leg at least 125 percent of nominal yield of reinforcing steel. This allows for increased force due to web shear connection. Strain hardening in the rebar is a factor, but strain hardening the angle would probably be as large. AISC SDGS-8 recommends 120 percent. ASCE TC recommends 133 percent, but then uses 125 percent to check the bolts. This is a check in compression, and the authors elected to use 125 percent.

Lines 29-33: Provide high strength bolts in normal (not oversized) holes to transfer force between beam flange and angle by shear; conventional rules regarding threads in the shear plane apply. The references do apply a resistance factor to the bolts, which may be an inconsistent design methodology. A check based on overstrength might be more consistent. The capacity at bolt slip could be compared against service loads, which would be a good idea for designs subject to strong wind forces.

Step 3: Establish nominal positive moment capacity: This connection is less stiff and less linear for positive moment than for negative moment, and generally weaker. There is not a simple, clear mechanism for a nominal positive moment. The authors of this example suggest the following procedure which follows the normal methods of structural engineering and yields a point relevant to the results of connection tests, in so far as construction of a bilinear approximation is concerned. It significantly underestimates the ultimate capacity.

Lines 35-38: Compute the shear in the vertical leg associated with bending. Figure 8-7 shows the mechanics, which is based on methods in the AISC Manual, for computing prying in hanger-type connections. Compute the nominal plastic moment of the angle leg bending out of plane (line 36) and assume that the location of the maximum moments are at the end of the fillet on the vertical leg (line 35) and at the edge of the bolt shaft (line 37). The moment near the bolt is reduced for the material lost at the bolt hole.

Lines 39-40: Check the shear capacity, compare with the shear governed by moment, and use the smaller. Shear will control if the angle is thick.

Line 41: Compute the nominal positive moment as a couple with the force and the distance from the bottom of the beam to the center of the compression area of the slab on the column. The concrete compression area uses the idealized Whitney stress block (ACI 318). Note that the capacity to transfer concrete compression force to the steel column flange is checked later. The nominal positive moment is also shown on Figures 8-5 and 8-6 at the break point in the bilinear relation.

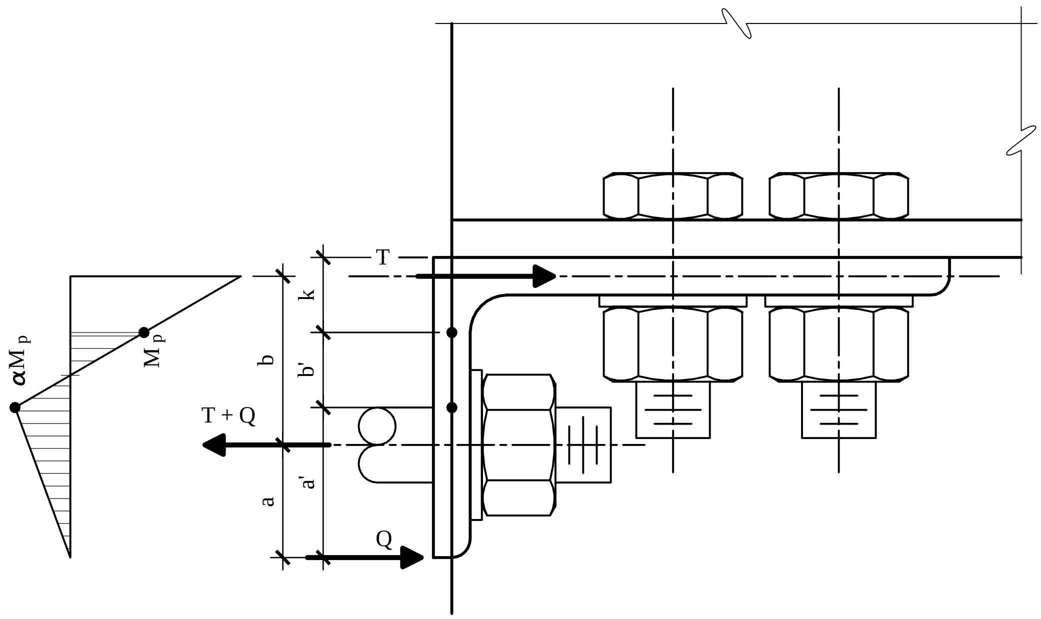


Figure 8-7 Analysis of seat angle for tension.

Step 4: Design the bolts to transfer positive moment tension to the column:

Lines 44-45: Compute the prying force following AISC's recommended method. The moment in the vertical leg is computed as described above, and the moment arm extends from the edge of the bolt shaft (closest to the beam) to the bottom edge of the angle. Refer to Figure 8-7.

Lines 46-48: Add the basic tension to the prying force and compare to the factored design capacity of the bolts. Note that the resistance factor is used here to be consistent with step 2. It is common to use the same size and grade of bolt as used for the connection to the beam flange, which generally means that these bolts have excess capacity. Also, for seismic design, another check at maximum positive moment is recommended (see step 9).

Step 5: Compute the flexural demand on the column: AISC Seismic, Part II, 7 and 8, require that the flexural resistance of the column be greater than the demand from the connections, but it does not give any particular margin. ASCE TC recommends a ratio of 1.25.

Lines 51-52: The minimum nominal flexural strength of the column, summed above and below as well as adjusted for the presence of axial load, is set to be 125 percent of the demand from the sum of the nominal strengths of the connections.

Lines 53-54: AISC Seismic, Part II, 8.4 requires that the connection capacity exceed 50 percent of the plastic moment capacity of the beam. In this example, the negative moment connections are designed for 75 percent of the beam plastic moment, and this check shows that the average of negative and positive nominal moment capacities for the connection exceeds 50 percent of the plastic moment for the beam. A later check (step 10) will compare the maximum positive moment resistance to the 50 percent rule.

Step 6: Check the transfer of force from concrete slab to steel column: The tension in the reinforcing steel and the compression couple from positive bending must both transfer. Both flanges provide resistance if concrete fills the space between the flanges, but full capacity of the second flange has probably not been exercised in tests.

Line 56: Add the yield force of the reinforcement and the tension yield force of the seat angle, both previously computed.

Line 57: Compute an upper bound concrete compression capacity as $0.85f'_c$ times the area of concrete bearing on both flanges.

Lines 58-59: Compute the force that would yield the steel column flanges over the thickness of the slab by computing the projection beyond the web fillet that would yield at a load of 0.85f'c. This ignores the capacity of the flange beyond the slab thickness and is obviously conservative.

Line 60: Compare the demand with the smaller of the two capacities just computed.

Step 7: Select the web connection:

Line 62: The seismic shear is computed by assuming beam end moments equal to the nominal capacity of the connections, one in negative moment and one in positive.

Line 63: The gravity demand must be added, and straight gravity demand must also be checked before selecting the actual connection.

Lines 64-66: The web connection influences the overall stiffness and strength of the connection, especially at large rotation angles. The moment-rotation expression include the area of steel in the web angles, but also places a limitation based upon 150 percent of the area of the leg of the seat angle for use in the computation.

Step 8: Determine the effective stiffness of the beam and connection system: Determining the equivalent stiffness for a prismatic beam involves several considerations. Figure 8-8 shows how the moment along the beam varies for gravity and lateral loads as well as composite and non-composite conditions. The moment of inertia for the composite beam varies with the sense of the bending moment. The end connections can be modeled as regions with their own moments of inertia, as illustrated in the figure. Figure 8-9 shows the effective cross section for each of the four stiffnesses: positive and negative bending of the composite beam and positive and negative bending of the composite connection. Given a linear approximation of each connection stiffness expressed as moment per radian, flexural mechanics leads to a simple expression for a moment of inertia of an equivalent prismatic beam.

Lines 68-73: Compute the negative and positive moments at a rotation of 2.5 milliradians, which is the rotation angle that defines the effective stiffness for lateral analysis (per both AISC SDGS-8 and ASCE TC).

Lines 74-75: Using those moments, compute the rotations corresponding to the nominal strength, positive and negative. (This is useful when idealizing the behavior as bilinear, which is plotted in Figures 8-5 and 8-6.)

Lines 77-79: Compute the moment of inertia of the composite beam in positive bending. Note that the system is designed for full composite action, per the recommendations in AISC SDGS-8 and ASCE TC, using the criteria in the AISC manual. The positive bending moment of inertia here is computed using AISC's lower bound method, which uses an area of steel in the flange adequate to replace the Whitney stress block in the concrete flange. This moment of inertia is less than if one used the full concrete area in Figure 8-9.

Lines 80-81: Compute the moment of inertia of the composite beam in negative bending.

Line 82: Compute an equivalent moment of inertia for the beam recognizing that a portion of the span is in positive bending and the remainder is in negative bending. Following the recommendations in AISC SDGS-8 and ASCE TC, this is computed as 60 percent of I_{pos} and 40 percent of I_{neg}.

Lines 83-84: Compute the moment of inertia of a prismatic beam that will give the same total end rotation in a sway condition as the actual system. Gravity loads place both connections in negative moment, so one will be subject to increasing negative moment while the other will be subject to decreasing negative moment. Thus, initially, the negative moment stiffness is the appropriate stiffness, which is what is recommended in the AISC SDGS-8 and ASCE TC. For this example the positive and negative stiffnesses are combined, weighted by the nominal strengths in positive and negative bending, to yield a connection stiffness that is appropriate for analysis up to the nominal strengths defined earlier. Defining this weighted stiffness as K_{conn} and the equivalent composite beam moment of inertia as I_{comp}, the effective moment of inertia is found by:

$$I_{effective} = \frac{I_{comp}}{1 + \dfrac{6EI_{comp}}{L\,K_{conn}}}$$

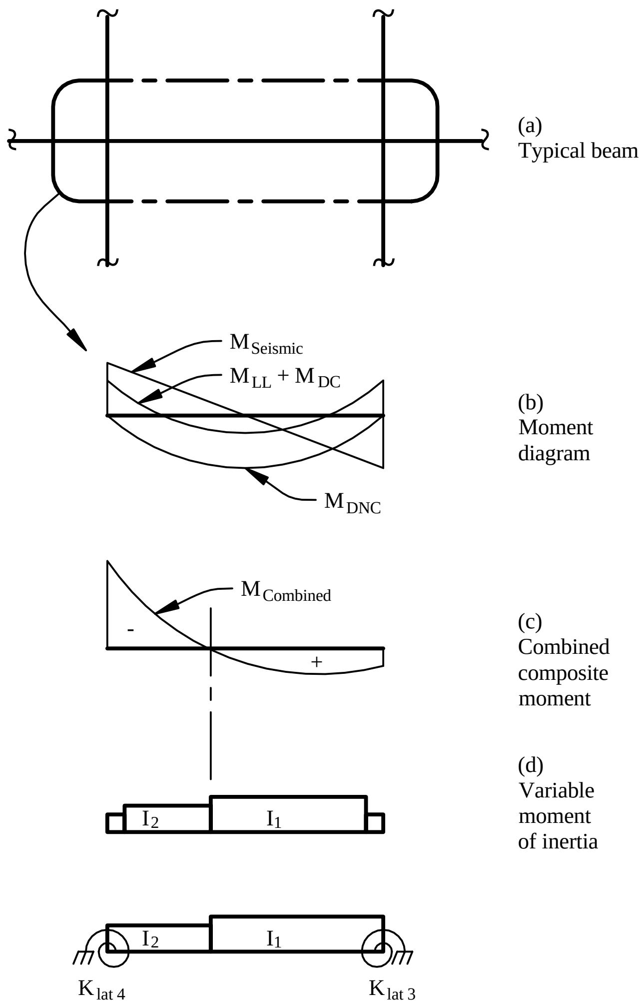


Figure 8-8 Moment diagram for typical beam.

Line 85: compute the ratio of the moment of inertia of the effective prismatic beam to that for the bare steel beam. When using standard computer programs for analysis that have a library of properties of steel cross sections, this ratio is a convenient way to adjust the modulus of elasticity and thus easily compute the lateral drift of a frame. This adjustment could invalidate routines in programs that automatically check various design criteria that depend on the modulus of elasticity.

Step 9: Check the tension bolts at maximum rotation

Line 87: Compute the rotation at total drift as C_d times the drift at the design positive moment.
Line 88: Compute the positive moment corresponding to that drift.
Line 89: Compute the tension force at the bottom seat angle, ignoring any contribution of the web angles, from the moment and a moment arm between the center of the slab thickness and the inflection point in the vertical leg of the seat angle, then add the prying force already calculated for a maximum demand on the tension bolts.
Line 90: Compare with the nominal capacity of the bolts (set $\phi = 1.0$)

Step 10: Check the maximum positive moment capacity:

Line 92: The positive moment at 20 milliradians, already calculated, is compared to the plastic moment capacity of the steel beam. This is the point at which the 50 percent requirement of AISC Seismic, Part II, 8.4 is checked.

Figure 8-10 shows many of the details of the connection for the W18x35. The headed studs shown develop full composite action of the beam between the end and midspan. They do not develop full composite action between the column and the inflection point, but it may be easily demonstrated that they are more than capable of developing the full force in the reinforcing steel within that distance. The transverse reinforcement is an important element of the design, which will be discussed subsequently. Alternating the position above and below is simply a preference of the authors.

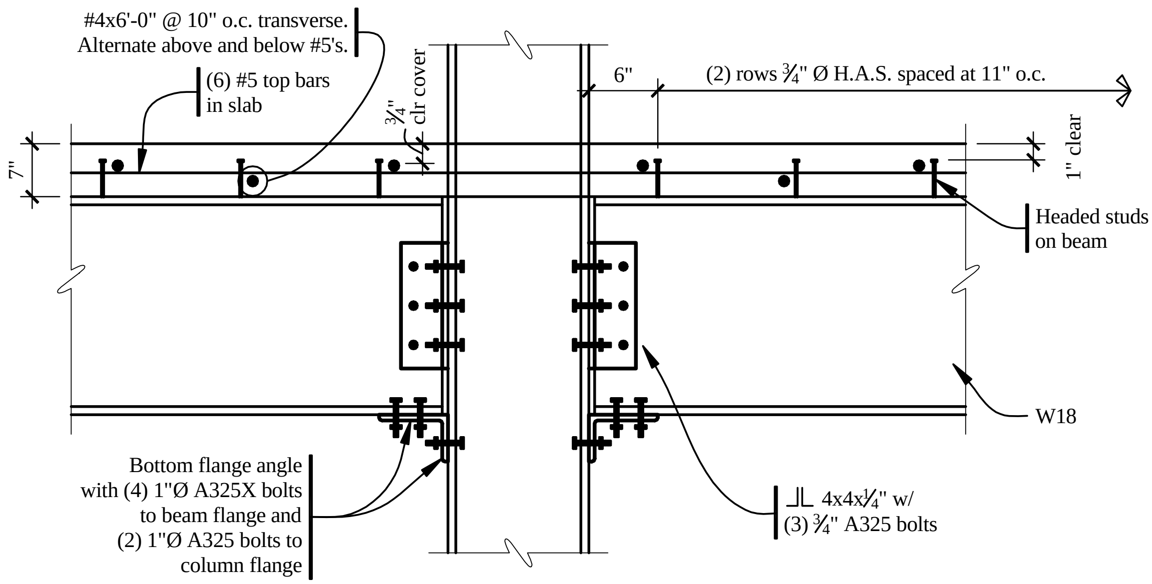


Figure 8-10 Elevation of typical connection (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

8.5 ANALYSIS

8.5.1 Load Combinations

A 3D model using Risa 3D was developed. Non-composite dead loads (steel beams, bar joists, form deck, and concrete) were input as concentrated loads at the columns on each level rather than uniformly distributed to the beams. This was because we want the model for the seismic load combinations to address the moments in the PRC connections. The loads subject to composite action are the composite dead loads, live loads, and seismic loads, not the non-composite dead loads. But the non-composite dead loads still contribute to mass, are subject to ground acceleration, and as such contribute to seismic loads. This gets confusing; so a detailed look at the load combinations is appropriate.

Let us consider four load cases (illustrated in Figures 8-11 and 8-12):

1. D_c - Composite dead load, which is uniformly distributed and applied to beams (based on 32 psf)

2. D_{nc} - Non-composite dead load, which is applied to the columns (based on 66 psf)

3. L - Composite live load, which is uniformly distributed to beams, using live load reductions

4. *E* - Earthquake load, which is applied laterally to each level of the building and has a vertical component applied as a uniformly distributed load to the composite beams

We will investigate two load combinations. Recall that composite loads are applied to beams, while non-composite loads are applied to columns. But there is an exception: the $0.2S_{DS}D$ component, which represents vertical acceleration from the earthquake is applied to all the dead load on the beams whether it is composite or non-composite. This is because even non-composite dead load contributes to mass, and is subject to the ground acceleration. Because the non-composite dead load is not distributed on the beams in the computer model, an adjustment to the load factor is necessary. The assignment of loads gets a little complicated, so pay careful attention:

$$
\begin{aligned}
\text{Combination 1} &= 1.2D + 0.5L + 1.0E \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.2S_{DS}D \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067\,(D_{nc} + D_c) \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067D_{nc}(D_c/D_c) + 0.067D_c \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067D_c(D_{nc}/D_c) + 0.067D_c \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067D_c(66\text{ psf}/32\text{ psf}) + 0.067D_c \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.138D_c + 0.067D_c \\
&= 1.2D_{nc} + 1.405D_c + 0.5L + Q_E
\end{aligned}
$$

Q_E will be applied in both the north-south and the east-west directions, so this really represents two load combinations.

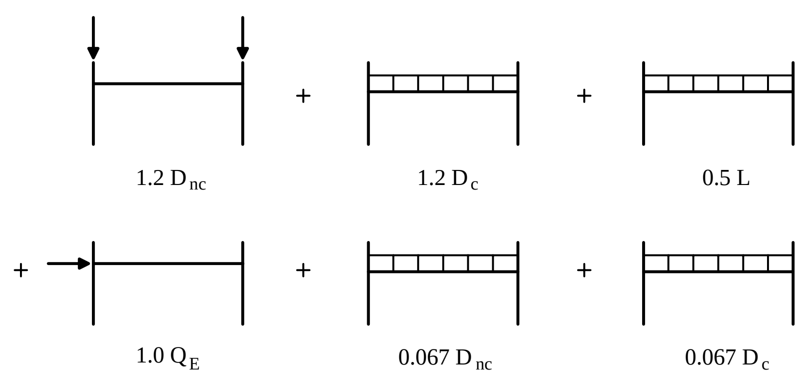


Figure 8-11 Illustration of input for load combination for $1.2D + 0.5L + 1.0Q_E + 0.2S_{DS}D$.

D_{nc} = non-composite dead load.
D_c = composite dead load
L = live load
Q_E = horizontal seismic load

Now consider at the second load combination:

$$
\begin{aligned}
\text{Combination 2} &= 0.9D + 1.0E \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.2S_{DS}D \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,(D_{nc} + D_c) \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,D_{nc}(D_c/D_c) - 0.067D_c \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,D_c(D_{nc}/D_c) - 0.067D_c \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,D_c(66\text{ psf}/32\text{ psf}) - 0.067D_c
\end{aligned}
$$

$$= 0.9D_{nc} + 0.9D_c + Q_E - 0.138D_c - 0.067D_c$$
$$= 0.9D_{nc} + 0.9D_c + Q_E - 0.205D_c$$
$$= 0.9D_{nc} + 0.695D_c + Q_E$$

Again, Q_E will be applied in both the north-south and the east-west directions, so this represents another two load combinations.

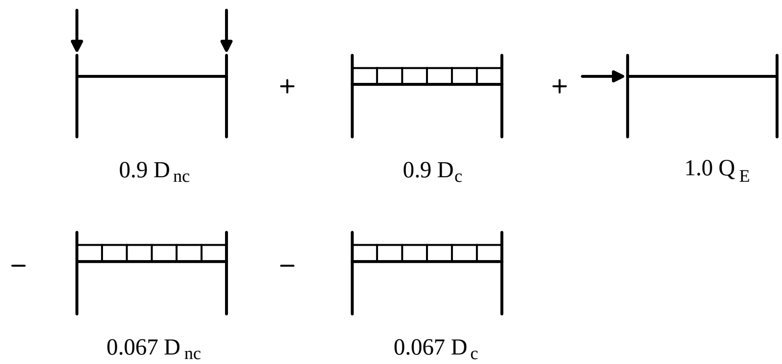


Figure 8-12 Illustration of input for load combination for $0.9D + 1.0Q_E - 0.2S_{DS}D$.

D_{nc} = non-composite dead load.
D_c = composite dead load
L = live load
Q_E = horizontal seismic load

8.5.2 Drift and P-delta

As defined by the *Provisions*, torsional irregularity is considered to exist when the maximum displacement computed including accidental torsion at one end of the structure transverse to an axis is more than 1.2 times the average of the displacements at the two ends of the structure (*Provisions* Sec. 5.4.4.3). For this building the maximum displacement at the roof including accidental torsion, is 1.65 in. The displacement at the other end of the building in this direction is 1.43 in. The average is 1.54 in. Because 1.65 in. < 1.85 in. = (1.2)(1.54 in.), the structure is not torsionally irregular. Consequently, it is not necessary to amplify the accidental torsion nor to check the story drift at the corners. A simple check at the center of the building suffices. [In the 2003 *Provisions*, the maximum limit on the stability coefficient has been replaced by a requirement that the stability coefficient is permitted to exceed 0.10 if and only "if the resistance to lateral forces is determined to increase in a monotonic nonlinear static (pushover) analysis to the target displacement as determined in Sec. A5.2.3. P-delta effects shall be included in the analysis." Therefore, in this example, the stability coefficient should be evaluated directly using 2003 *Provisions* Eq. 5.2.-16.]

The elastic story drifts were computed by the RISA 3D analysis for the required load combinations. Like most modern computer programs for structural analysis, a P-delta amplification can be automatically computed, but to illustrate the effect of P-delta in this structure and to check the limit on the stability index, two computer runs have been performed, one without the P-delta amplifier and one with it. The allowable story drift is taken from *Provisions* Table 5.2.8. The allowable story drift is 0.025 h_{sx} = (0.025)(13 ft × 12 in./ft) = 3.9 in. With a C_d of 5.5, this corresponds to a drift 0.71 in. under the equivalent elastic forces. At this point design for wind does influence the structure. A drift limit of h/400 (= 0.39 in.) was imposed, by office practice, to the service level wind load. In order to achieve the desired stiffness, the seismic story drift at elastic forces is determined thus:

Elastic story drift limit = (wind drift limit)(total seismic force)/(service level wind force)
Elastic story drift limit = (0.39 in.)(170 kip)/123 kip = 0.54 in.

The structure complies with the story drift requirements, but it was necessary to increase the size of the spandrel beams from the preliminary W18x35 to W21x44 to meet the desired wind stiffness. This is summarized in Table 8-3. The structure also complies with the maximum limit on the stability index (Provisions 5.4.6.2-2):

$$\theta_{\max} = \frac{0.5}{\beta C_d} = \frac{0.5}{0.5*5.5} = 0.18 \le 0.25$$

β is the ratio of demand to capacity for the story shear, and has not yet been computed. Maximum demand and design capacity are tabulated in the following section; the average is about two-thirds. The preceding data show that the maximum resistance is higher, especially for positive moment, than the value suggested here for design capacity. The average ratio of demand to maximum capacity with a resistance factor is well below 0.5, so that value is arbitrarily used to show that the actual stability index is within the limits of the *Provisions*.

Table 8-3 Story Drift (in.) and P-delta Analysis

Story	North-south (X direction)				East-west (Z direction)			
	without P-delta	with P-delta	P-delta amplifier	Stability index	without P-delta	with P-delta	P-delta amplifier	Stability index
1	0.358	0.422	1.179	0.152	0.312	0.360	1.154	0.133
2	0.443	0.517	1.167	0.143	0.410	0.471	1.149	0.130
3	0.449	0.513	1.143	0.125	0.402	0.453	1.127	0.113
4	0.278	0.304	1.094	0.086	0.239	0.259	1.084	0.077

8.5.3 Required and Provided Strengths

The maximum beam end moments from the frame analysis for the seismic load combinations are as follows:

Table 8-4 Maximum Connection Moments and Capacities (ft-kips)

Quantity	W18 Girders		W21 Spandrels	
	Negative	Positive	Negative	Positive
Demand (level 2), M_u	143	36.6	118	103
Nominal, M_n	216	67.2	325	149
Design capacity, ϕM_n	184	57.1	276	127

The capacities, using a resistance factor of 0.85, are well in excess of the demands. The girder member sizes are controlled by gravity load in the construction condition. All other member and connection capacities are controlled by the design for drift. The negative moment demands are somewhat larger than would result from a more careful analysis, because the use of a prismatic member overestimates the end moments due to distributed load (composite gravity load) along the member. The higher stiffness of the portion of the beam in positive bending with respect to the connections would result in higher positive moments at midspan and lower negative moments at the supports. This conservatism has no real effect on this design example. (The above demands and capacities do not include the girders supporting the storage bays, which are required to be W18x40 for the gravity load condition. The overall analysis does not take that larger member into account.)

Snow load is not included in the seismic load combinations. (According to the *Provisions*, snow load equal to or less than 30 psf does not have to be included in the mass.) Further, as a designer's judgment call, it was considered that the moments from 0.2S (= 6 psf) were so small, considering that the roof was designed with the same connections as the floors, that it would make no significant difference in the design analysis.

The maximum column forces are shown in Table 8-5; the particular column does support the storage load. The effective length of the columns about their weak axis will be taken as 1.0, because they are braced by perpendicular frames acting on the strong axes of the columns, and the P-delta analysis captures the secondary moments due to the "leaning" column effect. The effective length about their strong axis will exceed 1.0. The ratio of column stiffness to beam stiffness will use the same effective beam stiffness computed for the drift analysis, thus for the W10x77 framed into the W18x35 beams:

$I_{col} / L_{col} = 455/(13 \times 12) = 2.92$

$I_{beam} / L_{beam} = 1.25 \times 510 / (25 \times 12) = 2.12$

and the ratio of stiffnesses, $G = 2.92 / 2.12 = 1.37$

Although the column in the lowest story has greater restraint at the foundation, and thus a lower K factor, it is illustrative to determine K for a column with the same restraint at the top and bottom. From the nomographs in the AISC Manual or from equivalent equations, K = 1.45. It turns out that the effective slenderness about the strong axis is less than that for the weak axis, so the K factor does not really control this design.

Table 8-5 Column Strength Check, for W10x77

	Seismic Load Combination	Gravity Load Combination
Axial force, P_u	391 kip	557 kip
Moment, M_u	76.3 ft-kip	52.5 ft-kip
Interaction equation	0.72	0.89

8.6 DETAILS OF THE DESIGN

8.6.1 Overview

The requirements in AISC Seismic for C-PRMF systems are brief. Some of the requirements are references to Part I of AISC Seismic for the purely steel components of the system. A few of those detail checks are illustrated here. For this example, more attention is paid to the details of the joint.

8.6.2 Width-Thickness Ratios

The width-thickness ratio of the beam flanges, $b_f/2t_f$ is compared to λ_p given in AISC Seismic, Part I, Table I-9-1. Both beam sizes, W18x35 and W21x44 are found to be acceptable. The W21x44 is illustrated below:

$$\lambda_p = \frac{52}{\sqrt{F_y}} = \frac{52}{\sqrt{50}} = 7.35$$ (AISC Seismic, Table I-9-1)

$$\frac{b_f}{2t_f} = 7.22$$ (AISC Manual)

7.22 < 7.35 OK

The limiting h/t ratios for columns is also given in AISC Seismic, Part I, Table I-9-1. A W10x77 column from the lower level of an interior bay with storage load is illustrated (the axial load from the seismic load combination is used):

$$\frac{P_u}{\phi_b P_y} = \frac{391 \text{ kips}}{(0.9)(22.6 \text{ in.}^2 \text{ x } 50 \text{ ksi})} = 0.385 > 0.125$$ (AISC Seismic, Table I-9-1)

$$\lambda_p = \frac{191}{\sqrt{F_y}}\left[2.33 - \frac{P_u}{\phi_b P_y}\right] = \frac{191}{\sqrt{50}}\left[2.33 - 0.385\right] = 52.5$$ (AISC Seismic, Table I-9-1)

Check:

$$\lambda_p = 52.5 > 35.7 = \frac{253}{\sqrt{F_y}}$$ OK

$$\frac{h}{t_w} = 13.0$$ (AISC Manual)

13.0 < 52.5 OK

8.6.3 Column Axial Strength

AISC Seismic, Part I, 8.2 requires that when $P_u/\phi P_n > 0.4$ (in a seismic load combination), additional requirements be met. Selecting the same column as above for our illustration:

$$\frac{P_u}{\phi P_n} = \frac{391 \text{ kips}}{(0.85)(22.6 \text{ in.}^2)(38.4 \text{ ksi})} = 0.53 > 0.4$$ (AISC Seismic, Part I, 8.2)

Therefore the requirements of AISC Seismic, Part I, 8.2a, 8.2b, and 8.2c apply. These necessitate the calculation of axial loads using the System Overstrength Factor, $\Omega_0 = 3$. Analysis needs to be run for two additional load combinations:

$1.2D + 0.5L + 0.2S + \Omega_0 Q_E$ (AISC Seismic, Part I, Eq. 4.1)

and

$0.9D - \Omega_0 Q_E$ (AISC Seismic, Part I, Eq. 4.2)

The axial seismic force in this column is only 7.5 kips, therefore P_u becomes 397 kips, obviously much less than ϕP_n. The low seismic axial load is common for a moment-resisting frame system. Given that this requirement is a check ignoring bending moment, it does not control the design.

[The special load combinations have been removed from the 2002 edition of AISC Seismic to eliminate inconsistencies with other building codes and standards. Therefore, 2003 *Provisions* Eq. 4.2-3 and 4.2-4 should be used in conjunction with the load combinations in ASCE 7.]

8.6.4 Details of the Joint

Figure 8-13 shows a plan view at an edge column, concentrating on the arrangement of the steel elements. Figure 8-14 shows a section at the same location, showing the arrangement of the reinforcing steel. It is not required that the reinforcing bars be equally distributed on the two sides of the column, but it is necessary to place at least some of the bars on each side. This means that some overhang of the slab beyond the column flange is required. This example shows two of the six bars on the outside face. Figure 8-15 shows a plan view at a corner column. U shaped bent bars are used to implement the negative moment connection at such a location. Threaded bars directly attached to the column flange are also illustrated. Note the close spacing of the headed anchor studs for composite action. The reason for the close spacing at this location is that the beam span is half the normal span, yet full composite action is still provided.

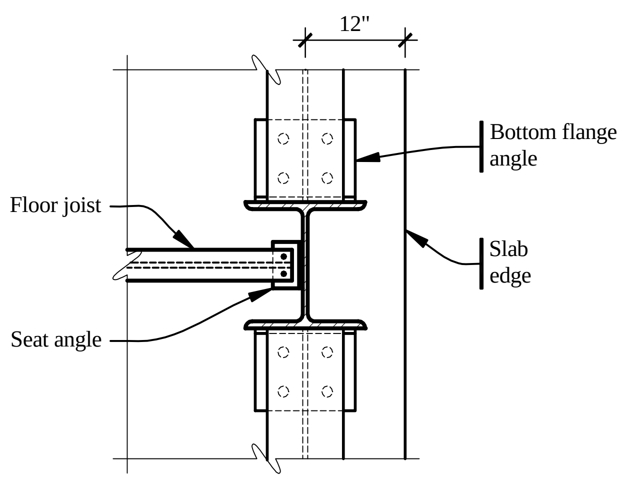


Figure 8-13 Detail at column.

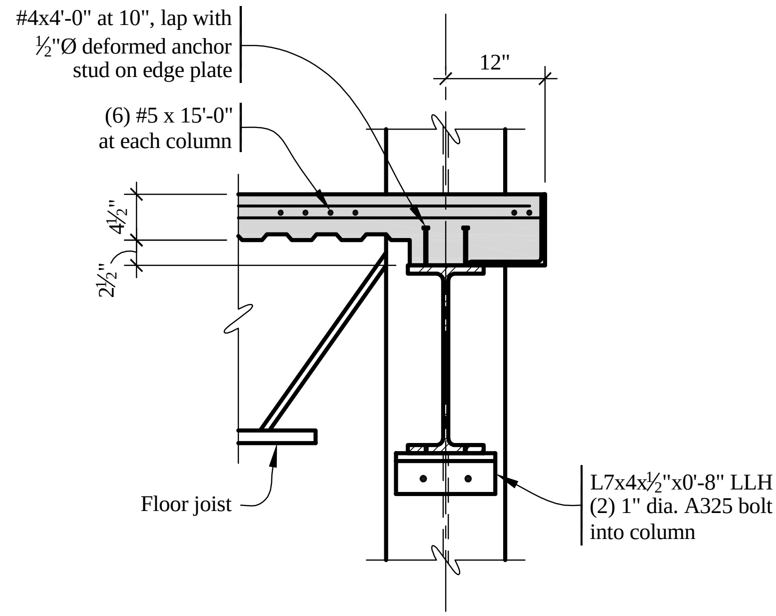


Figure 8-14 Detail at spandrel.

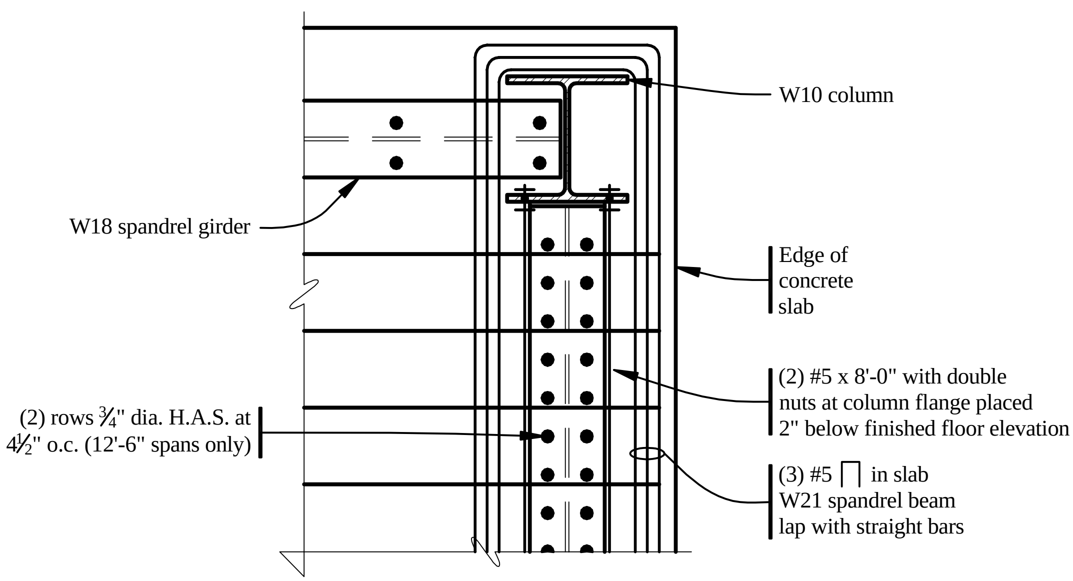


Figure 8-15 Detail at building corner.

The compressive force in the deck is transferred to both flanges of the column. This is shown in Figure 8-16. Note that both flanges can accept compressive forces from the concrete. Also note that the transverse reinforcement will carry tension as force is transferred from the principal tension reinforcement through the concrete to bearing on the column flange. Strut and tie models can be used to compute the appropriate tension.

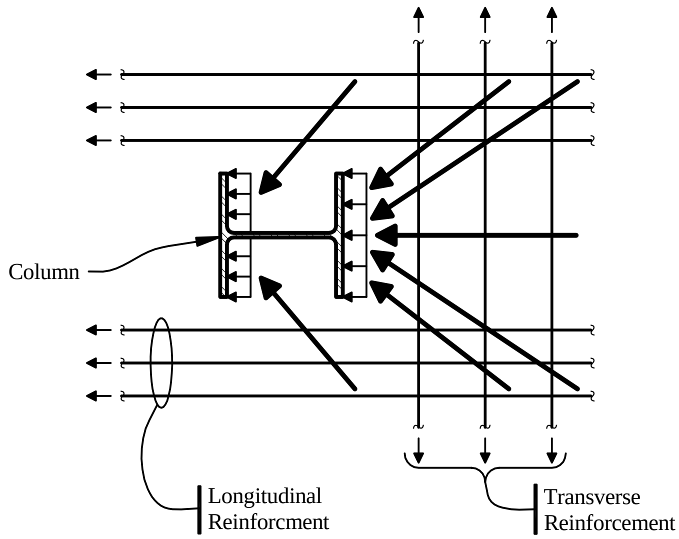


Figure 8-16 Force transfer from deck to column.

AISC SDGS-8 and ASCE TC include the following recommendations regarding the reinforcing steel:

1. Place the principal tension reinforcement within a strip of width equal to 7 times the width of the column flange (or less)

2. Use at least 6 bars for the principal reinforcement, extend it one quarter of the span from the column, but at least 24 bar diameters beyond the inflection point, and extend at least two of the bars over the full span

3. Do not use bars larger than number 6 (0.75 in. diameter)

4. Provide transverse reinforcement consistent with a strut and tie model to enable the transfer of forces (in the authors' observation such reinforcement is also necessary to preserve the capacity of the headed studs for shear transfer)

9

MASONRY

James Robert Harris, P.E., Ph.D., Frederick R. Rutz, P.E., Ph.D. and Teymour Manzouri, P.E., Ph.D.

This chapter illustrates application of the 2000 *NEHRP Recommended Provisions* (herein after the *Provisions),* to the design of a variety of reinforced masonry structures in regions with different levels of seismicity. Example 9.1 features a single-story masonry warehouse building with tall, slender walls; Example 9.2 presents a five-story masonry hotel building with a bearing wall system designed in areas with different seismicities; and Example 9.3 covers a twelve-story masonry building having the same plan as the hotel but located in a region of high seismicity. Selected portions of each building are designed to demonstrate specific aspects of the design provisions.

Masonry is a discontinuous and heterogeneous material. The design philosophy of reinforced grouted masonry approaches that of reinforced concrete; however, there are significant differences between masonry and concrete in terms of restrictions on the placement of reinforcement and the effects of the joints. These physical differences create significant differences in the design criteria.

All structures were analyzed using two-dimensional (2-D) static methods. Examples 9.2 and 9.3 use dynamic analyses to determine the structural periods. Example 9.2 employs the SAP 2000 program, V6.11 (Computers and Structures, Berkeley, California); Example 9.3 employs the RISA 2D program, V.5.5 (Risa Technologies, Foothill Ranch, California).

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

The most significant change to the masonry chapter in the 2003 *Provisions* is the incorporation by reference of ACI 530-02 for strength design in masonry. A significant portion of 2003 *Provisions* Chapter 11 has been replaced by a reference to this standard as well as a limited number of modifications to the standard, similar to other materials chapters. This updated chapter, however, does not result in significant technical changes as ACI 530-02 is in substantial agreement with the strength design methodology contained in the 2000 *Provisions*.

Another change to the provisions for masonry structures is the addition of a new lateral system, prestressed masonry shear walls. This system is not covered in this volume of design examples.

Some general technical changes in the 2003 *Provisions* that relate to the calculations and/or design in this chapter include updated seismic hazard maps, changes to Seismic Design Category classification for short period structures, revisions to the redundancy requirements, revisions to the wall anchorage design requirement for flexible diaphragms, and a new "Simplified Design Procedure" that could be applicable to some of the examples in this chapter.

Where they affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the *Provisions*, the following documents are referenced in this chapter:

ACI 318 American Concrete Institute. 1999 [2002]. *Building Code Requirements for Concrete Structures*.

ACI 530 American Concrete Institute. 1999 [2002]. *Building Code Requirements for Masonry Structures*, ACI 530/ASCE 5/TMS 402.

ASCE 7 American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures*.

Amrhein Amrhein, J, and D. Lee. 1994. *Tall Slender Walls, 2nd Ed.* Masonry Institute of America.

Drysdale Drysdale R., A. Hamid, and L. Baker. 1999. *Masonry Structures, Behavior and Design.* Boulder Colorado: The Boulder Masonry Society.

IBC International Code Council. 2000. *International Building Code*.

UBC International Conference of Building Officials. 1997. *Uniform Building Code*.

NCMA National Concrete Masonry Association. *A Manual of Facts on Concrete Masonry*, NCMA-TEK is an information series from the National Concrete Masonry Association, various dates.

SEAOC Seismology Committee, Structural Engineers Association of California. 1999. *Recommended Lateral Force Requirements and Commentary*, 7th Ed.

The short form designations for each citation are used throughout. The citation to the IBC exists for two reasons. One of the designs employees a tall, slender wall that is partially governed by wind loads and the IBC provisions are used for that design. Also, the *R* factors for masonry walls are significantly different in the IBC than in the *Provisions*; this is not true for other structural systems.

9.1 WAREHOUSE WITH MASONRY WALLS AND WOOD ROOF, LOS ANGELES, CALIFORNIA

This example features a one-story building with reinforced masonry bearing walls and shear walls.

9.1.1 Building Description

This simple rectangular warehouse is 100 ft by 200 ft in plan (Figure 9.1-1). The masonry walls are 30 ft high on all sides, with the upper 2 ft being a parapet. The wood roof structure slopes slightly higher towards the center of the building for drainage. The walls are 8 in. thick on the long side of the building, for which the slender wall design method is adopted, and 12 in. thick on both ends. The masonry is grouted in the cells containing reinforcement, but it is not grouted solid. The assumed strength of masonry is 2,000 psi. Normal weight concrete masonry units (CMU) with type S mortar are assumed.

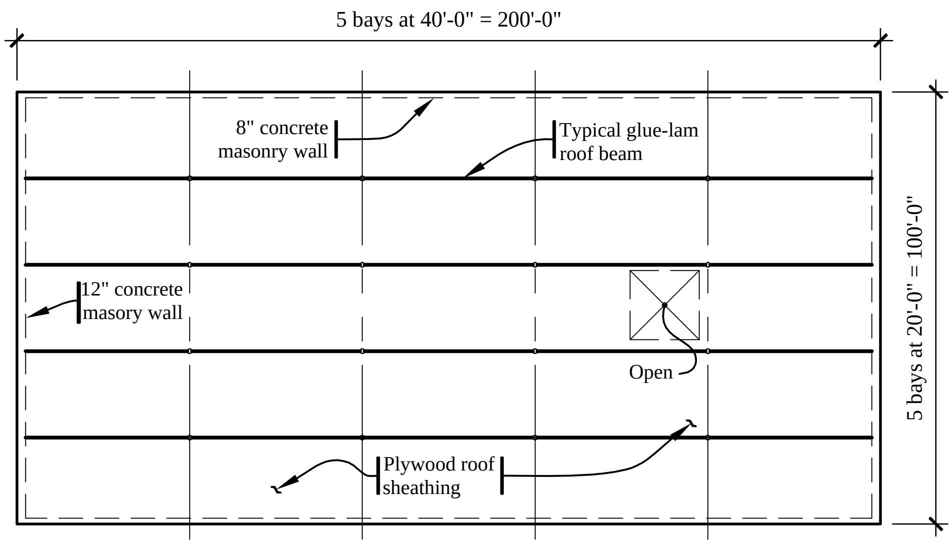


Figure 9.1-1 Roof plan (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

The long side walls are solid (no openings). The end walls are penetrated by several large doors, which results in more highly stressed piers between the doors (Figure 9.1-2); thus, the greater thickness for the end walls.

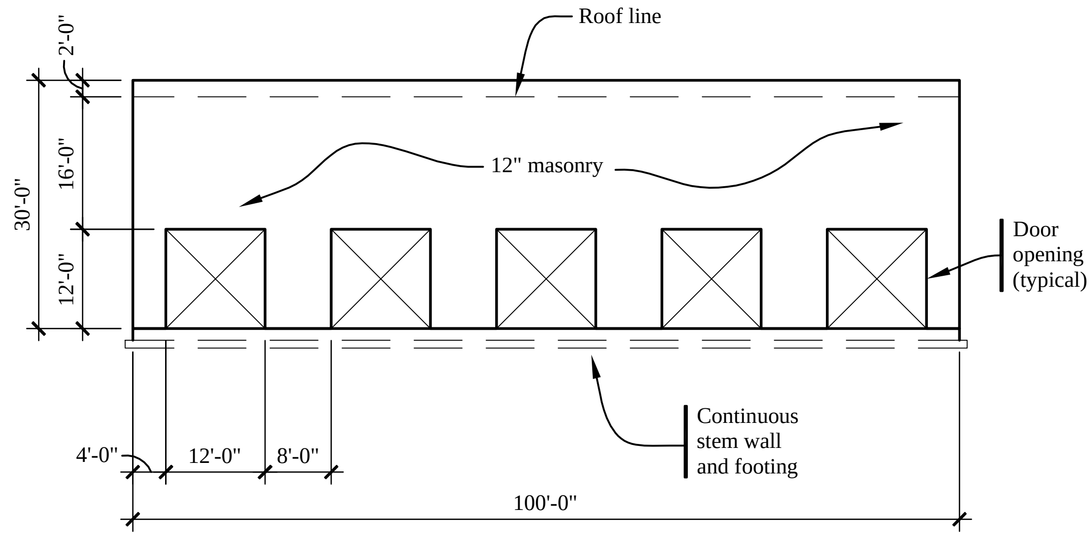


Figure 9.1-2 End wall elevation (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

The floor is concrete slab-on-grade construction. Conventional spread footings are used to support the interior steel columns. The soil at the site is a dense, gravelly sand.

The roof structure is wood and acts as a diaphragm to carry lateral loads in its plane from and to the exterior walls. The roofing is ballasted, yielding a total roof dead load of 20 psf. There are no interior walls for seismic resistance. This design results in a highly stressed diaphragm with large calculated deflections. The design of the wood roof diaphragm and the masonry wall-to-diaphragm connections is illustrated in Sec. 10.2.

In this example, the following aspects of the structural design are considered:

1. Design of reinforced masonry walls for seismic loads and
2. Computation of P-delta effects.

9.1.2 Design Requirements

[Note that the new “Simplified Design Procedure” contained in the 2003 *Provisions* Simplified Alternate Chapter 4 as referenced by the 2003 *Provisions* Sec. 4.1.1 is likely to be applicable to this example, subject to the limitations specified in 2003 *Provisions* Sec. Alt. 4.1.1.]

9.1.2.1 *Provisions* Parameters

Site Class (*Provisions* Sec. 4.1.2.1 [Sec. 3.5])	= C
S_S (*Provisions* Map 5 [Figure 3.3-3])	= 1.50
S_1 (*Provisions* Map 6 [Figure 3.3-4])	= 0.60
Seismic Use Group (*Provisions* Sec. 1.3[Sec. 1.2])	= I

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps , and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

The remaining basic parameters depend on the ground motion adjusted for site conditions.

9.1.2.2 Response Parameter Determination

The mapped spectral response factors must be adjusted for site class in accordance with *Provisions* Sec. 4.1.2.4 [3.3.2]. The adjusted spectral response acceleration parameters are computed according to *Provisions* Eq. 4.1.2.4-1 [3.3-1] and 4.1.2.4-2 [3.3-2] for the short period and one-second period, respectively, as follows:

$S_{MS} = F_a S_S$ = 1.0(1.50) = 1.50
$S_{M1} = F_v S_1$ = 1.3(0.60) = 0.78

Where F_a and F_v are site coefficients defined in *Provisions* Tables 4.1.2.4a [3.3-1] and 4.1.2.4b [3.3-2], respectively. The design spectral response acceleration parameters (*Provisions* Sec. 4.1.2.5 [Sec. 3.3.3]) are determined in accordance with *Provisions* Eq. 4.1.2.5-1 [Eq. 3.3-3] and 4.1.2.5-2 [3.3-4] for the short-period and one-second period, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.50) = 1.00$$
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.78) = 0.52$$

The Seismic Design Category may be determined by the design spectral acceleration parameters combined with the Seismic Use Group. For buildings assigned to Seismic Design Category D, masonry shear walls must satisfy the requirements for special reinforced masonry shear walls in accordance with *Provisions* Sec. 11.3.8.2 [ACI 530 Sec. 1.13.6.4]. A summary of the seismic design parameters follows:

Seismic Design Category (*Provisions* Sec. 4.2.1 [1.4])	= D
Seismic Force Resisting System (*Provisions* Table 5.2.2 [4.3-1])	= Special Reinforced Masonry Shear Wall
Response Modification Factor, R (*Provisions* Table 5.2.2 [4.3-1])	= 3.5
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [4.3-1])	= 3.5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [4.3-1])	= 2.5
Reliability Factor, ρ (*Provisions* Sec. 5.2.4.2 [Sec. 4.3.3])	= 1.0

(Determination of ρ is discussed in Sec. 9.1.3 below [see Sec. 9.1.3.1 for changes to the reliability factor in the 2003 *Provisions*].)

Note that the R factor for this system in the IBC and in ASCE 7 is 4.5. [5.0 in the 2003 IBC and ASCE 7-02] This difference would have a substantial effect on the seismic design; however, the vertical reinforcement in the tall 8-in. walls is controlled by wind loads so it would not change.

9.1.2.3 Structural Design Considerations

With respect to the load path, the roof diaphragm supports the upper 16 ft of the masonry walls (half the clear span plus the parapet) in the out-of-plane direction, transferring the lateral force to in-plane masonry shear walls.

Soil structure interaction is not considered.

The building is of bearing wall construction.

Other than the opening in the roof, the building is symmetric about both principal axes, and the vertical elements of the seismic resisting system are arrayed entirely at the perimeter. The opening is not large enough to be considered an irregularity (per *Provisions* Table 5.2.3.2[Table 4.3-2]); thus, the building is regular, both horizontally and vertically. *Provisions* Table 5.2.5.1[Table 4.4-1], permits several analytical procedures to be used; the equivalent lateral force (ELF) procedure (*Provisions* Sec. 5.4) is selected for used in this example. The orthogonality requirements of *Provisions* Sec. 5.2.5.2 Sec. 4.4.2 are potentially significant for the piers between the door openings at the end walls. Thus, those walls will be designed for 100 percent of the forces in one direction plus 30 percent of the forces in the perpendicular direction.

There will be no inherent torsion because the building is symmetric. The effects of accidental torsion, and its potential amplification, need not be included because the roof diaphragm is flexible. This is the authors' interpretation of what amounts to a conflict between *Provisions* Table 5.2.3.2[Table 4.3-2], Item 1, and *Provisions* Sec. 5.4.4.2[Sec. 5.24.2] and Sec. 5.4.4.3[Sec. 5.2.4.3].

The masonry bearing walls also must be designed for forces perpendicular to their plane (*Provisions* Sec. 5.2.6.2.7)[Sec. 4.6.1.3].

For in-plane loading, the walls will be treated as cantilevered shear walls. For out-of-plane loading, the walls will be treated as pinned at the bottom and simply supported at the top. The assumption of a pinned connection at the base is deemed appropriate because the foundation is shallow and narrow which permits rotation near the base of the wall.

9.1.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [Sec. 4.2.2]) are the same as specified in ASCE 7 (and similar to the IBC). The seismic load effect, E, is defined by *Provisions* Eq. 5.2.7-1 [4.2-1] and Eq. 5.2.7-2 [4.2-2] as:

$$E = \rho Q_E \pm 0.2 S_{DS} D = (1.0) Q_E \pm 0.2(1.00) D = Q_E \pm 0.2D$$

This assumes $\rho = 1.0$ as will be confirmed in the following section.

9.1.3.1 Reliability Factor

In accordance with *Provisions* Sec. 5.2.4.2[4.3.3], the reliability factor, ρ, applies to the in-plane load direction.

For the long direction of building:

$$r_{max_x} = \left(\frac{V_{wall}}{V_{story}}\right)\left(\frac{10}{l_w}\right)$$

$$r_{max_x} = (0.5)\left(\frac{10}{200}\right) = 0.025$$

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{20,000}} = 2 - \frac{20}{0.025\sqrt{20,000}} = -3.66$$

ρ = -3.66 < 1.0 = ρ_{min}, so use ρ = 1.0.

For the short direction of the building:

$$r_{max_x} = \left(\frac{V_{wall}}{V_{story}}\right)\left(\frac{10}{l_w}\right) = \left(\frac{(V_{wall})(0.23)}{V_{story}}\right)\left(\frac{10}{8}\right) = (0.5)(0.23)(1) = 0.115$$

Although the calculation is not shown here, note that a single 8-ft-long pier carries approximately 23 percent (determined by considering the relative rigidities of the piers) of the in-plane load for each end wall.

Also, 1.0 was used for the $10/l_w$ term even though 10/8 ft > 1.0. According to *Provisions* 5.2.4.2, the $10/l_w$ term need not exceed 1.0 *only* for walls of light frame construction. This example was created based on a draft version of the 2000 *Provisions*, which limited the value of the $10/l_w$ term to 1.0 for all shear walls, a requirement that was later changed for the published edition. Thus, this calculation is not strictly correct. Using the correct value of r_{max_x} would result in ρ = 1.02 rather than the 0.77 computed below. This would result in a slight change in the factor on Q_E, 1.02 vs. 1.00, which has not been carried through the remainder of this example.

(When the redundancy factor was developed by the Structural Engineers Association of California in the wake of the 1994 Northridge earthquake, the upper bound of 1.0 for $10/l_w$ was simply not mentioned. The 1997 *Provisions*, the UBC, and the IBC were published with no upper bound on $10/l_w$. However, the original authors of the concept published their intent with the SEAOC document in 1999 with the upper bound of 1.0 on $10/l_w$ for all types of shear walls. The same change was adopted within BSSC for the 2000 *Provisions*. A subsequent change to the 2000 *Provisions* limited the upper bound of 1.0 to apply only to light frame walls.)

Therefore,

$$r_{max_x} = 0.12$$

$$\rho = 2 - \frac{20}{0.115\sqrt{20,000}} = 0.77$$

ρ = 0.77 < 1.0 = ρ_{min}, so use ρ = 1.0.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Therefore, the redundancy factor would have to be investigated only in the transverse direction where the aspect ratios of the piers between door openings are greater than 1.0. In the longitudinal direct, where the aspect ratio is (significantly) less than 1.0, ρ = 1.0 by default.]

9.1.3.2 Combination of Load Effects

Load combinations for the in-plane loading direction from ASCE 7 are:

$1.2D + 1.0E + 0.5L + 0.2S$

and

$0.9D + 1.0E + 1.6H$.

L, S, H do not apply for this example so the load combinations become:

$1.2D + 1.0E$

and

$0.9D + 1.0E$.

When the effect of the earthquake determined above, $1.2D + 1.0(Q_E \pm 0.2D)$, is inserted in each of the load combinations:

$1.4D + 1.0\ Q_E$
$1.0D - 1.0\ Q_E$

and

$0.9D + 1.0(Q_E \pm 0.2D)$

which results in:

$1.1D + 1.0\ Q_E$

and

$0.7D - 1.0\ Q_E$

Thus, the controlling cases from all of the above are:

$1.4D + 1.0\ Q_E$

when gravity and seismic are additive and

$0.7D - 1.0\ Q_E$

when gravity and seismic counteract.

These load combinations are for the in-plane direction of loading. Load combinations for the out-of-plane direction of loading are similar except that the reliability coefficient (ρ) is not applicable. Thus, for this example (where $\rho = 1.0$), the load combinations for both the in-plane and the out-of-plane directions are:

$1.4D + 1.0\ Q_E$

and

$0.7D - 1.0\,Q_E$.

The combination of earthquake motion (and corresponding loading) in two orthogonal directions must be considered (*Provisions* Sec. 5.2.5.2.3) [Sec. 4.4.2.3].

9.1.4 Seismic Forces

9.1.4.1 Base Shear

Base shear is computed using the parameters determined previously. The *Provisions* does not recognize the effect of long, flexible diaphragms on the fundamental period of vibration. The approximate period equations, which limit the computed period, are based only on the height. Since the structure is relatively short and stiff, short-period response will govern the design equations. According to *Provisions* Sec. 5.4.1 [Sec. 5.2.1.1] and Eq. 5.4.1.1-1 [Eq. 5.2-3] (for short-period structures):

$$V = C_S W = \left[\frac{S_{DS}}{R/I}\right] W = \left[\frac{1.0}{3.5/1}\right] W = 0.286\ W$$

The seismic weight for forces in the long direction is:

Roof = 20 psf (100)200	= 400 kips
End walls = 103 psf (2 walls)[(30 ft)(100 ft) - 5(12 ft)(12 ft)](17.8 ft/28 ft)	= 299 kips
Side walls = 65 psf (30ft)(200ft)(2 walls)	= 780 kips
Total	= 1,479 kips

Note that the centroid of the end walls is determined to be 17.8 ft above the base, so the portion of the weight distributed to the roof is approximately the total weight multiplied by 17.8 ft/28 ft (weights and section properties of the walls are described subsequently).

Therefore, the base shear to each of the long walls is:

V = (0.286)(1,479 kips)/2 = 211 kips.

The seismic weight for forces in the short direction is:

Roof = 20 psf (100)200	= 400 kips
Side walls = 65 psf (2 walls)(30ft)(200ft)(15ft/28ft)	= 418 kips
End walls = 103 psf (2 walls)[(30ft)(100ft)-5(12ft)(12ft)]	= 470 kips
Total	= 1,288 kips

The base shear to each of the short walls is:

V = (0.286)(1,288 kips)/2 = 184 kips.

9.1.4.2 Diaphragm Force

See Sec. 10.2 for diaphragm forces and design.

9.1.4.3 Wall Forces

because the diaphragm is flexible with respect to the walls, shear is distributed to the walls on the basis of beam theory ignoring walls perpendicular to the motion (this is the "tributary" basis).

The building is symmetric. Given the previously explained assumption that accidental torsion need not be applied, the force to each wall becomes half the force on the diaphragm.

All exterior walls are bearing walls and, according to *Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3], must be designed for a normal (out-of-plane) force of $0.4S_{DS}W_c$. The out-of-plane design is shown in Sec. 9.1.5.3 below.

9.1.5 Longitudinal Walls

The total base shear is the design force. *Provisions* Sec. 11.7 [Sec. 11.2] is the reference for design strengths. The compressive strength of the masonry (f_m') is 2,000 psi. *Provisions* Sec. 11.3.10.2 gives $E_m = 750 f_m' = (750)(2 \text{ ksi}) = 1{,}500$ ksi.

[2003 *Provisions* Sec. 11.2 adopts ACI 530 as a design basis for strength design masonry and provides some modifications to ACI 530. In general, the adoption of ACI 530 as a reference does not have a significant effect on this design example. Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1, eliminating the conflict discussed below.]

Be careful to use values consistent with the *Provisions*. Different standards call for different values. To illustrate this point, the values of E_m from different standards are shown in Table 9.1-1.

Table 9.1-1 Comparison of E_m

Standard	E_m	E_m for this example
Provisions	750 f_m'	1,500 ksi
IBC	900 f_m'	1,800 ksi
ACI 530	900 f_m'	1,800 ksi

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm.

For 8-inch thick CMU with vertical cells grouted at 24 in. o.c. and horizontal bond beams at 48 inch o.c., the weight is conservatively taken as 65 psf (recall the CMU are normal weight) and the net bedded area is 51.3 in.^{2}/ft based on tabulations in NCMA-TEK 141.

9.1.5.1 Horizontal Reinforcement

As determined in Sec. 9.1.4.1, the design base shear tributary to each longitudinal wall is 211 kips. Based on *Provisions* Sec. 11.7.2.2 [ACI 530, Sec. 3.1.3], the design shear strength must exceed either the shear corresponding to the development of 1.25 times the nominal flexure strength of the wall, which is very unlikely in this example due to the length of wall, or 2.5 times V_u = 2.5(211) = 528 kips.

From *Provisions* Eq. 11.7.3.2[ACI 530, Eq. 3-21] , the masonry component of the shear strength capacity for reinforced masonry is:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f_m'} + 0.25\,P\,.$$

Conservatively treating M/Vd as equal to 1.0 for the long walls and conservatively treating P as the weight of the wall only without considering the roof weight contribution:

$$V_m = [4.0 - 1.75(1.0)](51.3)(200)\sqrt{2000} + 0.25(390) = 1130 \text{ kips}$$

and

$$\phi V_m = 0.8(1{,}130) = 904 \text{ kips} > 528 \text{ kips} \qquad \text{OK}$$

where ϕ = 0.8 is the resistance factor for shear from *Provisions* Table11.5.3[ACI 530, sec. 3.1.4] .

Horizontal reinforcement therefore is not required for shear but is required if the wall is to qualify as a "Special Reinforced Masonry Wall."

According to *Provisions* Sec. 11.3.8.3[ACI 530, Sec. 1.13.6.3] , minimum reinforcement is (0.0007)(7.625 in.)(8 in.) = 0.043 in.^{2} per course, but it may be wise to use more horizontal reinforcement for shrinkage in this very long wall and then use minimum reinforcement in the vertical direction (this concept applies even though this wall requires far more than the minimum reinforcement in the vertical direction due to its large height-to-thickness ratio). Two #5 bars at 48 in. on center provides 0.103 in.^{2} per course. This amounts to 0.4 percent of the area of masonry plus the grout in the bond beams. The actual shrinkage properties of the masonry and the grout and local experience should be considered in deciding how much reinforcement to provide. For long walls that have no control joints, as in this example, providing more than minimum horizontal reinforcement is appropriate.

9.1.5.2 Vertical Reinforcement

Steps for verifying a trial design are noted in the sections that follow.

9.1.5.3 Out-of Plane Flexure

As indicated previously, the design demand for seismic out-of-plane flexure is $0.4S_{DS}W_c$. For a wall weight of 65 psf for the 8-in.-thick CMU side walls, this demand is 0.4(1.00)(65 psf) = 26 psf.

Calculations for out-of-plane flexure become somewhat involved and include the following:

1. Select a trial design.

2. Investigate to ensure ductility.

3. Make sure the trial design is suitable for wind (or other nonseismic) lateral loadings using the IBC.

    Note that many section properties determined in accordance with the IBC are different from those indicated in the *Provisions* so section properties will have to be determined multiple times. The IBC portion of the calculation is not included in this example.

[2003 *Provisions* and the 2003 IBC both adopt ACI 530-02 by reference, so the section properties should be the same for both documents.]

4. Calculate midheight deflection due to wind by the IBC. (While the *Provisions* have story drift requirements, they do not impose a midheight deflection limit for walls).

5. Calculate seismic demand.

6. Determine seismic resistance and compare to demand determined in Step 5.

Proceed with these steps as follows:

9.1.5.3.1 Trial design

A trial design of #7 bars at 24 in. on center is selected. See Figure 9.1-3.

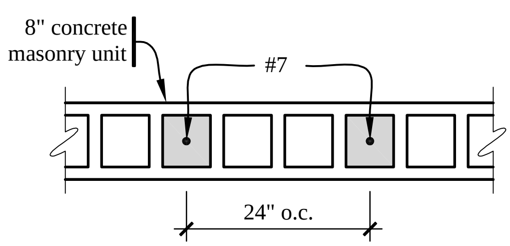


Figure 9.1-3 Trial design for 8-in.-thick CMU wall (1.0 in = 25.4 mm).

9.1.5.3.2 Investigate to ensure ductility

The critical strain condition corresponds to a strain in the extreme tension reinforcement (which is a single #7 centered in the wall in this example) equal to 1.3 times the strain at yield stress.

Based on *Provisions* Sec. 11.6.2.2[ACI 530, Sec. 3.2.3.5.1] for this case:

t = 7.63 in.
$d = t/2$ = 3.81 in.
$\varepsilon_m = 0.0025$
$\varepsilon_s = 1.3\varepsilon_y = 1.3(f_y/E_s) = 1.3(60 \text{ ksi} /29{,}000 \text{ ksi}) = 0.0027$

$$c = \left[\frac{\varepsilon_m}{(\varepsilon_m + \varepsilon_s)} \right] d = 1.83 \text{ in.}$$

$a = 0.8c$ = 1.46 in.

The Whitney compression stress block, a = 1.46 in. for this strain distribution, is greater than the 1.25 in. face shell width. Thus, the compression stress block is broken into two components: one for full compression against solid masonry (the face shell) and another for compression against the webs and grouted cells, but accounting for the open cells. These are shown as C_1 and C_2 in Figure 9.1-4:

$C_1 = 0.80 f_m' (1.25 \text{ in.}) b = (0.80)(2 \text{ ksi})(1.25)(24) = 48$ kips (for a 24-in. length)
$C_2 = 0.80 f_m' (a - 1.25 \text{ in.})(8 \text{ in.}) = (0.80)(2 \text{ ksi})(1.46-1.25)(8) = 2.69$ kips (for a 24-in. length)

The 8-in. dimension in the C_2 calculation is for combined width of grouted cell and adjacent mortared webs over a 24-in. length of wall. The actual width of one cell plus the two adjacent webs will vary with various block manufacturers, and may be larger or smaller than 8 in. The 8-in. value has the benefit of simplicity and is correct when considering solidly grouted walls.

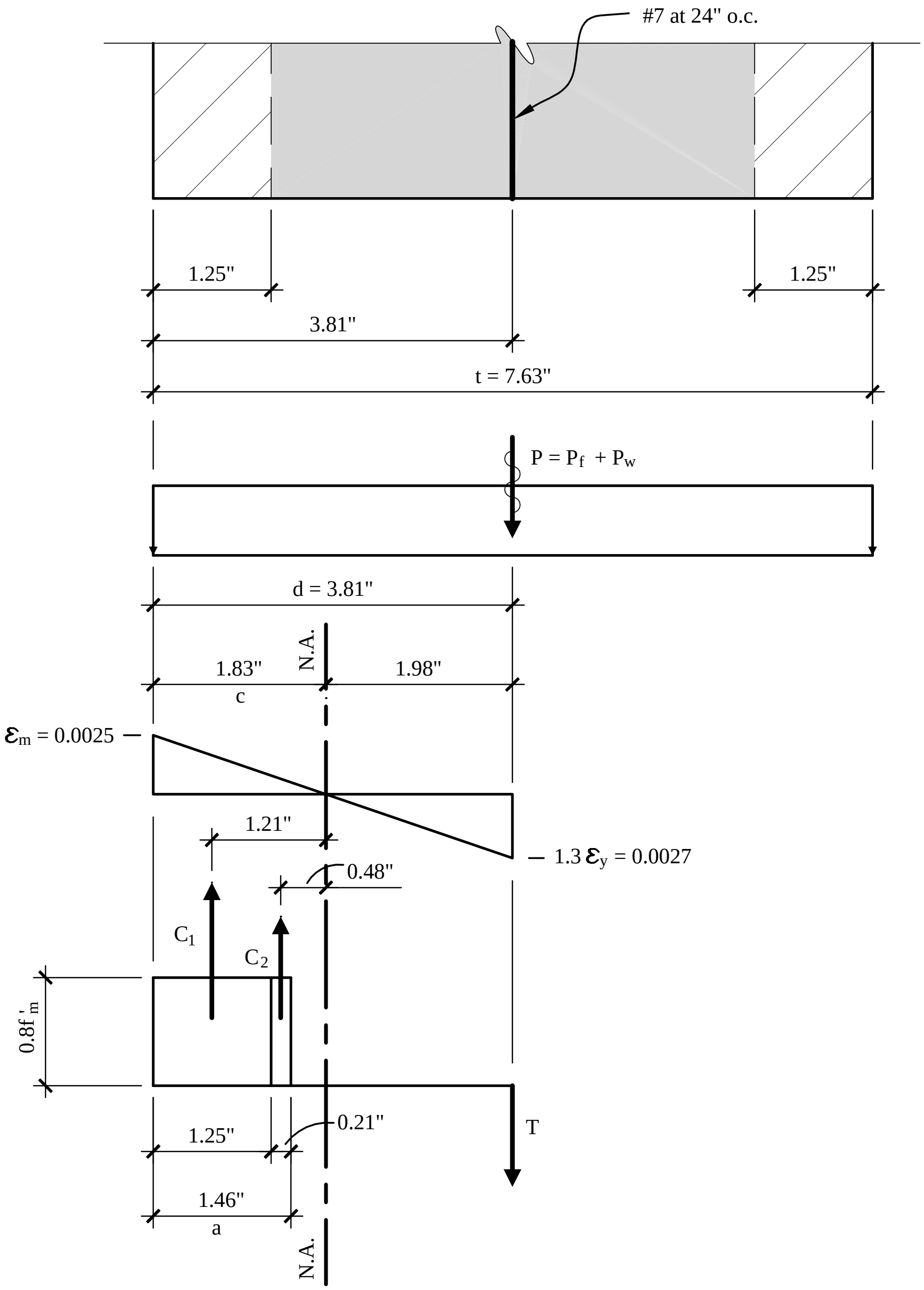


Figure 9.1-4 Investigation of out-of-plane ductility for the 8-in.-thick CMU side walls (1.0 in = 25.4 mm).

T is based on 1.25 F_y (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]:

$$T = 1.25F_yA_s = (1.25)(60 \text{ ksi})(0.60 \text{ in.}^2) = 45 \text{ kips (for a 24-in. length)}$$

Use unfactored P (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]:

$$P = (P_f + P_w) = (20 \text{ psf } (10 \text{ ft.}) + 65 \text{ psf } (16 \text{ ft.})) = 1.24 \text{ klf} = 2.48 \text{ kips (for a 24-in. length)}$$

Check $C_1 + C_2 > T + P$:

$T + P$ = 47.5 kips
$C_1 + C_2$ = 50.7 kips > 47.5 kips. OK

The compression capacity is greater than the tension capacity; therefore, the ductile failure mode criterion is satisfied.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (1.5 times) and axial force to consider when performing the ductility check (factored loads).]

9.1.5.3.3 Check for wind load using the IBC

Load factors and section properties are not the same in the IBC and the *Provisions* (The wind design check is beyond the scope of this seismic example so it is not presented here.) Both strength and deflection need to be ascertained in accordance with IBC.

Note that, for comparison, selected properties for the *Provisions* (and IBC) ductility check, IBC wind strength check, and *Provisions* seismic strength check are tabulated below. Keeping track of which version of a given parameter is used for each of the calculations can get confusing; be careful to apply the correct property for each analysis.

Table 9.1-2 Comparison of Variables (explanations in the following text)

Parameter	*Provisions* Ductility Calculation	*Provisions* Strength Calculation	*IBC* Wind Calculation
P	1.24 klf	0.87 & 1.74 klf	1.12 klf
E_m	NA	1,500,000 psi	1,800,000 psi
f_r	NA	80 psi	112 psi
w	NA	26 psf	19 psf (service)
ε_s	0.0027	NA	NA
d	3.82 in.	3.82 in.	3.82 in.
c	1.83 in.	1.25 in.	1.25 in.
a	1.46 in.	1.00 in.	1.00 in.
$C_{res} = C_1+C_2$	50.1 kips	52.1 kips	56.4 kips
$n = E_s/E_m$	NA	19.33	16.11
I_g	NA	355 in.^{4}	355 in.^{4}
S_g	NA	93.2 in.^{3}	93.2 in.^{3}
A_{se}	NA	0.32 in.^{2}/ft	0.32 in.^{2}/ft
I_{cr}	NA	48.4 in.^{4}/ft	48.4 in.^{4}/ft
$M_{cr} = f_r S$	NA	7.46 in.-kips	10.44 in.-kips
δ_{allow}	NA	NA	2.32 in.

NA = not applicable, 1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 ksi = 6.98 MPa, 1.0 in.-kip = 0.113 kN-m.

9.1.5.3.4 Calculate midheight deflection due to wind by the IBC

The actual calculation is not presented here. For this example the midheight deflection was calculated using IBC Eq. 21-41[ACI 530, Eq. 3-31] with I_{cr} = 47.3 in.^{4} per ft. Using IBC Eq. 21.41[ACI 530, Eq. 3-31], the calculated deflection is 2.32 in., which is less than 2.35 in. = 0.007h (IBC Eq. 21-39[ACI 530, Eq. 3-29]).

9.1.5.3.5 Calculate seismic demand

For this case, the two load factors for dead load apply: 0.7D and 1.4D. Conventional wisdom holds that the lower dead load will result in lower moment-resisting capacity of the wall so the 0.7D load factor would be expected to govern. However, the lower dead load also results in lower P-delta so both cases should be checked. (As it turns out, the higher factor of 1.4D governs).

Check moment capacity for 0.7D:

$P_u = 0.7(P_f + P_w)$.

For this example, the iterative procedure for addressing P-delta from Amrhein will be used, not *Provisions* Eq. 11.5.4.3[ACI 530, Commentary Sec. 3.1.5.3] which is intended for in-plane deflections:

Roof load, P_f = 0.7(0.2 klf) = 0.14 klf
Eccentricity, e = 7.32 in. (distance from wall centerline to roof reaction centerline)
Modulus of elasticity (*Provisions* Eq. 11.3.10.2 [ACI 530, 1.8.2.2]), $E_m = 750 f_m'$ = 1,500,000 psi

[Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1.]

Modular ratio, $n = \dfrac{E_s}{E_m} = 19.3$

The modulus of rupture (f_r) is found in *Provisions* Table 11.3.10.5.1[ACI 530, Sec. 3.1.7.2.1]. The values given in the table are for either hollow CMU or fully grouted CMU. Values for partially grouted CMU are not given; Footnote a indicates that interpolation between these values must be performed. As illustrated in Figure 9.1-6, the interpolated value for this example is 80 psi:

(f_r - 50 psi)/(103 in.^{2} - 60 in.^{2}) = (136 psi - 50 psi)/(183 in.^{2} - 60 in.^{2})
f_r = 80 psi
I_g = 355 in.^{4}/ft
S_g = 93.2 in.^{3}/ft
$M_{cr} = f_r S_g$ = 7460 in-lb/ft

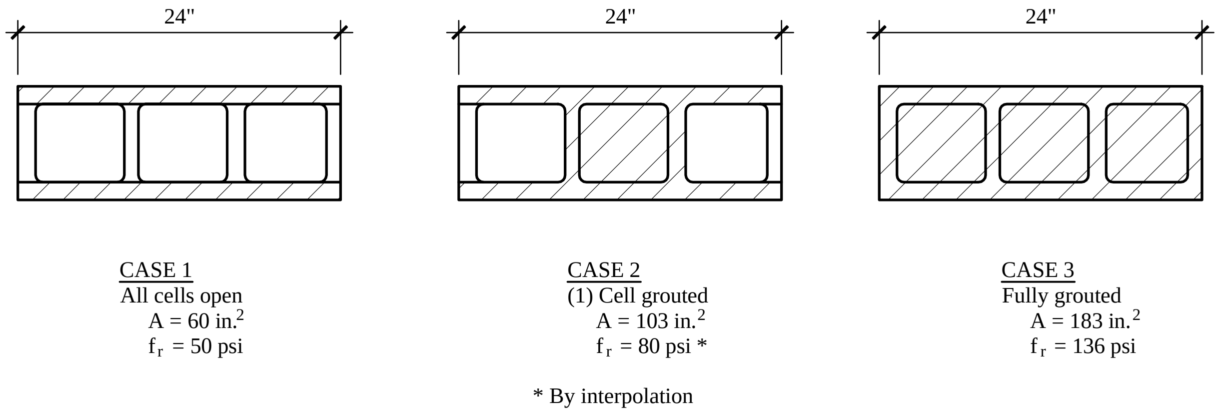


Figure 9.1-6 Basis for interpolation of modulus of rupture, f_r (1.0 in = 25.4 mm, 1.0 psi = 6.89 kPa).

Refer to Figure 9.1-7 for determining I_{cr}. The neutral axis shown on the figure is not the conventional neutral axis by linear analysis; instead it is the plastic centroid, which is simpler to locate, especially when the neutral axis position results in a T beam cross-section. (For this wall, the neutral axis does not produce a T section, but for the other wall in this building, a T section does result.) Cracked moments of inertia computed by this procedure are less than those computed by linear analysis but generally not so much less that the difference is significant. (This is the method used for computing the cracked section moment of inertia for slender walls in the standard for concrete structures, ACI 318.) Axial load does enter the computation of the plastic neutral axis and the effective area of reinforcement. Thus:

P = 1.24 klf
T = ((0.60 in.^{2})/(2 ft.))(60 ksi) = 18.0 klf
$C = T + P$ = 19.24 klf
$a = C/(0.8\, f'_m b)$ = (19.24 klf)/(0.8(2.0 ksi)(12 in./ft.)) = 1.002 in.
$c = a/0.8$ = 1.253 in.
$I_{cr} = nA_{se}(d\text{-}c)^2 + bc^3/3$ = 19.33(0.30 in.^{2} + (1.24 klf)/60 ksi)(3.81 in. - 1.25 in.)^2 + (12 in./ft)(1.25 in.)^3/3 = 4.84 in.^{4}/ft

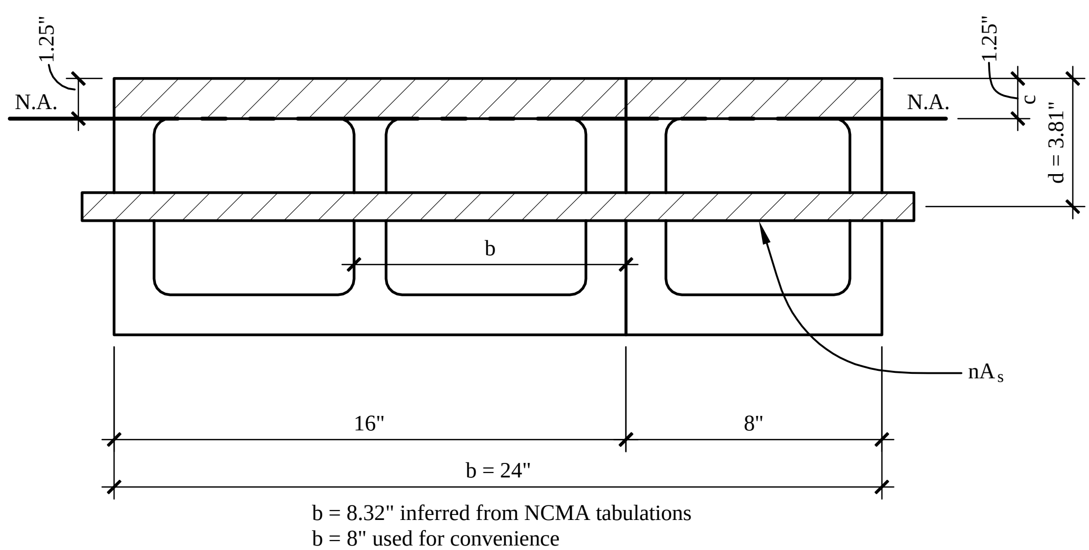


Figure 9.1-7 Cracked moment of inertia (I_{cr}) for 8-in.-thick CMU side walls (1.0 in = 25.4 mm).

Note that I_{cr} could be recomputed for P = 0.7D and P = 1.4D but that refinement is not pursued in this example.

The standard technique is to compute the secondary moment in an iterative fashion as shown below:

Axial load

$$P_u = 0.7(P_f + P_w) = 0.7(0.2 \text{ klf} + 1.04 \text{ klf}) = 0.868 \text{ klf}$$

First iteration

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{0.868 + (0.60)(60)}{60} = 0.614 \text{ in.}^2 / 2 \text{ ft.} = 0.312 \text{ in.}^2/\text{ft}$$

$$M_{u1} = w_u h^2/8 + P_0 e + (P_0 + P_w)\Delta$$

$$M_{u1} = \frac{(26 \text{ psf}/12)(336 \text{ in.})^2}{8} + (140 \text{ plf})\left(\frac{7.32 \text{ in.}}{2}\right) + (140\text{plf} + 728 \text{ plf})(0)$$

$$M_{u1} = 31{,}088 \text{ in.-lb/lf} > M_{cr} = 7460$$

$$\Delta_{s1} = \frac{5(7460)(336)^2}{48(1{,}500{,}000)(355)} + \frac{5(31{,}088 - 7460)(336)^2}{48(1{,}500{,}000)(48.4)} = 0.165 + 3.827 = 3.99 \text{ in.}$$

Second iteration

$$M_{u2} = 30{,}576 + 512 + (140 + 728)(3.99) = 34{,}551 \text{ in.-lb}$$

$$\Delta_{s2} = 0.165 + \frac{5(34{,}551 - 7460)(336)^2}{48(1{,}500{,}000)(48.4)} = 0.165 + 4.388 = 4.55 \text{ in.}$$

Third iteration

$$M_{u3} = 30{,}576 + 512 + (140 + 728)(4.55) = 35{,}040 \text{ in.-lb/lf}$$

$$\Delta_{s3} = 0.165 + \frac{5(35{,}040 - 7460)(336)^2}{48(1{,}500{,}000)(48.4)} = 0.165 + 4.467 = 4.63 \text{ in.}$$

Convergence check

$$\frac{4.63 - 4.55}{4.55} = 1.8\% < 5\%$$

M_u = 35,040 in.-lb (for the 0.7D load case)

Using the same procedure, find M_u for the 1.4D load case. The results are summarized below:

First iteration

P = 7360 plf
M_{u1} = 31,601 in.-lb/ft

Δ_{u1} = 4.08 in.

Second iteration

M_{u2} = 38,684 in.-lb/ft
Δ_{u2} = 5.22 in.

Third iteration

M_{u3} = 40,667 in.-lb/ft
Δ_3 = 5.54 in.

Fourth iteration

M_{u4} = 41,225 in.-lb/ft
Δ_{u4} = 5.63 in.

Check convergence

$$\frac{5.63-5.54}{5.54}=1.7\%<5\%$$

M_u = 41,225 in.-lb (for the 1.4*D* load case)

9.1.5.3.6 Determine flexural strength of wall

Refer to Figure 9.1-8. As in the case for the ductility check, a strain diagram is drawn. Unlike the ductility check, the strain in the steel is not predetermined. Instead, as in conventional strength design of reinforced concrete, a rectangular stress block is computed first and then the flexural capacity is checked.

$T = A_s f_y$ = (0.30 in.^{2}/ft.)60 ksi = 18.0 klf

The results for the two axial load cases are tabulated below.

Load Case	0.7*D* + *E*	1.4*D* + *E*
Factored *P*, klf	0.87	1.74
$T + P = C$, klf	18.87	19.74
$a = C / (0.8f'_m b)$, in.	0.981	1.028
$M_N = C (d - a/2)$, in.-kip/ft.	62.6	65.1
$\varphi M_N = 0.85M_N$, in.-kip/ft.	53.2	55.3
M_U, in.-kip/ft.	35.0	41.2
Acceptance	OK	OK

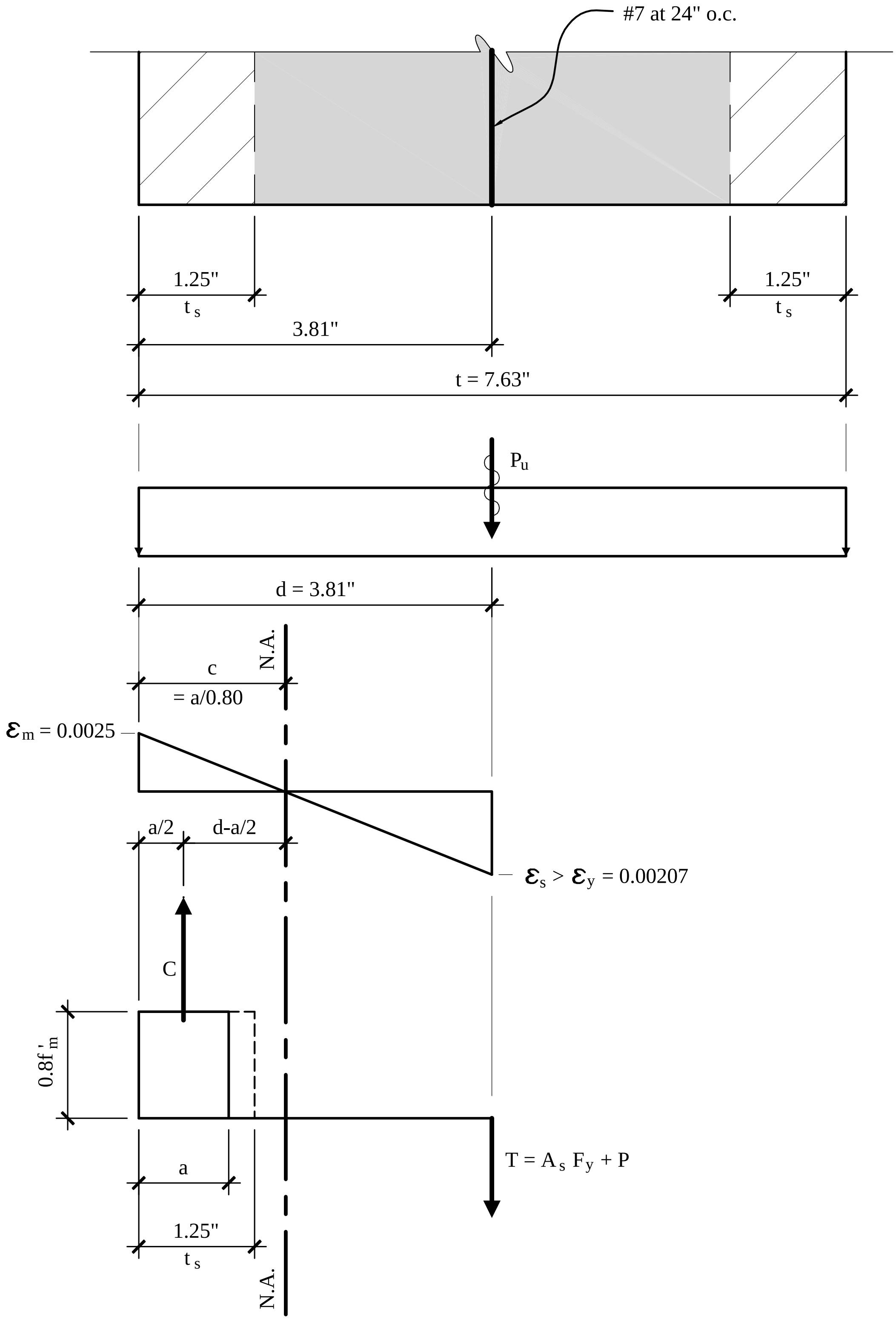


Figure 9.1-8 Out-of-plane strength for 8-in.-thick CMU walls (1.0 in = 25.4 mm).

Note that wind actually controls the stiffness and strength out-of-plane and that this is only a “tentative” acceptance for seismic. The *Provisions* requires a check of the combined orthogonal loads in accordance with *Provisions* Sec. 5.2.5.2, Item a [Sec. 4.4.2.3]. However, as discussed below, a combined orthogonal load check was deemed unnecessary for this example.

9.1.5.4 In-Plane Flexure

In-plane calculations for flexure in masonry walls include two items per the *Provisions*:

1. Ductility check and
2. Strength check.

It is recognized that this wall is very strong and stiff in the in-plane direction. In fact, most engineers would not even consider these checks necessary in ordinary design. The ductility check is illustrated here for two reasons: to show a method of implementing the requirement and to point out an unexpected result. (In the authors' opinion, the *Provisions* should reconsider the application of the ductility check where the M/Vd_v ratio is substantially less than 1.0.)

9.1.5.4.1 Ductility check

Provisions Sec. 11.6.2.2 [ACI 530, 3.2.3.5.1] requires that the critical strain condition correspond to a strain in the extreme tension reinforcement equal to 5 times the strain associated with F_y. This calculation uses unfactored gravity loads. (See Figure 9.1-9.)

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

$$P = P_w + P_f = (0.065 \text{ ksf } (30 \text{ ft.}) + 0.02 \text{ ksf } (10 \text{ ft.}))(200 \text{ ft.}) = 430 \text{ kips}$$

P is at the base of the wall rather than at the midheight.

$$c = \left(\frac{\varepsilon_m}{\varepsilon_m + \varepsilon_s}\right) d = \left(\frac{0.0025}{0.0025 + 0.0103}\right) 200 \text{ ft} = 38.94 \text{ ft}$$

$$a = 0.8c = 31.15 \text{ ft} = 373.8 \text{ in.}$$

$$C_m = 0.8 f_m' a b_{avg} = 2{,}560 \text{ kips}$$

Where b_{avg} is taken from the average area used earlier, 51.3 in.^{2}/ft.; see Figure 9.1-9 for locations of tension steel and compression steel (the rebar in the compression zone will act as compression steel). From this it can be seen that:

$$T_{s1} = (1.25 f_y)\left(\frac{40.27}{(2)(2 \text{ ft o.c.})}\right)(0.60) = 453 \text{ kips}$$

$$T_{s2} = (1.25 f_y)\left(\frac{120.79}{2}\right)(0.60) = 2{,}718 \text{ kips}$$

$$C_{s1} = f_y\left(\frac{6.73}{2 \text{ ft. o.c.}}\right)(0.60) = 121 \text{ kips}$$

$$C_{s2} = (f_y)\left(\frac{32.21}{(2)(2)}\right)(0.60) = 290 \text{ kips}$$

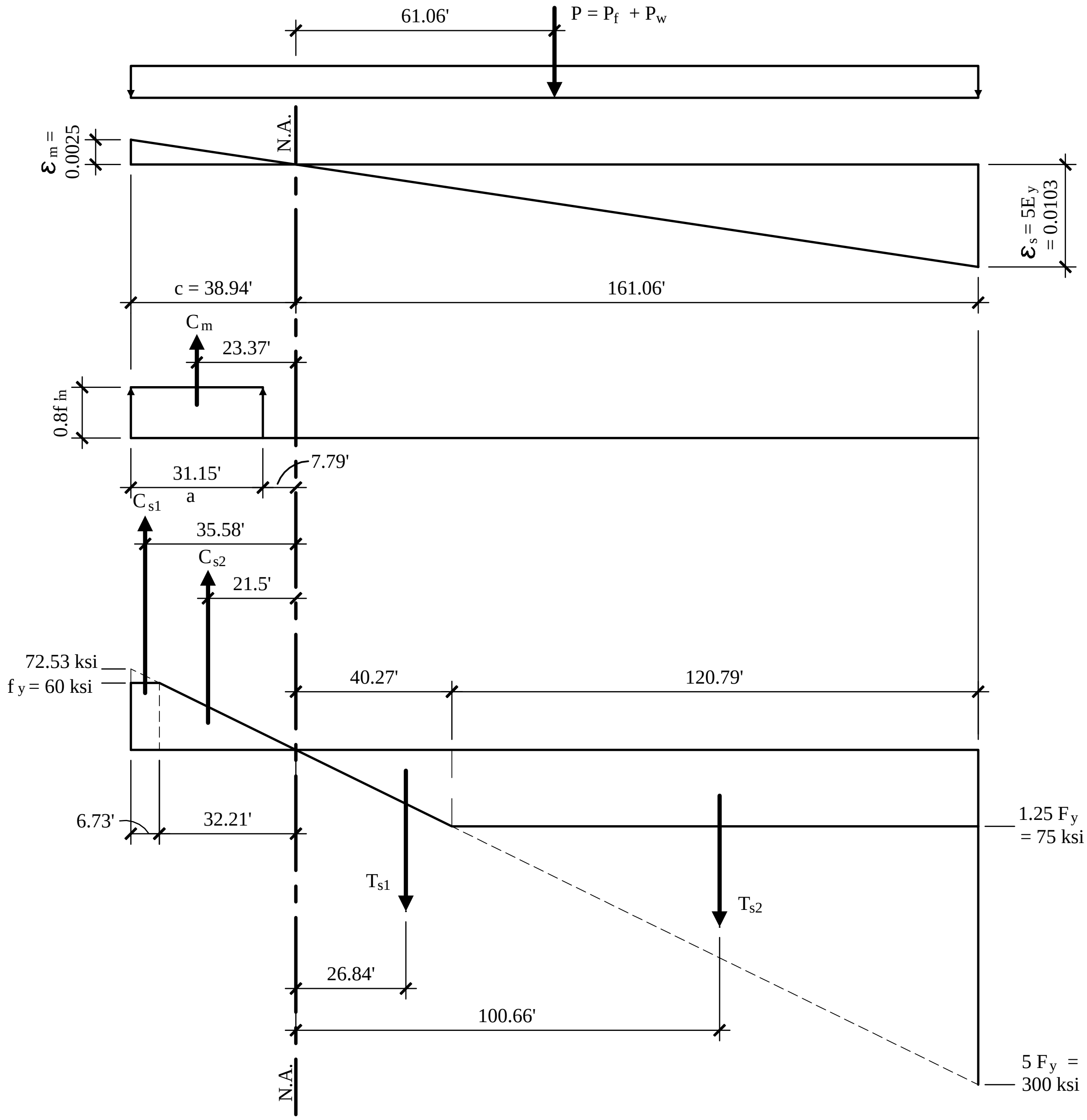


Figure 9.1-9 In-plane ductility check for side walls (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

Note that some authorities would not consider the compression resistance of reinforcing steel that is not enclosed within ties. The *Provisions* clearly allows inclusion of compression in the reinforcement.

$$\Sigma C > \Sigma T$$

$$C_m + C_{s1} + C_{s2} > P + T_{s1} + T_{s2}$$

$$2{,}560 + 121 + 290 = 2{,}971 < 3{,}601 = 430 + 453 + 2{,}718$$

Therefore, there is not enough compression capacity to ensure ductile failure.

In order to ensure ductile failure with #7 bars at 24 in. on center, one of the following revisions must be made: either add (3,601 kips - 2,971 kips) = 630 kips to C_m or reduce T by reducing A_s. Since this amount of reinforcement is needed for out-of-plane flexure, A_s cannot be reduced.

Try filling all cells for 10 ft - 0 in. from each end of the wall. As shown in Figure 9.1-10, this results in 10 additional grouted cells.

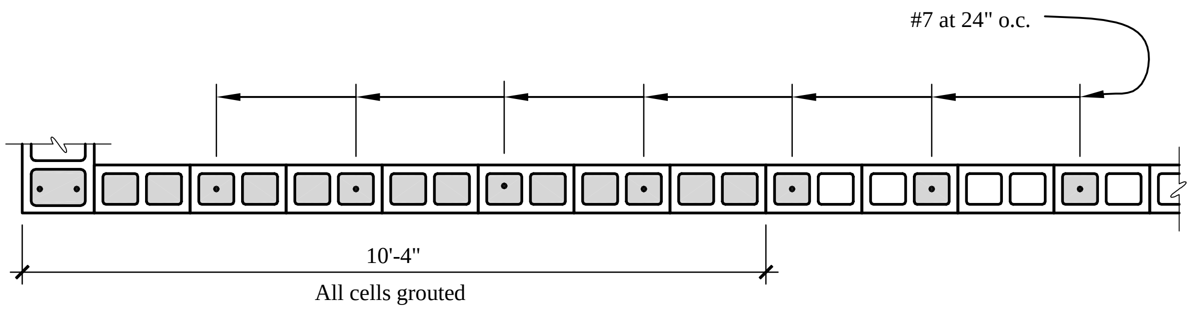


Figure 9.1-10 Grout cells solid within 10 ft of each end of side walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

Area of one grouted cell:	(8 in.)(5.13 in.) = 41 in.^{2}	
Volume of grout for one cell:	(6 in.)(5.13 in.)(30 ft.)/(144 in.^{2}/ft.^{2}) = 6.41 ft.^{3}	
Weight of grout for one cell:	(0.140 kcf)(6.41) = 0.90 kips/cell	
Additional P:	(10 additional cells)(0.9) = 9.0 kips	
Additional C_m:	0.8 f_m' (41 in.^{2})(10 cells) = 656 kips	
Additional C_m - additional P:	656 kips - 9 kips = 647 kips	
Net additional C_m:	647 kips > 630 kips	OK

or, as expressed in terms of the above equation:

$\Sigma C > \Sigma T$

2,971 kips + 656 kips = 3,627 kips > 3,610 kips = 3,601 kips + 9 kips OK

Since C > T, the ductile criterion is satisfied.

This particular check is somewhat controversial. In the opinion of the authors, flexural yield is feasible for walls with *M/Vd* in excess of 1.0; this criterion limits the compressive strain in the masonry, which leads to good performance in strong ground shaking. For walls with *M/Vd* substantially less than 1.0, the wall will fail in shear before a flexural yield is possible. Therefore, the criterion does not affect performance. Well distributed and well developed reinforcement to control the shear cracks is the most important ductility attribute for such walls.

9.1.5.4.2 Strength check

The wall is so long with respect to its height that in-plane strength for flexure is acceptable by inspection.

9.1.5.5 Combined Loads

Combined loads are not calculated here because the in-plane strength is obviously very high. Out-of-plane resistance governs the flexural design.

9.1.5.6 Shear in Longitudinal Walls (Side Walls)

Compute out-of-plane shear at base of wall in accordance with *Provisions* Sec. 5.2.6.2.7[Sec. 4.6.1.3]:

$F_p = 0.4S_{DS}W_c$ = (0.4)(1.00)(65 psf)(28 ft/2) = 364 plf.

Information from the flexural design from Sec. 9.1.5.3 is needed to determine the required shear strength based upon development of the flexural capacity. The ratio of φM_N to M_U is the largest for the load case $0.7D + E$. The load that would develop the flexural capacity is approximated by ratio (a second P-delta analysis does not seem justified for this check):

$$w' = w \times \frac{\phi M_N / \phi}{M_U} = 26 \text{ psf} \times \frac{53.2/0.85}{35.0} = 46.5 \text{ psf}$$

1.25 times this results in a load for shear design of 58 psf. Thus V_U = (58 psf)(28 ft. /2) = 818 plf. The capacity of computed per *Provisions* Eq.11.7.3.2[ACI 530, Eq. 3.2.1] :

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right]A_n\sqrt{f'_m} + 0.25\,P$$

M/Vd need not be taken larger than 1.0. A_n is taken as $b_w d$ = 8.32(3.81) = 31.7 in.^{2} per cell from Figure 9.17. Because this shear exists at both the bottom and the top of the wall, conservatively neglect the effect of *P*:

$$V_m = [4.0 - 1.75(1.0)](51.3\text{in.}^2 / 2\text{ ft.})\sqrt{2{,}000} + 0 = 1.595\text{ klf}$$

ϕV_m = (0.8)(1.595) = 1.28 klf > 0.81 klf

As indicated in Sec. 9.1.4.1 and Sec. 9.1.5.1, the in-plane demand at the base of the wall, V_u = 2.5(211 kips) = 528 kips, and the shear capacity, ϕV_m is larger than 904 kips.

For the purpose of understanding likely behavior of the building somewhat better, V_n is estimated more accurately for these long walls:

$M/Vd = h/l$ = 28/200 = 0.14
$P = 0.7D$ = 0.7(430) = 301 kip
V_m = [4.0 - 1.75(0.14)][200(51.3) + 2(10)91.5-51.3)](0.045) + 0.25(301) = 1870 + 75 = 1945 kip
$V_s = 0.5(A_v/s)f_y d$ = 0.5(0.62/4.0)(60)(200) = 930 kip
V_n = 1945 + 930 = 2875 kip
Maximum $V_n = 6\sqrt{f'_m}A$ = 6(0.045 ksi)(9234 in.^{2}) = 2493 < 2875 kip
φV_n = 0.8(2493) = 1994 kip
V_E = 211kip
V_n/V_E = 11.8 >> *R* used in design

In other words, it is unlikely that the long masonry walls will yield in either in-plane shear or flexure at the design seismic ground motion. The walls will likely yield in out-of-plane response and the roof diaphragm may also yield. The roof diaphragm for this building is illustrated in Sec. 10.2.

The combined loads for shear (orthogonal loading, per *Provisions* Sec. 5.2.5.2.2, Item a)[Sec. 4.4.2.3] are shown in Table 9.1-3.

Table 9.1-3 Combined Loads for Shear in Side Wall

	Out-of-Plane	In-Plane	Total
Case 1	1.00(810/1,280)+	0.30(528/1994)=	0.71 < 1.00 OK
Case 2	0.30(810/1,280)+	1.00(528/1994)=	0.45 < 1.00 OK

Values are in kips; 1.0 kip = 4.45 kN.

9.1.6 Transverse Walls

The transverse walls will be designed in a manner similar to the longitudinal walls. Complicating the design of the transverse walls are the door openings, which leave a series of masonry piers between the doors.

9.1.6.1 Horizontal Reinforcement

The minimum reinforcement, per *Provisions* Sec. 11.3.8.3[ACI 530, Sec.1.13.6.3] , is (0.0007)(11.625 in.)(8 in.) = 0.065 in.^{2} per course. The maximum spacing of horizontal reinforcement is 48 in., for which the minimum reinforcement is 0.39 in.^{2}. Two #4 in bond beams at 48 in. on center would satisfy the requirement. The large amount of vertical reinforcement would combine to satisfy the minimum total reinforcement requirement. However, given the 100-ft length of the wall, a larger amount is desired for control of restrained shrinkage as discussed in Sec. 9.1.5.1. Two #5 at 48 in. on center will be used.

9.1.6.2 Vertical Reinforcement

The area for each bay subject to out-of-plane wind is 20 ft wide by 30 ft high because wind load applied to the doors is transferred to the masonry piers. However, the area per bay subject to both in-plane and out-of-plane seismic is reduced by the area of the doors. This is because the doors are relatively light compared to the masonry. See Figures 9.1-12 and 9.1-13.

9.1.6.3 Out-of-Plane Flexure

Out-of-plane flexure will be considered in a manner similar to that illustrated in Sec. 9.1.5.3 . The design of this wall must account for the effect of door openings between a row of piers. The steps are the same as identified previously and are summarized here for convenience:

1. Select a trial design,
2. Investigate to ensure ductility,
3. Make sure the trial design is suitable for wind (or other non-seismic) lateral loadings using IBC,
4. Calculate midheight deflection due to wind by IBC,
5. Calculate the seismic demand, and
6. Determine the seismic resistance and compare to the demand determined in Step 5.

9.1.6.3.1 Trial design

A trial design of 12-in.-thick CMU reinforced with two #6 bars at 24 in. on center is selected. The self-weight of the wall, accounting for horizontal bond beams at 4ft on center, is conservatively taken as 103 psf. Adjacent to each door jamb, the vertical reinforcement will be placed into two cells. See Figure 9.1-11.

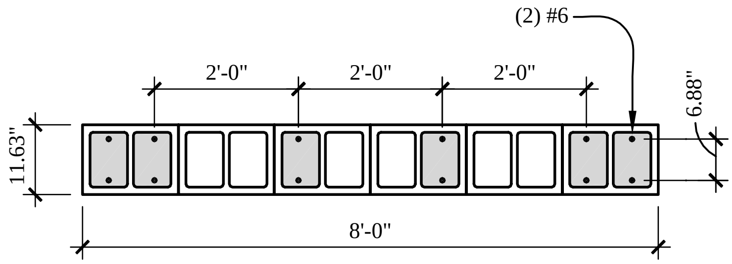


Figure 9.1-11 Trial design for piers on end walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

Next, determine the design loads. The centroid for seismic loads, accounting for the door openings, is determined to be 17.8 ft above the base. See Figures 9.1-12 and 9.1-13.

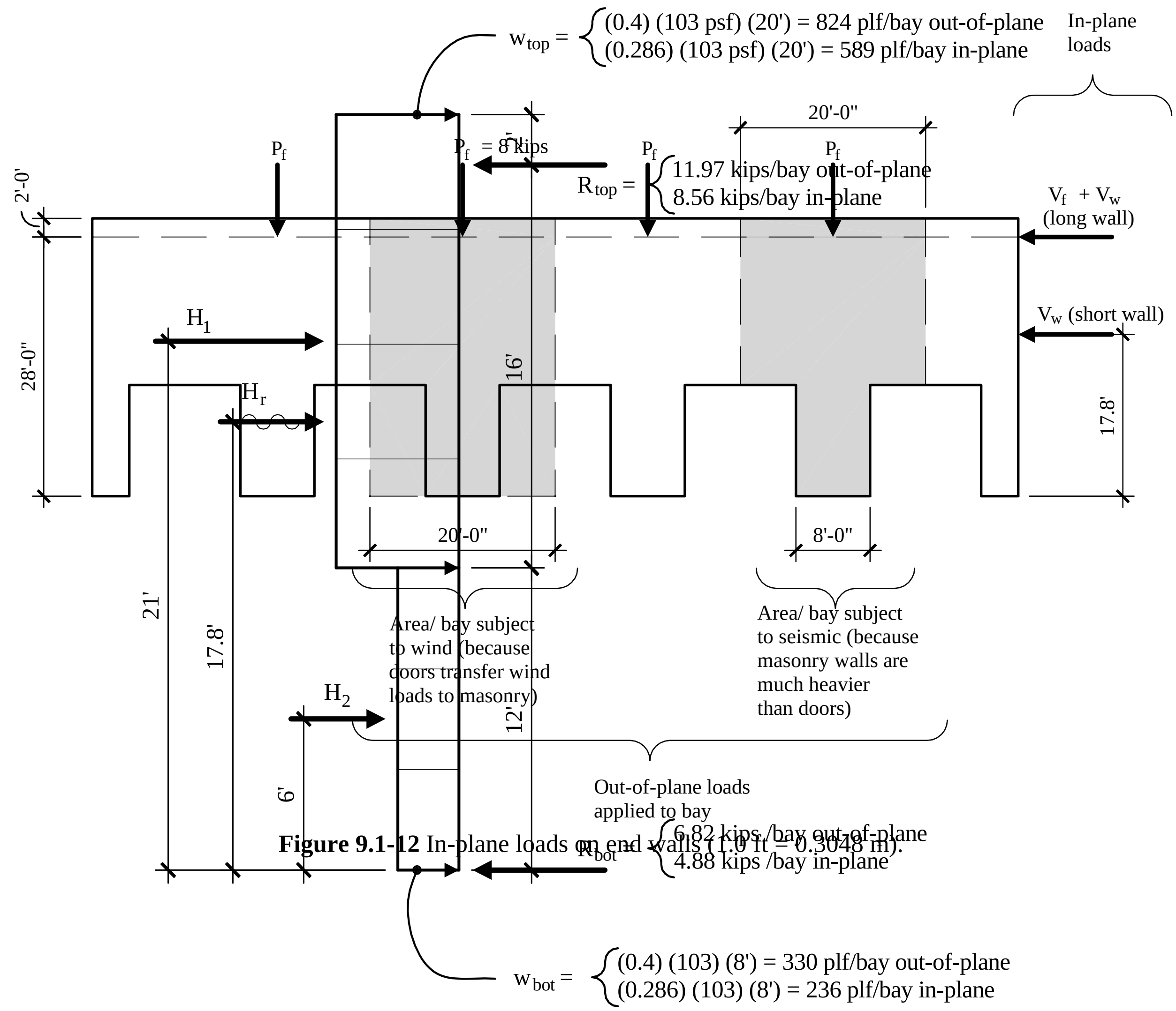


Figure 9.1-12 In-plane loads on end walls (1.0 ft = 0.3048 m).

Figure 9.1-13 Out-of-plane load diagram and resultant of lateral loads (1.0 ft = 0.3048 m, 1.0 lb = 4.45 N, 1.0 kip = 4.45 kN).

9.1.6.3.2 Investigate to ensure ductility

The critical strain condition is corresponds to a strain in the extreme tension reinforcement (which is a pair of #6 bars in the end cell in this example) equal to 1.3 times the strain at yield stress. See Figures 9.1-11 and 9.1-14.

For this case:

t = 11.63 in.
d = 11.63 - 2.38 = 9.25 in.
ε_m = 0.0025 (*Provisions* Sec. 11.6.2.1.b)[ACI 530, Sec. 3.2.2]
$\varepsilon_s = 1.3\,\varepsilon_y = 1.3\,(f_y/E_s)$ = 1.3 (60 ksi /29,000 ksi) = 0.0027 (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]

$$c = \left[\frac{\varepsilon_m}{(\varepsilon_m + \varepsilon_s)}\right] d = 4.45 \text{ in.}$$

$a = 0.8c$ = 3.56 in. (*Provisions* Sec. 11.6.2.2)

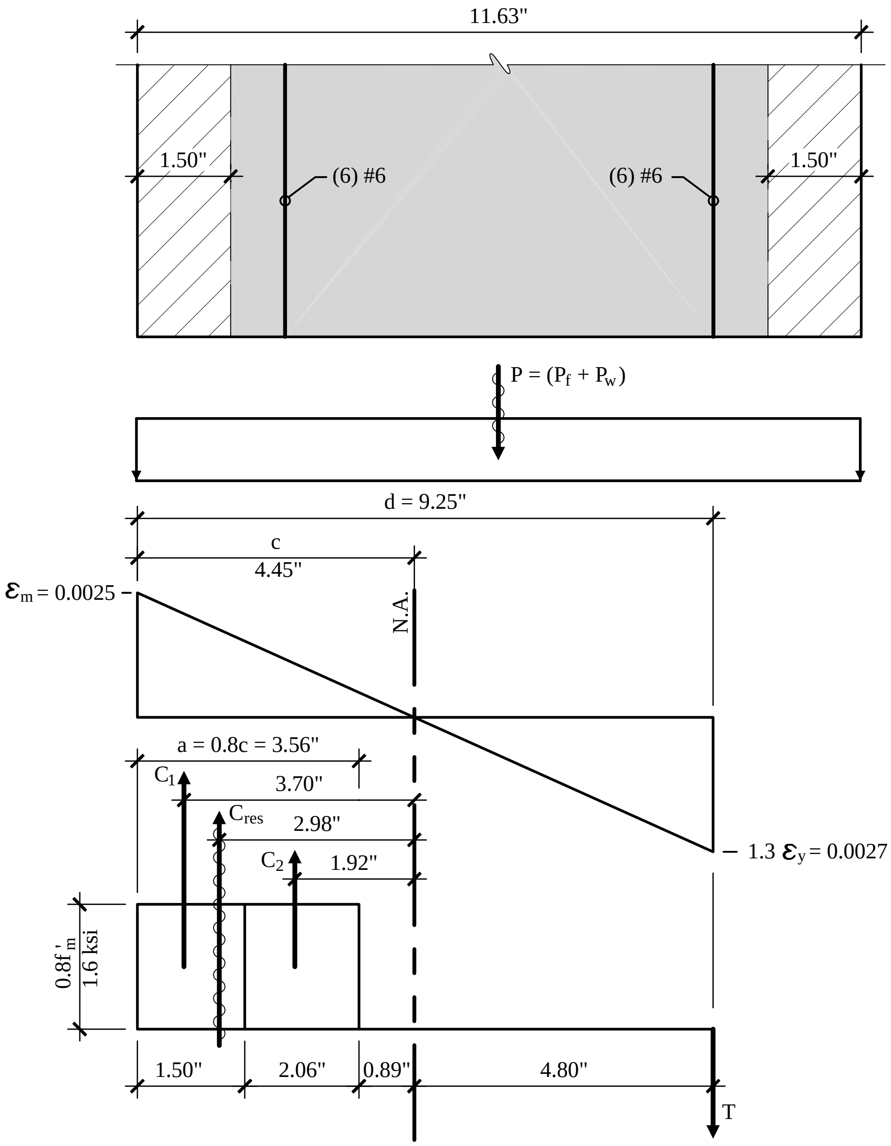


Figure 9.1-14 Investigation of out-of-plane ductility for end wall (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

Note that the Whitney compression stress block, a = 3.56 in. deep, is greater than the 1.50-in. face shell thickness. Thus, the compression stress block is broken into two components: one for full compression against solid masonry (the face shell) and another for compression against the webs and grouted cells but accounting for the open cells. These are shown as C_1 and C_2 on Figure 9.1-15. The values are computed using *Provisions* Sec. 11.6.2.1e:[ACI 530, 3.2.2.e];

C_1 = $0.80f_m'$ (1.50 in.)b = (0.80)(2 ksi)(1.50)(96) = 230 kips (for full length of pier)
C_2 = $0.80f_m'$ (a - 1.50 in.)(6(8 in.)) = (0.80)(2 ksi)(3.56 - 1.50)(48) = 158 kips

The 48 in. dimension in the C_2 calculation is the combined width of grouted cell and adjacent mortared webs over the 96-in. length of the pier.

T is based on $1.25F_y$ (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]:

$T \quad = 1.25F_yA_s = (1.25)(60 \text{ ksi})(6 \times 0.44 \text{ in.}^2) = 198 \text{ kips/pier}$
$P \quad = (P_f + P_w) = 8.0 \text{ k} + (0.103 \text{ ksf})(18 \text{ ft.})(20 \text{ ft.}) = 45.1 \text{ kips/pier}$

P is computed at the head of the doors:

$C_1 + C_2 > P + T$
388 kip > 243 kips

Since the compression capacity is greater than the tension capacity, the ductility criterion is satisfied.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (1.5 times) and axial force to consider when performing the ductility check (factored loads).]

9.1.6.3.3 Check for wind loading using IBC

Note that load factors and section properties are different in the IBC and the *Provisions*. Note also that wind per bay is over the full 20 ft wide by 30 ft high bay as discussed above. (The calculations are not presented here.)

9.1.6.3.4 Calculate midheight deflection due to wind by IBC

Although the calculations are not presented here, note that in Figure 9.1-15 the neutral axis position and partial grouting results in a T beam cross section for the cracked moment of inertia. Use of the plastic neutral axis is a simplification for computation of the cracked moment of inertia. For this example, midheight out-of-plane deflection is 1.27 in. < 2.35 in. = 0.007*h*, which is acceptable.

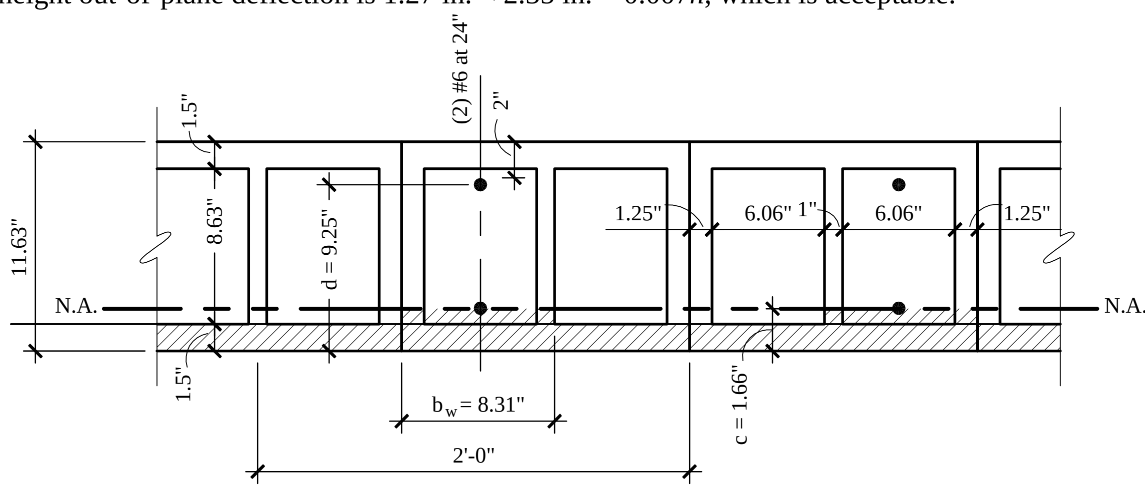


Figure 9.1-15 Cracked moment of inertia (I_{cr}) for end walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m). Dimension "c" depends on calculations shown for Figure 9.1-16.

9.1.6.3.5 Calculate Seismic Demand

For this example, the load combination with 0.7D has been used and, for this calculation, forces and moments over a single pier (width = 96 in.) are used. This does not violate the "$b > 6t$" rule (ACI 530 Sec. 7.3.3)[ACI 530, Sec. 3.2.4.3.3] because the pier is reinforced at 24 in. o.c. The use of the full width of the pier instead of a 24 in. width is simply for calculation convenience.

For this example, a P-delta analysis using RISA-2D was run. This resulted in:

Maximum moment, M_u = 66.22 ft-kips/bay = 66.22/20 ft = 3.31 klf (does not govern)
Moment at top of pier, M_u = 62.12 ft-kips/pier = 62.12 / 8 ft = 7.77 klf (governs)
Shear at bottom of pier, V_u = 6.72 kips/pier
Reaction at roof, V_u = 12.07 kips/bay
Axial force at base, R_u = 54.97 kips/pier

The shears do not agree with the reactions shown in Figure 9.1-13; because the results in Figure 9.1-13 do not include the P-delta consideration.

9.1.6.3.6 Determine moment resistance at the top of the pier

See Figure 9.1-16.

A_s = 6-#6 = 2.64 in.^{2}
d = 9.25 in.
T = 2.64(60) = 158.4 kip
$C = T + P$ = 203.5 kip
$a = C / (0.8f'_m b)$ = 203.5 / [0.8(2)96] = 1.32 in.

Because a is less than the face shell thickness (1.50 in.), compute as for a rectangular beam. Moments are computed about the centerline of the wall.

$$M_N = C\,(t/2 - a/2) + P\,(0) + T\,(d - t/2)$$
$$= 203.5(5.81 - 1.32/2) + 158.4(9.25-1.32/2) = 1593 \text{ in.-kip} = 132.7 \text{ ft.-kip}$$
$$\varphi M_N = 0.85(132.7) = 112.8 \text{ ft.-kip}$$

Because moment capacity at the top of the pier, ϕM_n = 112.8 ft-kips, exceeds the maximum moment demand at top of pier, M_u = 62.1 ft-kips, the condition is acceptable but note that this is only tentative acceptance.

The *Provisions* requires a check of the combined loads in accordance with *Provisions* 5.2.5.2, Item a [Sec. 4.4.2.3]. See Sec. 9.1.6.5 for the combined loads check.

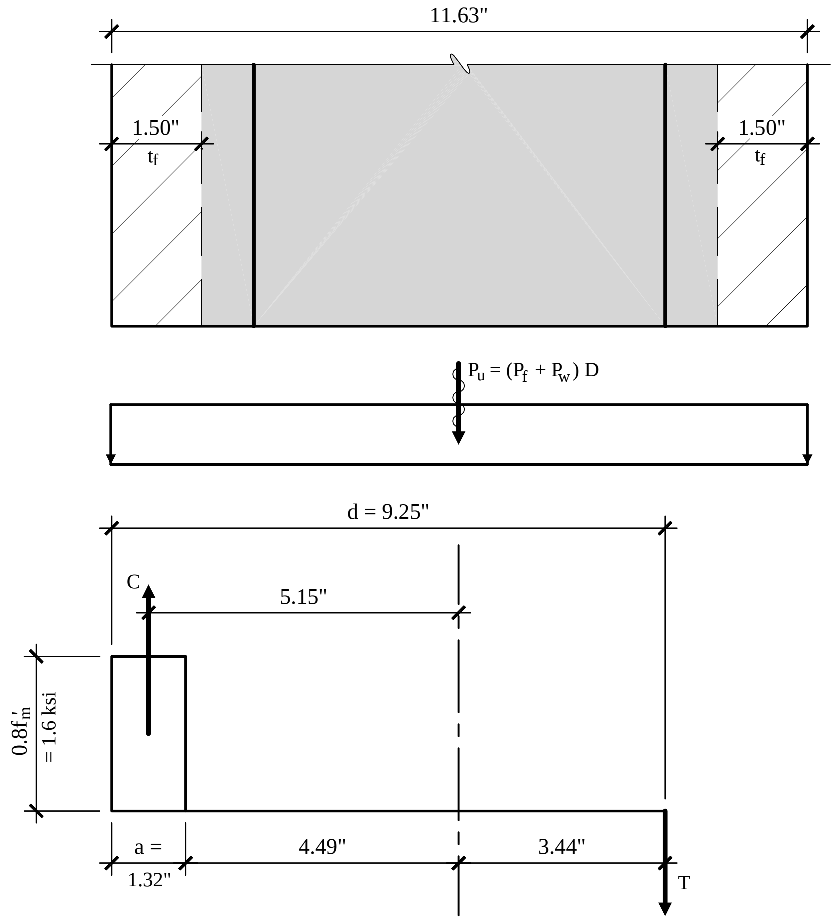


Figure 9.1-16 Out-of-plane seismic strength of pier on end wall (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

9.1.6.4 In-Plane Flexure

There are several possible methods to compute the shears and moments in the individual piers of the end wall. For this example, the end wall was modeled using RISA-2D. The horizontal beam was modeled at the top of the opening, rather than at its midheight. The in-plane lateral loads (from Figure 9.1-12) were applied at the 12-ft elevation and combined with joint moments representing transfer of the horizontal forces from their point of action down to the 12-ft elevation. Vertical load due to roof beams and the self-weight of the end wall were included. The input loads are shown on Figure 9.1-17. For this example:

w = (18 ft.)(103 psf) + (20 ft.)(20 psf) = 2.254 klf
H = (184 kip)/5 = 36.8 kip
M = 0.286((400 + 418)(28 - 12) + 470(17.8 - 12)) = 452 ft-kip

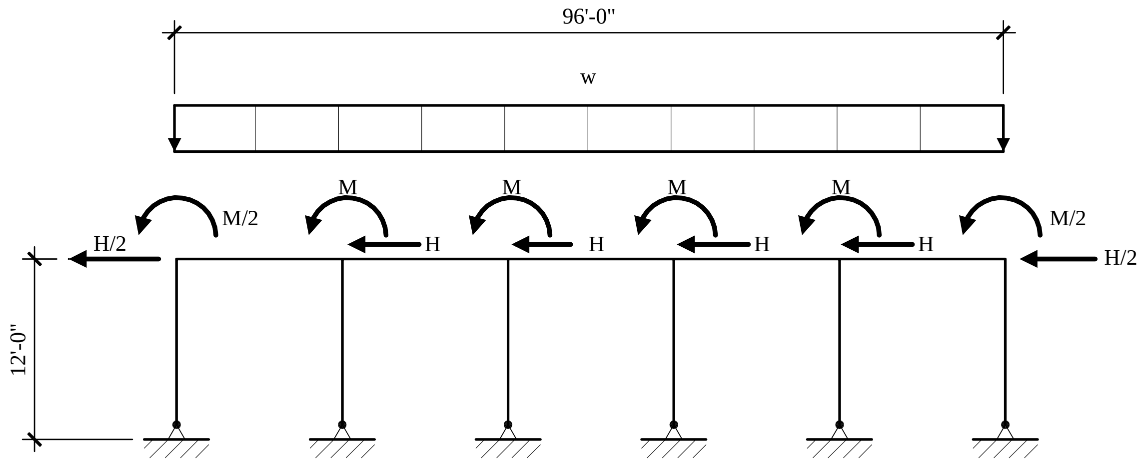


Figure 9.1-17 Input loads for in-plane end wall analysis (1.0 ft = 0.3048 m).

The input forces at the end wall are distributed over all the piers to simulate actual conditions. The RISA-2D frame analysis accounts for the relative stiffnesses of the 4-ft-and 8-ft-wide piers. The final distribution of forces, shears, and moments for an interior pier is shown on Figure 9.1-18.

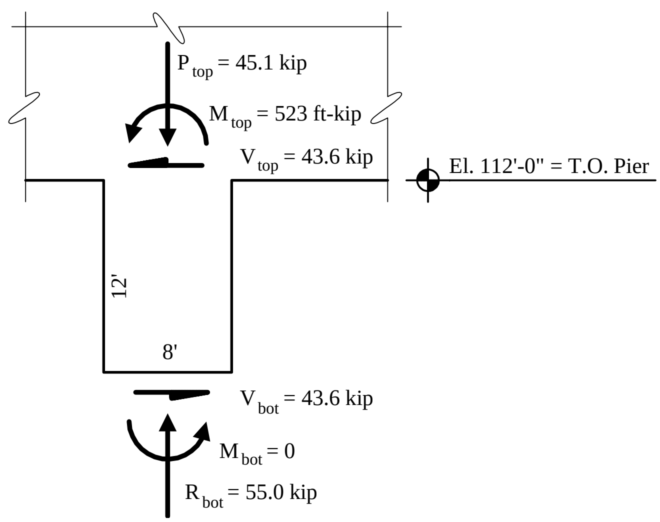


Figure 9.1-18 In-plane design condition for 8-ft-wide pier (1.0 ft = 0.3048 m).

As a trial design for in-plane pier design, use two #6 bars at 24 in. on center supplemented by adding two #6 bars in the cells adjacent to the door jambs (see Figure 9.1-19).

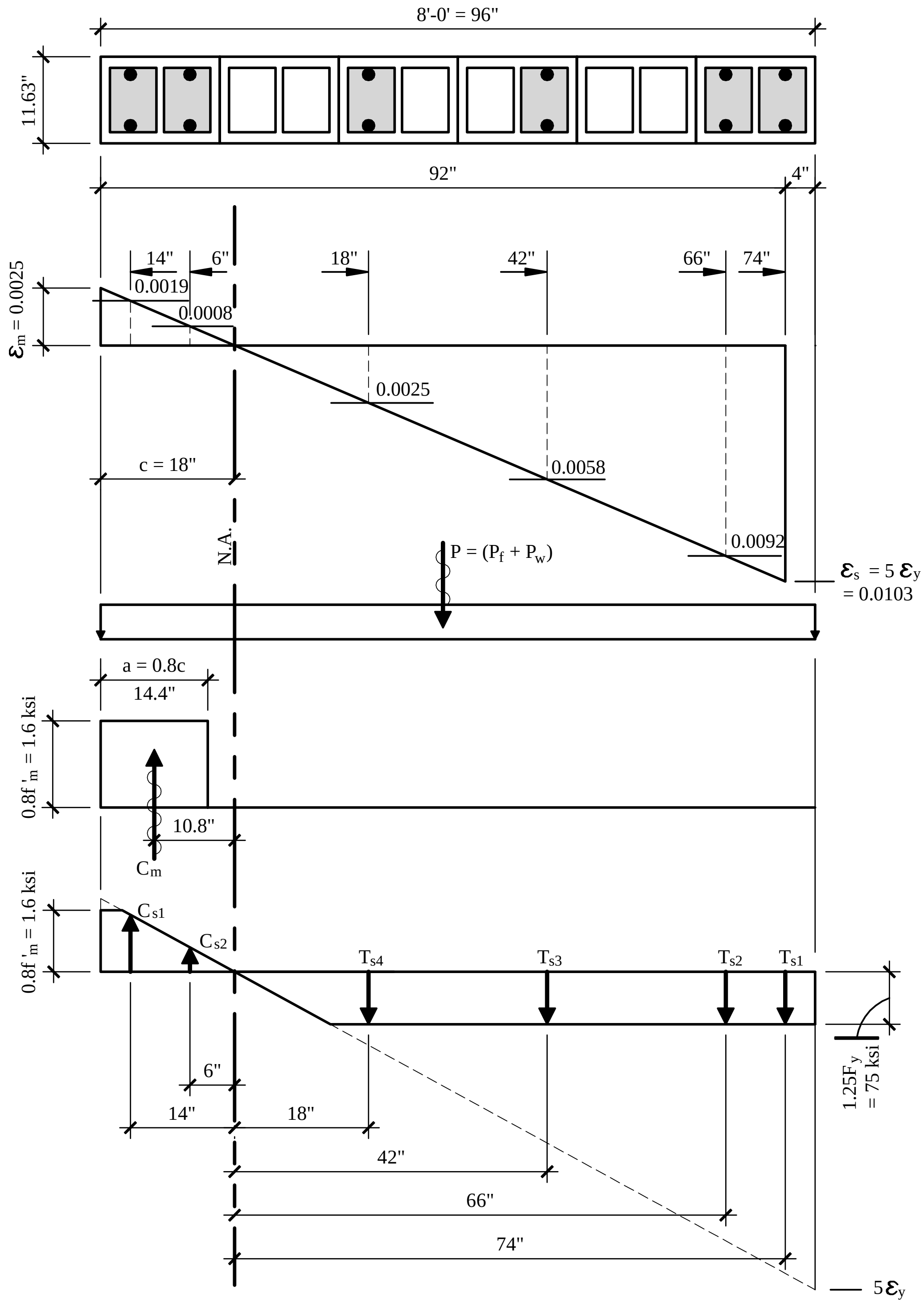


Figure 9.1-19 In-plane ductility check for 8-ft-wide pier (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

The design values for in-plane design at the top of the pier are:

	Unfactored	$0.7D + 1.0E$	$1.4D + 1.0E$
Axia	P = 45.1 kips	P_u = 31.6 kips	P_u = 63.2 kips
Shea	V = 43.6 kips	V_u = 43.6 kips	V_u = 43.6 kips
Mom	M = 523 ft-kips	M_u = 523 ft-kips	M_u = 523 ft-kips

The ductility check is illustrated in Figure 9.1-19:

$\varepsilon_m = 0.0025$
$\varepsilon_s = 5\varepsilon_y = (5)(60/29{,}000) = 0.0103$
d = 92 in.

From the strain diagram, the strains at the rebar locations are:

$\varepsilon_{66} = 0.0092$
$\varepsilon_{42} = 0.0058$
$\varepsilon_{18} = 0.0025$
$\varepsilon_{6} = 0.0008$
$\varepsilon_{14} = 0.0019$

To check ductility, use unfactored loads:

$P = P_f + P_w$ = (0.020 ksf)(20 ft)(20 ft) + (0.103 ksf)(18 ft)(20 ft)
P = 8 kips + 37.1 kips = 45.1 kips

$a = 0.8c$ = 14.4 in.
$C_m = (0.8f_m{}')ab$ = 1.6 ksi)(14.4 in.)(11.63 in.) = 268.0 kips
$T_{s1} = T_{s2} = T_{s3} = T_{s4} = (1.25F_y)(A_s)$ = (1.25)(60 ksi)(2 × 0.44 in.2) = 66 kips
$C_{s1} = F_yA_s(\varepsilon_{14}/\varepsilon_y)$ = (60 ksi)(2 × 0.44 in.^{2})(0.0019/0.00207) = 48.5 kips
$C_{s2} = F_yA_s(\varepsilon_6/\varepsilon_y)$ = (60 ksi)(2 × 0.44 in.^{2})(0.0008/0.00207) = 20.4 kips

$\Sigma C > \Sigma T + P$
$C_m + C_{s1} + C_{s2} > T_{s1} + T_{s2} + T_{s3} + T_{s4} + P$
268 + 48.5 + 20.4 > 66 + +66 + 66 + 66 + 45.1
336.9 kips > 309.1 kips

Since compression capacity exceeds tension capacity, ductile failure is ensured. Note that $1.25F_y$ is used for tension calculations per *Provisions* Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5-1] .

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

For the strength check, see Figure 9.1-20.

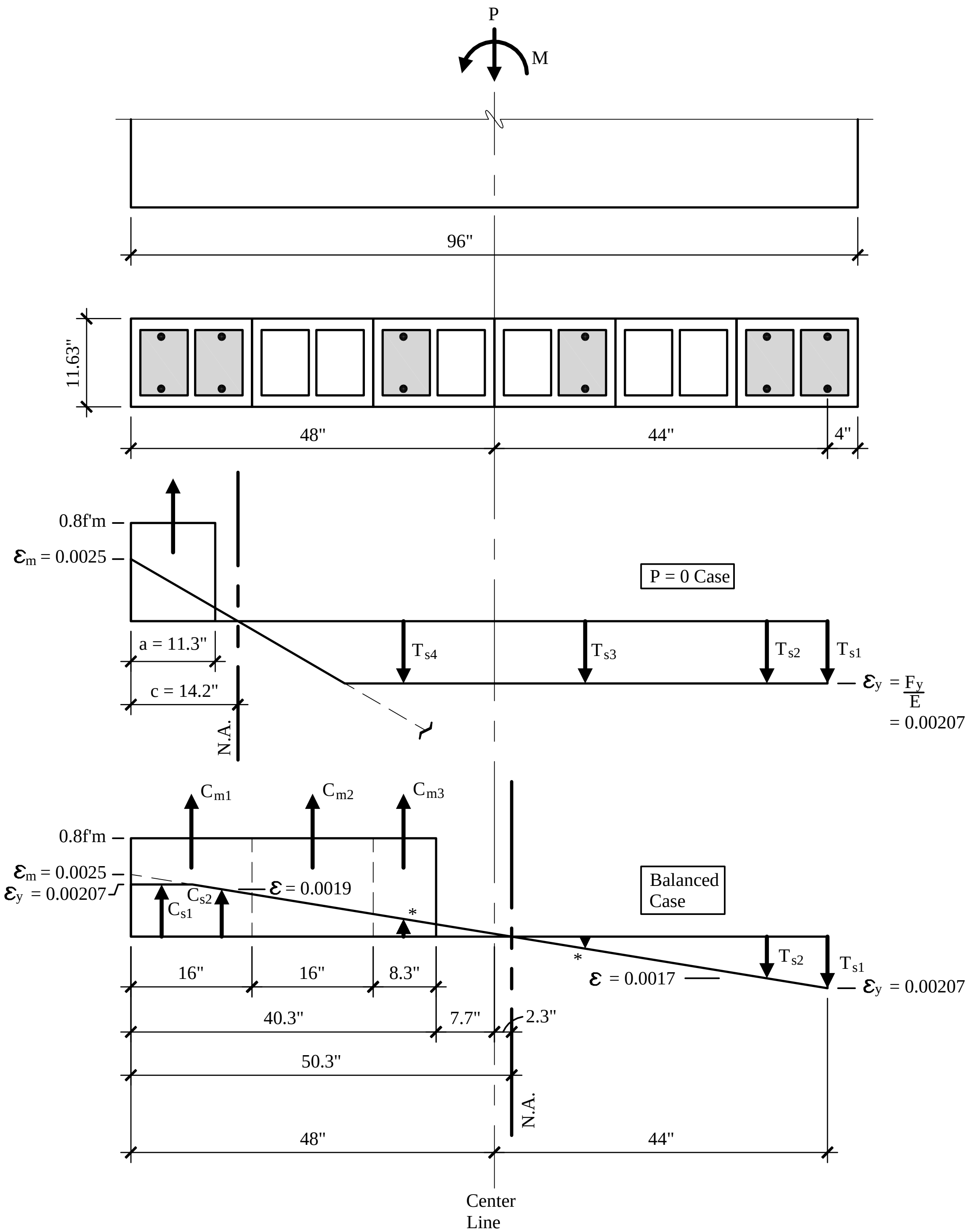


Figure 9.1-20 In-plane seismic strength of pier (1.0 in = 25.4 mm). Strain diagram superimposed on strength diagram for both cases. Note that low force in reinforcement is neglected in calculations.

To ascertain the strength of the pier, a ϕP_n - ϕM_n curve will be developed. Only the portion below the "balance point" will be examined as that portion is sufficient for the purposes of this example. Ductile failures occur only at points on the curve that are below the balance point so this is consistent with the overall approach).

For the P = 0 case, assume all bars in tension reach their yield stress and neglect compression steel (a conservative assumption):

$T_{s1} = T_{s2} = T_{s3} = T_{s4}$ = (2)(0.44 in.2)(60 ksi) = 52.8 kips
C_m = ΣT_s = (4)(52.8) = 211.2 kips
C_m = $0.8f'_m ab$ = (0.8)(2 ksi)a(11.63 in.) = 18.6a

Thus, a = 11.3 in. and $c = a/\phi$ = 11.3 / 0.8 = 14.2 in.

$\Sigma M_{cl} = 0$:
M_n = 42.35 $C_m + 44T_{s1} + 36T_{s2} + 12T_{s3} - 12T_{s4}$ = 13,168 in.-kips
ϕM_n = (0.85)(13,168) = 11,193 in.-kips = 933 ft-kips

For the balanced case:

d = 92 in.
ε = 0.0025
ε_y = 60/29,000 = 0.00207

$$c = \left(\frac{\varepsilon_m}{\varepsilon_m + \varepsilon_y}\right) d = 50.3 \text{ in.}$$

$a = 0.8c$ = 40.3 in.

Compression values are determined from the Whitney compression block adjusted for fully grouted cells or nongrouted cells:

C_{m1} = (1.6 ksi)(16 in.)(11.63 in.) = 297.8 kips
C_{m2} = (1.6 ksi)(16 in.)(2 × 1.50 in.) = 76.8 kips
C_{m3} = (1.6 ksi)(8.3 in.)(11.63 in.) = 154.4 kips
C_{s1} = (0.88 in.2)(60 ksi) = 52.8 kips
C_{s2} = (0.88 in.2)(60 ksi)(0.0019 / 0.00207) = 48.5 kips
T_{s2} = (0.88 in.2)(60 ksi) = 52.8 kips
T_{s2} = (0.88 in.2)(60 ksi)(0.0017 / 0/00207) = 43.4 kips

$\Sigma F_y = 0$:
$P_n = \Sigma C - \Sigma T$ = 297.8 + 76.8 + 154.4 + 52.8 + 48.5 -52.8 - 43.4 = 534 kips
ϕP_n = (0.85)(534) = 454 kips

$\Sigma M_{cl} = 0$:
$M_n = 40C_{m1} + 24C_{m2} + 11.85C_{m3} + 44C_{s1} + 36C_{s2} + 44T_{s1} + 36T_{s2}$ = 23,540 in.-kips
ϕM_n = (0.85)(23,540) = 20,009 in. - kips = 1,667 ft-kips

The two cases are plotted in Figure 9.1-21 to develop the ϕP_n - ϕM_n curve on the pier. The demand (P_u , M_u) also is plotted. As can be seen, the pier design is acceptable because the demand is within the ϕP_n - ϕM_n curve. (See the Birmingham 1 example in Sec. 9.2 for additional discussion of ϕP_n - ϕM_n curves.) By linear interpolation, φM_n at the minimum axial load is 968 kip.

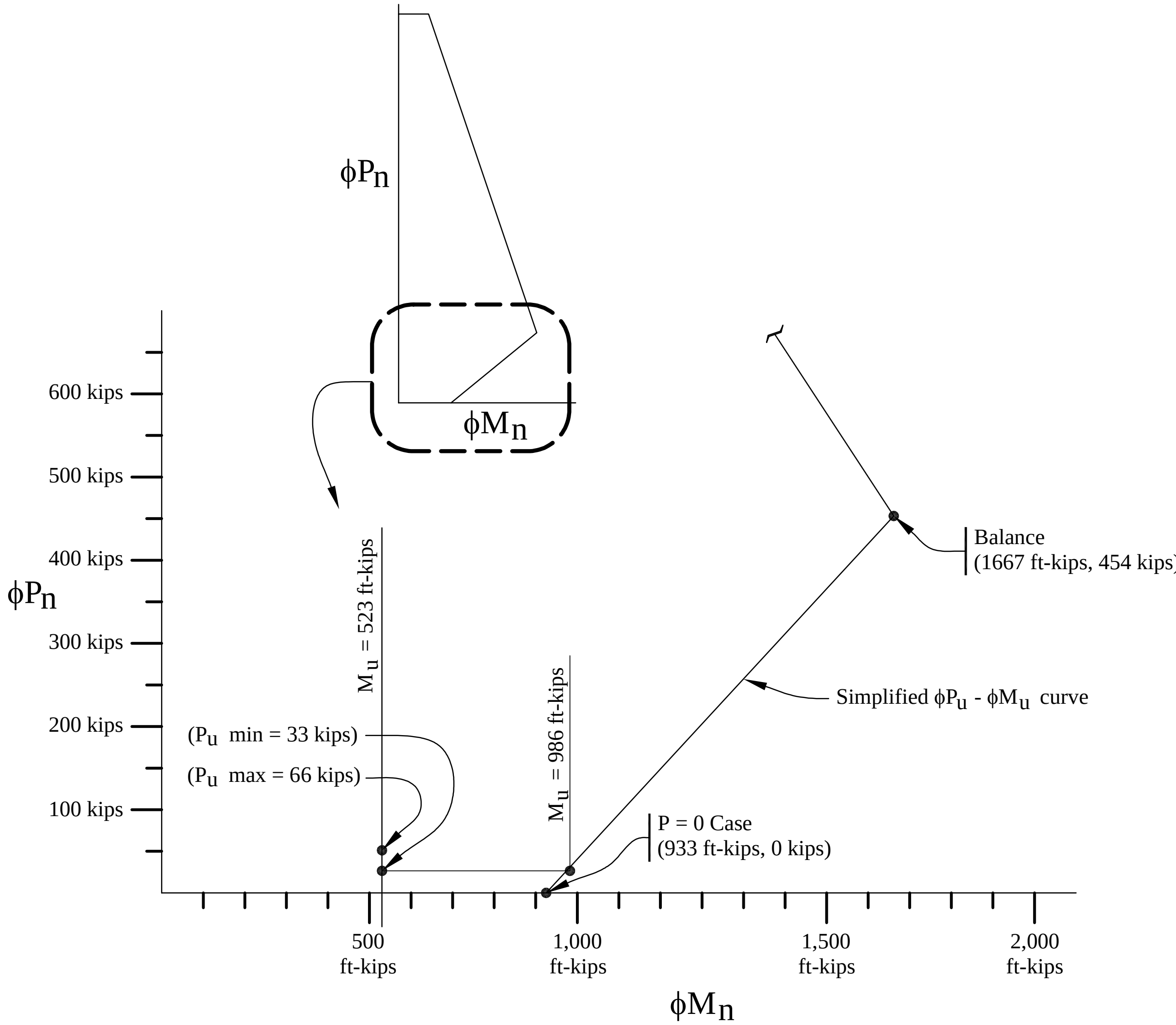


Figure 9.1-21 In-plane φP_{11} - φM_{11} diagram for pier (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.1.6.5 Combined Loads

Combined loads for in-plane and out-of-plane moments in piers at end walls, per *Provisions* Sec. 5.2.5.2.2, Item a, are shown in Table 9.1-4.

Table 9.1-4 Combined Loads for Flexure in End Pier

0.7*D*	Out-of-Plane		In-Plane		Total	
Case 1	1.0(62.12/112.8)	+	0.3(523/986)	=	0.71 < 1.00	OK
Case 2	0.3(62.12/112.8)	+	1.0(523/986)	=	0.70 < 1.00	OK

Values are in kips; 1.0 kip = 4.45 kN.

9.1.6.6 Shear at Transverse Walls (End Walls)

The shear at the base of the pier is 43.6 kips/bay. At the head of the opening where the moment demand is highest, the in-plane shear is slightly less (based on the weight of the pier). There, V = 43.6 kips - 0.286(8 ft)(12 ft)(0.103 ksf) = 40.8 kips. (This refinement in shear is not shown in Figure 9.1-18 although the difference in axial load at the two locations is shown.) The capacity for shear must exceed 2.5 times the demand or the shear associated with 125 percent of the flexural capacity. Using the results in Table 9.1-4, the 125 percent implies a factor on shear by analysis of:

$$1.25\left(\frac{1}{\text{Demand to capacity ratio}}\right)\left(\frac{1}{\phi}\right) = 1.25\left(\frac{1}{0.7}\right)\left(\frac{1}{0.85}\right) = 2.10$$

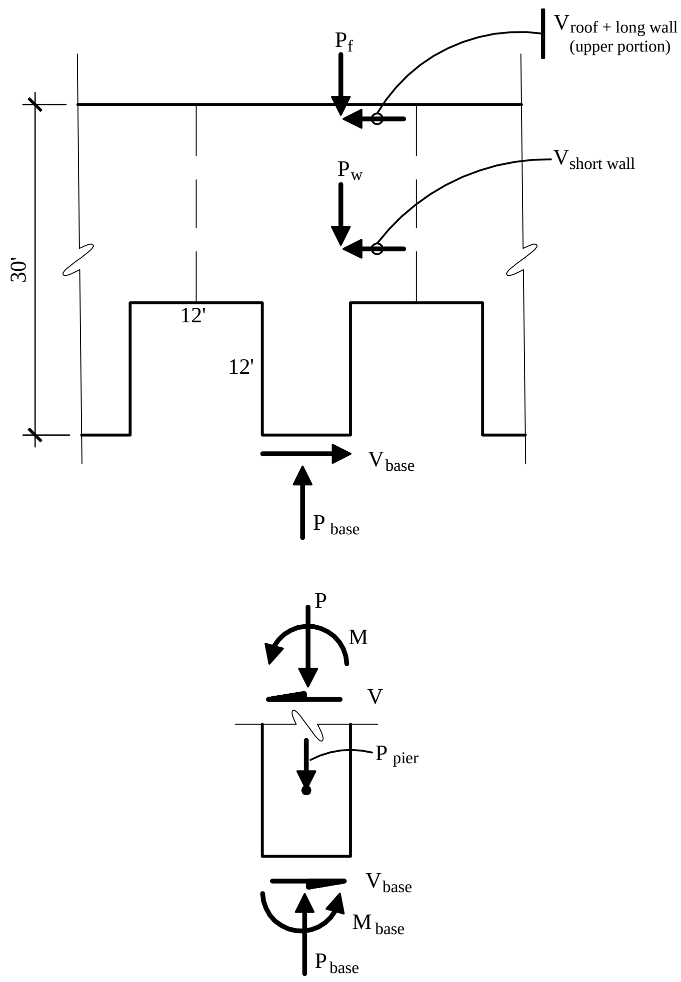


Figure 9.1-22 In-plane shear on end wall and pier (1.0 ft = 0.3048 m).

Therefore, the required shear capacities at the head and base of the pier are 91.6 kips and 85.7 kips, respectively.

The in-plane shear capacity is computed as follows where the net area, A_n, of the pier is the area of face shells plus the area of grouted cells and adjacent webs:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25P$$

A_n = (96 in. × 1.50 in.× 2) + (6 cells × 8 in. × 8.63 in.) = 702 in.^{2}

$$V_S = 0.5\left(\frac{A_V}{s}\right) f_y d_V$$
$$= 0.5\left(\frac{0.62 \text{ in.}^2}{48 \text{ in.}}\right)(60 \text{ ksi})(96 \text{ in.})$$
$$= 37.2 \text{ kips / bay}$$

At the head of the opening:

V_m = [4.0 - 1.75(1.0)](702 in.^{2})(0.0447 ksi) + (0.25)(0.7)(45.1 kips) = 78.5 kips/bay

ϕV_N = (0.8)(78.5 + 37.2) = 92.6 kips/bay

At the base:

V_m = [4.0 - 1.75(0)](702 in.^{2})(0.0447 ksi) + (0.25)(0.7)(55.0 kips) = 135.2 kips/bay

ϕV_N = (0.8)(135.2 + 37.2) = 137.9 kips/bay

As discussed previously, *M/Vd* need not exceed 1.0 in the above equation.

For out-of-plane shear, see Figure 9.1-13. Shear at the top of wall is 12.07 kips/bay and shear at the base of the pier is 6.72 kips/bay. From the values in the figure, the shear at the head of the opening is computed as 6.72 kips - (12 ft)(0.33 kip/ft) = 2.76 kips. The same multiplier of 2.10 for development of 125 percent of flexural capacity will be applied to out-of-plane shear resulting in 25.3 kips at the top of the wall, 5.80 kips at the head of the opening, and 14.11 kips at the base.

Out-of-plane shear capacity is computed using the same equation. $\Sigma b_w d$ is taken as the net area A_n. Note that *M/Vd* is zero at the support because the moment is assumed to be zero; however, a few inches into the span, *M/Vd* will exceed 1.0 so the limiting value of 1.0 is used here. This is typically the case when considering out-of-plane loads on a wall.

For computing shear capacity at the top of the wall:

A_n = $b_w d$ =((8 in./2 ft.) × 20 ft)(9.25 in.) = 740 in.^{2}
V_m = [4.0 - 1.75(1)](740 in.^{2})(0.0447 ksi) + (0.25)(8.0) = 76.9 kips/bay
ϕV_m = (0.8)(76.9) = 61.5 kips/bay

For computing shear capacity in the pier:

A_n =(8 in./cell)(6 cells)(9.25 in.) = 444 in.^{2}
V_m = [4.0 - 1.75(1)](444 in.^{2})(0.0447 ksi) + (0.25)(41.67) = 55.4 kips/bay
ϕV_m = (0.8)(55.4) = 44.3 kips/bay

The combined loads for shear at the end pier (per *Provisions* 5.2.5.2.2, Item a [Sec. 4.4-23]) are shown in

Table 9.1-5.

Table 9.1-5 Combined Loads for Shear in End Wall

	In-Plane		Out-of-Plane		Total
Case 1 Pier base	1.0(91.6/137.9)	+	0.3(14.11/44.3)	=	0.76 < 1.00 OK
Case 2 Pier base	0.3(91.6/137.9)	+	1.0(14.11/44.3)	=	0.52< 1.00 OK
Case 1 Pier head	1.0(85.7/92.6)	+	0.3(5.80/44.3)	=	0.96 > 1.00 OK
Case 2 Pier head	0.3(85.7/92.6)	+	1.0(5.80/44.3)	=	0.41 < 1.00 OK

Values are in kips; 1.0 kip = 4.45 kN.

9.1.7 Bond Beam

Reinforcement for the bond beam located at the elevation of the roof diaphragm can be used for the diaphragm chord. The uniform lateral load for the design of the chord is the lateral load from the long wall plus the lateral load from the roof and is equal to 0.87 klf. The maximum tension in rebar is equal the maximum moment divided by the diaphragm depth:

M/d = 4,350 ft-kips/100 ft = 43.5 kips

The seismic load factor is 1.0. The required reinforcement is:

$A_{reqd} = T/\phi F_y$ = 43.5/(0.85)(60) = 0.85 in.^{2}

This will be satisfied by two #6 bars, A_s = (2 × 0.44 in.^{2}) = 0.88 in.^{2}

In Sec. 10.2, the diaphragm chord is designed as a wood member utilizing the wood ledger member. Using either the wood ledger or the bond beam is considered acceptable.

9.1.8 In-Plane Deflection

Deflection of the end wall (short wall) has two components as illustrated in Figure 9.1-23.

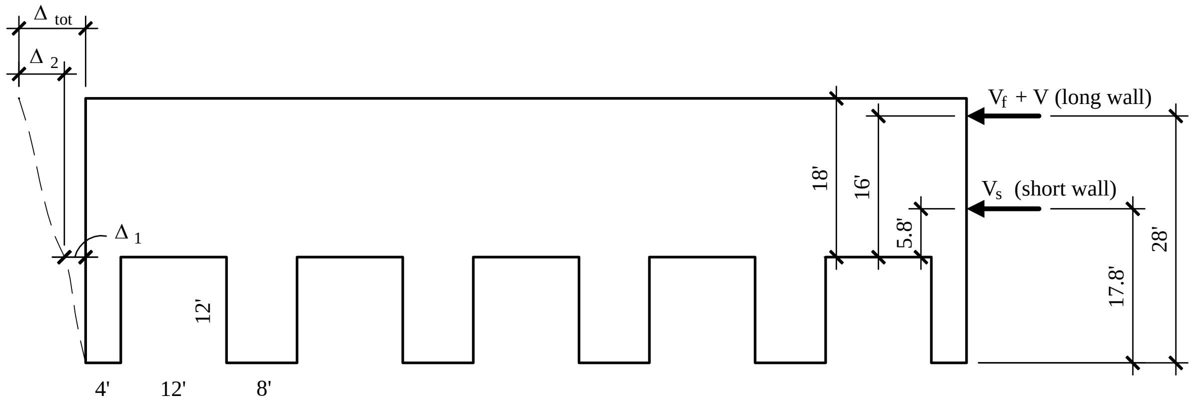


Figure 9.1-23 In-plane deflection of end wall (1.0 ft = 0.3048 m).

As obtained from the RISA 2D analysis of the piers, Δ_1 = 0.047 in.:

$$\Delta_2 = \sum \frac{\alpha VL}{AG}$$

where α is the form factor equal to 6/5 and

$G = E_m/2(1 + \mu)$ = 1500 ksi / 2(1 + 0.15) = 652 ksi
$A = A_n$ = Area of face shells + area of grouted cells
= (100 ft × 12 in./ft × 2 × 1.50 in.^{2}) +(50)(8 in.)(8.63 in.) = 7,050 in.^{2}

Therefore:

$$\Delta_2 = \left(\frac{6}{5}\right)\frac{(67.15)(5.8\times 12)}{(7{,}050)(652)} + \left(\frac{6}{5}\right)\frac{(116.9)(16\times 12)}{(7{,}050)(652)} = 0.0013 + 0.0059 = 0.007 \text{ in.}$$

and,

$\Delta_{total} = C_d(0.047 + 0.007) = 3.5(0.054 \text{ in.}) = 0.19 \text{ in.} < 3.36 \text{ in.}$
(3.36 = $0.01h_n = 0.01h_{sx}$) (*Provisions* Sec. 11.5.4)

Note that the drift limits for masonry structures are smaller than for other types of structure. It is possible to interpret *Provisions* Table 5.2.8 [Table 4.5-1] to give a limit of $0.007h_n$ for this structure but that limit also is easily satisfied. The real displacement in this structure is in the roof diaphragm; see Sec. 10.2.

9.2 FIVE-STORY MASONRY RESIDENTIAL BUILDINGS IN BIRMINGHAM, ALABAMA; NEW YORK, NEW YORK; AND LOS ANGELES, CALIFORNIA

9.2.1 Building Description

In plan, this five-story residential building has bearing walls at 24 ft on center (see Figures 9.2-1 and 9.2-2). All structural walls are of 8-in.-thick concrete masonry units (CMU). The floor is of 8-in.-thick hollow core precast, prestressed concrete planks. To demonstrate the incremental seismic requirements for masonry structures, the building is partially designed for four locations: two sites in Birmingham, Alabama; a site in New York, New York; and a site in Los Angeles, California. The two sites in Birmingham have been selected to illustrate the influence of different soil profiles at the same location. The building is designed for Site Classes C and E in Birmingham. The building falls in Seismic Design Categories B and D in these locations, respectively. For Site Class D soils, the building falls in Seismic Design Categories C and D for New York and Los Angeles, respectively.

[Note that the method for assigning seismic design category for short period buildings has been revised in the 2003 *Provisions*. If the fundamental period, T_a, is less than $0.8T_s$, the period used to determine drift is less than T_s, and the base shear is computed using 2003 *Provisions* Eq 5.2-2, then seismic design category is assigned using just 2003 *Provisions* Table 1.4-1 (rather than the greater of 2003 *Provisions* Tables 1.4-1 and 1.4-2). This change results in the Birmingham Site Class E building being assigned to Seismic Design Category C instead of D. The changes to this example based on the revised seismic design category are not noted in the remainder of the example. The New York building provides an example of what the Seismic Design Category C requirements would be for the Birmingham Site Class E building.]

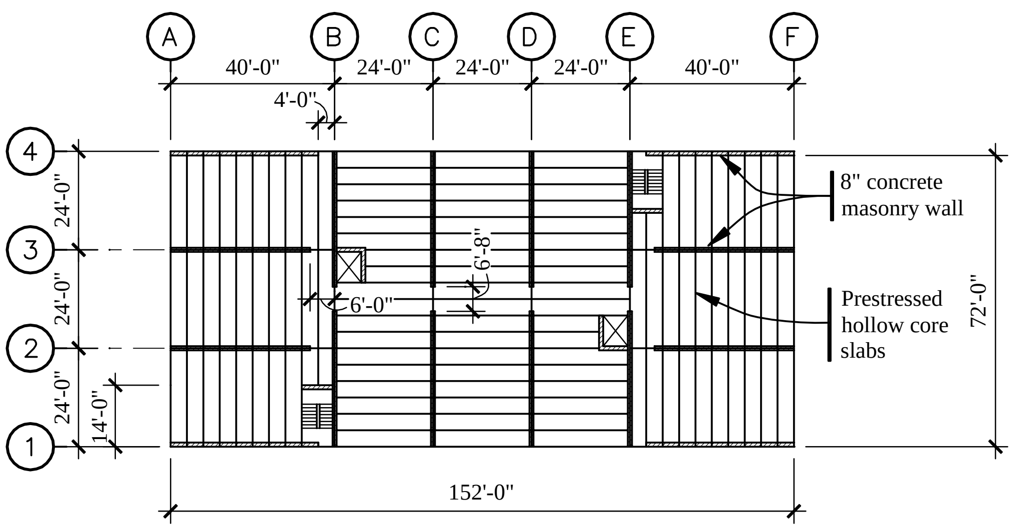


Figure 9.2-1 Typical floor plan (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

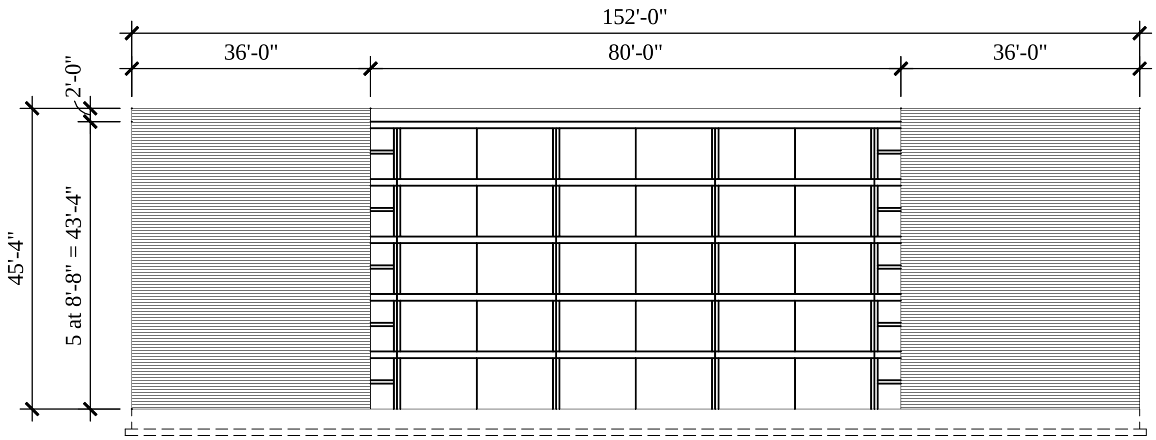


Figure 9.2-2 Building elevation (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

For the New York and both Birmingham sites, it is assumed that shear friction reinforcement in the joints of the diaphragm planks is sufficient to resist seismic forces, so no topping is used. For the Los Angeles site, a cast-in-place 2 ½-in.-thick reinforced lightweight concrete topping is applied to all floors. The structure is free of irregularities both in plan and elevation. The *Provisions*, by reference to ACI 318, requires reinforced cast-in-place toppings as diaphragms in Seismic Design Category D and higher. Thus, the Birmingham example in Site Class E would require a topping, although that is not included in this example.

Provisions Chapter 9 has an appendix (intended for trial use and feedback) for the design of untopped precast units as diaphragms. The design of an untopped diaphragm for Seismic Design Categories A, B, and C is not explicitly addressed in ACI 318. The designs of both untopped and topped diaphragms for these buildings are described in Chapter 7 of this volume using ACI 318 for the topped diaphragm in the Los Angeles building and using the appendix to *Provisions* Chapter 9 for untopped diaphragms in the New York building. It is assumed here that the diaphragm for the Birmingham 2 example would be similar to the New York example, and the extra weight of the Birmingham 2 topping is not included in the illustration here.

No foundations are designed in this example. However, for the purpose of determining the site class coefficient (*Provisions* Sec. 4.1.2.1 [Sec. 3.5]), a stiff soil profile with standard penetration test results of $15 < N < 50$ is assumed for Los Angeles and New York sites resulting in a Site Class D for these two locations. For Birmingham, however, one site has soft rock with $N > 50$ and the other has soft clay with $N < 15$, which results in Site Classes C and E, respectively. The foundation systems are assumed to be able to carry the superstructure loads including the overturning moments.

The masonry walls in two perpendicular directions act as bearing and shear walls with different levels of axial loads. The geometry of the building in plan and elevation results in nearly equal lateral resistance in both directions. The walls are constructed of CMU and are typically minimally reinforced in all locations. The walls are assumed to act as columns in their planes. Figure 9.2-3 illustrates the wall layout.

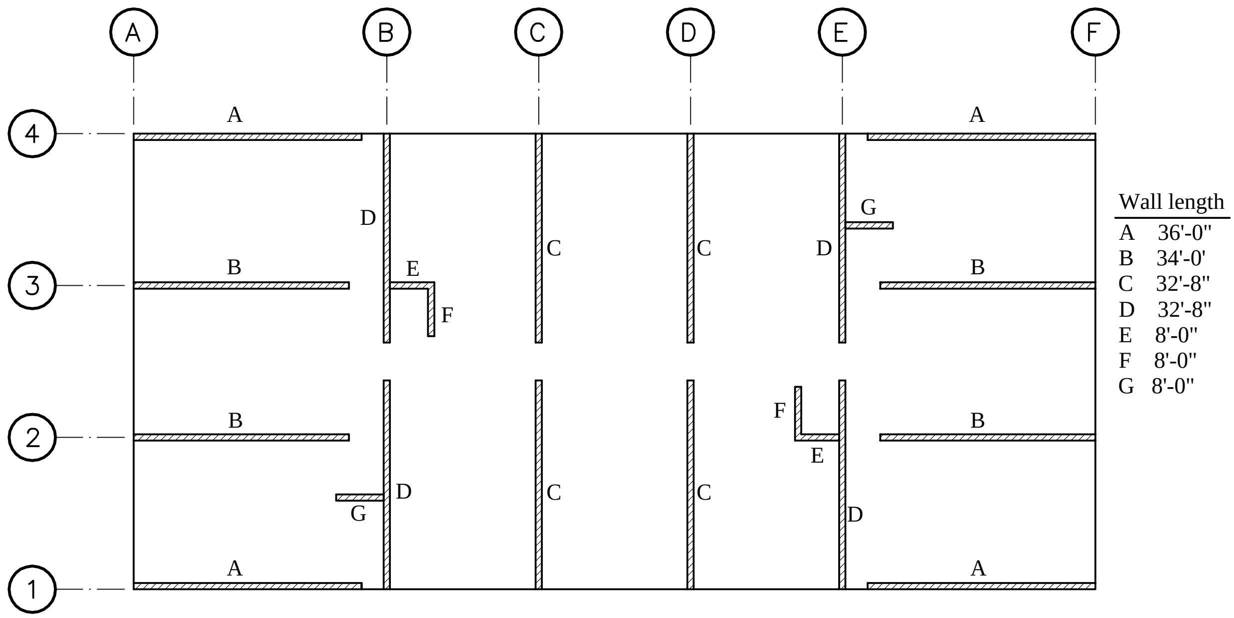


Figure 9.2-3 Plan of walls (1.0 ft = 0.3048 m).

The floors serve as horizontal diaphragms distributing the seismic forces to the walls and are assumed to be stiff enough to be considered rigid. There is little information about the stiffness of untopped precast diaphragms. The design procedure in the appendix to *Provisions* Chapter 9 results in a diaphragm intended to remain below the elastic limit until the walls reach an upper bound estimate of strength, therefore it appears that the assumption is reasonable.

Material properties are as follows:

The compressive strength of masonry, f'_m, is taken as 2,000 psi and the steel reinforcement has a yield limit of 60 ksi.

The design snow load (on an exposed flat roof) is taken as 20 psf for New York; design for snow does not control the roof design in the other locations.

This example covers the following aspects of a seismic design:

1. Determining the equivalent lateral forces,
2. Design of selected masonry shear walls for their in-plane loads, and
3. Computation of drifts.

See Chapter 7 of this volume for the design and detailing of untopped and topped precast diaphragms.

9.2.2 Design Requirements

9.2.2.1 *Provisions* Parameters

The basic parameters affecting the design and detailing of the buildings are shown in Table 9.2-1.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

9.2.2.2 Structural Design Considerations

The floors act as horizontal diaphragms and the walls parallel to the motion act as shear walls for all four buildings

The system is categorized as a bearing wall system (*Provisions* Sec. 5.2.2[Sec. 4.3]). For Seismic Design Category D, the bearing wall system has a height limit of 160 ft and must comply with the requirements for special reinforced masonry shear walls (*Provisions* Sec. 11.11.5[Sec. 11.2.1.5]). Note that the structural system is one of uncoupled shear walls. Crossing beams over the interior doorways (their design is not included in this example) will need to continue to support the gravity loads from the deck slabs above during the earthquake, but are not designed to provide coupling between the shear walls.

The building is symmetric and appears to be regular both in plan and elevation. It will be shown, however, that the building is actually torsionally irregular. *Provisions* Table 5.2.5 [Table 4.4-1] permits use of the equivalent lateral force (ELF) procedure in accordance with *Provisions* Sec. 5.4 [Sec. 5.2] for Birmingham 1 and New York City (Seismic Design Categories B and C). By the same table, the Category D buildings must use a dynamic analysis for design. For this particular building arrangement, the modal response spectrum analysis does not identify any particular effect of the torsional irregularity, as will be illustrated.

Table 9.2-1 Design Parameters

Design Parameter	Value for Birmingham 1	Value for Birmingham 2	Value for New York	Value for Los Angeles
S_s (Map 1) [Figure 3.3-1]	0.3	0.3	0.4	1.5
S_1 (Map 2) [Figure 3.3-2]	0.12	0.12	0.09	0.6
Site Class	C	E	D	D
F_a	1.2	2.34	1.48	1
F_v	1.68	3.44	2.4	1.5
$S_{MS} = F_aS_s$	0.36	0.7	0.59	1.5
$S_{M1} = F_vS_1$	0.2	0.41	0.22	0.9
S_{DS} = 2/3 S_{MS}	0.24	0.47	0.39	1
S_{D1} = 2/3 S_{M1}	0.13	0.28	0.14	0.6
Seismic Design Category	B	D	C	D
Masonry Wall Type	Ordinary Reinforced	Special Reinforced	Intermediate Reinforced	Special Reinforced
Provisions Design Coefficients (Table 5.2.2 [4.3-1])				
R	2.0	3.5	2.5	3.5

Design Parameter	Value for Birmingham 1	Value for Birmingham 2	Value for New York	Value for Los Angeles
Ω_0	2.5	2.5	2.5	2.5
C_d	1.75	3.5	2.25	3.5
IBC Design Coefficients (presented for comparison with *Provisions* coefficients)				
R	2.5	5.0	3.5	5.0
Ω_0	2.5	2.5	2.5	2.5
C_d	1.75	3.5	2.25	3.5

The orthogonal effect (*Provisions* Sec. 5.2.5.2, Item a [Sec. 4.4.2]) applies to structures assigned to Seismic Design Categories C and D (all of the example buildings except for Birmingham 1). However, the arrangement of this building is not particularly susceptible to orthogonal effects. This is because the stresses developed under out-of-plane loading for short-height walls (story clear height is 8 ft) are low and, their contribution to orthogonal effects is minimal.

The walls are all solid and there are no significant discontinuities, as defined by *Provisions* Sec. 5.2.6.2.3 [Sec. 4.3.2.3], in the vertical elements of the seismic-force-resisting system.

Ignoring the short walls at stairs and elevators, there are eight shear walls in each direction, therefore, the system appears to have adequate redundancy (*Provisions* Sec. 5.2.6.2.4 [Sec. 4.3.3]). The reliability factor, however, will be computed. [See Sec. 9.2.3.1 for changes to the reliability factor.]

Tie and continuity requirements (*Provisions* Sec. 5.2.6.1.2 [Sec. 4.6]) must be addressed when detailing connections between floors and walls (see Chapter 7 of this volume).

Nonstructural elements (*Provisions* Chapter 14 [Chapter 6]) are not considered in this example.

Collector elements are required in the diaphragm for longitudinal response (*Provisions* Sec. 5.2.6.2.5 [Sec. 4.6]). Rebar in the longitudinal direction, spliced into bond beams, will be used for this purpose (see Chapter 7 of this volume).

Diaphragms must be designed for the required forces (*Provisions* Sec. 5.2.6.2.6 [Sec. 4.6]).

The bearing walls must be designed for the required force perpendicular to their plane (*Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3]).

Each wall is a vertical cantilever; there are no coupling beams. The walls are classified as masonry cantilever shear wall structures in *Provisions* Table 5.2.8 [Table 4.5-1], which limits interstory drift to 0.01 times the story height. *Provisions* Sec.11.5.4.1.1 also limits drift to 0.01 times the wall height for such a structure.

[The deflection limits have been removed from Chapter 11 of the 2003 *Provisions* because they were redundant with the general deflection limits. Based on ACI 530 Sec. 1.13.3.2, the maximum drift for all masonry structures is 0.007 times the story height. Thus, there appears to be a conflict between ACI 530 and 2003 *Provisions* Table 4.5-1.]

Vertical accelerations must be considered for the prestressed slabs in Seismic Design Category D (*Provisions* Sec. 5.2.6.4.3 [Sec. 4.6.3.1]); refer to Chapter 7 of this volume. The evaluation of such components involves the earthquake effect determined using *Provisions* Eq. 5.2.7-1 [4.2-1] and 5.2.7-2 [4.2-1]. The important load is the vertical effect ($-0.2S_{DS}D$), which reduces the effect of dead loads. Because the system is prestressed, application of this load might lead to tension where there would otherwise be no reinforcement. The reinforcement within the topping will control this effect. Refer to Sec. 7.1 of this volume for the design of precast, prestressed slabs and topping.

Design, detailing, and structural component effects are presented in the chapters of the *Provisions* that are relevant to the materials used.

9.2.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [Sec. 4.2.2]) are the same as those in ASCE 7 (and are similar to those in the IBC). The seismic load effect, E, is defined by *Provisions* Eq. 5.2.7-1 [4.2-1] and 5.2.7-2 [4.2-2] as:

$$E = \rho Q_E \pm 0.2S_{DS}D$$

9.2.3.1 Reliability Factor

Note that ρ is a multiplier on design force effects and applies only to the in-plane direction of the shear walls. For structures in Seismic Design Categories A, B and C, $\rho = 1.0$ (*Provisions* Sec. 5.2.4.1 [Sec. 4.3.3.1]). For structures in Seismic Design Category D, ρ is determined per *Provisions* Sec. 5.2.4.2 [Sec. 4.3.3.2].

For the transverse direction, ignoring accidental torsion:

$$r_{max_x} = \left(\frac{V_{wall}}{V_{story}}\right)\left(\frac{10}{l_w}\right) \cong \left(\frac{1}{8}\right)\left(\frac{10}{33}\right) = 0.038$$

and,

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}} = 2 - \frac{20}{0.038\sqrt{10,944}} = -3.03$$

Since the computed $\rho < 1.0$ use $\rho = 1.0$ for the transverse direction. Accidental torsion does not change r_{max_x} enough to change this conclusion.

Based on similar calculations for the longitudinal direction, ρ is determined to be 1.0.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For structures assigned to Seismic Design Categories B and C, $\rho = 1.0$ in all cases. For a shear wall building assigned to Seismic Design Category D, $\rho = 1.0$ as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. The intent is that the aspect ratio is based on story height, not total height. Therefore, the redundancy factor would not have to be investigated ($\rho = 1.0$) for the structure(s) assigned to Seismic Design Category D.]

9.2.3.2 Combination of Load Effects

The seismic load effect, E, determined for each of the buildings is:

Birmingham 1	$E = (1.0)Q_E \pm (0.2)(0.24)D = Q_E \pm 0.05D$
Birmingham 2	$E = (1.0)Q_E \pm (0.2)(0.47)D = Q_E \pm 0.09D$
New York	$E = (1.0)Q_E \pm (0.2)(0.39)D = Q_E \pm 0.08D$
Los Angeles	$E = (1.0)Q_E \pm (0.2)(1.00)D = Q_E \pm 0.20D$

The applicable load combinations from ASCE 7 are:

$$1.2D + 1.0E + 0.5L + 0.2S$$

when the effects of gravity and seismic loads are additive and

$$0.9D + 1.0E + 1.6H$$

when the effects of gravity and seismic loads are counteractive. (H is the effect of lateral pressures of soil and water in soil.)

Load effect H does not apply for this design, and the snow load effect, S, exceeds the minimum roof live load only at the building in New York. However, even for New York, the snow load effect is only used for combinations of gravity loading. Consideration of snow loads is not required in the effective seismic weight, W, of the structure when the design snow load does not exceed 30 psf (*Provisions* Sec. 5.3 [Sec. 5.2.1]).

The basic load combinations are combined with E as determined above, and the load combinations representing the extreme cases are:

Birmingham 1	$1.25D + Q_E + 0.5L$ $0.85D - Q_E$
Birmingham 2	$1.29D + Q_E + 0.5L$ $0.81D - Q_E$
New York	$1.28D + Q_E + 0.5L + 0.2S$ $0.82D - Q_E$
Los Angeles	$1.40D + Q_E + 0.5L$ $0.70D - Q_E$

These combinations are for the in-plane direction. Load combinations for the out-of-plane direction are similar except that the reliability coefficient (1.0 in all cases for in-plane loading) is not applicable.

It is worth noting that there is an inconsistency in the treatment of snow loads combined with seismic loads. IBC Sec. 1605.3 clearly deletes the snow term from the ASD combinations where the design snow load does not exceed 30 psf. There is no similar provision for the strength load combinations in the IBC for reference standard, ASCE 7.

[The strength design load combinations in the 2003 IBC do have a similar exemption for snow loads, but ASCE 7-02 load combinations do not.]

9.2.4 Seismic Design for Birmingham 1

9.2.4.1 Birmingham 1 Weights

Use 67 psf for 8-in.-thick, normal weight hollow core plank plus the nonmasonry partitions. This site is assigned to Seismic Design Category B, and the walls will be designed as ordinary reinforced masonry shear walls (*Provisions* Sec. 11.11.3 [Sec. 4.2.1.3]), which do not require prescriptive seismic reinforcement. However, both ACI 530 and IBC 2106.1.1.2 stipulate that ordinary reinforced masonry shear walls have a minimum of vertical #4 bars at 120 in. on center. [By reference to ACI 530, the 2003 *Provisions* (and 2003 IBC) do have prescriptive seismic reinforcement requirements for ordinary reinforced masonry shear walls. Refer to ACI 530 Sec. 1.13.2.2.3.] Given the length of the walls, vertical reinforcement of #4 bars at 8 ft on center works well for detailing reasons and will be used here. For this example, 45 psf will be assumed for the 8-in.-thick lightweight CMU walls. The 45 psf value includes grouted cells and bond beams in the course just below the floor planks.

Story weight, w_i, is computed as follows:

For the roof:

Roof slab (plus roofing) = (67 psf) (152 ft)(72 ft) = 733 kips
Walls = (45 psf)(589 ft)(8.67 ft/2) + (45 psf)(4)(36 ft)(2 ft) = <u>128 kips</u>
Total = 861 kips

Note that there is a 2-ft-high masonry parapet on four walls and the total length of masonry wall, including the short walls, is 589 ft.

For a typical floor:

Slab (plus partitions) = 733 kips
Walls = (45 psf)(589 ft)(8.67 ft) = <u>230 kips</u>
Total = 963 kips

Total effective seismic weight, W = 861 + (4)(963) = 4,713 kips

This total excludes the lower half of the first story walls, which do not contribute to seismic loads that are imposed on CMU shear walls.

9.2.4.2 Birmingham 1 Base Shear Calculation

The seismic response coefficient, C_s, is computed using *Provisions* Sec. 5.4.1.1 [Sec. 5.2.1.1].
Per *Provisions* Eq. 5.4.1.1-1 [Eq. 5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.24}{2/1} = 0.12$$

The value of C_s need not be greater than *Provisions* Eq. 5.4.1.1-2 [Eq. 5.2-3]:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.13}{0.338(2/1)} = 0.192$$

T is the fundamental period of the building approximated per *Provisions* Eq. 5.4.2.1-1[Eq. 5.2-6] as:

$$T_a = C_r h_n^x = (0.02)(43.33^{0.75}) = 0.338 \text{ sec}$$

where C_r = 0.02 and x = 0.75 are from *Provisions* Table 5.4.2.1 [Table 5.2-2].

The value for C_s is taken as 0.12 (the lesser of the two computed values). This value is still larger than the minimum specified in *Provisions* Eq. 5.4.1.1-3:

$$C_s = 0.044IS_{DS} = (0.044)(1.0)(0.24) = 0\ 0.0106$$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4.1 [Eq. 5.2-1]as:

$$V = C_sW = (0.12)(4{,}713) = 566 \text{ kips}$$

9.2.4.3 Birmingham 1 Vertical Distribution of Seismic Forces

Provisions Sec. 5.4.4 [Sec. 5.2.3] stipulates the procedure for determining the portion of the total seismic load assigned to each floor level. The story force, F_x, is calculated using *Provisions* Eq. 5.4.3-1 [Eq. 5.2-10] and 5.4.3.-2 [Eq. 5.2-11], respectively, as:

$$F_x = C_{vx}V$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

For T = 0.338 sec < 0.5 sec, k = 1.0.

The seismic design shear in any story is determined from *Provisions* Eq. 5.4.4[Eq. 5.2-12]:

$$V_x = \sum_{i=x}^{n} F_i$$

The story overturning moment is computed from *Provisions* Eq. 5.4.5[Eq. 5.2-14]:

$$M_x = \sum_{i=x}^{n} F_i(h_i - h_x)$$

The application of these equations for this building is shown in Table 9.2-2.

Table 9.2-2 Birmingham 1 Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	861	43.34	37,310	0.3089	175	1.8e+14	1,515
4	963	34.67	33,384	0.2764	156		4,385
3	963	26.00	25,038	0.2073	117		8,272
2	963	17.33	16,692	0.1382	78		12,836
1	963	8.67	8,346	0.0691	39		17,739
∑	4,715		120,770	1.0000	566		

1.0 kips = 4.45 kN, 1.0 ft = 0.3048 m.

A note regarding locations of V and M: the vertical weight at the roof (5^{th} level), which includes the upper half of the wall above the 5^{th} floor (4^{th} level), produces the shear V applied at the 5^{th} level. That shear in turn produces the moment applied at the top of the 4^{th} level. Resisting this moment is the rebar in the wall combined with the wall weight above the 4^{th} level. Note that the story overturning moment is applied to the level below the level thatreceives the story shear. This is illustrated in Figure 9.2-4.

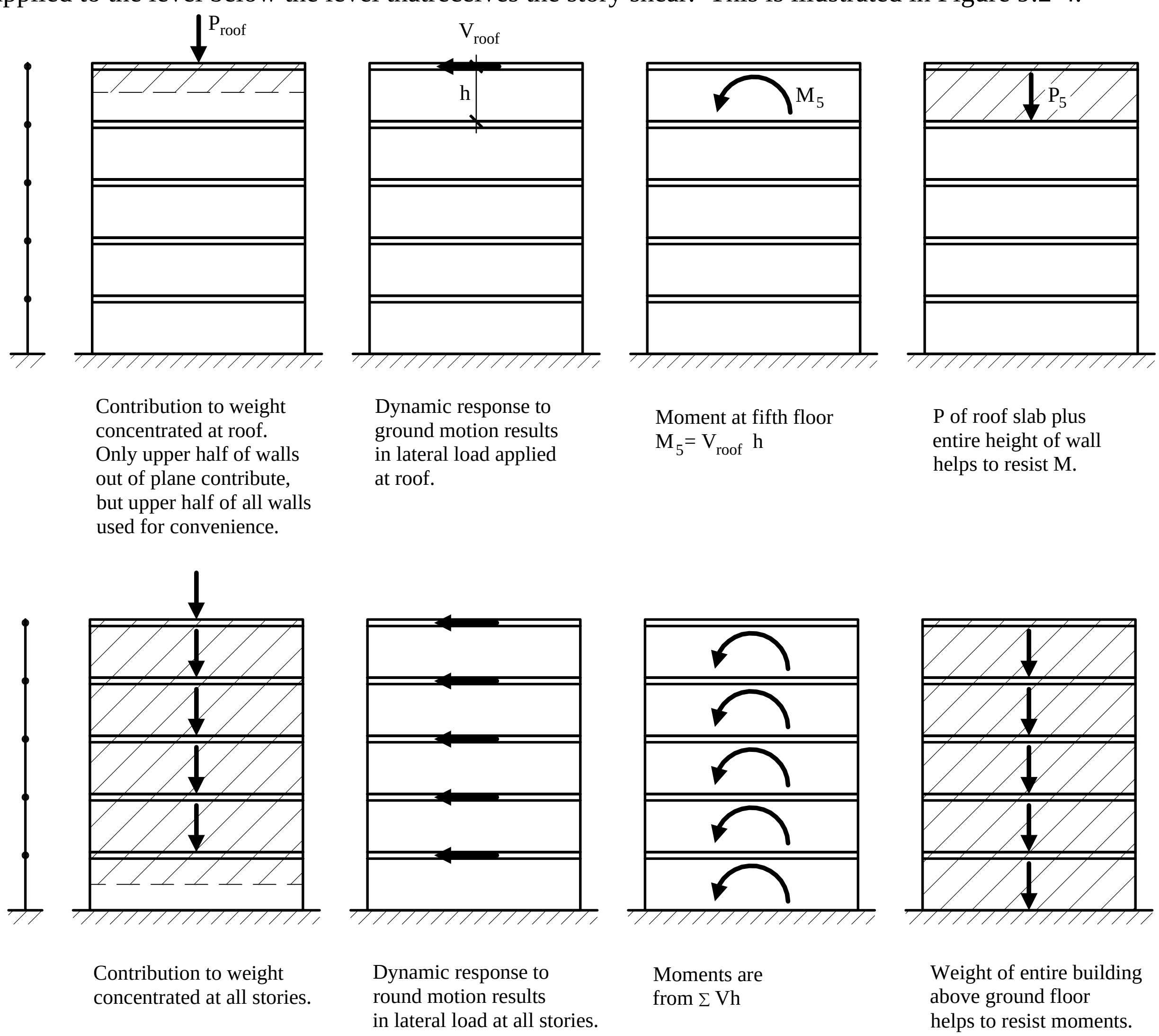


Figure 9.2-4 Location of moments due to story shears.

9.2.4.4 Birmingham 1 Horizontal Distribution of Forces

The wall lengths are shown in Figure 9.2-3. The initial grouting pattern is basically the same for walls A, B, and C. Because of a low relative stiffness, the effects Walls D, E, and F are ignored in this analysis. Walls A, B, and C are so nearly the same length that their stiffnesses will be assumed to be the same for this example.

Torsion is considered according to *Provisions* Sec. 5.4.4[Sec. 5.2.4]. For a symmetric plan, as in this example, the only torsion to be considered is the accidental torsion, M_{ta}, caused by an assumed eccentricity of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied loads.

Dynamic amplification of the torsion need not be considered for Seismic Design Category B per *Provisions* Sec. 5.4.4.3 [Sec. 5.2.4.3].

For this example, the building will be analyzed in the transverse direction only. The evaluation of Wall D is selected for this example. The rigid diaphragm distributes the lateral forces into walls in both directions. Two components of force must be considered: direct shear and shear induced by torsion.

The direct shear force carried by Wall D is one-eighth of the total story shear (eight equal walls). The torsional moment per *Provisions* Sec. 5.4.4.2 [Sec. 5.2.4.2] is:

$$M_{ta} = 0.05bV_x = (0.05)(152 \text{ ft})V_x = 7.6V_x$$

The torsional force per wall, V_t, is:

$$V_t = \frac{M_t Kd}{\sum Kd^2}$$

where K is the stiffness (rigidity) of each wall.

Because all the walls in this example are assumed to be equally stiff:

$$V_t = M_t \left[\frac{d}{\sum d^2} \right]$$

where d is the distance from each wall to the center of twisting.

$$\sum d^2 = 4(36)^2 + 4(12)^2 + 4(36)^2 + 4(12)^2 = 11{,}520$$

The maximum torsional shear force in Wall D, therefore is:

$$V_t = 7.6\ V(36/11{,}520) = 0.0238V$$

Total shear in Wall D is:

$$V_{tot} = 0.125V + 0.0238V = 0.149V$$

The total story shear and overturning moment may now be distributed to Wall D and the wall proportions checked. The wall capacity will be checked before considering deflections.

9.2.4.5 Birmingham 1 Transverse Wall (Wall D)

The strength or limit state design concept is used in the *Provisions*. This method was introduced in the 2002 edition of ACI 530, the basic reference standard for masonry design. Because strength design was not in prior editions of ACI 530, strength design of masonry as defined in the *Provisions* is illustrated here.

[The 2003 *Provisions* adopts by reference the ACI 530-02 provisions for strength design in masonry, and the previous strength design section has been removed. This adoption does not result in significant technical changes, and the references to the corresponding sections in ACI 530 are noted in the following sections.]

9.2.4.5.1 Birmingham 1 Shear Strength

Provisions Sec. 11.7.2 [ACI 530, Sec. 3.1.3] states that the ultimate shear loads must be compared to the design shear strength per *Provisions* Eq. 11.7.2.1:

$$V_u \leq \phi V_n$$

The strength reduction factor, ϕ, is 0.8 (*Provisions* Table 11.5.3, ACI 530 [See 3.1.4.3]). The design shear strength, ϕV_n, must exceed the shear corresponding to the development of 1.25 times the nominal flexural strength of the member but need not exceed 2.5 times V_u (*Provisions* Sec. 11.7.2.2 [ACI 530, Sec. 3.1.3]). The nominal shear strength, V_n, is (*Provisions* Eq. 11.7.3.1-1 [ACI 530, Eq. 3-18]):

$$V_n = V_m + V_s$$

The shear strength provided by masonry is (*Provisions* Eq. 11.7.3.2 [ACI 530, Eq. 3-21]):

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25\,P$$

For grouted cells at 8 ft on center:

$$A_n = (2 \times 1.25 \text{ in.} \times 32.67 \text{ ft x } 12 \text{ in.}) + (8 \times 5.13 \text{ in.}^2 \times 5 \text{ cells}) = 1{,}185 \text{ in.}^2$$

The shear strength provided by reinforcement is given by *Provisions* Eq. 11.7.3.3 [ACI 530, Sec. 3.2.4.1.2.2] as:

$$V_s = 0.5\left(\frac{A_v}{s}\right) F_y d_v$$

The wall will have a bond beam with two #4 bars at each story to bear the precast floor planks and wire joint reinforcement at alternating courses. Common joint reinforcement with 9 gauge wires at each face shell will be used; each wire has a cross-sectional area of 0.017 in.^{2} With six courses of joint reinforcement and two #4 bars, the total area per story is 0.60 in.^{2} or 0.07 in.^{2}/ft.

$$V_s = 0.5(0.07 \text{ in.2/ft.})(60 \text{ ksi})(32.67 \text{ ft.}) = 68.3 \text{ kips}$$

The maximum nominal shear strength of the member (Wall D in this case) for $M/Vd_v > 1.00$ (the *Provisions* has a typographical error for the inequality sign) is given by *Provisions* Eq. 11.7.3.1-3 [ACI 530, Eq. 3-22]:

$$V_n(\max) = 4\sqrt{f'_m}A_n$$

The coefficient 4 becomes 6 for $M/Vd_v < 0.25$. Interpolation between yields the following:

$$V_N\ (\max) = (6.67 - 2.67\left(\frac{M}{Vd_V}\right))\sqrt{f'_m}A_n$$

The shear strength of Wall D, based on the equations listed above, is summarized in Table 9.2-3. Note that V_x and M_x in this table are values from Table 9.2-2 multiplied by 0.149 (which represents the portion of direct and torsional shear assigned to Wall D). P is the dead load of the roof or floor times the tributary area for Wall D. (Note that there is a small load from the floor plank parallel to the wall.)

Table 9.2-3 Shear Strength Calculations for Birmingham 1 Wall D

Story	V_x (kips)	M_x (ft-kips)	M_x/V_xd	2.5 V_x (kips)	P (kips)	φV_m (kips)	φV_s (kips)	φV_n (kips)	φV_n *max* (kips)
5	26	225	0.265	65.0	41	158.1	54.6	212.7	252.7
4	49.3	652	0.405	123.3	89	157.3	54.6	211.9	236.9
3	66.7	1230	0.564	166.8	137	155.1	54.6	209.7	218.8
2	78.4	1910	0.746	196.0	184	151.0	54.6	205.6	198.3
1	84.2	2640	0.960	210.5	232	144.8	54.6	199.4	174.1

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

V_U exceeds both φV_n and φV_n *max* at the first story. It would be feasible to add grouted cells in the first story to remedy the deficiency. However, it will be shown following the flexural design that the shear to develop 1.25 times the flexural capacity is 1.94(84.2 kips) = 163 kips, which is OK.

9.2.4.5.2 Birmingham 1 Axial and Flexural Strength

All the walls in this example are bearing shear walls since they support vertical loads as well as lateral forces. In-plane calculations include:

1. Strength check and
2. Ductility check

9.2.4.5.2.1 Strength check

The wall demands, using the load combinations determined previously, are presented in Table 9.2-4 for Wall D. In the table, Load Combination 1 is 1.25D + Q_E + 0.5L and Load Combination 2 is 0.85D + Q_E.

Table 9.2-4 Demands for Birmingham 1 Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
54321	4.2e+12	8172534	5.111518e+13	2.2565e+17	3.576116e+12	2.256521e+17

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

Strength at the bottom story (where P, V, and M are the greatest) will be examined. (For a real design, all levels should be examined). The strength design will consider Load Combination 2 from Table 9.2-4 to be the governing case because it has the same lateral load as Load Combination 1 but with lower values of axial force.

For the base of the shear walls:

$P_{u_{min}}$ = 197 kips plus factored weight of lower ½ of 1st story wall = 197 + (0.85)(6.4) = 202 kips

$P_{u_{max}}$ = 307 + (1.25)(6.4) = 315 kips

M_u = 2,640 ft-kips

Try one #4 bars in each end cell and a #4 bar at 8 ft on center for the interior cells. A ϕP_n - ϕM_n curve, representing the wall strength envelope, will be developed and used to evaluate P_u and M_u determined above. Three cases will be analyzed and their results will be used in plotting the ϕP_n - ϕM_n curve.

In accordance with *Provisions* Sec. 11.6.2.1 [ACI 530, Sec. 3.2.2], the strength of the section is reached as the compressive strains in masonry reach their maximum usable value of 0.0025 for CMU. The force equilibrium in the section is attained by assuming an equivalent rectangular stress block of $0.8f'_m$ over an effective depth of $0.8c$, where c is the distance of the neutral axis from the fibers of maximum compressive strain. Stress in all steel bars is taken into account. The strains in the bars are proportional to their distance from the neutral axis. For strains above yield, the stress is independent of strain and is taken as equal to the specified yield strength F_y. See to Figure 9.2-5 for strains and stresses for all three cases selected.

<u>Case 1 (P = 0)</u>

Assume all tension bars yield (which can be verified later):

T_{s1} = (0.20 in.^{2})(60 ksi) = 12.0 kips
T_{s2} = (0.20 in.^{2})(60 ksi) = 12.0 kips each

Because the neutral axis is close to the compression end of the wall, compression steel, C_{s1}, will be neglected (it would make little difference anyway) for Case 1:

$\Sigma F_y = 0$:
$C_m = \Sigma T$
C_m =(4)(12.0) = 48.0 kips

The compression block will be entirely within the first grouted cell:

$C_m = 0.8 f'_m ab$
48.0 = (0.8)(2.0 ksi)a(7.625 in)
a = 3.9 in. = 0.33 ft
$c = a/0.8$ = 0.33/0.8 = 0.41 ft

Thus, the neutral axis is determined to be 0.41 ft from the compression end on the wall, which is within the first grouted cell:

$\Sigma M_{cl} = 0$: (The math will be a little easier if moments are taken about the wall centerline.)
M_n = (16.33-0.33/2 ft)C_m + (16.00 ft) T_{s1} + (0.00 ft)ΣT_{s2} + (0.00 ft)P_n
M_n = (16.17)(48.0) + (16.00)(12) + 0 + 0 = 968 ft-kips
ϕM_n = (0.85)(968) = 823 ft-kips

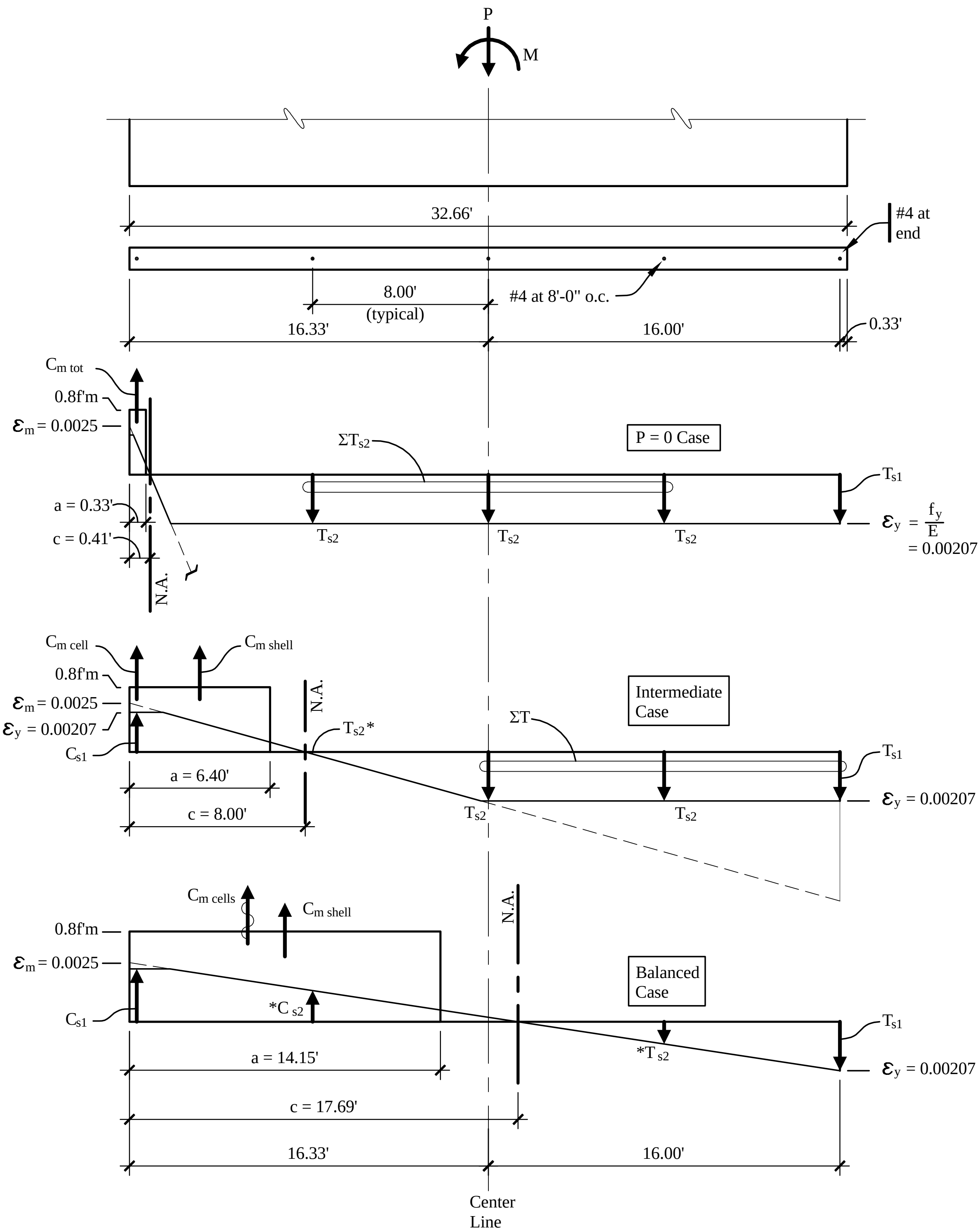


Figure 9.2-5 Strength of Birmingham 1 Wall D (1.0 ft = 0.3048 m). Strain diagram superimposed on strength diagram for the three cases. The low force in the reinforcement is neglected in the calculations.

To summarize, Case 1:

ϕP_n = 0 kips
ϕM_n = 823 ft-kips

<u>Case 2 (Intermediate case between P = 0 and P_{bal})</u>

Let c = 8.00 ft.(this is an arbitrary selection). Thus, the neutral axis is defined at 8 ft from the compression end of the wall:

$a = 0.8c = (0.8)(8.00) = 6.40$ ft

$C_{m\ shells} = 0.8f'_m$(2 shells)(1.25 in. / shell)(6.40 ft. (12 in./ft) = 307.2 kips
$C_{m\ cells} = 0.8\ f'_m$(41 in.^{2}) = 65.6 kips
$C_{m\ tot} = C_{m\ shells} + C_{m\ cells}$ = 307.2 + 65.6 = 373 kips

C_{s1} = (0.20 in.^{2})(60 ksi) = 12 kips (Compression steel is included in this case)
T_{s1} = (0.20 in.^{2})(60 ksi) = 12 kips
T_{s2} = (0.20 in.^{2})(60 ksi) = 12 kips each

Some authorities would not consider the compression resistance of reinforcing steel that is not enclosed within ties. The *Provisions* clearly allows inclusion of compression in the reinforcement.

$\Sigma F_y = 0$:
$C_{m\ tot} + C_{s1} = P_n + T_{s1} + \Sigma T_{s2}$
$373 + 12 = P_n + (3)(12.0)$
P_n = 349 kips
ϕP_n = (0.85)(349) = 297 kips

$\Sigma M_{cl} = 0$:
M_n = (13.13 ft)$C_{m\ shell}$ + (16.00 ft)($C_{m\ cell} + C_{s1}$) + (16.00 ft)T_{s1} + (8.00 ft)T_{s2}
M_n = (13.13)(307.2) + (16.00)(65.6 + 12) + (16.00)(12.0) + (8.00 ft)(12.0) = 5,563 ft-kips
ϕM_n = (0.85)(5,563) = 4,729 ft-kips

To summarize Case 2:

ϕP_n = 297 kips
ϕM_n = 4,729 ft-kips

<u>Case 3 (Balanced case)</u>

In this case, T_{s1} just reaches its yield stress:

$$c = \left[\frac{0.0025}{(0.0025+0.00207)}\right](32.33\text{ ft}) = 17.69\text{ ft}$$

$a = 0.8c = (0.8)(17.69) = 14.15$ ft
$C_{m\ shells} = 0.8f'_m$(2 shells)(1.25 in. / shell)(14.15 ft.) (12 in./ft) = 679.2 kips
$C_{m\ cells} = 0.8f'_m$(2 cells)(41 in.^{2}/cell) = 131.2 kips
$C_{m\ tot} = C_{m\ shells} + C_{m\ cells}$ = 810.4 kips

C_{s1} = (0.20 in.^{2})(60 ksi) = 12.0 kips

T_{s1} = (0.20 in.^{2})(60 ksi) = 12.0 kips

C_{s2} and T_{s2} are neglected because they are small, constituting less than 2 percent of the total P_n.

$\Sigma F_y = 0$:
$P_n = \Sigma C - \Sigma T$
$P_n = C_{m\ tot} + C_{s1} - T_{s1}$ = 810.4 + 12.0 - 12.0 = 810.4 kips
ϕP_n = (0.85)(810.4) = 689 kips

$\Sigma M_{cl} = 0$:
$M_n = 9.26\ C_{m\ shells} + ((16 + 8)/2)\ C_{m\ cells} + 16\ C_{s1} + 8\ T_{s2} + 16\ T_{s1}$
M_n = (9.26)(679.2) + (12.0)(131.2) + (16.00)(12.0) + (ignore small T_{s2}) + (16.0)(12.0) = 8,248 kips
ϕM_n = (0.85)(8,248) = 7,011 ft-kips

To summarize Case 3:

ϕP_n = 689 kips
ϕM_n = 7,011 ft-kips

Using the results from the three cases above, the ϕP_n - ϕM_n curve shown in Figure 9.2-6 is plotted. Although the portion of the ϕP_n - ϕM_n curve above the balanced failure point could be determined, it is not necessary here. Thus, only the portion of the curve below the balance point will be examined. This is the region of high moment capacity.

Similar to reinforced concrete beam-columns, in-plane compression failure of the cantilevered shear wall will occur if $P_u > P_{bal}$, and tension failure will occur if $P_u < P_{bal}$. A ductile failure mode is essential to the design, so the portion of the curve above the “balance point” is not useable.

As can be seen, the points for $P_{u\ min}$, M_u and $P_{u\ max}$, are within the ϕP_n - ϕM_n envelope; thus, the strength design is acceptable with the minimum reinforcement. Figure 9.2-6 shows two schemes for determining the design flexural resistance for a given axial load. One interpolates along the straight line between pure bending and the balanced load. The second makes use of intermediate points for interpolation. This particular example illustrates that there can be a significant difference in the interpolated moment capacity between the two schemes for axial loads midway between the balanced load and pure bending.

For the purpose of shear design, the value of φM_N at the design axial load is necessary. Interpolating between the intermediate point and the P = 0 point for P = 202 kips yields φM_N = 3,480 ft-kip. Thus, the factor on shear to represent development of 125 percent of flexural capacity is:

$$1.25\frac{\phi M_N/\phi}{M_U} = 1.25\frac{3480/0.85}{2640} = 1.94$$

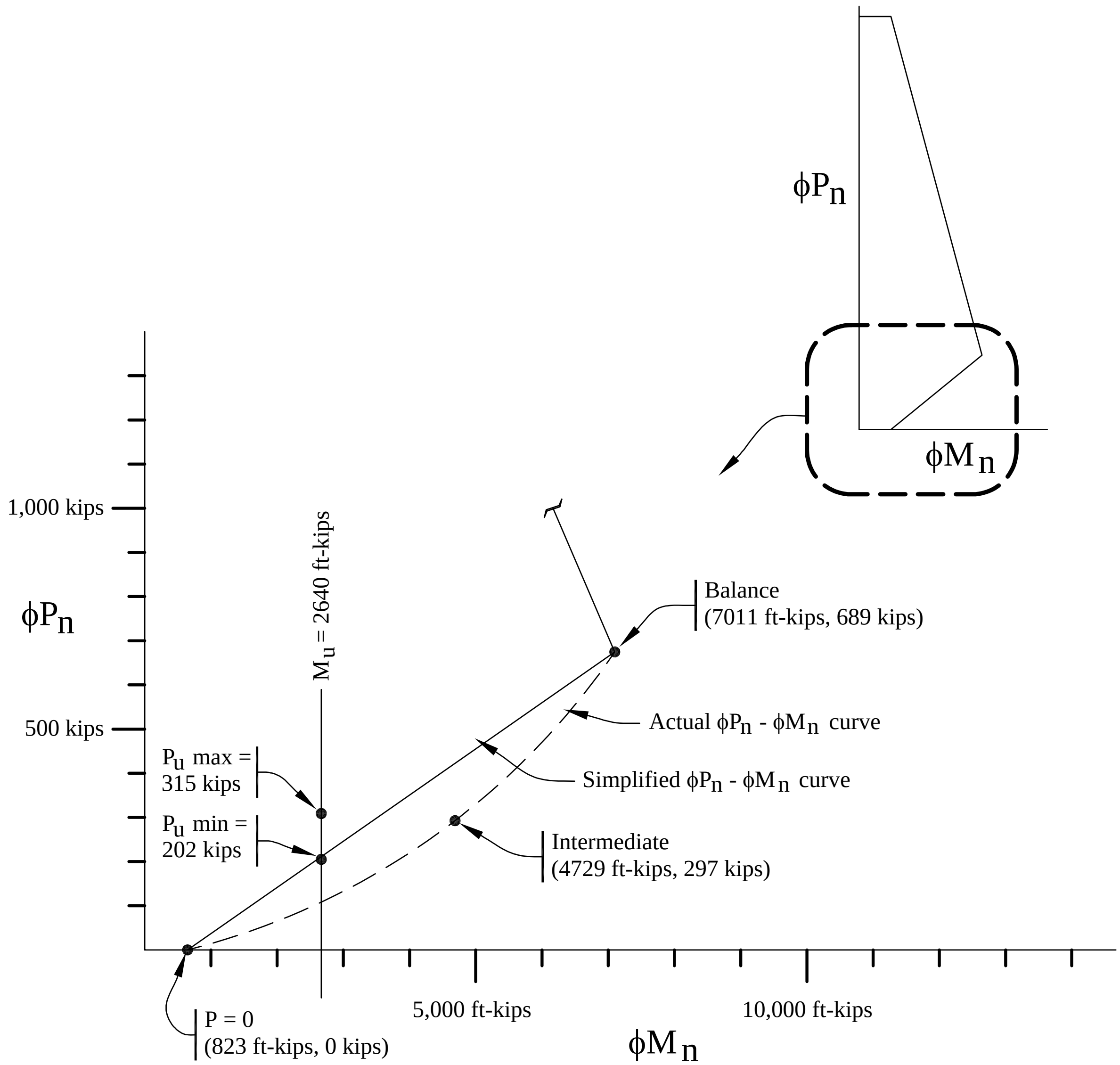


Figure 9.2-6 φP_{11} - φM_{11} diagram for Birmingham 1 Wall D (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.2.4.5.2.2 Ductility check

Provisions Sec.11.6.2.2 [ACI 530, Sec. 3.2.3.5] requires that the critical strain condition correspond to a strain in the extreme tension reinforcement equal to 5 times the strain associated with F_y. Note that this calculation uses unfactored gravity axial loads (*Provisions* Sec.11.6.2.2 [ACI 530, Sec. 3.2.3.5]). Refer to Figure 9.2-5 and the following calculations which illustrate this using loads at the bottom story (highest axial loads). Calculations for other stories are not presented in this example.

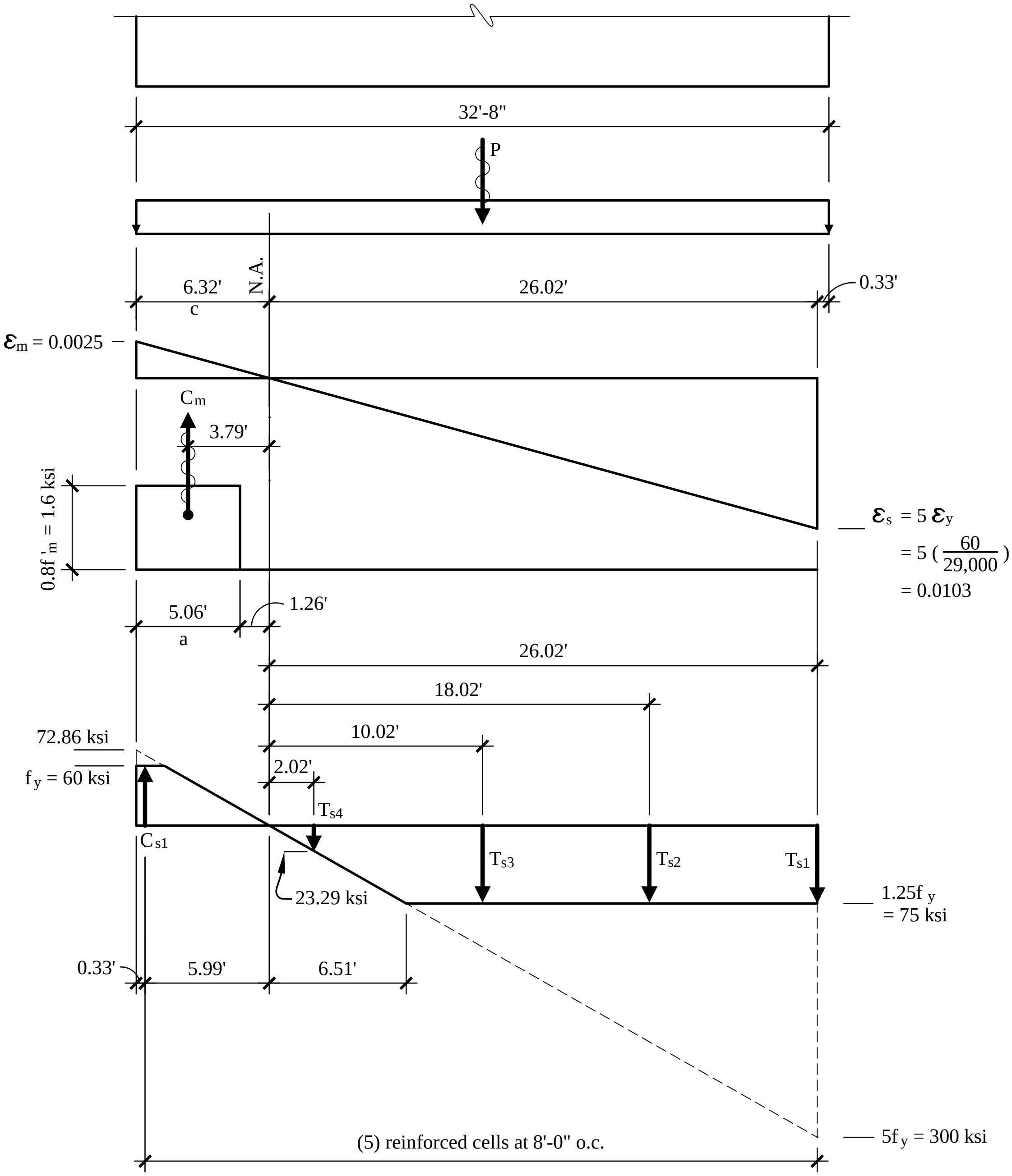


Figure 9.2-7 Ductility check for Birmingham 1 Wall D (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa).

For Level 1 (bottom story), the unfactored axial loads are:

P = 232 kips + weight of half of first story wall = 232 + 6.4 = 238.4 kips

Refer to Figure 9.2-7:

$C_m = 0.8 f'_m(ab + A_{cell})$ = (1.6 ksi)[(5.06 ft. x 12 in./ft.)(2.5 in.) + 41 in.^{2}] = 308.5 kips (same as above)
$C_{s1} = F_yA_s$ = (60 ksi)(0.20 in.^{2}) = 12.0 kips
$T_{s1} = T_{s2} = T_{s3}$ = (1.25 × 60 ksi)(0.20 in.^{2}) = 15 kips

T_{s4} = (23.29 ksi)(0.20 in.^{2}) = 4.6 kips

$\sum C > \sum P + T$
$C_m + C_{s1} > P + T_{s1} + T_{s2} + T_{s3} + T_{s4}$
308.5 + 12.0 > 238.4 + 15 + 15 + 15 + 4.6
320.5 kips > 288 kips OK

There is more compression capacity than required so ductile failure condition governs.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

9.2.4.6 Birmingham 1 Deflections

The calculations for deflection involve many variables and assumptions, and it must be recognized that any calculation of deflection is approximate at best.

Deflections are to be calculated and compared with the prescribed limits set forth by *Provisions* Table 5.2.8. Deformation requirements for masonry structures are given in *Provisions* Sec. 11.5.4 [Table 4.5-1].

The following procedure will be used for calculating deflections:

1. For each story, compare M_x (from Table 9.2-3) to $M_{cr} = S(f_r + P_{u\ min} / A)$ to determine if wall will crack.
2. If $M_{cr} < M_x$, then use cracked moment of inertia and *Provisions* Eq. 11.5.4.3.
3. If $M_{cr} > M_x$, then use $I_n = I_g$ for moment of inertia of wall.
4. Compute deflection for each level.

Other approximations can be used such as the cubic interpolation formula given in *Provisions* 11.5.4.3, but that equation was derived for reinforced concrete members acting as single span beams, not cantilevers. In the authors' opinion, all these approximations pale in comparison to the approximation of nonlinear deformation using C_d.

For the Birmingham 1 building:

b_e = effective masonry wall width
b_e = [(2 × 1.25 in.)(32.67 ft × 12) + (5 cells)(41 in.^{2}/cell)]/(32.67 ft × 12) = 3.02 in.
$S = b_e l^2/6$ = (3.02)(32.67 × 12)^2/6 = 77,434 in.^{3}
f_r = 0.250 ksi
$A = b_e l$ = (3.02 in.)(32.67 ft × 12) = 1,185 in.^{2}

P_u is calculated using 1.00D (see Table 9.2-4). 1.00D is considered to be a reasonable value for axial load for this admittedly approximate analysis. If greater conservatism is desired, P_u could be calculated using 0.85D.

The results are shown in Table 9.2-5.

Table 9.2-5 Birmingham 1 Cracked Wall Determination

Level	$P_{u_{min}}$ (kips)	M_{cr} (ft-kips)	M_u (ft-kips)	Status
5	41	1836	225	uncracked
4	89	2098	652	uncracked
3	137	2359	1230	uncracked
2	185	2621	1910	uncracked
1	232	2877	2640	uncracked

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

For uncracked walls:

$$I_n = I_g = bl^3/12 = (3.02\text{ in.})(32.67 \times 12)^3/12 = 1.52 \times 10^7\text{ in.}^4$$

The calculation of δ will consider flexural and shear deflections. For the final determination of deflection, a RISA-2D analysis was made. The result is summarized Table 9.2-6 below. Figure 9.2-8 illustrates the deflected shape of the wall.

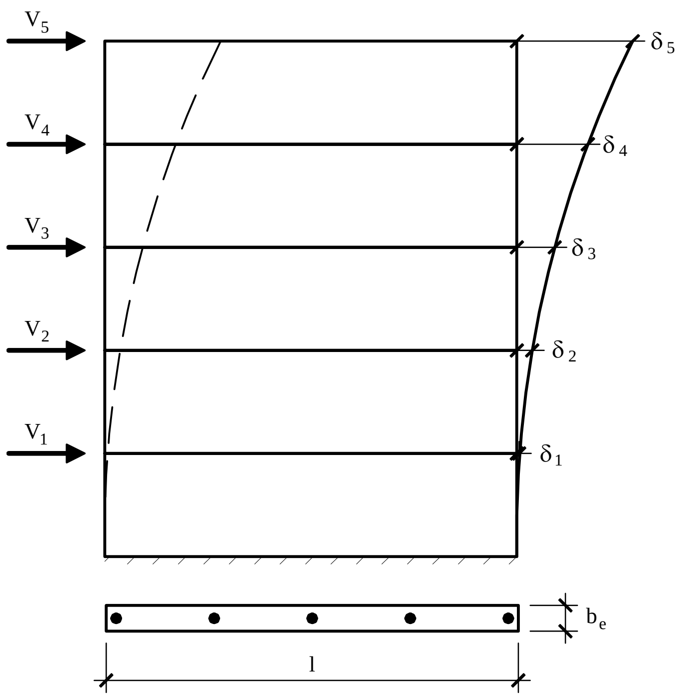


Figure 9.2-8 Shear wall deflections.

Table 9.2-6 Deflections, Birmingham 1

Level	F (kips)	I_{eff} (in.^{4})	$\delta_{flexural}$ (in.)	δ_{shear} (in.)	δ_{total} (in.)	$C_d\,\delta_{total}$ (in.)	Δ (in.)
54321	26.0	1.52×10^7	0.108	0.054	0.162	0.284	0.061
	23.2	1.52×10^7	0.078	0.049	0.128	0.223	0.066
	17.4	1.52×10^7	0.049	0.041	0.090	0.157	0.066
	11.7	1.52×10^7	0.024	0.028	0.052	0.091	0.054
	5.8	1.52×10^7	0.007	0.015	0.021	0.037	0.037

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm.

The maximum story drift occurs at Level 4 (*Provisions* Table 5.2.8 [Table 4.5-1]):

[The specific procedures for computing deflection of shear walls have been removed from the 2003 Provisions. ACI 530 does not contain the corresponding provisions in the text, however, the commentary contains a discussion and equations that are similar to the procedures in the 2000 *Provisions*. However, as indicated previously, there is a potential conflict between the drift limits in 2003 *Provisions* Table 4.5-1 and ACI 530 Sec. 1.13.3.2.]

Δ = 0.066 in. < 1.04 in. = $0.01h_n$ OK

9.2.4.7 Birmingham 1 Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4-6.1.3] requires that the bearing walls be designed for out-of-plane loads determined as follows:

$w = 0.40S_{DS}W_c \geq 0.1W_c$
$w = (0.40)(0.24)(45 \text{ psf}) = 4.3 \text{ psf} < 4.5 \text{ psf} = 0.1W_c$

The calculated seismic load, w = 4.5 psf, is much less than wind pressure for exterior walls and is also less than the 5 psf required by IBC Sec. 1607.13 for interior walls. Thus, seismic loads do not govern the design of any of the walls for loading in the out-of-plane direction.

9.2.4.8 Birmingham 1 Orthogonal Effects

Orthogonal effects do not have to be considered for Seismic Design Category B (*Provisions* Sec. 5.2.5.2.1 [Sec. 4.4.2.1]).

This completes the design of Transverse Wall D.

9.2.4.9 Summary of Design for Birmingham 1 Wall D

8 in. CMU
f'_m = 2,000 psi

Reinforcement:

One vertical #4 bar at wall end cells
Vertical #4 bars at 8 ft on center at intermediate cells throughout
Bond beam with two - #4 bars at each story just below the floor and roof slabs

Horizontal joint reinforcement at 16 inches

Grout at cells with reinforcement and at bond beams.

9.2.5 Seismic Design for New York City

This example focuses on differences from the design for the Birmingham 1 site.

9.2.5.1 New York City Weights

As before, use 67 psf for 8-in.-thick normal weight hollow core plank plus the nonmasonry partitions. This site is assigned to Seismic Design Category C, and the walls will be designed as intermediate reinforced masonry shear walls (*Provisions* Sec. 11.11.4 [Sec. 11.2.1.4] and Sec. 11.3.7 [Sec. 11.2.1.4]), which requires prescriptive seismic reinforcement (*Provisions* Sec. 11.3.7.3 [ACI 530, Sec. 1.13.2.2.4]). Intermediate reinforced masonry shear walls have a minimum of #4 bars at 4 ft on center. For this example, 48 psf will be assumed for the 8-in. CMU walls. The 48 psf value includes grouted cells and bond beams in the course just below the floor planks. In Seismic Design Category C, more of the regularity requirement must be checked. It will be shown that this symmetric building with a seemingly well distributed lateral force system is torsionally irregular by the *Provisions*.

Story weight, w_i:

Roof

Roof slab (plus roofing) = (67 psf) (152 ft)(72 ft) = 733 kips
Walls = (48 psf)(589 ft)(8.67 ft/2) + (48 psf)(4)(36 ft)(2 ft) = <u>136 kips</u>
Total = 869 kips

There is a 2-ft high masonry parapet on four walls and the total length of masonry wall is 589 ft.

Typical floor

Slab (plus partitions) = 733 kips
Walls = (48 psf)(589 ft)(8.67 ft) = <u>245 kips</u>
Total = 978 kips

Total effective seismic weight, W = 869 + (4)(978) = 4781 kips

This total excludes the lower half of the first story walls, which do not contribute to seismic loads that are imposed on CMU shear walls.

9.2.5.2 New York City Base Shear Calculation

The seismic response coefficient, C_s, is computed from *Provisions* Sec. 5.4.1.1 [Sec. 5.2-1.1]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.39}{2.5/1} = 0.156$$

The value of C_s need not be greater than:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.14}{0.338(2.5/1)} = 0.166$$

where T is the same as found in Sec. 9.2.4.2.

The value for C_s is taken as 0.156 (the lesser of the two computed values). This value is still larger than the minimum specified in *Provisions* Eq. 5.3.2.1-3. Using *Provisions* Eq. 5.4.1.1-3:

$$C_s = 0.044 S_{D1} I = (0.044)(0.14)(1) = 0.00616$$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4.1 [Eq. 5.2-1]:

$$V = C_s W = (0.156)(4{,}781) = 746 \text{ kips}$$

9.2.5.3 New York City Vertical Distribution of Seismic Forces

The vertical distribution of seismic forces is determined in accordance with *Provisions* Sec. 5.4.4 [Sec. 5.2.3], which was described in Sec. 9.2.4.3. Note that for *Provisions* Eq. 5.4.3-2 [Eq. 5.2-11], $k = 1.0$ since $T = 0.338$ sec (similar to the Birmingham 1 building).

The application of the *Provisions* equations for this building is shown in Table 9.2-7:

Table 9.2-7 New York City Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_X (kips)	M_x (ft-kips)
5	869	43.34	37,657	0.3076	229	2.3e+14	1,985
4	978	34.67	33,904	0.2770	207		5,765
3	978	26.00	25,428	0.2077	155		10,889
2	978	17.33	16,949	0.1385	103		16,907
1	978	8.67	8,476	0.0692	52		23,370
$\sum$	4,781		122,414	1.000	746		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m.

9.2.5.4 New York City Horizontal Distribution of Forces

The initial distribution is the same as Birmingham 1. See Sec. 9.2.4.4 and Figure 9.2-3 for wall designations.

Total shear in Wall Type D:

$$V_{tot} = 0.125V + 0.0238V = 0.149V$$

Provisions Sec.5.4.4.3 [Sec. 4.3.2.2] requires a check of torsional irregularity using the ratio of maximum displacement at the end of the structure, including accidental torsion, to the average displacement of the two ends of the building. For this simple and symmetric structure, the actual displacements do not have to be computed to find the ratio. Relying on symmetry and the assumption of rigid diaphragm behavior used to distribute the forces, the ratio of the maximum displacement of Wall D to the average displacement of the floor will be the same as the ratio of the wall shears with and without accidental torsion:

$$\frac{F_{max}}{F_{ave}} = \frac{0.149V}{0.125V} = 1.190$$

This can be extrapolated to the end of the rigid diaphragm therefore:

$$\frac{\delta_{max}}{\delta_{ave}} = 1 + 0.190\left(\frac{152/2}{36}\right) = 1.402$$

Provisions Table 5.2.3.2 [Table 4.3-2] defines a building as having a "Torsional Irregularity" if this ratio exceeds 1.2 and as having an "Extreme Torsional Irregularity" if this ratio exceeds 1.4. Thus, an important result of the Seismic Design Category C classification is that the total torsion must be amplified by the factor:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{ave}}\right)^2 = \left(\frac{1.402}{1.2}\right)^2 = 1.365$$

Therefore, the portion of the base shear for design of Wall D is now:

$$V_D = 0.125V + 1.365(0.0238V) = 0.158V$$

which is a 5.8 percent increase from the fraction before considering torsional irregularity.

The total story shear and overturning moment may now be distributed to Wall D and the wall proportions checked. The wall capacity will be checked before considering deflections.

9.2.5.5 New York City Transverse Wall D

The strength or limit state design concept is used in the *Provisions*.

9.2.5.5.1 New York City Shear Strength

Similar to the design for Birmingham 1, the shear wall design is governed by:

$$V_u \le \phi V_n$$

$$V_n = V_m + V_s$$

$$V_n \text{ max} = 4 \text{ to } 6\sqrt{f'_m}A_n \quad \text{depending on } M/Vd$$

$$V_m = \left[4 - 1.75\left(\frac{M}{Vd}\right)\right]A_n\sqrt{f'_m} + 0.25\,P$$

$$V_s = 0.5\left(\frac{A_v}{s}\right)f_y d_v$$

where

$$A_n = (2 \times 1.25 \text{ in.} \times 32.67 \text{ ft} \times 12 \text{ in.}) + (41 \text{ in.}^2 \times 9 \text{ cells}) = 1{,}349 \text{ in.}^2$$

The shear strength of each Wall D, based on the aforementioned formulas and the strength reduction factor of $\phi = 0.8$ for shear from *Provisions* Table 11.5.3 [ACI 530, Sec. 3.1.4.3], is summarized in Table 9.2-8. Note that V_x and M_x in this table are values from Table 9.2-7 multiplied by 0.158 (representing the portion of direct and indirect shear assigned to Wall D), and P is the dead load of the roof or floor times the tributary area for Wall D.

Table 9.2-8 New York City Shear Strength Calculation for Wall D

Story	V_x (kips)	M_x (ft-kips)	M_x/V_xd	2.5 V_x (kips)	P (kips)	φV_m (kips)	φV_s (kips)	φV_n (kips)	φV_n *max* (kips)
5	36.1	313	0.265	90.3	42	179.0	54.6	233.6	287.6
4	68.7	908	0.405	171.8	90	176.9	54.6	231.5	269.7
3	93.1	1715	0.564	232.8	139	173.2	54.6	227.8	249.2
2	109.3	2663	0.746	273.3	188	167.7	54.6	222.3	225.8
1	117.5	3680	0.959	293.8	236	159.3	54.6	213.9	198.4

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

V_u exceeds φV_n at the lower three stories. As will be shown at the conclusion of the design for flexure, the factor to achieve 125 percent of the nominal flexural capacity is 1.58. This results in V_u being less than φV_n at all stories. If that were not the case, it would be necessary to grout more cells to increase A_n or to increase f'_m.

9.2.5.5.2 New York City Axial and Flexural Strength

The walls in this example are all load-bearing shear walls because they support vertical loads as well as lateral forces. In-plane calculations include:

1. Strength check and
2. Ductility check.

9.2.5.5.2.1 Strength check

Wall demands, using load combinations determined previously, are presented in Table 9.2-9 for Wall D. In the table, Load Combination 1 is 1.28D + Q_E + 0.5L and Load Combination 2 is 0.82D + Q_E.

Table 9.2-9 Demands for New York City Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
5	42	0	54	313	34	313
4	90	8	119	908	74	908
3	139	17	186	1715	114	1715
2	188	25	253	2663	154	2663
1	236	34	319	3680	194	3680

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

As in Sec. 9.2.4.5.2, strength at the bottom story (where P, V, and M are the greatest) will be examined. The strength design will consider Load Combination 2 from Table 9.2-9 to be the governing case because it has the same lateral load as Load Combination 1 but with lower values of axial force. Refer to Fig. 9.2-9 for notation and dimensions.

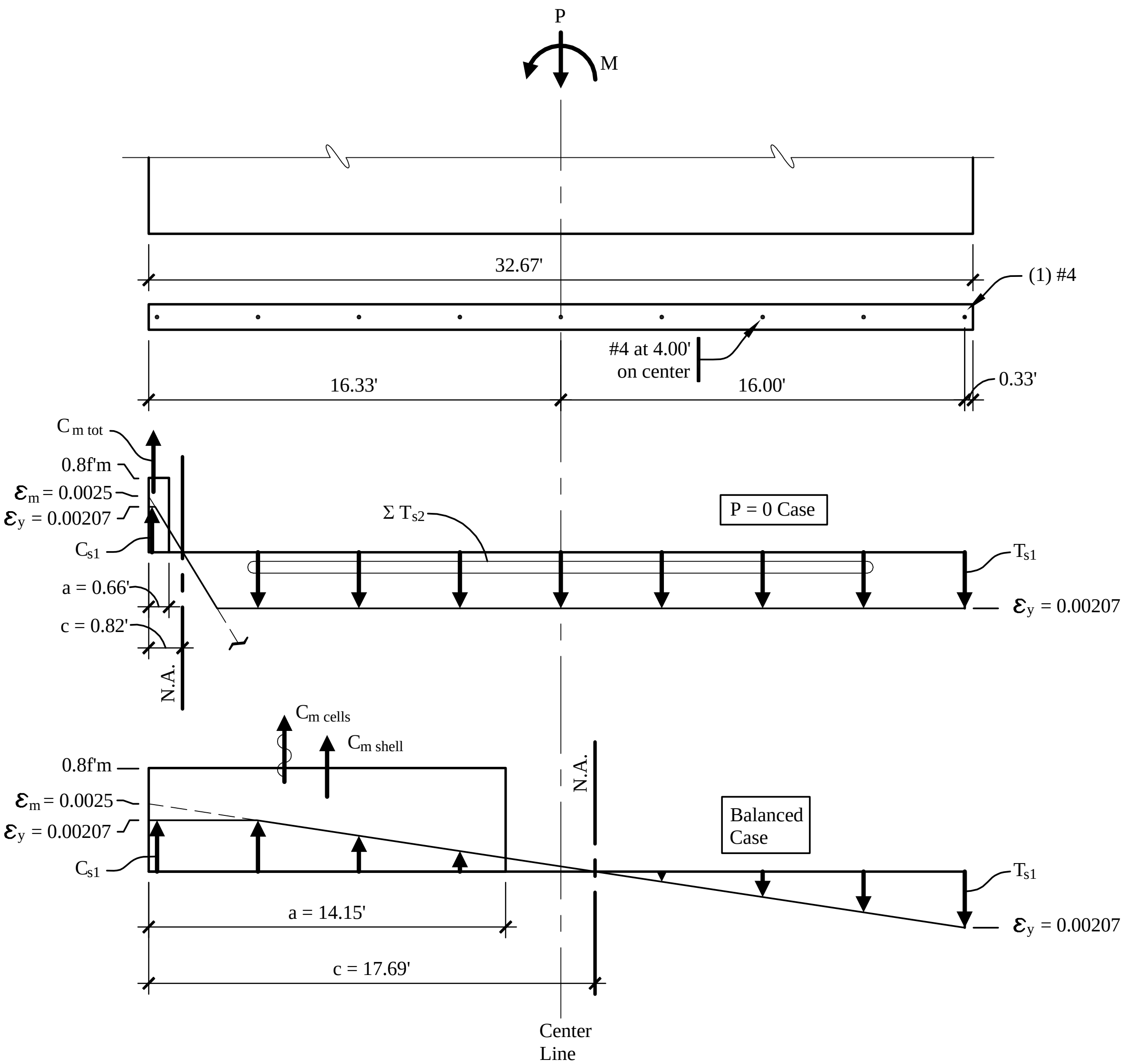


Figure 9.2-9 Strength of New York City and Birmingham 2 Wall D. Strength diagrams are superimposed over the strain diagrams for the two cases (intermediate case is not shown) (1.0 ft = 0.3048 m).

Examine the strength of Wall D at Level 1:

$$P_{u_{min}} = 0.82\,D = 0.82\text{ (236 + factored weight of lower half of first story wall)}$$
$$= 0.82(236 + 6.4) = 199\text{ kips}$$
$$P_{u_{max}} = 1.28\,D + 0.5\,L\ 319 = 1.28(236 + 6.4) + 0.5(34) = 327\text{ kips}$$
$$M_u = 3{,}680\text{ ft-kips}$$

Because intermediate reinforced masonry shear walls are used (Seismic Design Category C), vertical reinforcement at is required at 4 ft on center in accordance with *Provisions* Sec. 11.3.7.3 [ACI 530, Sec. 1.13.2.2.4]. Therefore, try one #4 bar in each end cell and #4 bars at 4 ft on center at all intermediate cells.

The calculation procedure is similar to that for the Birmingham 1 building presented in Sec. 9.2.4.5.2. The results of the calculations (not shown) for the New York building are summarized below.

<u>P = 0 case</u>

$\phi P_n = 0$
ϕM_n = 1,475 ft-kips

<u>Intermediate case</u>

c = 8.0 ft
ϕP_n = 330 kips
ϕM_n = 5,600 ft-kips

<u>Balanced case</u>

ϕP_n = 807
ϕM_n = 8,214 ft-kips

With the intermediate case, it is simple to use the three points to make two straight lines on the interaction diagram. Use the simplified ϕP_n - ϕM_n curve shown in Figure 9.2-10. The straight line from pure bending to the balanced point is conservative and can easily be used where the design is not as close to the criterion. It is the nature of lightly reinforced and lightly loaded masonry walls that the intermediate point is frequently useful.

Use one #4 bar in each end cell and one #4 bar at 4 ft on center throughout the remainder of the wall.

As shown in the design for Birmingham 1,for the purpose of shear design, the value of φM_N at the design axial load is necessary. Interpolating between the intermediate point and the P = 0 point for P = 199 kips yields φM_N = 3,960 ft-kip. Thus, the factor on shear to represent development of 125 percent of flexural capacity is:

$$1.25\frac{\phi M_N / \phi}{M_U} = 1.25\frac{3960 / 0.85}{3680} = 1.58$$

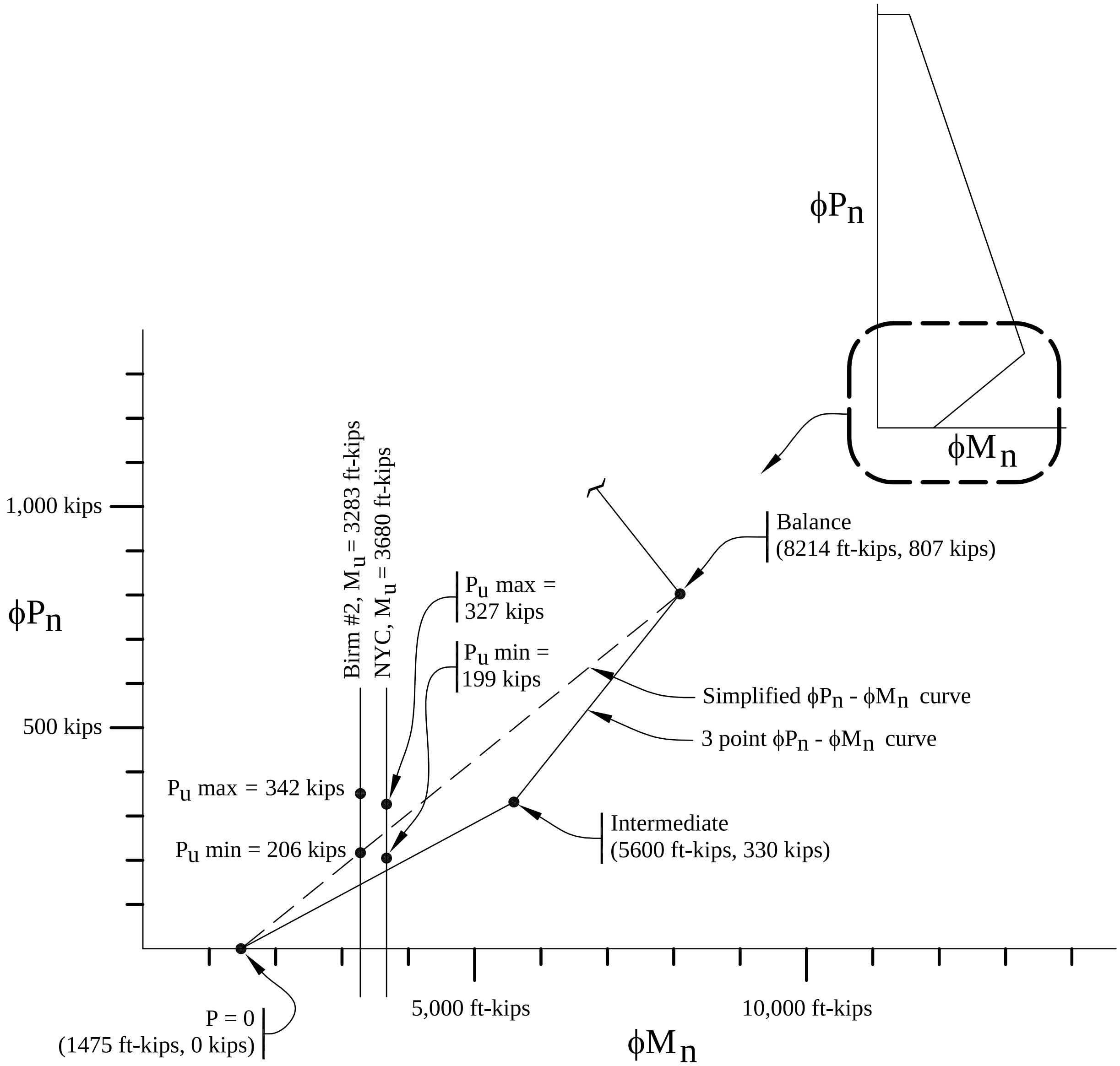


Figure 9.2-10 φP_{11} - φM_{11} Diagram for New York City and Birmingham 2 Wall D (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.2.5.5.2.2 Ductility check

Refer to Sec. 9.2.4.5.2, Item 2, for explanation [see Sec. 9.2.4.5.2 for discussion of revisions to the ductility requirements in the 2003 *Provisions*.]. For Level 1 (bottom story), the unfactored loads are:

P = 236 + weight of lower ½ of first story wall = 236 + 6.4 = 242.4 kips
M = 3,483 ft-kips

$C_m = 0.8\ f_m'\ [(a)(b) + A_{cells}]$
where b = face shells = (2 × 1.25 in.) and A_{cell} = 41 in.^{2}

C_m = (1.6 ksi)[(5.03 ft × 12)(2.5 in.) + (2)(41)] = 372.6 kips

$C_{s1} = F_y A_s$ = (60 ksi)(0.20 in.^{2}) = 12 kips
C_{s2} = (22.6 ksi)(0.20 in.^{2}) = 4.5 kips
$T_{s1} = T_{s2} = T_{s3} = T_{s4} = T_{s5}$ = (75 ksi)(0.20 in.^{2}) = 15 kips
T_{s6} = (69.6 ksi)(0.20 in.^{2}) = 13.9 kips
T_{s7}= (23.5 ksi)(0.20 sq. in.) = 4.7 kips

$\sum C > \sum P + T$
$C_m + C_{s1} + C_{s2} > P + T_{s1} + T_{s2} + T_{s3} + T_{s4} + T_{s5} + T_{s6} + T_{s7}$
372.6 + 12.0 + 4.5 > 242.5 + 5(15) + 13.9 + 4.7
389 kips > 336 kips OK

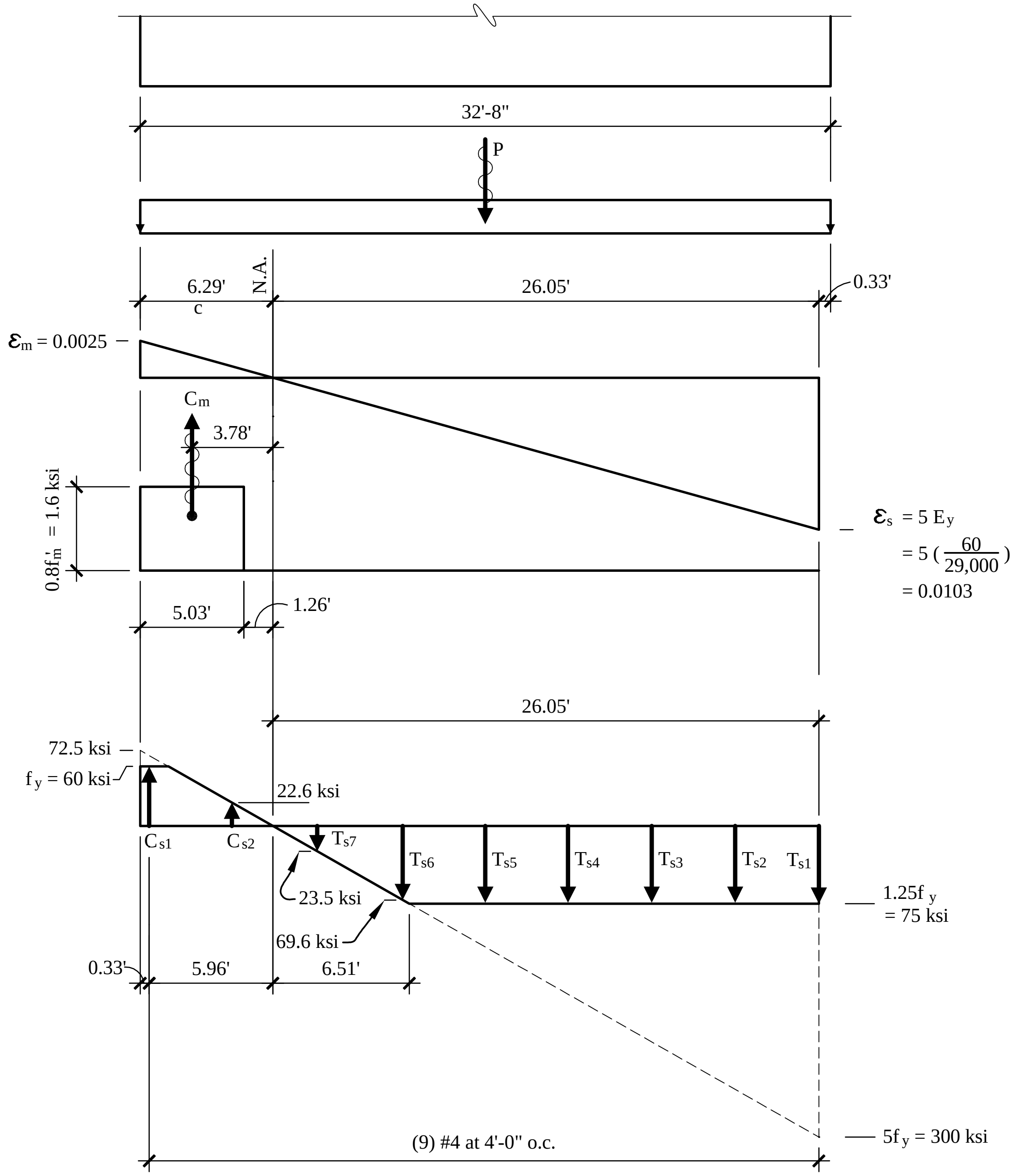


Figure 9.2-11 Ductility check for New York City and Birmingham 2 Wall D (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa).

9.2.5.6 New York Deflections

Refer to 9.2.4.6 for more explanation [see Sec. 9.2.4.6 for discussion of revisions to the deflection computations and requirements in the 2003 *Provisions*, as well as the potentially conflicting drift limits]. For the New York City building, the determination of whether the walls will be cracked is:

b_e = effective masonry wall width
b_e = [(2 × 1.25 in.)(32.67 ft × 12) + (9 cells)(41 in.^{2}/cell)]/(32.67 ft × 12) = 3.44 in.
$A = b_e\, l$ = (3.44 in.)(32.67 × 12) = 1,349 in.^{2}

$S = b_e l^2/6 = (3.44)(32.67 \times 12)^2 /6 = 88{,}100$ in.^{3}
$f_r = 0.250$ ksi

P_u is calculated using 1.00D (see Table 9.2-8 for values and refer to Sec. 9.2.4.6 for discussion). Table 9.2-10 a summarizes of these calculations.

Table 9.2-10 New York City Cracked Wall Determination

Level	P_u (kips)	M_{cr} (ft-kips)	M_x (ft-kips)	Status
5	42	2064	313	uncracked
4	90	2325	908	uncracked
3	139	2592	1715	uncracked
2	188	2860	2663	uncracked
1	236	3120	3680	cracked

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

For the uncracked walls:

$I_n = I_g = bl^3/12 = (3.44 \text{ in.})(32.67 \times 12)^3/12 = 1.73 \times 10^7$ in.^{4}

For the cracked wall, observe that the intermediate point on the interaction diagram is relatively close to the design point. Therefore, as a different type of approximation, compute a cracked moment of inertia using the depth to the neutral axis of 8.0 ft:

$I_{cr} = b_e c^3/3 + \sum nA_s d^2$

$I_{cr} = (3.44 \text{ in.})(8.0 \text{ ft} \times 12)^3/3 + 19.3(0.2)(4.33^2 + 8.33^2 + 12.33^2 + 16.33^2 + 20.33^2 + 24.33^2)144 =$
$= 1.01 \times 10^6 + 0.84 \times 10^6 = 1.85 \times 10^6$ in.^{4}

Per *Provisions* Eq. 11.5.4.3:

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n$$

$I_{eff} = 1.13 \times 10^7$ in.^{4}

Provisions 11.5.4.3 would imply that I_{eff} would be used for the full height. Another reasonable option is to use I_{cr} at the first story and I_g above that. The calculation of δ should consider shear deflections in addition to the flexural deflections. For this example I_{eff} will be used over the full height for the final determination of deflection (a RISA 2D analysis was made). The result is summarized in Table 9.2-11.

Table 9.2-11 New York City Deflections

Level	F (kips)	I_{eff} (in.^{4})	$\delta_{flexural}$ (in.)	δ_{shear} (in.)	δ_{total} (in.)	$C_d\,\delta_{total}$ (in.)	Δ (in.)
54321	34.1	1.13×10^7	0.256	0.080	0.336	0.757	0.163
	30.9	1.13×10^7	0.189	0.075	0.264	0.593	0.171
	23.1	1.13×10^7	0.124	0.064	0.188	0.422	0.163
	15.3	1.13×10^7	0.065	0.050	0.115	0.259	0.141
	7.7	1.13×10^6	0.020	0.033	0.053	0.118	0.118

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm

The maximum story drift occurs at Level 4:

Δ_4 = 0.171 in. < 1.04 in. = 0.01 h_4 (*Provisions* Table 5.2.8 [Table 4.5-1]) OK

The total displacement at the top of the wall is

Δ = 0.757 in. < 5.2 in. = 0.01 h_n (*Provisions* 11.5.4.1.1) OK

9.2.5.7 New York City Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4.4.2.2] requires that the bearing walls be designed for out-of-plane loads determined as

$$w = 0.40\, S_{DS}\, W_c \geq 0.1 W_c$$

With S_{DS} = 0.39 , $w = 0.156W_c > 0.1W_c$, so w = (0.156)(48 psf) = 7.5 psf, which is much less than wind pressure for exterior walls. Even though Wall D is not an exterior wall, the lateral pressure is sufficiently low that it is considered acceptable by inspection, without further calculation. Seismic loads do not govern the design of Wall D for loading in the out-of-plane direction.

9.2.5.8 New York City Orthogonal Effects

According to *Provisions* Sec. 5.2.5.2.2, orthogonal interaction effects have to be considered for Seismic Design Category C when the equivalent lateral force (ELF) procedure is used (as it is here). However, the out-of-plane component of only 30 percent of 7.5 psf on the wall will not produce a significant effect when combined with the in-plane direction of loads, so no further calculation will be made.

This completes the design of the transverse Wall D for the New York building.

9.2.5.9 Summary of New York City Wall D Design

8 in. CMU
f'_m = 2,000 psi

Reinforcement:

Vertical #4 bars at 4 ft on center throughout the wall
Bond beam with two #4 at each story just below the floor or roof slabs
Horizontal joint reinforcement at alternate courses

9.2.6 Birmingham 2 Seismic Design

The emphasis here is on differences from the previous two locations for the same building. Per *Provisions* Table 5.2.5.1 [Table 4.4-1], the torsional irregularity requires that the design of a Seismic Design Category D building be based on a dynamic analysis. Although not explicitly stated, the implication is that the analytical model should be three-dimensional in order to capture the torsional response. This example will compare both the equivalent lateral force procedure and the modal response spectrum analysis procedure and will demonstrate that, as long as the torsional effects are accounted for, the static analysis could be considered adequate for design.

9.2.6.1 Birmingham 2 Weights

The floor weight for this examples will use the same 67 psf for 8-in.-thick, normal weight hollow core plank plus roofing and the nonmasonry partitions as used in the prior examples (see Sec. 9.2.1). This site is assigned to Seismic Design Category D, and the walls will be designed as special reinforced masonry shear walls (*Provisions* Sec. 11.11.5 and Sec. 11.3.8[ACI 530, Sec. 1.13.2.2.5), which requires prescriptive seismic reinforcement (*Provisions* Sec. 11.3.7.3). Special reinforced masonry shear walls have a maximum spacing of rebar at 4 ft on center both horizontally and vertically. Also, the total area of horizontal and vertical reinforcement must exceed 0.0020 times the gross area of the wall, and neither direction may have a ratio of less than 0.0007. The vertical #4 bars at 48 in. used for the New York City design yields a ratio of 0.00055, so it must be increased. Two viable options are #5 bars at 48 in. (yielding 0.00085) and #4 bars at alternating spaces of 32 in. and 40 in. (12 bars in the wall), which yields 0.0080. The latter is chosen in order to avoid unnecessarily increasing the shear demand. Therefore, the horizontal reinforcement must be (0.0020 - 0.0008)(7.625 in.)(12 in./ft.) = 0.11 in.^{2}/ft. or 0.95 in.^{2} per story. Two #5 bars in bond beams at 48 in. on center will be adequate. For this example, 56 psf weight for the 8-in.-thick CMU walls will be assumed. The 56 psf value includes grouted cells and bond beams.

Story weight, w_i:

Roof:

Roof slab (plus roofing) = (67 psf) (152 ft)(72 ft)	= 733 kips
Walls = (56 psf)(589 ft)(8.67 ft/2) + (56 psf)(4)(36 ft)(2 ft)	= 159 kips
Total	= 892 kips

There is a 2-ft-high masonry parapet on four walls and the total length of masonry wall is 589 ft.

Typical floor:

Slab (plus partitions)	= 733 kips
Walls = (56 psf)(589 ft)(8.67 ft)	= 286 kips
Total	= 1,019 kips

Total effective seismic weight, W = 892 + (4)(1,019) = 4,968 kips

This total excludes the lower half of the first story walls which do not contribute to seismic loads that are imposed on CMU shear walls.

9.2.6.2 Birmingham 2 Base Shear Calculation

The ELF analysis proceeds as described for the other locations. The seismic response coefficient, C_s, is computed using *Provisions* Eq. 5.4.1.1-1 [Eq. 5.2-2] and 5.4.1.1-2 [Eq.5.2-3]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.47}{3.5/1} = 0.134 \qquad \text{(Controls)}$$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.28}{0.338(3.5/1)} = 0.237$$

This is somewhat less than the 746 kips computed for the New York City design due to the larger *R* factor.

The fundamental period of the building, based on *Provisions* Eq. 5.4.2.1-1 [Eq.5.2-6], is 0.338 sec as computed previously (the approximate period, based on building system and building height, will be the same for all locations). The value for C_s is taken as 0.134 (the lesser of the two values). This value is still larger than the minimum specified in *Provisions* Eq. 5.3.2.1-3 which is:

$$C_s = 0.044 S_{D1} I = (0.044)(0.28)(1) = 0.012$$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4.1 [Eq.5.2-1] as:

$$V = C_s W = (0.134)(4{,}968) = 666 \text{ kips}$$

A three-dimensional (3D) model was created in SAP 2000 for the modal response spectrum analysis. The masonry walls were modeled as shell bending elements and the floors were modeled as an assembly of beams and shell membrane elements. The beams have very little mass and a large flexural moment of inertia to avoid consideration of models of vertical vibration of the floors. The flexural stiffness of the beams was released at the bearing walls in order to avoid a wall slab frame that would inadvertently increase the torsional resistance. The mass of the floors was captured by the shell membrane elements. Table 9.2-12 shows data on the modes of vibration used in the analysis.

Provisions Sec. 4.1.2.6 [Sec. 3.3.4] was used to create the response spectrum for the modal analysis. The key points that define the spectrum are:

$T_S = S_{D1}/S_{DS} = 0.28/0.47 = 0.60$ sec
$T_0 = 0.2\ T_S = 0.12$ sec
at $T = 0$, $S_a = 0.4\ S_{DS}/R = 0.0537\ g$
from $T = T_0$ to T_S, $S_a = S_{DS}/R = 0.1343\ g$
for $T > T_S$, $S_a = S_D 1/(RT) = 0.080/T$

The computed fundamental period is less than the approximate period. The transverse direction base shear from the SRSS combination of the modes is 457.6 kips, which is considerably less than that obtained using the ELF method.

Provisions Sec. 5.5.7 [Sec. 5.3.7] requires that the modal base shear be compared with the ELF base shear computed using a period somewhat larger than the approximate fundamental period (C_uT_a). Per Sec. 9.2.4.2, T_a = 0.338 sec. and per *Provisions* Table 5.4.2 [Table 5.2-1] C_u = 1.4. Thus, C_uT_a = 0.48 sec., which is less that S_{D1}/S_{DS}. Therefore, the ELF base shear for comparison is 666 kips as just computed. Because 85 percent of 666 kips = 566 kips, *Provisions* Sec. 5.5.7 [Sec. 5.3.7] dictates that all the results of the modal analysis be factored by:

$$\frac{0.85V_{ELF}}{V_{Modal}} = \frac{566}{458} = 1.24$$

Both analyses will be carried forward as discussed in the subsequent sections.

Table 9.2-12 Birmington 2 Periods, Mass Participation Factors, and Modal Base Shears in the Transverse Direction for Modes Used in Analysis

Mode number	Period, (seconds)	Individual mode (percent) Long.	Individual mode (percent) Trans.	Individual mode (percent) Vert.	Cumulative sum (percent) Long.	Cumulative sum (percent) Trans.	Cumulative sum (percent) Vert.	Trans. base shear
1	0.2467	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	0.1919	0.00	70.18	0.00	0.00	70.18	0.00	451.1
3	0.1915	70.55	0.00	0.00	70.55	70.18	0.00	0.0
4	0.0579	0.00	18.20	0.00	70.55	88.39	0.00	73.9
5	0.0574	17.86	0.00	0.00	88.41	88.39	0.00	0.0
6	0.0535	0.00	4.09	0.00	88.41	92.48	0.00	16.1
7	0.0532	4.17	0.00	0.00	92.58	92.48	0.00	0.0
8	0.0413	0.00	0.01	0.00	92.58	92.48	0.00	0.0
9	0.0332	1.50	0.24	0.00	94.08	92.72	0.00	0.8
10	0.0329	0.30	2.07	0.00	94.38	94.79	0.00	7.1
11	0.0310	1.28	0.22	0.00	95.66	95.01	0.00	0.8
12	0.0295	0.22	1.13	0.00	95.89	96.14	0.00	3.8
13	0.0253	1.97	0.53	0.00	97.86	96.67	0.00	1.7
14	0.0244	0.53	1.85	0.00	98.39	98.52	0.00	5.9
15	0.0190	1.05	0.36	0.00	99.44	98.89	0.00	1.1
16	0.0179	0.33	0.94	0.00	99.77	99.82	0.00	2.8
17	0.0128	0.19	0.07	0.00	99.95	99.90	0.00	0.2
18	0.0105	0.03	0.10	0.00	99.99	99.99	0.00	0.3

1 kip = 4.45 kN.

9.2.6.3 Birmingham 2 Vertical Distribution of Seismic Forces

The dynamic analysis will be revisited for the horizontal distribution of forces in the next section but as demonstrated there, the ELF procedure is considered adequate to account for the torsional behavior in this example. The dynamic analysis can certainly be used to deduce the vertical distribution of forces. This analysis was constructed to study amplification of accidental torsion. It would be necessary to integrate the shell forces to find specific story forces, and it is not necessary to complete the design. Therefore, the vertical distribution of seismic forces for the ELF analysis is determined in accordance with *Provisions* Sec. 5.4.4 [Sec. 5.2.3], which was described in Sec. 9.2.4.3. For *Provisions* Eq. 5.4.3-2 [Sec. 5.2-11], k = 1.0 since T = 0.338 sec (similar to the Birmingham 1 and New York City buildings). It should be noted that the response spectrum analysis may result in moments that are less than those calculated using the ELF method; however, because of its relative simplicity, the ELF is used in this example.

Application of the *Provisions* equations for this building is shown in Table 9.2-13:

Table 9.2-13 Birmingham 2 Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	w_xh_x (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	892	43.34	38,659	0.3045	203	203	1,760
4	1,019	34.67	35,329	0.2782	185	388	5,124
3	1,019	26.00	26,494	0.2086	139	527	9,693
2	1,019	17.33	17,659	0.1391	93	620	15,068
1	1,019	8.67	8,835	0.0695	46	666	20,843
∑	4,968		126,976	1.000	666		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m.

9.2.6.4 Birmingham 2 Horizontal Distribution of Forces

For the ELF analysis, this is the same as that for New York City location; see Sec. 9.2.5.4.

Total shear in wall type D:

$$V_{tot} = 0.125V + 1.365(0.0238)V = 0.158V = 104.9\text{kips}$$

The dynamic analysis shows that the fundamental mode is a pure torsional mode. The fact that the fundamental mode is torsional does confirm, to an extent, that the structure is torsionally sensitive. This modal analysis does not show any significant effect of the torsion, however. The pure symmetry of this structure is somewhat idealistic. Real structures usually have some real eccentricity between mass and stiffness, and dynamic analysis then yields coupled modes, which contribute to computed forces.

The *Provisions* does not require that the accidental eccentricity be analyzed dynamically. For illustration, however, this was done by adjusting the mass of the floor elements to generate an eccentricity of 5 percent of the 152-ft length of the building. Table 9.2-14 shows the results of such an analysis. (Accidental torsion could also be considered using a linear combination of the dynamic results and a statically applied moment equal to the accidental torsional moment.)

The transverse direction base shear from the SRSS combination of the modes is 403.8 kips, significantly less than the 457.6 kips for the symmetric model. The amplification factor for this base shear is 566/404 = 1.4. This smaller base shear from modal analysis of a model with an artificially introduced eccentricity is normal. For two primary reasons. First, the mass participates in more modes. The participation in the largest mode is generally less, and the combined result is dominated by the largest single mode. Second, the period for the fundamental mode generally increases, which will reduce the spectral response except for structures with short periods (such as this one).

The base shear in Wall D was computed by adding the in-plane reactions. For the symmetric model the result was 57 kips, which is 12.5 percent of the total of 458 kips, as would be expected. Amplifying this by the 1.24 factor yields 71 kips The application of a static horizontal torsion equal to the 5 percent eccentricity times a base shear of 566 kips (the “floor”) adds 13 kips, for a total of 84 kips. If the static horizontal torsion is amplified by 1.365, as found in the analysis for the New York location, the total becomes 89 kips, which is less than the 99 kips and 105 kips computed in the ELF analysis without and with, respectively, the amplification of accidental torsion. The Wall D base shear from the eccentric model was 66 kips; with the amplification of base shear = 1.4, this becomes 92 kips. Note that this value is less than the direct shear from the symmetric model plus the amplified static torsion. The obvious conclusion is that more careful consideration of torsional instability than actually required by the

Provisions does not indicate any more penalty than already given by the procedures for the ELF in the *Provisions*. Therefore the remainder of the example designs for this building are completed using the ELF.

Table 9.2-14 Birmingham periods, Mass Participation Factors, and Modal Base Shears in the Transverse Direction for Modes Used in Analysis

Mode	Period	Individual mode (percent)			Cumulative sum (percent)			Trans.
Number	(sec)	Long.	Trans.	Vert.	Long.	Trans.	Vert.	Base Shear
1	0.2507	0.0	8.8	0.0	0.0	8.8	0.0	56.3
2	0.1915	70.5	0.0	0.1	70.5	8.8	0.1	0.0
3	0.1867	0.0	61.4	0.0	70.5	70.2	0.1	394.9
4	0.0698	0.0	2.9	0.0	70.5	73.1	0.1	12.7
5	0.0613	1.1	0.0	23.0	71.6	73.1	23.1	0.0
6	0.0575	19.2	0.0	0.0	90.9	73.1	23.2	0.0
7	0.0570	0.0	13.7	0.0	90.9	86.8	23.2	55.5
8	0.0533	0.0	5.6	0.0	90.9	92.4	23.2	22.0
9	0.0480	1.2	0.0	12.8	92.0	92.4	35.9	0.0
10	0.0380	1.4	0.0	0.0	93.5	92.4	35.9	0.0
11	0.0374	0.0	0.4	0.0	93.5	92.8	35.9	1.3
12	0.0327	1.7	0.0	0.2	95.2	92.8	36.1	0.0
13	0.0322	0.0	3.1	0.0	95.2	95.9	36.1	10.4
14	0.0263	2.8	0.0	0.1	98.0	95.9	36.2	0.0
15	0.0243	0.0	3.0	0.0	98.0	98.8	36.2	9.5
16	0.0201	1.6	0.0	0.1	99.6	98.8	36.3	0.0
17	0.0164	0.0	1.1	0.0	99.6	100.0	36.3	3.4
18	0.0141	0.4	0.0	0.1	100.0	100.0	36.3	0

The total story shear and overturning moment (from the ELF analysis) may now be distributed to Wall D and the wall proportions checked. The wall capacity will be checked before considering deflections.

The “extreme torsional irregularity” has an additional consequence for Seismic Design Category D: *Provisions* 5.6.2.4.2 [Sec. 4.6.3.2] requires that the design forces for connections between diaphragms, collectors, and vertical elements (walls) be increased by 25 percent above the diaphragm forces given in *Provisions* 5.4.1 [Sec. 4.6.3.4]. For this example, the diaphragm of precast elements is designed using the different requirements of the appendix to *Provisions* Chapter 9 (see Chapter 7 of this volume).

9.2.6.5 Birmingham 2 Transverse Wall (Wall D)

The design demands are slightly smaller than for the New York City design, yet there is more reinforcement, both vertical and horizontal in the walls. This illustration will focus on those items where the additional reinforcement has special significance.

9.2.6.5.1 Birmingham 2 Shear Strength

Refer to Sec. 9.2.5.5.1 for most quantities. The additional horizontal reinforcement raises V_s and the additional grouted cells raises A_n and, therefore both V_m and V_n *max*.

A_V/s = (4)(0.31 in.^{2})/(8.67 ft.) = 0.1431 in.^{2}/ft
V_s = 0.5(0.1431)(60 ksi)(32.67 ft) = 140.2 kips
A_n = (2 × 1.25 in. × 32.67 ft x 12 in.) + (41 in.^{2} × 12 cells) = 1,472 in.^{2}

The shear strength of Wall D is summarized in Table 9.2-15 below. (Note that V_x and M_x in this table are values from Table 9.2-13 multiplied by 0.158, the portion of direct and torsional shear assigned to the wall). Clearly, the dynamic analysis would make it possible to design this wall for smaller forces, but the minimum configuration suffices. The 1.96 multiplier on V_x to determine V_u is explained in the subsequent section on flexural design.

Table 9.2-15 Shear Strength Calculations for Wall D, Birmingham 2

Level (x)	V_x (kips)	M_x (ft-kips)	M_x/V_xd	$1.98V_x$ (kips)	P (kips)	ϕV_m (kips)	ϕV_s (kips)	ϕV_n (kips)	ϕV_n *max* (kips)
5	32.0	277	0.265	63.4	42	194.6	112.2	306.8	313.9
4	61.1	907	0.454	121	90	186.8	112.2	299	287.3
3	83.0	1527	0.563	164.3	139	186.6	112.2	298.8	272.0
2	97.7	2373	0.743	193.4	188	179.7	112.2	291.9	246.7
1	104.9	3283	0.958	207.7	236	169.6	112.2	281.8	216.6

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

Note that V_n *max* is less than V_n at all levels except the top story. The capacity is greater than the demand at all stories, therefore, the design is satisfactory for shear.

9.2.6.5.2 Birmingham 2 Axial and Flexural Strength

Once again, the similarities to the design for the New York City location will be exploited. Normally, the in-plane calculations include:

1. Strength check
2. Ductility check

9.2.6.5.2.1 Strength check

The wall demands, using the load combinations determined previously, are presented in Table 9.2-16 for Wall D. In the table, Load Combination 1 is $1.29D + Q_E + 0.5L$ and Load Combination 2 is $0.81D + Q_E$.

Table 9.2-16 Birmingham 2 Demands for Wall D

Level	P_D (kips)	P_L (kips)	Load Combination 1		Load Combination 2	
			P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
5	43	0	55	277	36	277
4	94	8	125	807	76	807
3	145	17	196	1527	117	1527
2	196	25	265	2373	159	2373
1	247	34	336	3283	200	3283

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m

Strength at the bottom story (where P, V, and M are the greatest) are less than required for the New York City design. The demands are plotted on Figure 9.2-10, showing that the design for New York City has sufficient axial and flexural capacity for this Birmingham 2 location. For this design, the interaction capacity line will be shifted to the right, due to the presence of additional reinforcing bars. The only calculation here will be an estimate of the factor to develop the flexural capacity at the design axial load.

The flexural capacity for lightly load walls is approximately proportional to the sum of axial load plus the yield of the reinforcing steel:

$$\frac{\text{Birmingham \#2 capacity}}{\text{NewYorkCapacity}} = \frac{200 \text{ kips } + \ 12\times 0.20 \text{ in.}^2 \times 60 \text{ ksi}}{199 \text{ kips } + \ 9\times 0.20 \text{ in.}^2 \times 60 \text{ ksi}} = \frac{344}{307} = 1.12$$

Therefore the factor by which the walls shears must be multiplied to represent 125 percent of flexural capacity, given that the factor was 1.58 for the New York design is:

$$1.58\times 1.12\frac{\text{New York base shear}}{\text{Birmingham \#2 base shear}} = 1.77\times\frac{746}{666} = 1.98$$

9.2.6.5.2.2 Ductility check

The *Provisions* requirements for ductility are described in Sec. 9.2.4.5.2 and 9.2.5.5.2. Since the wall reinforcement and loads are so similar to those for the New York City building, the computations are not repeated here.

[Refer to Sec. 9.2.4.5.2 for discussion of revisions to the ductility requirements in the 2003 *Provisions*.]

9.2.6.6 Birmingham 2 Deflections

The calculations for deflection would be very similar to that for the New York City location. Ironically, that procedure will indicate that the wall is not cracked at the design load. The C_d factor is larger, 3.5 vs. 2.25. However, the calculation is not repeated here; refer to Sec. 9.2.4.6 and Sec. 9.2.5.6.

[Refer to Sec. 9.2.4.6 for discussion of revisions to the deflection computations and requirements in the 2003 *Provisions*, as well as the potentially conflicting drift limits.]

9.2.6.7 Birmingham 2 Out-of-Plane Forces

Provisions Sec. 5.2.6.2.7 [Sec. 4.6.1.3] requires that the bearing walls be designed for out-of-plane loads determined:

$w = 0.40\, S_{DS}\, W_c \geq 0.1W_c$
$w = (0.40)(0.47)(56 \text{ psf}) = 10.5 \text{ psf} \geq 0.1W_c$

The calculated seismic load, w = 10.5 psf, is less than wind pressure for exterior walls. Even though Wall D is not an exterior wall, the lateral pressure is sufficiently low that it is considered acceptable by inspection without further calculation. Seismic loads do not govern the design of Wall D for loading in the out-of-plane direction.

9.2.6.8 Birmingham 2 Orthogonal Effects

According to *Provisions* Sec. 5.2.5.2.2 [Sec. 4.4.2.3], orthogonal interaction effects have to be considered for Seismic Design Category D when the ELF procedure is used (as it is here). However, the out-of-plane component of only 30 percent of 10.5 psf on the wall will not produce a significant effect when combined with the in-plane direction of loads so no further calculation will be made.

This completes the design of the Transverse Wall D.

9.2.6.9 Birmingham 2 Summary of Wall Design for Wall D

8-in. CMU
f'_m = 2,000 psi

Reinforcement:

12 vertical #4 bars per wall (spaces alternate at 32 and 40 in. on center)
Two bond beams with 2 - #5 at each story, at bearing for the planks, and at 4 ft above each floor.
Horizontal joint reinforcement at alternate courses is recommended, but not required.

9.2.7 Seismic Design for Los Angeles

Once again, the differences from the designs for the other locations will be emphasized. As explained for the Birmingham 2 building, the *Provisions* would require a dynamic analysis for design of this building. For the reasons explained in Sec. 9.2.6.4, this design is illustrated using the ELF procedure.

9.2.7.1 Los Angeles Weights

Use 91 psf for 8-in.-thick, normal weight hollow core plank, 2.5 in. lightweight concrete topping (115 pcf), plus the nonmasonry partitions. This building is Seismic Design Category D, and the walls will be designed as special reinforced masonry shear walls (*Provisions* Sec. 11.11.5 and Sec. 11.3.8 [Sec.11.2.1.5]), which requires prescriptive seismic reinforcement (*Provisions* Sec. 11.3.8.3 [ACI 530, Sec. 1.13.2.2.5]). Special reinforced masonry shear walls have a minimum spacing of vertical reinforcement of 4 ft on center. For this example, 60 psf weight for the 8-in. CMU walls will be assumed. The 60 psf value includes grouted cells and bond beams in the course just below the floor planks and in the course 4 ft above the floors. A typical wall section is shown in Figure 9.2-12.

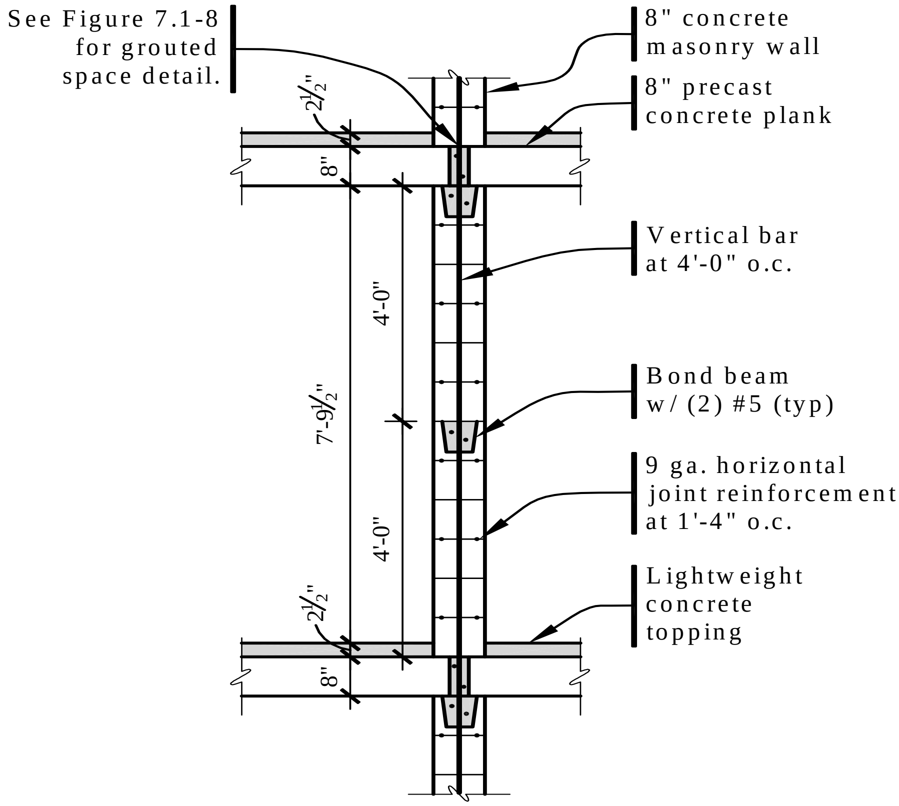


Figure 9.2-12 Typical wall section for the Los Angeles location (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

Story weight ,w_i:

Roof weight:

Roof slab (plus roofing) = (91 psf) (152 ft)(72 ft)	=	996 kips
Walls = (60 psf)(589 ft)(8.67 ft/2) + (60 psf)(4)(36 ft)(2 ft)	=	170 kips
Total	=	1,166 kips

There is a 2-ft-high masonry parapet on four walls and the total length of masonry wall is 589 ft.

Typical floor:

Slab (plus partitions)	=	996 kips
Walls = (60 psf)(589 ft)(8.67 ft)	=	306 kips
Total	=	1,302 kips

Total effective seismic weight, W = 1,166 + (4)(1,302) = 6,374 kips

This total excludes the lower half of the first story walls, which do not contribute to seismic loads that are not imposed on the CMU shear walls.

9.2.7.2 Los Angeles Base Shear Calculation

The seismic response coefficient, C_s, is computed using *Provisions* Eq. 5.4.1.1-1 [Eq. 5.2-2] and 5.4.1.1-2 [Eq. 5.2-3]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.00}{3.5/1} = 0.286 \qquad \text{Controls}$$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.60}{0.338(3.5/1)} = 0.507$$

where T is the fundamental period of the building, which is 0.338 sec as computed previously (the approximate period, based on building system and building height, will be the same for all locations). The value for C_s is taken as 0.286 (the lesser of these two). This value is still larger than the minimum specified in *Provisions* Eq. 5.3.2.1-3 which is:

$C_s = 0.044 S_{D1} I = (0.044)(0.60)(1) = 0.026$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated *Provisions* Eq. 5.4.1 [Eq.5.2-1]:

$V = C_s W = (0.286)(6{,}374) = 1{,}823$ kips

9.2.7.3 Los Angeles Vertical Distribution of Seismic Forces

The vertical distribution of seismic forces is determined in accordance with *Provisions* Sec. 5.4.4 [Sec. 5.2.3], which as described in Sec. 9.2.4.3. Note that for *Provisions* Eq. 5.4.3-2 [Eq. 5.2-11], $k = 1.0$ since $T = 0.338$ sec (similar to the previous example buildings).

The application of the *Provisions* equations for this building is shown in Table 9.2-17:

Table 9.2-17 Los Angeles Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	1,166	43.34	50,534	0.309	564	564	4,890
4	1,302	34.67	45,140	0.276	504	1,608	14,150
3	1,302	26.00	33,852	0.207	378	1,446	26,686
2	1,302	17.33	22,564	0.138	252	1,698	41,409
1	1,302	8.67	11,288	0.069	126	1,824	57,222
$\sum$	6,374		163,378	1.000	1,824		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m

9.2.7.4 Los Angeles Horizontal Distribution of Forces

This is the same as for the Birmingham 2 design; see Sec. 9.2.6.4.

Total shear in Wall Type D:

$$V_{tot} = 0.125V + 1.365(0.0238)V = 0.158V$$

The total story shear and overturning moment may now be distributed to each wall and the wall proportions checked. The wall capacity will be checked before considering deflections.

9.2.7.5 Los Angeles Transverse Wall D

The strength or limit state design concept is used in the *Provisions*.

9.2.7.5.1 Los Angeles Shear Strength

The equations are the same as for the prior locations for this example building. Looking forward to the design for flexural and axial load, the amplification factor on the shear is computed as:

$$1.25\frac{M_n}{M_u} = 1.25\frac{9156/0.85}{9012} = 1.49 \quad \text{(which is less than the 2.5 upper bound)}$$

Therefore, the demand shear is 1.49 times the value from analysis. (This design continues to illustrate the ELF analysis and; as explained for the Birmingham 2 design, smaller demands could be derived from the dynamic analysis.) All other parameters are similar to those for Birmingham 2 except that:

$$A_n = (2 \times 1.25 \text{ in.} \times 32.67 \text{ ft x } 12 \text{ in.}) + (41 \text{ in.}^2 \times 15 \text{ cells}) = 1{,}595 \text{ in.}^2$$

The shear strength of each Wall D, based on the aforementioned formulas and data, are summarized in Table 9.2-18.

Table 9.2-18 Los Angeles Shear Strength Calculations for Wall D

Story	V_x (kips)	M_x (ft-kips)	M_x/V_xd	$1.49V_x$ (kips)	P (kips)	ϕV_m (kips)	ϕV_s (kips)	ϕV_n (kips)	ϕV_n *max* (kips)
5	88.8	770	0.265	132.3	42	210.1	112.2	322.3	340
4	168.2	2229	0.406	250.6	90	205.7	112.2	317.9	318.7
3	227.7	4203	0.565	339.3	139	199.6	112.2	311.8	294.5
2	267.4	6522	0.747	398.4	188	191.3	112.2	303.8	266.8
1	287.2	9012	0.960	427.9	236	179.5	112.2	291.7	234.3

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m

Just as for the Birmingham 2 design, the maximum on V_n controls over the sum of V_m and V_s at all stories except the top. Unlike the prior design, the shear capacity is inadequate in the lower three stories. The solution is to add grout. At the first story, solid grouting is necessary:

$$A_n = (7.625 \text{ in.})(32.67 \text{ ft.})(12 \text{ in./ft.}) = 2989 \text{ in.}^2$$
$$\varphi V_n\ max = 0.8(4.11)(0.0447 \text{ ksi})(2989 \text{ in.2}) = 439 \text{ kips} > 428 \text{ kips} \qquad \text{OK}$$

At the third story, six additional cells are necessary, and at the second story, approximately two out of three cells must be grouted. The additional weight adds somewhat to the demand but only about 2 percent. If the entire building were grouted solid (which would be common practice in the hypothetical location), the weight would increase enough that the shear strength criterion might be violated.

9.2.7.5.2 Los Angeles Axial and Flexural Strength

The basics of the flexural design have been demonstrated for the previous locations. The demand is much higher at this location, however, which introduces issues about the amount and distribution of reinforcement in excess of the minimum requirements. Therefore, the strength and ductility checks will both be examined.

9.2.7.5.2.1 Strength check

Load combinations, using factored loads, are presented in Table 9.2-19 for Wall D. In the table, Load Combination 1 is 1.4D + Q_E + 0.5L, and Load Combination 2 is 0.7D + Q_E.

Table 9.2-19 Los Angeles Load Combinations for Wall D

			Load Combination 1		Load Combination 2	
Level (x)	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
5	63	0	88	770	44	770
4	126	8	180	2229	88	2229
3	189	17	273	4203	132	4203
2	251	25	364	6522	176	6522
1	314	34	456	9012	220	9012

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m

Strength at the bottom story (where P, V, and M are the greatest) is examined. This example considers Load Combination 2 from Table 9.2.19 to be the governing case, because it has the same lateral load as Load Combination 1 but lower values of axial force.

Refer to Figure 9.2-13 for notation and dimensions.

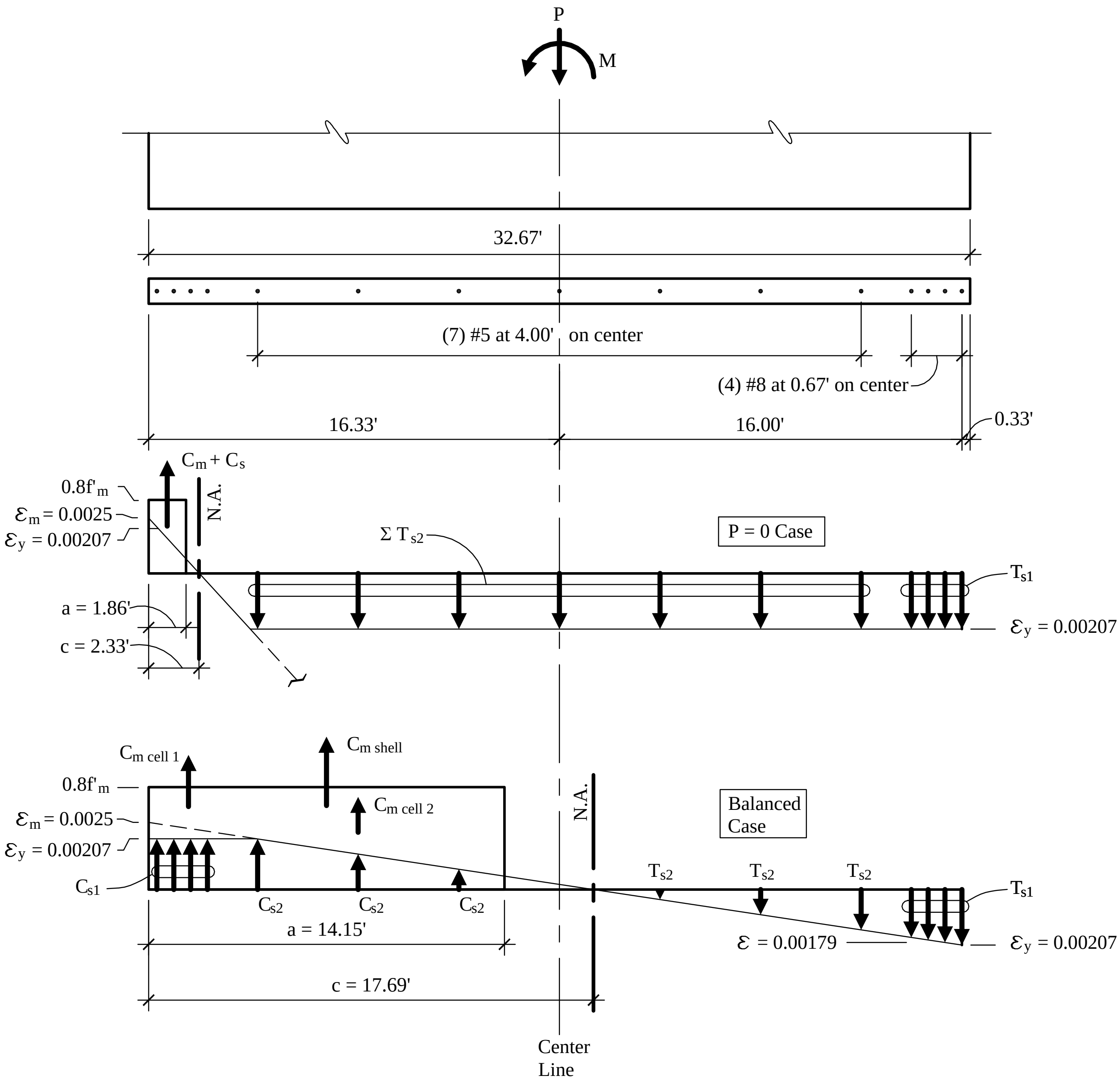


Figure 9.2-13 Los Angeles: Strength of wall D (1.0 ft = 0.3048 m).
Strength diagrams superimposed on strain diagrams for the two cases.

Examine the strength of Wall D at Level 1:

$P_{u_{min}}$ = 220 kips

$P_{u_{max}}$ = 456 kips

M_u = 9,012 ft-kips

Because special reinforced masonry shear walls are used (Seismic Design Category D), vertical reinforcement at 4 ft. on center and horizontal bond beams at 4 ft on center are prescribed (*Provisions* Sec. 11.3.7.3 [ACI 530, Sec. 1.13.2.2.5]). (Note that the wall is 43.33 ft high, not 8 ft high, for purposes of determining the maximum spacing of vertical and horizontal reinforcement.)

For this bending moment, the minimum vertical reinforcement will not suffice. For reinforcement uniformly distributed, a first approximation could be taken from a simple model using an effective internal moment arm of 80 percent of the overall length of the wall:

$$A_s = \frac{M - 0.8Pl/2}{60(\text{ksi})(0.8l/2)} = \frac{9012 - 0.8 \times 220 \times 32.67/2}{60 \times 0.8 \times 32.67/2} = 7.8 \text{ in}^2 = 0.24 \text{ in}^2\text{/ft.}$$

The minimum vertical steel is 0.0007 times the gross area, which is 0.064 in.^{2}/ft. At the maximum spacing of 4 ft, a #5 bar is slightly above the minimum. Experimental evidence indicates that uniformly distributed reinforcement will deliver good performance. This could be accomplished with a #9 bar at 48 in. or a #8 bar at 40 in. This design will work well in a wall that is solidly grouted; however, for walls that are grouted only at cells containing reinforcement, it will be found that this wall fails the ductility check (which can be remedied by placing several extra grouted cells near each end of the wall as was shown in Sec. 9.1.5.4). The flexural design was completed before the shear design (described in the previous section) discovered the need for solid grout in the first story. The remainder of this flexural design check is carried out without consideration of the added grout. (It is unlikely that the interaction line will be affected near the design points, but the balanced point will definitely change.)

It has long been common engineering practice to concentrate flexural reinforcement near the ends of the wall. (This a normal result of walls that intersect to form flanges with reinforcement in both web and flange.) For this design, if one uses the minimum #5 bar at 48 inches, then the extra steel at the ends of the walls is approximately:

$$A_{s\ end} = (7.8 - 7 \times 0.31)/2 = 2.8 \text{ in}^2$$

Try #8 bars in each of the first four end cells and #5 bars at 4 ft on center at all intermediate cells.

The calculation procedure is similar to that presented in Sec. 9.2.4.5.2. The strain and stress diagrams are shown in Figure 9.2-13 for the Birmingham 1 building and the results are as follows:

P = 0 case

$\phi P_n = 0$
ϕM_n = 6,636 ft-kips

Intermediate case, setting *c* = 4.0 ft

ϕP_n = 223 kips
ϕM_n = 9,190 ft-kips

Balanced case

ϕP_n = 1049 kips
ϕM_n = 14,436 ft-kips

The simplified ϕP_n - ϕM_n curve is shown in Figure 9.2-14 and indicates the design with #8 bars in the first four end cells and #5 bars at 4 ft on center throughout the remainder of the wall is satisfactory.

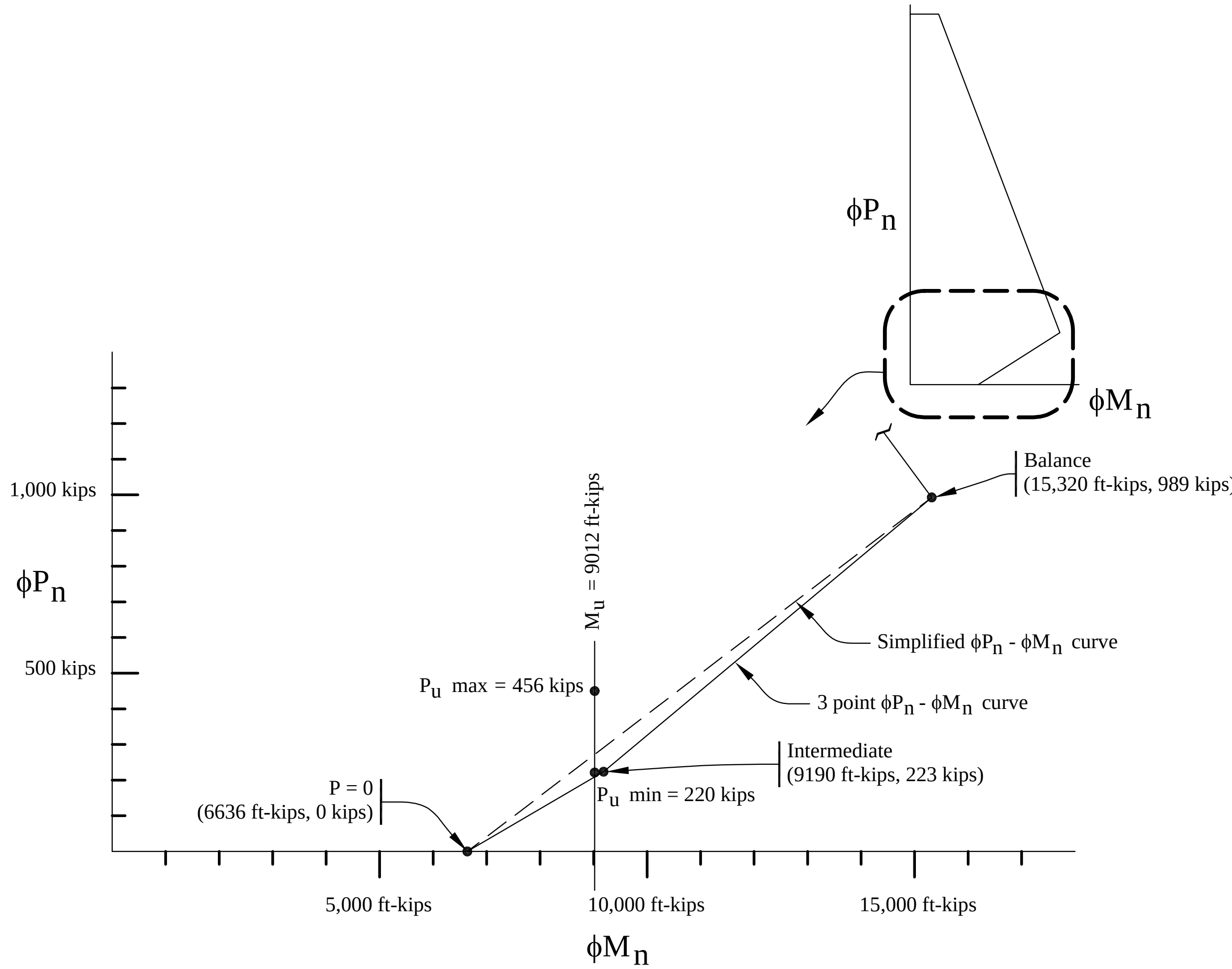


Figure 9.2-14 φP_{11} - φM_{11} diagram for Los Angeles Wall D (1.0 kip = 4.45 kN, 1.0 kip-ft = 1.36 kN-m).

9.2.7.5.2.2 Ductility check

Provisions Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5] has been illustrated in the prior designs. Recall that this calculation uses unfactored gravity axial loads (*Provisions* Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5]). Refer to Figure 9.2-15 and the following calculations which illustrate this using loads at the bottom story (highest axial loads). The extra grout required for shear is also ignored here. More grout gives higher compression capacity, which is conservative.

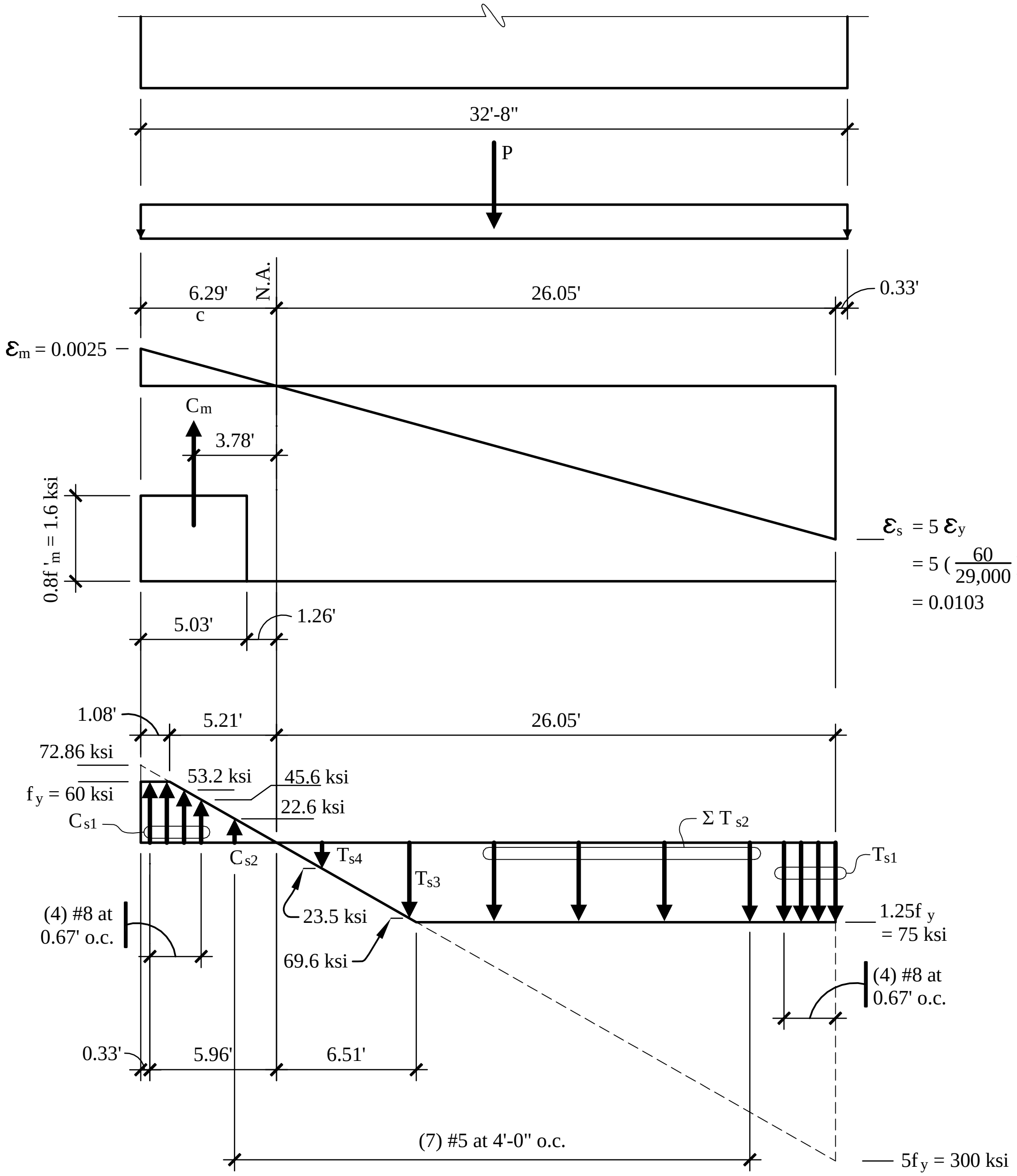


Figure 9.2-15 Ductility check for Los Angeles Wall D (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa)

For Level 1 (bottom story), the unfactored loads are:

P = 314 kips
$C_m = 0.8f'_m[(a)(b) + A_{cells}]$

where b = flange width = (2 × 1.25 = 2.5 in.) and A_{cells} = 41 in.^{2}

C_m = (1.6 ksi)[(5.03 ft × 12)(2.5 in.) + (5 cells)(41)] = 569.4 kips
C_{s1} = 0.79(2 x 60 + 53.2 + 45.6) = 172.9 kips
C_{s2} = (22.6 ksi)(0.31 in.^{2}) = 7.0 kips

$\sum C$ = 749 kips
$\sum T_{s1}$ = (4 × 0.79 in.^{2})(75 ksi) = 237 kips
$\sum T_{s2}$ = (4 × 0.31 in.^{2})(75 ksi) = 93.0 kips
T_{s3} = (0.31 in.^{2})(69.6 ksi) = 21.6 kips
T_{s4} = (0.31 in.^{2})(23.5 ksi) = 7.3 kips
$\sum T$ = 359 kips

$\sum C > \sum P + T$
749 kips > 673 kips

If a solution with fully distributed reinforcement were used, the tension from reinforcement would increase while the compression from grout at the end of the wall, as well as compression of steel at the compression would also decrease. The criterion would not be satisfied. Adding grout would be required.

[Refer to Sec. 9.2.4.5.2 for discussion of revisions to the ductility requirements in the 2003 *Provisions*.]

9.2.7.6 Los Angeles Deflections

Recall the assertion that the calculations for deflection involve many variables and assumptions and that any calculation of deflection is approximate at best. The requirements and procedures for computing deflection are provided in Sec. 9.2.4.6. [Refer to Sec. 9.2.4.6 for discussion of revisions to the deflection computations and requirements in the 2003 *Provisions*, as well as the potentially conflicting drift limits.]

For the Los Angeles building, the determination of whether the walls will be cracked is as follows:

b_e = effective masonry wall width
b_e = [(2 × 1.25 in.)(32.67 ft × 12) + (15 cells)(41 in.^{2}/cell)]/32.67 ft × 12) = 4.07 in.
$A = b_e\, l$ = (4.07 in.)(32.67 × 12) = 1595 in.^{2}
$S = b_e\, l^2/6$ = (7.07)(32.67 × 12)^2/6 = 104,207 in.^{3}
f_r = 0.250 ksi

P_u is calculated using 1.00D (See Table 9.2-18 for values, and refer to Sec. 9.2.4.6 for discussion). Table 9.2-20 provides a summary of these calculations. (The extra grout required for shear strength is also not considered here; the revision would slightly reduce the computed deflections by raising the cracking moment.)

Table 9.2-20 Los Angeles Cracked Wall Determination

Level	$P_{u_{min}}$ (kips)	M_{cr} (ft-kips)	M_x (ft-kips)	Status
5	63	2514	770	uncracked
4	126	2857	2229	uncracked
3	189	3200	4203	cracked
2	251	3538	6522	cracked
1	314	3880	9012	cracked

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

For the uncracked walls (Levels 4 and 5):

$I_n = I_g = b_e l^3/12$ = (4.07 in.)(32.67 × 12)^3/12 = 2.04 × 10^7 in.^{4}

For the cracked walls, the transformed cross section will computed by classic methods. Assuming the neutral axis to be about 10 ft in from the compression face gives five #5 bars in tension. The tension reinforcement totals:

A_s = 4(0.79) + 5(0.31) = 3.16 + 1.55 = 4.71 in.^{2}

The axial compression stiffens the wall. The effect is approximated with an equivalent area of tension reinforcement equal to half the compression. Thus, the total reinforcement becomes:

A_{se} = 4.71 + 0.5(314)/60 = 4.71 + 2.62 = 7.33 in.^{2}

The centroid of this equivalent reinforcement is 29.5 ft from the compression face. Following the classic method for transformed cracked cross sections and with n = 19.3:

ρ = 7.33/(4.04 x 29.5 x 12) = 0.0051
ρn = 0.0051(19.3) = 0.099
k = sqrt(ρn^2 + 2ρn) - ρn = 0.36
$kd = c$ = 10.5 feet (which is close enough to the assumed 10 feet)

$I_{cr} = bc^3/3 + \sum nA_s d^2$ = 4.04(29.5 x 12)^3 + 19.3(7.33)(29.5 - 10.5)^2(144) = 1.01 x 10^7 in.^{4}

The *Provisions* encourages the use of the cubic interpolation formula illustrated for the previous locations. For the values here, this yields I_{eff} = 1.09 x 10^7 in.^{4}, which is about half the gross moment of inertia (which in itself is not a bad approximation for a cracked and well reinforced cross section). For this example, the deflection computation will instead use the cracked moment of inertia in the lower three stories and the gross moment of inertia in the upper two stories. The results from a RISA 2D analysis are shown in Table 9.2-21, and are about 5 percent higher than use of I_{eff} over the full height.

Table 9.2-21 Los Angeles Deflections

Level	F (kips)	I_{eff} (in.^{4})	$\delta_{flexural}$ (in.)	δ_{shear} (in.)	δ_{total} (in.)	$C_d\,\delta_{total}$ (in.)	Δ (in.)
5	84.0	2.04 × 10^7	0.491	0.127	0.618	2.163	0.455
4	75.1	2.04 × 10^7	0.370	0.118	0.488	1.708	0.543
3	56.3	1.01 × 107	0.237	0.096	0.333	1.166	0.515
2	37.6	1.01 × 10^7	0.118	0.068	0.186	0.651	0.413
1	18.8	1.01 × 10^7	0.033	0.035	0.068	0.238	0.238

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm.

The maximum drift occurs at Level 4 per *Provisions* Table 5.2.8 is:

Δ = 0.543 in. < 1.04 in. = 0.01 h_n (*Provisions* Table 5.2.8 [Table 4.5-1]) OK

9.2.7.7 Los Angeles Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4.6.1.3] requires that the bearing walls be designed for out-of-plane loads determined as follows:

w = 0.40 S_{DS} W_c ≥ 0.1W_c
w = (0.40)(1.00)(60 psf) = 24psf ≥ 0.1W_c

The out-of-plane bending moment, using the strength design method for masonry, for a pressure, w = 24 psf and considering the P-delta effect, is computed to be 2,232 in.-lb/ft. This compares to a computed strength of the wall of 14,378 in.-lb/ft, considering only the #5 bars at 4 ft on center. Thus, the wall is loaded to about 16 percent of its capacity in flexure in the out-of-plane direction. (See Sec. 9.1 for a more detailed discussion of strength design of masonry walls, including the P-delta effect.)

9.2.7.8 Los Angeles Orthogonal Effects

According to *Provisions* Sec. 5.2.5.2.2 [Sec. 4.4.2.3], orthogonal interaction effects have to be considered for Seismic Design Category D when the ELF procedure is used (as it is here).

The out-of-plane effect is 16 percent of capacity, as discussed in Sec. 9.2.7.7 above. When considering the 0.3 combination factor, the out-of-plane action adds about 5 percent overall to the interaction effect. For the lowest story of the wall, this could conceivably require a slight increase in capacity for in-plane actions. In the authors' opinion, this is on the fringe of requiring real consideration (in contrast to the end walls of Example 9.1).

This completes the design of the transverse Wall D.

9.2.7.9 Los Angeles Summary of Wall Design for Wall D

8-in. CMU
f'_m = 2,000 psi

Reinforcement:

Four vertical #8 bars, one bar in each cell for the four end cells
Vertical #5 bars at 4 ft on center at intermediate cells
Two bond beams with two #5 bars at each story, at floor bearing and at 4 ft above each floor
Horizontal joint reinforcement at alternate courses recommended, but not required

Grout:

All cells with reinforcement and bond beams, plus solid grout at first story, at two out of three cells in the second story, and at six extra cells in the third story

Table 9.2-22 compares the reinforcement and grout for Wall D designed for each of the four locations.

Table 9.2-22 Variation in Reinforcement and Grout by Location

	Birmingham 1	New York City	Birmingham 2	Los Angeles
Vertical bars	5 - #4	9 - #4	12 - #4	8 - #8 + 7 - #5
Horizontal bars	10 - -#4 + jt. reinf	10 - #4 + jt. reinf	20 - #5	20 - #5
Grout (cu. ft.)	91	122	189	295

1 cu. ft. = 0.0283 m^3.

9.3 TWELVE-STORY RESIDENTIAL BUILDING IN LOS ANGELES, CALIFORNIA

9.3.1 Building Description

This 12-story residential building has a plan form similar to that of the five-story masonry building described in Sec. 9.2. The floor plan and building elevation are illustrated in Figures 9.3-1 and 9.3-2, respectively. The floors are composed of 14-in.-deep open web steel joists spaced at 30 in. that support a 3-in. concrete slab on steel form deck. A fire-rated ceiling is included at the bottom chord of the joists. Partitions, including the shaft openings, are gypsum board on metal studs, and the exterior nonstructural curtain walls are glass and aluminum.

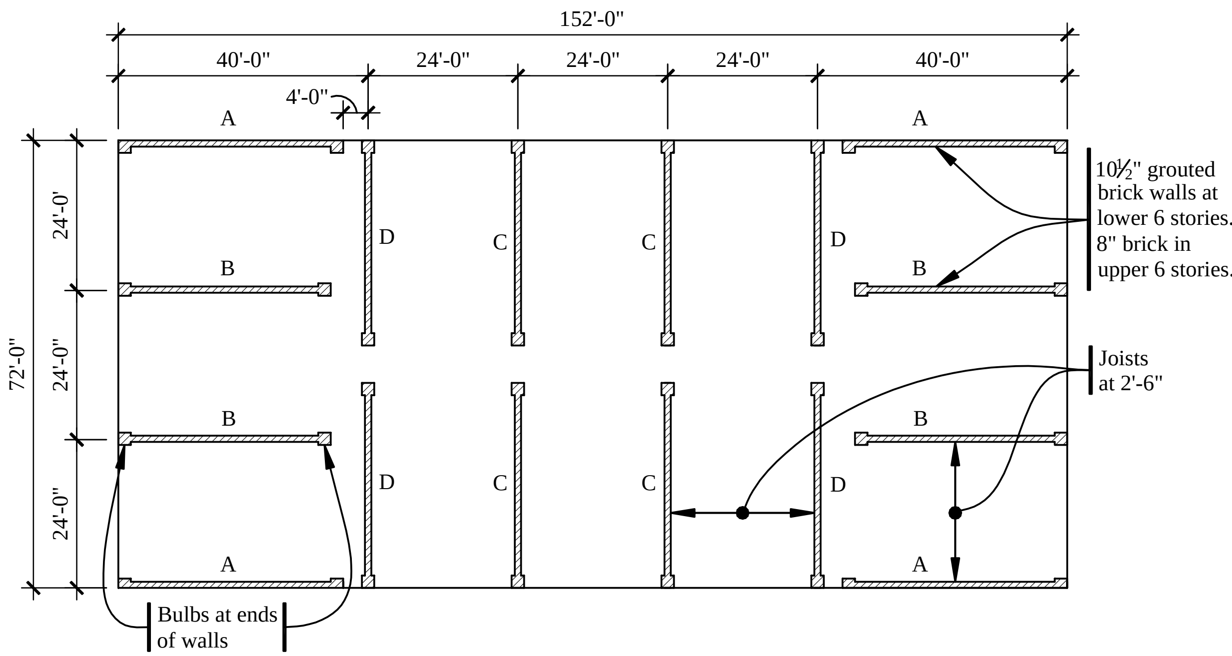


Figure 9.3-1 Floor plan (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

All structural walls are of grouted brick. For purposes of illustration, two styles of wall are included. The lower six stories have 10-1/2in.-thick walls consisting of two wythes of 4-in. (nominal) brick and a 3-1/4-in. grout space. The upper six stories have 8-in. (nominal) brick, hollow unit style, with the vertical reinforcing in the cells and the horizontal reinforcing in bond beams. (In actual construction, however, a single style wall might be used throughout: either a two-wythe grouted wall or a through-the-wall unit of an appropriate thickness). The walls are subject to high overturning moments and have a reinforced masonry column at each end. The column concentrates the flexural reinforcement and increases resistance to overturning. (Similar concentration and strength could be obtained with transverse masonry walls serving as flanges for the shear walls had the architectural arrangement been conducive to this approach.) Although there is experimental evidence of improved performance of walls with all vertical reinforcement uniformly distributed, concentration at the ends is common in engineering practice and the flexural demands are such for this tall masonry building that the concentration of masonry and reinforcement at the ends is simply much more economical.

The compressive strength of masonry, f'_m, used in this design is 2,500 psi for Levels 1 through 6 and 3,000 psi for Levels 7 through 12.

This example illustrates the following aspects of the seismic design of the structure:

1. Development of equivalent lateral forces
2. Reinforced masonry shear wall design
3. Check for building deflection and story drift
4. Check of diaphragm strength.

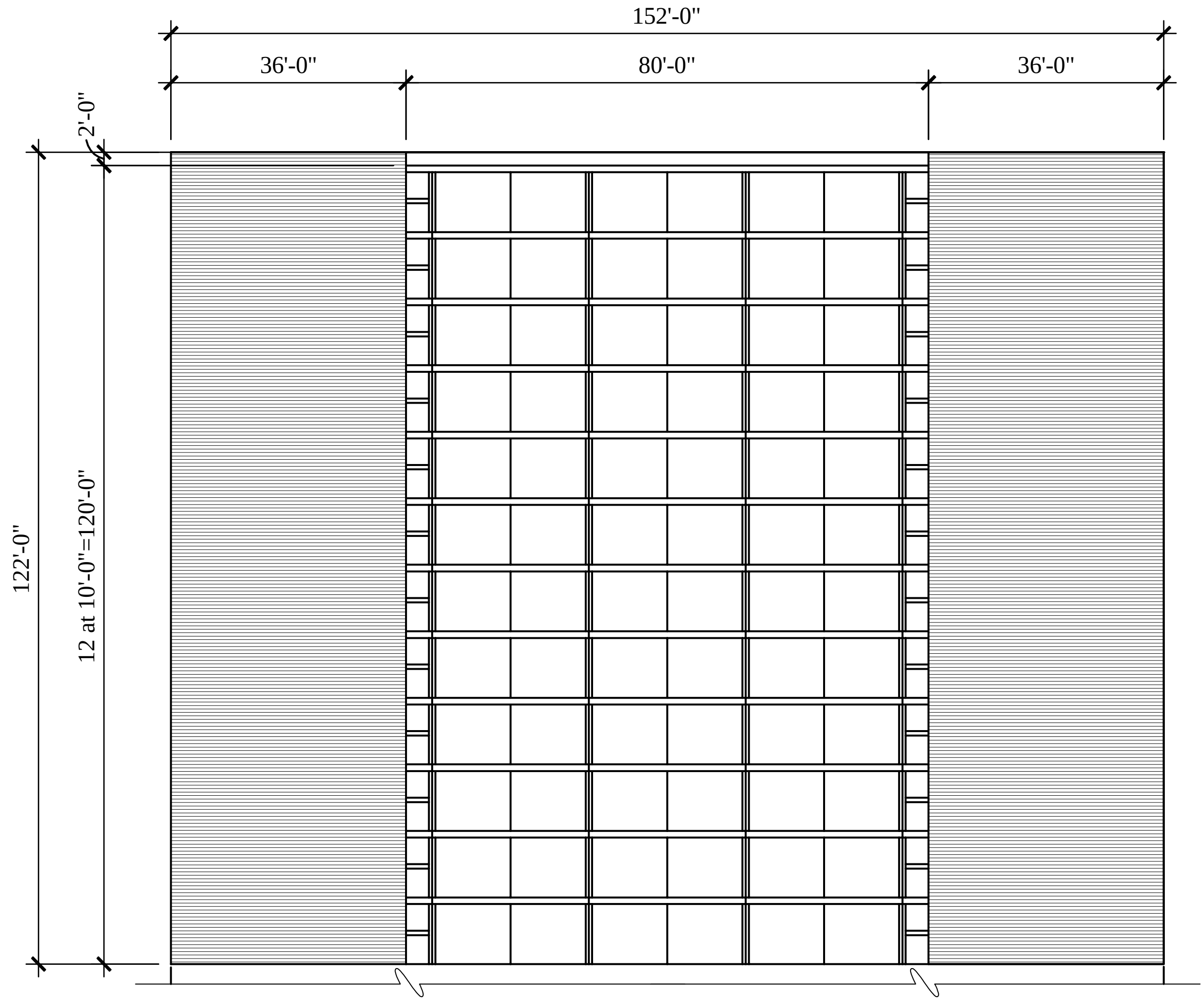


Figure 9.3-2 Elevation (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

9.3.2 Design Requirements

9.3.2.1 *Provisions* Design Parameters

Table 9.3-1 shows the design parameters for building design.

Table 9.3-1 Design Parameters

Design Parameter	Value
S_s (Map 1 [Figure 3.3-3])	1.5
S_1 (Map 2 [Figure 3.3-4])	0.6
Site Class	C
F_a	1
F_v	1.3
$S_{MS} = F_a S_s$	1.5
$S_{M1} = F_v S_1$	0.78
S_{DS} = 2/3 S_{MS}	1
S_{D1} = 2/3 S_{M1}	0.52
Seismic Design Category	D
Masonry Wall Type	Special Reinforced
R	3.5
Ω_0	2.5
C_d	3.5

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

9.3.2.2 Structural Design Requirements

The load path consists of the floors acting as horizontal diaphragms and the walls parallel to the motion acting as shear walls.

Soil-structure interaction is not considered.

The building is a bearing wall system (*Provisions* Table 5.2.2 [4.3-1]) .

Deformational compatibility must be assured (*Provisions* Sec. 5.2.2.4.3 [Sec. 4.5.3]). The structural system is one of non-coupled shear walls. Crossing beams over the halls (their design is not included in this example) will need to continue to support the gravity loads from the floors and roof during an earthquake but will not provide coupling between the shear walls.

The building is symmetric in plan but has the same torsional irregularity described in Sec. 9.2.5.4. The vertical configuration is regular except for the change in wall type between the sixth and seventh stories, which produces a significant discontinuity in stiffness and strength, both for shear and flexure (*Provisions* Sec. 5.2.3.3 [Sec. 4.3.2.3] and 5.2.6.2.3 [Sec. 4.6.1.6]). There is no weak story because the strength does not increase as one goes upward. The stiffness discontinuity will be shown to qualify as regular.

Provisions Table 5.2.5.1 [Table 4.4-1] would not permit the use of the ELF procedure of *Provisions* Sec. 5.4 [Sec. 5.2]; instead a dynamic analysis of some type is required. As will be illustrated, this particular building does not really benefit from this requirement.

The design and detailing must comply with the requirements of *Provisions* Sec. 5.2.6 [Sec. 4.6].

The walls must resist forces normal to their plane (*Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3]). These forces will be used when addressing the orthogonal effects (*Provisions* Sec.5.2.5.2.2 [Sec. 4.4.2.3]).

With eight walls in each direction, the system is expected to be redundant.

Tie and continuity requirements for anchorage of masonry walls must be considered when detailing the connections between floors and walls (*Provisions* Sec. 5.2.6.1.2 [Sec. 4.6.2.1] and 5.2.6.1.3).

Openings in walls and diaphragms need to be reinforced (*Provisions* Sec. 5.2.6.2.2 [Sec. 4.6.1.4]).

Diaphragms need to be designed to comply with *Provisions* Sec. 5.2.6.2.6 [Sec. 4.6.3.4].

The story drift limit is $0.01h_{sx}$ (*Provisions* Sec. 5.2.8 [Sec. 4.5.1]) and the overall drift limit is $0.01h_{sn}$ (*Provisions* Sec. 11.5.4.1). For this structure the difference between these two is significant, as will be shown.

[The deflection limits have been removed from Chapter 11 of the 2003 *Provisions* because they were redundant with the general deflection limits. Based on ACI 530 Sec. 1.13.3.2, the maximum drift for all masonry structures is 0.007 times the story height. Thus, there appears to be a conflict between ACI 530 and 2003 *Provisions* Table 4.5-1.]

9.3.2.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [Sec. 4.2.2]) are the same as those in ASCE 7 except that the seismic load effect, E, is defined by *Provisions* Eq. 5.2.7-1 [Eq. 4.2-1] and 5.2.7-2 [Eq. 4.2-2] as:

$$E = \rho Q_E \pm 0.2S_{DS}D$$

Based on the configuration of the shear walls and the results presented in Sec. 9.2, the reliability factor, ρ, is treated as equal to 1.0 for both directions of loading. Refer to Sec. 9.2.3.1 for additional information.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. The intent is that the aspect ratio is based on story height, not total height. Therefore, the redundancy factor would not have to be investigated (ρ = 1.0) for this building.]

The discussion on load combinations for the Los Angeles site in Sec.9.2 is equally applicable to this example. Refer to Sec. 9.2.3.2 for determination of load combinations.

The load combinations representing the extreme cases are:

$$1.4D + Q_E + 0.5L$$
$$0.7D + Q_E$$

9.3.3 Seismic Force Analysis

The analysis is performed using the ELF procedure of *Provisions* Sec. 5.4 [Sec. 5.2] and checked with a modal response spectrum (MRS) analysis in conformance with *Provisions* Sec. 5.5 [Sec. 5.3]. This example illustrates an analysis for earthquake motions acting in the transverse direction only. Earthquake motions in all directions will need to be addressed for an actual project.

9.3.3.1 Building Weights

For the ELF analysis, the masses are considered to be concentrated at each floor level whereas, for the MRS analysis, it is distributed on both wall and floor elements. Note that the term "level" corresponds to the slab above each story. Thus Level 1 is the second floor; Level 12 is the roof.

Lower Levels:

Slab, joists, partitions, ceiling, mechanical/electrical (M/E), curtain wall at 53 psf
(0.053 ksf)(152 ft)(72 ft) = 580 kips/story

Walls: 10.5 in. at 114 psf (brick at 73 psf + grout at 41 psf)
(0.114 ksf)(10 ft)[(8)(29 ft) + (4)(30 ft) + (4)(32 ft)] = 547 kips/story

Bulbs at ends of walls: (24 in. × 24 in. bulb)
Brick: 2.01 ft^2 /bulb; Grout: 1.99 ft^2/bulb
[(2.01 ft^2)(0.120 kcf) + (1.99 ft^2)(0.150 kcf)] (10 ft)(32 bulbs) = 173 kips/story

Upper Levels:

Slab, joists, partitions, ceiling, M/E, curtain wall at 53 psf
(0.053 ksf)(152 ft)(72 ft) = 580 kips/story

Walls: 8 in. Partially grouted brick at 48 psf
(0.048 ksf)(10 ft)[(8)(29.67 ft) + (4)(30.67 ft) + (4)(32.67 ft)] = 236 kips/story

Bulbs at ends of walls (grouted 20 in. × 20 in. brick bulb)
(0.315 klf/bulb)(10 ft)(32 bulbs) = 101 kips/story

Roof:

Slab, roofing, joists,, ceiling, M&E, curtain wall at 53 psf :
(0.053 ksf)(152 ft)(72 ft) = 580 kips

Walls
(238 kips/story + 101 kips/story)/2 = 170 kips

Parapet
(4 parapets)(2 ft)[(0.048 kips/lf)(33 ft) +(3.15 kips/bulb)(2 bulbs)/(10 ft)] = 18 kips

Preliminary design indicates a 10-1/2-in. wall with bulbs for the six lower stories and an 8-in. wall with bulbs for the six upper stories. Therefore, effective seismic weight, *W*, is computed as follows:

Levels 1-5	(5)(580 + 547 + 173)	= (5)(1,300 kips/level)	= 6,500 kips
Level 6	580 + (547 + 236)/2 + (173 + 101)/2	= 1,109 kips	= 1,109 kips
Levels 7-11	(5)(580 + 236 + 101)	= (5)(917 kips/level)	= 4,585 kips
Level 12 (roof)	(580 + 170 + 18)	= 768 kips	= 768 kips
Total			= 12,962 kips

The weight of the lower half of walls for the first story are not included with the walls for Level 1 because the walls do not contribute to the seismic loads.

9.3.3.2 Base Shear

The seismic coefficient, C_s, for the ELF analysis is computed as:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.00}{3.5/1.0} = 0.286$$

The value of C_s need not be greater than:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.52}{(0.75)(3.5/1)} = 0.198$$

The value of the fundamental period, T, was determined from a dynamic analysis of the building modeled as a cantilevered shear wall. RISA 2D was used for this analysis, with cracked sections taken into account. From this analysis, a period of T = 0.75 sec was determined. See Sec. 9.3.4. This value is also obtained from the 3D dynamic analysis (described subsequently) for the first translational mode in the transverse direction when using a reduced modulus of elasticity to account for cracking in the masonry (approximately 60 percent of the nominal value for E). *Provisions* Sec. 5.4.2 [Sec. 5.2-2] requires that the fundamental period, T, established in a properly substantiated analysis be no larger than the approximate period, T_a, multiplied by C_u, determined from *Provisions* Table 5.4.2 [Table 5.2-1]. The approximate period of the building, T_a, is calculated based as:

$$T_a = C_r h_n^{3/4} = (0.02)(120)^{0.75} = 0.725 \text{ sec}$$

where C_r = 0.02 from *Provisions* Table 5.4.2.1 [Table 5.2-2], and h_n = 120 ft

$$T_a C_u = (0.725)(1.4) = 1.015 \text{ sec} > 0.75 \text{ sec} = T$$

(Note that T = 0.75 sec will be verified later when deflections are examined).

The value for C_s is taken to be 0.198 (the minimum of the two values computed above). This value is still larger than the minimum specified:

$$C_s = 0.044 I S_{D1} = (0.044)(1.0)(0.60) = 0.0264$$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated by *Provisions* Eq. 5.4.1 [Eq. 5.2-1]:

$$V = C_s W = (0.198)(12{,}962 \text{ kips}) = 2{,}568 \text{ kips}$$

A 3-D model was created in SAP 2000 for the MRS analysis. Just as for the five-story building described in Sec. 9.2, the masonry walls were modeled as shell bending elements and the floors were modeled as an assembly of beams and shell membrane elements. See Sec. 9.2.6.2 for further description. The difference in f'_m between upper and lower stories was not modeled; the value of E_m used was 1,100 ksi, which is 59 percent of the value from *Provisions* Eq. 11.3.10.2 for the lower stories. [Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1.] As mentioned, this value was selected as an approximation of the effects of flexural cracking. Unlike the five-story building, the difference in length between the longitudinal and transverse walls was modeled. However, to simplify construction of the model, wall types A and B are the same length. Because this example illustrates design in the transverse direction, this liberty has little effect. Table 9.3-2 shows data on the modes of vibration used in the analysis.

Table 9.3-2 Periods, mass participation ratios, and modal base shears in the transverse direction for modes used in analysis

Mode number	Period (seconds)	Individual mode (percent) Long.	Trans.	Vert.	Cumulative sum (percent) Long.	Trans.	Vert.	Trans. base shear
1	0.9471	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	0.7469	0.00	59.12	0.00	0.00	59.12	0.00	1528.0
3	0.6941	59.16	0.00	0.00	59.16	59.12	0.00	0.0
4	0.2247	0.00	0.00	0.00	59.16	59.12	0.00	0.0
5	0.1763	0.00	24.38	0.00	59.16	83.50	0.00	896.2
6	0.1669	24.57	0.00	0.00	83.73	83.50	0.00	0.0
7	0.1070	0.00	0.01	0.00	83.73	83.51	0.00	0.5
8	0.1059	0.00	0.00	0.28	83.74	83.51	0.28	0.0
9	0.1050	0.00	0.00	29.48	83.74	83.51	29.76	0.0
10	0.0953	0.00	0.00	0.00	83.74	83.51	29.76	0.0
11	0.0900	0.00	0.00	1.51	83.74	83.51	31.27	0.0
12	0.0858	0.00	0.03	0.01	83.74	83.54	31.28	1.1
13	0.0832	0.00	7.25	0.00	83.74	90.79	31.28	234.4
14	0.0795	7.11	0.00	0.00	90.85	90.79	31.28	0.0
15	0.0778	0.04	0.00	0.19	90.88	90.79	31.48	0.0
16	0.0545	0.00	4.47	0.00	90.88	95.26	31.48	117.5
17	0.0526	4.44	0.00	0.00	95.32	95.26	31.48	0.0
18	0.0413	0.01	1.24	0.00	95.33	96.51	31.48	29.1
19	0.0392	1.66	0.05	0.00	96.99	96.55	31.48	1.1
20	0.0358	0.07	0.87	0.00	97.06	97.43	31.48	19.5
21	0.0288	1.59	0.33	0.01	98.66	97.76	31.49	7.0
22	0.0278	0.33	1.40	0.00	98.99	99.16	31.49	28.9
23	0.0191	0.76	0.23	0.01	99.75	99.39	31.50	4.3
24	0.0186	0.23	0.60	0.00	99.98	99.98	31.50	11.1

1 kip = 4.45kN.

The combined modal base shear is 1,791 kips

The fundamental mode captures no translation of mass; it is a pure torsional response. This is a confirmation of the intent of the torsional irregularity provision. The first translational mode has a period of 0.75 sec, confirming the earlier statements. Also note that the base shear is only about 70 percent of the ELF base shear (2,568 kips) even though the fundamental period is the same. The ELF analysis assumes that all the mass participates in the fundamental mode whereas the dynamic analysis does not. The absolute sum of modal base shears is higher than the ELF but the statistical sum is not. *Provisions* Sec. 5.5.7 requires that the modal base shear be compared with 85 percent of the ELF base shear. The comparison value is 0.85(2,568 kips), which is 2,183 kips. Because this is greater than the value from the modal analysis, the modal analysis results would have to be factored upwards by the ratio 2,183/1,791 = 1.22. The period used for this comparison cannot exceed C_uT_a, which is 1.015 sec as described previously. Note that the period used is from Mode 2, because Mode 1 is a purely torsional mode. The 1.22 factor is very close to the factor for the five story building computed in Sec. 9.2.6.2; an additional comparison will follow.

9.3.3.3 Vertical Distribution of Seismic Forces

Carrying forward with the ELF analysis, *Provisions* Sec. 5.4.3 [Sec. 5.2.3] provides the procedure for determining the portion of the total seismic loads assigned to each floor level. The story force, F_x, is calculated as:

$$F_x = C_{vx}V$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

For T = 0.75 sec, which is between 0.5 sec and 2.5 sec, the value of k is determined to be 1.125 based on interpolation (*Provisions* Sec. 5.4.3 [Sec. 5.2.3]).

The seismic design shear in any story shall be determined from:

$$V_x = \sum_{i=x}^{n} F_i$$

The story overturning moment is computed from:

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

Table 9.3-3 shows the application of these equations for this building.

Table 9.3-3 Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (kips)	$w_x h_x^{1.125}$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (kips)
12	768	120	167,700	0.128	329	329	3,300
11	917	110	181,500	0.139	357	686	10,200
10	917	100	163,100	0.125	320	1,006	20,200
9	917	90	144,800	0.111	284	1,291	33,100
8	917	80	126,900	0.097	249	1,540	48,500
7	917	70	109,200	0.084	214	1,754	66,000
6	1,109	60	111,000	0.085	218	1,972	85,800
5	1,300	50	106,000	0.081	208	2,181	107,600
4	1,300	40	82,500	0.063	162	2,342	131,000
3	1,300	30	59,700	0.046	117	2,460	155,600
2	1,300	20	37,800	0.029	74	2.534	181,000
1	1,300	10	17,300	0.013	34	2,568	206,600
			1,307,400	1.00	2,568		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m.

The dynamic modal analysis does give a direct output for the gross overturning moment, of about 66 percent of the moment from the ELF analysis. Because the model is built with shell elements, there is no direct value for the variation of moment with height.

9.3.3.4 Horizontal Distribution

For the ELF analysis, the approach is essentially the same as used for the five-story masonry building described in Sec. 9.2.4.4:

Direct shear: All transverse walls have the same properties, except axial load. Axial load affects cracking but, each wall considered has the same stiffness. Therefore, each will resist an equivalent amount in direct shear:

$$V = V/8 = 0.125V_x$$

Torsion: The center of mass corresponds with the center of resistance; therefore, the only torsion is due to the 5 percent accidental eccentricity in accordance with *Provisions* Sec. 5.4.4.2 [Sec. 5.2.4.2]:

$$M_{ta} = 0.05\,bV = (0.05)(152\text{ ft})V = 7.6V$$

The longitudinal walls are slightly longer than the transverse walls. Unlike the example in Sec. 9.2, this difference will be illustrated here in a simple fashion. The diaphragm is assumed to be rigid. When the walls are not identical, a measure of the actual stiffness is necessary; for masonry walls, this involves both flexural and shear deformations. The conventional technique is an application of the following equation for deformation of a simple cantilever wall without bulbs or flanges at the ends:

$$\Delta_{wall} = \frac{Vh^3}{3E_m I} + \frac{6Vh}{5G_m A}$$

Considering $G_m = 0.4\,E_m$, $A = Lt$, and $I = L^3t/12$, this can be simplified to :

$$\Delta_{wall} = \frac{V}{Et}\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]$$

Rigidity, K, is inversely proportional to deflection. Considering E and t as equal for all walls:

$$K = \frac{1}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$$

Figure 9.3-3 identifies the walls.

For a multistory building, the quantity h is not easy to pin down. For this example, the authors suggest the following approach: use h= 10 ft (one story) to evaluate the shear in the wall at the base and also use h = 80 ft (two thirds of total height) to evaluate the moments in the walls. Table 9.3-4 shows some of the intermediate steps for these two assumptions.

(d, as used here, is the distance of the wall to the centroid of the building, not the length of the wall, as used elsewhere)

Table 9.3-4 Relative Rigidities

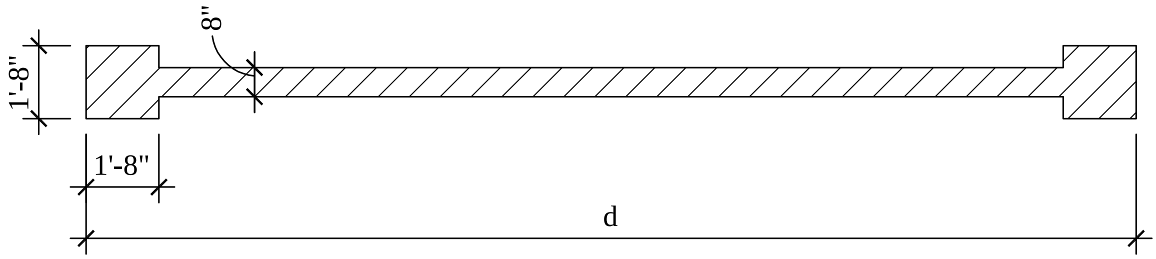
Wall	Length (ft)	Arm, d (ft)	for shear, h = 10 ft			for moment, h = 80 ft		
			h/d	K	Kd^2 (ft^2)	h/d	K	Kd^2 (ft^2)
A	36	36	0.278	1.088	1,410	2.22	0.01978	25.63
B	34	12	0.294	1.017	146	2.35	0.01690	2.43
C	33	12	0.303	0.980	141	2.42	0.01556	2.24
D	33	36	0.303	0.980	1,270	2.42	0.01556	20.17
					2,967			50.47

1.0 ft = 03.048 m

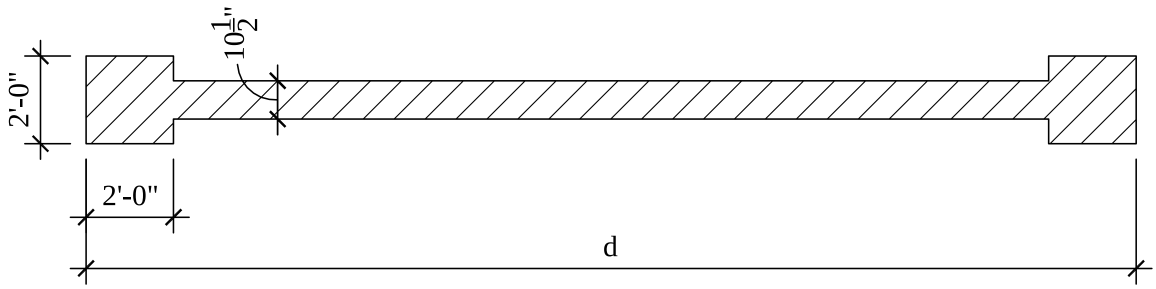
The total torsional rigidity is four times the amount in Table 9.3-4, since there are four walls of each type. When considering torsion, Wall D is the critical member (shortest length, greatest *d*).

For shear due to accidental torsion:

$$V_t = \frac{MKd}{\Sigma Kd^2} = 7.6V\left[\frac{(0.980)(36)}{4(2,967)}\right] = 0.0226V \text{ or } 7.6V\left[\frac{0.01556(36)}{4(50.47)}\right] = 0.0211V$$



Wall at Stories 7 to 12



Wall at Stories 1 to 6

Wall length	
Wall	d
A	36'-0"
B	34'-0"
C	33'-0"
D	33'-0"

Figure 9.3-3 Wall dimensions (1.0 in. = 25.4 mm, 1.0 ft = 0.0348 m).

When considering the approximations involved, the remainder of the ELF example will simply use 0.0226V for V_t. Because a 3D analytical model exists, a simplistic load case with a static horizontal torsion at each level was defined. The couple varied directly with height, so the variation of mass with height was ignored. Examining the base reactions for Wall D yields a torsional shear equal to 0.0221V and an overturning moment corresponding to 0.0191V. Therefore, the hand computations illustrated are somewhat conservative.

Total shear for Wall D is equal to the direct shear plus shear due to accidental torsion, which is computed as:

$$0.125V + 0.0226V = 0.148V$$

The resulting shears and overturning moments for Wall D are shown in Table 9.3-5.

Table 9.3-5 Shear for Wall D

Level	Story Shear (kips)	Wall Shear (kips)	Story Moment (ft-kips)	Wall Moment (ft-kips)
12	329	49	3,300	500
11	686	102	10,200	1,500
10	1,006	149	20,200	3,000
9	1,291	191	33,100	4,900
8	1,540	228	48,500	7,200
7	1,754	260	66,000	9,800
6	1,972	292	85,800	12,700
5	2,181	323	107,600	15,900
4	2,342	347	131,000	19,400
3	2,460	364	155,600	23,000
2	2,534	375	181,000	26,800
1	2,568	380	206,600	30,60

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

When considering accidental torsion, a check for torsional irregularity must be made. First consider the case used for design: a direct shear of 0.125V and a torsional shear of 0.0226V. The ratio of extreme displacement to average displacement can be found from these values and the dimensions, considering symmetry:

Average displacement is proportional to 0.125V
Torsional displacement at Wall D is proportional to 0.0226V
Torsional displacement at the corner is proportional to (0.0226V)((152 ft./2)/36 ft. = 0.0447V
Ratio of corner to average displacement = (0.125 + 0.0447)/0.125 = 1.38

If the lower value of torsional shear, 0.0191V, found from the 3D computer analysis for the static torsion is used, the ratio becomes 1.32. In either case, the result is a torsional irregularity (ratio exceeds 1.2) but not an extreme torsional irregularity (ratio does not exceed 1.4). The reason for the difference from the five-story building, in which the ratio exceeded 1.4, is that the longer walls in the longitudinal directions. For the ELF analysis, *Provisions* Sec. 5.4.4.3 [Sec. 5.2.4.3] requires the accidental torsion to be amplified:

$$A_x = \left(\frac{Max\ displacement}{1.2\ Ave\ displacement} \right)^2 \leq 3.0$$

If one uses the ratio of 1.32 based on the 3D computer analysis, the amplifier is 1.21 and the torsional shear becomes 1.32(0.0191V) = 0.0231V. This is close enough to the unamplified 0.0226V that the ELF analysis will simply proceed with a torsional shear of 0.0226V.

As described in Sec. 9.2.6.4 for the five-story building, the 3D analytical model was altered to offset the center of mass from the center of rigidity. The modal periods, mass participation ratios, and base shears are given in Table 9.3-6. The total base shear is 1620 kips, down from the 1,791 kips found without the eccentricity. The 1,620 kips still slightly exceeds the minimum for design of 1,613 kips described earlier. Thus, MRS analysis can be used directly in the load combinations and can be considered to include the amplified accidental torsion.

Table 9.3-6 Periods, Mass Participation Ratios, and Modal Base Shears in the Transverse Direction for Modes Used in Analysis of Building with Deliberate Eccentricity

Mode number	Period (seconds)	Individual mode (percent) Long.	Trans.	Vert.	Cumulative sum (percent) Long.	Trans.	Vert.	Trans. base shear
1	0.965	0.0	8.5	0.0	0.0	8.5	0.0	169.4
2	0.723	0.0	50.6	0.0	0.0	59.1	0.0	1352.7
3	0.694	59.2	0.0	0.0	59.2	59.1	0.0	0.0
4	0.229	0.0	3.3	0.0	59.2	62.5	0.0	122.7
5	0.171	0.0	21.0	0.0	59.2	83.5	0.0	772.7
6	0.167	24.6	0.0	0.0	83.7	83.5	0.0	0.0
7	0.120	0.0	0.0	20.3	83.7	83.5	20.3	0.0
8	0.108	0.0	1.0	0.0	83.7	84.4	20.3	35.3
9	0.105	0.0	0.0	0.0	83.7	84.4	20.3	0.1
10	0.097	0.0	0.0	0.0	83.7	84.4	20.3	0.1
11	0.090	0.0	0.0	11.4	83.8	84.4	31.7	0.0
12	0.081	0.0	6.2	0.0	83.8	90.7	31.7	198.6
13	0.079	7.1	0.0	0.1	90.9	90.7	31.8	0.0
14	0.074	0.0	0.0	3.1	90.9	90.7	34.8	0.1
15	0.072	0.0	0.6	0.1	90.9	91.3	34.9	17.5
16	0.061	0.0	0.5	0.1	90.9	91.7	34.9	12.8
17	0.053	4.3	0.0	0.0	95.2	91.7	34.9	0.0
18	0.052	0.0	4.1	0.0	95.2	95.8	34.9	104.8
19	0.043	0.7	0.0	0.0	95.9	95.8	34.9	0.0
20	0.037	1.5	0.0	0.0	97.4	95.8	35.0	0.1
21	0.035	0.0	2.5	0.0	97.4	98.3	35.0	54.7
22	0.027	1.8	0.0	0.0	99.2	98.3	35.0	0.0
23	0.022	0.0	1.7	0.0	99.2	100.0	35.0	32.8
24	0.018	0.8	0.0	0.0	100.0	100.0	35.0	0

1 kip = 4.45 kN

The combined modal base shear is 1620 kips.

Mode 1 now includes a translational component, and the comparison to an ELF base shear would be performed with its period. For T = 0.965 sec, the ELF base shear becomes 1,996 kips, and the comparison value is 0.85 (1996) = 1,696 kips. This is 78 percent of the value for the symmetric model and illustrates one of the problems in handling accidental torsion in a consistent fashion.

Without factoring the modal results up to achieve a base shear of 1,696 kips (a factor of 1.047), the reactions indicate that the base shear for wall D is 266.5 kips, or 0.1645 times the total base shear. If one takes the direct shear as one-eighth (0.125V), that leaves 0.0395V for the dynamically amplified shear due to accidental torsion, which could be interpreted to be an amplification of 1.79 over the shear of 0.0221V found for the static torsion. Thus, it is clear that the amplification value of 1.21 from the equation given for the ELF analysis underestimates the dynamic amplification of accidental torsion. The bottom line is that the Wall D shear of 266.5 kips from the dynamic analysis is significantly less than the shear of 380 kips found in the ELF analysis without amplification of accidental torsion. The example will proceed based upon the shear of 380 kips.

9.3.3.5 Transverse Wall (Wall D)

The strength or limit state design concept is used in the *Provisions*.

[The 2003 *Provisions* adopts by reference the ACI 530-02 provisions for strength design in masonry, and the previous strength design section has been removed. This adoption does not result in significant technical changes, and the references to the corresponding sections in ACI 530 are noted in the following sections.]

9.3.3.5.1 Axial and Flexural Strength General

The walls in this example are all bearing shear walls since they support vertical loads as well as lateral forces.

The demands for the representative design example, Wall D, are presented in this section. The design of the lower and upper portions of Wall D is presented in the next two sections. For both locations, in-plane calculations include:

1. Strength check and
2. Ductility check.

The axial and flexural demands for Wall D, using the load combinations identified in Sec. 9.3.2.3, are presented in Table 9.3-7. In the table, Load Combination 1 represents $1.4D + 1.0\ Q_E + 0.5L$, and Load Combination 2 represents $0.7D + 1.0\ Q_E$.

Table 9.3-7 Load Combinations for Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
12	37	0	51	500	26	500
11	80	8	117	1,500	56	1,500
10	124	17	182	3,000	87	3,000
9	168	25	247	4,900	117	4,900
8	212	34	313	7,200	148	7,200
7	255	43	379	9,800	179	9,800
6	308	50	457	12,700	216	12,700
5	370	59	548	15,900	259	15,900
4	432	67	639	19,400	303	19,400
3	494	76	730	23,000	346	23,000
2	556	84	821	26,800	389	26,800
1	618	92	912	30,600	433	30,60

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

Strength at the lowest story (where *P*, *V*, and *M* are the greatest) for both the lower wall (Level 1) and the upper wall (Level 7) constructions will be examined. The design for both locations is based on the values for Load Combination 2 in Table 9.3-7.

9.3.3.5.2 Axial and Flexural Strength Lower Levels

Examine the strength of Wall D at Level 1:

9.3.3.5.2.1 Strength Check (Level 1)

$P_{u_{min}}$ = 433 kips + factored weight of half of 1st story wall = 433 + (0.7)(21.9) = 448 kips

M_u = 30,600 ft-kips

For this Seismic Design Category D building, the special reinforced masonry shear walls must have vertical and horizontal reinforcement spaced at no more than 4 ft on center. The minimum in either direction is 0.0007(10.5 in.) = 0.0074 in.^{2}/in. (vertical #5 at 42 in. on center). That will be used as the vertical reinforcement (although some of the subsequent calculations of flexural resistance are based upon a spacing of 48 in. on center); the shear strength demands for horizontal reinforcement will be greater and will satisfy the total amount of 0.0020(10.5 in.) = 0.021 in.^{2}/in.(horizontal #5 at 22 in. on center will suffice).

Try 16 #9 bars in each bulb. Refer to Figure 9.3-4 for the placement of the reinforcement in the bulb. In some of the strength calculations, the #5 bars in the wall will be neglected as a conservative simplification.

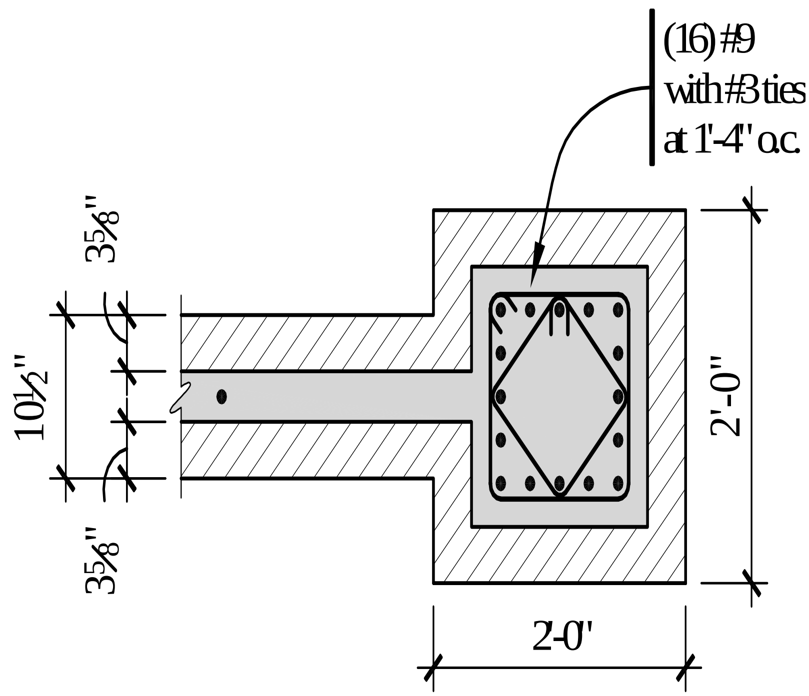


Figure 9.3-4 Bulb reinforcement at lower levels
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

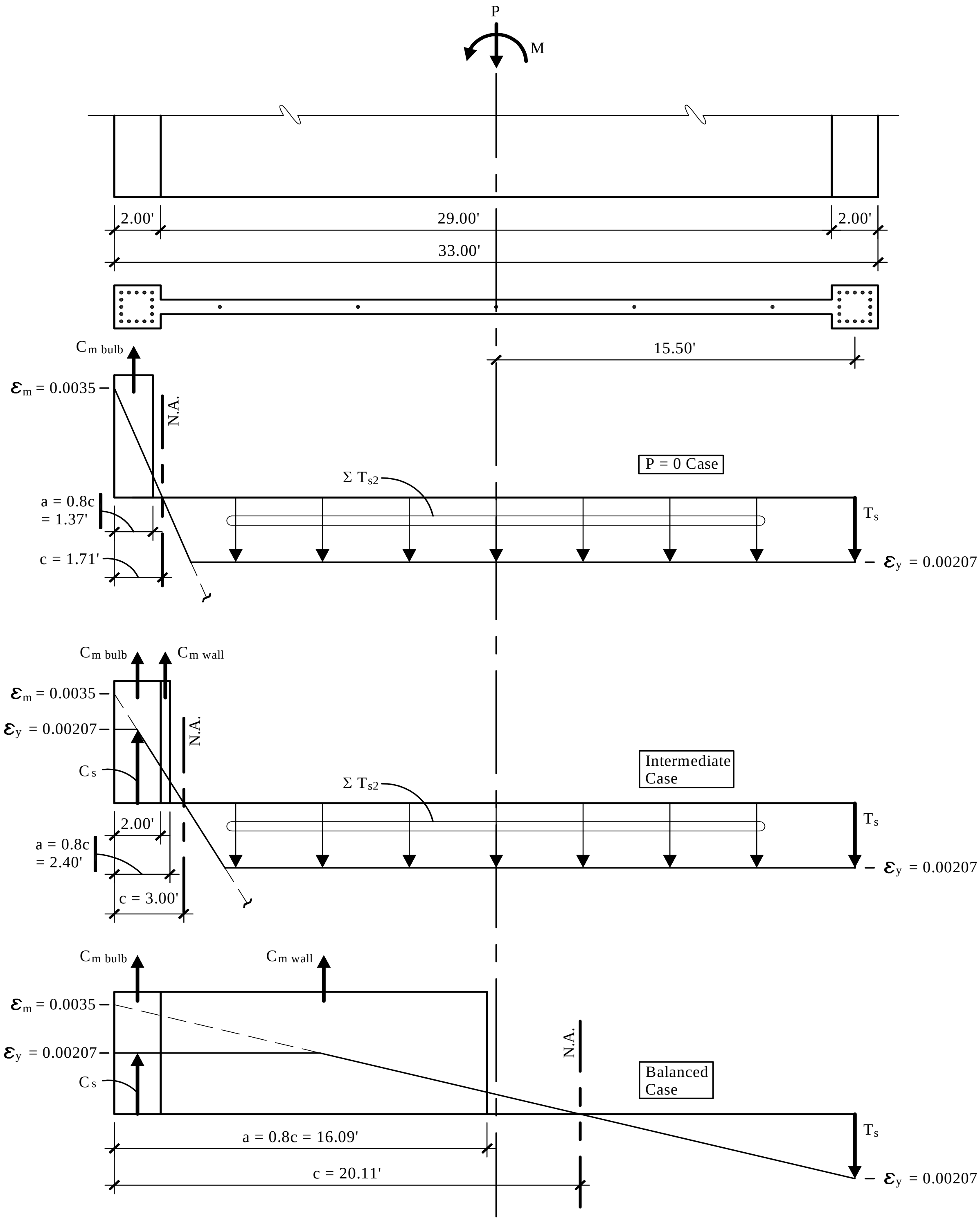


Figure 9.3-5 Strength of Wall D, Level 1 (1.0 ft = 0.3048 m)

For evaluating the capacity of the wall, a ϕP_n - ϕM_n curve will be developed to represent the wall strength envelope. The demands (P_u and M_u determined above) will then be compared to this curve. Several cases will be analyzed and their results used in plotting the ϕP_n - ϕM_n curve. Refer to Figure 9.3-5 for notation and dimensions.

Case 1 (P = 0)

The neutral axis will be within the compression bulb, so assume that only the bars closest to the compression face are effective in compression. (Also recall that the *Provisions* clearly endorses the use of compression reinforcement in strength computations.)

T_{s1} = (16 bars)(1.00 in.^{2})(60 ksi) = 960 kips
T_{s2} = (7 bars)(0.31 in.^{2})(60 ksi) = 130 kips
C_s = (5 bars)(1.00 in.^{2})(60 ksi) = 300 kips

$\Sigma C = \Sigma T + P$
$C_m + C_s = T_{s1} + T_{s2} + P$
C_m = 960 + 130 +0 - 300 = 790 kips

C_m = 790 kips = $\phi f'_m$(24 in.)a = (0.8)(2.5 ksi)(24 in.)a
a = 16.46 in. = 1.37 ft
$c = a$/0.8 = 1.37/0.8 = 1.71 ft = 20.7 in.

Check strain in compression steel

ε_s = 0.0035(20.7 in. - 6 in.)/(20.7 in.) = 0.0025 > yield; assumption OK

$\Sigma M_{cl} = 0$
M_n = (790 kips)(16.5 ft- 1.37 ft/2) + (300 + 960 kips)(15.5 ft) + (130 kips)(0 ft.) = 32,170 ft-kips
ϕM_n = (0.85)(32,170) = 27,340 ft-kips

Case 2 (Intermediate case between P = 0 and balanced case):

Select an intermediate value of c. Let c = 3.0 ft, and determine P_n and M_n for this case.

$a = 0.8c$ = 2.4 ft
$C_{m\ bulb}$ = (0.8)(2.5 ksi)(24 in.)^2 = 1152 kips
$C_{m\ wall}$ = (0.8)(2.5 ksi)(10.5 in.)(0.4 ft. × 12) = 101 kips
C_s = (16 bars)(1.00 in.^{2})(60 ksi) = 960 kips (approximate; not all bars reach full yield)
ΣC = (1152 + 101 + 960) = 2213 kips
ΣT = 960 +130 kips = 1090 kips

$\Sigma F_y = 0$
$P_n = \Sigma C - \Sigma T$ = 2213 - 1090 = 1123 kips
ϕP_n = (0.85)(1123) = 955 kips

$\Sigma M_{cl} = 0$
M_u = (1152 +960 kips)(15.5 ft) + (101 kips)(14.1 ft) + (960 kips)(15.5 ft) = 49,040 ft-kips
ϕM_n = (0.85)(49,040) = 41,680 ft-kips

Case 3 (Balanced case):

$$c = \left[\frac{0.0035}{0.0035 + 0.00207}\right](32.00\ \text{ft}) = 20.11\ \text{ft}$$

$a = (0.8)c$ = 16.09 ft

Ignore the distributed #5 bars for this case.

$C_{m\ bulb}$ = 1152 kips
$C_{m\ wall}$ = (0.8)(2.5 ksi)(10.5 in.)(14.09 ft. × 12) = 3,550 kips
C_s = 960 kips
ΣC = (1152 + 3550 + 960) = 5,662 kips
T_s = 960 kips

$\Sigma F_y = 0$
$P_n = \Sigma C - \Sigma T$ = 5662 - 960 = 4,702 kips
φP_n = (0.85)(4,702) = 3,997 kips

$\Sigma M_{cl} = 0$
M_u = (1152 +960 kips)(15.5 ft) + (3550 kips)(7.46 ft) + (960 kips)(15.5 ft) = 74,100 ft-kips
ϕM_n = (0.85)(74,100) = 62,980 ft-kips

The actual design strength, ϕM_n, at the level of minimum axial load can be found by interpolation to be 33,840 ft.-kip.

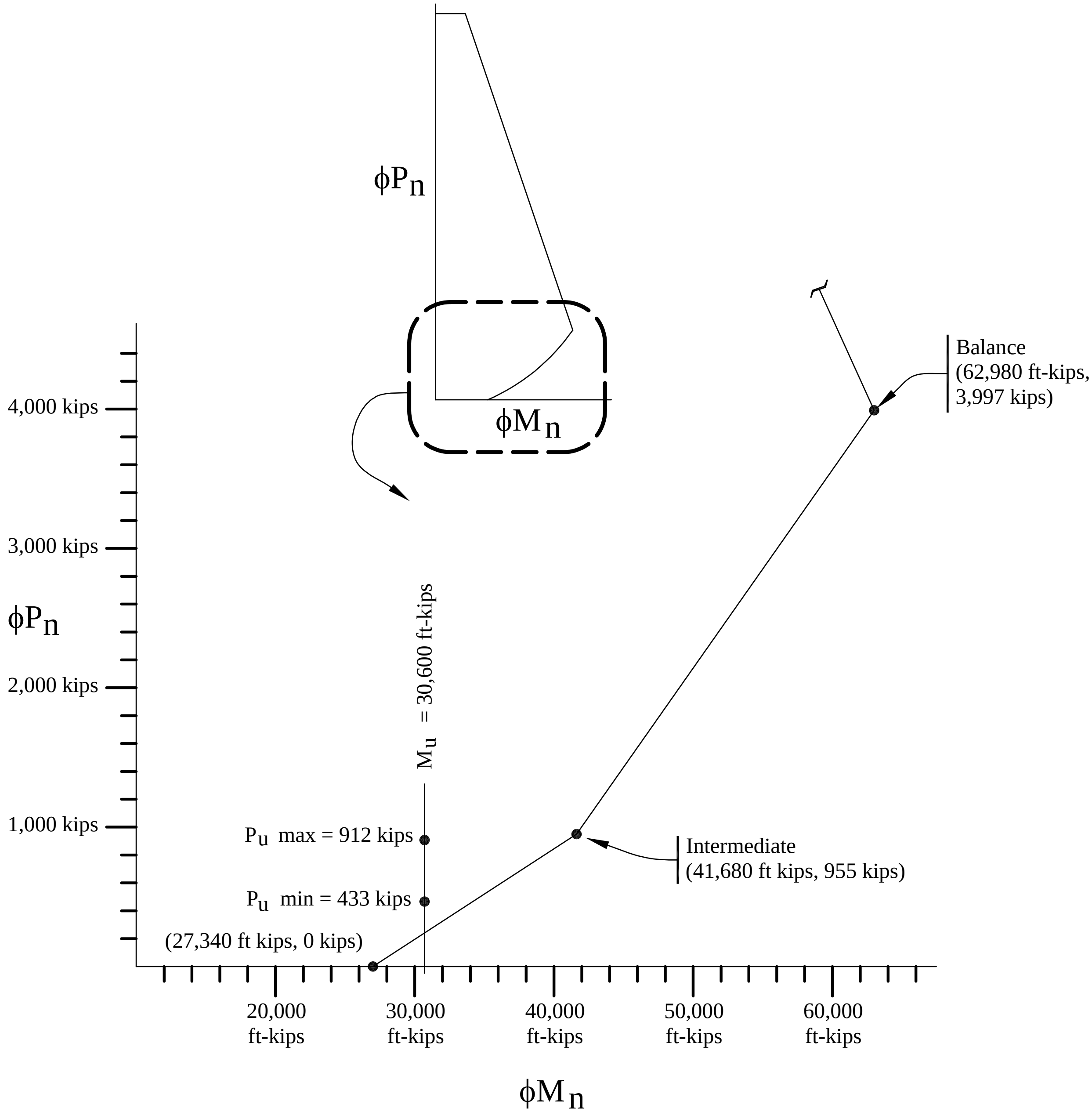


Figure 9.3-6 φP_{11} - φM_{11} Diagram for Level 1 (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.3.3.5.2.2 Ductility check (Level 1)

Provisions Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5] requires that the critical strain condition correspond to a strain in the extreme tension reinforcement equal to 5 times the strain associated with F_y. Note that this calculation uses unfactored gravity axial loads (*Provisions* 11.6.2.2 [ACI 530, Sec. 3.2.3.5]). See Figure 9.3-7 and the following calculations.

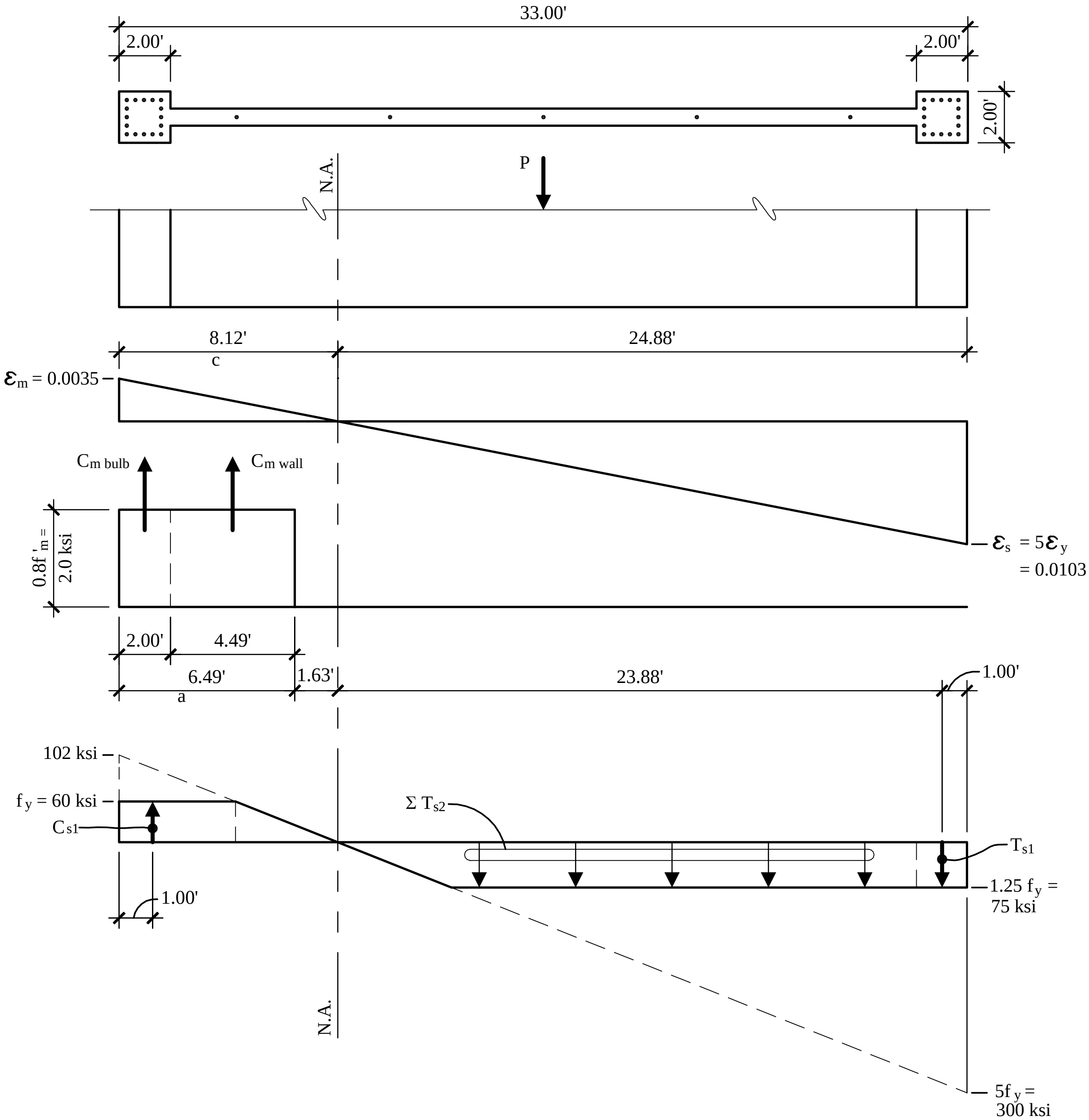


Figure 9.3-7 Ductility check for Wall D, Level 1 (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa).

$$c = \left[\frac{0.0035}{(0.0035+0.0103)}\right](32.00\text{ ft}) = 8.12\text{ ft}$$

$a = 0.8\,c = 6.49$ ft

For Level 1, the unfactored loads are:

$P = 618$ kips
$M = 30{,}600$ ft-kips
$$C_{m_{bulb}} = 0.8\,f'_m A_{bulb} = 1{,}152\text{ kips}$$
$$C_{m_{wall}} = 0.8\,f'_m(10.5\text{ in.})(4.49\text{ ft} \times 12) = 1131\text{ kips}$$

The distributed wall rebar that is near the neutral axis is divided between tension and compression, and therefore it will not have much effect on the result of this check, so it will be neglected.

C_{s1} = (60 ksi)(16 × 1.00 in.^{2}) = 960 kips
T_{s1} = (16 × 1.00 in.^{2})(75 ksi) = 1,200 kips
T_{s2} = (4 × 0.31 in.^{2})(75 ksi) = 93 kips

P = 618 kips (unfactored dead load)

$\sum C > \sum P + \sum T$
1152 +1131 + 960 > 618 + 1200 + 93
3,243 kips > 1,818 kips

There is more compression capacity than tension capacity, so a ductile failure condition governs.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

9.3.3.5.3 Axial and Flexural Strength Upper Levels

Examine the strength of Wall D at Level 7.

$P_{u_{min}}$ = 179 kips + factored weight of ½ of 7th story wall = 179 + (0.7)(11.4) =190 kips
M_u = 9,800 ft-kips

This is a point, however, where some of the reinforcement in the lower wall will be terminated. Although not required by the *Provisions*, most design standards require the longitudinal reinforcement to be extended a distance *d* beyond the point where it could theoretically be terminated. (The ASD chapter of ACI 530 has such a requirement.) Therefore, the reinforcement at level 6 (7th floor) should be capable of resisting the moment *d* below. *d* is approximately three stories for this wall, therefore, take M_u = 19,400 ft-kip (and P = 303 kip) from Level 3.

Try eight #9 in each bulb and vertical #5 bars at 4 ft on center in the wall. Refer to Figure 9.3-8 for the placement of the bulb reinforcement.

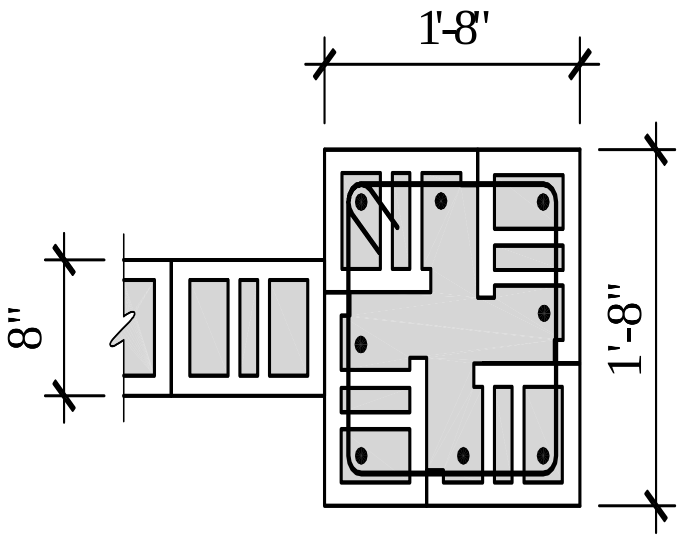


Figure 9.3-8 Bulb reinforcement at upper levels
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

For evaluating the capacity of the wall, a ϕP_n - ϕM_n curve will be developed to represent the wall strength envelope for Level 7. The demands (P_u and M_u determined above) will then be compared to this curve. Several cases will be analyzed and their results used in plotting the ϕP_n - ϕM_n curve. Refer to Figure. 9.3-9 for notation and dimensions.

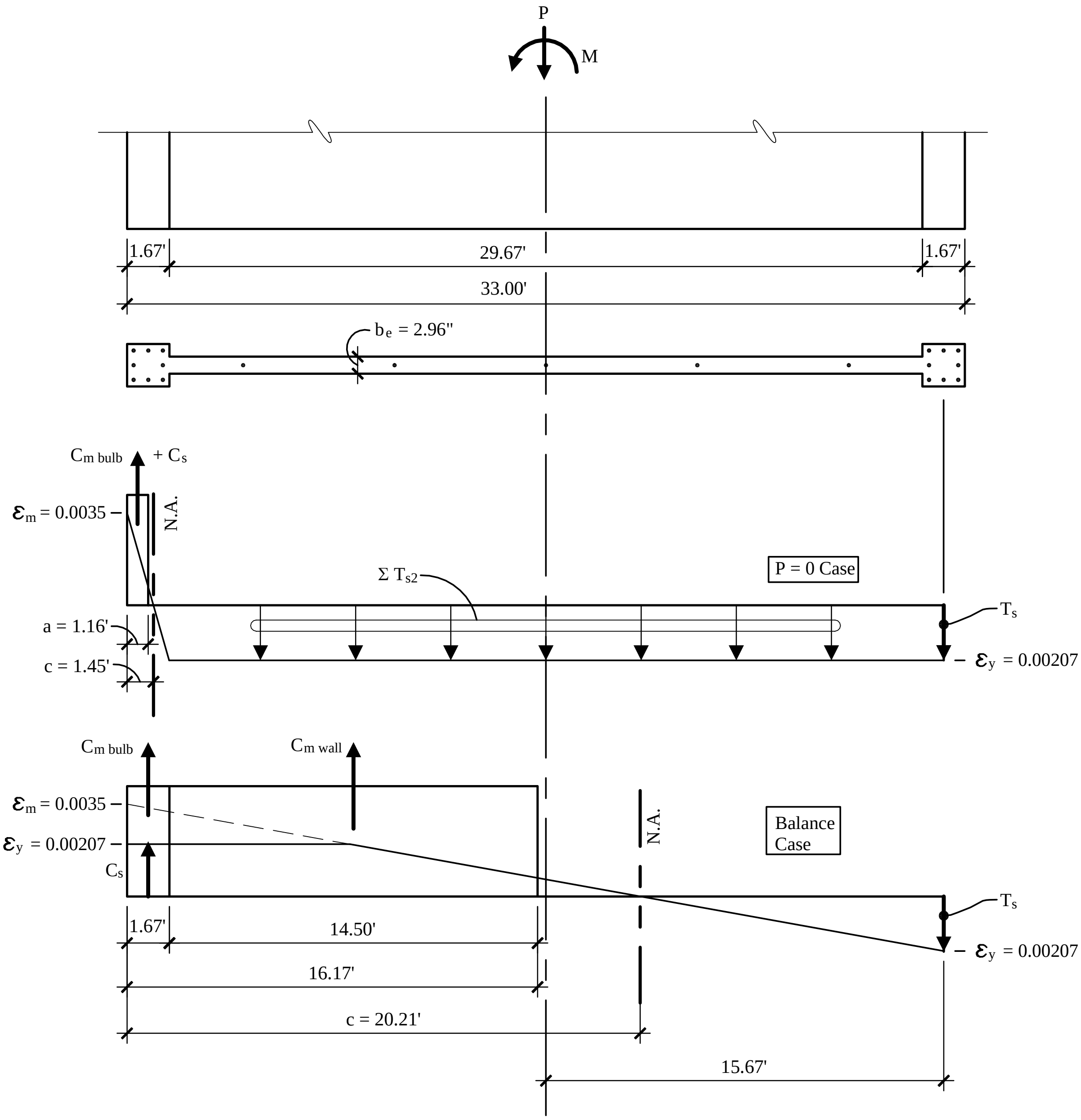


Figure 9.3-9 Strength of Wall D at Level 7 (1.0 ft = 0.3048 m)

Case 1 (P = 0)

Tension forces:

T_{s1} = (8 bars)(1.00 in.^{2})(60 ksi) = 480 kips
T_{s2} = (7 bars)(0.31 in.^{2})(60 ksi) = 130 kips

Equilibrium:

$$\Sigma C = \Sigma T + P$$

ΣC = 480 + 130 + 0 = 610 kips

Assume bars closest to compression face yield:

$\Sigma C = C_s + C_m$
C_s = (3 bars)(1.00 in.^{2})(60 ksi) = 180 kips
C_m = 610 - 180 = 430 kips

Locate equivalent stress block and neutral axis:

430 = $\phi f'_m$(20 in.)a = (0.8)(3 ksi)(20 in.)a
a = 8.96 in. = 0.75 ft
$c = a/0.8$ = 0.75/0.8 = 0.93 ft = 11.2 in.

Verify strain in compression steel:

At the outside layer, ε = (0.0035)(7.2 in. / 11.2 in.) = 0.0023 > yield,
At the central layer, ε = (0.0035)(1.2 in. / 11.2 in.) = 0.0004 => f_s = 11 ksi

Resultant moment:

$\Sigma M_{cl} = 0$:
M_n = (430 kips)(16.5 ft- 0.75 ft/2) + (180 kips)(16.5 - 0.33 ft) + (480 kips)(16.5 - 0.83 ft)
+ (130 kips)(0 ft.) = 17,370 ft-kips
ϕM_n = (0.85)(17,280) = 14,760 ft-kips

<u>Case 2 (Intermediate)</u>

Assume the neutral axis at the face of the bulb, c = 1.67 ft

$a = 0.8c$ = 1.33 ft. = 16 in.
C_m = (2.4 ksi)(16 in.)(20 in.) = 768 kip

For the compression steel, it is necessary to compute the strains:

At the outside layer, ε = (0.0035)(16 in. / 20 in.) = 0.0028 > yield
At the central layer, ε = (0.0035)(10 in. / 20 in.) = 0.00175 => f_s = 50 ksi
At the inside layer, ε = (0.0035)(4 in. / 20 in.) = 0.0007 => f_s = 20 ksi
C_s = (3.0 x 60 ksi + 2.0 x 50 ksi + 3.0 x 20 ksi) = 340 kips

T_{s1} = (8 bars)(1.00 in.^{2})(60 ksi) = 480 kips
T_{s2} = (7 bars)(0.31 in.^{2})(60 ksi) = 130 kips

P_n = 768 + 340 - 480 -130 = 498 kips
ϕP_n = (0.85)(498) = 423 kips

M_n = (768 kips)(16.5 ft- 1.33 ft/2) + (340 + 480 kips)(16.5 - 1.67/2 ft) + (130 kips)(0 ft)
= 25,010 ft-kips
ϕM_n = (0.85)(25,010) = 21,260 ft-kips

At P = 303 kips, ϕM_n = 19,990 ft.-kips by interpolation (exceeds 19,400 ft.-kips, OK)

Case 3 (Balanced Case):

$$c = \left[\frac{0.0035}{0.0035+0.00207}\right](32.17\ \text{ft}) = 20.21\ \text{ft}$$

$a = (0.8)c = 16.17$ ft

b_e = effective width of wall
b_e = [(2)(1.3125 in.)(29.67 ft x 12) + (8 cells)(15 in.^{2}/cell)] = 2.96 in./ft
$C_{m\ bulb}$ = 960 kips
$C_{m\ wall}$ = (0.8)(3 ksi)(2.96 in.)(14.17 ft x 12) = 101 kips
C_s = 480 kips
ΣC = (960 + 101 + 480) = 1,541 kips
T_s = 480 kips, ignoring the distributed #5 bars

$\Sigma F_y = 0$
$P_n = \Sigma C - \Sigma T$ = 1,541 - 480 = 1,061 kips
ϕP_n = (0.85)(1,061) = 902 kips

$\Sigma M_{cl} = 0$
M_u = (480 +960 kips)(15.67 ft) + (101 kips)(7.42 ft) + (480 kips)(15.67 ft) = 30,830 ft-kips
ϕM_n = (0.85)(30,830) = 26,210 ft-kips

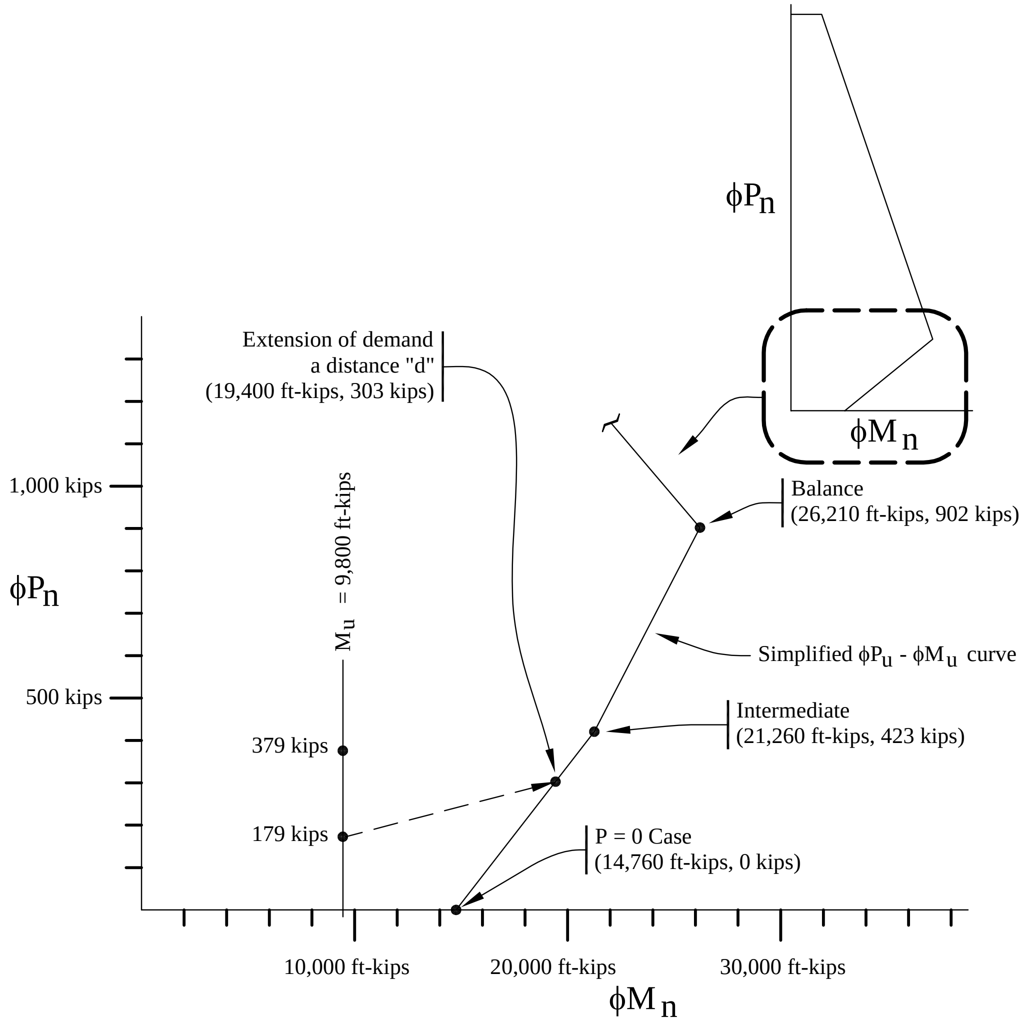


Figure 9.3-10 φP_{11} - φM_{11} Diagram for Level 7 (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

The ductility check is performed similar to that for the wall at Level 1. See Figure 9.3-11 and the following calculations.

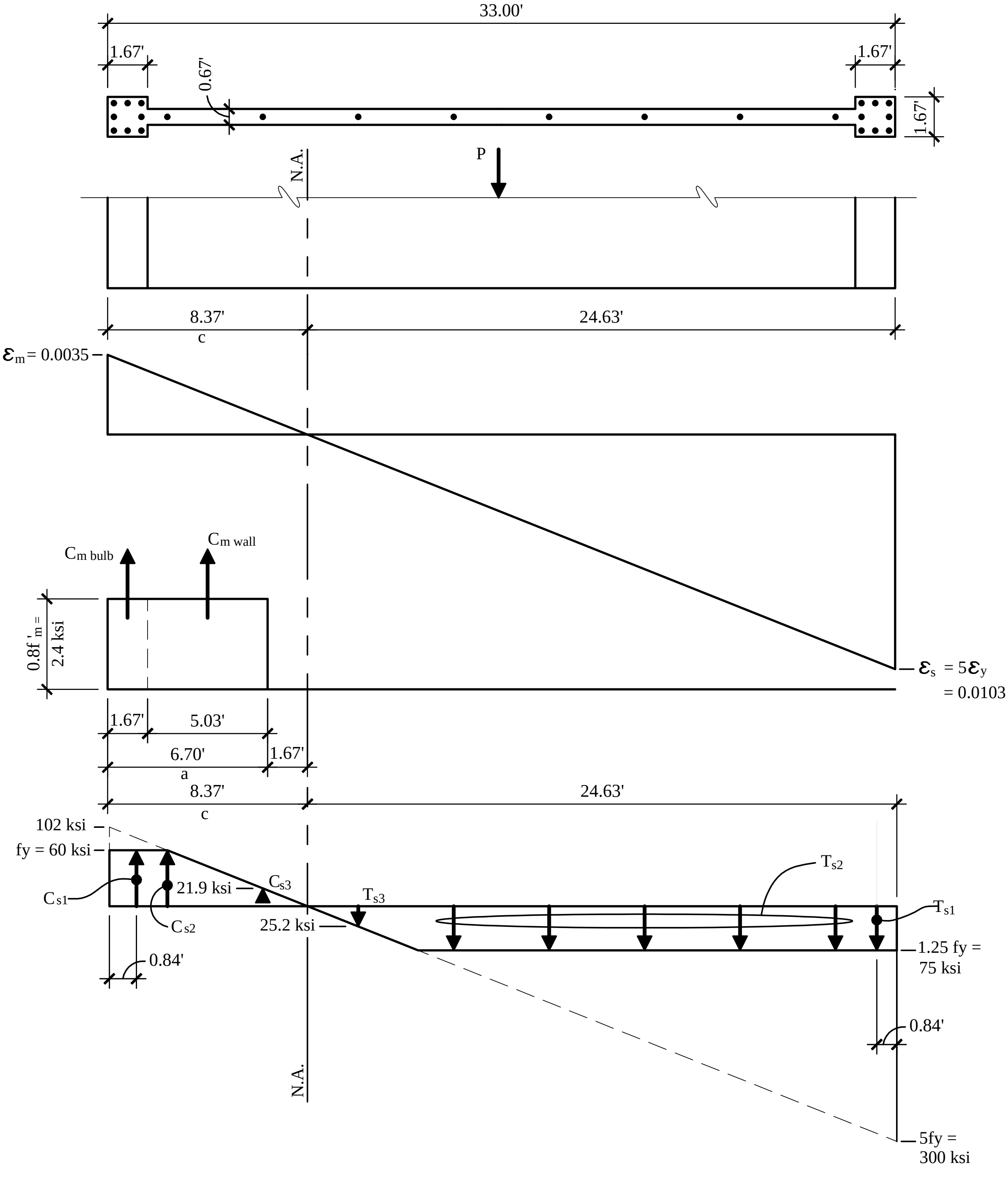


Figure 9.3-11 Ductility check for Wall D, Level 7 (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa)

For Level 7, the unfactored loads are:

P = 255 kips
M = 11,600 ft-kips

$C_{mb} = 0.8f'_mA_b = 0.8(3.0 \text{ ksi})(400 \text{ in.}2) = 960 \text{ kips}$
$C_{mw} = 0.8f'_mA_w = 0.8(3.0 \text{ ksi})[2(1.3125 \text{ in.})(5.03 \text{ ft.})(12 \text{ in/ft}) + (1 \text{ cell})(25.6 \text{ in.}^2/\text{cell})]$
$= 442 \text{ kips}$

$C_{s1} = (60 \text{ ksi})(8\times 1.00 \text{ in.}^2) = 480 \text{ kips}$
$C_{s2} = (60 \text{ ksi})(0.31 \text{ in.}^2) = 19 \text{ kips}$

$T_{s1} = (8 \times 1.00 \text{ in.}^2)(75 \text{ ksi}) = 600 \text{ kips}$
$T_{s2} = (4 \times 0.31 \text{ in.}^2)(75 \text{ ksi}) = 116 \text{ kips}$
$T_{s3} = (0.31 \text{ in.}^2)(25.2 \text{ ksi}) = 7 \text{ kips}$

$P = 255$ kips (unfactored dead load)

$\Sigma C > P + \Sigma T$
$960 + 442 + 480 + 19 > 255 + 600 + 116 + 7$?
1901 kips > 978 kips OK

There is more compression capacity than tension capacity, so a ductile failure condition governs.

9.3.3.5.4 Shear Strength

The first step is to determine the net area, A_n, for Wall D. The definition of A_n in the *Provisions*, however, does not explicitly address bulbs or flanges at the ends of walls. Following an analogy with reinforced concrete design, the area is taken as the thickness of the web of the wall times the overall length. In partially grouted walls this is not extended to the point of ignoring the grouted cores because the implication is that grouted cores are intended to be included. (However, if the spacing of bond beams greatly exceeds the spacing of grouted cores, even that assumption might be questionable.)

For Levels 1 through 6:

$A_n = (10.5 \text{ in.}(33 \text{ ft} \times 12)) = 4{,}158 \text{ in.}^2$

For Levels 7 through 12 (using 8 x 8 x 12 clay brick units):

$A_n = (2)(1.3125 \text{ in.})(12 \text{ in.})(33 \text{ ft}) + (7 + 2 \times 3 \text{ cells})(25.6 \text{ in.}^2 \text{/cell with adjacent webs})$
$= 1{,}372 \text{ in.}^2$

Shear strength is determined as described in Sec. 9.2 using *Provisions* Eq. 11.7.2.1 [ACI 520, Sec. 3.13] and *Provisions* Eq. 11.7.3.1-1 [ACI 530, Eq. 3-18], respectively:

$V_u \leq \phi V_n$

$V_n = V_m + V_s$

For Levels 1 through 6 using *Provisions* Eq. 11.7.3.1-3 where $\dfrac{M_x}{V_x d} > 1.0$:

$$V_n(\max) = 4\sqrt{f'_m}A_n = (4)(\sqrt{2{,}500})(4{,}806) = 961 \text{ kips} \qquad (P3)$$

For Levels 7 through 12 where $\frac{M_x}{V_x d}$ varies from 0.30 to 1.14:

V_n (max) varies from $5.87\sqrt{f'_m}A_n$ to $4\sqrt{f'_m}A_n$

Therefore, V_n (max) varies from $5.87\sqrt{3{,}000}(1836) = 590$ kips to $4\sqrt{3{,}000}(1836) = 402$ kips

depending on the value of $\frac{M_x}{V_x d}$. The masonry shear strength is computed as:

$$V_m = \left[4 - 1.75\left(\frac{M}{Vd}\right)\right]A_n\sqrt{f'_m} + 0.25P$$

The shear strength of Wall D, based on the aforementioned formulas and the strength reduction factor of $\phi = 0.8$ for shear from *Provisions* Table 11.5.3 [ACI 530, Sec. 3.1.4.3] , is summarized in Table 9.3-8. V_x and M_x in this table are values from Table 9.3-2 multiplied by 0.148 (the portion of direct and torsional shear assigned to Wall D). P is the dead load of the roof or floor multiplied by the tributary area for Wall D, and d is the wall length, not height (d = 32.67 ft for Wall D).

The demand shear, Vu, is found by amplifying the loads to a level that produces a moment of 125 percent of the nominal flexural strength at the base of the wall. Given the basic flexural demand of 30,600 ft-kips, a design resistance of 33,840 ft-kips, $\varphi = 0.85$, and the 1.25 factor, the overall amplification of design load is 1.63.

Table 9.3-8 Shear Strength for Wall D

Level	V_x/wall (kips)	M_x/wall (ft-kips)	M_x/V_xd	$1.63V_x$	ϕV_n *max* (kips)	ϕV_m (kips)	P (kips)	ϕV_m (kips)	Req'd ϕV_s (kips)
12	49	500	0.309	80	351.2	OK	37	215	-
11	102	1500	0.446	166	329.3	OK	80	210	-
10	149	3000	0.610	243	303.0	OK	124	201	42
9	191	4900	0.777	311	276.2	NG	168	192	119
8	228	7200	0.957	372	247.4	NG	212	182	190
7	260	9800	1.142	424	240.5	NG	255	186	238
6	292	12700	1.318	476	665.3	OK	308	436	40
5	323	15900	1.492	526	665.3	OK	370	448	78
4	347	19400	1.694	566	665.3	OK	432	461	106
3	364	23000	1.915	593	665.3	OK	494	473	120
2	375	26800	2.166	611	665.3	OK	556	485	126
1	380	30600	2.440	619	665.3	OK	618	498	121

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

Note that 1.63 V_x exceeds $V_{n\,max}$ at Levels 7, 8, and 9. The next most economical solution appears to be to add grout to increase A_n and, therefore, both V_m and $V_{n\,max}$. Check Level 7 using solid grout:

A_n = (7.5 in.)(33 ft)(12 in./ft) = 2970 in.^{2}

$V_n\,(\max) = 4\sqrt{f'_m}A_n = (4)(\sqrt{2{,}500})(2{,}970) = 650$ kips

$\varphi V_n\, max$ = 0.8(650) = 520 kips > 424 kips OK

$$\phi V_m = (0.80)\left[\left(4.0 - 1.75(1.0)\right)2{,}970\sqrt{3000}/1000 + 0.25(255)\right] = 344 \text{ kips}$$

$$\varphi V_s = V_u - \varphi V_m = 424 - 344 = 80 \text{ kips}$$

Check minimum reinforcement for capacity. With vertical #5 at 48 in., a reinforcement ratio of 0.00086 is provided. Thus the horizontal reinforcement must exceed (0.0020 - 0.00086)(7.5 in.)(12 in.) = 0.1025 in.^{2}/ft. With story heights of 10 ft., bond beams at 40 in. on center are convenient, which would require 0.34 in.^{2} Therefore, for 2 - #4 at 40 in. on center:

$$\varphi V_s = 0.80(0.5)(0.4 / 40)(60 \text{ ksi})(33 \text{ ft.})(12 \text{ in./ft.}) = 95 \text{ kips} > 80 \text{ kips} \qquad \text{OK}$$

The largest demand for φV_x in the lower levels is 126 kips at level 2. As explained in the design of the lower level walls for flexural and axial loads (Sec. 9.3.3.5.1), horizontal #5 at 22 in. are required to satisfy minimum reinforcement. Given the story height, check for horizontal #5 at 20 in.:

$$\varphi V_s = 0.8(0.5)(0.31 / 20)(60 \text{ ksi})(33 \text{ ft.})(12 \text{ in./ft.}) = 147 \text{ kips} > 126 \text{ kips} \qquad \text{OK}$$

In summary, for shear it is necessary to grout the hollow units at story 7 solid, and to add some grout at stories 8 and 9. Horizontal reinforcement is 2 - #4 in bond beams at 40 in. on center in the upper stories and one #5 at 20 in. on center in the grouted cavity of the lower stories.

9.3.4 Deflections

The calculations for deflection involve many variables and assumptions, and it must be recognized that any calculation of deflection is approximate at best.

Deflections are to be calculated and compared with the prescribed limits set forth by *Provisions* Table 5.2.8 [Table 4.5-1]. Deformation requirements for masonry structures are discussed in *Provisions* Sec. 11.5.4.

The following procedure will be used for calculating deflections:

1. Determine if the wall at each story will crack by comparing M_x (see Table 9.3-6) to M_{cr} where $M_{cr} = S\left(f_r + P_{u_{min}}/A\right)$
2. If $M_{cr} < M_x$, then use cracked moment of inertia and *Provisions* Eq. 11.5.4.3.
3. If $M_{cr} > M_x$, then use $I_n = I_g$ for moment of inertia of wall.
4. Compute deflection for each level.
5. $\delta_{max} = \sum$ story drift

[The specific procedures for computing deflection of shear walls have been removed from the 2003 *Provisions*. ACI 530 does not contain the corresponding provisions in the text, however, the commentary contains a discussion and equations that are similar to the procedures in the 2000 *Provisions*. Based on ACI 530 Sec. 1.13.3.2, the maximum drift for all masonry structures is 0.007 times the story height. Thus, there appears to be a conflict between ACI 530 and 2003 *Provisions* Table 4.5-1.]

For the upper levels (the additional grout required for shear strength is not considered here):

b_e = effective masonry wall width
b_e = [(2 × 1.3125 in.)(356) + (7 cells)(15 in.^{2}/cell)]/(356) = 2.92 in.

$A = A_{wall} + 2A_{bulb}$ = (2.92 in.)(356 in.) + (2)(400 in.^{2}) = 1,840 in.^{2}

[Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1]

Per *Provisions* Eq. 11.3.10.2 [ACI 530, 1.8.2.2]:

$E = 750\, f'_m = 2{,}250$ ksi ($n = 12.89$)

$I_g = I_{wall} + I_{bulb}$

$$I_g = \frac{(2.96)(356)^3}{12} + (2\text{ bulbs})(20\times 20)\left(\frac{376}{2}\right)^2 = 39.4\times 10^6\text{ in.}^4$$

$S = I_g/c = 39.4 \times 10^6/(198) = 199{,}000$ in.^{3}
$f_r = 0.250$ ksi
$P_{u_{min}} = 1.00D$ (see Table 9.3-6.)

For the lower levels:

$A = A_{wall} + 2A_{bulb} = (10.5\text{ in.})(348\text{ in.}) + (2)(576\text{ in.}^2) = 4{,}806\text{ in.}^2$
$E = 750\, f'_m = 1{,}875$ ksi ($n = 15.47$)
$I = I_{wall} + I_{bulb}$

$$I_g = \frac{(10.5)(348)^3}{12} + (2\text{ bulbs})(24\times 24)\left(\frac{372}{2}\right)^2 = 76.7\times 10^6\text{ in.}^4$$

$S = I_g / c = 76.7 \times 10^6/(198) = 387{,}000$ in.^{3}
$f_r = 0.250$ ksi
$P_{u_{min}} = 1.00D$ (see Table 9.3-6.)

Table 9.3-9 provides a summary of these calculations.

Table 9.3-9 Cracked Wall Determination

Level	$P_{u_{min}}$ (kips)	M_{cr} (ft-kips)	M_x (ft-kips)	Status
12	37	7,620	500	uncracked
11	80	8,820	1,500	uncracked
10	124	8,950	3,000	uncracked
9	168	9,620	4,900	uncracked
8	212	10,300	7,200	uncracked
7	255	11,000	9,800	uncracked
6	308	15,400	12,700	uncracked
5	370	16,000	15,900	uncracked
4	432	16,700	19,400	cracked
3	494	17,300	23,000	cracked
2	556	18,000	26,800	cracked
1	618	18,600	30,600	cracked

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

For the uncracked walls at the upper levels:

$I_n = I_g = 39.4 \times 10^6$ in.^{4}

For the uncracked walls at the lower levels:

$I_n = I_g = 76.7 \times 10^6$ in.^{4}

For the cracked walls at the lower levels, the determination of I_{cr} will be for the load combination of 1.0D + 0.5L. The 0.5L represents an average condition of live load. Making reference to Figure 9.3-6, it can be observed that at this level of P_u, the point on the ϕP_n - ϕM_n curve is near the "intermediate point" previously determined. This is where c = 3.0 ft. (The actual c dimension will be very close to 3.0 ft). For this case, and referring to Figure 9.3-5, the cracked moment of inertia is:

$$I_{cr} = I_{bulb} + I_{wall} + I_{nAs}$$
$$= [24^4 + (24 \times 24)(24)^2\,] + [10.5 \times 12^3/3\,] + [(15.47 \times 16 \text{ in.}^2)(29 \text{ ft} \times 12)^2]$$
$$= 30.3 \times 10^6 \text{ in}^4.$$

Note that 98.9 percent of the value comes from one term: the reinforcement in the tension bulb. If the distributed #5 bars is added to this computation, the value becomes 31.5×10^6 in.^{4}

With the other masonry examples, the interpolation between gross and cracked section properties was used. The application of that is less clear here, where the properties step at midheight, so two analyses are performed. First, each story is considered to be cracked or uncracked. Second is the author's interpretation of the effective moment of inertia equation as:

For all the cracked walls (*Provisions* Eq. 11.5.4.3 [ACI 530, Commentary Sec. 3.1.5.3]):

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n$$

$$I_{eff} = \left(76.7\times10^6\right)\left(\frac{18,600}{30,600}\right)^3 + \left(30.6\times10^6\right)\left[1-\left(\frac{18,600}{30,600}\right)^3\right] = 41.0\times10^6 \text{ in.}^4$$

The entire 12-story Wall D will be treated as a stepped, vertical masonry cantilever shear wall. For the lower step (Levels 1-6), $I_{eff} = 41.0 \times 10^6$ in.^{4} Even though the upper walls are uncracked, I_{eff} of the upper step (Levels 7-12), will be I_n reduced in the same proportion as the lower levels:

$$I_{eff} = (39.4\times10^6)\left(\frac{41.0\times10^6}{76.7\times10^6}\right) = 21.1\times10^6 \qquad \text{(upper levels)}$$

Both the deflections and the fundamental period can now be found. Two RISA 2D analyses were run, and the deflections shown in Table 9.3-10 were obtained. The deflection from the RISA 2D analysis at each level is multiplied by C_d (= 3.5) to determine the inelastic deflection at each level. From these, the story drift, Δ, at each level can be found.

The periods shown in the table validate the period of T = 0.75 sec previously used to determine the base shear in Sec. 9.3.3.2.

Table 9.3-10 Deflections for ELF Analysis (inches)

	Using gross and cracked properties,	Using effective moment of inertia

	story by story (T = 0.798 sec)				(T = 0.755 sec)			
Level	Elastic	Total	Drift	Ratio	Elastic	Total	Drift	Ratio
12	3.40	11.90			3.14	10.99		
11	3.04	10.65	1.25		2.77	9.71	1.28	
10	2.68	9.38	1.27	1.01	2.40	8.41	**1.30**	1.01
9	2.32	8.10	1.27	1.00	2.04	7.13	1.28	0.99
8	1.95	6.84	1.26	0.99	1.68	5.87	1.26	0.98
7	1.60	5.60	1.24	0.98	1.34	4.68	1.19	0.95
6	1.26	4.41	1.19	0.96	1.02	3.58	1.10	0.92
5	0.95	3.34	1.07	0.90	0.76	2.65	0.93	0.85
4	0.66	2.31	1.03	0.96	0.52	1.82	0.83	0.90
3	0.40	1.40	0.91	0.88	0.32	1.11	0.71	0.85
2	0.20	0.68	0.71	0.78	0.16	0.55	0.56	0.79
1	0.06	0.20	0.48	0.67	0.05	0.17	0.38	0.68
0	0	0.00	0.20	0.42	0	0.00	0.17	0.44

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm

The two methods give comparable results. The maximum building deflection is compared to the maximum deflection permitted by *Provisions* Sec. 11.5.4.1.1 as follows:

$C_d\delta_{max}$ = 11.90 in. < 14.4 in. = $0.1h_n$ OK

The maximum story drift occurs at Story 11 and is compared to the maximum story drift permitted by *Provisions* Table 5.2.8 [Table 4.5-1] as follows:

Δ = 1.30 in. > 1.20 in. = $0.01h_{sx}$ NG

Although this indicates a failure to satisfy the *Provisions*, in the author's opinion the drift is satisfactory for two reasons. First the MRS analysis shows smaller drifts (Table 9.3-11) that are within the criteria. On a more fundamental level, however, the authors believe the basic check for drift of a masonry wall is performed according to *Provisions* Sec. 11.5.4.1.1, which applies only to the total displacement at the top of the wall, and that the story drift for any particular story is more properly related to the values for non-masonry buildings. That limit is 0.020 h_{sx}, or 2.4 in. per story. For a building with a torsional irregularity, *Provisions* Sec. 5.4.6.1 [Sec. 4.5.1] requires that the story drift be checked at the plan location with the largest drift, which would be a corner for this building. That limit is satisfied for this building, both by ELF and MRS analyses.

Table 9.3-11 Displacements from Modal Analysis, inches

	At corner of floor plate with maximum displacements. Story drift would be pertinent, although not at 0.010			At wall with maximum in-plane displacement. Roof limit for masonry would be pertinent.		
Level	Elastic	Total	Approx. Drift	Elastic	Total	Approx. Drift
12	2.48	**8.67**		1.91	**6.70**	
11	2.20	7.69	0.98	1.69	5.90	0.79
10	1.91	6.70	0.99	1.48	5.17	0.74
9	1.63	5.70	**1.00**	1.26	4.40	**0.77**
8	1.35	4.72	0.98	1.04	3.64	0.76
7	1.08	3.77	0.95	0.83	2.91	0.73
6	0.83	2.90	0.88	0.64	2.23	0.68
5	0.62	2.16	0.74	0.48	1.67	0.57
4	0.43	1.50	0.66	0.33	1.16	0.51
3	0.27	0.93	0.57	0.21	0.72	0.44
2	0.14	0.48	0.46	0.11	0.37	0.35
1	0.05	0.16	0.32	0.04	0.12	0.25
0	0.00	0.00	0.16	0.00	0.00	0.12

1.0 kip = 4.45 kN, 1.0 in. = 25.4 mm.

The drifts in Table 9.3-11 are not the true modal drifts. The values are computed from the modal sum maximum displacements, rather than being a modal sum of drifts in each mode. The values in the table are less than the true value.

Both tables also confirm that the change in stiffness at midheight does not produce a stiffness irregularity. *Provisions* Sec. 5.2.3.3, Exception 1 [Sec. 4.3.2.3, Exception 1], clarifies that if the drift in a story never exceeds 130 percent of the drift in the story above, then there is no vertical stiffness irregularity. Note that the inverse does not apply; even though the drift in Story 2 is more than double that in Story 1, it does not constitute a stiffness irregularity.

9.3.5 Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4.6.1.3] states that the bearing walls shall be designed for out-of-plane loads equal to:

$$w = 0.40\, S_{DS}\, W_c \geq 0.1 W_c$$
$$w = (0.40)(1.00)(114 \text{ psf}) = 45.6 \text{ psf} \geq 0.1 W_c$$

Therefore, w = 45.6 psf. Out-of-plane bending, using the strength design method for masonry, for a load of 45.6 psf acting on a 10 ft story height is approximated as 456 ft.-lb. per linear ft of wall. This compares to a computed strength of the wall of 1,600 in.-lb per linear foot of wall, considering only the #5 bars at 4 ft on center. Thus the wall is loaded to 28.5 percent of its capacity in flexure in the out-of-plane direction. The upper wall has the same reinforcement, about 42 percent of the load and about 71 percent of the thickness. Therefore, it will be loaded to a smaller fraction of its capacity. (Refer to Example 9.1 for a more detailed discussion of strength design of masonry walls, including the P-delta effect.)

9.3.6 Orthogonal Effects

In accordance with *Provisions* Sec. 5.2.5.2.2 [Sec. 4.4.2.3], orthogonal interaction effects must be considered for buildings in Seismic Design Category D when the ELF procedure is used. Any out-of-plane effect on the heavily reinforced bulbs is negligible compared to the in-plane effect, so orthogonal effects on the bulbs need not be considered further. Considering only the #5 bars and the load combination of 100 percent of in-plane load plus 30 percent of the out-of-plane load, yields a result that 0.3(0.285), or 8.6 percent of the capacity of the #5 bars is not available for in-plane resistance. Given that the #5 bars contribute about 12 percent to the tension resistance (130 kips, vs 960 kips for the bulb reinforcement), the overall effect is a change of about 1 percent in in-plane resistance, which is negligible.

This completes the design of the transverse Wall D.

9.3.7 Wall Anchorage

The anchorage for the bearing walls must be designed for the force, F_p, determined in accordance with *Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3] as:

$F_p = 0.4S_{DS}W_c$ = (0.4)(1.00)(10 ft)(114 psf) = 456 lb/ft
Minimum force = $0.10W_c$ = (0.10)(10 ft)(114 psf) = 114 lb/ft

Provisions Sec. 5.2.6.3.2 [Sec. 4.6.2.1] references *Provisions* Sec. 6.1.3 [Sec. 6.2.2] for anchorage of walls where diaphragms are not flexible. For the lower wall:

$$F_p = \frac{0.4a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2z/h\right) = \frac{0.4(1.0)(1.0)(10\,\text{ft.} \times 114\,\text{psf})}{2.5/1.0}\left(1 + 2(0.5)\right) = 364 \text{ lb./ft.}$$

Therefore, design for 456 lb/ft. For a 2 ft-6 in. joist spacing, the anchorage force at each joist is F_p = 1,140 lb.

Refer to Figure 9.3-12 for the connection detail. A 3/16-in. fillet, weld 2 in. long on each side of the joist seat to its bearing plate will be more than sufficient. Two1/2 in.-diameter headed anchor studs on the bottom of the bearing plate also will be more than sufficient to transfer 4,560 lb into the wall.

9.3.8 Diaphragm Strength

See Example 7.1 for a more detailed discussion on the design of horizontal diaphragms.

To compute the story force associated with the diaphragm on each level, use *Provisions* Eq. 5.2.6.4-4 [Eq. 4.6-2]:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

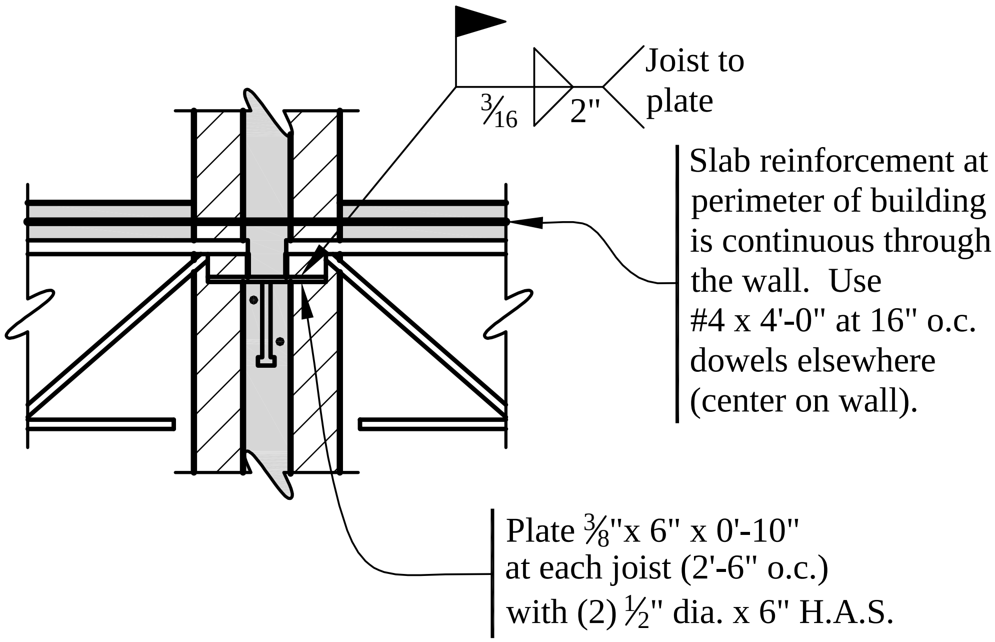


Figure 9.3-12 Floor anchorage to wall (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The results are shown in Table 9.3-12. Note that w_i is approximately the same as w_{px} for this case, the only difference being the weight of the walls perpendicular to the force direction, so the w_i values were used for both.

Table 9.3-12 Diaphragm Seismic Forces

Level	w_i (kips)	F_i (kips)	F_{px} (kips)
12	768	329	329
11	917	357	373
10	917	320	355
9	917	284	336
8	917	249	318
7	917	214	301
6	1109	218	338
5	1300	208	365
4	1300	162	336
3	1300	117	309
2	1300	74	282
1	1300	34	258

1.0 kip = 4.45 kN

The maximum story force is 373 kips. Therefore, use 373 kips/152 ft = 2.45 kips/ft in the transverse direction. The shear in the diaphragm is shown in Figure 9.3-13b. The reaction, R, at each wall pair is 373/4 = 93.25 kips. The diaphragm force at each wall pair is 93.25 kips/(2 × 33 ft) = 1.41 kips/ft.

The maximum diaphragm shear stress is $v = V/td$ = 1410 plf/(2.5 in.)(12 in.) = 47 psi. This compares to an allowable shear of

$$\phi v_c = (0.85)(2)\sqrt{f'_c} = (0.85)(2)\sqrt{3{,}000} = 93 \text{ psi}$$

for 3,000 psi concrete. Thus, no shear reinforcing is necessary. Provide $\rho = 0.0018$ as minimum reinforcement, so A_s = 0.054 in.^{2}/ft. Use WWF 6 × 6-2.9/2.9, which has A_s = 0.058 in.^{2}/ft.

The moment in the diaphragm is shown in Figure 9.3-13c. The maximum moment is 2,460 ft-kips.

Perimeter reinforcement in the diaphragm is determined from:

$T = M/d$ = (2,460 ft-kips)/(72 ft) = 34.1 kips

$A_s = T/\phi F_y$ = 34.1 kips / (0.85)(60 ksi) = 0.67 in.^{2}

Boundary elements of diaphragms may also serve as collectors. The collector force is not usually the same as the chord force. *Provisions* Sec. 5.2.6.4.1 [Sec. 4.6.2.2] requires that collector forces be amplified by Ω_0. Collector elements are required in this diaphragm for the longitudinal direction. A similar design problem is illustrated in Chapter 7 of this volume. Where reinforcing steel withing a topping slab is used for chords or collectors, ACI 318, Sec. 21.9.8 (2002 edition) imposes special spacing and cover requirements. Given the thin slab in this building, the chord reinforcement will have to be limited to bars with couplers at the splices or a thickened edge will be required.

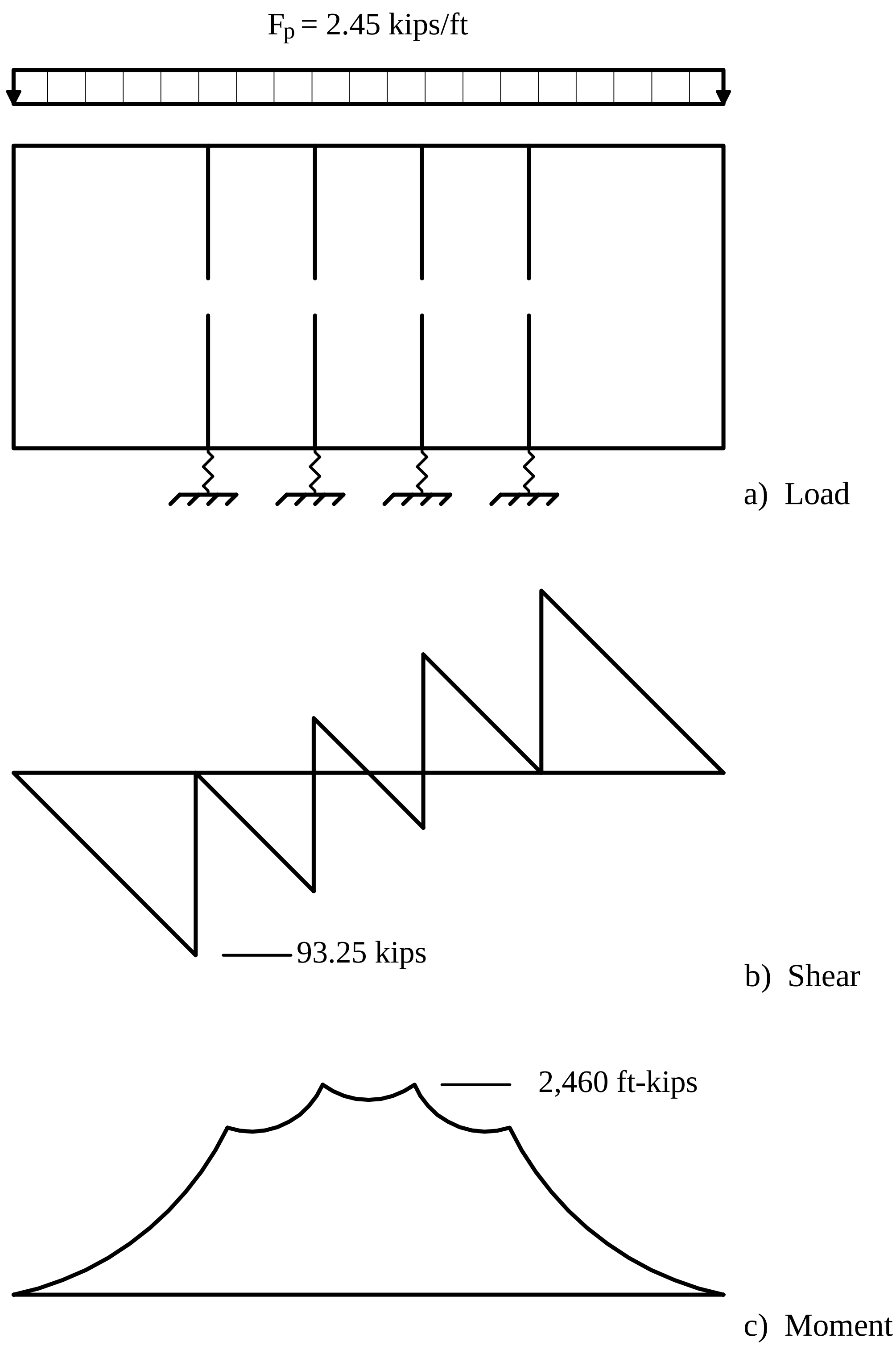


Figure 9.3-13.

Figure 9.3-13 Shears and moments for diaphragm
(1.0 kip/ft = 14.6 kN/m, 1.0 kip = 4.45 kN, 1.0 kip-ft = 1.36kN-m)

10

WOOD DESIGN

Peter W. Somers, P.E. and Michael Valley, P.E.

This chapter examines the design of a variety of wood building elements. Section10.1 features a BSSC trial design prepared in the early 1980s as a starting point. Section 10.2 completes the roof diaphragm design for the building featured in Section 9.1. In both cases, only those portions of the designs necessary to illustrate specific points are included.

Typically, the weak links in wood systems are the connections, but the desired ductility must be developed by means of these connections. Wood members have some ductility in compression (particularly perpendicular to grain) but little in tension. Nailed plywood shear panels develop considerable ductility through yielding of nails and crushing of wood adjacent to nails. Because wood structures are composed of many elements that must act as a whole, the connections must be considered carefully to ensure that the load path is complete. "Tying the structure together ," which is as much an art as a science, is essential to good earthquake-resistant construction.

Wood elements often are used in low-rise masonry and concrete buildings. The same basic principles apply to the design of wood elements, but certain aspects of the design (for example, wall-to-diaphragm anchorage) are more critical in mixed systems than in all-wood construction.

Wood structural panel sheathing is referred to as "plywood" in this chapter. As referenced in the 2000 *NEHRP Recommended Provisions*, wood structural panel sheathing includes plywood and other products, such as oriented-strand board (OSB), that conform to the materials standards of Chapter 12. According to *Provisions* Chapter 12, panel materials other than wood structural panel sheathing do not have a recognized capacity for seismic-force resistance in engineered construction.

In addition to the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereafter, the *Provisions* and *Commentary*), the documents edited below are either referenced directly, or are useful design aids for wood construction.

AF&PA Manual	American Forest & Paper Association 1996. *Manual for Engineered Wood Construction (LRFD)*, including supplements, guidelines, and ASCE 16-95, AF&PA.
[AF&PA Wind & Seismic	American Forest and Paper Association. 2001. *Special Design Provisions for Wind and Seismic*, ADS/LRFD Supplement. AF&PA.]

ANSI/AITC A190.1	American National Standards Institute/American Institute of Timber Construction. 1992. *American National Standard for Wood Products: Structural Glued-Laminated Timber*, A190.1. AITC.
APA PDS	American Plywood Association. 1998. *Plywood Design Specification*, APA.
APA 138	American Plywood Association. 1998. *Plywood Diaphragms*, APA Research Report 138. APA.
ASCE 7	American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures*. ASCE.
ASCE 16	American Society of Civil Engineers. 1995. *Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction*. ASCE.
UBC Std 23-2	International Conference of Building Officials. 1997. *UBC Standard 23-2 Construction and Industrial Plywood, Uniform Building Code*. ICBO.
Roark	Roark, Raymond. 1985. *Formulas for Stress and Strain*, 4^{th} Ed. McGraw-Hill.
USGS CD-ROM	United States Geological Survey. 1996. *Seismic Design Parameters*, CD-ROM. USGS.
WWPA Rules	Western Wood Products Association. 1991. *Western Lumber Grading Rules*. WWPA.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets,[ ], indicated both organizational changes (as a result of a reformat of all the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and it's primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

The most significant change to the wood chapter in the 2003 *Provisions* is the incorporation by reference of the AF&PA, ADS/LRFD Supplement, *Special Design Provisions for Wind and Seismic* for design of the engineered wood construction. A significant portion of the 2003 *Provisions* Chapter 12, including the diaphragm and shear wall tables, has been replaced by a reference to this document. This updated chapter, however, does not result in significant technical changes , as the Supplement, (referred to herein as AF&PA Wind&Seismic) is in substantial agreement with the 2000 *Provisions.* There are, however, some changes to the provisions for perforated shear walls, which are covered in Section 10.1

Some general technical changes in the 2003 *Provisions* that relate to the calculations and/or design in this chapter include updated seismic hazard maps, changes to the Seismic Design Category classification for short period structures, revisions to the redundancy requirements, revisions to the wall anchorage design requirement for flexible diaphragms, and a new "Simplified Design Procedure" that could be applicable to the examples in this chapter.

Where the affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

10.1 THREE-STORY WOOD APARTMENT BUILDING; SEATTLE, WASHINGTON

This example features a wood frame building with plywood diaphragms and shear walls. It is based on a BSSC trial design by Bruce C. Olsen, Structural Engineer, Seattle, Washington.

10.1.1 BUILDING DESCRIPTION

This three-story, wood frame apartment building has plywood floor diaphragms and shear walls. The building has a double-loaded central corridor. Figure 10.1-1 shows a typical floor plan, and Figure 10.1-2 shows a longitudinal section and elevation. The building is located in a neighborhood a few miles north of downtown Seattle. The site coordinates for determining the seismic design parameters are 47.69° N, 122.32° W.

The shear walls in the longitudinal direction are located on the exterior faces of the building and along the corridor. (In previous versions of this volume of design examples, the corridor walls were gypsum wallboard sheathed shear walls; however, gypsum wallboard sheathing, is no longer recognized for engineered design of shear walls per *Provisions* Sec. 12.3.5. Therefore, plywood shear walls are provided at the corridors.) The entire solid (non-glazed) area of exterior walls plywood sheathing, but only a portion of the corridor walls will require sheathing. For the purposes of this example, assume that each corridor wall will have a net of 55 ft of plywood (the reason for this is explained later). In the transverse direction, the end walls and one line of interior shear walls provide lateral resistance. (In previous versions of this example, only the end walls were shear walls. The interior walls now are required for control of diaphragm deflections given the increased seismic ground motion design parameters for the Seattle area.)

The floor and roof systems consist of wood joists supported on bearing walls at the perimeter of the building, the corridor lines, plus one post-and-beam line running through each bank of apartments. Exterior walls are framed with 2×6 studs for the full height of the building to accommodate insulation. Interior bearing walls require 2×6 or 3×4 studs on the corridor line up to the second floor and 2×4 studs above the second floor. Apartment party walls are not load bearing; however, they are double walls and are constructed of staggered, 2×4 studs at 16 in. on center. Surfaced, dry (seasoned) lumber, is used for all framing to minimize shrinkage. Floor framing members are assumed to be composed of Douglas Fir-Larch material, and wall framing is Hem-fir No. 2, as graded by the WWPA. The material and grading of other framing members associated with the lateral design is as indicated in the example. The lightweight concrete floor fill is for sound isolation, and is interrupted by the party walls, corridor walls, and bearing walls.

The building is founded on interior footing pads, continuous strip footings, and grade walls (Figure 10.1-3). The depth of the footings, and the height of the grade walls, are sufficient to provide crawl space clearance beneath the first floor.

The building is typical of apartment construction throughout the western United States, and has the weight necessary to balance potential overturning forces in the transverse direction. If the ground floor were a slab-on-grade, however, the resulting shallower grade wall might well require special attention, due to the possibility of overturning on some of the shear wall units.

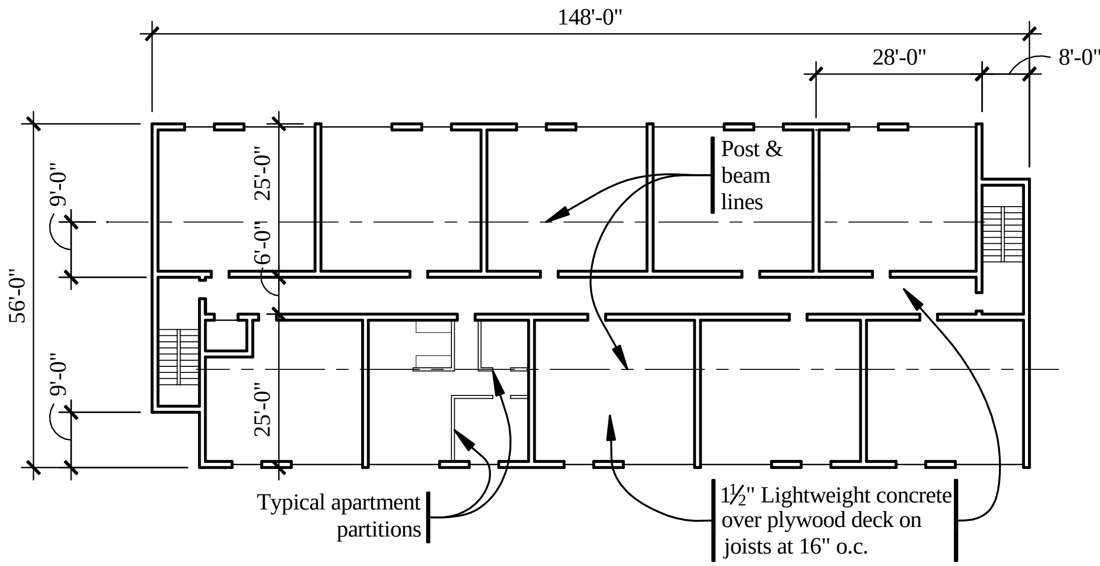


Figure 10.1-1 Typical floor plan (1.0 ft = 0.3048 m).

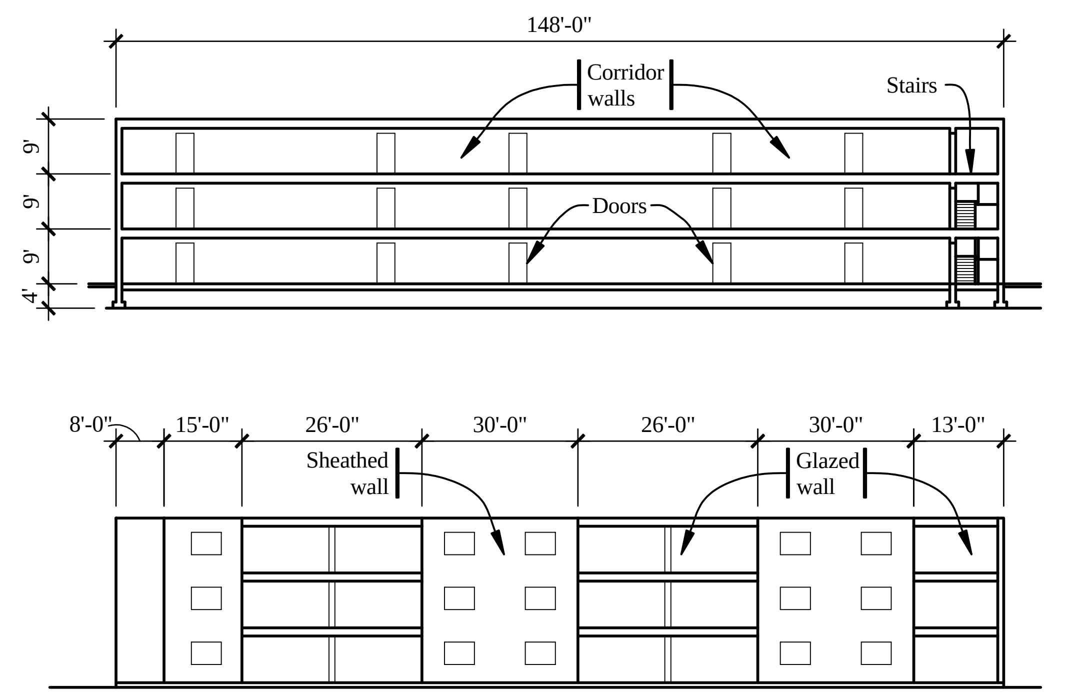


Figure 10.1-2 Longitudinal section and elevation (1.0 ft = 0.3048 m).

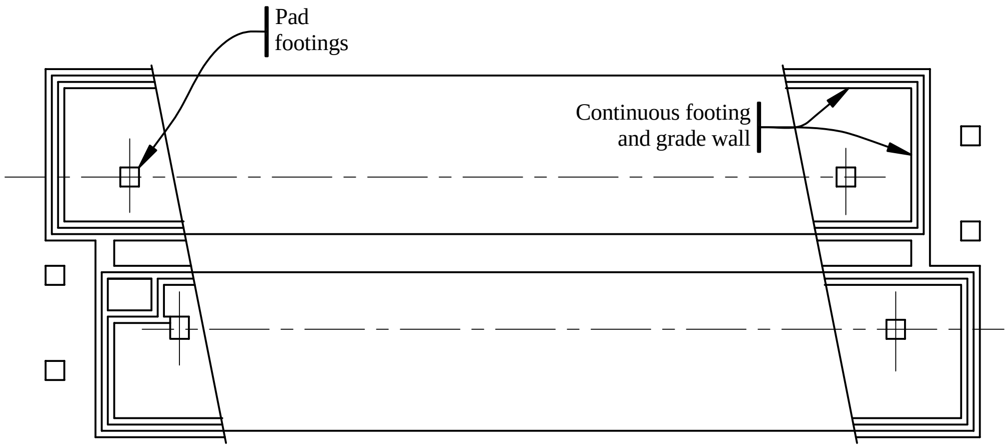


Figure 10.1-3 Foundation plan.

10.1.1.1 Scope

In this example, the structure is designed and detailed for forces acting in the transverse and longitudinal directions. However, greater attention is paid to the transverse direction, because of diaphragm flexibility. The example includes the following

1. Development of equivalent static loads, including torsional effects on plywood diaphragms,
2. Design and detailing of transverse plywood walls for shear and overturning moment,
3. Design and detailing of plywood floor and roof diaphragms,
4. Design and detailing of wall and diaphragm chord members,
5. Detailed deflection and P-delta calculations, and
6. Design and detailing of longitudinal plywood walls using the requirements for perforated shear walls.

[Note that the new "Simplified Design Procedure" contained in 2003 *Provisions* Simplified Alternate Chapter 4 as referenced by 2003 *Provisions* Sec. 4.1.1 is likely to be applicable to this example, subject to the limitations specified in 2003 *Provisions* Sec. Alt. 4.1.1.]

10.1.2 BASIC REQUIREMENTS

10.1.2.1 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	= I
Occupancy Importance Factor, *I* (*Provisions* Sec. 1.4 [1.3])	= 1.0
Site Coordinates	= 47.69° N, 122.32° W
Short Period Response, S_S (USGS CD-ROM)	= 1.34
One Second Period Response, S_1 (USGS CD-ROM)	= 0.46
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	= D
Seismic Design Category (*Provisions* Sec. 4.2 [1.4])	= D
Seismic-Force-Resisting System (*Provisions* Table 5.2.2 [4.3-1])	= Wood panel shear wall
Response Modification Coefficient, *R* (*Provisions* Table 5.2.2 [4.3-1)	= 6.5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [4.3-1)	= 3
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [4.3-1)	= 4

10.1.2.2 Structural Design Criteria

10.1.2.2.1 Ground Motion (Provisions Sec. 4.1.2 [3.3])

Based on the site location, the spectral response acceleration values can be obtained from either the seismic hazard maps accompanying the *Provisions* or from the USGS CD-ROM. For site coordinates 47.69° N, 122.32° W, the USGS CD-ROM returns short period response, S_S = 1.34 and one second period response, S_1 = 0.46.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). A CD-ROM containing the site response parameters based on the 2002 maps is also available.]

The spectral response factors are then adjusted for the site class (*Provisions* Sec. 4.1.2.4 [3.5]). For this example, it is assumed that a site class recommendation was not part of the soils investigation, which would not be uncommon for this type of construction. When soil properties are not known, *Provisions* Sec. 4.1.2.1 [3.5] defaults to Site Class D, provided that soft soils (Site Class E or F) are not expected to be present at the site (a reasonable assumption for soils sufficient to support a multistory building on shallow spread footings). The adjusted spectral response acceleration parameters are computed, according to *Provisions* Eq. 4.1.2.4-1 and 4.1.2.4-2 [3.3-1 and 3.3-4], for the short period and one second period, respectively, as:

$$S_{MS} = F_a S_S = 1.0(1.34) = 1.34$$
$$S_{M1} = F_v S_1 = 1.54(0.46) = 0.71$$

where F_a and F_v are site coefficients defined in *Provisions* Tables 4.1.2.4a and 4.1.2.4b [3.3-1 and 3.3-2], respectively. Note that Straight line interpolation was used for F_v.

Finally, the design spectral response acceleration parameters (*Provisions* Sec. 4.1.2.5 [3.3.3]) are determined in accordance with *Provisions* Eq. 4.1.2.5-1 and 4.1.2.5-2 [3.3-3 and 3.3-4], for the short period and one second period, respectively, as:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.34) = 0.89$$
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.71) = 0.47$$

10.1.2.2.3 Seismic Design Category (Provisions Sec. 4.2 [1.4])

Based on the Seismic Use Group and the design spectral response acceleration parameters, the Seismic Design Category is assigned to the building based on *Provisions* Tables 4.2.1a and 4.2.1b [1.4-1 and 1.4-2]. For this example, the building is assigned Seismic Design Category D.

[Note that the method for assigning seismic design category for short period buildings has been revised in the 2003 *Provisions*. If the fundamental period, T_a, is less than 0.8T_s, the period used to determine drift is less than T_s, and the base shear is computed using 2003 *Provisions* Eq 5.2-2, then seismic design category is assigned using just 2003 *Provisions* Table 1.4-1 (rather than the greater of 2003 *Provisions* Tables 1.4-1 and 1.4-2). The change does not affect this example.]

10.1.2.2.4 Load Path (Provisions Sec. 5.2.1 [14.2-1])

See Figure 10.1-4. For both directions, the load path for seismic loading consists of plywood floor and roof diaphragms and plywood shear walls. Because the lightweight concrete floor topping is discontinuous at each partition and wall, it is not considered to be a structural diaphragm.

10.1.2.2.5 Basic Seismic-Force-Resisting Systems (Provisions Sec. 5.2.2 [4.3])

Building Class (*Provisions* Table 5.2.2 [4.3-1]): Bearing Wall System

Seismic-Force-Resisting System (*Provisions* Table 5.2.2 [4.3-1]): Light frame walls with shear panels with R = 6.5, Ω_0 = 3, and C_d = 4 for both directions

10.1.2.2.6 Structure Configuration (Provisions Sec. 5.2.3 [5.3.2])

Diaphragm Flexibility (*Provisions* Sec. 5.2.3.1 [4.3.2.1]): Rigid (wood panel diaphragm in light frame structure with structural panels for shear resistance). *Provisions* Sec. 12.4.1.1 [4.3.2.1]defines a structural panel diaphragm as flexible, if the maximum diaphragm deformation is more than two times the average story drift. Due to the central shear wall, this is not expected to be the case in this building.

Plan Irregularity (*Provisions* Sec. 5.2.3.2 [4.3.2.2]): Since the shear walls are not balanced for loading in the transverse direction (see Figure 10.1-4), there will be some torsional response of the system. The potential for torsional response combined with the rigid diaphragm requires that the building be checked for a torsional irregularity (*Provisions* Table 5.2.3.2 [4.3-2]). This check will be performed following the initial determination of seismic forces, and the final seismic forces will be modified if required.

Vertical Irregularity (*Provisions* Sec. 5.2.3.3 [4.3.2.3]): Regular

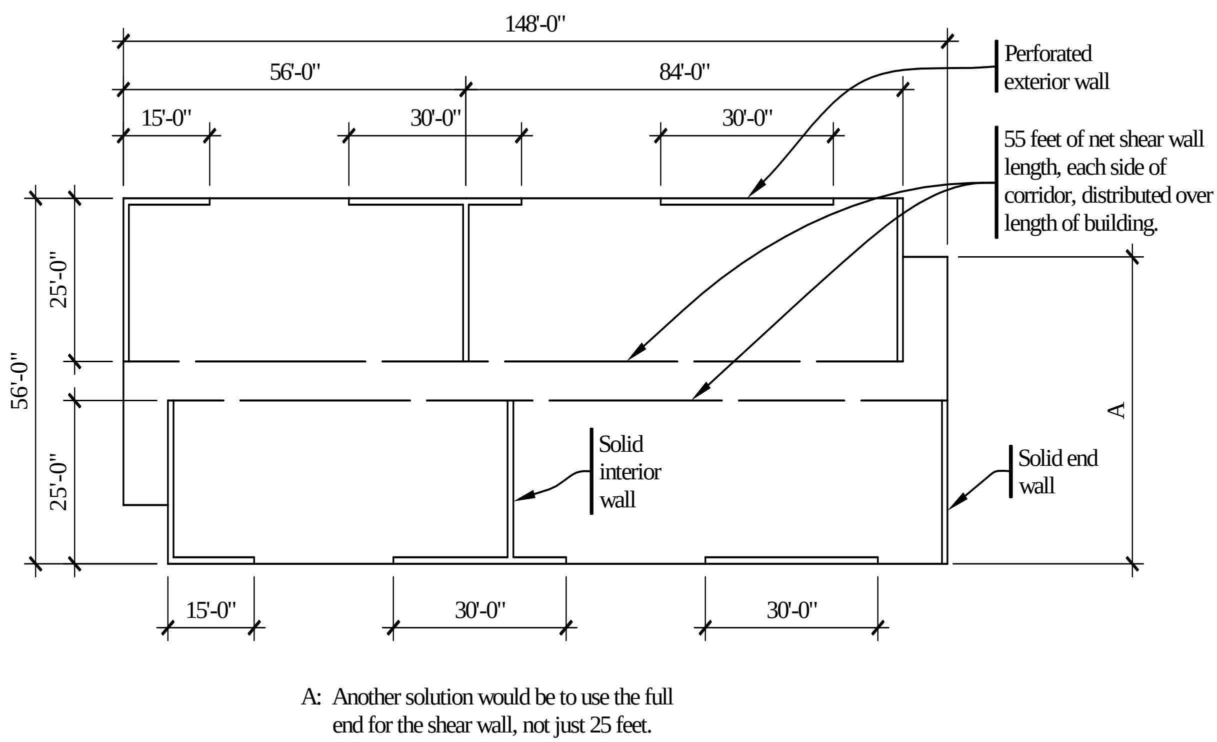


Figure 10.1-4 Load path and shear walls (1.0 ft = 0.3048 m).

10.12.2.7 Redundancy (Provisions Sec. 5.2.4 [4.3.3])

For Seismic Design Category D, the reliability factor, ρ, is computed per *Provisions* Eq. 5.2.4.2 [4.3.3.2]. Since the computation requires more detailed information than is known prior to the design, assume ρ = 1.0 for the initial analysis and verify later. If, in the engineer's judgement, the initial design appears to possess relatively few lateral elements, the designer may wish to use an initial ρ greater than 1.0 (but no greater than 1.5).

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. The intent is that the aspect ratio is based on story height, not total height. Therefore, the redundancy factor would have to be investigated only in the longitudinal direction where the aspect ratios of the perforated shear walls would be interpreted as having aspect ratios greater than 1.0 at individual piers. In the longitudinal direct, where the aspect ratio is less than 1.0, ρ = 1.0 by default.]

10.1.2.2.8 Analysis Procedure (Provisions Sec. 5.2.5 [4.4.1])

Design in accordance with the equivalent lateral force (ELF) procedure (*Provisions* Sec. 5.4 [5.2]): No special requirements. In accordance with *Provisions* Sec. 5.2.5.2.3 [4.4.2], the structural analysis must consider the most critical load effect due to application of seismic forces in any direction for structures assigned to Seismic Design Category D. For the ELF procedure, this requirement is commonly satisfied by applying 100 percent of the seismic force in one direction, and 30 percent of the seismic force in the perpendicular direction; as specified in *Provisions* Sec. 5.2.5.2.2, Item a [4.4.2.2, Item 1]. For a light-framed shear wall building, with shear walls in two orthogonal directions, the only element affected by this directional combination would be the design of the shear wall end post and tie-down, located where the ends of two perpendicular walls intersect. In this example, the requirement only affects the shear wall intersections at the upper left and lower left corners of Figure 10.1-4. The directional requirement is satisfied using a two-dimensional analysis in the design of the remainder of the shear wall and diaphragm elements.

10.1.2.2.9 Design and Detailing Requirements (Provisions Sec. 5.2.6 [4.6])

See *Provisions* Chapters 7 and 12 for special foundation and wood requirements, respectively. As discussed in greater detail below, *Provisions* Sec. 12.2.1, now utilizes Load and Resistance Factor Design (LRFD) for the design of engineered wood structures. Therefore, the design capacities are consistent with the strength design demands of *Provisions* Chapter 5 [4 and 5].

10.1.2.2.10 Combination of Load Effects (Provisions Sec. 5.2.7 [4.2.2])

The basic design load combinations are as stipulated in ASCE 7 and modified by the *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-1]. Seismic load effects according to the *Provisions* are:

$$E = \rho Q_E + 0.2S_{DS}D$$

and

$$E = \rho Q_E - 0.2S_{DS}D$$

when seismic and gravity are additive and counteractive, respectively.

For S_{DS} = 0.89 and assuming ρ = 1.0 (both discussed previously), the design load combinations are as stipulated in ASCE 7:

$$1.2D + 1.0E + 0.5L + 0.2S = 1.38D + 1.0Q_E + 0.5L + 0.2S$$

and

$$0.9D - 1.0E = 0.72D - 1.0Q_E$$

10.1.2.2.11 Deflection and Drift Limits (Provisions Sec. 5.2.8 [4.5.1])

Assuming that interior and exterior finishes have not been designed to accommodate story drifts, then the allowable story drift is (*Provisions* Table 2.5.8 [4.5-1]):

$$\Delta_a = 0.020\, h_{sx}$$

where interstory drift is computed from story drift as (*Provisions* Eq. 5.4.6.1 [5.2-15]):

$$\Delta_x = \delta_x - \delta_{x-1} = \frac{C_d\left[\delta_{xe} - \delta_{(x-1)e}\right]}{I}$$

where C_d is the deflection amplification factor, I is the occupancy importance factor, and δ_{xe} is the total elastic deflection at Level x.

10.1.2.3 Basic Gravity Loads

Roof:

Live/Snow load (in Seattle, snow load governs over roof live load; in other areas this may not be the case)	= 25 psf
Dead load (including roofing, sheathing, joists, insulation, and gypsum ceiling)	= 15 psf

Floor:

Live load	= 40 psf
Dead load (1½ in. lightweight concrete, sheathing, joists, and gypsum ceiling. At first floor, omit ceiling but add insulation.)	= 20 psf

Interior partitions, corridor walls (8 ft high at 11 psf)	= 7 psf distributed floor load
Exterior frame walls (wood siding, plywood sheathing, 2×6 studs, batt insulation, and 5/8-in. gypsum drywall)	= 15 psf of wall surface
Exterior double glazed window wall	= 9 psf of wall surface
Party walls (double-stud sound barrier)	= 15 psf of wall surface
Stairways	= 20 psf of horiz. projection

Perimeter footing (10 in. by 1 ft-4 in.) and grade beam (10 in. by 3 ft-2 in.) = 562 plf

Corridor footing (10 in by 1 ft-4 in.) and grade wall (8 in. by 1 ft-3 in.); 18 in. minimum crawl space under first floor = 292 plf

Applicable seismic weights at each level

W_{roof} = area (roof dead load + interior partitions + party walls) + end walls + longitudinal walls = 182.8 kips

W_3 = W_2 = area (floor dead load + interior partitions + party walls) + end walls + longitudinal walls = 284.2 kips

Effective total building weight, W = 751 kips

For modeling the structure, the first floor is assumed to be the seismic base, because the short crawl space with concrete foundation walls is quite stiff compared to the superstructure.

10.1.3 SEISMIC FORCE ANALYSIS

The analysis is performed manually following a step-by-step procedure for determining the base shear (*Provisions* Sec. 5.4.1 [5.2.1]), and the distribution of vertical (*Provisions* Sec. 5.4.3 [5.2.3]) and horizontal (*Provisions* Sec. 5.4.4 [5.2.4]) shear forces. Since there is no basic irregularity in the building mass, the horizontal distribution of forces to the individual shear walls is easily determined. These forces need only be increased to account for accidental torsion (see subsequent discussion).

No consideration is given to soil-structure interaction since there is no relevant soil information available; a common situation for a building of this size and type. The soil investigation ordinarily performed for this type of structure is important, but is not generally focused on this issue. Indeed, the cost of an investigation sufficiently detailed to permit soil-structure interaction effects to be considered, would probably exceed the benefits to be derived.

10.1.3.1 Period Determination and Calculation of Seismic Coefficient (*Provisions* Sec. 5.4.1 [5.2.1])

Using the values for S_{D1}, S_{DS}, R, and I from Sec. 10.1.2.1, the base shear is computed per *Provisions* Sec. 5.4.1 [5.2.1]. The building period is based on *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = 0.237$$

where C_r = 0.020, h_n = 27 ft, and x = 0.75.

According to *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.89}{6.5/1.0} = 0.137$$

but need not exceed *Provisions* Eq. 5.4.1.1-2 [5.2-3]:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.47}{(0.237)(6.5/1.0)} = 0.305$$

Although it would probably never govern for this type of structure, also check minimum value according to *Provisions* Eq. 5.4.1.1-3:

$C_s = 0.044IS_{DS} = (0.044)(1.0)(0.89) = 0.039$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

The calculation of actual T as based on the true dynamic characteristics of the structure would not affect C_s; thus, there is no need to compute the actual period because the *Provisions* does not allow a calculated period that exceeds C_uT_a where C_u = 1.4 (see *Provisions* Sec. 5.4.2 [5.2.2]). Computing the base shear coefficient, per *Provisions* Eq. 5.4.1.1-1 [5.2-2], using this maximum period, would give C_s = 0.218, which still exceeds 0.137. Therefore, short period response governs the seismic design of the structure, which is common for low-rise buildings.

10.1.3.2 Base Shear Determination

According to *Provisions* Eq. 5.4.1 [5.2-1]:

$V = C_sW = 0.137(751) = 103$ kips (both directions)

where effective total weight is W = 751 kips as computed previously.

10.1.3.3 Vertical Distribution of Forces

Forces are distributed as shown in Figure 10.1-5, where the story forces are calculated according to *Provisions* Eq. 5.4.3-1 and 5.4.3-2 [5.2-10 and 5.2-11]:

$$F_x = C_{vx}V = \left(\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \right) V$$

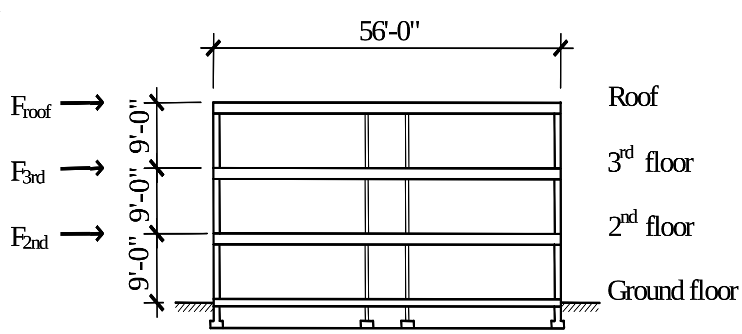


Figure 10.1-5 Vertical shear distribution (1.0 ft = 0.3048 m).

For $T < 0.5$, $k = 1.0$ and $\sum w_i h_i^k = 182.2(27) + 284.2(18) + 284.2(9) = 12,610$

F_{roof} = [182.8(27)/12,608](103.2) = 40.4 kips
F_{3rd} = [284.2(18)/12,608](103.2) = 41.9 kips
F_{2nd} = [284.2(9)/12,608](103.2) = 20.9 kips
Σ = 103 kips

It is convenient and common practice, to perform this calculation along with the overturning moment calculation. Such a tabulation is given in Table 10.1-1.

Table 10.1-1 Seismic Coefficients, Forces, and Moments

Level x	W_x (kips)	h_x (ft)	$w_x h_x^k$ (k = 1)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
Roof	182.8	27	4,936	0.391	40.4		
						40.4	364
3	284.2	18	5,115	0.406	41.9		
						82.3	1,104
2	284.2	9	2,557	0.203	20.9		
						103.2	2,033
Σ	751.2		12,608				

1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip= 1.36 kN-m.

10.1.3.4 Horizontal Distribution of Shear Forces to Walls

Since the diaphragms are defined as “rigid” by the *Provisions* (Sec 5.2.3.1 [4.3.2.1] and 12.4.1.1), the horizontal distribution of forces must account for relative rigidity of the shear walls, and horizontal torsion must be included. As discussed below, for buildings with a Type 1a or 1b torsional irregularity (per *Provisions* Sec. 5.2.3.2 [4.3.2.2]) the torsional amplification factor (*Provisions* Sec. 5.4.4.3 [5.2.4.3]) must be calculated.

It has been common practice for engineers to consider wood diaphragms as flexible, regardless of the relative stiffness between the walls and the diaphragms. Under the flexible diaphragm assumption, loads are distributed to shear walls based on tributary area, without taking diaphragm continuity, or relative wall rigidity into account. Recognizing that diaphragm stiffness should not be ignored (even for wood structural panel sheathing), the *Provisions* provides limits on when the flexible diaphragm assumption may be used and when it may not be used.

The calculation of horizontal force distribution for rigid diaphragms, can be significantly more laborious than the relatively simple tributary area method. Therefore, this example illustrates some simplifications that can be used as relatively good approximations (as confirmed by using more detailed calculations). The design engineer is encouraged to verify all simplifying assumptions that are used to approximate rigid diaphragm force distribution.

For this example, forces are distributed as described below.

10.1.3.4.1 Longitudinal Direction

Based on the rigid diaphragm assumption, force is distributed based on the relative rigidity of the longitudinal walls, and the transverse walls are included for resisting torsion. By inspection, however, the center of mass coincides with the center of rigidity and , thus, the torsional demand is just the accidental torsional moment resulting from a 5 percent eccentricity of force from the center of mass (*Provisions* Sec. 5.4.4.2 [5.2.4.2]).

In this direction, there are four lines of resistance and the total torsional moment is relatively small. Although the walls are unequal in length, the horizontal distribution of the forces can be simplified by making two reasonable assumptions. First, for plywood shear walls, it is common to assume that stiffness is proportional to net in-plane length of sheathing (assuming sheathing thickness, nailing, and chord elements are roughly similar). Second, for this example, assume that all of the torsional moment is resisted by the end walls in the transverse directions. This is a reasonable assumption because the walls have a greater net length than the longitudinal exterior shear walls and are located much farther from the center of rigidity (and thus contribute more significantly to the rotational resistance).

Therefore, direct shear is distributed in proportion to wall length and torsional shear is neglected. Each exterior wall has 45 ft net length and each corridor wall has 55 ft net length for a total of 200 ft of net shear wall. The approximate load to each exterior wall is (45/200)F_x = 0.225F_x, and the load to each corridor wall is (55/200)F_x = 0.275F_x. (The force distribution was also computed using a complete rigidity model, including accidental torsion, with all transverse and longitudinal wall segments. The resulting distribution is 0.231F_x to the exterior walls and 0.276 F_x to the corridor walls, for a difference of 2.7 percent and 0.4 percent, respectively).

10.1.3.4.2 Transverse Direction

Again, based on the rigid diaphragm assumption, force is to be distributed based on relative rigidity of the transverse walls, and the longitudinal walls are included for resisting torsion because the center of mass does not coincide with the center of rigidity. The torsional demand must be computed. Assuming that all six transverse wall segments have the same rigidity, the distance from the center of rigidity (*CR*) to the center of mass (*CM*) can be computed as:

$$CM - CR = 148/2 - \frac{4+60+144}{3} = 4.67\text{ft}$$

The accidental torsional moment resulting from a 5 percent eccentricity of force from the center of mass (*Provisions* Sec. 5.4.4.2 [5.2.4.2]) must also be considered.

As in the longitudinal direction, the force distribution in the transverse direction can be computed with reasonable accuracy by utilizing a simplified model. This simplification is made possible largely because the transverse wall segments are all of the same length and, thus, the same rigidity (assuming nailing and chord members are also the same for all wall segments). First, although the two wall segments at each line of resistance are offset in plan (Figure 10.1-4), assume that the wall segments do align, and are located at the centroid of the two segments. Second, assume that the longitudinal walls do not resist the torsional moment. Third, since interior transverse wall segments are located close to the center of rigidity, assume that they do not contribute to the resistance of the torsional moment.

To determine the forces on each wall, split the seismic force into two parts: that due to direct shear and that due to the torsional moment. Because their rigidities are the same, the direct shear is resisted by all six wall segments equally and is computed as 0.167F_x. The torsional moment is resisted by the four end wall segments. Including accidental torsion, the total torsional moment is computed as:

Total Torsional Moment, $M_t + M_{ta} = F_x \times (4.67 \text{ ft} + 0.05 \times 148 \text{ ft}) = 12.07F_x$
Torsional Shear to End Walls = $12.07F_x/140 \text{ ft} = 0.086F_x$

Therefore, the simplified assumption yields a total design force of $0.167F_x + 0.086F_x/2 = 0.210F_x$, at each of the end wall segments on the right side of Figure 10.4-1, $0.167F_x - 0.086F_x/2 = 0.124F_x$ at each of the end wall segments on the left side of the figure, and $0.167F_x$ at each interior wall segment.

Next, determine the relationship between the maximum and average story drifts, to determine if a torsional irregularity exists, as defined in *Provisions* Table 5.2.3.2 [4.3-2]. Assuming that all of the shear walls will have the same plywood and nailing, the deflection of each wall will be proportional to the force in that wall. Therefore, the maximum drift for any level, will be proportional to $0.210F_x$, and the average drift proportional to $(0.210F_x + 0.124F_x)/2 = 0.167F_x$. The ratio of maximum to average deflection is $0.210F_x/0.167F_x = 1.26$. Since the ratio is greater than 1.2, the structure is considered to have a torsional irregularity Type 1a, in accordance with *Provisions* Table 5.2.3.2 [4.3-2]. Therefore, the horizontal force distribution must be computed again using the torsional amplification factor, A_x, from *Provisions* Sec. 5.4.4.3 [5.2.4.3]. (Diaphragm connections and collectors, not considered in this example, must satisfy *Provisions* Sec. 5.2.6.4.2. [4.6.3.2])

In accordance with *Provisions* Eq 5.4.4.3-1 [5.2-13]:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 = \left[\frac{0.210F_x}{1.2(0.167F_x)}\right]^2 = 1.10$$

Therefore, recompute the torsional moment and forces as:

Total torsional moment, $A_x(M_t + M_{ta}) = 1.10[F_x \times (4.67 \text{ ft} + 0.05 \times 148 \text{ ft})] = 13.28F_x$
Torsional shear to End walls = $13.28F_x/140 \text{ ft} = 0.095F_x$

The revised simplified assumption yields a maximum total design force of $0.167F_x + 0.095F_x/2 = 0.214F_x$ at the end walls (right side of Figure 10.4-1), which will be designed in the following sections.

(The force distribution was also computed using a complete rigidity model including all transverse and longitudinal wall segments. Relative rigidities were based on length of wall and the torsional moment was resisted based on polar moment of inertia. The resulting design forces to the shear wall segments were $0.208F_x$, $0.130F_x$, and $0.168F_x$ for the right end, left end, and interior segments, respectively. The structure is still torsionally irregular ($\delta_{max} / \delta_{ave} = 1.23$), but the irregularity is less substantial and the resulting $A_x = 1.05$. This more rigorous analysis, has shown that the simplified approach provides reasonable results in this case. Since the simplified method is more likely to be used in design practice, the values from that approach will be used for the remainder of this example.)

10.1.3.5 Verification of Redundancy Factor

Once the horizontal force distribution is determined, the assumed redundancy factor must be verified. *Provisions* Eq. 5.2.4.2, is used to compute the redundancy factor, ρ, for each story as:

$$\rho_x = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

where A_x is the floor area in square feet and r_{max_x} is the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear. As defined for shear walls in *Provisions* Sec. 5.2.4.2, r_{max_x} shall be taken as the shear in the most heavily loaded wall, multiplied by $10/l_w$, where l_w is the wall length in feet.

The most heavily loaded wall is the end wall, farthest away from the interior shear wall. Since the force distribution is the same for all three levels, the redundancy factor need only be determined for one level, in this case, at the first floor. The redundancy factor is computed as:

Floor area = (140)(56) = 7,840 ft^2
Max load to wall = 0.214V
Wall length = 25 ft
$r_{max_x} = 0.214V(10/25)/V = 0.0856$

Therefore:

$$\rho_x = 2 - \frac{20}{0.0856\sqrt{7,840}} = -0.64$$

Because the calculated redundancy factor is less than the minimum permitted value of 1.0, the initial assumption of $\rho = 1.0$ is correct, and the design can proceed using the previously computed lateral forces.

[See Sec. 10.1.2.2 for discussion of the changes to the redundancy requirements in the 2003 *Provisions*.]

10.1.3.6 Diaphragm Design Forces

As specified in *Provisions* Sec. 5.2.6.4.4 [4.6.3.4], floor and roof diaphragms must be designed to resist a force, F_{px}, in accordance with *Provisions* Eq. 5.2.6.4.4 [4.6-2]:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

plus any force due to offset walls (not applicable for this example). The diaphragm force as computed above need not exceed $0.4S_{DS}Iw_{px}$, but shall not be less than $0.2S_{DS}Iw_{px}$. This latter value often governs at the lower levels of the building, as it does here. The maximum required diaphragm demand does not govern in this example.

The weight tributary to the diaphragm, w_{px}, need not include the weight of walls parallel to the force. For this example, however, since the shear walls in both directions are relatively light compared to the total tributary diaphragm weight, the diaphragm force is computed based on the total story weight, for convenience.

Transverse direction

Roof Level
$\Sigma F_i = 40.4$
$\Sigma w_i = 182.8$
$F_{P,roof} = (40.4/182.8)(182.8)$ = 40.4 kips (controls for roof)
$0.2S_{DS}Iw_{px} = (0.2)(0.89)(1.0)(182.8)$ = 32.7 kips

Third Floor
ΣF_i = 40.4 + 41.9 = 82.3
Σw_i = 182.8 + 284.2 = 447.0
$F_{P,3rd}$ =(82.3/447.0)(284.2) = 50.1 kips
$0.2S_{DS}Iw_{px}$ = (0.2)(0.89)(1.0)(284.2) = 50.8 kips (controls for 3rd floor)

Second Floor
ΣF_i = 40.4 + 41.9 + 20.9 = 103,231
Σw_i = 182.8 + 284.2 + 284.2 = 751.2
$F_{P,2nd}$ = (103.2/751.2)(284.2) = 39.1 kips
$0.2S_{DS}Iw_{px}$ = (0.2)(0.89)(1.0)(284.2) = 50.8 kips (controls for 2nd floor)

Diaphragm forces in the longitudinal direction are computed in a similar manner. Since the weight of the exterior walls is more significant in the longitudinal direction, the designer may wish to subtract this weight from the story force in order to compute the diaphragm demands.

10.1.4 Basic Proportioning

Designing a plywood diaphragm and plywood shear wall building, principally involves the determination of sheathing thicknesses and nailing patterns to accommodate the applied loads. In doing so, some successive iteration of steps may be needed to satisfy the deflection limits.

Nailing patterns in diaphragms and shear walls have been established on the basis of tabulated requirements included in the *Provisions*. It is important to consider the framing requirements for a given nailing pattern and capacity as indicated in the notes following the tables. In addition to strength requirements, *Provisions* Sec. 12.4.1.2 [12.4.1.1] places aspect ratio limits on plywood diaphragms (length/width shall not exceed 4/1 for blocked diaphragms), and *Provisions* Sec. 12.4.2.3 [12.4.2.3] places similar limits on shear walls (height/width shall not exceed 2/1 for full design capacities). However, it should be taken into consideration that compliance with these aspect ratios does not guarantee that the drift limits will be satisfied.

Therefore, diaphragms and shear walls have been analyzed for deflection as well as for shear capacity. A procedure for computing diaphragm and shear wall deflections is provided in *Commentary* Sec. 12.4.1. This procedure is illustrated below. The *Commentary* does not indicate how to compute the nail deformation (nail slip) factor, but there is a procedure contained in the commentary of ASCE16.

In the calculation of diaphragm deflections, the chord slip factor can result in large additions to the total deflection. This can be overcome by using “neat” holes for bolts, and proper shimming at butt joints. However, careful attention to detailing and field inspection are essential, to ensure that they are provided.

[AF&PA Wind & Seismic also contains procedures for computing diaphragm and shear wall deflections. The equations are slightly different from the more commonly used equations that appear in the *Commentary* and AF&PA LRFD *Manual*. In AF&PA Wind & Seismic, the shear and nail slip terms are combined using an “apparent shear stiffness” parameter. However, the apparent shear stiffness values are only provided for OSB. Therefore, the deflection equations in the *Commentary* or AF&PA LRFD *Manual* must be used in this example which has plywood diaphragms and shear walls. The apparent shear stiffness values for plywood will likely be available in future editions of AF&PA Wind & Seismic.]

10.1.4.1 Strength of Members and Connections

The 2000 *Provisions* has adopted Load and Resistance Factor Design (LRFD) for engineered wood structures. The *Provisions* includes the ASCE 16 standard by reference and uses it as the primary design procedure for engineered wood construction. Strength design of members and connections is based on the requirements of ASCE 16. The AF&PA Manual and supplements contain reference resistance values for use in design. For convenience, the *Provisions* contain design tables for diaphragms and shear walls that are identical to those contained in the AF&PA Structural-Use Panels Supplement. However, the modification of the tabulated design resistance for shear walls and diaphragms with framing other than Douglas Fir-Larch or Southern Pine is different between the two documents. This example illustrates the modification procedure contained in the *Provisions* tables, which is new to the 2000 edition.

Throughout this example, the resistance of members and connections subjected to seismic forces, acting alone, or in combination with other prescribed loads, is determined in accordance with ASCE 16 and the AF&PA Manual; with the exception of shear walls and diaphragms for which design resistance values are taken from the *Provisions*. The LRFD standard incorporates the notation *D*', *T*', *Z*', etc. to represent adjusted resistance values, which are then modified by a time effect factor, λ, and a capacity reduction factor, ϕ, to compute a design resistance, which is defined as "factored resistance" in the *Provisions* and ASCE 16. Additional discussion on the use of LFRD is included in *Commentary* Sec. 12.2 and 12.3, and in the ASCE 16 commentary. It is important to note that ACSE 16 and the AF&PA Manual use the term, "resistance," to refer to the design capacities of members and connections while "strength" refers to material property values.

It is worth noting that the AF&PA Manual contains a Pre-engineered Metal Connections Guideline for converting allowable stress design values for cataloged metal connection hardware (for example, tie-down anchors) into ultimate capacities for use with strength design. This procedure is utilized in this example.

[The primary reference for design of wood diaphragms and shear walls in the 2003 *Provisions* is AF&PA Wind & Seismic. Much of the remaining text in the 2003 *Provisions* results from differences between AF&PA Wind & Seismic and Chapter 12 of the 2000 *Provisions* as well as areas not addressed by AF&PA Wind & Seismic. Because the AF&PA Wind & Seismic tabulated design values for diaphragms and shear walls do not completely replace the tables in the 2000 *Provisions*, portions of the tables remain in the 2003 *Provisions*. Therefore, some diaphragm and shear wall design values are in the 2003 *Provisions* and some are in AF&PA Wind & Seismic. The design values in the tables are different between the two documents. The values in the 2003 *Provisions* represent factored shear resistance ($\lambda\varphi D'$), while the values in AF&PA Wind & Seismic represent nominal shear resistance that must then be multiplied by a resistance factor, φ, (0.65) and a time effect factor ,λ, (1.0 for seismic loads). Therefore, while the referenced tables may be different, the factored resistance values based on the 2003 *Provisions* should be the same as those in examples based on the 2000 *Provisions*. The calculations that follow are annotated to indicate from which table the design values are taken.]

10.1.4.2 Transverse Shear Wall Nailing

The design will focus on the more highly loaded end walls; interior walls are assumed to be similar.

10.1.4.2.1 Load to Any One of Four 25-ft End Walls

$$F_{roof} = 0.214(40.4) = 8.65 \text{ kips}$$
$$F_{3rd} = 0.214(41.9) = 8.97 \text{ kips}$$
$$F_{2nd} = 0.214(20.9) = \underline{4.47 \text{ kips}}$$
$$\Sigma = 22.09 \text{ kips}$$

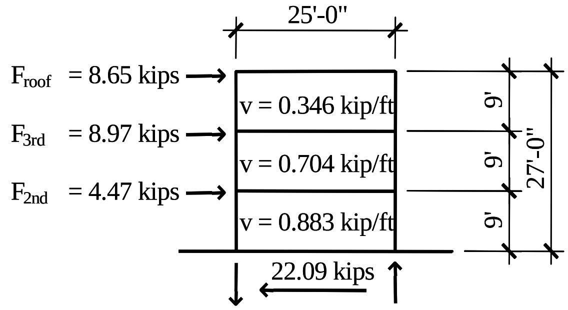


Figure 10.1-6 Transverse section: end wall (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 kip/ft = 14.6 kN/m).

10.1.4.2.2 Roof to Third Floor Design

V = 8.65 kips
v = 8.65/25 = 0.346 klf

Try a ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d common nails at six-in. on center at panel edges and 12 in. on center at intermediate framing members. According to Note b of *Provisions* Table 12.4.3-2a [12.4-3a], the design shear resistance must be adjusted for Hem-fir wall framing. The specific gravity adjustment factor equals 1-(0.5-SG) where SG is the specific gravity of the framing lumber. From Table 12A of the AF&PA Structural Connections Supplement, the SG = 0.43 for Hem-fir. Therefore, the adjustment factor is 1-(0.5-0.43) = 0.93.

From *Provisions* Table 12.4.3-2a [AF&PA Wind/Seismic Table 4.3A], $\lambda\phi D'$ = 0.40 klf
Adjusted shear resistance = 0.93(0.40) = 0.37 klf > 0.346 klf OK

8d nails could be used at this level but, because 10d nails are required below, 10d nails are used here for consistency.

Deflection of plywood shear panels depends, in part, on the slip at the nails which, in turn, depends on the load per nail. See Sec. 10.1.4.3 for a detailed discussion of the appropriate slip for use with the *Provisions* and see Table 10.1-2 for fastener slip equations used here.

Load per nail = 0.346(6/12)(1000) = 173 lb
Nail slip $e_n = 1.2(173/769)^{3.276} = 0.00904$ in.

In the above equation, 1.2 = factor for plywood other than Structural I, and 769 and 3.276 are constants explained in Sec. 10.1.4.3.

[As indicated previously, AF&PA Wind & Seismic does not provide apparent shear stiffness values for plywood sheathing. Therefore, the deflection equations in the *Commentary* or AF&PA LRFD *Manual* must be used in this case.]

10.1.4.2.3 Third Floor to Second Floor

V = 8.65 + 8.97 = 17.62 kips

v = 17.62/25 = 0.704 klf

Try ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d nails at 3-in. on center at panel edges, and at 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A], $\lambda\phi D'$ = 0.78 klf
Adjusted shear resistance = 0.93(0.78) = 0.73 klf > 0.704 klf OK

Table Note e [AF&PA Wind&Seismic Sec. 4.3.7.1] requires 3-in. framing at adjoining panel edges.

Load per nail = 0.704(3/12)(1000) = 176 lb
Nail slip $e_n = 1.2(176/769)^{3.276} = 0.00960$ in.

10.1.4.2 Second Floor to First Floor

V = 17.62 + 4.47 = 22.09 kips
v = 22.09/25 = 0.883 klf

Try a ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d common nails at 2-in. on center at panel edges and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A], $\lambda\phi D'$ = 1.00 klf
Adjusted shear resistance = 0.93(1.00) = 0.93 klf > 0.883 klf OK

Table Notes d and e [AF&PA Wind&Seismic Sec. 4.3.7.1]require 3-in. framing at adjoining panel edges.

Load per nail = 0.883(2/12)(1000) = 147 lb
Nail slip $e_n = 1.2(147/769)^{3.276} = 0.00534$ in.

10.1.4.3 General Note on the Calculation of Deflections for Plywood Shear Panels

The commonly used expressions for deflection of diaphragms and shear walls, which are contained in *Commentary* Sec. 12.4.1, and the ASCE 16 commentary standard (see also UBC Std 23-2 or APA 138), include a term for nail slip. The ASCE 16 commentary includes a procedure for estimating nail slip that is based on LRFD design values.

The values for e_n used in this example (and in Sec. 10.2) are calculated according to Table 10.1-2, which is taken from Table C9.5-1 of ASCE 16.

Table 10.1-2 Fastener Slip Equations

Fastener	Minimum Penetration (in.)	For Maximum Loads Up to (lb.)	Approximate Slip, e_n*	
			Green/Dry	Dry/Dry
6d common nail	1-1/4	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
8d common nail	1-7/16	220	$(V_n/857)^{1.869}$	$(V_n/616)^{3.018}$
10d common nail	1-5/8	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$
14-ga staple	1 to 2	140	$(V_n/902)^{1.464}$	$(V_n/596)^{1.999}$
14-ga staple	2	170	$(V_n/674)^{1.873}$	$(V_n/361)^{2.887}$

*Fabricated green/tested dry (seasoned); fabricated dry/tested dry. V_n = fastener load in pounds. Values based on Structural I plywood fastened to Group II lumber. Increase slip by 20 percent when plywood is not Structural I.

1.0 in. = 25.4 mm, 1.0 lb = 4.45 N.

This example is based on the use of surfaced dry lumber so the "dry/dry" values are used. The appearance of the equations for both shear wall and diaphragm deflection, implies greater accuracy than is justified. The reader should keep in mind that the deflections calculated are only rough estimates.

10.1.4.4 Transverse Shear Wall Deflection

From *Commentary* Sec. 12.4, modified as described below, shear wall deflection is computed as:

$$\delta = \frac{8vh^3}{wEA} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{w}d_a$$

The above equation produces displacements in inches and the individual variables must be entered in the force or length units as described below:

$v = V/w$ where V is the total shear on the wall and v is in units of pounds/foot

$8vh3/wEA$ = bending deflection, as derived from the formula $\delta_b = Vh^3/3EI$, where V is the total shear in pounds, acting on the wall and $I = Aw^2/2$ (in.^{4})

vh/Gt = shear deflection, as derived from the formula $\delta_v = Vh/GA'$, where $A' = wt$ (in.^{2})

$0.75\ he_n$ = nail slip, in inches. Note that with h being given in feet, the coefficient 0.75 carries units of 1/ft.

$(h/w)d_a$ = deflection due to anchorage slip, in inches. Note that for use in the deflection equation contained in the *Commentary*, the term d_a represents the vertical deflection due to anchorage details. In the deflection equation contained in the commentary in the AF&PA Manual, there is no h/w factor, so it should be assumed that the term d_a represents the horizontal deflection at the top of the wall due to anchorage details.

For this example:

E = 1,600,000 psi
G = 75,000 psi
A (area) = 2(2.5)5.5 = 27.5 in.^{2} for assumed double 3×6 end posts

w (shear wall length) = 25 ft
h (story height) = 9 ft
t (effective thickness) = 0.298 in. for ½-in. unsanded plywood (not Structural I)
e_n = nail deformation factor from prior calculations, inches.

This equation is designed for a one-story panel and some modifications are in order for a multistory panel. The components due to shear distortion and nail slip are easily separable (see Table 10.1-3).

Table 10.1-3 Wall Deflection (per story) Due to Shear and Nail Slip

Story	v (plf)	vh/Gt (in.)	e_n (in.)	$0.75e_nh$ (in.)
Roof	346	0.139	0.00904	0.061
3	704	0.284	0.00960	0.065
2	883	0.356	0.00534	0.036

1.0 in. = 25.4 mm, 1.0 plf = 14.6 N/m.

Likewise, the component due to anchorage slip is easily separable; it is a rigid body rotation. If a 1/8-in. upward slip is assumed (on the tension side only), the deflection per story is (9/25)(1/8) = 0.045 in. (Table 10.1-4).

The component due to bending is more difficult to separate. For this example, a grossly simplified, distributed triangular load on a cantilever beam is used (Figure 10.1-7).

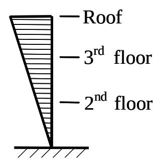


Figure 10.1-7 Force distribution for flexural deflections.

The total load *V* is taken as 21.3 kips, the sum of the story forces on the wall. The equation for the deflection, taken from Roark's *Formulas for Stress and Strain*, is:

$$\delta_x = \frac{2V}{5EAb^2h^2}(11h^5 - 15h^4x + 5hx^4 - x^5)$$

where x is the distance (ft) from the top of the building to the story in question, h is the total height (ft), b is the wall width (ft), A is the chord cross sectional area (in.^{2}), E is the modulus of elasticity (psi), and the resulting displacement, δ, is in inches.

This somewhat underestimates the deflections, but it is close enough for design. The results are shown in Table 10.1-4.

The total wall deflections, shown in Table 10.1-5, are combined with the diaphragm deflections (see Sec. 10.1.4.7, 10.1.4.8, and 10.1.4.9). Drift limits are checked after diaphragm deflections are computed.

Table 10.1-4 Wall Deflection (per story) Due to Bending and Anchorage Slip

Level	Effective $8vh^3/wEA$ (in.)	$(h/w)d_a$ (in.)
Roof	0.031	0.045
3	0.027	0.045
2	0.012	0.045

1.0 in. = 25.4 mm.

Table 10.1-5 Total Elastic Deflection and Drift of End Wall

Level	Shear (in.)	Bending (in.)	Nail Slip (in.)	Anchor Slip (in.)	Drift Δ_e (in.)	Total δ_e (in.)
Roof	0.139	0.031	0.061	0.045	0.277	1.145
3	0.284	0.027	0.065	0.045	0.420	0.869
2	0.356	0.012	0.036	0.045	0.448	0.448

1.0 in. = 25.4 mm.

10.1.4.5 Transverse Shear Wall Anchorage

Provisions Sec. 12.4.2.4 [12.4.2.4.1] requires tie-down (hold-down) anchorage at the ends of shear walls where net uplift is induced. Net uplift is computed as the combination of the seismic overturning moment and the dead load counter-balancing moment using the load combination indicated in *Provisions* Eq. 5.2.7-2 [4.2-2].

Traditionally, tie-down devices were designed to resist this net overturning demand. However, *Provisions* Sec. 12.4.2.4 [12.4.2.4.1] requires that the nominal strength of tie-down devices exceeds the expected maximum vertical force that the shear panels can deliver to the tension-side end post. Specifically, the nominal strength (ϕ = 1.0) of the tie-down device must be equal to, or greater than the net uplift forces resulting from Ω_0/1.3 times the factored shear resistance of the shear panels, where Ω_0 is the system overstrength factor. These uplift forces are cumulative over the height of the building. Also, dead load forces are not used in these calculations to offset the uplift forces for the design of the tie-down anchorage.

[The requirements for uplift anchorage are slightly different in the 2003 *Provisions*. The tie down force is based on the "nominal strength of the shear wall" rather than the Ω_0/1.3 times the factored resistance as specified in the 2000 *Provisions*. This change is primarily intended to provide consistent terminology and should not result in a significant impact on the design of the tie down.]

An additional requirement of *Provisions* Sec. 12.4.2.4 [12.4.2.4.1], is that end posts must be sized such that failure across the net section does not control the capacity of the system. That is, the tensile strength of the net section must be greater than the tie-down strength. Note that the tie-downs designed in the following section are not located at wall intersections where the directional combination requirements of *Provisions* Sec. 5.2.5.2.3 [4.4.2.3] would apply (see also the "Analysis Procedure" in Sec. 10.1.2.2).

10.1.4.5.1 Tie-down Anchors at Third Floor

For the typical 25-ft end wall segment, the overturning moment at the third floor is:

$$M_0 = 9(8.65) = 77.8 \text{ ft-kip} = Q_E$$

The counter-balancing moment, 0.72 Q_D = 0.72D(25 ft/2). The width of the floor contributing to this, is taken as half the span of the exterior window header equal to 6.5 feet (see Figures 10.1-1 and 10.1-2). For convenience, the same length is used for the longitudinal walls, the weight of which (interior and exterior glazed wall) is assumed to be 9 psf.

End wall self weight = 9 ft (25ft)15 psf/1000	=	3.4 kips
Tributary floor = 6.5 ft (25 ft) (15) psf/1000	=	2.4 kips
Tributary longitudinal walls = 9 ft (6.5 ft) 9 psf (2)/1000	=	<u>1.1 kips</u>
Σ	=	6.9 kips

$0.72Q_D$ = 0.72(6.9)12.5 = 62.1 ft-kip

M_0 (net) = 77.8 - 62.1 = 15.7 ft-kip

Therefore, uplift anchorage is required. Using the procedure described above, nominal tie-down strength must be equal to or exceed the tension force, T, computed as:

T = 0.37 klf (3/1.3)(8ft) = 6.83 kips

where

0.37 klf = factored shear wall resistance at the third floor
3/1.3 = Ω_0/1.3
8 ft = net third floor wall height (9-ft story height minus approximately 1 ft of framing).

Use a double tie-down device to eliminate the eccentricity associated with a single tie-down. Also, use the same size end post over the full height of the wall to simplify the connections and alignments. As will be computed below, a 6×6 (Douglas fir-Larch) post is required at the first floor. At the third floor, try two sets of double tie-down anchors connected through the floor with a 5/8-in. threaded rod to the end posts with two 5/8-in. bolts similar to Figure 10.1-8.

According to *Provisions* Sec. 12.4.2.4 [12.4.2.4.1], the nominal tie-down strength is defined as the "maximum test load the device can resist under cyclic testing without connection failure by either metal or wood failure." [Note that this definition for nominal tie down strength has been removed in the 2003 *Provisions*. This change is primarily intended to provide consistent terminology and should not result in a significant impact on the design of the tie down.] For a single tie-down device as described above, the cataloged allowable uplift capacity is 2.76 kips and the cataloged average ultimate load is 12.15 kips. However, according to the documentation in this particular supplier's product catalog, the ultimate values are based on static testing and the type of failure is not indicated. Therefore, for this example, the cataloged ultimate value is not considered to satisfy the requirements of the *Provisions*. As an alternate approach, this example will utilize the methodology contained in the AF&PA Manual for converting cataloged allowable stress values to strength values.

Based on the procedure in the AF&PA Manual, Pre-Engineered Metal Connectors Guide, the strength conversion factor for a connector for which the catalog provides an allowable stress value with a 1.33 factor for seismic is 2.88/1.33.

Since the typical product catalogs provide design capacities only for single tie-downs, the design of double tie-downs requires two checks. First, consider twice the capacity of one tie-down, and second, the capacity of the bolts in double shear. ϕ = 1.0 for both these calculations.

For the double tie-down, the nominal strength is computed as:

$2\lambda\phi Z' = 2(1.0)(1.0)(2.76)(2.88/1.33) = 12.0$ kips > 6.83 kips OK

For the two bolts through the end post in double shear, the AF&PA Manual gives:

$2\lambda\phi Z' = 2(1.0)(1.0)(4.90) = 9.80$ kips > 6.83 kips OK

The factored capacity of the tie-downs must also be checked for the design loads, which in this case will not govern the design.

10.1.4.5.2 Tie-down Anchors at Second Floor

Since tie-downs are required at the third floor, it would be common practice to provide tie-downs at the second and first floors, whether or not calculations indicate that they are required. Nevertheless, the overturning calculations are performed for illustrative purposes. The overturning moment at the second floor is:

$M_0 = 18(8.65) + 9(8.97) = 236$ ft-kip.

The counter-balancing moment, $0.72\ Q_D = 0.72(\text{DL})12.5$.

End wall self weight = 18 ft (25 ft) 15 psf/1000	= 6.75 kips
Tributary floor = 6.5 ft (25 ft) (15 + 27) psf/1000	= 6.83 kips
Tributary longitudinal walls = 18 ft (6.5 ft) 9 psf (2)/1000	= 2.10 kips
Σ	= 15.7 kips

$0.72Q_D = 0.72(15.7)12.5 = 141$ ft-kips

M_0 (net) = 236-141 = 95 ft-kips

As expected, uplift anchorage is required. The design uplift force is computed using an adjusted factored shear resistance of 0.73 klf at the second floor, and a net length of wall height equal to 8 ft. Note that 8 ft. is appropriate for this calculation given the detailing for this structure. As shown in Figure 10.1-10, the plywood sheathing is not detailed as continuous across the floor framing, which results in a net sheathing height of about 8ft. If the sheathing were detailed across the floor framing, then 9 ft would be the appropriate wall height for use in computing tie-down demands. Combined with the uplift force at the third floor, the total design uplift force at the second floor is:

$T = 6.83$ kips + 0.73 klf (3/1.3)(8ft) = 20.3 kips

Use two sets of double tie-down anchors to connect the 6×6 end posts. Using the same procedure as for the third floor, tie-downs with a seven-eighths-in. threaded rod and three seven-eighths-in. bolts are computed to be adequate. See Figure 10.1-8.

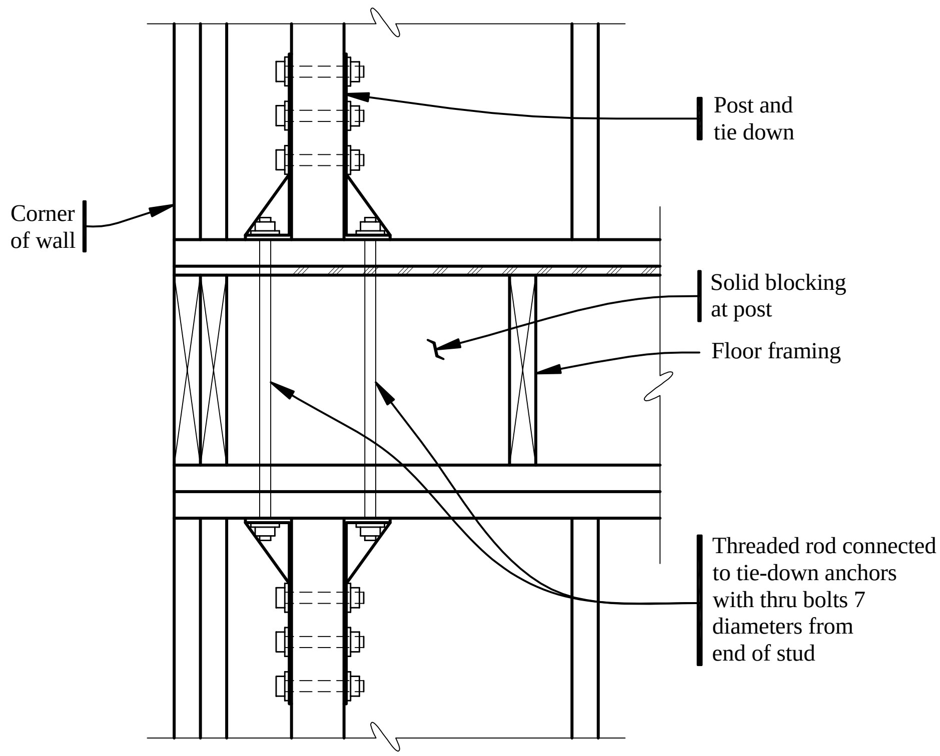


Figure 10.1-8 Shear wall tie down at suspended floor framing.

10.1.4.5.3 Tie-down Anchors at First Floor

The overturning moment at the first floor is:

M_0 = 27(8.65) + 18(8.97) + 9(4.47) = 435 ft-kip = $Q_{E.}$

The counter-balancing moment, 0.72 Q_D = 0.72D(25 ft/2).

End wall self weight = 27 ft (25ft)15 psf/1000	= 10.1 kips
Tributary floor = 6.5 ft (25 ft) (15 + 27 + 27) psf/1000	= 11.2 kips
Tributary longitudinal walls = 27 ft (6.5 ft) 9 psf (2)/1000	= 3.2 kips
Σ	= 24.5 kips

0.72Q_D = 0.72(24.5)12.5 = 221 ft-kip

M_0 (net) = 435 - 221 = 214 ft-kip

As expected, uplift anchorage is required. The design uplift force is computed using an adjusted factored shear resistance of 0.93 klf at the first floor, and a net wall height of 8ft. Combined with the uplift force at the floors above, the total design uplift force at the first floor is:

T = 20.3 kips + 0.93 klf (3/1.3)(8ft.) = 37.5 kips.

Use a double tie-down anchor that extends through the floor with an anchor bolt into the foundation. Tie-downs with a 7/8-in. threaded rod, and four 7/8-in. bolts are adequate.

The strength of the end post, based on failure across the net section, must also be checked (*Provisions* Sec. 12.4.2.4 [12.4.2.4.1]). For convenience, the same size post has been used over the height of the building, so the critical section is at the first floor. A reasonable approach to preclude net tension failure from being a limit state would be to provide an end post, whose factored resistance exceeds the nominal strength of the tie-down device. (Note that using the factored resistance rather than nominal strength of the end post provides an added margin of safety that is not explicitly required by the *Provisions*.) The nominal strength of the first floor double tie-down is 42.9 kips, as computed using the procedure described above. Therefore, the tension capacity at the net section must be greater than 42.9 kips.

Try a 6×6 Douglas Fir-Larch No. 1 end post. In some locations, the shear wall end post also provides bearing for the window header, so this size is reasonable. Accounting for 1-in. bolt holes, the net area of the post is 24.75 in.^{2} According to the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi T'$ = (1.0)(0.8)(2.23ksi)(24.75in^2) = 44.2 kips > 42.9 kips OK

For the maximum compressive load at the end post, combine maximum gravity load, plus the seismic overturning load. In the governing condition, the end post supports the header over the glazed portion of the exterior wall (end wall at right side of Figure 10.1-1). Assume that the end post at the exterior side of the wall supports all the gravity load from the header, and resists one-half of the seismic overturning load.

Compute gravity loads based on a 6.5-foot tributary length of the header:

Tributary DL = ((27 ft)(9 psf) + (8 ft)(15 + 27 + 27)psf)(6.5ft)/1000 = 5.17 kips
Tributary LL = 8 ft (7 ft) (40 + 40) psf/1000 = 4.48 kips
Tributary SL = 8 ft (6.5 ft) (25 psf)/1000 = 1.30 kips

The overturning force is based on the seismic demand (not wall capacity as used for tension anchorage) assuming a moment arm of 23 ft:

Overturning, Q_E = 435/23 ft = 18.9 kips

Per load combination associated with *Provisions* Eq. 5.2.7.1-1 [4.2-1]:

Maximum compression = 1.38(5.17) + 1.0(18.9/2) + 0.5(4.48) + 0.2(1.30) = 19.1 kips.

Due to the relatively short clear height of the post, the governing condition is bearing perpendicular to the grain on the bottom plate. Check bearing of the 6×6 end post on a 3×6 Douglas fir-Larch No. 2 plate, per the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi P'$= (1.0)(0.8)(1.30ksi)(30.25 in.^{2}) = 31.5 kips > 19.1 kips. OK

The 6×6 end post is slightly larger than the double 3×6 studs assumed above for the shear wall deflection calculations. Therefore, the computed shear wall deflection is slightly conservative, but the effect is minimal.

10.1.4.5.4 Check Overturning at the Soil Interface

A summary of the overturning forces is shown in Figure 10.1-9. To compute the overturning at the soil interface, the overturning moment must be increased for the 4-ft foundation height:

M_0 = 435 + 22.09(4.0) = 523 ft-kip

However, it then may be reduced in accordance with *Provisions* Sec. 5.3.6:

M_0 = 0.75(523) = 392 ft-kip

To determine the total resistance, combine the weight above with the dead load of the first floor and foundation.

Load from first floor = 25 ft (6.5 ft) (27 - 4 + 1)psf /1000 = 3.9 kips

where 4 psf is the weight reduction due to the absence of a ceiling, and 1 psf is the weight of insulation.

The length of the longitudinal foundation wall included, is a conservative approximation of the amount carried by minimum nominal reinforcement in the foundation.

Foundation weight = (562 plf (13 ft + 25 ft) + 292 plf (13 ft))/1000	= 25.1 kips
First floor	= 3.9 kips
Structure above	= 24.5 kips
Σ	= 53.5 kips

Therefore, $0.72D - 1.0Q_E$ = 0.72(53.5)12.5 ft - 1.0(392) = 89.5 ft-kips, which is greater than zero, so the wall will not overturn.

10.1.4.5.5 Anchor Bolts for Shear

At the first floor:

v = 0.883 klf.

A common anchorage for non-engineered construction is a ½-in. bolt at 4ft-0 in. For a ½-in. bolt in a Douglas fir-Larch 3×6 plate, in single shear, parallel to the grain:

$\lambda\phi Z'$/4ft = (1.0)(0.65)(2.38 kips)/4 = 0.39 klf < 0.883 klf NG

Try a larger bolt and tighter spacing. For this example, use a 5/8-in. bolt at 32 in. on center:

$\lambda\phi Z'$/2.67ft = (1.0)(0.65)(3.72 kips)/2.67 = 0.91 klf > 0.883 klf OK

Provisions Sec. 12.4.2.4 [12.4.2.4.2]requires plate washers at all shear wall anchor bolts. A summary of the transverse shear wall elements is shown in Figure 10.1-9.

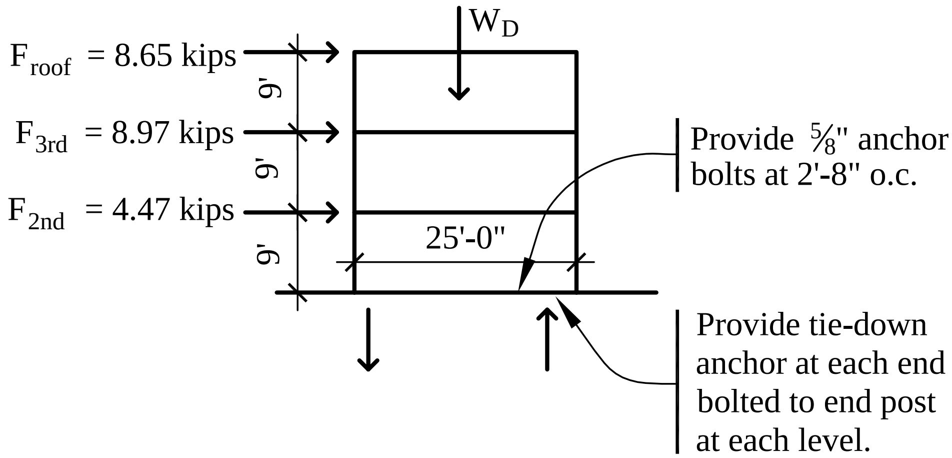


Figure 10.1-9 Transverse wall: overturning
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN).

10.1.4.6 Remarks on Shear Wall Connection Details

In normal platform frame construction, details must be developed that will transfer the lateral loads through the floor system and, at the same time, accommodate normal material sizes and the cross-grain shrinkage in the floor system. The connections for wall overturning in Sec. 10.1.4.5 are an example of one of the necessary force transfers. The transfer of diaphragm shear to supporting shear walls is another important transfer as is the transfer from a shear wall on one level to the level below.

The floor-to-floor height is nine-ft with about one-ft occupied by the floor framing. Using standard 8-ft-long plywood sheets for the shear walls, a gap occurs over the depth of the floor framing. It is common to use the floor framing to transfer the lateral shear force. Figures 10.1-10 and 10.1-11 depict this accomplished by nailing the plywood to the bottom plate of the shear wall, which is nailed through the floor plywood to the double 2×12 chord in the floor system.

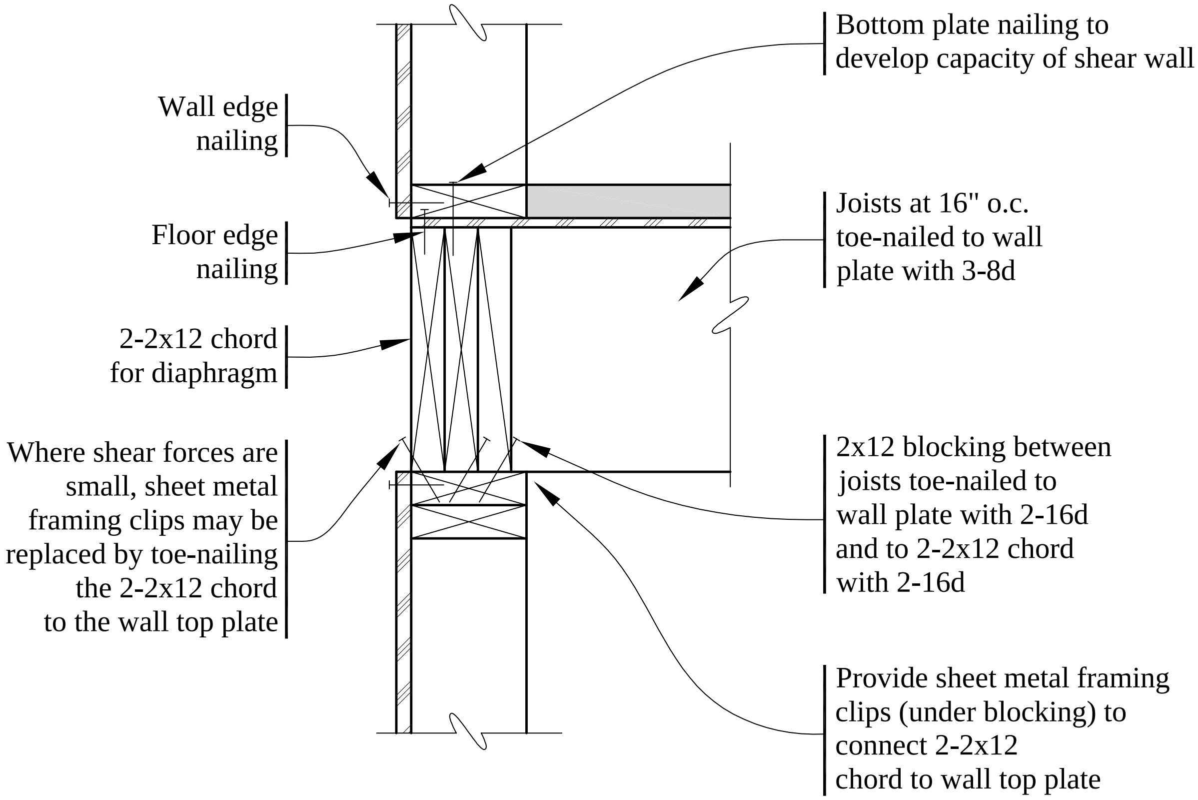


Figure 10.1-10 Bearing wall (1.0 in = 25.4 mm).

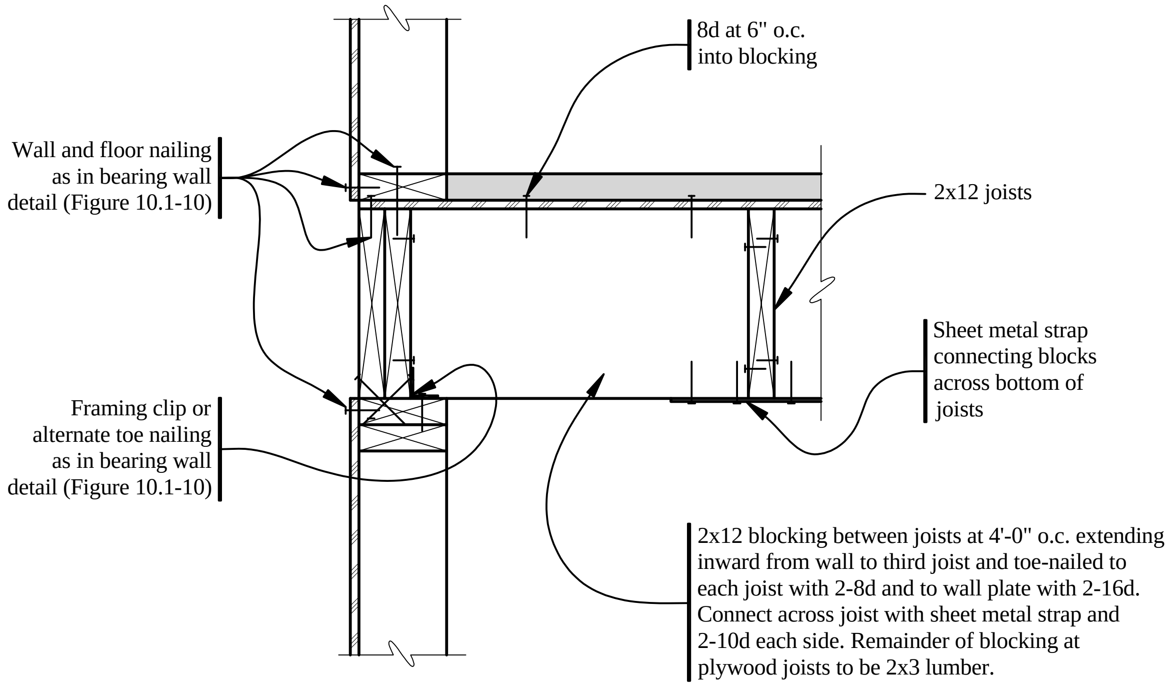


Figure 10.1-11 Nonbearing wall (1.0 in. = 25.4 mm).

The top plate of the lower shear wall also is connected to the double 2×12 by means of sheet metal framing clips to the double 2×12 to transfer the force back out to the lower plywood. (Where the forces are small, using toe nails between the double 2×12 and the top plate may be used for this connection.) This technique leaves the floor framing free for cross-grain shrinkage. Although some designers in the past may have used a short tier of plywood nailed to the plates of the stud walls to accomplish the transfer, *Provisions* Sec. 12.4.2.6 prohibits this type of detailing.

The floor plywood is nailed directly to the framing at the edge of the floor, before the plate for the upper wall is placed. Also, the floor diaphragm is connected directly to framing that spans over the openings between shear walls. The axial strength, and the connections of the double 2×12 chords, allow them to function as collectors to move the force from the full length of the diaphragm to the discrete shear walls. (According to *Provisions* Sec. 5.2.6.4.1 [4.6.2.2], the design of collector elements in wood shear wall buildings in Seismic Design Category D need not consider increased seismic demands due to overstrength.)

The floor joist is toe nailed to the wall below for forces normal to the wall. Likewise, full-depth blocking is provided adjacent to walls that are parallel to the floor joists, as shown in Figure 10.1-11. (Elsewhere the blocking for the floor diaphragm only need be small pieces, flat 2×4s, for example.) The connections at the foundation are similar (see Figure 10.1-12).

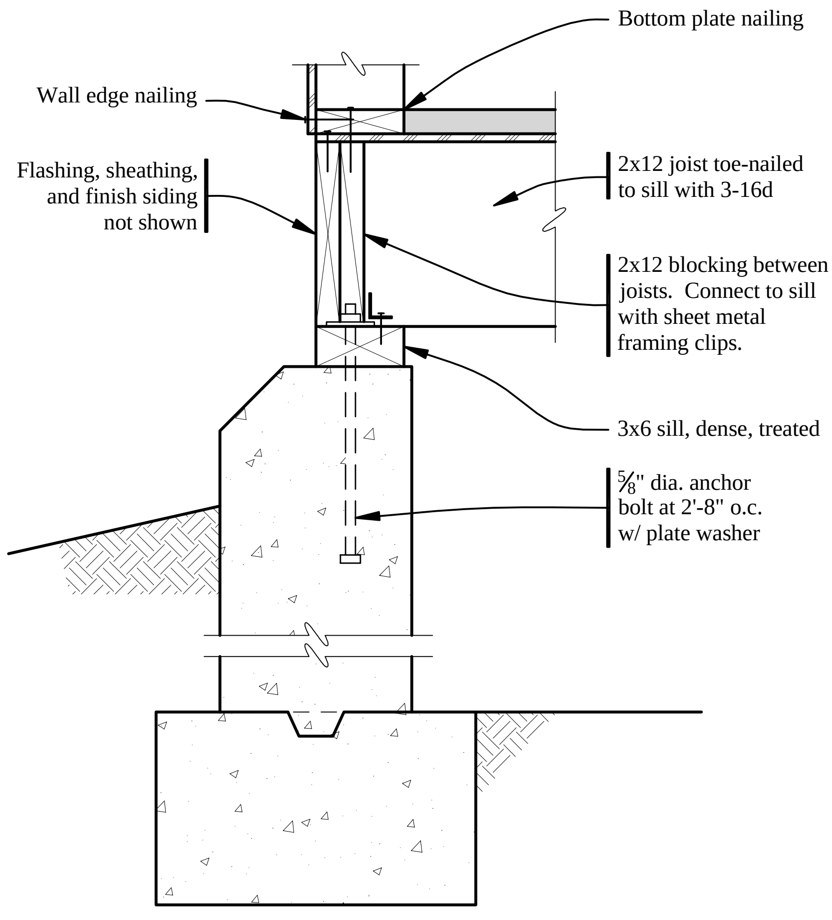


Figure 10.1-12 Foundation wall detail (1.0 in. = 25.4 mm).

The particular combinations of nails and bent steel framing clips shown in Figures 10.1-10, 10.1-11, and 10.1-12, to accomplish the necessary force transfers, are not the only possible solutions. A great amount of leeway exists for individual preference, as long as the load path has no gaps. Common carpentry practices often will provide most of the necessary transfers but, just as often, a critical few will be missed. As a result, careful attention to detailing and inspection is an absolute necessity.

10.1.4.7 Roof Diaphragm Design

While it has been common practice to design plywood diaphragms as simply supported beams spanning between shear walls, the diaphragm design for this example must consider the continuity associated with rigid diaphragms. The design will be based on the maximum shears and moments that occur over the entire diaphragm. From Sec. 10.1.3.5, the diaphragm design force at the roof is, $F_{P,roof}$ = 40.4 kips.

As discussed previously, the design force computed in this example includes the internal force due to the weight of the walls parallel to the motion. Particularly for one-story buildings, it is common practice to remove that portion of the design force. It is conservative to include it, as is done here.

10.1.4.7.1 Diaphragm Nailing

The maximum diaphragm shear occurs at the end walls. From Sec. 10.1.3.4, each 25-ft end wall segment resists 21 percent of the total story (diaphragm, in this case) force. Distributing the diaphragm force at the same rate, the diaphragm shear over the entire diaphragm width at the end walls is:

V = (0.214)(2)(40.4) = 17.3 kips
v = 17.3/56 ft. = 0.308 klf

Try ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center, with 8d nails at 6 in. on center at panel edges and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A], $\lambda\phi D'$ = 0.35 klf > 0.308 klfOK

The determination of nail slip for diaphragms is included below.

10.1.4.7.2 Chord and Splice Connection

Diaphragm continuity is an important factor in the design of the chords. The design must consider the tension/compression forces, due to positive moment at the middle of the span, as well as negative moment at the interior shear wall. It is reasonable (and conservative) to design the chord for the positive moment assuming a simply supported beam and for the negative moment accounting for continuity. The positive moment is $wl^2/8$, where w is the unit diaphragm force, and l is the length of the governing diaphragm span. For a continuous beam of two unequal spans, under a uniform load, the maximum negative moment is:

$$M^- = \frac{wl_1^3 + wl_2^3}{8(l_1 + l_2)}$$

where w is the unit diaphragm force, and l_1 and l_2 are the lengths of the two diaphragm spans. For w = 40.4 kips /140 ft = 0.289 klf, the maximum positive moment is:

$0.289(84)^2 / 8$ = 255 ft-kip

and the maximum negative moment is:

$$\frac{0.289(84)^3+0.289(56)^3}{8(84+56)}=198\text{ ft-kip}$$

The positive moment controls, and the design chord force is 255/56 = 4.55 kips. Try a double 2×12 Douglas fir-Larch No. 2 chord. Due to staggered splices, compute the tension capacity based on a single 2×12, with a net area of A_n = 15.37 in.^{2} (accounting for 1-in. bolt holes). Per the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi T'$ = (1.0)(0.8)(1.55psi)(15.37in^2) = 19.1 kips > 4.55 kips OK

For chord splices, use 4 in. diameter split-ring connectors with 3/4-in. bolts. For split rings is single shear, the capacity of one connector is:

$\lambda\phi T'$ = (1.0)(0.65)(17.1) = 11.1 kips > 4.55 kips. OK

This type of chord splice connection, shown in Figure 10.1-13, is generally used only for heavily loaded chords and is shown here for illustrative purposes. A typical chord splice connection for less heavily loaded chords can be accomplished more easily by using 16d nails to splice the staggered chord members.

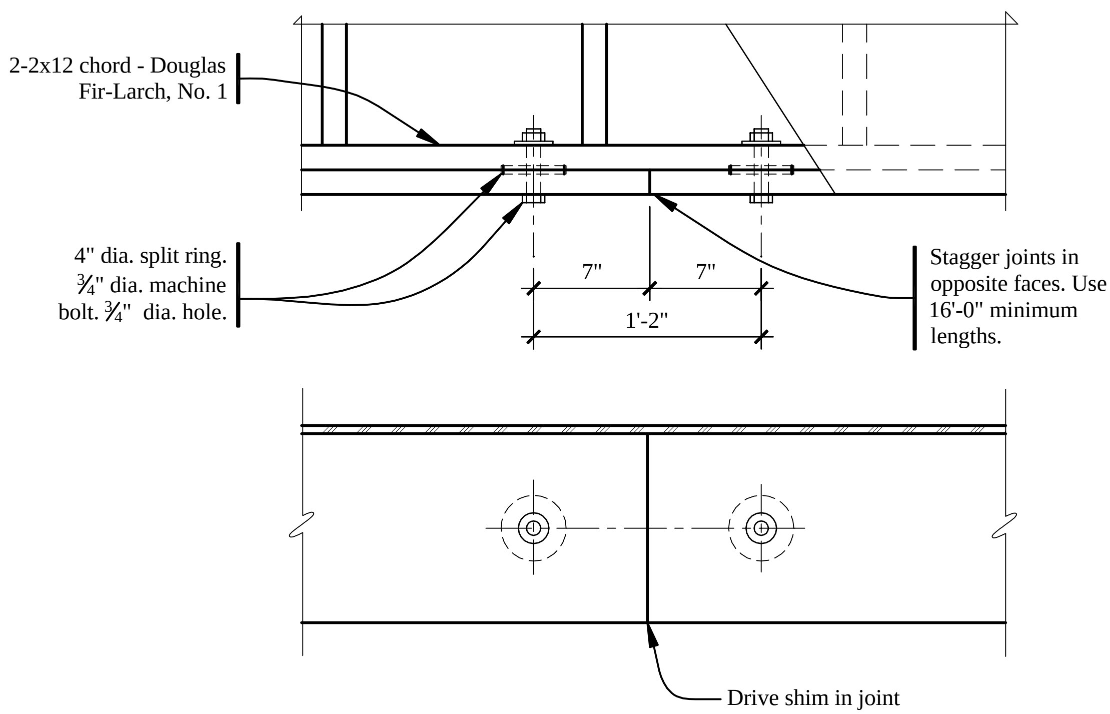


Figure 10.1-13 Diaphragm chord splice (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

10.1.4.7.3 Diaphragm Deflection

The procedure for computing diaphragm deflections, contained in *Commentary* Sec. 12.4 (similar to the commentary of ASCE 16), is intended for the single span, "flexible" diaphragm model that has been used in common practice. The actual deflections of multiple span "rigid" diaphragms may, in general, be similar to those of single-span diaphragms because shear deflection and nail slip (both based on shear demand) tend to dominate the behavior. As multiple span deflection computations tend to be cumbersome, it is suggested that the design engineer compute diaphragm deflections based on the single span model. This will result with a reasonable (and conservative), estimation of overall displacements. If these displacements satisfy the drift criteria, then the design is assumed to be adequate; if not, then a more rigorous computation of displacements could be performed.

It is the authors' opinion that for diaphragm displacement computations, the diaphragm loading should be according to *Provisions* Eq. 5.2.6.4.4 [5.2-11]; the minimum design demand ($0.2S_{DS}Iw_{px}$) need not be considered. Although the *Provisions* does not provide a specific requirement either way, this interpretation is consistent with *Provisions* Sec. 5.4.6.1 [5.2.6.1], which indicates that the minimum base shear equation (*Provisions* Eq. 5.4.1.1-3) need not be used for calculation of story drift.

[See Sec. 10.1.3.1 for a discussion of diaphragm deflection computation using the 2003 Provisions and AF&PA Wind & Seismic reference.]

From *Commentary* Sec. 12.4, the diaphragm deflection is computed as:

$$\delta = \frac{5vL^3}{8wEA} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2w}\ .$$

The equation produces the midspan diaphragm displacement in inches, and the individual variables *must* be entered in the force or length units as described below. For the single span approximation, consider the longer, 84 ft long span so that:

$$v = \frac{F_{px}(84/140)}{2w}$$

where F_{px} is the story diaphragm force at Level x, w is the diaphragm width, and v is in pounds/ft. For this calculation, the unit shear will be the same at both ends of the diaphragm. Therefore, it underestimates the actual unit shear at the end wall but overestimates the actual unit shear at the interior wall. These inaccuracies are assumed to be roughly offsetting. The individual terms of the above equation represent the following:

$5vL^3/8wEA$ = bending deflection, as derived from the formula $\delta_b = 5vL^4/384EI$, where v is the diaphragm unit force in pounds per ft and $I = Aw^2/2$ (in^2)

$vL/4Gt$ = shear deflection as derived from the formula $\delta_v = vL^2/8GA$, where v is the diaphragm unit force in pounds per ft and $A = wt$ (in^2)

$0.188Le_n$ = nail slip deflection in inches

$\Sigma(\Delta_c X)/2w$ = deflection due to chord slip in inches

For this example:

v = 40.4(84/140)/[(2(56))(1000)] = 216 plf (ignoring torsion)
L = 84 ft and w = 56 ft, diaphragm length and width
A = 2(1.5) 11.25 = 33.75 in.^{2} (double 2×12)
t = 0.298 in. for ½-in. unsanded plywood
E = 1,600,000 psi
G =75,000 psi

Nail slip is computed using the same procedure as for shear walls (Sec. 10.1.4.3), with the load per nail based on the diaphragm shear from this section. The diaphragm nailing is 8d at 6 in. on center.

Load per nail = 216(6/12) = 108 lb
Nail slip e_n = 1.2(108/616)$^{3.018}$ = 0.0063 in.

In the above equation, 1.2 is the factor for plywood other than Structural I and 616 and 3.018 are coefficients for seasoned lumber.

Although a tight chord splice has been specified, the chord slip is computed for illustrative purposes. Assume chord slip occurs only at the side tension since the compression splices are tightly shimmed as shown in Figure 10.1-13. For this example:

Δ_c = chord splice slip (1/16 in. used for this example)
X = distance from chord splice to nearest support

Assuming splices at 20 ft on center, along the 84-ft diaphragm length (ignore diaphragm continuity for this term), the sum of the chord splice slip is:

$\Sigma(\Delta_c X)$ = (1/16)(20 + 40 + 24 + 4) = 5.5

Thus:

$$\delta = \frac{5(216)(84^3)}{8(1{,}600{,}000)(33.75)(56)} + \frac{216(84)}{4(75{,}000)(0.298)} + 0.188(84)(0.0063) + \frac{5.5}{2(56)}$$

$$= 0.027 + 0.203 + 0.100 + 0.055 = 0.385 \text{ in.}$$

Wall and floor drifts are added, and checked in Sec. 10.1.4.9.

10.1.4.8 Second and Third Floor Diaphragm Design

The design of the second and third floor diaphragms follows the same procedure as the roof diaphragm. From Sec. 10.1.3.6, the diaphragm design force for both floors is $F_{p,3rd} = F_{p,2nd}$ = 50.8 kips.

10.1.4.8.1 Diaphragm Nailing

The maximum diaphragm shear occurs at the end walls. From Sec. 10.1.3.4, each end wall segment resists 21 percent of the total story (diaphragm, in this case) force. Distributing the diaphragm design force at the same rate, the diaphragm shear at the end walls is:

V = (0.214)(2)(50.8) = 21.7 kips
v = 21.7/56 ft = 0.388 klf

Try ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center, with 8d nails at 4 in. on center at boundaries and continuous panel edges, at 6 in. on center at other panel edges, and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A], $\lambda\varphi D'$ = 0.47 klf > 0.388 klf OK

10.1.4.8.2 Chord and Splice Connection

Computed as described above for the roof diaphragm, the maximum positive moment is 320 kips and the design chord force is 5.71 kips.

By inspection, a double 2×12 chord spliced with 4-in. diameter split ring connectors, as at to the roof level, is adequate. A typical chord splice connection is shown in Figure 10.1-13.

10.1.4.8.3 Diaphragm Deflection

Using the same procedure as before:

L = 84 ft and w = 56 ft, diaphragm length and width
A = 2(1.5) 11.25 = 33.75 in.^{2} (double 2×12)
t = 0.298 in. for ½-in. unsanded plywood
E = 1,600,000 psi
G = 75,000 psi

As discussed previously, diaphragm deflection computations need not consider the minimum diaphragm design forces.

Therefore, at the third floor:

v = 50.1(84/140)/[(2(56))(1000)] = 268 plf
Load per nail = 268(4/12) = 89 lb
Nail slip $e_n = 1.2(89/616)^{3.018} = 0.0035$ in.

$$\delta = \frac{5(268)(84^3)}{8(1,600,000)(33.75)(56)} + \frac{268(84)}{4(75,000)(0.298)} + 0.188(84)(0.0035) + \frac{5.5}{2(56)}$$

$$= 0.033 + 0.252 + 0.056 + 0.055 = 0.396 \text{ in.}$$

At the second floor:

v = 39.1(84/140)/(2(56))(1000) = 209 plf
Load per nail = 209(4/12) = 70 lb
Nail slip $e_n = 1.2(70/616)^{3.018} = 0.0017$ in.

$$\delta = \frac{5(209)(84^3)}{8(1,600,000)(33.75)(56)} + \frac{209(84)}{4(75,000)(0.298)} + 0.188(84)(0.0017) + \frac{5.5}{2(56)}$$

$$= 0.026 + 0.197 + 0.026 + 0.055 = 0.304 \text{ in.}$$

10.1.4.9 Transverse Deflections, Drift, and P-delta Effects

Transverse deflections for walls and diaphragms were calculated above. The diaphragm deflections for the second and third floors are based on the seismic force analysis and not the minimum diaphragm design forces (*Provisions* Sec. 5.2.6.4.4 [5.2.3]).

To determine the maximum story deflections and drifts (see Section 10.1.2.2), the midspan diaphragm deflection is combined with the wall deflection. This is summarized in Table 10.1-6, which shows the drift below the level considered.

Table 10.1-6 Total Deflection and Drift

Level	Wall (in.)	Diaphragm (in.)	Total δ_e (in.)	$\delta = C_d\delta_e/I$ (in.)	Δ,drift (in.)

Roof	1.14	0.38	1.52	6.08	1.00
3	0.87	0.40	1.27	5.08	2.08
2	0.45	0.30	0.75	3.00	3.00

1.0 in. = 25.4 mm.

The maximum permissible drift is 0.020 h_s = 2.16 in. Therefore, the drift limitations are satisfied at the third floor and roof. The drift in the first story is about 38 percent too large so the design must be revised.

One beneficial aspect of the design that has been ignored in these calculations is the lightweight concrete floor fill. Although it is discontinuous at the stud walls, it will certainly stiffen the diaphragm. No studies that would support a quantitative estimate of the effect have been found. Because the shear deformation of the ½-in. plywood diaphragm represents a significant portion of the total drift at the first and second levels, any significant increase in shear stiffness that might be provided by the concrete would further reduce the expected drifts. In this case, about 60 percent of the drift is contributed by the shear walls. The diaphragm stiffness would need to be tripled to meet the drift criteria, so the shear walls will be revised.

If Structural I plywood is used for the shear walls in this direction, the drift criteria are satisfied at all levels. In Sec. 10.1.4.4, G becomes 90,000 ksi, and t (effective thickness) becomes 0.535 in. The deflections due to shear and nail slip (the 1.2 factor no longer applies) are reduced. The resulting total elastic deflection of the shear walls at Levels 2, 3, and Roof are, 0.242 in., 0.488 in., and 0.669 in., respectively. Table 10.1-6b shows the revised results, which satisfy the drift criteria.

Table 10.1-6b Total Deflection and Drift (Structural I Plywood Shear Walls)

Level	Wall (in.)	Diaphragm (in.)	Total δ_e (in.)	$\delta = C_d\delta_e/I$ (in.)	Δ,drift (in.)
Roof	0.67	0.38	1.05	4.20	0.64
3	0.49	0.40	0.89	3.56	1.40
2	0.24	0.30	0.54	2.16	2.16

1.0 in. = 25.4 mm.

Because the tie-down calculations (Sec. 10.1.4.5) depend on the tabulated capacities of the shear wall panels and Structural I panels have slightly higher capacities, the connection designs must be verified. Additional calculations (not shown here) confirm that the hardware that was previously selected still works. It is also worth noting that although many designers fail to perform deflection calculations for wood construction, the drift criteria control the selection of sheathing grade in this example.

The P-delta provision also must be examined. Following *Provisions* Sec. 5.4.6.2 [5.2.6.2] and assuming the total mass deflects two-thirds of the maximum diaphragm deflection, the P-delta coefficient is as shown in Table 10.1-7.

[Note that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, *I*, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because I = 1.0.]

Table 10.1-7 P-delta Stability Coefficient

Level	P_D (kips)	P_L (kips)	ΣP (kips)	Δ (in.)	V (kips)	$\theta = P\Delta/VhC_d$

Roof	183	204	387	0.67	40.4	0.015
3	284	130	801	1.27	82.3	0.029
2	284	130	1,215	1.76	103.2	0.048

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

For example, to compute the effective P-delta drift at the roof:

$\delta_{roof} = C_d[\delta_{wall} + 2/3(\delta_{diaphragm})] = 4[0.67 + 2/3(0.38)] = 3.693$ in.
$\delta_3 = 4[0.49 + 2/3(0.40)] = 3.027$ in.
$\delta = 3.693 - 3.027 = 0.67$ in.

The story dead load is the same as shown in Table 10.1-1, and the story live load is based on 25 psf snow load for the roof and 16 psf reduced live load (0.4×40 psf) acting over the entire area of Levels 2 and 3. For $\theta < 0.10$, no deflection amplification due to P-delta effects is necessary.

10.1.4.10 Longitudinal Direction

Only one exterior shear wall section will be designed here. The design of the corridor shear walls would be similar to the transverse walls. (This example has assumed a greater length of corridor wall than would likely be required to resist the design forces. This increased length is intended to reduce the demand to the exterior walls, assuming rigid diaphragm distribution, to a level below the maximum permitted in-plane shear for the perforated shear wall design procedure as discussed below.) For loads in the longitudinal direction, diaphragm stresses and deflections are negligible.

The design of the exterior wall will utilize the guidelines for perforated shear walls (*Provisions* Sec. 12.4.3 [AF&PA Wind&Seismic Sec. 4.3]), which are new to the 2000 *Provisions*. [The provisions for perforated shear walls are contained in AF&PA Wind & Seismic and therefore have been removed from the 2003 *Provisions*. The design provisions are spread throughout AF&PA Wind & Seismic Sec. 4.3.3, but are not substantially different for the provisions contained in the 2000 *Provisions* except for the revisions to the tie down requirements as noted below.] The procedure for perforated shear walls applies to walls with openings that have not been specifically designed and detailed for forces around the openings. Essentially, a perforated wall is treated in its entirety, rather than as a series of discrete wall piers. The use of this design procedure is limited by several conditions (*Provisions* Sec. 12.4.3.2 [AF&PA Wind&Seismic Sec. 4.3.5.2]), the most relevant to this example is that the factored design shear resistance shall not exceed 0.64 klf. This requirement essentially limits the demand on perforated shear walls such that the required factored design shear resistance is less than 0.64 klf. If the configuration required higher design values, then walls must be added in order to reduce the demand.

The main aspects of the perforated shear wall design procedure are as follows. The design shear capacity of the shear wall is the sum of the capacities of each segment (all segments shall have the same sheathing and nailing) reduced by an adjustment factor that accounts for the geometry of the openings. Uplift anchorage (tie-down) is required only at the ends of the wall (not at the ends of all wall segments), but all wall segments must resist a specified tension force (using anchor bolts at the foundation and with strapping or other means at upper floors). Requirements for shear anchorage and collectors (drag struts) across the openings are also specified. It should be taken into account that the design capacity of a perforated shear wall, is less than a standard segmented wall with all segments restrained against overturning. However, the procedure is useful in eliminating interior hold downs for specific conditions and, thus, is illustrated in this example.

The portion of the story force resisted by each exterior wall was computed previously as $0.225F_x$. The exterior shear walls are composed of three separate perforated shear wall segments (two at 30 ft long and one at 15 ft long, all with the same relative length of full height sheathing), as shown in Figure 10.1-2. This section will focus on the design of a 30-ft section. Assuming that load is distributed to the wall sections based on relative length of shear panel, then the total story force to the 30-ft section is $(30/75)0.225F_x = 0.090F_x$ per floor. The load per floor is

$F_{roof} = 0.090(40.4)$	= 3.64 kips
$F_{3rd} = 0.090(41.9)$	= 3.77 kips
$F_{2nd} = 0.090(20.9)$	= $\underline{1.88 \text{ kips}}$
Σ	= 9.29 kips

10.1.4.10.1 Perforated Shear Wall Resistance

The design shear capacity (*Provisions* Sec. 12.4.3.3 [AF&PA Wind&Seismic Table 4.3.3.4]) is computed as the factored shear resistance for the sum of the wall segments, multiplied by an adjustment factor that accounts for the percentage of full height (solid) sheathing and the ratio of the maximum opening to the story height. At each level, the design shear capacity, V_{wall}, is:

$$V_{wall} = (vC_0)\Sigma L_i$$

where

v = unadjusted factored shear resistance (*Provisions* Table 12.4.3-2a [Table 12.4.3a or AF&PA Wind&Seismic Table 4.3A).

C_0 = shear capacity adjustment factor (*Provisions* Table 12.4.3-1 [AF&PA Wind&Seismic Table 4.3.3.4]) = 0.83

The percent of full height sheathing is (4 + 10 + 4)/30 = 0.60, and the maximum opening height ratio is 4 ft /8 ft = 0.5. Per *Provisions* Table 12.4.3-1 [AF&PA Wind&Seismic Table 4.3.3.4], C_0 = 0.83.

ΣL_i = sum of widths of perforated shear wall segments = 4 + 10 + 4 = 18 ft

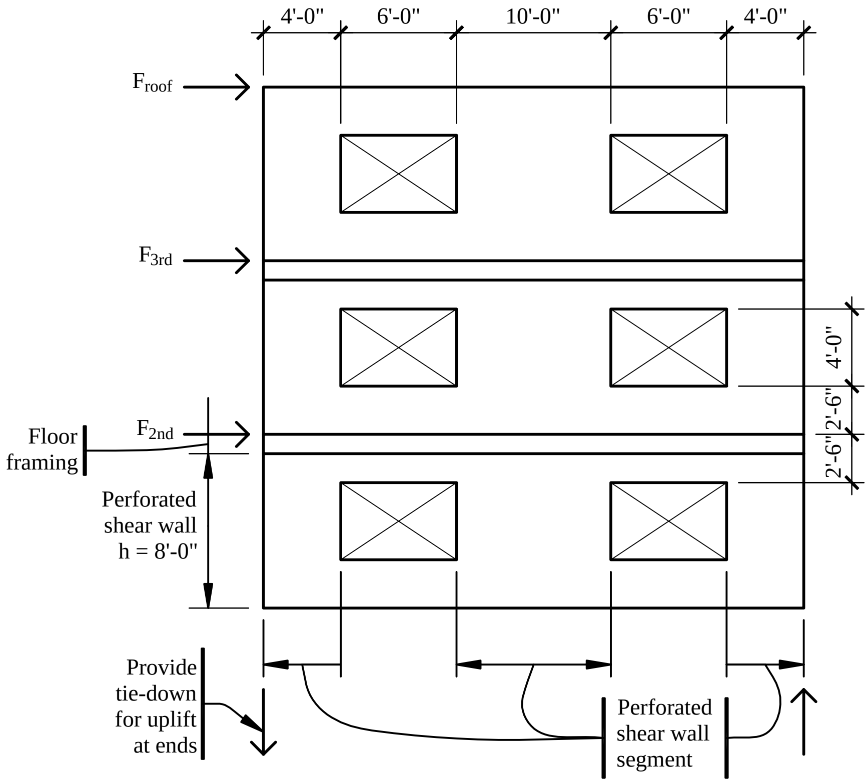


Figure 10.1-14 Perforated shear wall at exterior (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

The wall geometry (and thus the adjustment factor and total length of wall segments) is the same at all three levels, as shown in Figure 10.1-14. Perforated shear wall plywood and nailing are determined below.

Roof to third floor:

V = 3.64 kips
Required v = 3.64/0.83/18 = 0.244 klf

Try ½-in. (15/32) Structural I plywood rated sheathing on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d common nails at 6 in. on center at panel edges and 12 in. on center at intermediate framing members. (Structural I plywood would not be required to satisfy the strength requirements. However, to minimize the possibility for construction errors, the same grade of sheathing is used on walls in both directions.)

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A], $\lambda\varphi D'$ = 0.44 klf > 0.244 klf OK

Third floor to second floor:

V = 3.64 + 3.77 = 7.41 kips
Required v = 7.41/0.83/18 = 0.496 klf

Try ½-in. (15/32) Structural I plywood rated sheathing on blocked 2 in. Douglas fir-larch members at 16 in. on center with 10d common nails at four in. on center at panel edges and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A]:

$\lambda\varphi D'$ = 0.66 klf (0.64 klf max) > 0.496 klf OK

As discussed above, *Provisions* Sec. 12.4.3.2, Item b [12.4.3], limits the factored shear resistance in *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A] to 0.64 klf, which still exceeds the demand at this level, so the limitation is satisfied.

Second floor to first floor:

V = 7.41 + 1.88 = 9.29 kips
Required v = 9.29/0.83/18 = 0.622 klf

By inspection, the same plywood and nailing from above will work.

10.1.4.10.2 Perforated Shear Wall Uplift Anchorage

According to *Provisions* Sec. 12.4.3.4.1 [AF&PA Wind&Seismic Table 4.3A], the requirements for uplift anchorage must be evaluated at the ends of the wall only. Uplift at each wall segment is treated separately as described later. Uplift forces, based on the strength of the shear panels (per *Provisions* Sec. 12.4.2.4 [12.4.2.4.1]), are to be computed as discussed in Sec. 10.1.4.5. For this example, calculations involving seismic overturning and counter-balancing moments are assumed not to be applicable for perforated shear walls, as they are not expected to act as rigid bodies in resisting global overturning.

The tie-down design force is determined as Ω_0/1.3 times the factored shear resistance of the shear panels, as discussed previously. For this example, the tie-down will be designed at the first floor only; the other floors would be computed similarly, and tie-down devices, as shown in Figure 10.1-8, would be used.

[The requirements for uplift anchorage are slightly different in the 2003 *Provisions*. The tie down force is based on the "nominal strength of the shear wall" rather than the Ω_0/1.3 times the factored resistance as specified in the 2000 *Provisions*. This change is primarily intended to provide consistent terminology and should not result in a significant impact on the design of the tie down.]

The uplift forces are computed as:

Roof: T = 0.44 klf (3.0/1.3) (8 ft)	= 8.12 kips
Third floor: T = 0.66 klf (3.0/1.3) (8 ft)	= 12.18 kips
Second floor: T = 0.66 klf (3.0/1.3) (8 ft)	= <u>12.18 kips</u>
Σ = 32.48 kips	

Since the chord member supports the window header as well, use a 6×6 Douglas fir-Larch No. 1 similar to the transverse walls. Try a double tie-down device with a 7/8-in. anchor bolt and three 7/8-in. stud bolts. Using the method described above for computing the strength of a double tie-down, the nominal design strength is 34.3 kips, which is greater than the demand of 32.48 kips.

The design of the tie-downs at the second and third floors is similar.

10.1.4.10.3 Perforated Shear Wall Compression Chords

Provisions Sec. 12.4.3.4.4 [AF&PA Wind&Seismic Sec. 4.3.6.1], requires each end of a perforated shear wall to have a chord member designed for the following compression force from each story:

$$C = \frac{Vh}{C_0 \Sigma L_i}$$

where

V = design shear force in the shear wall (not wall capacity as used for uplift)
h = shear wall height (per floor)
C_0 = shear capacity adjustment factor
ΣL_i = sum of widths of perforated shear wall segments

For h = 8 ft, C_0 = 0.83 and ΣL_i = 18 ft, the compression force is computed as:

Third floor: C = 3.64(8)/0.83/18	= 1.95 kips
Second floor: C = (3.64 + 3.77)(8)/0.83/18	= 3.96 kips
First floor: C = (3.64 + 3.77 + 1.88)(8)/0.83/18	= 4.98 kips
Σ	= 10.89 kips

Again, just the chord at the first floor will be designed here; the design at the upper floors would be similar. Although not explicitly required by *Provisions* Sec. 12.4.3.4.4 [AF&PA Wind&Seismic Sec. 4.3.6.1], it is rational to combine the chord compression with gravity loading (using the load combination $1.38D + 1.0Q_E + 0.5L + 0.2S$ in accordance with *Provisions* Eq. 5.2.7.1-1 [4.2-1]), in order to design the chord member. The end post of the longitudinal shear wall supports the same tributary weight at the end post of the transverse shear walls. Using the weights computed previously in Sec. 10.1.4.5, the design compression force is:

$$1.38(5.17) + 1.0(10.89) + 0.5(4.48) + 0.2(1.30) = 20.5 \text{ kips.}$$

The bearing capacity on the bottom plate was computed previously as 31.5 kips, which is greater than 20.5 kips. Where end posts are loaded in both directions, orthogonal effects must be considered in accordance with *Provisions* Sec. 5.2.5.2 [4.4.2.3].

10.1.4.10.4 Anchorage at Shear Wall Segments

The anchorage at the base of a shear wall segment (bottom plate to floor framing or foundation wall), is designed per *Provisions* Sec. 12.4.3.4.2 [AF&PA Wind&Seismic Sec. 4.3.6.4]. While this anchorage need only be provided at the full height sheathing, it is usually extended over the entire length of the perforated shear wall to simplify the detailing and reduce the possibility of construction errors.

$$v = \frac{V}{C_0 \sum L_i}$$

where

V = design shear force in the shear wall
C_0 = shear capacity adjustment factor
ΣL_i = sum of widths of perforated shear wall segments

This equation is the same as was previously used to compute unit shear demand on the wall segments. Therefore, the in-plane anchorage will be designed to meet the following unit, in-plane shear forces:

Third floor: v = 0.244 klf
Second floor: v = 0.496 klf
First floor: v = 0.622 klf.

In addition to resisting the in-plane shear force, *Provisions* Sec. 12.4.3.4.3 [AF&PA Wind&Seismic Sec. 4.3.6.4] requires that the shear wall bottom plates be designed to resist a uniform uplift force, t, equal to the unit in-plane shear force. Per *Provisions* Sec. 12.4.3.4.5 [AF&PA Wind&Seismic Sec. 4.3.6.4], this uplift force must be provided with a complete load path to the foundation. That is, the uplift force at each level must be combined with the uplift forces at the levels above (similar to the way overturning moments are accumulated down the building).

At the foundation level, the unit in-plane shear force, v, and the unit uplift force, t, are combined for the design of the bottom plate anchorage to the foundation wall. The design unit forces are:

Shear: v = 0.622 klf
Tension: t = 0.244 + 0.496 + 0.622 = 1.36 klf

Assuming that stresses on the wood bottom plate govern the design of the anchor bolts, the anchorage is designed for shear (single shear, wood-to-concrete connection) and tension (plate washer bearing on bottom plate). The interaction between shear and tension need not be considered in the wood design for this configuration of loading. As for the transverse shear walls, try a 5/8-in. bolt at 32 in. on center with a 3-in. square plate washer (*Provisions* Sec. 12.4.2.4 [12.4.2.4.2] requires 1/4×3×3 in. plate washer for 5/8-in. anchor bolts). As computed previously, the shear capacity is 0.91 klf so the bolts are adequate.

For anchor bolts at 32 in. on center, the tension demand per bolt is 1.36 klf (32/12) = 3.63 kips. Bearing capacity of the plate washer (using a Douglas fir No. 2 bottom plate) is computed per the AF&PA Manual, Structural Connections Supplement, as:

$\lambda\phi P'$ = (1.0)(0.8)(1.30 ksi)(9 in.^{2}) = 9.36 kips > *3.63* kips OK

The anchor bolts themselves must be designed for combined shear, and tension in accordance with *Provisions* Sec. 9.2 [11.2].

In addition to designing the anchor bolts for uplift, a positive load path must be provided to transfer the uplift forces into the bottom plate. One method for providing this load path continuity, is to use metal straps nailed to the studs and lapped around the bottom plate, as shown in Figure 10.1-15. Attaching the studs directly to the foundation wall (using embedded metal straps) for uplift and using the anchor bolts for shear only is an alternative approach.

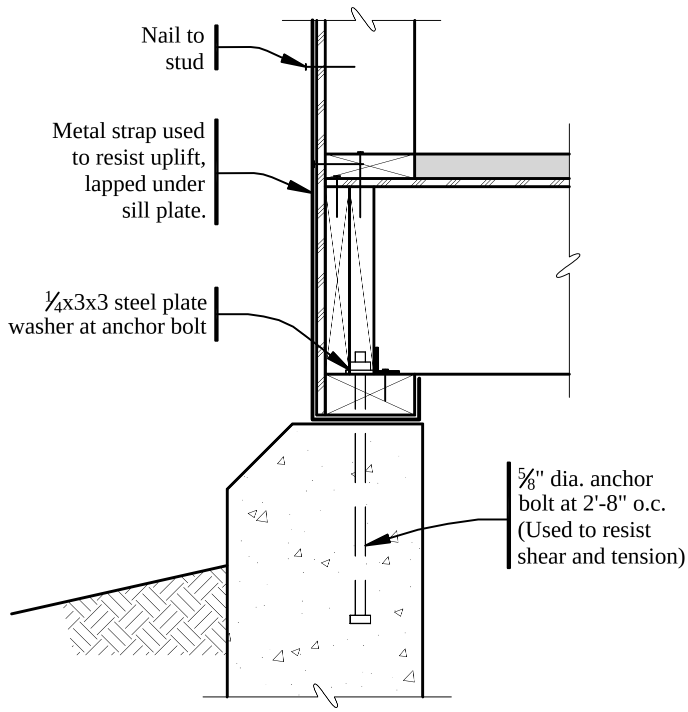


Figure 10.1-15 Perforated shear wall detail at foundation (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

At the upper floors, the load transfer for in-plane shear is accomplished by using nailing or framing clips between the bottom plates, rim joists, and top plates in a manner similar to that for standard shear walls. The uniform uplift force can be resisted either by using the nails in withdrawal (for small uplift demand) or by providing vertical metal strapping between studs above and below the level considered. This type of connection is shown in Figure 10.1-16. For this type of connection (and the one shown in Figure 10.1-15) to be effective, shrinkage of the floor framing must be minimized using dry or manufactured lumber.

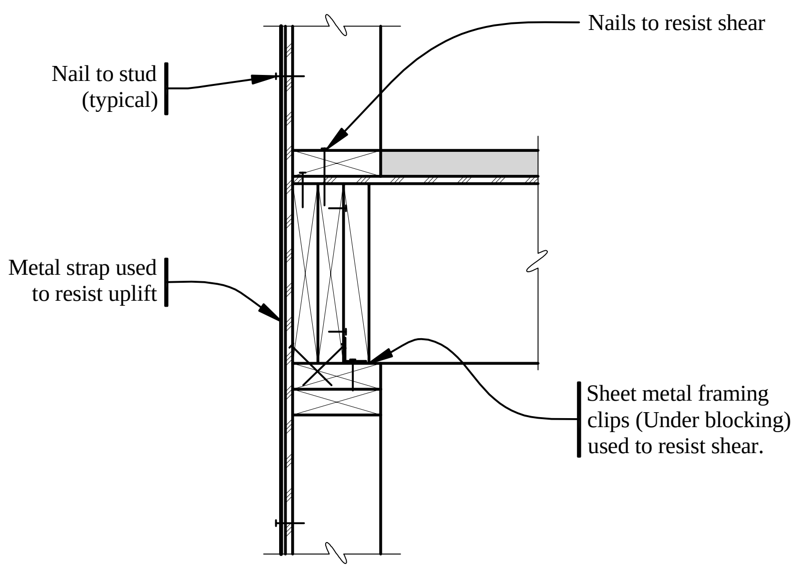


Figure 10.1-16 Perforated shear wall detail at floor framing.

For example, consider the second floor. The required uniform uplift force, t = 0.244 + 0.496 = 0.740 klf. Place straps at every other stud, so the required strap force is 0.740 (32/12) = 1.97 kips. Provide an 18-gauge strap with 1210d nails at each end.

10.2 WAREHOUSE WITH MASONRY WALLS AND WOOD ROOF, LOS ANGELES, CALIFORNIA

This example features the design of the wood roof diaphragm, and wall-to-diaphragm anchorage for the one-story masonry building, described in Sec. 9.1 of this volume of design examples. Refer to that example for more detailed building information and the design of the masonry walls.

10.2.1 Building Description

This is a very simple rectangular warehouse, 100 ft by 200 ft in plan (Figure 10.2-1), with a roof height of 28 ft. The wood roof structure slopes slightly, but it is nominally flat. The long walls (side walls) are 8 in. thick and solid, and the shorter end walls are 12 in. thick and penetrated by several large openings.

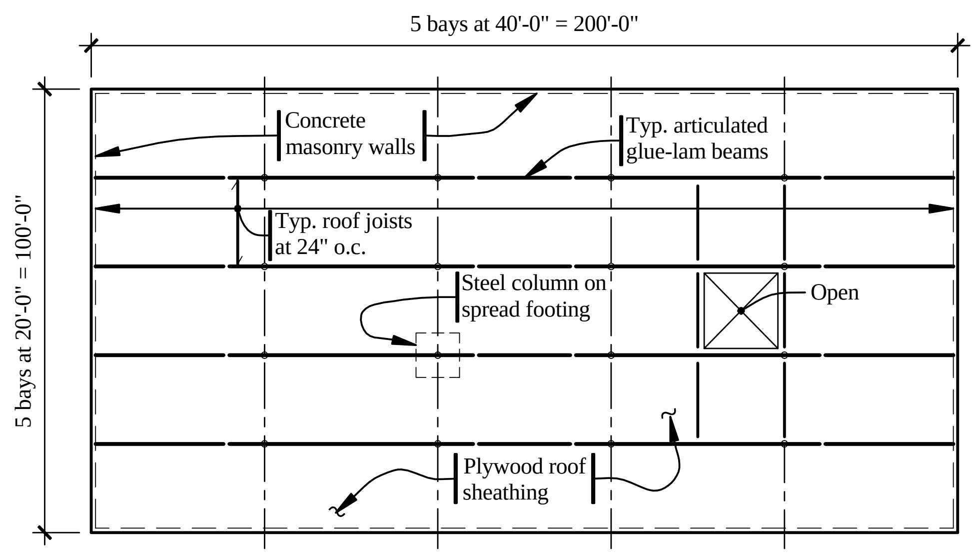


Figure 10.2-1 Building plan (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

Based on gravity loading requirements, the roof structure consists of wood joists, supported by 8¾-in. wide, by 24-in. deep, glued-laminated beams, on steel columns. The joists span 20 ft and the beams span 40 ft, as an articulated system. Typical roof framing is assumed to be Douglas fir-Larch No.1 as graded by the WWPA. The glued-laminated beams meet the requirements of combination 24F-V4 per ANSI/AITC A190.1.

The plywood roof deck acts as a diaphragm to carry lateral loads to the exterior walls. There are no interior walls for seismic resistance. The roof contains a large opening that interrupts the diaphragm continuity.

The diaphragm contains continuous cross ties in both principal directions. The details of these cross ties and the masonry wall-to-diaphragm anchorage are substantially different from those shown in previous versions of this example. This is primarily due to significant revisions to the *Provisions* requirements for anchorage of masonry (and concrete) walls to flexible wood diaphragms.

The following aspects of the structural design are considered in this example:

1. Development of diaphragm forces based on the equivalent lateral force procedure used for the masonry wall design (Sec. 9.1)
2. Design and detailing of a plywood roof diaphragm with a significant opening
3. Computation of drift and P-delta effects
4. Anchorage of diaphragm and roof joists to masonry walls and
5. Design of cross ties and subdiaphragms

[Note that as noted in Sec. 9.1, the new "Simplified Design Procedure" contained in 2003 *Provisions* Simplified Alternate Chapter 4 as referenced by 2003 *Provisions* Sec. 4.1.1 is likely to be applicable to this example, subject to the limitations specified in 2003 *Provisions* Sec. Alt. 4.1.1.]

10.2.2 Basic Requirements

10.2.2.1 *Provisions* Parameters

S_S (*Provisions* Maps [Figure 3.3.3])	= 1.50
S_1 (*Provisions* Maps [Figure 3.3.4])	= 0.60
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	= C
Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	= I
Seismic Design Category (*Provisions* Sec. 4.2 [1.4])	= D
Seismic Force Resisting System (*Provisions* Table 5.2.2 [4.3-1])	= Special reinforced masonry shear wall
Response Modification Factor, R (*Provisions* Table 5.2.2 [4.3-1])	= 3.5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [4.3-1])	= 2.5
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 4.3-1])	= 3.5

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

10.2.2.2 Structural Design Criteria

A complete discussion on the criteria for ground motion, seismic design category, load path, structural configuration, redundancy, analysis procedure, and shear wall design, is included in Sec. 9.1 of this volume of design examples.

10.2.2.2.1 Design and Detailing Requirements (Provisions Sec. 5.2.6 [4.6])

See *Provisions* Chapter 12, for wood design requirements. As discussed in greater detail in Sec. 10.1, *Provisions* Sec. 12.2.1 utilizes load and resistance factor design (LRFD) for the design of engineered wood structures. The design capacities are therefore, consistent with the strength design demands of *Provisions* Chapter 5.

The large opening in the diaphragm must be fitted with edge reinforcement (*Provisions* Sec. 5.2.6.2.2 [4.6.1.4]). However, the diaphragm does not require any collector elements that would have to be designed for the special load combinations (*Provisions* Sec. 5.2.6.4.1 [4.6.2.2]).

The requirements for anchoring of masonry walls to flexible diaphragms (*Provisions* Sec. 5.2.6.3.2 [4.6.2.1]) are of great significance in this example.

10.2.2.2.2 Combination of Load Effects (Provisions Sec. 5.2.7 [4.2.2])

The basic design load combinations for the lateral design, as stipulated in ASCE 7, and modified by the *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2], were computed in Sec. 9.1 of this volume of design examples as:

$$1.4D + 1.0Q_E$$

and

$$0.7D - 1.0Q_{E.}$$

The roof live load, L_r, is not combined with seismic loads, and the design snow load is zero for this Los Angeles location.

10.2.2.2.3 Deflection and Drift Limits (Provisions Sec. 5.2.8 [4.5.1])

In-plane deflection and drift limits for the masonry shear walls are considered in Sec. 9.1.

As illustrated below, the diaphragm deflection is much greater than the shear wall deflection. According to *Provisions* Sec. 5.2.6.2.6 [4.5.2], in-plane diaphragm deflection shall not exceed the permissible deflection of the attached elements. Because the walls are essentially pinned at the base, and simply supported at the roof, they are capable of accommodating large deflections at the roof diaphragm.

For illustrative purposes, story drift is determined and compared to the requirements of *Provisions* Table 5.2.8 [4.5-1]. However, according to this table, there is essentially no drift limit for a single story structure as long as the architectural elements can accommodate the drift (assumed to be likely in a warehouse structure with no interior partitions). As a further check on the deflection, P-delta effects (*Provisions* Sec. 5.4.6.2 [5.2.6.2]) are evaluated.

10.2.3 Seismic Force Analysis

Building weights and base shears are as computed in Sec. 9.1 of this volume of design examples. (The building weights used in this example are based on a preliminary version of Example 9.1 and, thus, minor numerical differences may exist between the two examples). *Provisions* Sec. 5.2.6.4.4 [4.6.3.4]specifies that floor and roof diaphragms be designed to resist a force, F_{px}, in accordance with *Provisions* Eq. 5.2.6.4.4 [4.6-2]as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

plus any force due to offset walls (not applicable for this example). For one-story buildings, the first term of this equation will be equal to the seismic response coefficient, C_s, which is 0.286. The effective diaphragm weight, w_{px}, is equal to the weight of the roof, plus the tributary weight of the walls perpendicular to the direction of the motion. The tributary weights are:

Roof = 20(100)(200)	= 400 kips
Side walls = 2(65)(28/2+2)(200)	= 416 kips
End walls = 2(103)(28/2+2)(100)	= 330 kips

The diaphragm design force is computed as:

Transverse	$F_{p,roof} = 0.286(400 + 416)$	= 233 kips
Longitudinal	$F_{p,roof} = 0.286(400 + 330)$	= 209 kips

These forces exceed the minimum diaphragm design forces given in *Provisions* Sec. 5.2.6.4.4 [4.6.3.4], because C_s exceeds the minimum factor of $0.2S_{DS}$.

10.2.4 Basic Proportioning of Diaphragm Elements

The design of plywood diaphragms primarily involves the determination of sheathing sizes and nailing patterns to accommodate the applied loads. Large openings in the diaphragm and wall anchorage requirements, however, can place special requirements on the diaphragm capacity. Diaphragm deflection is also a consideration.

Nailing patterns for diaphragms are established on the basis of tabulated requirements included in the *Provisions*. It is important to consider the framing requirements for a given nailing pattern and capacity as indicated in the notes following the tables. In addition to strength requirements, *Provisions* Sec. 12.4.1.2 places aspect ratio limits on plywood diaphragms (length-to-width shall not exceed 4/1 for blocked diaphragms). However, it should be taken into consideration that compliance with this aspect ratio does not guarantee that drift limits will be satisfied.

While there is no specific limitation on deflection for this example, the diaphragm has been analyzed for deflection as well as for shear capacity. A procedure for computing diaphragm deflections is illustrated in detail, in Sec. 10.1.4.7.

In the calculation of diaphragm deflections, the chord splice slip factor can result in large additions to the total deflection. This chord splice slip, however, is often negligible where the diaphragm is continuously anchored to a bond beam in a masonry wall. Therefore, chord splice slip is assumed to be zero in this example.

10.2.4.1 Strength of Members and Connections

The 2000 *Provisions* have adopted Load and Resistance Factor Design (LRFD) for engineered wood structures. The *Provisions* includes the ASCE 16 standard by reference and uses it as the primary design procedure for engineered wood construction. Strength design of members and connections is based on the requirements of ASCE 16. The AF&PA Manual and supplements contain reference resistance values for use in design. For convenience, the *Provisions* contains design tables for diaphragms that are identical to those contained in the AF&PA Structural-Use Panels Supplement. Refer to Sec. 10.1.4.1 for a more complete discussion of the design criteria.

[The primary reference for design of wood diaphragms in the 2003 *Provisions* is AF&PA Wind & Seismic. Much of the remaining text in the 2003 *Provisions* results from differences between AF&PA Wind & Seismic and Chapter 12 of the 2000 *Provisions* as well as areas not addressed by AF&PA Wind & Seismic. Because the AF&PA Wind & Seismic tabulated design values for diaphragms do not completely replace the tables in the 2000 *Provisions*, portions of the tables remain in the 2003 *Provisions*. Therefore, some diaphragm design values are in the 2003 *Provisions* and some are in AF&PA Wind & Seismic. The design values in the tables are different between the two documents. The values in the 2003 *Provisions* represent factored shear resistance ($\lambda\varphi D'$), while the values in AF&PA Wind & Seismic represent nominal shear resistance that must then be multiplied by a resistance factor, φ, (0.65) and a time effect factor ,λ, (1.0 for seismic loads). Therefore, while the referenced tables may be different, the factored resistance values based on the 2003 *Provisions* should be the same as those in examples based on the 2000 *Provisions*. The calculations that follow are annotated to indicate from which table the design values are taken.]

10.2.4.2 Roof Diaphragm Design for Transverse Direction

10.2.4.2.1 Plywood and Nailing

The diaphragm design force, $F_{p,roof}$ = 233 kips. Accounting for accidental torsion (*Provisions* Sec. 5.4.4.2 [5.2.4.2]), the maximum end shear = 0.55$F_{p,roof}$ = 128 kips. This corresponds to a unit shear force v = (128/100) = 1.28 klf. Although there is not a specific requirement in the *Provisions*, it is the authors' opinion that accidental torsion should be considered, even for "flexible" diaphragms, to account for the possibility of a non-uniform mass distribution in the building.

Because the diaphragm shear demand is relatively high, the plywood sheathing and roof framing at the ends of the building, must be increased in size over the standard roof construction for this type of building. Assuming 3-in. nominal framing, try blocked 3/4-in. (23/32) Structural I plywood rated sheathing with two lines of 10d common nails at 2-1/2 in. on center at diaphragm boundaries, continuous panel edges, and two lines at 3 in. on center at other panel edges.

From *Provisions* Table 12.4.3-1a [12.4-1a], $\lambda\phi D'$ = 1.60 klf > 1.28 klf OK

Because the diaphragm shear decreases towards the midspan of the diaphragm, the diaphragm capacity may be reduced towards the center of the building. Framing size and plywood thickness are likely to have a more significant impact on cost than nail spacing, determine a reasonable location to transition to 2 in. nominal roof joists and ½ in. (15/32) plywood. A reasonable configuration for the interior of the building utilizes ½ -in. (15/32) Structural I plywood rated sheathing with a single line of 10d at 2 ½ in. on center nailing at diaphragm boundaries, continuous panels edges, and 4 in. on center nailing at other panel edges. Using 2×4 flat blocking at continuous panel edges, the requirements found in Notes f, and g of *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Sec. 4.2.7.1] are met. Determine the distance, X, from the end wall where the transition can be made as:

$\lambda\phi D'$ = 0.83 klf (*Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A]),
Shear Capacity = 0.83(100) = 83.0 kips
Uniform Diaphragm Demand = 233/200 = 1.165 klf
X = (128 - 83)/1.165 = 38.6 ft, say 40 ft from the diaphragm edge

[Here is an example where both the *Provisions* tables and the AF&PA Wind & Seismic tables are required to complete the design. The design value for this plywood thickness and nailing pattern is contained in AF&PA Wind & Seismic, but the design value for the higher-capacity diaphragm at the ends is contained in the *Provisions*.]

In a building of this size, it may be beneficial to further reduce the diaphragm nailing towards the middle of the roof. However, due to the requirements for subdiaphragms, (see below) and diaphragm capacity, in the longitudinal direction and for simplicity of design, no additional nailing pattern is used.

Table 10.2-1 contains a summary of the diaphragm framing and nailing requirements (All nails are 10d common). See Figure 10.2-2 for designation of framing and nailing zones, and Figure 10.2-3 for typical plywood layout.

Table 10.2-1 Roof Diaphragm Framing and Nailing Requirements

Zone*	Framing	Plywood	Nail Spacing (in.)	Capacity (kip/ft)

			Boundaries and Cont. Panel Edges	Other Panel Edges	Intermediate Framing Members	
a	3×12	¾ in.	2½ (2 lines)	3 (2 lines)	12 (1 line)	1.60
b	2×12	½ in.	2½ (1 line)	4 (1 line)	12 (1 line)	0.83

1.0 in. = 25.4 mm, 1.0 kip/ft = 14.6 kN/m.
* Refer to Figure 10.2-2 for zone designation.

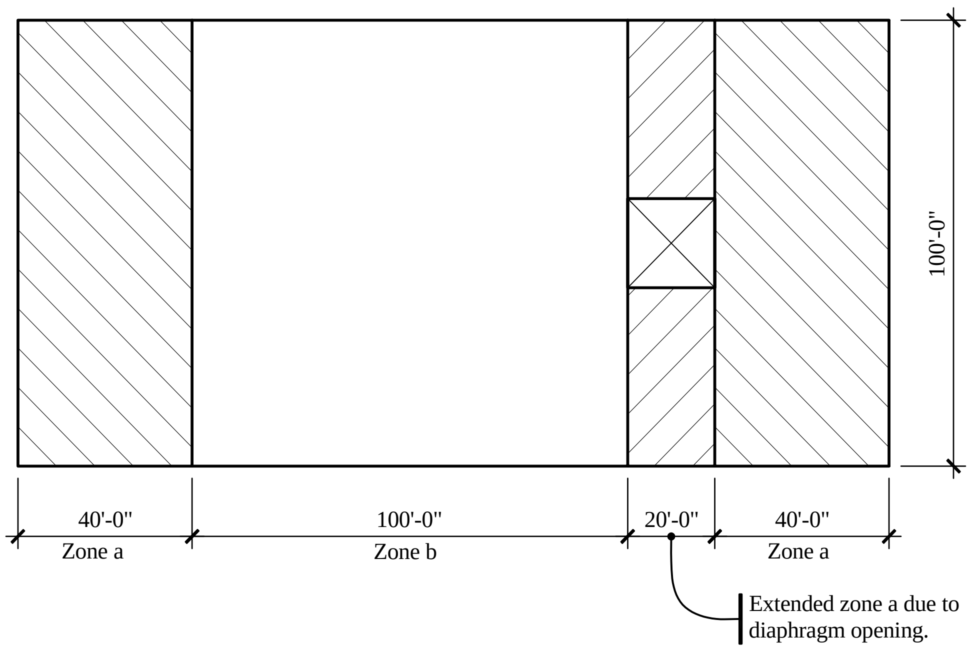


Figure 10.2-2 Diaphragm framing and nailing layout (1.0 ft = 0.3048 m).

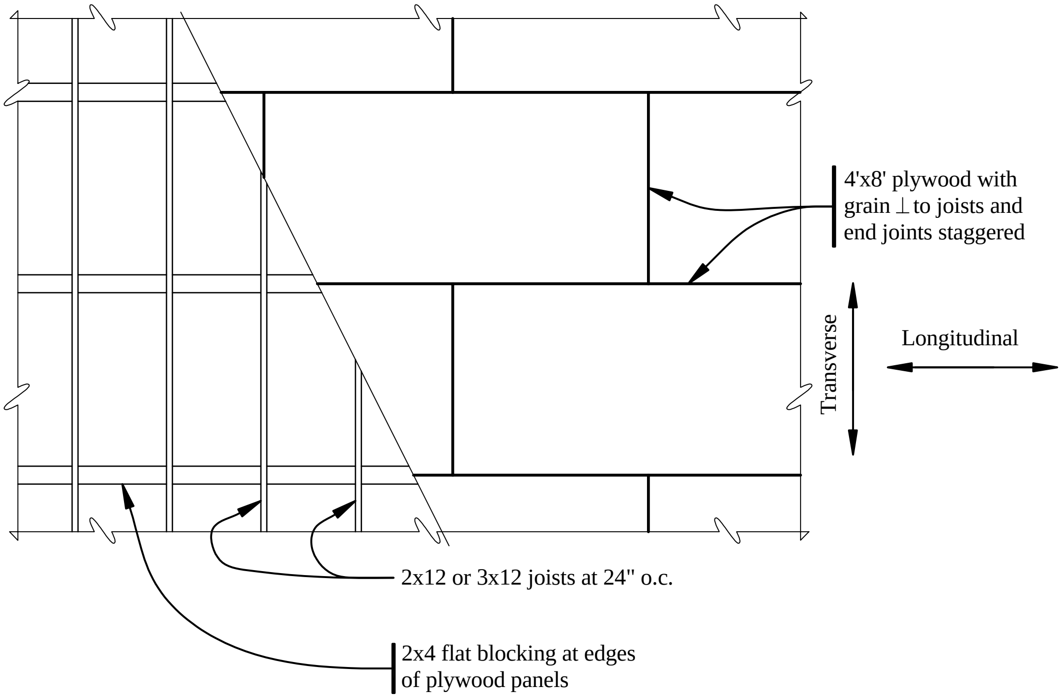


Figure 10.2-3 Plywood layout (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

10.2.4.2.2 Chord Design

Although the bond beam at the masonry wall could be used as a diaphragm chord, this example illustrates the design of the wood ledger member as a chord. Chord forces are computed using a simply supported beam analogy, where the design force is the maximum moment divided by the diaphragm depth.

Diaphragm moment, $M = wL^2/8 = F_{p,roof}L/8$ = 233(200/8) = 5,825 ft-kips
Chord force, $T = C$ = 5,825/(100 - 16/12) = 59.0 kips

Try a select structural Douglas fir-larch 4×12, for the chord. Assuming two 1/16 in. bolt holes (for 1-in. bolts) at splice locations, the net chord area is 31.9 in.^{2} Tension strength (parallel to wood grain), per the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi T'$ (1.0)(0.8)(2.70)(31.9) = 68.9 kips > 59.0 kips *OK*

Design the splice for the maximum chord force of 59.0 kips. Try bolts with steel side plates using 1 in. A307 bolts, with a 3½ in. length in the main member. The capacity, according to the AF&PA Manual, Structural Connections Supplement, is:

$\lambda\phi Z'$ = (1.0)(0.65)(16.29) = 10.6 kips per bolt.

Number of bolts required = 59.0/10.6 = 5.6

Use two rows of three bolts. The reduction (group action factor) for multiple bolts is negligible. Net area of the 4×12 chord with two rows of 1-1/16 in. holes is 31.9 in.^{2} as assumed above. Therefore, use six 1 in. A307 bolts on each side of the chord splice (Figure 10.2-4). Although it is shown for illustration, this type of chord splice may not be the preferred splice against a masonry wall since the bolts, and side plate, would have to be recessed into the wall.

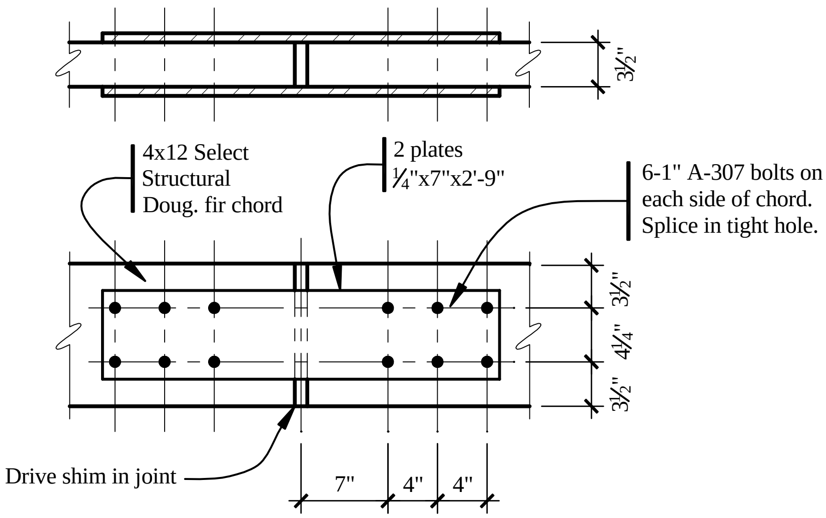


Figure 10.2-4 Chord splice detail (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

10.2.4.2.3 Diaphragm Deflection and P-delta Check

The procedure for computing diaphragm deflection is described in Sec. 10.1.4.7.

[AF&PA Wind & Seismic also contains procedures for computing diaphragm deflections. The equations are slightly different from the more commonly used equations that appear in the *Commentary* and AF&PA LRFD *Manual*. In AF&PA Wind & Seismic, the shear and nail slip terms are combined using an "apparent shear stiffness" parameter. However, the apparent shear stiffness values are only provided for OSB. Therefore, the deflection equations in the *Commentary* or AF&PA LRFD *Manual* must be used in this example which has plywood diaphragms. The apparent shear stiffness values for plywood will likely be available in future editions of AF&PA Wind & Seismic.]

As stated in *Commentary* Sec. 12.4, the diaphragm deflection is computed as:

$$\delta = \frac{5vL^3}{8wEA} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma\left(\Delta_c X\right)}{2w}$$

The equation produces the midspan diaphragm displacement in inches, and the individual variables *must* be entered in the force or length units as described below. A small increase in diaphragm deflection due to the large opening is neglected. An adjustment factor, F, for non-uniform nailing is applied to the third term of the above equation for this example.

Bending deflection = $5vL^3/8wEA$	= 0.580 in.
Shear deflection = $vL/4Gt$	= 1.210 in.
Effective nails slip deflection = 0.188 Le_nF	= 0.265 in.
Deflection due to chord slip at splices = $\Sigma(\Delta_c X)/2w$	= 0.000 in.

(The chord slip deflection is assumed to be zero because the chord is connected to the continuous bond beam at the top of the masonry wall.)

The variables above and associated units used for computations are:

v = (233/2)/100 = 1,165 plf (shear per foot at boundary, ignoring torsion)

L = 200 ft and w = 100 ft (diaphragm length and depth)
A = effective area of 4×12 chord and two-#6 rebars assumed to be in the bond beam
= 39.38 in.^{2} + 2(0.44)(29,000,000/1,900,000) = 52.81 in.^{2}
t = 0.535 in. (effective, for ½ in. Structural I plywood, unsanded; neglect ¾ in. plywood at edge)
E = 1,900,000 psi (for Douglas fir-larch select structural chord)
G = 90,000 psi (Structural I plywood)
e_n = nail slip for 10d nail at end of diaphragm (use one and one-half-in. nail spacing, two lines at 3 in., not the 1-1/4 in. perimeter spacing)

Design nail load = 1,165/(12/1.5) = 146 lb/nail

Using the procedure in Sec. 10.1.4.3 and the value for dry/dry lumber from Table 10.1-2, e_n = $(146/769)^{3.276}$ = 0.00433 in.

F = adjustment for non-uniform nail spacing

The 0.188 coefficient assumes a uniform nail spacing, which implies an average load per nail of one-half the maximum. One common practice for calculating deformation is simply to use the load per nail that would result from the larger spacing should that spacing be used throughout. However, this gives a large estimate for deformation due to nail slip. Figure 10.2-5 shows the graphic basis for computing a more accurate nail slip term. The basic amount is taken as that for the smaller nail spacing (1-1/2 in.), which gives 146 lbs per nail.

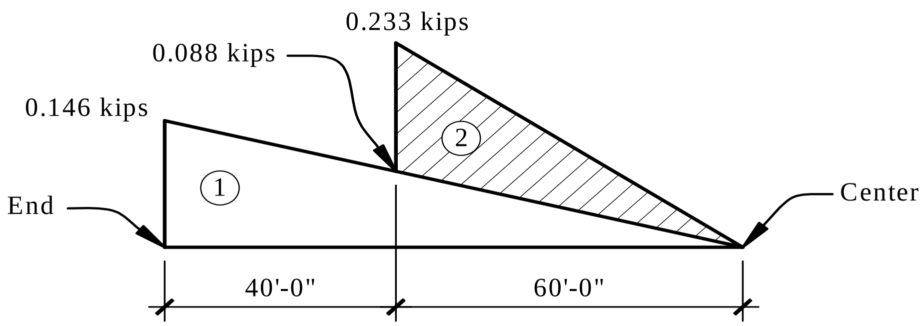


Figure 10.2-5 Adjustment for nonuniform nail spacing (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN)

Using a larger nail spacing at the interior increases the deformation. The necessary increase may be determined as the ratio of the areas of the triangles in Figure 10.2-5, which represents the load per nail along the length of the diaphragm. The ratio is for Triangle 2 representing zone b compared to Triangle 1 representing zone a, where zones a and b are as shown Figure 10.2-2.

Ratio = (233 - 88)60 / 146(100) = 0.63

thus, F = 1 + 0.63 = 1.63.

Total for diaphragm:

δ = 0.580 + 1.210 + 0.265 + 0.00 = 2.055 in.

End wall deflection = 0.037 in. (see Sec. 9.1 of this volume of design examples)

Therefore, the total elastic deflection δ_{xe} = 2.055 + 0.037 = 2.092 in.

Total deflection, $\delta_x = C_d\ \delta_{xe}/I$ = 3.5(2.092)/1.0 = 7.32 in. = Δ

P-delta effects are computed according to *Provisions* Sec. 5.4.6.2 [5.2.6.2]using the stability coefficient computed per *Provisions* Eq. 5.4.6.2-1 [5.2-16]:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

Because the midspan diaphragm deflection is substantially greater than the deflection at the top of the masonry end walls, it would be overly conservative to consider the entire design load at the maximum deflection. Therefore, the stability coefficient is computed by splitting the P-delta product into two terms one for the diaphragm and one for the end walls.

For the diaphragm, consider the weight of the roof and side walls at the maximum displacement. (This overestimates the P-delta effect. The computation could consider the average displacement of the total weight, which would lead to a reduced effective delta. Also, the roof live load need not be included.)

P = 400 + 416 = 816 kips
Δ= 7.32 in.
V = 233 kips (diaphragm force)

For the end walls, consider the weight of the end walls at the wall displacement:

P = 330 kips
Δ = (3.5)(0.037) = 0.13 in.
V = 264 kips (additional base shear for wall design)

For story height, h = 28 feet, the stability coefficient is:

$$\theta = \left(\frac{P\Delta}{V} + \frac{P\Delta}{V}\right) \Big/ hC_d = \left(\frac{816(7.32)}{233} + \frac{330(0.13)}{264}\right) \Big/ (28)(12)(3.5) = 0.022$$

For $\theta < 0.10$, no deflection amplification due to P-delta effects is necessary.

[Note that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, I, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because I = 1.0.]

Drift index =Δ/h_{sx} = 7.32/[28(12)] = 0.022.

This is slightly less than the limiting drift ratio of 0.025 applied for most low-rise buildings in Seismic Use Group I (*Provisions* Sec. 5.2.8 [4.5.1] and Table 5.2.8 [4.5-1]). However, for one-story buildings, Table 5.2.8, Note b [4.5-1, Note c], and *Provisions* Sec. 5.2.6.2.6 [4.5.2], permit unlimited drift provided that the structural elements and finishes can accommodate the drift. The limit for masonry cantilever shear wall structures (0.007) should only be applied to the in-plane movement of the end walls (0.13/h = 0.0004 << 0.007). The construction of the out-of-plane walls allows them to accommodate very large drifts. It is further expected that the building does not contain interior elements that are sensitive to drift.

Given the above conditions, and the fact that P-delta effects are not significant for this structure, the computed diaphragm deflections appear acceptable.

10.2.4.2.4 Detail at Opening

Consider diaphragm strength at the roof opening (Figure 10.2-6), as required by *Provisions* Sec. 5.2.6.2.2 [4.6.1.4].

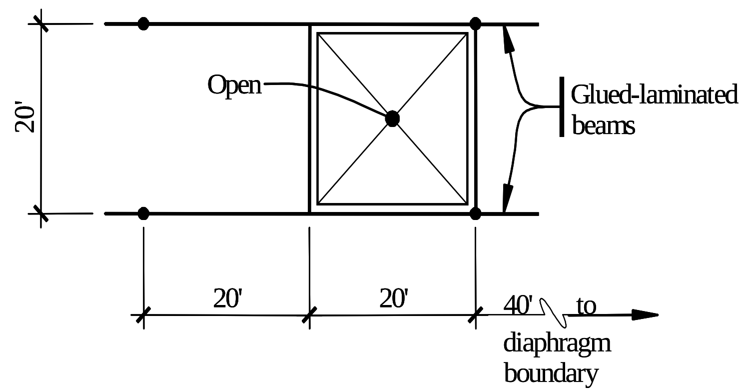


Figure 10.2-6 Diaphragm at roof opening (1.0 ft = 0.3048 m).

Check diaphragm nailing for
required shear area (shear in diaphragm at edge of opening):

Shear = 128 - [40(1.165)] = 81.4 kips
v = 81.4/(100 - 20) = 1.02 klf

Because the opening is centered in the width of the diaphragm, half the force to the diaphragm must be distributed on each side of the opening.

Diaphragm capacity in this area = 0.830 klf as computed previously (see Table 10.2-1 and Figure 10.2-2). Because the diaphragm demand at the reduced section exceeds the capacity, the extent of the Zone a nailing and framing should be increased. For simplicity, extend the Zone a nailing to the interior edge of the opening (60 ft from the end wall). The diaphragm strength is now adequate for the reduced overall width at the opening.

10.2.4.2.5 Framing around Opening

The opening is located 40 ft from one end of the building and is centered in the other direction (Figure 10.2-6). This does not create any panels with very high aspect ratios.

In order to develop the chord forces, continuity will be required across the glued-laminated beams in one direction and across the roof joists in the other direction.

10.2.4.2.6 Chord Forces at Opening

To determine the chord forces on the edge joists, model the diaphragm opening as a Vierendeel truss and assume the inflection points will be at the midpoint of the elements (Figure 10.2-7). Compute forces at the opening using a uniformly distributed diaphragm demand of 233/200 = 1.165 klf.

For Element 1 (shown in Figure 10.2-7):

w_1 = 1.165/2 = 0.582 kips/ft
V_{1B} = 0.5[128-(40)(1.165)] = 40.7 kips
V_{1A} = 40.7 - 20(0.582) = 29.1 kips
M_1 due to Vierendeel action = (½)[40.7(10) + 29.1(10)] = 349 ft-kips

Chord force due to M_1 = 349/40 = 8.72 kips. This is only 41 psi on the glued-laminated beam on the edge of the opening. This member is adequate by inspection. On the other side of this diaphragm element, the chord force is much less than the maximum global chord force (59.0 kips), so the ledger and ledger splice are adequate.

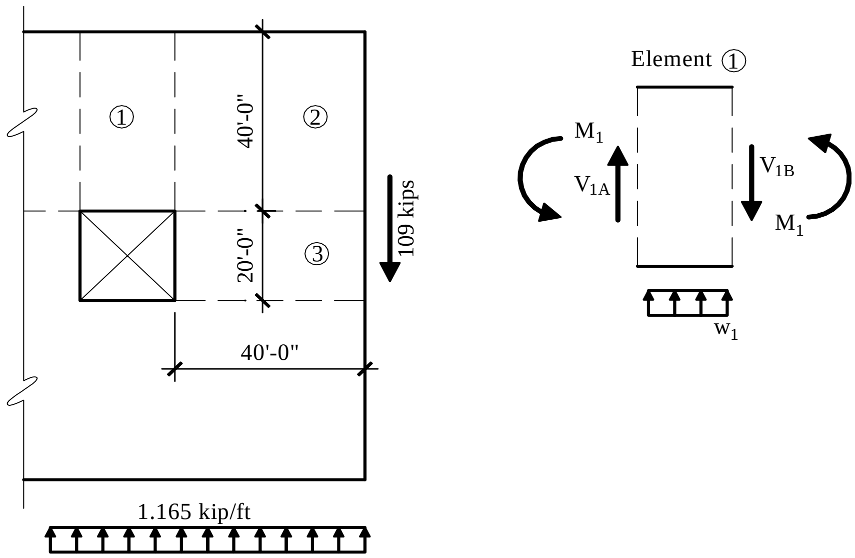


Figure 10.2-7 Chord forces and Element 1 free-body diagram (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 kip/ft = 14.6 kN/m).

For Element 3, analyze Element 2 (shown in Figure 10.2-8):

w_2 = 1.165(40/100) = 0.466 kips/ft
V_3 = 128(40/100) = 51.2 kips
V_{1B} = 40.7 kips
M_1 = 349 ft-kips.

T_{1B} is the chord force due to moment on the total diaphragm:

$M = 128(40) - 1.165(40^2/2)$ = 4,188 ft-kips
T_{1B} = 4,188/100 = 41.9 kips
ΣM_0: $M_3 = M_1 + 40V_3 - 40T_{1B} - w_2 40^2/2$ = 348 ft-kips

Therefore, the chord force on the roof joist = 348/40 = 8.7 kips

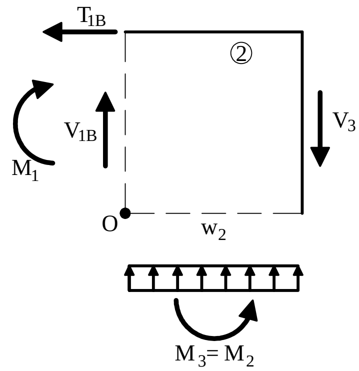


Figure 10.2-8 Free-body diagram for Element 2.

Alternatively, the chord design should consider the wall anchorage force interrupted by the opening. As described in Sec. 10.2.4.3, the edge members on each side of the opening are used as continuous cross-ties, with maximum cross-tie force of 25.0 kips. Therefore, the cross-tie will adequately serve as a chord at the opening.

10.2.4.3 Roof Diaphragm Design for Longitudinal Direction

Force = 209 kips
Maximum end shear = 0.55(209) = 115 kips
Diaphragm unit shear v = 115/200 = 0.58 klf

For this direction, the plywood layout is Case 3 in *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A]. Using ½ in. Structural I plywood rated sheathing, blocked, with 10d common nails at 2 ½ in. on center at diaphragm boundaries and continuous panel edges parallel to the load (ignoring the capacity of the extra nails in the outer zones):

$\lambda\varphi D'$ = 0.83 plf > 0.58 plf, *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A] OK

Therefore, use the same nailing designed for the transverse direction. Compared with the transverse direction, the diaphragm deflection and P-delta effects will be satisfactory.

10.2.4.4 Masonry Wall Anchorage to Roof Diaphragm

As stipulated in *Provisions* Sec. 5.2.6.3.2 [4.6.2.1], masonry walls must be anchored to flexible diaphragms to resist out-of-plane forces computed per *Provisions* Eq. 5.2.6.3.2 [4.6-1]as:

$F_P = 1.2 S_{DS} I W_p = 1.2(1.0)(1.0) W_p = 1.2\ W_p$
Side walls, F_P = 1.2(65psf)(2+28/2)/1000 = 1.25 klf
End walls, F_P = 1.2(103psf)(2 + 28/2)/1000 = 1.98 klf

[In the 2003 *Provisions* the anchorage force for masonry walls connected to a flexible diaphragm has been reduced to $0.8 S_{DS} I W_p$.]

10.2.4.3.1 Anchoring Joists Perpendicular to Walls (Side Walls)

Because the roof joists are spaced at 2 ft on center, provide a connection at each joist that will develop 2(1.25) = 2.50 kip/joist.

A common connection for this application is metal tension tie or hold-down device that is anchored to the masonry wall with an embedded bolt and is either nailed or bolted to the roof joist. Other types of anchors include metal straps that are embedded in the wall and nailed to the top of the joist. The ledger is not used for this force transfer because the eccentricity between the anchor bolt and the plywood creates tension perpendicular to the grain in the ledger (cross-grain bending), which is prohibited. Also, using the edge nails to resist tension perpendicular to the edge of the plywood, is not permitted.

Try a tension tie with a 3/4 in. headed anchor bolt, embedded in the bond beam and 18 10d nails into the side of the joist (Figure 10.2-10). Modifying the allowable values using the procedure in Sec. 10.1.4.5 results in a design strength of:

$\lambda\phi Z'$ = (1.0)(0.65)(4.73) = 3.07 kips per anchor > 2.50 kips OK

The joists anchored to the masonry wall must also be adequately connected to the diaphragm sheathing. Determine the adequacy of the typical nailing for intermediate framing members. The nail spacing is 12 in. and the joist length is 20 ft, so there are 20 nails per joist. From the AF&PA Manual, Structural Connections Supplement, the strength of a single 10d common nail is:

$\lambda\phi Z'$ = (1.0)(0.65)(0.298) = 0.194 kips per nail
20(0.194) = 3.88 kips > 2.50 kips *OK*

The embedded bolt also serves as the ledger connection, for both gravity loading, and in-plane shear transfer at the diaphragm. Therefore, the strength of the anchorage to masonry, and the strength of the bolt in the wood ledger, must be checked.

For the anchorage to masonry, check the combined tension and shear, resulting from the out-of-plane seismic loading (2.50 kips/bolt), and the vertical gravity loading. Assuming 20 psf dead load (roof live load is not included), and a 10-foot tributary roof width, the vertical load per bolt = (20 psf)(10 ft)(2 ft)/1000 = 0.40 kip. Using the load combination described previously, the design horizontal tension and vertical shear on the bolt are:

$b_a = 1.0Q_E = 2.50$ kips
$b_v = 1.4D = 1.4(0.40) = 0.56$ kip

The anchor bolts in masonry, are designed according to *Provisions* Sec. 11.3.12. Using 3/4 in. headed anchor bolts, both axial strength, B_a, and shear strength, B_v, will be governed by masonry breakout. Per *Provisions* Eq. 11.3.12.3-1, and 11.3.12.1-1, respectively, the design strengths are:

$$B_a = 4\phi A_p \sqrt{f'_m}$$

$$B_v = 1750\phi\sqrt[4]{f'_m A_b}$$

where $\varphi = 0.5$, $f'_m = 2{,}000$ psi, A_b = tensile area of bolt = 0.44 in.^{2}, and A_p = projected area on the masonry surface of a right circular cone = 113 in.^{2} (assuming 6 in. effective embedment). Therefore, B_a = 10.1 kips and B_v = 4.8 kips. Shear and tension are combined per *Provisions* Eq. 11.3.12.4 as:

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} = \frac{2.50}{10.1} + \frac{0.56}{4.8} = 0.36 < 1.0 \qquad \text{OK}$$

[The 2003 *Provisions* refers to ACI 530, Sec. 3.1.6 for strength design of anchorage to masonry, as modified by 2003 Provisions Sec. 11.2. In general, the methodology for anchorage design using ACI 530 should be comparable to the 2000 *Provisions*, though some of the equations and reduction factors may be different. In addition, the 2003 *Provisions* require that anchors are either controlled by a ductile failure mode or are designed for 2.5 times the anchorage force.]

Figure 10.2-9 summarizes the details of the connection. In-plane seismic shear transfer (combined with gravity) and orthogonal effects are considered in a subsequent section.

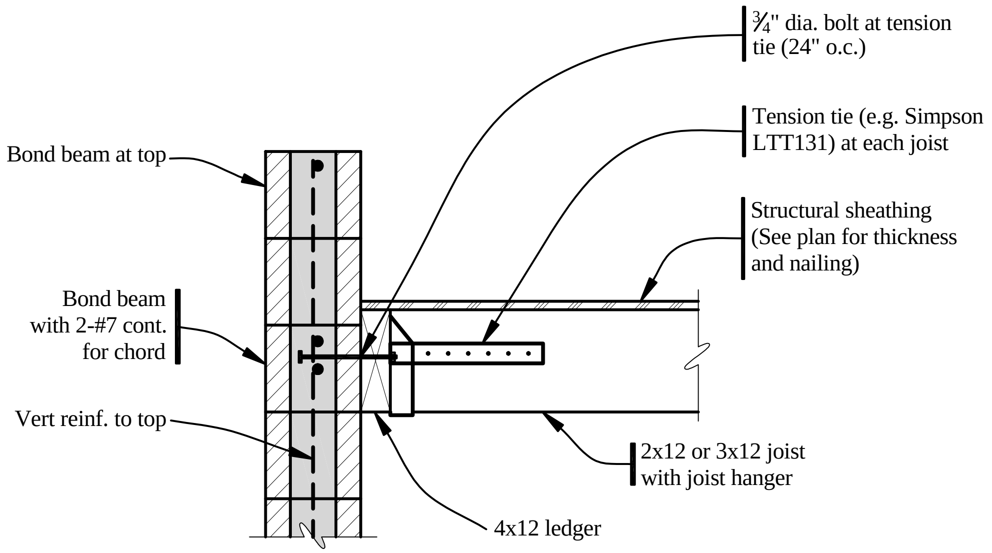


Figure 10.2-9 Anchorage of masonry wall perpendicular to joists (1.0 in. = 25.4 mm).

According to *Provisions* Sec. 5.2.6.3.2 [4.6.2.1, diaphragms must have continuous cross-ties to distribute the anchorage forces into the diaphragms. Although the *Provisions* do not specify a maximum spacing, 20 ft is common practice for this type of construction and Seismic Design Category.

For cross-ties at 20 ft on center, the wall anchorage force per cross-tie is:

(1.25 klf)(20 ft) = 25.0 kips

Try a 3×12 (Douglas fir-Larch No. 1) as a cross-tie. Assuming one row of 15/16 in. bolt holes, the net area of the section is 25.8 in.^{2} Tension strength (parallel to wood grain) per the AF&PA Structural Lumber Supplement, is:

$\lambda\phi T'$ = (1.0)(0.8)(1.82)(25.8) = 37.5 kips > 25.0 kips *OK*

At the splices, try a double tie-down device with four 7/8 in. bolts in double shear through the 3×12 (Figure 10.2-10). Product catalogs provide design capacities for single tie-downs only; the design of double hold downs requires two checks. First, consider twice the capacity of one tie-down and, second, consider the capacity of the bolts in double shear.

For the double tie-down, use the procedure in Sec. 10.1.4.5 to modify the allowable values:

$2\lambda\phi Z'$ = 2(1.0)(0.65)(20.66) = 26.9 kips > 25.0 kips OK

The reduction (group action) factor for multiple bolts, C_g = 0.97.

For the four bolts, the AF&PA Manual, Structural Connections Supplement, gives:

$4\lambda\phi C_g Z'$ = 4(1.0)(0.65)(0.97)(10.17) = 25.6 kips > 25.0 kips

OK

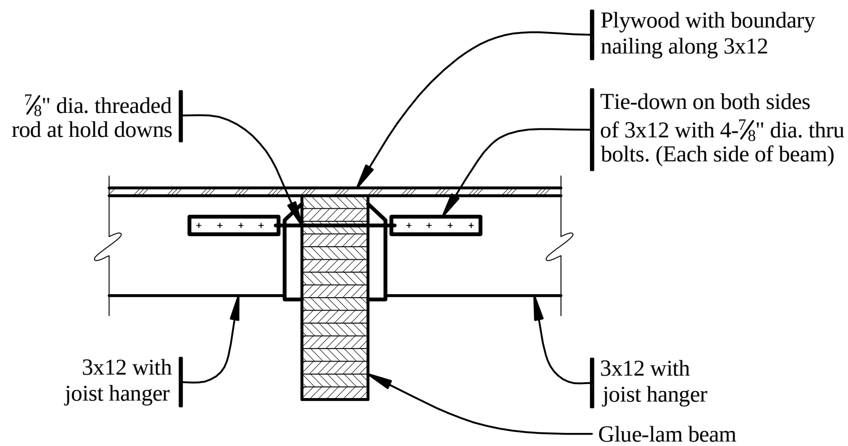


Figure 10.2-10 Chord tie at roof opening (1.0 in. = 25.4 mm).

In order to transfer the wall anchorage forces into the cross-ties, the subdiaphragms between these ties must be checked per *Provisions* Sec. 5.2.6.3.2 [4.6.2.1]. There are several ways to perform these subdiaphragm calculations. One method is illustrated in Figure 10.2-11. The subdiaphragm spans between cross-ties and utilizes the glued-laminated beam and ledger as it's chords. The 1-to-1 aspect ratio meets the requirement of 2½ to 1 for subdiaphragms per *Provisions* Sec. 5.2.6.3.2 [4.2.6.1].

For the typical subdiaphragm (Figure 10.2-11):

F_p = 1.25 klf
v = (1.25)(20/2)/20 = 0.625 klf.

The subdiaphragm demand is less than the minimum diaphragm capacity (0.83 klf along the center of the side walls). In order to develop the subdiaphragm strength, and boundary nailing must be provided along the cross-tie beams.

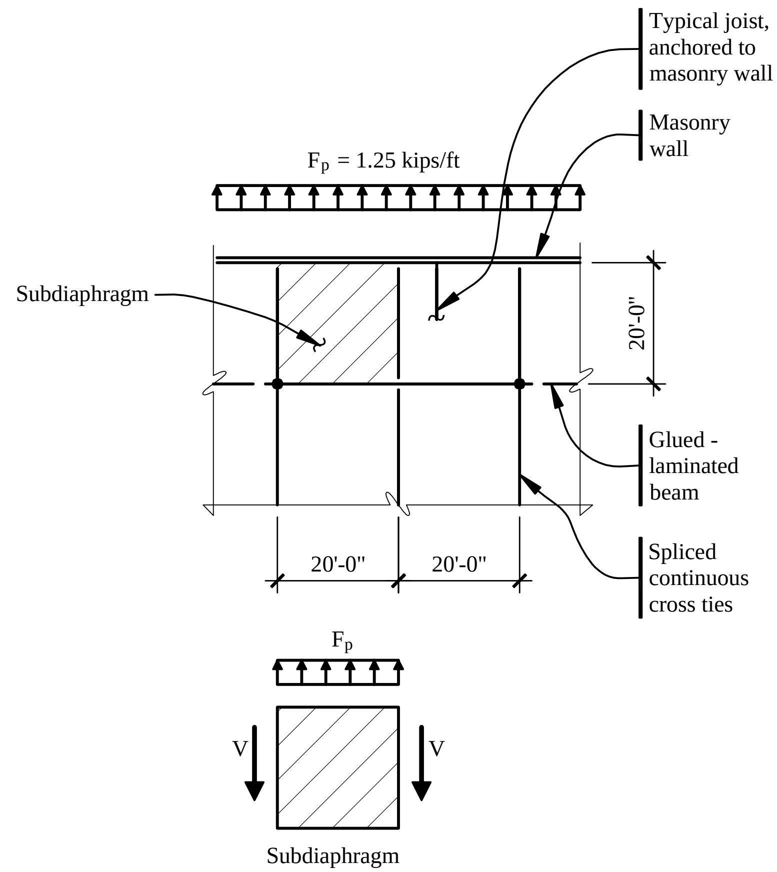


Figure 10.2-11 Cross tie plan layout and subdiaphragm free-body diagram for side walls (1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m).

10.2.4.3.2 Anchorage at Joists Parallel to Walls (End Walls)

Where the joists are parallel to the walls, tied elements must transfer the forces into the main body of the diaphragm, which can be accomplished by using either metal strapping and blocking or metal rods and blocking. This example uses threaded rods that are inserted through the joists and coupled to the anchor bolt (Figure 10.2-12). Blocking is added on both sides of the rod to transfer the force into the plywood sheathing. The tension force in the rod causes a compression force on the blocking through the nut, and bearing plate at the innermost joist.

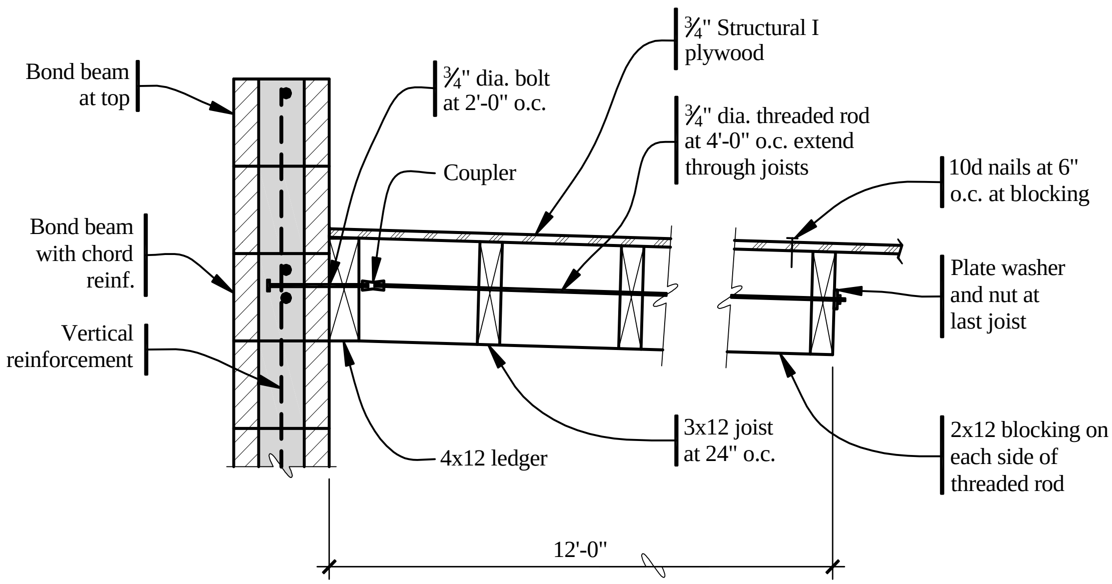


Figure 10.2-12 Anchorage of masonry wall parallel to joists (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

The anchorage force at the end walls is 1.98 klf. Space the connections at 4 ft on center so that the wall need not be designed for flexure (*Provisions* Sec. 5.2.6.1.2 [4.6.1.2]). Thus, the anchorage force is 7.92 kips per anchor.

Try a 3/4 in. headed anchor bolt, embedded into the masonry. In this case, gravity loading on the ledger is negligible and can be ignored, and the anchor can be designed for tension only. (In-plane shear transfer and orthogonal effects are considered later.)

As computed for 3/4 in. headed anchor bolts (with 6 in. embedment), the design axial strength B_a = 10.1 kips > 7.92 kips. Therefore, the bolt is acceptable.

Using couplers rated for 125 percent of the strength of the rod material, the threaded rods are then coupled to the anchor bolts and extend six joist spaces (12 ft) into the roof framing. (The 12 ft are required for the subdiaphragm force transfer as discussed below.)

Nailing the blocking to the plywood sheathing is determined, using the AF&PA Structural Connections Supplement. As computed previously, the capacity of a single 10d common nail, $\lambda\varphi Z'$ = 0.194 kips. Thus, 7.92/0.194 = 41 nails are required. This corresponds to a nail spacing of about 7 in. for two 12-ft rows of blocking. Space nails at 6 in. for convenience.

Use the glued-laminated beams (at 20 ft on center) to provide continuous cross-ties, and check the subdiaphragms between the beams to provide adequate load transfer to the beams per Provisions Sec. 5.2.6.3.2 [4.6.2.1].

Design tension force on beam = (1.98 klf)(20 ft) = 39.6 kips

The stress on the beam is f_t = 39.600/[8.75(24)] = 189 psi, which is small. The beam is adequate for combined moment due to gravity loading and axial tension.

At the beam splices, try one-in. bolts with steel side plates. Per the AF&PA Manual, Structural Connections Supplement:

lfZ' = (1.0)(0.65)(18.35) = 11.93 kips per bolt.

The number of bolts required = 39.6/11.93 = 3.3.

Use four bolts in a single row at midheight of the beam, with 1/4-in. by 4-in. steel side plates. The reduction (group action factor) for multiple bolts is negligible. Though not included in this example, the steel side plates should be checked for tension capacity on the gross and net sections. There are pre-engineered hinged connectors for glued-laminated beams that could provide sufficient tension capacity for the splices.

In order to transfer the wall anchorage forces into the cross-ties, the subdiaphragms between these ties must be checked per *Provisions* Sec. 5.2.6.3.2 [4.6.2.1]. The procedure is similar to that used for the side walls as described previously. The end wall condition is illustrated in Figure 10.2-13. The subdiaphragm spans between beams and utilizes a roof joist as its chord. In order to adequately engage the subdiaphragm, the wall anchorage ties must extend back to this chord. The maximum aspect ratio for subdiaphragms is 2½ to 1 per *Provisions* Sec. 5.2.6.3.2 [4.6.2.1]. Therefore, the minimum depth is 20/2.5 = 8 ft.

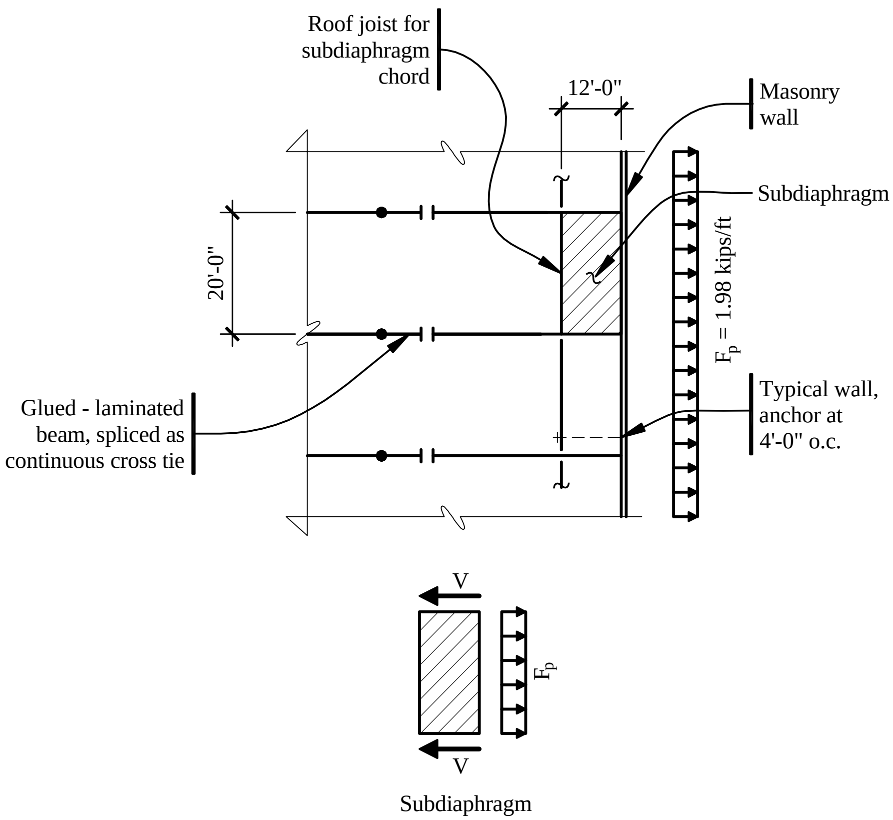


Figure 10.2-13 Cross tie plan layout and subdiaphragm free-body diagram for end walls (1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m).

For the typical subdiaphragm (Figure 10.2-13):

F_p = 1.98 klf
v = (1.98)(20/2)/8 = 2.48 klf

As computed previously (see Table 10.2-1 and Figure 10.2.2), the diaphragm strength in this area is 1.60 klf < 2.48 klf. Therefore, increase the subdiaphragm depth to 12 ft (six joist spaces):

v = (1.98)(20/2)/12 = 1.65 klf ≈1.60 klf OK

In order to develop the subdiaphragm strength, boundary nailing must be provided along the cross-tie beams. There are methods of refining this analysis using multiple subdiaphragms so that all of the tension anchors need not extend 12 ft into the building.

10.2.4.3.3 Transfer of Shear Wall Forces

The in-plane diaphragm shear must be transferred to the masonry wall by the ledger, parallel to the wood grain. The connection must have sufficient capacity for the diaphragm demands as:

Side walls = 0.575 klf
End walls = 1.282 klf

For each case, the capacity of the bolted wood ledger and the capacity of the anchor bolts embedded into masonry must be checked. Because the wall connections provide a load path for both in-plane shear transfer and out-of-plane wall forces, the bolts must be checked for orthogonal load effects in accordance with *Provisions* Sec. 5.2.5.2.3 [4.4.2.3]. That is, the combined demand must be checked for 100 percent of the lateral load effect in one direction (e.g., shear) and 30 percent of the lateral load effect in the other direction (e.g., tension).

At the side walls, the wood ledger with 3/4-in. bolts (Figure 10.2-9) must be designed for gravity loading (0.56 kip per bolt as computed above) as well as seismic shear transfer. The seismic load per bolt (at 2 ft on center) is 0.575(2) = 1.15 kips.

Combining gravity shear and seismic shear, produces a resultant force of 1.38 kips at an angle of 26 degrees from the axis of the wood grain. Using the formulas for bolts at an angle to the grain per the AF&PA Structural Connections Supplement gives

lfZ' = (1.0)(0.65)(3.47) = 2.26 kips > 1.38. OK

This bolt spacing satisfies the load combination for gravity loading only.

For the check of the embedded anchor bolts, the factored demand on a single bolt is 1.15 kips in horizontal shear (in-plane shear transfer), 2.50 kips in tension (out-of-plane wall anchorage), 0.56 kip in vertical shear (gravity). Orthogonal effects are checked, using the following two equations:

$$\frac{(0.3)(2.5)}{19.1}+\frac{\sqrt{1.15^2+0.56^2}}{4.8}=0.34$$

and

$$\frac{(2.5)}{10.1}+\frac{\sqrt{[0.3(1.15)]^2+0.56^2}}{4.8}=0.38 \text{ (controls)} < 1.0 \qquad \text{OK}$$

At the end walls, the ledger with 3/4-in. bolts (Figure 10.2-12) need only be checked for in-plane seismic shear because gravity loading is negligible. For bolts spaced at 4 ft on center, the demand per bolt is 1.282(4) = 5.13 kips parallel to the grain of the wood. Per the AF&PA Structural Manual, Connections Supplement:

Z' = (1.0)(0.65)(5.37) = 3.49 kips < 5.13 kips NG

Therefore, add 3/4-in. headed bolts evenly spaced between the tension ties such that the bolt spacing is 2 ft on center and the demand per bolt is 1.282(2) = 2.56 kips. These added bolts are used for in-plane shear only and do not have coupled tension tie rods.

For the check of the embedded bolts, the factored demand on a single bolt is 2.56 kips in horizontal shear (in-plane shear transfer), 7.92 kips in tension (out-of-plane wall anchorage), 0 kip in vertical shear (gravity is negligible). Orthogonal effects are checked using the following two equations:

$$\frac{(0.3)(7.92)}{10.1}+\frac{2.56}{4.8}=0.77$$

and

$$\frac{7.92}{10.1}+\frac{0.3(2.56)}{4.8}=0.94 \text{ (controls)} < 1.0 \qquad \text{OK}$$

Therefore, the wall connections satisfy the requirements for combined gravity, and seismic loading, including orthogonal effects.

11

SEISMICALLY ISOLATED STRUCTURES

Charles A. Kircher, P.E., Ph.D.

Chapter 13 of the 2000 *NEHRP Recommended Provisions* addresses the design of buildings that incorporate a seismic isolation system. The *Provisions* provides essentially a stand alone set of design and analysis criteria for an isolation system. Chapter 13 defines load, design, and testing requirements specific to the isolation system and interfaces with the appropriate materials chapters for design of the structure above the isolation system and of the foundation and structural elements below.

A discussion of background, basic concepts, and analysis methods is followed by an example that illustrates the application of the *Provisions* to the structural design of a building with an isolation system. In this example, the building is a three-story emergency operations center (EOC) with a steel concentrically braced frame above the isolation system. Although the facility is hypothetical, it is of comparable size and configuration to actual base-isolated EOCs, and is generally representative of base-isolated buildings.

The EOC is located in San Francisco and has an isolation system that utilizes elastomeric bearings, a type of bearing commonly used for seismic isolation of buildings. The example comprehensively describes the EOC's configuration, defines appropriate criteria and design parameters, and develops a preliminary design using the equivalent lateral force (ELF) procedure of Chapter 13. It also includes a check of the preliminary design using dynamic analysis as required by the *Provisions* and specifies isolation system design and testing criteria.

Located in a region of very high seismicity, the building is subject to particularly strong ground motions. Large seismic demands pose a challenge for the design of base-isolated structures in terms of the capacity of the isolation system and the configuration of the structure above the isolation system. The isolation system must accommodate large lateral displacements (e.g., in excess of 2 ft). The structure above the isolation system should be configured to produce the smallest practical overturning loads (and uplift displacements) on the isolators. The example addresses these issues and illustrates that isolation systems can be designed to meet the requirements of the *Provisions*, even in regions of very high seismicity. Designing an isolated structure in a region of lower seismicity would follow the same approach. The isolation system displacement, overturning forces, and so forth would all be reduced, and therefore, easier to accommodate using available isolation system devices.

The isolation system for the building in the example is composed of high-damping rubber (HDR) elastomeric bearings. HDR bearings are constructed with alternating layers of rubber and steel plates all sheathed in rubber. The first base-isolated building in the United States employed this type of isolation system. Other types of isolation systems used to base isolate buildings employed lead-core elastomeric bearings (LR) and sliding isolators, such as the friction pendulum system (FPS). In regions of very high seismicity, viscous dampers have been used to supplement isolation system damping (and reduce displacement demand). Using HDR bearings in this example should not be taken as an endorsement of this particular type of isolator to the exclusion of others. The concepts of the *Provisions* apply to all types of isolations systems, and other types of isolators (and possible supplementary dampers) could have been used equally well in the example.

In addition to the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereafter, the *Provisions* and *Commentary*), the following documents are either referenced directly or are useful aids for the analysis and design of seismically isolated structures.

ATC 1996	Applied Technology Council. 1996. Seismic Evaluation and Retrofit of Buildings, ATC40.
Constantinou	Constantinou, M. C., P. Tsopelas, A. Kasalanati, and E. D. Wolff. 1999. *Property Modification Factors for Seismic Isolation Bearings*, Technical Report MCEER-99-0012. State University of New York.
CSI	Computers and Structures, Inc. (CSI). 1999. *ETABS Linear and Nonlinear Static and Dynamic Analysis and Design of Building Systems*.
FEMA 273	Federal Emergency Management Agency. 1997. *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273.
FEMA 222A	Federal Emergency Management Agency. 1995. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings,* FEMA 222A.
91 UBC	International Conference of Building Officials. 1991. *Uniform Building Code*.
94 UBC	International Conference of Building Officials. 1994. *Uniform Building Code*.
Kircher	Kircher, C. A., G. C. Hart, and K. M. Romstad. 1989. "Development of Design Requirements for Seismically Isolated Structures" in *Seismic Engineering and Practice*, Proceedings of the ASCE Structures Congress, American Society of Civil Engineers, May 1989.
SEAOC 1999	Seismology Committee, Structural Engineers Association of California. 1999. *Recommended Lateral Force Requirements and Commentary*, 7th Ed.
SEAOC 1990	Seismology Committee, Structural Engineers Association of California. 1990. *Recommended Lateral Force Requirements and Commentary*, 5th Ed.
SEAONC Isolation	Structural Engineers Association of Northern California. 1986. *Tentative Seismic Isolation Design Requirements*.

Although the guide is based on the 2000 *Provisions* , it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantiative technical changes to the 2003 *Provisions* and its primary reference documents. While the general changes to the document are described , the deign examples and calculations have not been revised to reflect the changes to the 2003 *Provisions.*

In the 2003 edition of the *Provisions*, Chapter 13 has been restructured so that it is better integrated into the *Provisions* as a whole and is less of a stand alone set of requirements. Where they affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents may be noted.

11.1 BACKGROUND AND BASIC CONCEPTS

Seismic isolation, commonly referred to as base isolation, is a design concept that presumes a structure can be substantially decoupled from potentially damaging earthquake ground motions. By decoupling the structure from ground shaking, isolation reduces the level of response in the structure that would otherwise occur in a conventional, fixed-base building. Conversely, base-isolated buildings may be designed with a reduced level of earthquake load to produce the same degree of seismic protection. That decoupling is achieved when the isolation scheme makes the fundamental period of the isolated structure several times greater than the period of the structure above the isolation system.

The potential advantages of seismic isolation and the advancements in isolation system products led to the design and construction of a number of isolated buildings and bridges in the early 1980s. This activity, in turn, identified a need to supplement existing seismic codes with design requirements developed specifically for such structures. These requirements assure the public that isolated buildings are safe and provide engineers with a basis for preparing designs and building officials with minimum standards for regulating construction.

Initial efforts developing design requirements for base-isolated buildings began with ad hoc groups of the Structural Engineers Association of California (SEAOC), whose Seismology Committee has a long history of contributing to codes. The northern section of SEAOC was the first to develop guidelines for the use of elastomeric bearings in hospitals. These guidelines were adopted in the late 1980s by the California Office of Statewide Health Planning and Development (OSHPD) and were used to regulate the first base-isolated hospital in California. At about the same time, the northern section of SEAOC published SEAONC Isolation, first set of general requirements to govern the design of base-isolated buildings. Most of the basic concepts for the design of seismically isolated structures found in the *Provisions* can be traced back to the initial work by the northern section of SEAOC.

By the end of the 1980s, the Seismology Committee of SEAOC recognized the need to have a more broadly based document and formed a statewide committee to develop design requirements for isolated structures Kircher. The "isolation" recommendations became an appendix to the 1990 SEAOC Blue Book. The isolation appendix was adopted with minor changes as a new appendix in the 1991 Uniform Building Code and has been updated every three years, although it remains largely the same as the original 91 UBC appendix. (SEAOC 1990 and 1999 are editions of SEAOC's *Recommended Lateral Force Requirements and Commentary*, which is also known as the *Blue Book*.)

In the mid-1990s, the Provisions Update Committee of the Building Seismic Safety Council incorporated the isolation appendix of the 94 UBC into the 1994 *Provisions* (FEMA 222A). Differences between the *Uniform Building Code* (UBC) and the *Provisions* were intentionally minimized and subsequent editions of the *UBC* and the *Provisions* are nearly identical. Additional background may be found in the commentary to the 1999 SEAOC *Blue Book*.

The *Provisions* for designing the isolation system of a new building were used as the starting point for the isolation system requirements of the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* (FEMA 273). FEMA 273 follows the philosophy that the isolation system for a rehabilitated building should be comparable to that for a new building (for comparable ground shaking criteria, etc.). The superstructure, however, could be quite different, and FEMA 273 provides more suitable design requirements for rehabilitating existing buildings using an isolation system.

11.1.1 Types of Isolation Systems

The *Provisions* requirements are intentionally broad, accommodating all types of acceptable isolation systems. To be acceptable, the *Provisions* requires the isolation system to:

1. Remain stable for maximum earthquake displacements,
2. Provide increasing resistance with increasing displacement,
3. Have limited degradation under repeated cycles of earthquake load, and
4. Have well-established and repeatable engineering properties (effective stiffness and damping).

The *Provisions* recognizes that the engineering properties of an isolation system, such as effective stiffness and damping, can change during repeated cycles of earthquake response (or otherwise have a range of values). Such changes or variability of design parameters are acceptable provided that the design is based on analyses that conservatively bound (limit) the range of possible values of design parameters.

The first seismic isolation systems used in buildings in the United States were composed of elastomeric bearings that had either a high-damping rubber compound or a lead core to provide damping to isolated modes of vibration. Other types of isolation systems now include sliding systems, such as the friction pendulum system (FPS), or some combination of elastomeric and sliding isolators. Some applications at sites with very strong ground shaking use supplementary fluid-viscous dampers in parallel with either sliding or elastomeric isolators to control displacement. While generally applicable to all types of systems, certain requirements of the *Provisions* (in particular, prototype testing criteria) were developed primarily for isolation systems with elastomeric bearings.

Isolation systems typically provide only horizontal isolation and are rigid or semi-rigid in the vertical direction. A rare exception to this rule is the full isolation (horizontal and vertical) of a building in southern California isolated by large helical coil springs and viscous dampers. While the basic concepts of the *Provisions* can be extended to full isolation systems, the requirements are only for horizontal isolation systems. The design of a full isolation system requires special analyses that explicitly include vertical ground shaking and the potential for rocking response.

Seismic isolation is commonly referred to as base isolation because the most common location of the isolation system is at or near the base of the structure. The *Provisions* does not restrict the plane of isolation to the base of the structure but does require the foundation and other structural elements below the isolation system to be designed for unreduced (R_I = 1.0) earthquake forces.

11.1.2 Definition of Elements of an Isolated Structure

The design requirements of the *Provisions* distinguish between structural elements that are either components of the isolation system or part of the structure below the isolation system (e.g., foundation) and elements of the structure above the isolation system. The isolation system is defined by the *Provisions* as:

> The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, energy-dissipation devices, and/or the displacement restraint system if such systems and devices are used to meet the design requirements of Chapter 13.

Figure 11.1-1 illustrates this definition and shows that the isolation system consists not only of the isolator units but also of the entire collection of structural elements required for the system to function properly. The isolation system typically includes segments of columns and connecting girders just above the isolator units because such elements resist moments (due to isolation system displacement) and their yielding or failure could adversely affect the stability of isolator units.

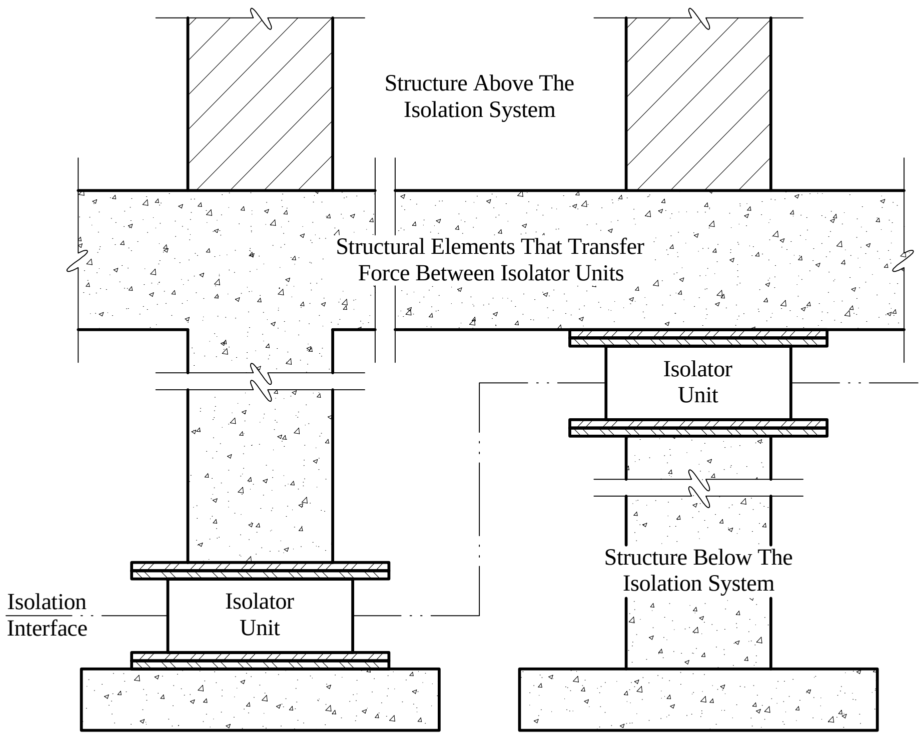


Figure 11.1-1 Isolation system terminology.

The isolation interface is an imaginary boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which is assumed to move rigidly with the ground. Typically, the isolation interface is a horizontal plane, but it may be staggered in elevation in certain applications. The isolation interface is important for design of nonstructural components, including components of electrical and mechanical systems that cross the interface and must accommodate large relative displacements.

The wind-restraint system is typically an integral part of isolator units. Elastomeric isolator units are very stiff at very low strains and usually satisfy drift criteria for wind loads, and the static (breakaway) friction force of sliding isolator units is usually greater than the wind force.

11.1.3 Design Approach

The design of isolated structures using the *Provisions* (like the *UBC* and SEAOC's *Blue Book*) has two objectives: achieving life safety in a major earthquake and limiting damage due to ground shaking. To meet the first performance objective, the isolation system must be stable and capable of sustaining forces and displacements associated with the maximum considered earthquake and the structure above the isolation system must remain essentially elastic when subjected to the design earthquake. Limited ductility demand is considered necessary for proper functioning of the isolation system. If significant inelastic response was permitted in the structure above the isolation system, unacceptably large drifts could result due to the nature of long-period vibration. Limiting ductility demand on the superstructure has the additional benefit of meeting the second performance objective of damage control.

The *Provisions* addresses the performance objectives by requiring:

1. Design of the superstructure for forces associated with the design earthquake, reduced by only a fraction of the factor permitted for design of conventional, fixed-base buildings (i.e., $R_I = 3/8\ R \leq 2.0$).

2. Design of the isolation system and elements of the structure below the isolation system (e.g., foundation) for unreduced design earthquake forces.

3. Design and prototype testing of isolator units for forces (including effects of overturning) and displacements associated with the maximum considered earthquake.

4. Provision of sufficient separation between the isolated structure and surrounding retaining walls and other fixed obstructions to allow unrestricted movement during the maximum considered earthquake.

11.1.4 Effective Stiffness and Effective Damping

The *Provisions* utilizes the concepts of effective stiffness and damping to define key parameters of inherently nonlinear, inelastic isolation systems in terms of amplitude-dependent linear properties. Effective stiffness is the secant stiffness of the isolation system at the amplitude of interest. Effective damping is the amount of equivalent viscous damping described by the hysteresis loop at the amplitude of interest. Figure 11.1-2 shows the application of these concepts to both hysteretic isolator units (e.g., friction or yielding devices) and viscous isolator units and shows the *Provisions* equations used to determine effective stiffness and damping from tests of prototypes. Ideally, the effective damping of velocity-dependent devices (including viscous isolator units) should be based on the area of hysteresis loops measured during cyclic testing of the isolation system at full-scale earthquake velocities. Tests of prototypes are usually performed at lower velocities (due to test facility limitations), resulting in hysteresis loops with less area, which produce lower (conservative) estimates of effective damping.

11.2 CRITERIA SELECTION

As specified in the *Provisions* the design of isolated structures must be based on the results of the equivalent lateral force (ELF) procedure, response spectrum analysis, or (nonlinear) time history analysis. Because isolation systems are typically nonlinear, linear methods (ELF procedure and response spectrum analysis) use effective stiffness and damping properties to model nonlinear isolation system components.

The ELF procedure is intended primarily to prescribe minimum design criteria and may be used for design of a very limited class of isolated structures (without confirmatory dynamic analyses). The simple equations of the ELF procedure are useful tools for preliminary design and provide a means of expeditious review and checking of more complex calculations. The *Provisions* also uses these equations to establish lower-bound limits on results of dynamic analysis that may be used for design. Table 11.2-1 summarizes site conditions and structure configuration criteria that influence the selection of an acceptable method of analysis for designing of isolated structures. Where none of the conditions in Table 11.2-1 applies, all three methods are permitted.

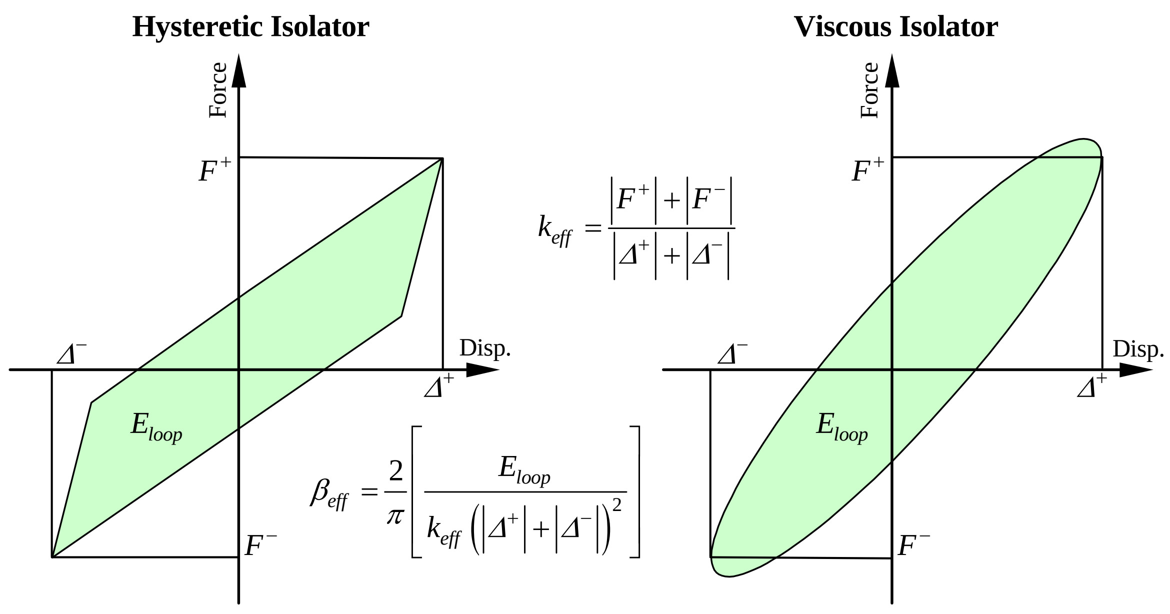


Figure 11.1-2 Effective stiffness and effective damping.

Table 11.2-1 Acceptable Methods of Analysis*

Site condition or Structure Configuration Criteria	ELF Procedure	Response Spectrum Analysis	Time History Analysis
Site Conditions			
Near-source (S_1 > 0.6)	NP	P	P
Soft soil (Site Class E or F)	NP	NP	P
Superstructure Configuration			
Flexible or irregular superstructure (height > 4 stories, height > 65 ft, or T_M > 3.0 sec., or $T_D \leq 3T$)	NP	P	P
Nonlinear superstructure (requiring explicit modeling of nonlinear elements; *Provisions* Sec. 13.2.5.3.1) [13.4.1.2]	NP	NP	P
Isolation System Configuration			
Highly nonlinear isolation system or system that otherwise does not meet the criteria of *Provisions* Sec. 13.2.5.2, Item 7 [13.2.4.1, Item 7]	NP	NP	P

* P indicates permitted and NP indicates not permitted by the *Provisions*.

Seismic criteria are based on the same site and seismic coefficients as conventional, fixed-base structures (e.g., mapped value of S_1 as defined in *Provisions* Chapter 4 [3]). Additionally, site-specific design criteria are required for isolated structures located on soft soil (Site Class E of F) or near an active source such that S_1 is greater than 0.6, or when nonlinear time history analysis is used for design.

11.3 EQUIVALENT LATERAL FORCE PROCEDURE

The equivalent lateral force (ELF) procedure is a displacement-based method that uses simple equations to determine isolated structure response. The equations are based on ground shaking defined by 1 second spectral acceleration and the assumption that the shape of the design response spectrum at long periods is inversely proportional to period as shown in *Provisions* Figure 4.1.2.6 [3.3-15]. [In the 2003 edition of the *Provisions* , there is also a $1/T^2$ portion of the spectrum at periods greater than T_L. However, in most parts of the Unites States T_L is longer than the period of typical isolated structures.] Although the ELF procedure is considered a linear method of analysis, the equations incorporate amplitude-dependent values of effective stiffness and damping to implicitly account for the nonlinear properties of the isolation system. The equations are consistent with the nonlinear static procedure of FEMA 273 assuming the superstructure is rigid and lateral displacements to occur primarily in the isolation system.

11.3.1 Isolation System Displacement

The isolation system displacement for the design earthquake is determined by using *Provisions* Eq. 13.3.3.1 [13.3-1]:

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D}$$

where the damping factor ,B_D, is based on effective damping, β_D, using *Provisions* Table 13.3.3.1 [13.3-1]. This equation describes the peak (spectral) displacement of a single-degree-of-freedom (SDOF) system with period, T_D, and damping, β_D, for the design earthquake spectrum defined by the seismic coefficient, S_{D1}. S_{D1} corresponds to 5 percent damped spectral response at a period of 1 second. B_D, converts 5 percent damped response to the level of damping of the isolation system. B_D is 1.0 when effective damping, β_D, is 5 percent of critical. Figure 11.3-1 illustrates the underlying concepts of *Provisions* Eq. 13.3.3.1 [13.3-1] and the amplitude-dependent equations of the *Provisions* for effective period, T_D, and effective damping, β_D.

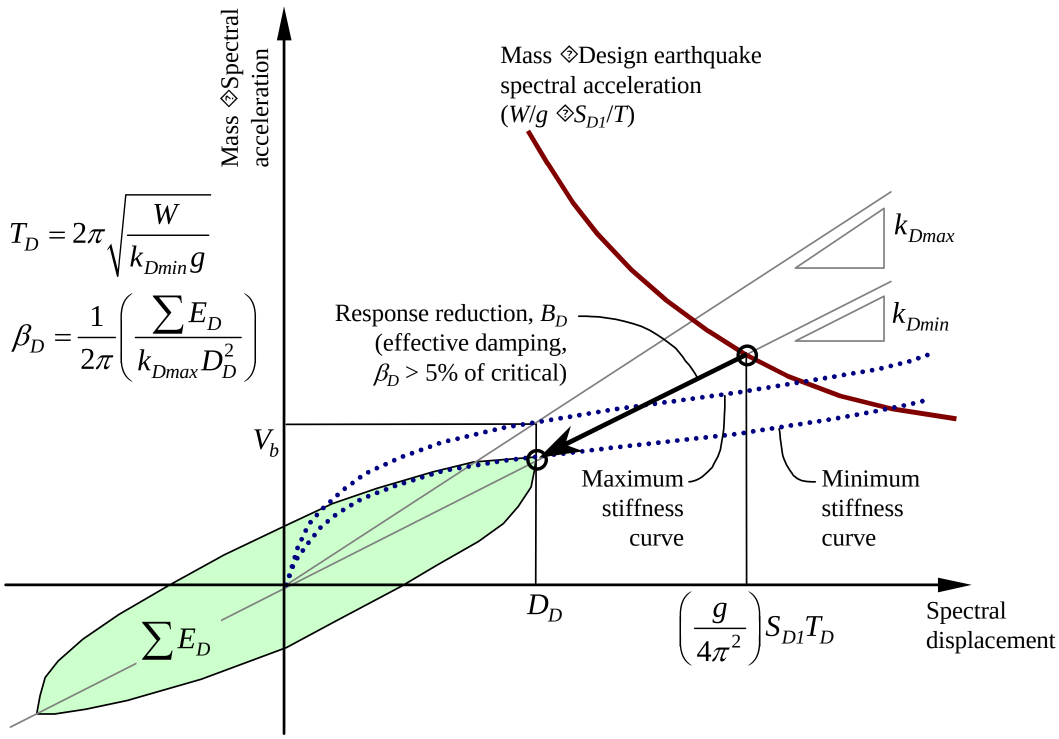


Figure 11.3-1 Isolation system capacity and earthquake demand.

The equations for maximum displacement, D_M, and design displacement, D_D, reflect differences due to the corresponding levels of ground shaking. The maximum displacement is associated with the maximum considered earthquake (characterized by S_{M1}) whereas the design displacement corresponds to the design earthquake (characterized by S_{D1}). In general, the effective period and the damping factor (T_M and B_M, respectively) used to calculate the maximum displacement are different from those used to calculate the design displacement (T_D and B_D) because the effective period tends to shift and effective damping may change with the increase in the level of ground shaking.

As shown in Figure 11.3-1, the calculation of effective period, T_D, is based on the minimum effective stiffness of the isolation system, k_{Dmin}, as determined by prototype testing of individual isolator units. Similarly, the calculation of effective damping is based on the minimum loop area, E_D, as determined by prototype testing. Use of minimum effective stiffness and damping produces larger estimates of effective period and peak displacement of the isolation system.

The design displacement, D_D, and maximum displacement, D_M, represent peak earthquake displacements at the center of mass of the building without the additional displacement, that can occur at other locations due to actual or accidental mass eccentricity. Equations for determining total displacement, including the effects of mass eccentricity as an increase in the displacement at the center of mass, are based on the plan dimensions of the building and the underlying assumption that building mass and isolation stiffness have a similar distribution in plan. The increase in displacement at corners for 5 percent mass eccentricity is about 15 percent if the building is square in plan, and as much as 30 percent if the building is long in plan. Figure 11.3-2 illustrates design displacement, D_D, and maximum displacement, D_M, at the center of mass of the building and total maximum displacement, D_{TM}, at the corners of an isolated building.

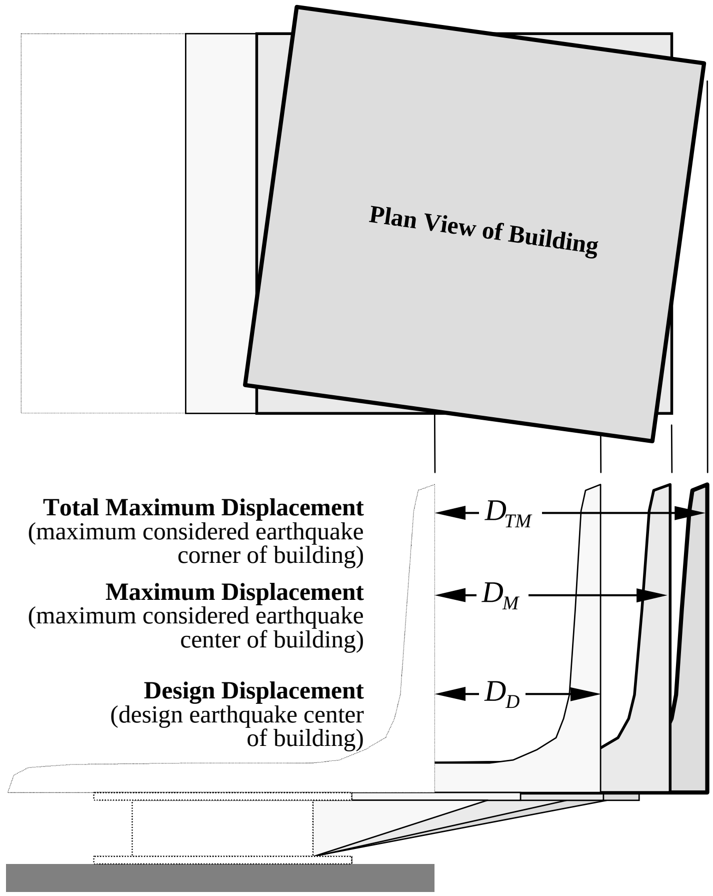


Figure 11.3-2 Design, maximum, and total maximum displacement.

11.3.2 Design Forces

Forces required by the *Provisions* for design of isolated structures are different for design of the superstructure and design of the isolation system and other elements of the structure below the isolation system (e.g., foundation). In both cases, however, use of the maximum effective stiffness of the isolation system is required to determine a conservative value of design force.

In order to provide appropriate overstrength, peak design earthquake response (without reduction) is used directly for design of the isolation system and the structure below. Design for unreduced design earthquake forces is considered sufficient to avoid inelastic response or failure of connections and other elements for ground shaking as strong as that associated with the maximum considered earthquake (i.e., shaking as much as 1.5 times that of the design earthquake). The design earthquake base shear, V_b, is given by *Provisions* Eq. 13.3.4.1 [13.3-7]:

$$V_b = k_{Dmax} D_{D,}$$

where k_{Dmax} is the maximum effective stiffness of the isolation system at the design displacement, D_D. Because the design displacement is conservatively based on minimum effective stiffness, *Provisions* Eq. 13.3.4.1 implicitly induces an additional conservatism of a worst case combination mixing maximum and minimum effective stiffness in the same equation. Rigorous modeling of the isolation system for dynamic analyses precludes mixing of maximum and minimum stiffness in the same analysis (although separate analyses are typically required to determine bounding values of both displacement and force).

Design earthquake response is reduced by a modest factor for design of the superstructure above the isolation interface, as given by *Provisions* Eq. 13.3.4.2 [13.3-8]:

$$V_s = \frac{V_b}{R_I} = \frac{k_{Dmax} D_D}{R_I}$$

The reduction factor, R_I, is defined as three-eighths of the R factor for the seismic-force-resisting system of the superstructure, as specified in *Provisions* Table 5.2.2 [4.3-1], with an upper-bound value of 2.0. A relatively small R_I factor is intended to keep the superstructure essentially elastic for the design earthquake (i.e., keep earthquake forces at or below the true strength of the seismic-force-resisting system). The *Provisions* also impose three limits on design forces that require the value of V_s to be at least as large as each of:

1. The shear force required for design of a conventional, fixed-base structure of period T_D.
2. The shear force required for wind design, and/or
3. A factor of 1.5 times the shear force required for activation of the isolation system.

These limits seldom govern design but reflect principles of good design. In particular, the third limit is included in the *Provisions* to ensure that isolation system displaces significantly before lateral forces reach the strength of the seismic-force-resisting system.

For designs using the ELF procedure, the lateral forces, F_x, must be distributed to each story over the height of the structure, assuming an inverted triangular pattern of lateral load (*Provisions* Eq.13.3.5 [13.3-9]):

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

Because the lateral displacement of the isolated structure is dominated by isolation system displacement, the actual pattern of lateral force in the isolated mode of response is distributed almost uniformly over height. The *Provisions* require an inverted triangular pattern of lateral load to capture possible higher-mode effects that might be missed by not modeling superstructure flexibility. Rigorous modeling of superstructure flexibility for dynamic analysis would directly incorporate higher-mode effects in the results.

Example plots of the design displacement, D_D, total maximum displacement, D_{TM}, and design forces for the isolation system, V_b, and the superstructure, V_s (R_I = 2), are shown in Figure 11.3-3 as functions of the effective period of the isolation system. The figure also shows the design base shear required for a conventional building, V (R/I = 5). The example plots are for a building assigned to Seismic Design Category D with a one-second spectral acceleration parameter, S_{D1}, equal to 0.6, representing a stiff soil site (Site Class D) located in a region of high seismicity but not close to an active fault. In this example, the isolation system is assumed to have 20 percent effective damping (at all amplitudes of interest) and building geometry is assumed to require 25 percent additional displacement (at corners/edges) due to the requisite 5 percent accidental eccentricity.

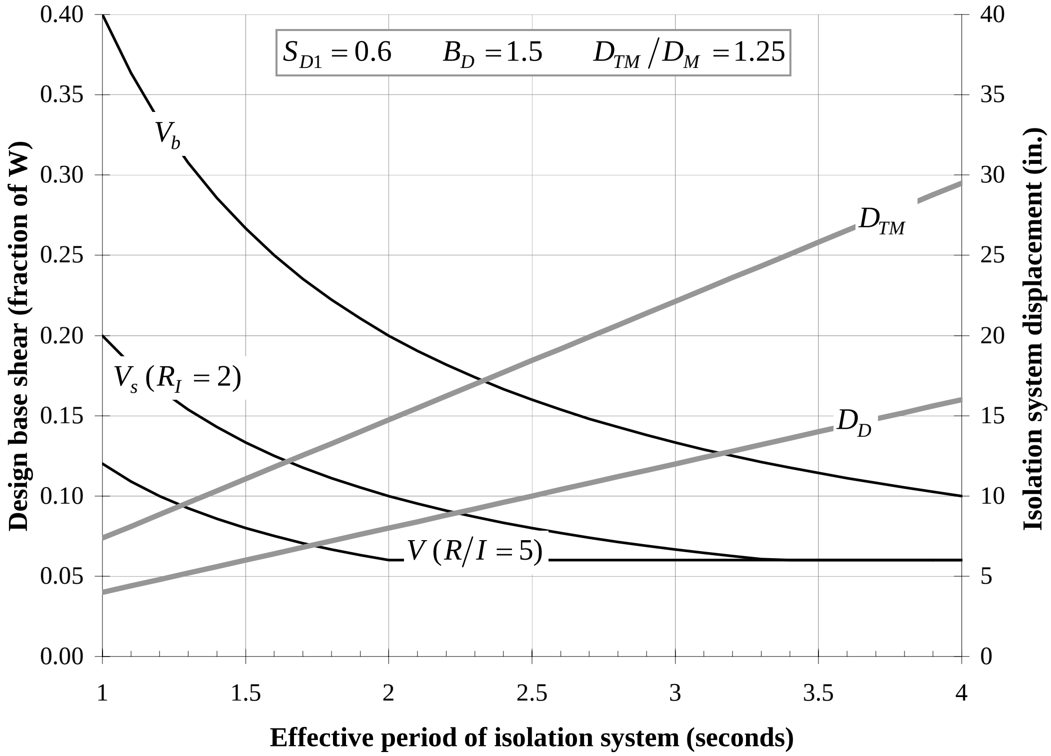


Figure 11.3-3 Isolation system displacement and shear force as function of period (1.0 in. = 25.4 mm).

The plots in Figure 11.3-3 illustrate the fundamental trade off between displacement and force as a function of isolation system displacement. As the period is increased, design forces decrease and design displacements increase linearly. Plots like those shown in Figure 11.3-3 can be constructed during conceptual design once site seismicity and soil conditions are known (or are assumed) to investigate trial values of effective stiffness and damping of the isolation system. In this particular example, an isolation system with an effective period falling between 2.5 and 3.0 seconds would not require more than 22 in. of total maximum displacement capacity (assuming $T_M \leq 3.0$ seconds). Design force on the superstructure would be less than about eight percent of the building weight (assuming $T_D \geq 2.5$ seconds and $R_I \geq 2.0$).

11.4 DYNAMIC LATERAL RESPONSE PROCEDURE

While the ELF procedure equations are useful tools for preliminary design of the isolations system, the *Provisions* requires a dynamic analysis for most isolated structures. Even where not strictly required by the *Provisions*, the use of dynamic analysis (usually time history analysis) to verify the design is common.

11.4.1 Minimum Design Criteria

The *Provisions* encourages the use of dynamic analysis but recognize that along with the benefits of more complex models and analyses also comes an increased chance of design error. To avoid possible under design, the *Provisions* establishes lower-bound limits on results of dynamic analysis used for design. The limits distinguish between response spectrum analysis (a linear, dynamic method) and time history analysis (a nonlinear, dynamic method). In all cases, the lower-bound limit on dynamic analysis is established as a percentage of the corresponding design parameter calculated using the ELF procedure equations. Table 11.4-1 summarizes the percentages that define lower-bound limits on dynamic analysis.

Table 11.4-1 Summary of Minimum Design Criteria for Dynamic Analysis

Design Parameter	Response Spectrum Analysis	Time History Analysis
Total design displacement, D_{TD}	90% D_{TD}	90% D_{TD}
Total maximum displacement, D_{TM}	80% D_{TM}	80% D_{TM}
Design force on isolation system, V_b	90% V_b	90% V_b
Design force on irregular superstructure, V_s	100% V_s	80% V_s
Design force on regular superstructure, V_s	80% V_s	60% V_s

The *Provisions* permits more liberal drift limits when the design of the superstructure is based on dynamic analysis. The ELF procedure drift limits of $0.010h_{sx}$ are increased to $0.015h_{sx}$ for response spectrum analysis and to $0.020h_{sx}$ for time history analysis (where h_{sx} is the story height at level x). Usually a stiff system (e.g., braced frames) is selected for the superstructure and drift demand is typically less than about $0.005h_{sx}$. *Provisions* Sec. 13.4.7.4 [13.4.4] requires an explicit check of superstructure stability at the maximum considered earthquake displacement if the design earthquake story drift ratio exceeds $0.010/R_I$.

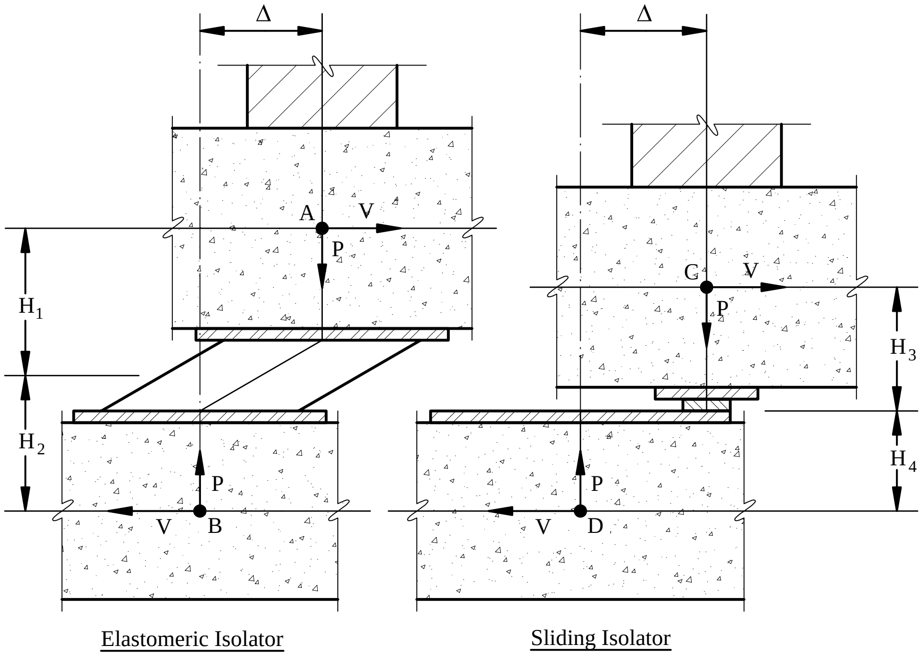
11.4.2 Modeling Requirements

As for the ELF procedure, the *Provisions* requires the isolation system to be modeled for dynamic analysis using stiffness and damping properties that are based on tests of prototype isolator units. Additionally, dynamic analysis models are required to account for:

1. Spatial distribution of individual isolator units,
2. Effects of actual and accidental mass eccentricity,
3. Overturning forces and uplift of individual isolator units, and
4. Variability of isolation system properties (due to rate of loading, etc.).

The *Provisions* requires explicit nonlinear modeling of elements if time history analysis is used to justify design loads less than those permitted for ELF or response spectrum analysis. This option is seldom exercised and the superstructure is typically modeled using linear elements and conventional methods. Special modeling concerns for isolated structures include two important and related issues: the uplift of isolator units, and the P-delta effects on the isolated structure. Isolator units tend to have little or no ability to resist tension forces and can uplift when earthquake overturning (upward) loads exceed factored gravity (downward) loads. Local uplift of individual elements is permitted (*Provisions* Sec. 13.6.2.7 [13.2.5.7]) provided the resulting deflections do not cause overstress or instability. To calculate uplift effects, gap elements may be used in nonlinear models or tension may be released manually in linear models.

The effects of P-delta loads on the isolation system and adjacent elements of the structure can be quite significant. The compression load, P, can be large due to earthquake overturning (and factored gravity loads) at the same time that large displacements occur in the isolation system. Computer analysis programs (most of which are based on small-deflection theory) may not correctly calculate P-delta moments at the isolator level in the structure above or in the foundation below. Figure 11.4-1 illustrates moments due to P-delta effects (and horizontal shear loads) for an elastomeric bearing isolation system and a sliding isolation system. For the elastomeric system, the P-delta moment is split one-half up and one-half down. For the sliding system, the full P-delta moment is applied to the foundation below (due to the orientation of the sliding surface). A reverse (upside down) orientation would apply the full P-delta moment on the structure above.



$$M_A = \frac{P\Delta}{2} + VH_1$$
$$M_B = \frac{P\Delta}{2} + VH_2$$
$$M_C = VH_3$$
$$M_D = P\Delta + VH_4$$

Figure 11.4-1 Moments due to horizontal shear and P-delta effects.

For time history analysis, nonlinear force-deflection characteristics of isolator units are explicitly modeled (rather than using effective stiffness and damping). Force-deflection properties of isolator units are typically approximated by a bilinear, hysteretic curve whose properties can be accommodated by commercially available nonlinear structural analysis programs. Such bilinear hysteretic curves should have approximately the same effective stiffness and damping at amplitudes of interest as the true force-deflection characteristics of isolator units (as determined by prototype testing).

Figure 11.4-2 shows a bilinear idealization of the response of a typical nonlinear isolator unit. Figure 11.4-2 also includes simple equations defining the yield point (D_y, F_y) and end point (D, F) of a bilinear approximation that has the same effective stiffness and damping as the true curve (at a displacement, D).

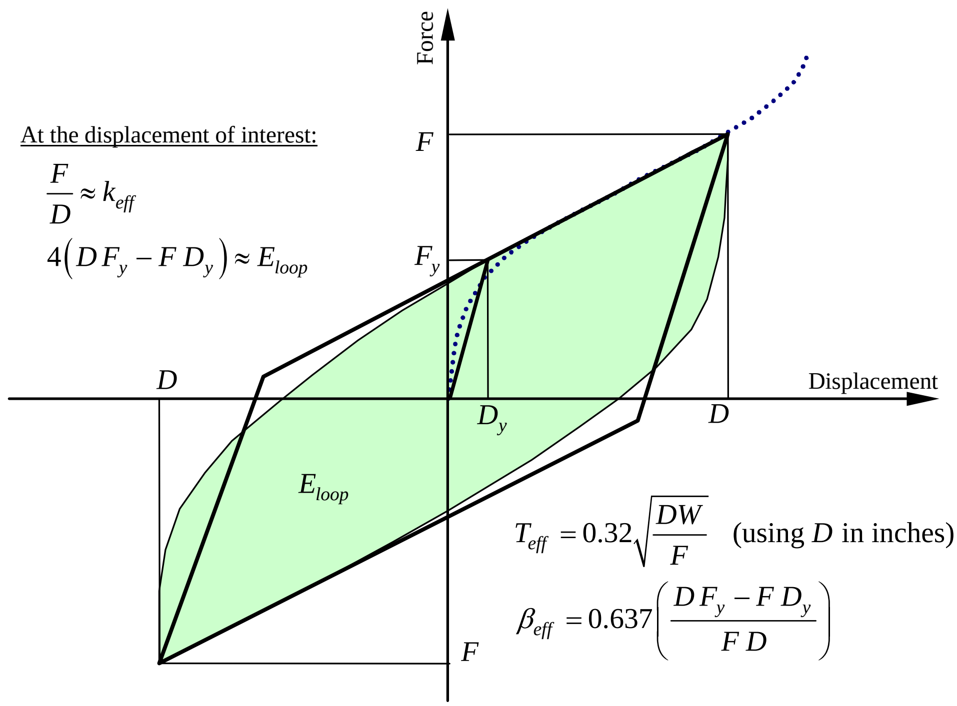


Figure 11.4-2 Bilinear idealization of isolator unit behavior.

11.4.3 Response Spectrum Analysis

Response spectrum analysis requires that isolator units be modeled using amplitude-dependent values of effective stiffness and damping that are the same as those for the ELF procedure. The effective damping of the isolated modes of response is limited to 30 percent of critical. Higher modes of response are usually assumed to have five percent damping—a value of damping appropriate for the superstructure, which remains essentially elastic. As previously noted, maximum and minimum values of effective stiffness are typically used to individually capture maximum displacement of the isolation system and maximum forces in the superstructure. Horizontal loads are applied in the two orthogonal directions, and peak response of the isolation system and other structural elements is determined using the 100 percent plus 30 percent combination method.

11.4.4 Time History Analysis

Time history analysis with explicit modeling of nonlinear isolator units is commonly used for the evaluation of isolated structures. Where at least seven pairs of time history components are employed, the values used in design for each response parameter of interest may be the average of the corresponding analysis maxima. Where fewer pairs are used (with three pairs of time history components being the minimum number permitted), the maximum value of each parameter of interest must be used for design.

The time history method is not a particularly useful design tool due to the complexity of results, the number of analyses required (e.g., to account for different locations of eccentric mass), the need to combine different types of response at each point in time, etc. It should be noted that while *Provisions* Chapter 5 does not require consideration of accidental torsion for either the linear or nonlinear response history procedures, Chapter 13 does require explicit consideration of accidental torsion, regardless of the analysis method employed. Time history analysis is most useful when used to verify a design by checking a few key design parameters, such as: isolation displacement, overturning loads and uplift, and story shear force.

11.5 EMERGENCY OPERATIONS CENTER USING ELASTOMERIC BEARINGS, SAN FRANCISCO, CALIFORNIA

This example features the seismic isolation of a hypothetical emergency operations center (EOC), located in the center of San Francisco, California, an area of very high seismicity. Using high-damping rubber bearings, other types of isolators could be designed to have comparable response properties. Isolation is an appropriate design strategy for EOCs and other buildings where the goal is to limit earthquake damage and protect facility function. The example illustrates the following design topics:

1. Determination of seismic design parameters,
2. Preliminary design of superstructure and isolation systems (using the ELF procedure),
3. Dynamic analysis of seismically isolated structures, and
4. Specification of isolation system design and testing criteria.

While the example includes development of the entire structural system, the primary focus is on the design and analysis of the isolation system. Examples in other chapters may be referred to for more in-depth descriptions of the provisions governing detailed design of the superstructure (i.e., the structure above the isolation system) and the foundation.

11.5.1 System Description

This EOC is a three-story, steel-braced frame structure with a large, centrally located mechanical penthouse. Story heights of 15 ft at all floors accommodate computer access floors and other architectural and mechanical systems. The roof and penthouse roof decks are designed for significant live load to accommodate a helicopter-landing pad and meet other functional requirements of the EOC. Figure 11.5-1 shows the three-dimensional model of the structural system.

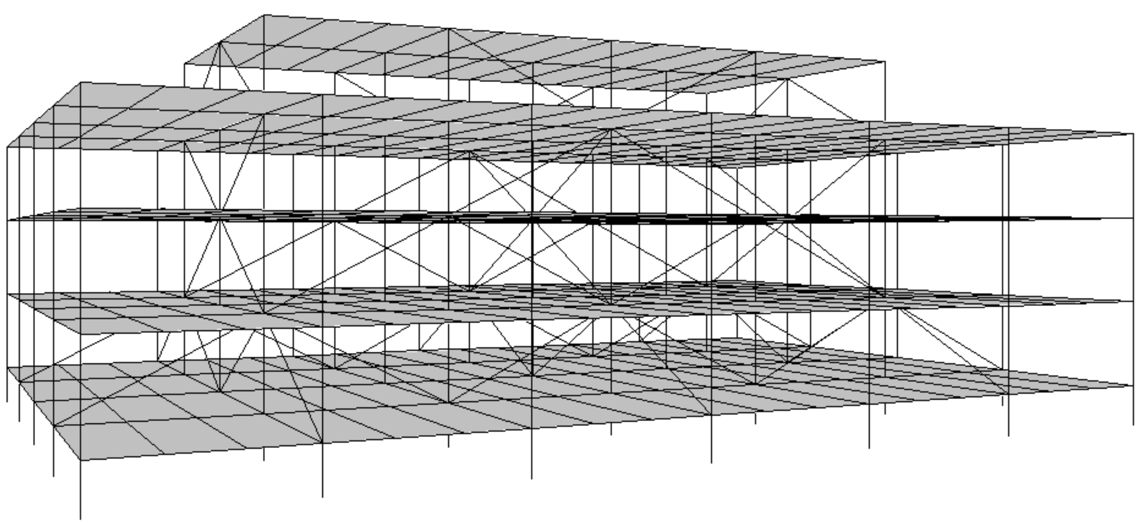


Figure 11.5-1 Three-dimensional model of the structural system.

The structure (which is regular in configuration) has plan dimensions of 120 ft. by 180 ft. at all floors except for the penthouse, which is approximately 60 ft by 120 ft in plan. Columns are spaced at 30 ft in both directions. This EOC's relatively large column spacing (bay size) is used to reduce the number of isolator units for design economy and to increase gravity loads on isolator units for improved earthquake performance. Figures 11.5-2 and 11.5-3, are framing plans for the typical floor levels (1, 2, 3, and roof) and the penthouse roof.

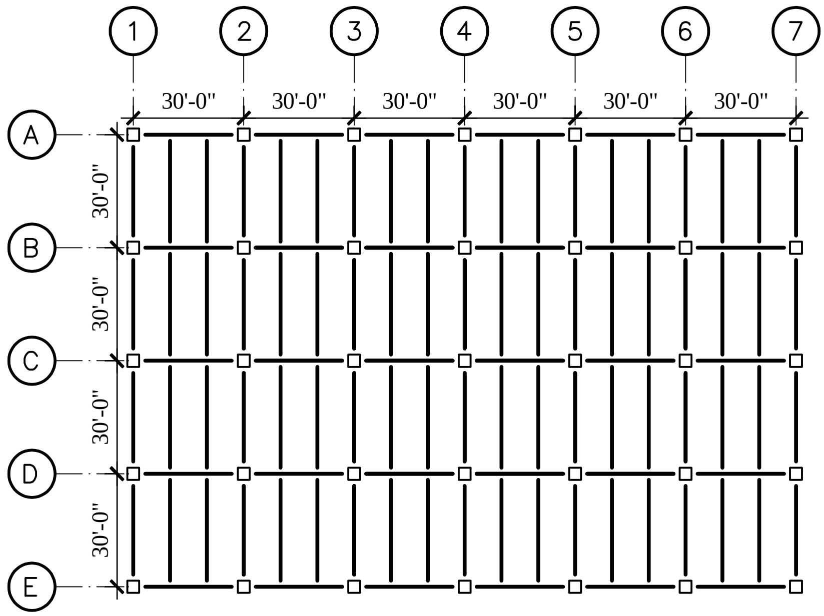


Figure 11.5-2 Typical floor framing plan (1.0 ft = 0.3048 m).

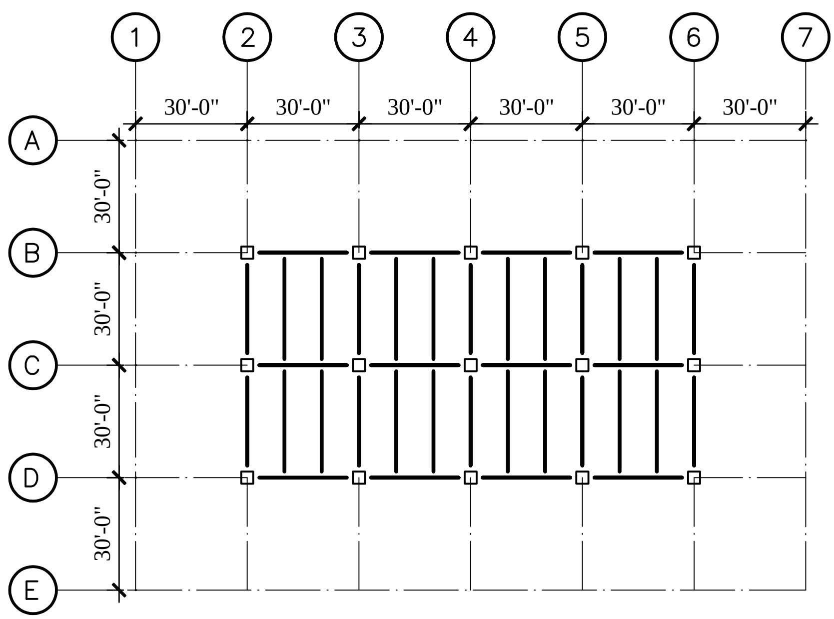


Figure 11.5-3 Penthouse roof framing plan (1.0 ft = 0.3048 m).

The vertical load carrying system consists of concrete fill on steel deck floors, roofs supported by steel beams at 10 ft on center, and steel girders at column lines. Isolator units support the columns below the first floor. The foundation is a heavy mat (although the spread footings or piles could be used depending on the soil type, depth to the water table, and other site conditions).

The lateral system consists of a symmetrical pattern of concentrically braced frames. These frames are located on Column Lines B and D in the longitudinal direction, and on Column Lines 2, 4 and 6 in the transverse direction. Figures 11.5-4 and 11.5-5, respectively, show the longitudinal and transverse elevations. Braces are specifically configured to reduce the concentration of earthquake overturning, and uplift loads on isolator units by:

1. Increasing the continuous length of (number of) braced bays at lower stories,
2. Locating braces at interior (rather than perimeter) column lines (more hold-down weight), and
3. Avoiding common end columns for transverse and longitudinal bays with braces.

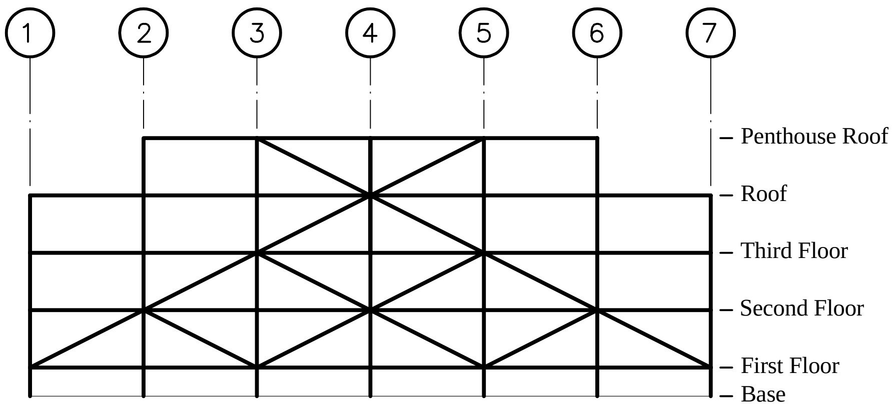


Figure 11.5-4 Longitudinal bracing elevation (Column Lines B and D).

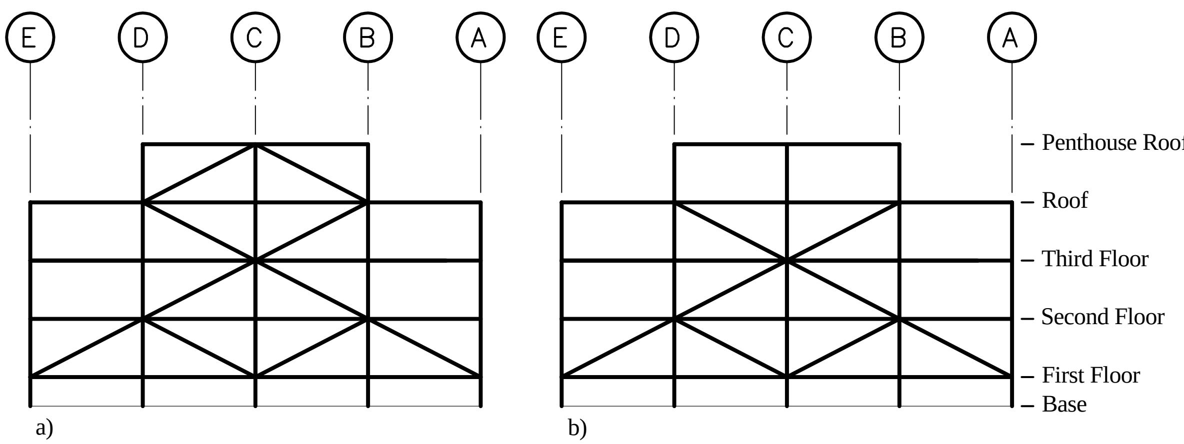


Figure 11.5-5 Transverse bracing elevations: (a) on Column Lines 2 and 6 and (b) on Column Line 4.

The isolation system is composed of 35 identical elastomeric isolator units, located below columns. The first floor is just above grade, and the isolator units are approximately 3 ft below grade to provide clearance below the first floor, for construction and maintenance personnel. A short retaining wall borders the perimeter of the facility and provides 30 in. of "moat" clearance for lateral displacement of the isolated structure. Access to the EOC is provided at the entrances by segments of the first floor slab, which cantilever over the moat.

Girders at the first-floor column lines are much heavier than the girders at other floor levels, and have moment-resisting connections to columns. These girders stabilize the isolator units by resisting moments due to vertical (P-delta effect) and horizontal (shear) loads. Column extensions from the first floor to the top plates of the isolator units are stiffened in both horizontal directions, to resist these moments and to serve as stabilizing haunches for the beam-column moment connections.

11.5.2 Basic Requirements

11.5.2.1 Specifications

General: 1997 *Uniform Building Code* (*UBC*)

Seismic: *NEHRP Recommended Provisions*

11.5.2.2 Materials

Concrete:

Strength	$f_c' = 3$ ksi
Weight (normal)	$\gamma = 150$ pcf

Steel:

Columns	$F_y = 50$ ksi
Primary first-floor girders (at column lines)	$F_y = 50$ ksi
Other girders and floor beams	$F_y = 36$ ksi
Braces	$F_y = 46$ ksi

Steel deck: 3-in.-deep, 20-gauge deck

Seismic isolator units (high-damping rubber):

Maximum long-term-load ($1.2D + 1.6L$) face pressure, σ_{LT}	1,400 psi
Maximum short-term-load ($1.5D + 1.0L + Q_{\mathrm{MCE}}$) face pressure, σ_{ST}	2,800 psi
Minimum bearing diameter (excluding protective cover)	$1.25D_{TM}$
Minimum rubber shear strain capacity (isolator unit), γ_{max}	300 percent
Minimum effective horizontal shear modulus, G_{min}	65 - 110 psi
Third cycle at γ = 150 percent (after scragging/recovery)	
Maximum effective horizontal shear modulus, G_{max}	$1.3 \times G_{min}$
First cycle at γ = 150 percent (after scragging/recovery)	
Minimum effective damping at 150 percent rubber shear strain, β_{eff}	15 percent

11.5.2.3 Gravity Loads

Dead loads:

Main structural elements (slab, deck, and framing)	self weight

Miscellaneous structural elements (and slab allowance)	10 psf
Architectural facades (all exterior walls)	750 plf
Roof parapets	150 plf
Partitions (all enclosed areas)	20 psf
Suspended MEP/ceiling systems and supported flooring	15 psf
Mechanical equipment (penthouse floor)	50 psf
Roofing	10 psf

Reducible live loads:

Floors (1-3)	100 psf
Roof decks and penthouse floor	50 psf

Live load reduction:

The 1997 *UBC* permits area-based live load reduction, of not more than 40 percent for elements with live loads from a single story (e.g., girders), and not more than 60 percent for elements with live loads from multiple stories (e.g., axial component of live load on columns at lower levels and isolator units).

EOC weight (dead load) and live load:

Penthouse roof	W_{PR}	= 965 kips
Roof (penthouse)	W_R	= 3,500 kips
Third floor	W_3	= 3,400 kips
Second floor	W_2	= 3,425 kips
First floor	W_1	= 3,425 kips
Total EOC weight	W	= 14,715 kips

(See Chapter 1 for a discussion of live load contributions to the seismic weight.)

Live load (L) above isolation system	L	= 7,954 kips
Reduced live load (RL) above isolation system	RL	= 3,977 kips

Table 11.5-1 Gravity Loads on Isolator Units*

Column line	Dead/live loads (kips)			
	1	2	3	4
A	182/73	349/175	303/153	345/173
B	336/166	570/329	606/346	539/309
C	295/149	520/307	621/356	639/358

1.0 kip = 4.45 kN.
* Loads at Column Lines 5, 6 and 7 (not shown) are the same as those at Column Lines 3, 2, and 1, respectively; loads at Column Lines D and E (not shown) are the same as those at Column Lines B and A, respectively.

11.5.2.4 *Provisions* Parameters

11.5.2.4.1 Performance Criteria (Provisions Sec. 1.3 [1.2 and 1.3])

Designated Emergency Operation Center	Seismic Use Group III
Occupancy Importance Factor	I = 1.5 (conventional)
Occupancy Importance Factor (*Provisions* Chapter 13)	I = 1.0 (isolated)

> Chapter 13 does not require use of the occupancy importance factor to determine the design loads on the structural system of an isolated building (that is, I = 1.0), but the component importance factor is required by Chapter 6, to determine seismic forces on components (I_p = 1.5 for some facilities).

11.5.2.4.2 Ground Motion (Provisions Chapter 4 [3])

Site soil type (assumed)	Site Class D
Maximum considered earthquake (MCE) spectral acceleration at short periods (*Provisions* Map 7)	S_S = 1.50

[The 2003 Provisions have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package.]

Site coefficient (*Provisions* Table 4.1.2.4b [3.3-1])	F_a = 1.0
MCE spectral acceleration adjusted for site class (F_aS_S)	S_{MS} = 1.50
Design earthquake (DE) spectral acceleration (2/3S_{MS})	S_{DS} = 1.0
MCE spectral acceleration at a period of 1 second (*Provisions* Map 8)	S_1 = 0.9

> On Map 8, the contour line of S_1 = 0.9 runs approximately through the center of the San Francisco peninsula, with other San Francisco peninsula contour lines ranging from 0.6 (greatest distance from San Andreas Fault), to about 1.6 (directly on top of the San Andreas Fault).

Site coefficient (*Provisions* Table 4.1.2.4b [3.3-2])	F_v = 1.5
MCE spectral acceleration adjusted for site class (F_vS_1)	S_{M1} = 1.35
Design earthquake (DE) spectral acceleration (2/3S_{M1})	S_{D1} = 0.9
Seismic Design Category (*Provisions* Table 4.2.1b [1.4-2])	Seismic Design Category F

[In the 2003 edition of the *Provisions*, the ground motion trigger for Seismic Design Categories E and F have been changed to $S_1 \geq 0.60$. No change would result for this example.]

11.5.2.4.3 Design Spectra (Provisions Sec. 13.4.4.1)

Figure 11.5-6 plots design earthquake and maximum considered earthquake response spectra as constructed in accordance with the procedure of *Provisions* Sec. 13.4.4.1 [3.4] using the spectrum shape defined by *Provisions* Figure 4.1.2.6 [3.3-15]. [In the 2003 edition of *Provisions*, the shape of the design spectrum changes at period beyond T_L, but no change would result for this example.] *Provisions* Sec. 13.4.4.1[13.2.3.1] requires site-specific design spectra to be calculated for sites with S_1 greater than 0.6 (e.g., sites near active sources). Site-specific design spectra may be taken as less than 100 percent but not less than 80 percent of the default design spectra of *Provisions* Figure 4.1.2.6 [3.3-15].

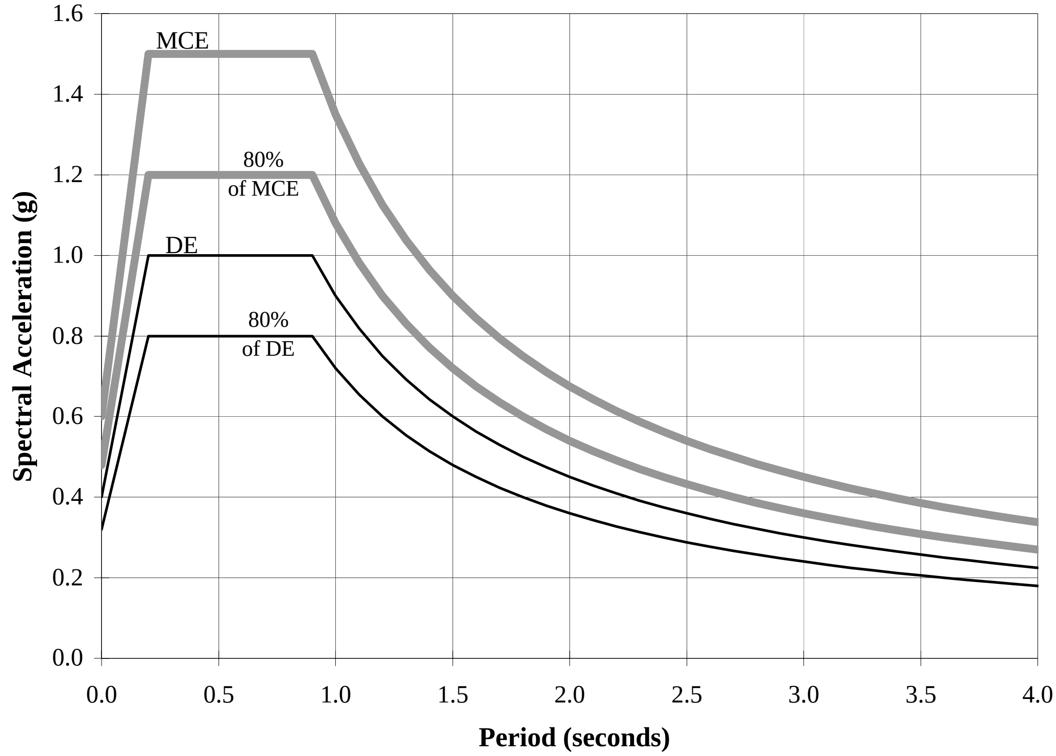


Figure 11.5-6 Example design spectra (5 percent damping).

For this example, site-specific spectra for the design earthquake and the maximum considered earthquake, were assumed to be 80 percent (at long periods) and 100 percent (at short periods) of the respective spectra shown in Figure 11.5-6. The 80 percent factor reflects a reduction in demand that could be achieved through a detailed geotechnical investigation of site soil conditions. In general, site-specific spectra for regions of high seismicity, with well defined fault systems (like those of the San Francisco Bay Area), would be expected to be similar to the default design spectra of *Provisions* Figure 4.1.2.6 [3.3-15].

11.5.2.4.4 Design Time Histories (Provisions Sec. 13.4.4.2 [13.2.3.2])

For time history analysis, *Provisions* Sec. 13.4.4.7 [13.4.2.3] requires no less than three pairs of horizontal ground motion time history components to be selected from actual earthquake records, and scaled to match either the design earthquake (DE) or the maximum considered earthquake (MCE) spectrum. The selection and scaling of appropriate time histories is usually done by an earth scientist, or a geotechnical engineer experienced in the seismology of the region; and takes the earthquake magnitudes, fault distances, and source mechanisms that influence hazards at the building site into consideration.

For this example, records from the El Centro Array Station No. 6, the Newhall Fire Station, and the Sylmar Hospital are selected from those recommended by *ATC-40* [ATC, 1996], for analysis of buildings at stiff soil sites with ground shaking of 0.2g or greater. These records, and the scaling factors required to match the design spectra are summarized in Table 11.5-2. These three records represent strong ground shaking recorded relatively close to fault rupture; and contain long-period pulses in the direction of strongest shaking, which can cause large displacement of the isolated structures. MCE scaling factors for the El Centro No. 6 and Sylmar records are 1.0, indicating that these records are used without modification for MCE time history analysis of the EOC.

Provisions Sec. 13.4.4.2 [13.2.3.2] provides criteria for scaling earthquake records to match a target spectrum over the period range of interest, defined as $0.5T_D$ to $1.25T_M$. In this example, T_D and T_M are both assumed to be 2.5 seconds, so the period range of interest is from 1.25 seconds to 3.125 seconds. For each period in this range, the average of the square-root-of-the-sum-of-the-squares (SRSS) combination of each pair of horizontal components of scaled ground motion, may not fall below the target spectrum by more 10 percent. The target spectrum is defined as 1.3 times the design spectrum of interest (either the DE or the MCE spectrum).

Figure 11.5-7 compares the spectra of scaled time history components with the spectrum for the design earthquake. Rather than comparing the average of the SRSS spectra with 1.3 times the design spectrum, as indicated in the *Provisions*, Figure 11.5-7 shows the (unmodified) design spectrum and the average of the SRSS spectra divided by 1.3. The effect is the same, but the method employed eliminates one level of obscurity and permits direct comparison of the calculated spectra without additional manipulation. A comparison of maximum, considered earthquake spectra would look similar (with values that were 1.5 times larger). The fit is very good at long periods - the period range of interest for isolated structures. At 2.5 seconds, the spectrum of the scaled time histories has the same value as the target spectrum. Between 1.25 seconds and 3.125 seconds, the spectrum of the scaled time histories is never less than the target spectrum by more than 10 percent. At short periods, the spectrum of scaled time history motions are somewhat greater than the target spectrum - a common result of scaling real time histories to match the long-period portion of the design spectrum.

Figure 11.5-7 also includes plots of upper-bound (maximum demand) and lower-bound (minimum demand) envelopes of the spectra of the six individual components of scaled time histories. The maximum demand envelope illustrates that the strongest direction (component) of shaking of at least one of the three scaled time histories is typically about twice the site-specific spectrum in the period range of the isolated structure - consistent with strong ground shaking recorded near sources in the fault normal direction.

Table 11.5-2 Earthquake Time History Records and Scaling Factors

Record No.	Earthquake Source and Recording Station			Scaling factor	
	Year	Earthquake	Station (owner)	DE	MCE
1	1979	Imperial Valley, CA	El Centro Array Station 6 (USGS)	0.67	1.0
2	1994	Northridge, CA	Newhall Fire Station (CDMG)	1.0	1.5
3	1994	Northridge, CA	Sylmar Hospital (CDMG)	0.67	1.0

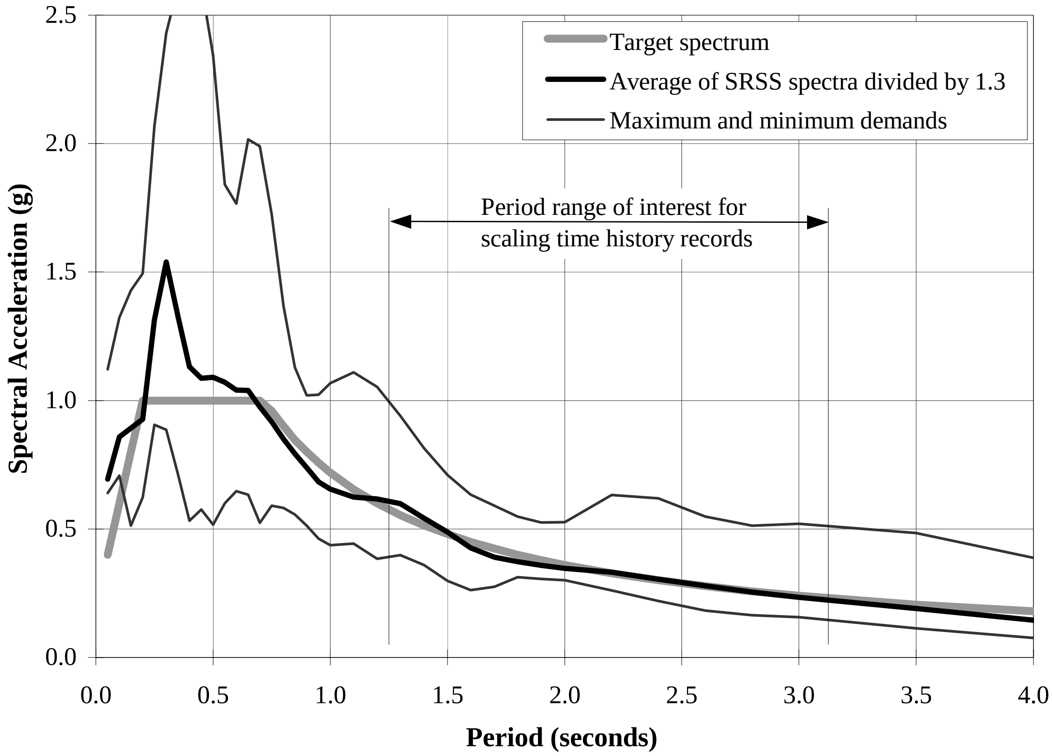


Figure 11.5-7 Comparison of design earthquake spectra.

11.5.2.5 Structural Design Criteria

11.5.2.5.1 Design Basis (Provisions Sec. 5.2 and 13.2 [4.3 and 13.2])

Seismic-force-resisting system.	Special steel concentrically braced frames (height < 100 ft)
Response modification factor, R (*Provisions* Table 5.2.2 [4.3-1]	6 (conventional)
Response modification factor for design of the superstructure, R_I (*Provisions* Sec. 13.3.4.2 [13.3.3.2], $3/8R \leq 2$)	2 (isolated)
Plan irregularity (of superstructure) - (*Provisions* Table 5.2.3.2)	None
Vertical irregularity (of superstructure) - from Table 5.2.3.3	None
Lateral response procedure (Provisions Sec. 13.2.5.2 [13.2.4.1], $S_1 > 0.6$)	Dynamic analysis
Redundancy/reliability factor - (*Provisions* Sec. 5.2.4.2 [4.3.3])	$\rho > 1.0$ (conventional) $\rho = 1.0$ (isolated)

Provisions Sec. 5.2.4.2 [4.3.3] requires the use of a calculated ρ value, which would be greater than 1.0 for a conventional structure with a brace configuration similar to the superstructure of the base-isolated EOC. However, in the author's opinion, the use of R_I equal to 2.0 (rather than R equal to 6) as required by *Provisions* Sec. 13.3.4.2 precludes the need to further increase superstructure design forces for redundancy/reliability. Future editions of the *Provisions* should address this issue.

[The redundancy factor is changed substantially in the 2003 Provisions. However, the rationale set forth above by the author still holds; use of $\rho = 1.0$ is reasonable since $R_I = 2.0$.]

11.5.2.5.2 Horizontal Earthquake Loads and Effects (Provisions Chapters 5 and 13)

Design earthquake (acting in either the X or Y direction)	DE (site specific)
Maximum considered earthquake (acting in either the X or Y direction)	MCE (site specific)
Mass eccentricity - actual plus accidental	$0.05b$ = 6 ft ($\perp$ to X axis) $0.05d$ = 9 ft ($\perp$ to Y axis)

The superstructure is symmetric about both principal axes, however, the placement of the braced frames results in a ratio of maximum corner displacement to average displacement of 1.25, exceeding the threshold of 1.2 per the definition of the *Provisions*. If the building were not on isolators, the accidental torsional moment would need to be increased from 5 percent to 5.4 percent of the building dimension. The input to the superstructure is controlled by the isolation system, and it is the author's opinion that the amplification of accidental torsion is not necessary for such otherwise regular structures. Future editions of the *Provisions* should address this issue. Also refer to the discussion of analytical modeling of accidental eccentricities in *Guide* Chapter 1.

Superstructure design (reduced DE response)	$Q_E = Q_{\mathrm{DE/2}}$ = DE/2.0
Isolation system and foundation design (unreduced DE response)	$Q_E = Q_{\mathrm{DE}}$ = DE/1.0
Check of isolation system stability (unreduced MCE response)	$Q_E = Q_{\mathrm{MCE}}$ = MCE/1.0

11.5.2.5.3 Combination of Horizontal Earthquake Load Effects (Provisions Sec. 5.25.2.3 [4.4.2.3] and 13.4.6.3 [13.4.2.2])

Response due earthquake loading in the X and Y directions	Q_E = Max $(1.0Q_{EX}+0.3Q_{EY},$ $0.3Q_{EX}+1.0Q_{EY})$

In general, the horizontal earthquake load effect, Q_E, on the response parameter of interest is influenced by only one direction of horizontal earthquake load, and $Q_E = Q_{EX}$ or $Q_E = Q_{EY}$. Exceptions include vertical load on isolator units due to earthquake overturning forces.

11.5.2.5.4 Combination of Horizontal and Vertical Earthquake Load Effects (Provisions Sec. 5.2.7 [4.2.2.1])

Design earthquake ($\rho Q_E \pm 0.2S_{DS}D$)	$E = Q_E \pm 0.2D$
Maximum considered earthquake ($\rho Q_E \pm 0.2S_{MS}D$)	$E = Q_E \pm 0.3D$

11.5.2.5.5 Superstructure Design Load Combinations (UBC Sec. 1612.2, using R_I = 2)

Gravity loads (dead load and reduced live load)	$1.2D + 1.6L$
Gravity and earthquake loads ($1.2D + 0.5L + 1.0E$)	$1.4D + 0.5L + Q_{\mathrm{DE/2}}$

Gravity and earthquake loads (0.9D - 1.0E)	0.7D - $Q_{DE/2}$

11.5.2.5.6 Isolation System and Foundation Design Load Combinations (UBC Sec. 1612.2)

Gravity loads (for example, long term load on isolator units)	1.2D + 1.6L
Gravity and earthquake loads (1.2D + 0.5L + 1.0E)	1.4D + 0.5L + Q_{DE}
Gravity and earthquake loads (0.9D - 1.0E)	0.7D - Q_{DE}

11.5.2.5.7 Isolation System Stability Load Combinations (Provisions Sec. 13.6.2.6 [13.2.5.6])

Maximum short term load on isolator units (1.2D + 1.0L + |E|)	1.5D + 1.0L + Q_{MCE}
Minimum short term load on isolator units (0.8D - |E|)	0.8D - Q_{MCE}

Note that in the above combinations, the vertical earthquake load (0.2$S_{MS}D$) component of |E| is included in the maximum (downward) load combination, but excluded from the minimum (uplift) load combination. It is the author's opinion that vertical earthquake ground shaking is of a dynamic nature, changing direction too rapidly to affect appreciably uplift of isolator units and need not be used with the load combinations of *Provisions* Sec. 13.6.2.6 [13.2.5.6] for determining minimum (uplift) vertical loads on isolator units due to the MCE. Future editions of the *Provisions* should explicitly address this issue.

11.5.3 Seismic Force Analysis

11.5.3.1 Basic Approach to Modeling

To expedite calculation of loads on isolator units, and other elements of the seismic-force-resisting system, a 3-dimensional model of the EOC is developed and analyzed using the ETABS computer program (CSI, 1999). While there are a number of programs to choose from, ETABS (version 7.0) is selected for this example since it permits the automated release of tension in isolator units subject to uplift and has built-in elements for modeling other nonlinear properties of isolator units. Arguably, all of the analyses performed by the ETABS program could be done by hand, or by spreadsheet calculation (except for confirmatory time history analyses).

The ETABS model is used to perform the following types of analyses and calculations:

1. Dead load weight and live loads for the building--calculate maximum long-term load on isolator units (*Guide* Table 11.5-1)

2. ELF procedure for gravity and reduced design earthquake loads--design superstructure (ignoring uplift of isolator units)

3. Nonlinear static analysis with ELF loads for gravity and unreduced design earthquake loads--design isolation system and foundation (considering uplift of isolator units)

4. Nonlinear static analysis with ELF loads - gravity and unreduced design earthquake loads--calculate maximum short-term load (downward force) on isolator units (*Guide* Table 11.5-5) and calculate minimum short-term load (downward force) of isolator units (*Guide* Table 11.5-6)

5. Nonlinear static analysis with ELF loads for gravity and unreduced MCE loads--calculate maximum short-term load (downward force) on isolator units (*Guide* Table 11.5-7) and calculate minimum short-term load (uplift displacement) of isolator units (*Guide* Table 11.5-8)

6. Nonlinear time history analysis - gravity and scaled DE or MCE time histories--verify DE displacement of isolator units and design story shear (*Guide* Table 11.5-10), verify MCE displacement of isolator units (*Guide* Table 11.5-11), verify MCE short-term load (downward force) on isolator units (*Guide* Table 11.5-12), and verify MCE short-term load (uplift displacement) of isolator units (*Guide* Table 11.5-13).

The *Provisions* requires a response spectrum analysis for the EOC (see *Guide* Table 11.2-1). In general, the response spectrum method of dynamic analysis is considered sufficient for facilities that are located at a stiff soil site, which have an isolation system meeting the criteria of *Provisions* Sec. 13.2.5.2.7, Item 7 [13.2.4.1, Item 7]. However, nonlinear static analysis is used for the design of the EOC, in lieu of response spectrum analysis, to permit explicit modeling of uplift of isolator units. For similar reasons, nonlinear time history analysis is used to verify design parameters.

11.5.3.2 Detailed Modeling Considerations

Although a complete description of the ETABS model is not possible, key assumptions and methods used to model elements of the isolation system and superstructure are described below.

11.5.3.2.1 Mass Eccentricity

Provisions Sec. 13.4.5.2 [13.4.1.1] requires consideration of mass eccentricity. Because the building in the example is doubly symmetric, there is no actual eccentricity of building mass (but such would be modeled if the building were not symmetric). Modeling of accidental mass eccentricity would require several analyses, each with the building mass located at different eccentric locations (for example, four quadrant locations in plan). This is problematic, particularly for dynamic analysis using multiple time history inputs. In this example, only a single (actual) location of mass eccentricity is considered, and calculated demands are increased moderately for the design of the seismic-force-resisting system, and isolation system (for example, peak displacements calculated by dynamic analysis are increased by 10 percent for design of the isolation system).

11.5.3.2.2 P-delta Effects

P-delta moments in the foundation and the first floor girders just above isolator units due to the large lateral displacement of the superstructure are explicitly modeled. The model distributes half of the P-delta moment to the structure above, and half of the *P*-delta moment to the foundation below the isolator units. ETABS (version 7.0) permits explicit modeling of the P-delta moment, but most computer programs (including older versions of ETABS) do not. Therefore, the designer must separately calculate these moments and add them to other forces for the design of affected elements. The P-delta moments are quite significant, particularly at isolator units that resist large earthquake overturning loads along lines of lateral bracing.

11.5.3.2.5 Isolator Unit Uplift

Provisions Sec. 13.6.2.7 [13.2.5.7] permits local uplift of isolator units provided the resulting deflections do not cause overstress of isolator units or other structural elements. Uplift of some isolator units is likely (for unreduced earthquake loads) due to the high seismic demand associated with the site. Accordingly, isolator units are modeled with gap elements that permit uplift when the tension load exceeds the tensile capacity of an isolator unit. Although most elastomeric bearings can resist some tension stress before yielding (typically about 150 psi), isolator units are assumed to yield as soon as they are loaded in tension, producing larger estimates of uplift displacement and overturning loads on isolator units that are in compression.

The assumption that isolator units have no tension capacity is not conservative, however, for design of the connections of isolator units to the structure above and the foundation below. The design of anchor bolts and other connection elements, must include the effects of tension in isolator units (typically based on a maximum stress of 150 psi).

11.5.3.2.4 Bounding Values of Bilinear Stiffness of Isolator Units

The design of elements of the seismic-force-resisting system is usually based on a linear, elastic model of the superstructure. When such models are used, *Provisions* Sec. 13.4.5.3.2 [13.4.1.2] requires that the stiffness properties of nonlinear isolation system components be based on the maximum effective stiffness of the isolation system (since this assumption produces larger earthquake forces in the superstructure). In contrast, the *Provisions* require that calculation of isolation system displacements be based on the minimum effective stiffness of the isolation system (since this assumption produces larger isolation system displacements).

The concept of bounding values, as discussed above, applies to all of the available analysis methods. For the ELF procedure, the *Provisions* equations are based directly on maximum effective stiffness, k_{Dmax} and k_{Mmax}, are used for calculating design forces; and on minimum effective stiffness, k_{Dmin} and k_{Dmax}, are used for calculating design displacements. Where (nonlinear) time history analysis is used, isolators are explicitly modeled as bilinear hysteretic elements with upper or lower-bound stiffness curves. Upper-bound stiffness curves are used to verify the forces used for the design of the superstructure and lower-bound stiffness curves are used to verify design displacements of the isolation system.

11.5.4 Preliminary Design Based on the ELF Procedure

11.5.4.1 Calculation of Design Values

11.5.4.1.1 Design Displacements

Preliminary design begins with the engineer's selection of the effective period (and damping) of the isolated structure, and the calculation of the design displacement, D_D. In this example, the effective period of the EOC facility (at the design earthquake displacement) is T_D = 2.5 seconds; and the design displacement is calculated as follows, using *Provisions* Eq. 13.3.3.1:

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D} = (9.8)\frac{0.9(2.5)}{1.35} = 16.3 \text{ in.}$$

The 1.35 value of the damping coefficient, B_D, is given in *Provisions* Table 13.3.3.1 [13.3-1] assuming 15 percent effective damping at 16.3 in. of isolation system displacement. Effective periods of 2 to 3 seconds and effective damping values of 10 to 15 percent are typical of high-damping rubber (and other types of) bearings.

Stability of the isolation system must be checked for the maximum displacement, D_M, which is calculated using *Provisions* Eq. 13.3.3.3 as follows:

$$D_M = \left(\frac{g}{4\pi^2}\right)\frac{S_{M1}T_M}{B_M} = (9.8)\frac{1.35(2.5)}{1.35} = 24.5 \text{ in.}$$

For preliminary design, the effective period and effective damping at maximum displacement are assumed to be the same as the values at the design displacement. While both the effective period and damping values may reduce slightly at larger rubber strains, the ratio of the two parameters tend to be relatively consistent.

The total displacement of specific isolator units (considering the effects of torsion) is calculated based on the plan dimensions of the building, the total torsion (due to actual, plus accidental eccentricity), and the distance from the center of resistance of the building to the isolator unit of interest. Using *Provisions* Eq. 13.3.3.5-1 and 13.3.3.5-2 [13.3-5 and 13.3-6], the total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of isolator units located on Column Lines 1 and 7 are calculated for the critical (transverse) direction of earthquake load as follows:

$$D_{TD} = D_D\left[1+y\left(\frac{12e}{b^2+d^2}\right)\right] = 16.3\left[1+90\left(\frac{12(0.05)(180)}{120^2+180^2}\right)\right] = 16.3(1.21) = 19.7\text{ in.}$$

$$D_{TM} = D_M\left[1+y\left(\frac{12e}{b^2+d^2}\right)\right] = 24.5\left[1+90\left(\frac{12(0.05)(180)}{120^2+180^2}\right)\right] = 24.5(1.21) = 29.6\text{ in.}$$

The equations above assume that mass is distributed in plan in proportion to isolation system stiffness and shifted by 5percent, providing no special resistance to rotation of the building on the isolation system. In fact, building mass is considerably greater toward the center of the building, as shown by the schedule of gravity loads in Table 11.5-1. The stiffness of the isolation system is uniform in plan (since all isolators are of the same size) providing significant resistance to dynamic earthquake rotation of the building. While the *Provisions* permit a reduction in the total displacements calculated using the ELF procedure (with proper substantiation of resistance to torsion), in this example the 21 percent increase is considered to be conservative for use in preliminary design and for establishing lower-bound limits on dynamic analysis results.

11.5.4.1.2 Minimum and Maximum Effective Stiffness

Provisions Eq. 13.3.3.2 [13.3-2] expresses the effective period at the design displacement in terms of building weight (dead load) and the minimum effective stiffness of the isolation system, k_{Dmin}. Rearranging terms and solving for minimum effective stiffness:

$$k_{Dmin} = \left(\frac{4\pi^2}{g}\right)\frac{W}{T_D^2} = \left(\frac{1}{9.8}\right)\frac{14{,}715}{2.5^2} = 240\text{ kips/in.}$$

This stiffness is about 6.9 kips/in. for each of 35 identical isolator units. The effective stiffness can vary substantially from one isolator unit to another and from one cycle of prototype test to another. Typically, an isolator unit’s effective stiffness is defined by a range of values for judging acceptability of prototype (and production) bearings. The minimum value of the stiffness range, k_{Dmin}, is used to calculate isolation system design displacements; the maximum value of the stiffness, k_{Dmax}, is used to define design forces.

The variation in effective stiffness depends on the specific type of isolator, elastomeric compound, loading history, etc., but must, in all cases, be broad enough to comply with the *Provisions* Sec. 13.9.5.1 [13.6.4.1]requirements that define maximum and minimum values of effective stiffness based on testing of isolator unit prototypes. Over the three required cycles of test at D_D, the maximum value of effective stiffness (for example, at the first cycle) should not be more than about 30 percent greater than the minimum value of effective stiffness (for example, at the third cycle) to comply with *Provisions* Sec.

13.9.4 [13.6.3]. On this basis, the maximum effective stiffness, k_{Dmax}, of the isolation system in this example, is limited to 312 kips/in. (that is, 1.3 × 240 kips/in.).

The range of effective stiffness defined by k_{Dmin} and k_{Dmax} (as based on cyclic tests of prototype isolator units) does not necessarily bound all the possible variations in the effective stiffness of elastomeric bearings. Other possible sources of variation include: stiffness reduction, due to post-fabrication "scragging" of bearings (by the manufacturer), and partial recovery of this stiffness over time. Temperature and aging effects of the rubber material, and other changes in properties that can also occur over the design life of isolator units. (Elastomeric bearings are typically “scragged” immediately following molding and curing to loosen up the rubber molecules by application of vertical load.) *Provisions* Sec. 13.6.2.1 [13.2.5.1] requires that such variations in isolator unit properties be considered in design, but does not provide specific criteria. A report - *Property Modification Factors for Seismic Isolation Bearings* (Constantinou, 1999) - provides guidance for establishing a range of effective stiffness (and effective damping) properties that captures all sources of variation over the design life of the isolator units. The full range of effective stiffness has a corresponding range of effective periods (with different levels of spectral demand). The longest effective period (corresponding to the minimum effective stiffness) of the range would be used to define isolation system design displacements; and the shortest effective period (corresponding to the maximum effective stiffness) of the range would be used to define the design forces on the superstructure.

11.5.4.1.3 Lateral Design Forces

The lateral force required for the design of the isolation system, foundation, and other structural elements below the isolation system, is given by *Provisions* Eq. 13.3.4.1 [13.3-7]:

$$V_b = k_{Dmax}\, D_D = 312(16.3) = 5{,}100 \text{ kips}$$

The lateral force required for checking stability and ultimate capacity of elements of the isolation system, may be calculated as follows:

$$V_{MCE} = k_{Dmax}\, D_M = 312(24.5) = 7{,}650 \text{ kips}$$

The (unreduced) base shear of the design earthquake is about 35 percent of the weight of the EOC, and the (unreduced) base shear of the MCE is just over 50 percent of the weight. In order to design the structure above the isolation system, the design earthquake base shear is reduced by the R_I factor in *Provisions* Eq. 13.3.4.2 [13.3-8]:

$$V_s = \frac{k_{Dmax} D_D}{R_1} = \frac{312(16.3)}{2.0} = 2{,}550 \text{ kips}$$

This force is about 17 percent of the dead load weight of the EOC, which is somewhat less than, but comparable to, the force that would be required for the design of a conventional, fixed-base building of the same size and height, seismic-force-resisting system, and site seismic conditions. Story shear forces on the superstructure are distributed vertically over the height of the structure in accordance with *Provisions* Eq. 13.3.5 [13.3-9], as shown in Table 11.5-3.

Table 11.5-3 Vertical Distribution of Reduced Design Earthquake Forces (DE/2)

Story level, x	Weight, w_x (kips)	Height above isolation system, h_x (ft)	Force, $F_x = \frac{w_x h_x V_s}{\sum_i w_i h_i}$ (kips)	Cumulative force (kips)	Force divided by weight, $\frac{F_x}{w_x}$
Penthouse roof	965	64	370	370	0.38
Roof	3,500	49	1,020	1,390	0.29
Third floor	3,400	34	690	2,080	0.20
Second floor	3,425	19	390	2,470	0.11
First floor	3,425	4	80	2,550	0.023

1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN.

Provisions Eq. 13.3.5 distributes lateral seismic design forces (DE/2) over the height of the building in an inverted, triangular, pattern as indicated by the ratio of F_x/w_x, shown in Table 11.5-3. Because the superstructure is much stiffer laterally, than the isolation system, it tends to move as a rigid body in the first mode with a pattern of lateral seismic forces that is more uniformly distributed over the height of the building. The use of a triangular load pattern for design is intended to account for higher-mode response that may be excited due to flexibility of the superstructure. *Provisions* Eq. 13.3.5 [13.3-9] is also used to distribute forces over the height of the building for unreduced DE and MCE forces, as summarized in Table 11.5-4.

Table 11.5-4 Vertical Distribution of Unreduced DE and MCE Forces

Story level, x	Design earthquake (DE)			Maximum considered earthquake (MCE)		
	Force (kips)	Cumulative force (kips)	Force divided by weight	Force (kips)	Cumulative force (kips)	Force divided by weight
Penthouse roof	740	740	0.76	1,110	1,110	1.14
Roof	2,040	2,780	0.59	3,060	4,170	0.88
Third floor	1,380	4,160	0.41	2,070	6,240	0.61
Second floor	780	4,940	0.23	1,170	7,410	0.34
First floor	160	5,100	0.047	240	7,650	0.071

1.0 kip = 4.45 kN.

11.5.4.1.4 Design Earthquake Forces for Isolator Units

Tables 11.5-5 and 11.5-6 show the maximum and minimum downward forces for design of the isolator units. These forces are a result from the simultaneous application of unreduced design earthquake story forces, summarized in Table 11.5-4 and appropriate gravity loads to the model of the EOC. (See *Guide* Sec. 11.5.2.5 for the design load combinations.) As described in *Guide* Sec. 11.5.2.5, loads are applied simultaneously in two horizontal directions. The tables report the results for both of the orientations: 100 percent in the X direction, plus 30 percent in the Y direction, and 30 percent in the X direction, plus 100 percent in the Y direction. Where the analyses indicate that certain isolator units could uplift during peak DE response (as indicated by zero downward force), the amount of uplift is small and would not appreciably affect the distribution of earthquake forces in the superstructure.

Table 11.5-5 Maximum Downward Force (kips) for Isolator Design ($1.4D + 0.5L + Q_{DE}$)*

Column line	Maximum downward force (kips) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	347/348	661/892	522/557	652/856
B	833/634	1,335/**1,519**	1,418/1,278	984/1,186
C	537/502	939/890	1,053/1,046	1,074/1,070

1.0 kip = 4.45 kN

.*Forces at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; loads at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

Table 11.5-6 Minimum Downward Force (kips) for Isolator Design ($0.7D - Q_{DE}$)*

Column line	Maximum downward force (kips) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	87/84	165/**0**	200/169	163/**0**
B	**0**/123	11/**0**	23/69	280/59
C	169/196	299/243	428/427	447/442

1.0 kip = 4.45 kN

* Forces at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; loads at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

11.5.4.1.5 Maximum Considered Earthquake Forces and Displacements for Isolator Units

Simultaneous application of the unreduced MCE story forces, as summarized in Table 11.5-4 and appropriate gravity loads to the model of the EOC, result in the maximum downward forces on isolator units shown in *Guide* Table 11.5-7, and the maximum uplift displacements shown in Table 11.5-8. The load orientations and MCE load combinations, are described in *Guide* Sec. 11.5.2.5. The tables report the results for both of the load orientations: 100 percent in the X direction, plus 30 percent in the Y direction, and 30 percent in the X direction, plus 100 percent in the Y direction. Since the nonlinear model assumes that the isolators have no tension capacity, the values given in *Guide* Table 11.5-8 are upper bounds on uplift displacements.

Table 11.5-7 Maximum Downward Force (kips) on Isolator Units ($1.5D + 1.0L + Q_{MCE}$)*

Column line	Maximum downward force (kips) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	444/445	829/1,202	645/710	819/1,146
B	1,112/794	1,739/**2,006**	1,848/1,635	1,219/1,521
C	680/618	1,171/1,091	1,298/1,282	1,316/1,307

1.0 kip = 4.45 kN.
* Forces at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; loads at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

Table 11.5-8 Maximum Uplift Displacement (in.) of Isolator Units ($0.8D - Q_{MCE}$)*

Column line	Maximum uplift displacement (in.) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	No uplift	0.00/**0.94**	No uplift	0.00/0.50
B	0.54/0.00	0.19/0.45	0.13/0.00	0.00/0.14
C	No uplift	No uplift	No uplift	No uplift

1.0 in. = 25.4 mm.
* Displacements at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; displacements at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

11.5.4.1.6 Limits on Dynamic Analysis

The displacements and forces determined by the ELF procedure provide a basis for expeditious assessment of size and capacity of isolator units and the required strength of the superstructure. The results of the ELF procedure also establish limits on design parameters when dynamic analysis is used as the basis for design. Specifically, the total design displacement, D_{TD}, and the total maximum displacement of the isolation system, D_{TM}, determined by dynamic analysis cannot be less than 90 percent and 80 percent, respectively, of the corresponding ELF procedure values:

$$D_{TD,\ dynamic} \geq 0.9 D_{TD,\ \mathrm{ELF}} = 0.9(19.7) = 17.7 \text{ in.}$$

$$D_{TM,\ dynamic} \geq 0.8 D_{TM,\ \mathrm{ELF}} = 0.8(29.6) = 23.7 \text{ in.}$$

The superstructure, if regular, can also be designed for less base shear, but not less than 80 percent of the base shear from the ELF procedure:

$$V_{s,\ dynamic} \geq 0.8 V_{s,\ \mathrm{ELF}} = 0.8(2{,}550) = 2{,}040 \text{ kips } (= 0.14W)$$

As an exception to the above, design forces less than 80 percent of the ELF results are permitted if justified by time history analysis (which is seldom, if ever, the case).

11.5.4.2 Design of the Superstructure

The lateral forces, developed in the previous section, in combination with gravity loads, provide a basis for the design of the superstructure, using methods similar to those used for a conventional building. In this example, selection of member sizes were made based on the results of ETABS model calculations. Detailed descriptions of the design calculations are omitted, since the focus of this section is on design aspects unique to isolated structures (i.e., design of the isolation system, which is described in the next section).

Figures 11.5-8 and 11.5-9 are elevation views at Column Lines 2 and B, respectively. Figure 11.5-10 is a plan view of the building that shows the framing at the first floor level.

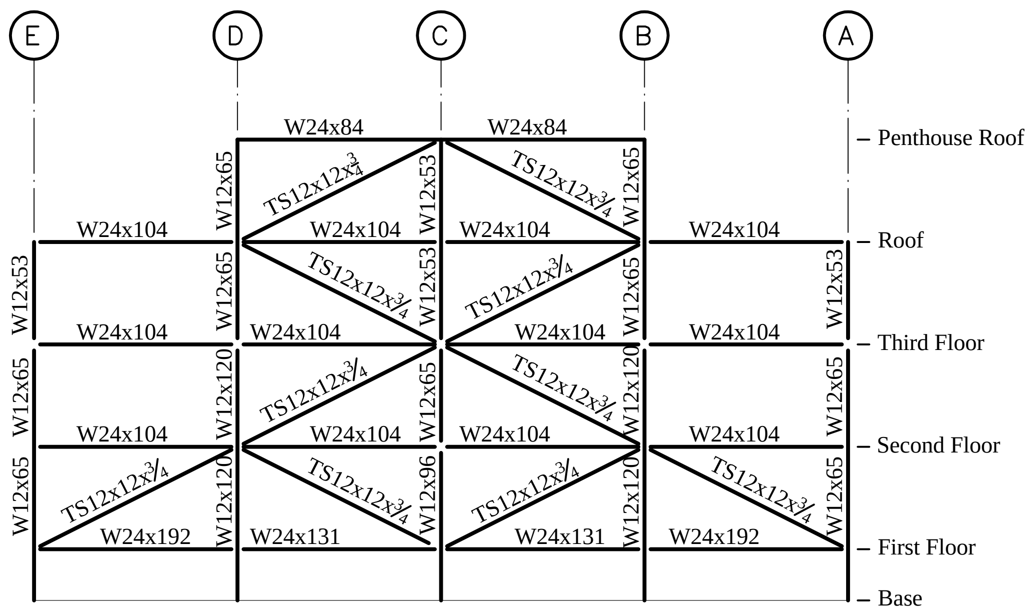


Figure 11.5-8 Elevation of framing on Column Line 2 (Column Line 6 is similar).

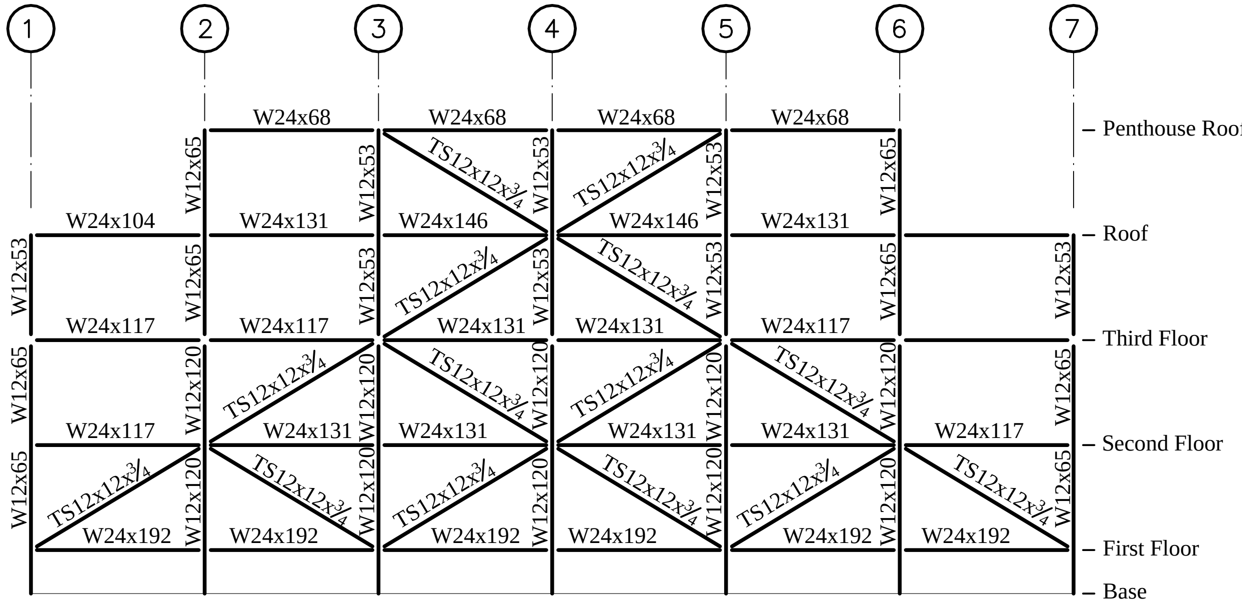


Figure 11.5-9 Elevation of framing on Column Line B (Column Line D is similar).

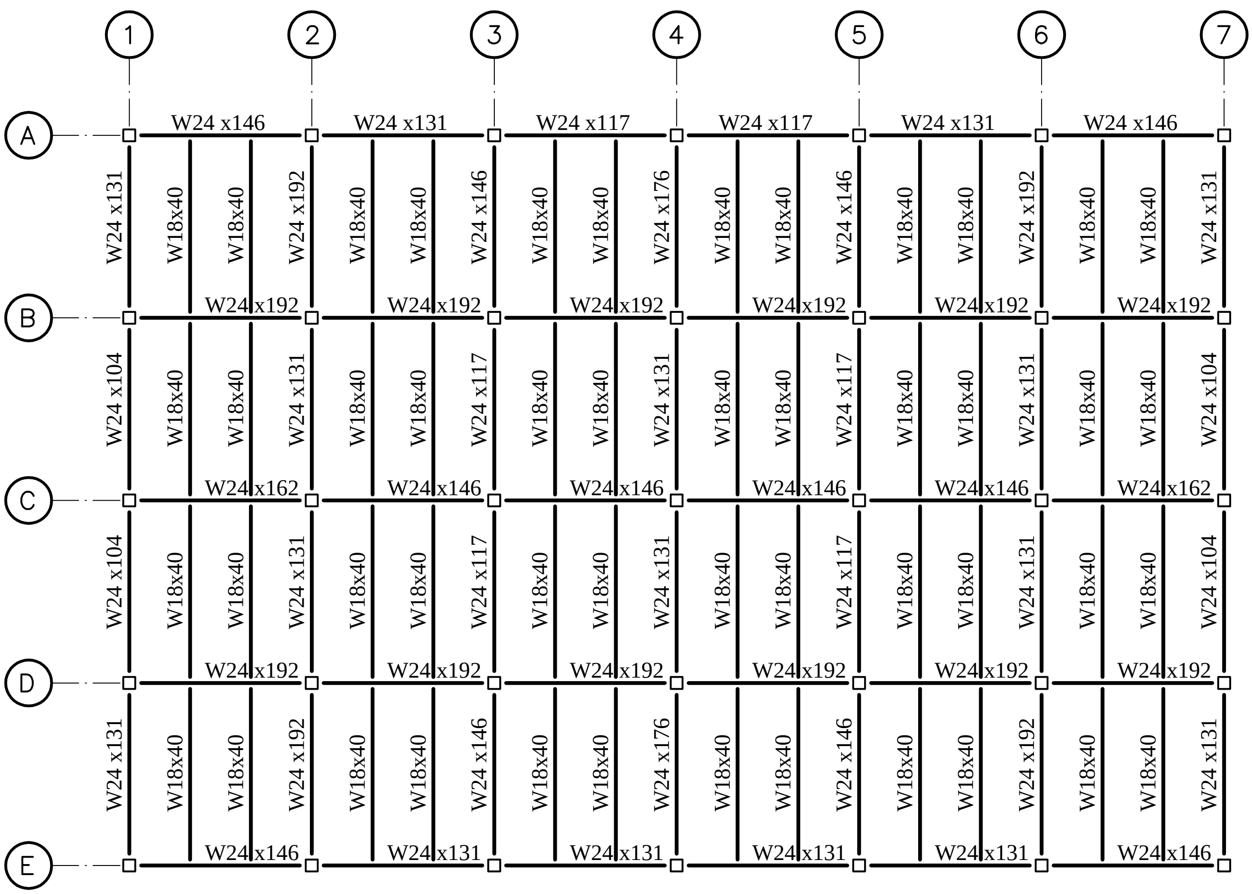


Figure 11.5-10 First floor framing plan.

As shown in the elevations (Figures 11.5-8 and 11.5-9), fairly large (12×12×¾ in.) tubes are consistently used throughout the structure for diagonal bracing. A quick check of these braces indicates that stresses will be at, or below yield for design earthquake loads. The six braces at the third floor, on lines 2, 4, and 6 (critical floor and direction of bracing) carry a reduced design earthquake force of about 400 kips each (= 2,080 kips/6 braces × cos(30°)). The corresponding stress is about 12.5 ksi for reduced design earthquake forces, or about 25 ksi for unreduced design earthquake forces; indicating that the structure is expected to remain elastic during the design earthquake.

As shown in Figure 11.5-10, the first floor framing has heavy, W24 girders along lines of bracing (lines B, D, 2, 4, and 6). These girders resist P-delta moments, as well as other forces. A quick check of these girders indicates that only limited yielding is likely, even for the MCE loads (up to about 2 ft of MCE displacement). Girders on Line 2 that frame into the column at Line B (critical columns and direction of framing), resist a P-delta moment due to the MCE of about 1,000 kip-ft (2,000 kips/2 girders × 2 ft/2). Moment in these girders due to MCE shear force in isolators is about 450 kip-ft (8.9 kips/in. × 24 in. × 4 ft/2 girders). Considering additional moment due to gravity loads, the plastic capacity of the first floor girders (1,680 kip-ft) would not be reached until isolation system displacements exceed about 2 ft. Even beyond this displacement, post-yield deformation would be limited (due to the limited extent and duration of MCE displacements beyond 2 ft), and the first floor girders would remain capable of stabilizing the isolator units.

11.5.4.3 Design of the Isolation System

The displacements and forces calculated in *Guide* Sec. 11.5.4.1 provide a basis to either:

1. Develop a detailed design of the isolator units or
2. Include appropriate design properties in performance-based specifications.

Developing a detailed design of an elastomeric bearing, requires a familiarity with rubber bearing technology, that is usually beyond the expertise of most structural designers; and often varies based on the materials used by different manufacturers. This example, like most recent isolation projects, will define design properties for isolator units that are appropriate for incorporation into a performance specification (and can be bid by more than one bearing manufacturer). Even though the specifications will place the responsibility for meeting performance standards with the supplier, the designer must still be knowledgeable of available products and potential suppliers, to ensure success of the design.

11.5.4.3.1 Size of Isolator Units

The design properties of the seismic isolator units are established based on the calculations of ELF demand, recognizing that dynamic analysis is required to verify these properties (and will likely justify slightly more lenient properties). The key parameters influencing size are:

1. Peak displacement of isolator units,
2. Average long-term (gravity) load on all isolator units,
3. Maximum long-term (gravity) load on individual isolator units, and
4. Maximum short-term load on individual isolator units (gravity plus MCE loads) including maximum uplift displacement.

These parameters are summarized in *Guide* Table 11.5-9. Loads or displacements on individual isolator units are taken as the maximum load or displacement on all isolator units (since the design is based on only one size of isolator unit). Reduced live load is used for determining long-term loads on isolators.

Table 11.5-9 Summary of Key Design Parameters [isolator unit location]

Key Design Parameter	ELF Procedure or Gravity Analysis	Dynamic Analysis Limit
DE displacement at corner of building (D_{TD})	19.7 in. [A1]	> 17.7 in. [A1]
MCE displacement at corner of building (D_{TM})	29.6 in. [A1]	> 23.7 in. [A1]
Average long-term load ($1.0D + 0.5L$)	477 kips [ALL]	N/A
Maximum long-term load ($1.2D + 1.6L$)	1,053 kips [C4]	N/A
DE max short-term load ($1.4D + 0.5L + Q_{\mathrm{DE}}$)	1,519 kips [B2]	N/A
DE minimum short-term load ($0.7D - Q_{\mathrm{DE}}$)	150 kips uplift [A2]	N/A
MCE max short-term load ($1.5D + 1.0L + Q_{\mathrm{MCE}}$)	2,006 kips [B2]	N/A
MCE minimum short-term load ($0.8D - Q_{\mathrm{MCE}}$)	0.94 in. uplift [A2]	N/A

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

As rule of thumb, elastomeric isolators should have a diameter, excluding the protective layer of cover, of no less than 1.25 times maximum earthquake displacement demand. In this case, the full displacement determined by the ELF procedure would require an isolator diameter of:

$$\varnothing_{\mathrm{ISO}} \geq 1.25(29.6) = 37 \text{ in. } (0.95 \text{ m})$$

If justified by dynamic analysis, then 80 percent of the ELF procedure displacement would require an isolator diameter of:

$$\varnothing_{\mathrm{ISO}} \geq 1.25(23.7) = 30 \text{ in. } (0.75 \text{ m})$$

For this example, a single size of isolator unit is selected with a nominal diameter of no less than 35.4 in. (0.90 m). Although the maximum vertical loads vary enough to suggest smaller diameter of isolator units at certain locations (such as at building corners), all of the isolator units must be large enough to sustain MCE displacements; which are largest at building corners, due to torsion. An isolator unit with a diameter of 35.4 in., has a corresponding bearing area of about A_b = 950 square in. The maximum long-term face pressure is about 1,100 psi (i.e., 1,053 kips/950 in.^{2}), which is less than the limit for most elastomeric bearing compounds. Average long-term face pressure is about 500 psi (i.e., 477 kips/950 in.^{2}) indicating the reasonably good distribution of loads among all isolator units.

The minimum effective stiffness is 6.9 kips/in. per isolator unit at the design displacement (i.e., about 16 in.). The height of the isolator unit is primarily a function of the height of the rubber, h_r, required to achieve this stiffness, given the bearing area, A_b, and the effective stiffness of the rubber compound. The EOC design accommodates rubber compounds with minimum effective shear modulus (at 150 percent shear strain) ranging from $G_{150\%}$ = 65 psi to 110 psi. Numerous elastomeric bearing manufacturers have rubber compounds with a shear modulus that falls within this range. Since rubber compounds (and in particular, high-damping rubber compounds) are nonlinear, the effective stiffness used for design must be associated with a shear strain that is close to the strain level for the design earthquake (e.g., 150 percent shear strain). For a minimum effective shear modulus of 65 psi, the total height of the rubber, h_r, would be:

$$h_r = \frac{G_{150\%} A_b}{k_{eff}} = \frac{65\ \text{lb/in.}^2 \times 950\ \text{in.}^2}{6{,}900\ \text{lb/in.}} = 8.9\ \text{in.} \cong 9\ \text{in.}$$

The overall height of the isolator unit, *H*, including steel shim and flange plates, would be about 15 in. For compounds with an effective minimum shear modulus of 110 psi, the rubber height would be proportionally taller (about 15 in.), and the total height of the isolator unit would be about 24 in.

11.5.4.3.2 Typical Isolation System Detail

For the EOC design, the isolation system has a similar detail at each column, as shown in Figure 11.5-11. The column has an extra large base plate that bears directly on the top of the isolator unit. The column base plate is circular, with a diameter comparable to that of the top plate of the isolator unit. Heavy, first floor girders frame into, and are moment connected to the columns (moment connections are required at this floor only). The columns and base plates are strengthened by plates that run in both horizontal directions, from the bottom flange of the girder to the base. Girders are stiffened above the seat plates, and at temporary jacking locations. The top plate of the isolator unit is bolted to the column base plate, and the bottom plate of the isolator unit is bolted to the foundation.

The foundation connection accommodates the removal and replacement of isolator units, as required by *Provisions* Sec. 13.6.2.8 [13.2.5.8]. The bottom plate of the isolator unit bears on a steel plate, that has a shear lug at the center grouted to the reinforced concrete foundation. Anchor bolts pass through holes in this plate and connect to threaded couplers that are attached to deeply embedded rods.

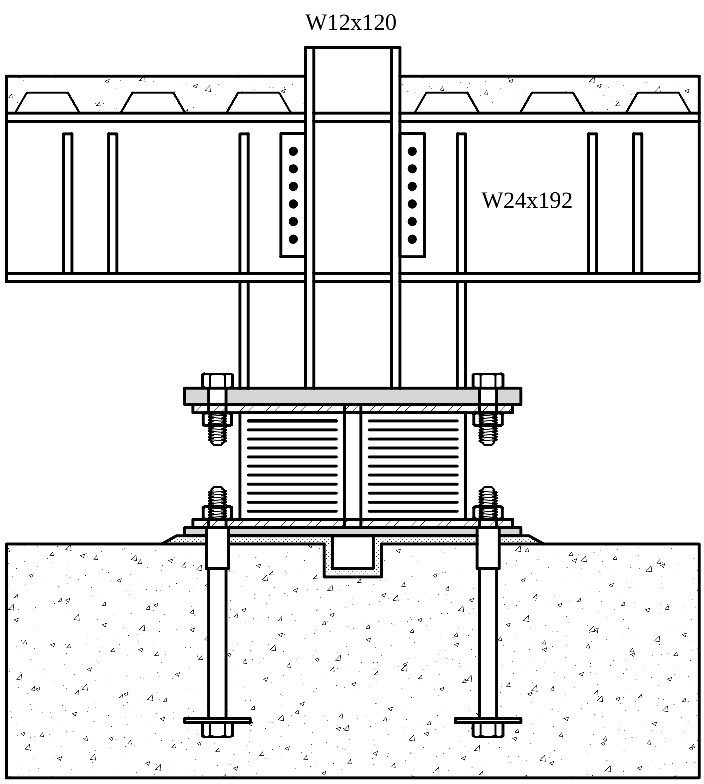


Figure 11.5-11 Typical detail of the isolation system at columns (for clarity, some elements not shown).

With the exception of the portion of the column above the first floor slab, each element shown in Figure 11.5-11 is an integral part of the isolation system (or foundation), and is designed for the gravity and unreduced design earthquake loads. In particular, the first floor girder, the connection of the girder to the column, and the connection of the column to the base plate, are designed for gravity loads and forces caused by horizontal shear and P-delta effects due to the unreduced design earthquake load (as shown earlier in Figure 11.4-1).

11.5.4.3.3 Design of Connections of Isolator Units

Connection of the top plate of the isolator unit, to the column base plate; and the connection of the bottom plate to the foundation, are designed for load combinations that include maximum downward forces (1.4D + 0.5L + Q_{DE}), and minimum downward (uplift) forces (0.7D - Q_{DE}). The reactions to bolts at the top and bottom plates of isolator units (ignoring shear friction and shear capacity of the lug at the base) are approximately equal, and include shear, axial load, and moment, in the most critical direction of response for individual bolts. Moments include the effects of P-delta and horizontal shear across the isolator unit, as described by the equations shown in Figure 11.4-1:

$$M = \frac{P\Delta}{2} + \frac{VH}{2}$$

In this case, $H_1 = H_2 = H/2$, where H is the height of the isolator unit (assumed to be 24 in., the maximum permissible height of isolator units). For maximum downward acting loads, maximum tension on any one of N (12) uniformly spaced bolts located in a circular pattern of diameter, D_b, must be equal to 43 in. and is estimated as follows:

$$F = \left(\frac{M}{S} - \frac{P}{A_b}\right)\frac{A_b}{N} = \left(\frac{4M}{A_b \cdot D_b} - \frac{P}{A_b}\right)\frac{A_b}{N} = \frac{1}{N}\left(\frac{4M}{D_b} - P\right) = \frac{1}{N}\left(\frac{2(P\Delta + VH)}{D_b} - P\right)$$
$$= \frac{1}{12}\left(\frac{2(1{,}519\text{ kips})(19.7\text{ in.}) + 175\text{ kips}(24\text{ in.})}{43\text{ in.}} - 1{,}519\text{ kips}\right) = \frac{1{,}587 - 1{,}519}{12} \cong 6\text{ kips}$$

In this calculation, the vertical load, P = 1,519 kips, is the maximum force occurring at Column B2, the deflection is based on D_{TD} = 19.7 in., and the shear force is calculated as V = 8.9 kips/in. × 19.7 in. The calculation indicates only a modest amount of tension force in anchor bolts, due to the maximum downward loads on the isolator units. However, the underlying assumption of the plane section's remaining plane is not conservative if top and bottom flange plates yield during a large lateral displacement of the isolator units. Rather than fabricate bearings with overly thick flange plates, manufacturers usually recommend anchor bolts that can carry substantial amounts of shear and tension, which can arise due to local bending of flanges.

Tension forces in anchor bolts can occur due to local uplift of isolator units. The ETABS model did not calculate uplift forces, since it was assumed that the isolator units would yield in tension. An estimate of the uplift load may be based on a maximum tension yield stress of about 150 psi, which produces a tension force, F_t, of no more than 150 kips for isolator units with a bearing area of 950 in.^{2} Applying uplift and shear loads to the isolator unit, the maximum tension force in each bolt is estimated at:

$$F_t = \left(\frac{M}{S} + \frac{P}{A_b}\right)\frac{A_b}{N} = \frac{1}{N}\left(\frac{2(P\Delta + VH)}{D_b} + P\right)$$
$$= \frac{1}{12}\left(\frac{2(150\text{ kips}(19.7\text{ in.}) + 175\text{ kips}(24\text{ in.})}{43\text{ in.}} + 150\text{ kips}\right) = \frac{333 + 150}{12} \cong 40\text{ kips}$$

The maximum shear load per bolt is:

$$F_v = \frac{V}{N} = \frac{175}{12} \cong 15 \text{ kips}$$

Twelve 1¼-in. diameter anchor bolts of A325 material are used for these connections. Alternatively, eight 1½-in. diameter bolts are permitted for isolator units manufactured with eight, rather than twelve, bolt holes. Bolts are tightened as required for a slip-critical connection (to avoid slip in the unlikely event of uplift).

Design of stiffeners and other components of the isolation system detail are not included in this example, but would follow conventional procedures for design loads. Design of the isolator unit including top and bottom flange plates is the responsibility of the manufacturer. *Guide* Sec. 11.5.6 includes example performance requirements of specifications that govern isolator design (and testing) by the manufacturer.

11.5.5 Design Verification Using Nonlinear Time History Analysis

The design is verified (and in some cases isolation system design properties are improved) using time history analysis of models that explicitly incorporate isolation system nonlinearity, including lateral force-deflection properties and uplift of isolator units, which are subject to net tension loads. Using three sets of horizontal earthquake components the EOC is analyzed separately for design earthquake (DE) and MCE ground shaking,. All analyses are repeated for two models of the EOC, one with upper-bound (UB) stiffness properties and the other with lower-bound (LB) stiffness properties of isolator units.

11.5.5.1 Ground Motion

Each pair of horizontal earthquake components is applied to the model in three different orientations relative to the principal axes of the building. Time histories are first applied to produce the maximum response along the X axis of the EOC model. The analyses are then repeated with the time histories, they are rotated 90° to produce the maximum response along the Y axis of the EOC model. Additionally, the time histories are rotated 45° to produce the maximum response along an X-Y line (to check if this orientation would produce greater response in certain elements than the two principal axis orientations). A total of 18 analyses (2 models × 3 component orientations × 3 sets of earthquake components) are performed separately for both DE and the MCE loads. Results are based on the maximum response of the parameter of interest calculated by each DE or MCE analysis, respectively. Parameters of interest include:

Design earthquake

1. Peak isolation system displacement
2. Maximum story shear forces (envelope over building height)

Maximum considered earthquake

1. Peak isolation system displacement
2. Maximum downward load on any isolator unit ($1.5D + 1.0L + Q_{\text{MCE}}$)
3. Maximum uplift displacement of any isolator unit ($0.8L - Q_{\text{MCE}}$).

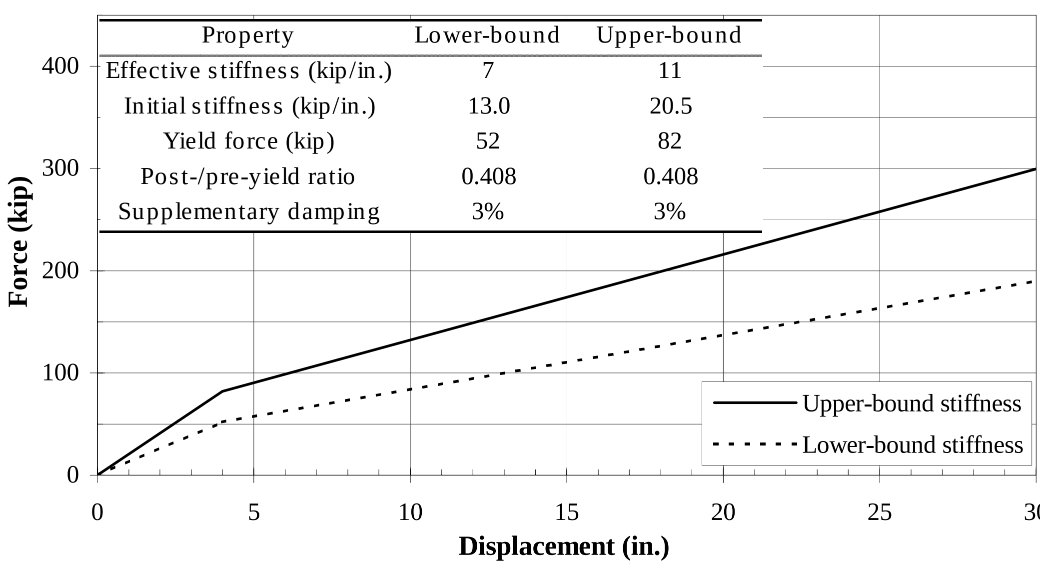
11.5.5.2 Bilinear Stiffness Modeling of Isolator Units

Nonlinear force-deflection properties of the isolator units are modeled using a bilinear curve; whose hysteretic behavior is a parallelogram, which is supplemented by a small amount of viscous damping. A bilinear curve is commonly used to model the nonlinear properties of elastomeric bearings; although other approaches are sometimes used, including a trilinear curve that captures stiffening effects of some rubber compounds at very high strains.

Using engineering judgement, the initial stiffness, the yield force, the ratio of post-yield to pre-yield stiffness for the bilinear curves, and the amount of supplementary viscous damping, β_v, are selected. The effective stiffness and effective damping values are used for preliminary design. The lower-bound bilinear stiffness curve is based on k_{Dmin} = 6.9 kips/in. (rounded to 7 kips/in.). The upper-bound bilinear stiffness curve is based on an effective stiffness of k_{Dmax} = 8.9 kips/in. times 1.2 (which the result is rounded to 11 kips/in.). The 1.2 factor is taken into account for the effects of aging over the design life of the isolators. Both upper-bound and lower-bound stiffness curves are based on effective damping (combined hysteretic and supplementary viscous) of $\beta_D \cong$ 15 percent. Figure 11.5-12 illustrates upper-bound and lower-bound bilinear stiffness curves, and summarizes the values of the parameters that define these curves.

For isolators with known properties, parameters defining bilinear stiffness may be based on test data provided by the manufacturer. For example, the bilinear properties shown in Figure 11.5-12, are compared with effective stiffness and effective damping test data that are representative of a HK090H6 high-damping rubber bearing manufactured by Bridgestone Engineered Products Company, Inc. Bridgestone is one of several manufacturers of high-damping rubber bearings and provides catalog data on design properties of standard isolators. The HK090H6 bearing has a rubber height of about 10 in., a diameter of 0.9 m, without the cover, and a rubber compound with a relatively low shear modulus (less than 100 psi at high strains).

Plots of the effective stiffness and damping of the HK090H6 bearing, are shown in Figure 11.5-13 (solid symbols), and are compared with plots of the calculated effective stiffness and damping (using the equations from Figure 11.4-2 and the bilinear stiffness curves shown in Figure 11.5-12). The effective stiffness curve of the HK090H6 bearing, is based on data from the third cycle of testing; and therefore, best represents the minimum effective (or lower-bound) stiffness. The plots indicate the common trend in effective stiffness of high-damping elastomeric bearings– significant softening up to 100 percent shear strain, fairly stable stiffness from 100 percent to 300 percent shear strain, and significant stiffening beyond 300 percent shear strain. The trend in effective damping is a steady decrease in damping with amplitude beyond about 150 percent shear strain.



Property	Lower-bound	Upper-bound
Effective stiffness (kip/in.)	7	11
Initial stiffness (kip/in.)	13.0	20.5
Yield force (kip)	52	82
Post-/pre-yield ratio	0.408	0.408
Supplementary damping	3%	3%

Figure 11.5-12 Stiffness and damping properties of EOC isolator units (1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN).

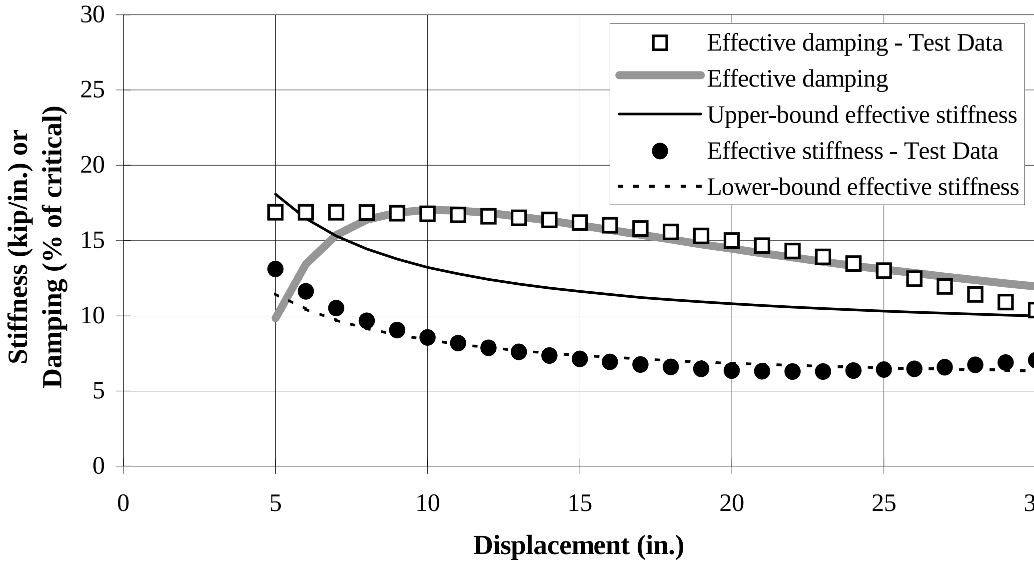


Figure 11.5-13 Comparison of modeled isolator properties to test data (1.0 in. = 25.4 mm, 1.0 kip/in. = 0.175 kN/mm).

The effective stiffness and damping of the bilinear stiffness curves capture the trends of the HK090H6 bearing reasonably well, as shown in Figure 11.5-13. Lower-bound effective stiffness fits the HK090H6 data well for response amplitudes of interest (i.e., the displacements in the range of about 15 in. through 25 in.). Likewise, the effective damping of the bilinear stiffness curves, is conservatively less than the effective damping based on test data for the same displacement range. These comparisons confirm that the bilinear stiffness properties of isolator units used for nonlinear analysis, are valid characterizations of the force-deflection properties of the Bridgestone HK090H6 bearing, also presumably, the bearings of other manufacturers that have comparable values of effective stiffness and damping.

11.5.5.3 Summary of Results for Time History Analyses

Tables 11.5-10 and 11.5-11, compare the results of the design earthquake and maximum considered earthquake time history analyses, respectively, with the corresponding values calculated using the ELF procedure. The peak response values noted for the time history analyses are, the maxima of X-axis and Y-axis directions of response. Because torsional effects were neglected in the time history analyses for this example, the displacement at the corner of the EOC was assumed to be 1.1 times the center displacement; this assumption may not be conservative. The reported shears occur below the indicated level.

Table 11.5-10 Design Earthquake Response Parameters - Results of ELF Procedure and Nonlinear Time History Analysis

Response parameter	ELF procedure	Time history analysis		
		Peak response	Model	Record
Isolation system displacement (in.)				
Displacement at center of EOC, D_D	16.3 in.	14.3 in.	LB stiffness	Sylmar
Displacement at corner of EOC, D_{TD}	19.7 in.	15.7 in.*		
Superstructure forces – story shear (kips)				
Penthouse roof	740 kips	400 kips	UB stiffness	Newhall
Roof (penthouse)	2,780 kips	1,815 kips		
Third floor	4,160 kips	3,023 kips		
Second floor	4,940 kips	4,180 kips		
First floor	5,100 kips	5,438 kips		

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.
* Displacement includes an arbitrary 10 percent increase for possible torsional response.

Table 11.5-11 Maximum Considered Earthquake Response Parameters – Results of ELF Procedure and Nonlinear Time History Analysis

Response parameter	ELF procedure	Time history analysis		
		Peak response	Model	Record
Displacement at center of EOC, D_M	24.5 in.	26.4 in.	LB stiffness	El Centro No. 6
Displacement at corner of EOC, D_{TM}	29.7 in.	29.0 in.[1]		

1.0 in. = 25.4 mm.
* Displacement includes an arbitrary 10 percent increase for possible torsional response.

The results of the design earthquake time history analyses, verify that the results of the ELF procedure are generally conservative. A design displacement of 16 in., and total design displacement of 20 in., are conservative bounds on calculated displacements, even if significant torsion should occur. The story shears calculated using the ELF procedure, are larger than those from the dynamic analyses at all superstructure elevations. The higher the elevation, the larger the margin between the story shears, which is an indication that the inverted triangular pattern (the results from *Provisions* Eq. 13.3.5 [13.3-9]), produces conservative results. At the second story (below the third floor), the critical level for the brace design the story shear from the dynamic analyses (3,023 kips) is only about three-quarters of the story shear from the ELF procedure (4,160 kips). The results of the MCE time history analyses show that the ELF procedure can underestimate the maximum displacement. Accordingly, a maximum displacement of 27 in., and total maximum displacement of 30 in., are used for the design specifications.

Tables 11.5-12 and 11.5-13, respectively, report the maximum downward forces on isolator units and the maximum uplift displacements determined from the maximum considered earthquake time history analyses.

These tables report two values for each isolator location representing both the X-axis and Y-axis orientations of the strongest direction of shaking of the time history record.

Table 11.5-12 Maximum Downward Force (kips) on Isolator Units ($1.5D + 1.0L + Q_{MCE}$)*

Column line	Upper-bound stiffness model – Newhall record – X-axis/Y-axis orientations			
	1	2	3	4
A	417/411	930/1,076	651/683	889/1,027
B	1,048/881	1,612/**1,766**	1,696/1,588	1,308/1,429
C	656/628	1,145/1,127	1,301/1,300	1,319/1,318

1.0 kip = 4.45 kN.
* Forces on Column Lines 5, 6 and 7 (not shown) are enveloped by those on Column Lines 3, 2, and 1, respectively; forces on Column Lines D and E (not shown) are enveloped by those on Column Lines B and A, respectively.

Table 11.5-13 Maximum Uplift Displacement (in.) Of Isolator Units ($0.8D - Q_{MCE}$)[1]

Column line	Upper-bound stiffness model – Newhall record – X-axis/Y-axis orientations			
	1	2	3	4
A	No uplift	0.00/**0.39**	No uplift	0.00/0.17
B	0.26/0.00	0.19/0.12	No uplift	No uplift
C	No uplift	No uplift	No uplift	No uplift

1.0 in. = 25.4 mm.
* Displacement on Column Lines 5, 6 and 7 (not shown) enveloped by those on Column Lines 3, 2, and 1, respectively; displacement on Column Lines D and E (not shown) are enveloped by those on Column Lines B and A, respectively.

Table 11.5-12 indicates a maximum downward force of 1,766 kips (at column B2), which is somewhat less than the value predicted using the ELF procedure (2,006 kips in Table 11.5-7). Table 11.5-13 includes the results from the controlling analysis, and indicates a maximum uplift displacement of 0.39 in. (at column A2), which is significantly less than the value predicted using the ELF procedure (0.94 in. as noted in Table 11.5-8). Based on these results, the design specification (*Guide* Sec. 11.5.2.10) uses values of 2,000 kips maximum downward force and ½ in. of maximum uplift displacement.

11.5.6 Design and Testing Criteria for Isolator Units

Detailed design of the isolator units for the EOC facility, is the responsibility of the bearing manufacturer subject to the design and testing (performance)of the criteria included in the construction documents (drawings and/or specifications). Performance criteria typically includes a basic description, and requirements for isolator unit sizes, the design life and durability, the environmental loads and fire-resistance criteria, Quality Assurance and Quality Control (including QC testing of production units), the design forces and displacements, and prototype testing requirements. This section summarizes key data and performance criteria for the EOC, including the criteria for prototype testing of isolator units, as required by *Provisions* Sec. 13.9.2 [13.6.1].

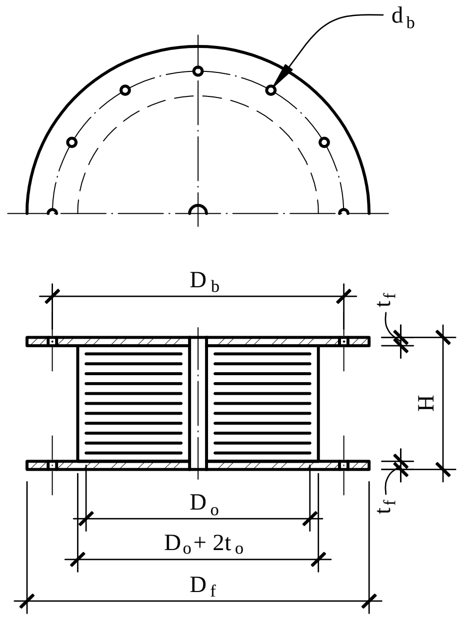


Figure 11.5-14 Isolator dimensions.

11.5.6.1 Dimensions and Configuration

Steel shim plate diameter, $D_o \geq 35.4$ in.

Rubber cover thickness, $t_o \geq 0.5$ in.

Rubber layer thickness, $t_r \leq 0.375$ in.

Gross rubber diameter, $D_o + 2t_o \leq 38$ in.

Bolt pattern diameter, $D_b = 43$ in.

Bolt hole diameter, $d_b = 1.5$ in.

Steel flange plate diameter, $D_f = 48$ in.

Steel flange plate thickness, $t_f \geq 1.5$ in.

Total height, $H \leq 24$ in.

11.5.6.2 Prototype Stiffness and Damping Criteria

Minimum effective stiffness (third cycle of test, typical vertical load) k_{Dmin}, $k_{Mmin} \geq 7.0$ kips/in.

Maximum effective stiffness (first cycle test, typical vertical load) k_{Dmax}, $k_{Mmax} \leq 9.0$ kips/in.

The k_{Dmin}, k_{Dmax}, k_{Mmin}, and $k_{Mmax,}$ are properties of the isolation system as a whole (calculated from the properties of individual isolator units using Provisions Eq. 13.9.5.1-1 through 13.9.5.1-4 [13.6-3 through 13.6-6]). Individual isolator units may have stiffness properties that fall outside the limits (by, perhaps, 10 percent), provided the average stiffness of all isolator units complies with the limits.

Effective damping	
(minimum of three cycles of test, typical vertical load)	$\beta_D \geq$ 15 percent
(minimum of three cycles of test, typical vertical load)	$\beta_M \geq$ 12 percent
Vertical Stiffness	
(average of three cycles of test, typical vertical load ± 50 percent)	$k_v \geq$ 5,000 kips/in.

11.5.6.3 Design Forces (Vertical Loads)

Maximum long-term load (individual isolator)	$1.2D + 1.6L$ = 1,200 kips
Typical load - cyclic load tests (average of all isolators)	$1.0D + 0.5L$ = 500 kips
Upper-bound load - cyclic load tests (all isolators)	$1.2D + 0.5L + \lvert E \rvert$ = 750 kips
Lower-bound load - cyclic load tests (all isolators)	$0.8D - \lvert E \rvert$ = 250 kips
Maximum short-term load (individual isolator)	$1.2D + 1.0L + \lvert E \rvert$ = 2,000 kips
Minimum short-term load (individual isolator)	$0.8D - \lvert E \rvert$ = tension force due to ½ in. of uplift

11.5.6.4 Design Displacements

Design earthquake displacement	D_D = 16 in.
Total design earthquake displacement	D_{TD} = 20 in.
Maximum considered earthquake displacement	D_M = 27 in.
Total maximum considered earthquake displacement	D_{TM} = 30 in.

11.5.6.5 Prototype Testing Criteria

Table 11.5-14 summarizes the prototype test criteria found in *Provisions* Sec. 13.9.2 [13.6.1] as well as the corresponding loads on isolator units of the EOC.

Table 11.5-14 Prototype Test Requirements

No. of cycles	*Provisions* criteria: Vertical load	*Provisions* criteria: Lateral load	EOC criteria: Vertical load	EOC criteria: Lateral load
Vertical stiffness test				
3 cycles	Typical ± 50%	None	500 ± 250 kips	None
Cyclic load tests to check wind effects (*Provisions* Sec. 13.9.2.3 [13.6.1.2])				
20 cycles	Typical	Design force	500 kips	± 20 kips
Cyclic load tests to establish effective stiffness and damping (*Provisions* Sec. 13.9.2.3 [13.6.1.2])				
3 cycles	Typical	$0.25D_D$	500 kips	± 4 in.
3 cycles	Upper-bound*	$0.25D_D$	750 kips	± 4 in.
3 cycles	Lower-bound*	$0.25D_D$	250 kips	± 4 in.
3 cycles	Typical	$0.5D_D$	500 kips	± 8 in.
3 cycles	Upper-bound*	$0.5D_D$	750 kips	± 8 in.
3 cycles	Lower-bound*	$0.5D_D$	250 kips	± 8 in.
3 cycles	Typical	$1.0D_D$	500 kips	± 16 in.
3 cycles	Upper-bound*	$1.0D_D$	750 kips	± 16 in.
3 cycles	Lower-bound*	$1.0D_D$	250 kips	± 16 in.
3 cycles	Typical	$1.0D_M$	500 kips	± 27 in.
3 cycles	Upper-bound*	$1.0D_M$	750 kips	± 27 in.
3 cycles	Lower-bound*	$1.0D_M$	250 kips	± 27 in.
3 cycles	Typical	$1.0D_{TM}$	500 kips	± 30 in.
Cyclic load tests to check durability (*Provisions* Sec. 13.9.2.3 [13.6.1.2])				
$30S_{D1}/S_{DS}B_D(= 20)$**	Typical load	$1.0D_{TD}$	500 kips	± 20 in.
Static load test of isolator stability (*Provisions* Sec. 13.9.2.6 [13.6.1.5])				
N/A	Maximum	$1.0D_{TM}$	2,000 kips	30 in.
N/A	Minimum	$1.0D_{TM}$	½-in. of uplift	30 in.

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

* Tests with upper-bound and lower-bound vertical loads are required by *Provisions* Sec. 13.9.2.3 [13.6.1.2] for isolator units that are vertical-load-carrying elements.

** The *Provisions* contains a typographical error where presenting this expression. The errata to the *Provisions* correct the error.

12

NONBUILDING STRUCTURE DESIGN

Harold O. Sprague Jr., P.E.

Chapter 14 of the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereafter, the *Provisions* and *Commentary*) is devoted to nonbuilding structures. Nonbuilding structures comprise a myriad of structures constructed of all types of materials with markedly different dynamic characteristics and a wide range of performance requirements.

Nonbuilding structures are a general category of structure distinct from buildings. Key features that differentiate nonbuilding structures from buildings include human occupancy, function, dynamic response, and risk to society. Human occupancy, which is incidental to most nonbuilding structures, is the primary purpose of most buildings. The primary purpose and function of nonbuilding structures can be incidental to society or the purpose and function can be critical for society.

In the past, many nonbuilding structures were designed for seismic resistance using building code provisions developed specifically for buildings. These code provisions were not adequate to address the performance requirements and expectations that are unique to nonbuilding structures. For example consider secondary containment for a vertical vessel containing hazardous materials. Nonlinear performance and collapse prevention, which are performance expectations for buildings, are inappropriate for a secondary containment structure, which must not leak.

Traditionally, the seismic design of nonbuilding structures depended on the various trade organizations and standards development organizations that were disconnected from the building codes. The *Provisions* have always been based upon strength design and multiple maps for seismic ground motion definition, whereas most of the industry standards were based on allowable stress design and a single zone map. The advent of the 1997 *Provisions* exacerbated the problems of the disconnect for nonbuilding structures with direct use of seismic spectral ordinates, and with the change to a longer recurrence interval for the seismic ground motion. It became clear that a more coordinated effort was required to develop appropriate seismic design provisions for nonbuilding structures.

This chapter develops examples specifically to help clarify Chapter 14 of the *Provisions*. The solutions developed are not intended to be comprehensive but instead focus on interpretation of *Provisions* Chapter 14 (Nonbuilding Structure Design Requirements). Complete solutions to the examples cited are beyond the scope of this chapter.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

Several noteworthy changes were made to the nonbuilding structures requirements of the 2003 *Provisions*. These include clearer definition of the scopes of Chapters 6 and 14, expanded, direct definition of structural systems (along with design parameters and detailing requirements) in Chapter 14, and a few specific changes for particular nonbuilding structural systems.

In addition to changes *Provisions* Chapter 14, the basic earthquake hazard maps were updated, the redundancy factor calculation was completely revised, and the minimum base shear equation for areas without near-source effects was eliminated.

Where they affect the design examples in this chapter, significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the *Provisions* and *Commentary*, the following publications are referenced in this chapter:

United States Geological Survey, 1996. *Seismic Design Parameters* (CD-ROM) USGS.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

American Water Works Association. 1996. Welded Steel Tanks for Water Storage. AWWA.

American Petroleum Institute (API), Welded steel tanks for oil storage. API 650, 10th Edition, November 1998.

12.1 NONBUILDING STRUCTURES VERSUS NONSTRUCTURAL COMPONENTS

Many industrial structures are classified as either nonbuilding structures or nonstructural components. This distinction is necessary to determine how the practicing engineer designs the structure. The intent of the *Provisions* is to provide a clear and consistent design methodology for engineers to follow regardless of whether the structure is a nonbuilding structure or a nonstructural component. Central to the methodology is how to determine which classification is appropriate.

The design methodology contained in *Provisions* Chapter 6, Architectural, Mechanical, and Electrical Components Design Requirements, focuses on nonstructural component design. As such, the amplification by the supporting structure of the earthquake-induced accelerations is critical to the design of the component and its supports and attachments. The design methodology contained in *Provisions* Chapter 14 focuses on the direct effects of earthquake ground motion on the nonbuilding structure.

Table 12-1 Applicability of the Chapters of the *Provisions*

Supporting Structure	Supported Item	
	Nonstructural Component	Nonbuilding Structure
Building	Chapter 5 [4 and 5]for supporting structure Chapter 6 for supported item	Chapter 5 [4 and 5]for supporting structure Chapter 14 for supported item
Nonbuilding	Chapter 14 for supporting structure Chapter 6 for supported item	Chapter 14 for both supporting structure and supported item

The example shown in Figure 12-1 is a combustion turbine, electric-power-generating facility with four bays. Each bay contains a combustion turbine and supports an inlet filter on the roof. The uniform seismic dead load of the supporting roof structure is 30 psf. Each filter weighs 34 kips.

The following two examples illustrate the difference between nonbuilding structures that are treated as nonstructural components, using *Provisions* Chapter 6, and those which are designed in accordance with *Provisions* Chapter 14. There is a subtle difference between the two chapters:

> 6.1: ". . .if the combined weight of the supported *components* and *nonbuilding structures* with flexible dynamic characteristics exceeds 25 percent of the weight of the *structure*, the *structure* shall be designed considering interaction effects between the *structure* and the supported items."

> 14.4: "If the weight of a *nonbuilding structure* is 25 percent or more of the combined weight of the *nonbuilding structure* and the supporting *structure*, the design seismic forces of the *nonbuilding structure* shall be determined based on the combined *nonbuilding structure* and supporting structural system.. . . "

The difference is the plural *components* and the singular *nonbuilding structure*, and that difference is explored in this example.

[The text has been cleaned up considerably in the 2003 edition but some inconsistencies persist. Sec. 14.1.5 indicates the scopes of Chapters 6 and 14. Both chapters consider the weight of an individual supported component or nonbuilding structure in comparison to the total seismic weight. Where the weight of such an individual item does not exceed 25 percent of the seismic weight, forces are determined in accordance with Chapter 6. Where a nonbuilding structure's weight exceeds 25 percent of the seismic weight, Sec. 14.1.5 requires a combined system analysis and the rigidity or flexibility of the supported nonbuilding structure is used in determining the R factor. In contrast, Sec. 6.1.1 requires consideration of interaction effects only where the weight exceeds 25 percent of the seismic weight **and** the supported item has flexible dynamic characteristics.]

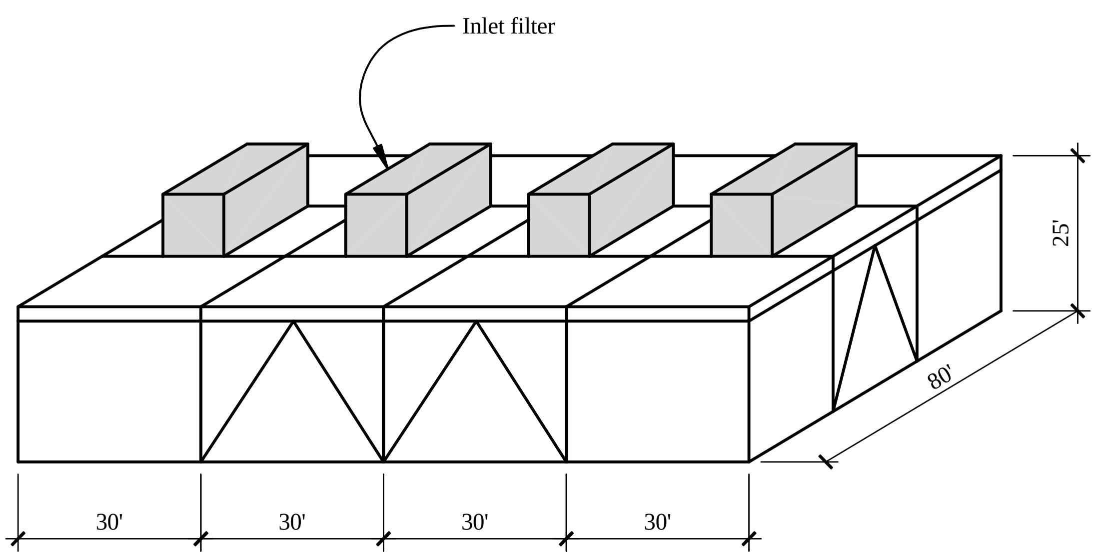


Figure 12-1 Combustion turbine building (1.0 ft = 0.3048 m).

12.1.1 Nonbuilding Structure

For the purpose of illustration assume that the four filter units are connected in a fashion that couples their dynamic response. Therefore, the plural components used in *Provisions* Sec. 6.1 is apparently the most meaningful provision.

[The text no longer contains a plural, but conceptually the frame could be considered a single item in this instance (just as the separate items within a single roof-top unit would be lumped together).]

12.1.1.1 Calculation of Seismic Weights

All four inlet filters = W_{IF} = 4(34 kips) = 136 kips

Support structure = W_{SS} = 4 (30 ft)(80 ft)(30 psf) = 288 kips

The combined weight of the nonbuilding structure (inlet filters) and the supporting structural system is

$$W_{combined} = 136 \text{ kips} + 288 \text{ kips} = 424 \text{ kips}$$

12.1.1.2 Selection of Design Method

The ratio of the supported weight to the total weight is:

$$\frac{W_{IF}}{W_{Combined}} = \frac{136}{424} = 0.321 > 25\%$$

Because the weight of the inlet filters is 25 percent or more of the combined weight of the nonbuilding structure and the supporting structure (*Provisions* Sec. 14.4 [14.1.5]), the inlet filters are classified as "nonbuilding structures" and the seismic design forces must be determined from analysis of the combined seismic-resistant structural systems. This would require modeling the filters, the structural components of the filters, and the structural components of the combustion turbine supporting structure to determine accurately the seismic forces on the structural elements as opposed to modeling the filters as lumped masses. [See the discussion added to Sec. 12.1.]

12.1.2 NONSTRUCTURAL COMPONENT

For the purpose of illustration assume that the inlet filters are independent structures, although each is supported on the same basic structure. In this instance, one filter is the nonbuilding structure. The question is whether it is heavy enough to significantly change the response of the combined system.

12.1.2.1 Calculation of Seismic Weights

One inlet filter = W_{IF} = 34 kips

Support structure = W_{SS} = 4 (30 ft)(80 ft)(30 psf) = 288 kips

The combined weight of the nonbuilding structures (all four inlet filters) and the supporting structural system is

$$W_{combined} = 4 \text{ (34 kips)} + 288 \text{ kips} = 424 \text{ kips}$$

12.1.2.2 Selection of Design Method

The ratio of the supported weight to the total weight is:

$$\frac{W_{IF}}{W_{Combined}} = \frac{34}{424} = 0.08 < 25\%$$

Because the weight of an inlet filter is less than 25 percent of the combined weight of the nonbuilding structures and the supporting structure (*Provisions* Sec. 14.4 [14.1.5]), the inlet filters are classified as "nonstructural components" and the seismic design forces must be determined in accordance with *Provisions* Chapter 6. In this example, the filters could be modeled as lumped masses. The filters and the filter supports could then be designed as nonstructural components.

12.2 PIPE RACK, OXFORD, MISSISSIPPI

This example illustrates the calculation of design base shears and maximum inelastic displacements for a pipe rack using the equivalent lateral force (ELF) procedure.

12.2.1 Description

A two-tier, 12-bay pipe rack in a petrochemical facility has concentrically braced frames in the longitudinal direction and ordinary moment frames in the transverse direction. The pipe rack supports four runs of 12-in.-diameter pipe carrying naphtha on the top tier and four runs of 8-in.-diameter pipe carrying water for fire suppression on the bottom tier. The minimum seismic dead load for piping is 35 psf on each tier to allow for future piping loads. The seismic dead load for the steel support structure is 10 psf on each tier.

Pipe supports connect the pipe to the structural steel frame and are designed to support the gravity load and resist the seismic and wind forces perpendicular to the pipe. The typical pipe support allows the pipe to move in the longitudinal direction of the pipe to avoid restraining thermal movement. The pipe support near the center of the run is designed to resist longitudinal and transverse pipe movement as well as provide gravity support; such supports are generally referred to as fixed supports.

Pipes themselves must be designed to resist gravity, wind, seismic, and thermally induced forces, spanning from support to support.

If the pipe run is continuous for hundreds of feet, thermal/seismic loops are provided to avoid a cumulative thermal growth effect. The longitudinal runs of pipe are broken up into sections by providing thermal/seismic loops at spaced intervals. In Figure 12-2, it is assumed thermal/seismic loops are provided at each end of the pipe run.

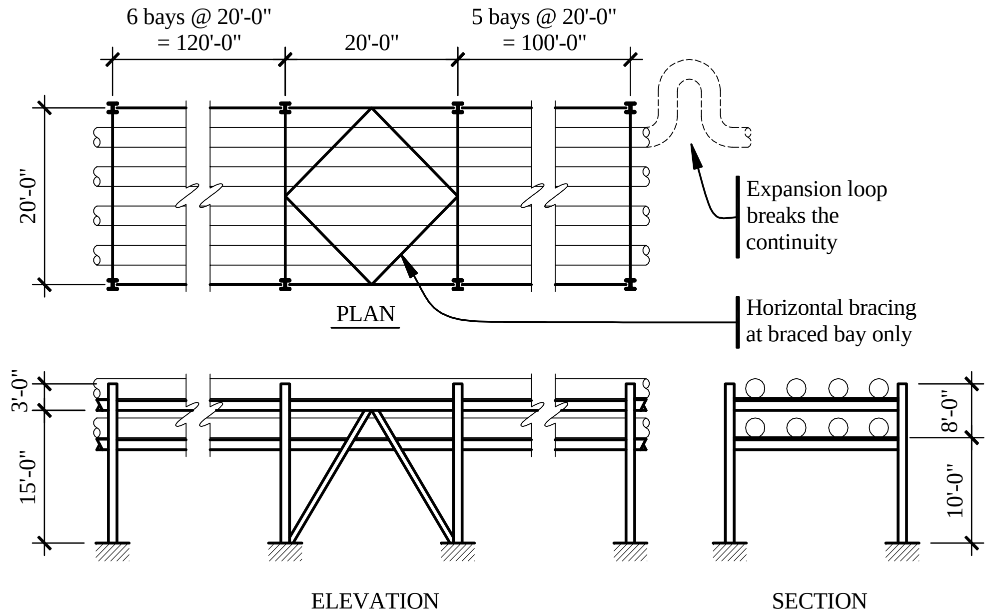


Figure 12-2 Pipe rack (1.0 ft = 0.3048 m).

12.2.2 *Provisions* Parameters

12.2.2.1 Ground Motion

The spectral response acceleration coefficients at the site are

S_{DS} = 0.40
S_{D1} = 0.18.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

12.2.2.2 Seismic Use Group and Importance Factor

The upper piping carries a hazardous material (naphtha) and the lower piping is required for fire suppression. The naphtha piping is included in *Provisions* Sec. 1.3.1, Item 11 [Sec. 1.2.1, Item 11], therefore, the pipe rack is assigned to Seismic Use Group III.

According to *Provisions* Sec. 14.5.1.2 [14.2.1], the importance factor, *I*, is 1.5 based on Seismic Use Group, Hazard, and Function. If these three measures yield different importance factors, the largest factor applies.

12.2.2.3 Seismic Design Category

For this structure assigned to Seismic Use Group III with S_{DS} = 0.40 and S_{D1} = 0.18, the Seismic Design Category is D according to *Provisions* Sec. 4.2.1 [1.4].

12.2.3 Design in the Transverse Direction

[Chapter 14 has been revised so that it no longer refers to Table 4.3-1. Instead values for design coefficients and detailing requirements are provided with the chapter.]

12.2.3.1 Design Coefficients

Using *Provisions* Table 14.5.1.1 [14.4-2] (which refers to *Provisions* Table 5.2.2 [4.3-1]), the parameters for this ordinary steel moment frame are

$R = 4$
$\Omega_0 = 3$
$C_d = 3½$

[In the 2003 *Provisions*, *R* factor options are presented that correspond to required levels of detailing. $R = 3.5$, $\Omega = 3$; $C_d = 3$.]

Ordinary steel moment frames are retained for use in nonbuilding structures such as pipe racks because they allow greater flexibility for accommodating process piping and are easier to design and construct than special steel moment frames.

12.2.3.2 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.4}{4/1.5} = 0.15$$

From analysis, $T = 0.42$ sec. For nonbuilding structures, the fundamental period is generally approximated for the first iteration and must be verified with final calculations. For many nonbuilding structures the maximum period limit contained in the first paragraph of *Provisions* Sec. 5.4.2 [5.2.2] is not appropriate. As a result, the examples in this chapter neglect that limit. Future editions of the *Provisions* will clarify that this limit does not apply to nonbuilding structures. [In the 2003 *Provisions*, Sec. 14.2.9 makes clear that the approximate period equations do not apply to nonbuilding structures.]

Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.18}{0.42\left(4/1.5\right)} = 0.161$$

Using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than

$$C_s = 0.044 I S_{DS} = 0.044(1.5)(0.4) = 0.0264$$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

Provisions Eq. 5.4.1.1-1 [5.2-2] controls; $C_s = 0.15$.

12.2.3.3 Seismic Weight

W = 2(20 ft)(20 ft)(35 psf + 10 psf) = 36 kips

12.2.3.4 Base Shear (*Provisions* Sec. 5.3.2 [5.2.1])

$V = C_sW$ = 0.15(36 kips) = 5.4 kips

12.2.3.5 Drift

Although not shown here, drift of the pipe rack in the transverse direction was calculated by elastic analysis using the design forces calculated above. The calculated lateral drift, δ_{xe} = 0.328 in. Using *Provisions* Eq. 14.3.2.1 [5.2-15],

$$\delta_x = \frac{C_d \delta_{xe}}{I} = \frac{3.5(0.328 \text{ in.})}{1.5} = 0.765 \text{ in.}$$

The lateral drift must be checked with regard to acceptable limits. The acceptable limits for nonbuilding structures are not found in codes. Rather, the limits are what is acceptable for the performance of the piping. In general, piping can safely accommodate the amount of lateral drift calculated in this example. P-delta effects must also be considered and checked as required in *Provisions* Sec. 5.4.6.2 [5.2.6.2].

12.2.3.6 Redundancy Factor

Some nonbuilding structures are designed with parameters from *Provisions* Table 5.2.2 [4.3-1]; if they are termed "nonbuilding structures similar to buildings". For such structures the redundancy factor applies, if the structure is in Seismic Design Category D, E, or F. Pipe racks, being fairly simple moment frames or braced frames, are in the category similar to buildings. Because this structure is assigned to Seismic Design Category D, *Provisions* Sec. 5.2.4.2 [4.3.3.2] applies. The redundancy factor is calculated as

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

where $r_{\max_x}$ is the fraction of the seismic force at a given level resisted by one component of the vertical seismic-force-resisting system at that level, and A_x is defined as the area of the diaphragm immediately above the story in question. Some interpretation is necessary for the pipe rack. Considering the transverse direction, the seismic-force-resisting system is an ordinary moment resisting frame with only two columns in a single frame. The frames repeat in an identical pattern. The "diaphragm" is the pipes themselves, which are not rigid enough to make one consider the 240 ft length between expansion joints as a diaphragm. Therefore, for the computation of ρ in the transverse direction, each 20-by-20 ft bay will be considered independently.

The maximum of the sum of the shears in the two columns equals the story shear, so the ratio r_{max} is 1.0. The diaphragm area is simply the bay area:

A_x = 20 ft × 20 ft = 400 ft^{2},

therefore,

$$\rho = 2 - \frac{20}{1.0\sqrt{400}} = 1.0$$

[The redundancy requirements have been changed substantially in the 2003 *Provisions*.]

12.2.3.7 Determining *E*

E is defined to include the effects of horizontal and vertical ground motions as follows:

$$E = \rho Q_E \pm 0.2\ S_{DS}\ D$$

where Q_E is the effect of the horizontal earthquake ground motions, which is determined primarily by the base shear just computed, and *D* is the effect of dead load. By putting a simple multiplier on the effect of dead load, the last term is an approximation of the effect of vertical ground motion. For the moment frame, the joint moment is influenced by both terms. *E* with the "+" on the second term when combined with dead and live loads will generally produce the largest negative moment at the joints, while *E* with the "-"on the second term when combined with the minimum dead load (0.9*D*) will produce the largest positive joint moments.

The *Provisions* also requires the consideration of an overstrength factor, Ω_0, on the effect of horizontal motions in defining *E* for components susceptible to brittle failure.

$$E = \rho\ \Omega_0\ Q_E \pm 0.2\ S_{DS}$$

The pipe rack does not appear to have components that require such consideration.

12.2.4 Design in the Longitudinal Direction

[In the 2003 *Provisions*, Chapter 14 no longer refers to Table 4.3-1. Instead, Tables 14.2-2 and 14.2-3 have design coefficient values and corresponding detailing requirements for each system.]

12.2.4.1 Design Coefficients

Using *Provisions* Table 14.5.1.1 [14.2-2] (which refers to *Provisions* Table 5.2.2 [4.3-1]), the parameters for this ordinary steel concentrically braced frame are:

$R = 4$
$\Omega_0 = 2$
$C_d = 4\frac{1}{2}$

[The 2003 *Provisions* allow selection of appropriate design coefficients and corresponding detailing for several systems. In the case of this example, *R* would equal 5, but the calculations that follow are not updated.]

Where *Provisions* Table 5.2.2 [4.3-1] is used to determine the values for design coefficients, the detailing reference sections noted in the table also apply. A concentric braced frame has an assigned *R* of 5, but an *R* of 4 is used to comply with *Provisions* Sec. 5.2.2.2.1 [4.3.1.2.1].

[In the 2003 *Provisions*, Chapter 14 no longer refers to Table 4.3-1. Instead, Tables 14.2-2 and 14.2-3 have design coefficient values and corresponding detailing requirements for each system. Chapter 14 contains no requirements corresponding to that found in Sec. 4.3.1.2.1 (related to *R* factors for systems in orthogonal directions).]

12.2.4.2 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.4}{4/1.5} = 0.15$$

From analysis, T = 0.24 seconds. The fundamental period for nonbuilding structures, is generally approximated for the first iteration and must be verified with final calculations. For many nonbuilding structures the maximum period limit contained in the first paragraph of *Provisions* Sec. 5.4.2 [5.2.2] is not appropriate. As a result, the examples in this chapter neglect that limit. Future editions of the *Provisions* are expected to clarify that this limit does not apply to nonbuilding structures. [In the 2003 *Provisions*, Sec. 14.2.9 makes clear that the approximate period equations do not apply to nonbuilding structures.]

Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.18}{0.24(4/1.5)} = 0.281$$

Provisions Sec. 14.5.1 [14.2.8] provides equations for minimum values of C_s that replace corresponding equations in *Provisions* Sec. 5.4.1.1 [5.2.1.1]. However, according to Item 2 of Sec. 14.5.1 [14.2.8, replacement of Chapter 5 equations for minima occurs only “for nonbuilding systems that have an R value provided in Table 14.5.1.1” [14.4-2]. In the present example the R values are taken from Table 5.2.2 so the minima defined in Sec. 5.4.1.1 apply. [In the 2003 *Provisions* this is no longer the case as reference to Table 4.3-1 has been eliminated. Since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed, no additional minimum base shear must be considered.]

Using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than:

$$C_s = 0.044IS_{DS} = 0.044(1.5)(0.4) = 0.0264$$

Provisions Eq. 5.4.1.1-1 [5.2-2] controls; C_s = 0.12.

12.2.4.3 Seismic Weight

$$W = 2(240\text{ ft})(20\text{ ft})(35\text{ psf} + 10\text{ psf}) = 432\text{ kips}$$

12.2.4.4 Base Shear

Using *Provisions* Eq. 5.3.2 [5.2-1]:

$$V = C_sW = 0.15(432\text{ kips}) = 64.8\text{ kips}$$

12.2.4.5 Redundancy Factor

For the longitudinal direction, the diaphragm is the horizontal bracing in the bay with the braced frames. However, given the basis for the redundancy factor, it appears that a more appropriate definition of A_x would be the area contributing to horizontal forces in the diagonal braces. Thus A_x = 20(240) = 4800 ft^2. The ratio r_x is 0.25; each of the four braces has the same stiffness, and each is capable of tension and compression. Therefore:

$$\rho = 2 - \frac{20}{0.25\sqrt{4800}} = 0.85 < 1.0, \quad \text{use } 1.0$$

[The redundancy requirements have been changed substantially in the 2003 *Provisions*.]

12.3 STEEL STORAGE RACK, OXFORD, MISSISSIPPI

This example uses the equivalent lateral force (ELF) procedure to calculate the seismic base shear in the east-west direction for a steel storage rack.

12.3.1 Description

A four-tier, five-bay steel storage rack is located in a retail discount warehouse. There are concentrically braced frames in the north-south and east-west directions. The general public has direct access to the aisles and merchandise is stored on the upper racks. The rack is supported on a slab on grade. The design operating load for the rack contents is 125 psf on each tier. The weight of the steel support structure is assumed to be 5 psf on each tier.

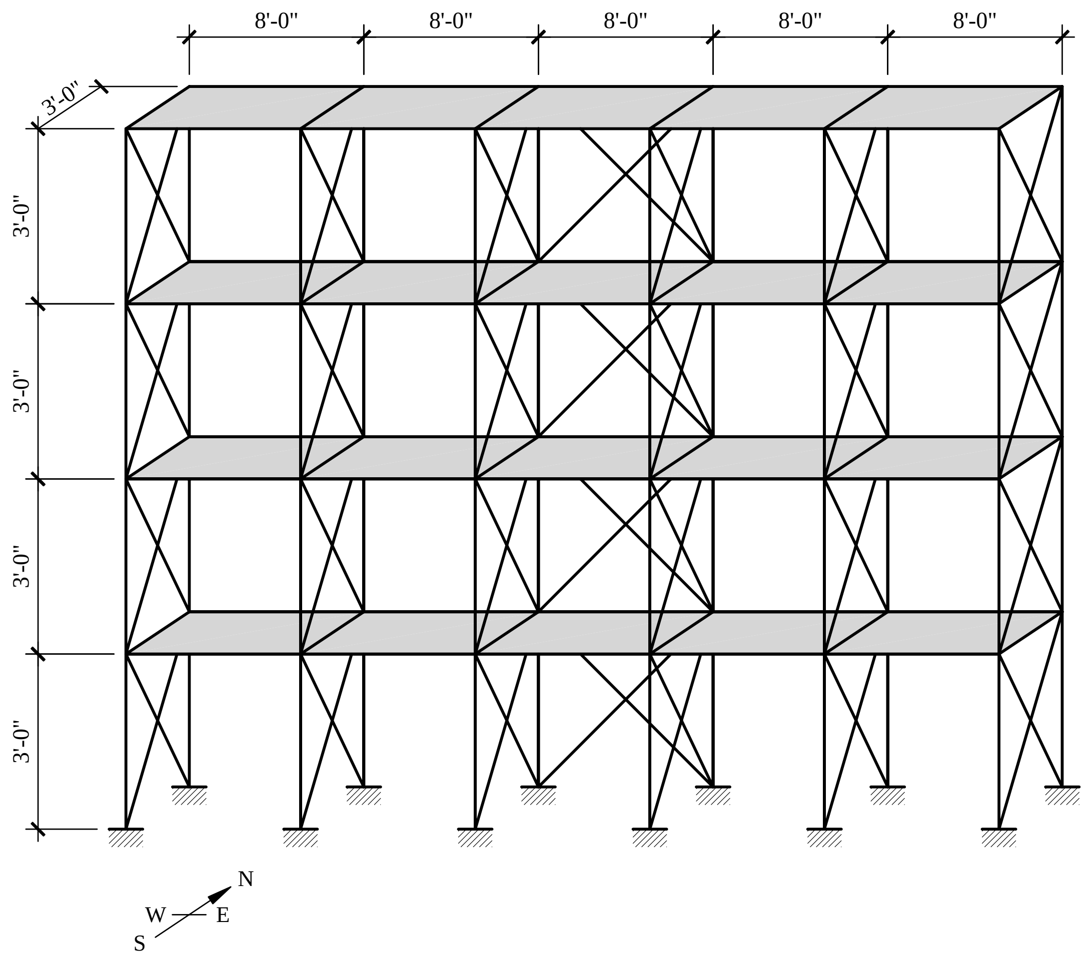


Figure 12-3 Steel storage rack (1.0 ft = 0.3048 m).

12.3.2 *Provisions* Parameters

12.3.2.1 Ground Motion

The spectral response acceleration coefficients at the site are as follows:

S_{DS} = 0.40
S_{D1} = 0.18

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

12.3.2.2 Seismic Use Group and Importance Factor

Use *Provisions* Sec. 1.3 [1.2]. The storage rack is in a retail facility. Therefore the storage rack is assigned to Seismic Use Group I. According to *Provisions* Sec. 14.6.3.1 and 6.1.5 [14.3.5.2], $I = I_p = 1.5$ because the rack is in an area open to the general public.

12.3.2.3 Seismic Design Category

Use *Provisions* Tables 4.2.1a and 4.2.1b [1.4-1 and 1.4-2]. Given Seismic Use Group I, S_{DS} = 0.40, and S_{D1} = 0.18, the Seismic Design Category is C.

12.3.2.4 Design Coefficients

According to *Provisions* Table 14.5.1.1 [14.2-3], the design coefficients for this steel storage rack are

R = 4
Ω_0 = 2
C_d = 3½

12.3.3 Design of the System

12.3.3.1 Seismic Response Coefficient

Provisions Sec. 14.6.3 [14.3.5]allows designers some latitude in selecting the seismic design methodology. Designers may use the Rack Manufacturer's Institute specification if they modify the equations to incorporate the seismic spectral ordinates contained in the *Provisions*; or they may use an R of 4 and use *Provisions* Chapter 5 according to the exception in *Provisions* Sec. 14.6.3.1. The exception is used in this example. [In the 2003 *Provisions* these requirements have been restructured so that the primary method is use of Chapter 5 with the design coefficients of Chapter 14; racks designed using the RMI method of Sec. 14.3.5.6 are deemed to comply.]

Using *Provisions* Eq. 5.4.1.1-1 [5.2-3]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.4}{4/1.5} = 0.15$$

From analysis, T = 0.24 seconds. For this particular example the short period spectral value controls the design. The period, for taller racks, however, may be significant and will be a function of the operating weight. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.18}{0.24\left(4/1.5\right)} = 0.281$$

Provisions Sec. 14.5.1 [14.2.8]provides equations for minimum values of C_s that replace corresponding equations in Sec. 5.4.1.1 [5.2.1.1]. The equations in Sec. 14.5.1 [14.2.8] are more conservative than those in Sec. 5.4.1.1 [5.2.1.1] because nonbuilding structures generally lack redundancy and are not as highly damped as building structures. These equations generally govern the design of systems with long periods. According to Item 2 of Sec. 14.5.1 [14.2.8], replacement of the Chapter 5 equations for minima occurs only "for nonbuilding systems that have an *R* value provided in Table 14.5.1.1" [14.2-2]. In the present example the *R* value is taken from Table 14.5.1.1 [14.2-2]and the Seismic Design Category is C so Eq. 14.5.1-1 [14.2-2] applies. Using that equation, C_s shall not be less than the following:

$$C_s = 0.14 S_{DS} I = 0.14(0.4)(1.5) = 0.084$$

Provisions Eq. 5.4.1.1-1[5.2-2] controls; $C_s = 0.15$.

12.3.3.2 Condition "a" (each rack loaded)

12.3.3.2.1 Seismic Weight

In accordance with *Provisions* Sec. 14.6.3.2 [14.3.5.3], Item a:

$$W_a = 4(5)(8\text{ ft})(3\text{ ft})[0.67(125\text{ psf})+5\text{ psf}] = 42.6\text{ kips}$$

12.3.3.2.2 Design Forces and Moments

Using *Provisions* Eq. 5.4.1 [5.2-1], the design base shear for condition "a" is calculated

$$V_a = C_s W = 0.15(42.6\text{ kips}) = 6.39\text{ kips}$$

In order to calculate the design forces, shears, and overturning moments at each level, seismic forces must be distributed vertically in accordance with *Provisions* Sec. 14.6.3.3 [14.3.5.4]. The calculations are shown in Table 12.3-1.

Table 12.3-1 Seismic Forces, Shears, and Overturning Moments

Level x	W_x (kips)	h_x (ft)	$w_x h_x^k$ (k = 1)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	10.65	12	127.80	0.40	2.56		
						2.56	7.68
4	10.65	9	95.85	0.30	1.92		
						4.48	21.1
3	10.65	6	63.90	0.20	1.28		
						5.76	38.4
2	10.65	3	31.95	0.10	0.63		
						6.39	57.6
Σ	42.6		319.5				

1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

12.3.3.2.3 Resisting Moment at the Base

$M_{OT,\,resisting} = W_a$ (1.5 ft) = 42.6(1.5 ft) = 63.9 ft-kips

12.3.3.3 Condition “b” (only top rack loaded)

12.3.3.3.1 Seismic Weight

In accordance with *Provisions* Sec. 14.6.3.2 [14.3.5.3], Item b:

W_b = 1(5)(8 ft)(3 ft)(125 psf) + 4(5)(8 ft)(3 ft)(5 psf) = 17.4 kips

12.3.3.3.2 Base Shear

Using *Provisions* Eq. 5.4.1 [5.2-1], the design base shear for condition “b” is calculated as follows:

$V_b = C_s W$ = 0.15(17.4 kips) = 2.61 kips

12.3.3.3.3 Overturning Moment at the Base

Although the forces could be distributed as shown above for condition “a”, a simpler, conservative approach for condition “b” is to assume that a seismic force equal to the entire base shear is applied at the top level. Using that simplifying assumption,

$M_{OT} = V_b$ (12 ft) = 2.61 kip (12 ft) = 31.3 ft-kips

12.3.3.3.4 Resisting Moment at the Base

$M_{OT,\,resisting} = W_b$ (1.5 ft) = 17.4(1.5 ft) = 26.1 ft-kips

12.3.3.4 Controlling Conditions

Condition “a” controls shear demands at all but the top level.
Although the overturning moment is larger under condition “a ,” the resisting moment is larger than the overturning moment. Under condition “b” the resistance to overturning is less than the applied

overturning moment. Therefore, the rack anchors must be designed to resist the uplift induced by the base shear for condition “b”.

12.3.3.5 Torsion

It should be noted that the distribution of east-west seismic shear will induce torsion in the rack system because the east-west brace is only on the back of the storage rack. The torsion should be resisted by the north-south braces at each end of the bay where the east-west braces are placed. If the torsion were to be distributed to each end of the storage rack, the engineer would be required to calculate the transfer of torsional forces in diaphragm action in the shelving, which may be impractical.

12.4 ELECTRIC GENERATING POWER PLANT, MERNA, WYOMING

This example highlights some of the differences between the design of nonbuilding structures and the design of building structures. The boiler building in this example illustrates a solution using the equivalent lateral force (ELF) procedure. Due to mass irregularities, the boiler building would probably also require a modal analysis. For brevity, the modal analysis is not illustrated.

12.4.1 Description

Large boilers in coal-fired electric power plants are generally suspended from support steel near the roof level. Additional lateral supports (called buck stays) are provided near the bottom of the boiler. The buck stays resist lateral forces but allow the boiler to move vertically. Lateral seismic forces are resisted at the roof and at the buck stay level. Close coordination with the boiler manufacturer is required in order to determine the proper distribution of seismic forces.

In this example, a boiler building for a 950 mW coal-fired electric power generating plant is braced laterally with ordinary concentrically braced frames in both the north-south and the east-west directions. The facility is part of a grid and is not for emergency back up of a Seismic Use Group III facility.

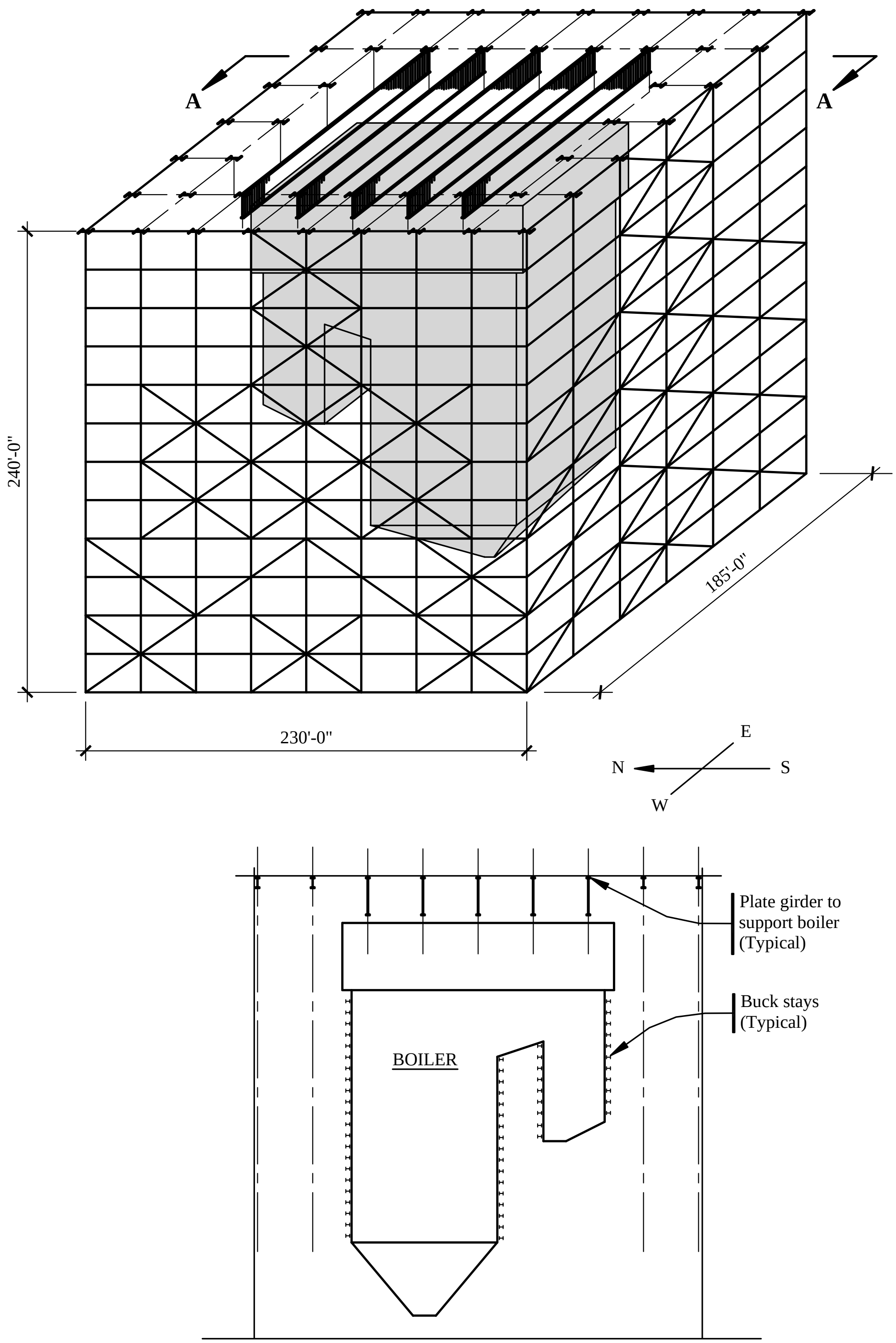
The dead load of the structure, equipment, and piping, W_{DL}, is 16,700 kips.

The weight of the boiler in service, W_{Boiler}, is 31,600 kips.

The natural period of the structure (determined from analysis) is as follows:

North-South, T_{NS} = 1.90 seconds

East-West, T_{EW} = 2.60 seconds


A
A
240'-0"
230'-0"
185'-0"
E
N
S
W
Plate girder to
support boiler
(Typical)
Buck stays
(Typical)
BOILER

Section A-A

12.4.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2]) (for continuous operation, but not for emergency back up of a Seismic Use Group III facility)	=	II
Occupancy Importance Factor, I (*Provisions* Sec. 1.4 [14.2.1])	=	1.25
Site Coordinates	=	42.800° N, 110.500° W
Short Period Response, S_S (*Seismic Design Parameters*)	=	0.966
One Second Period Response, S_1 (*Seismic Design Parameters*)	=	0.278
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	D (default)
Acceleration-based site coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.11
Velocity-based site coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.84
Design spectral acceleration response parameters		
$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.11)(0.966)$	=	0.715
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.84)(0.278)$	=	0.341
Seismic Design Category (*Provisions* Sec. 4.2 [1.4])	=	D
Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-2])	=	Steel concentrically braced frame (Ordinary)
Response Modification Coefficient, R (*Provisions* Table 5.2.2)	=	5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2)	=	2
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2)	=	4½
Height limit (*Provisions* Table 14.5.1.1)	=	None

Note: If the structure were classified as a "building," its height would be limited to 35 ft for a Seismic Design Category D ordinary steel concentrically braced frame, according to the *Provisions* Table 5.2.2. The structure is, however, defined as a nonbuilding structure according to *Provisions* Sec. 14.6.3.4. *Provisions* Table 14.5.1.1 does not restrict the height of a nonbuilding structure using an ordinary steel concentrically braced frame.

[Changes in the 2003 *Provisions* would affect this example significantly. Table 14.2-2 would be used to determine design coefficients and corresponding levels of detailing. For structures of this height using an ordinary concentrically braced frame system, R = 1.5, Ω_0 = 1, and C_d = 1.5. Alternatively, a special concentrically braced frame system could be employed.]

12.4.3 Design in the North-South Direction

12.4.3.1 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1[5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.715}{5/1.25} = 0.179$$

From analysis, T = 1.90 seconds. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.341}{1.90\left(5/1.25\right)} = 0.045$$

but using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than:

$C_s = 0.044IS_{DS} = 0.044(1.25)(0.715) = 0.0393$

[Under the 2003 *Provisions* no additional minimum base shear must be considered since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed.]

Provisions Eq. 5.4.1.1-2 [5.2-3] controls; C_s = 0.045.

12.4.3.2 Seismic Weight

Calculate the total seismic weight, W, as:

$W = W_{DL} + W_{Boiler}$ = 16,700 kips + 31,600 kips = 48,300 kips

12.4.3.3 Base Shear

Using *Provisions* Eq. 5.4.1 [5.2-1]:

$V = C_sW$ = 0.045(48,300 kips) = 2170 kips

12.4.3.4 Redundancy Factor

Refer to Sec. 12.2.3.6 for an explanation of the application of this factor to nonbuilding structures similar to buildings. The seismic force resisting system is an ordinary concentric braced frame with five columns in a single line of framing. The number of bays of bracing diminishes near the top, and the overall plan area is large. For the purposes of this example, it will be assumed that the structure lacks redundancy and ρ = 1.5.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. If it is assumed that the structure would fail the redundancy criteria, ρ = 1.3.]

12.4.3.5 Determining E

See Sec. 12.2.3.7.

12.4.4 Design in the East-West Direction

12.4.4.1 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.715}{5/1.25} = 0.179$$

From analysis, T = 2.60 seconds. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed:

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.341}{2.60\left(5/1.25\right)} = 0.0328$$

Using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than:

$$C_s = 0.044IS_{DS} = 0.044(1.25)(0.715) = 0.0393$$

[Under the 2003 *Provisions* no additional minimum base shear must be considered since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed.]

Provisions Eq. 5.4.1.1-3 controls; C_s = 0.0393. [Under the 2003 *Provisions*, Eq. 5.2-3 would control the base shear coefficient for this example.]

12.4.4.2 Seismic Weight

Calculate the total seismic weight, W, as

$$W = W_{DL} + W_{Boiler} = 16{,}700 \text{ kips} + 31{,}600 \text{ kips} = 48{,}300 \text{ kips}$$

12.4.4.3 Base Shear

Using *Provisions* Eq. 5.4.1 [5.2-1]:

$$V = C_sW = 0.0393(48{,}300 \text{ kips}) = 1900 \text{ kips}$$

12.5 PIER/WHARF DESIGN, LONG BEACH, CALIFORNIA

This example illustrates the calculation of the seismic base shear in the east-west direction for the pier using the ELF procedure.

12.5.1 Description

A private shipping company is developing a pier in Long Beach, California, to service container vessels. In the north-south direction, the pier is tied directly to an abutment structure supported on grade. In the east-west direction, the pier resists seismic forces using moment frames.

The design live load for container storage is 1000 psf.

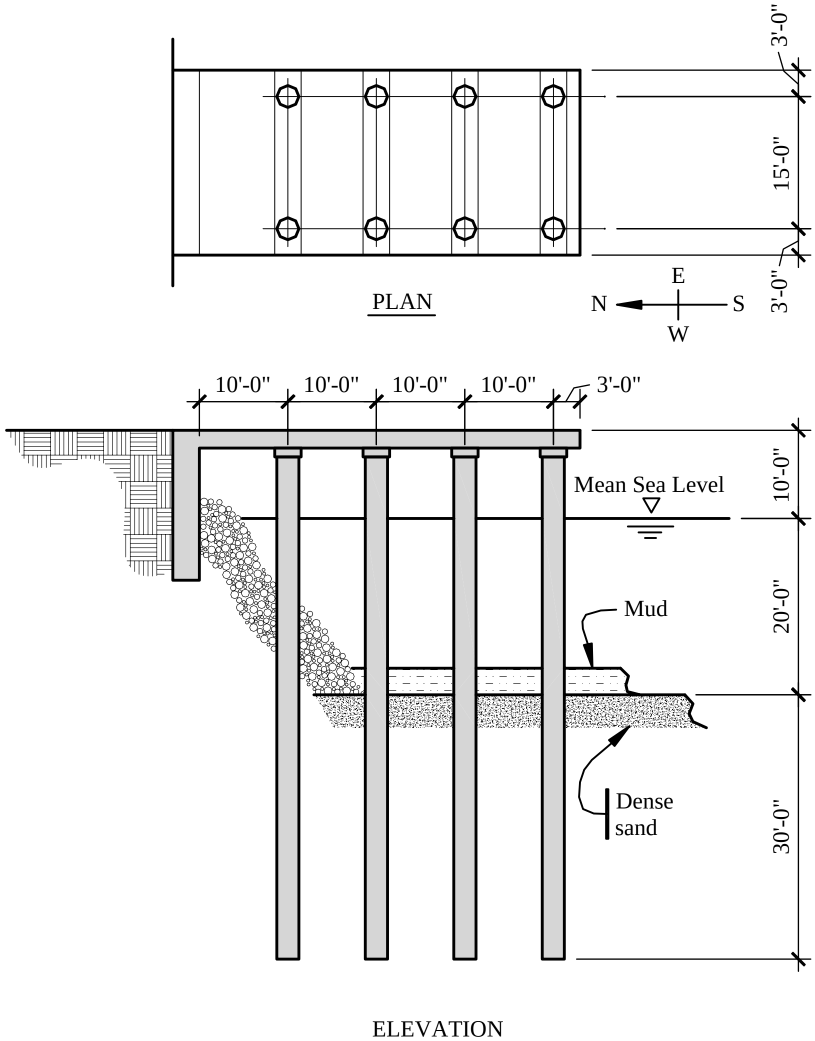


Figure 12-5 Pier plan and elevation (1.0 ft = 0.3048 m).

12.5.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2]) (The pier serves container vessels that carry no hazardous materials.)	=	I
Importance Factor, I (*Provisions* Sec. 14.5.1.2 [14.2.1])	=	1.0
Short Period Response, S_S	=	1.75
One Second Period Response, S_1	=	0.60
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	D (dense sand)
Acceleration-based Site Coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.0

Velocity-based Site Coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.5

Design spectral acceleration response parameters

$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.0)(1.75)$	=	1.167
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.5)(0.60)$	=	0.60
Seismic Design Category (*Provisions* Sec. 4.2)	=	D
Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-2])	=	Intermediate concrete moment frame
Response Modification Coefficient, R (*Provisions* Table 5.2.2)	=	5

(The *International Building Code* and the 2002 edition of ASCE 7 would require an R value of 3.)

System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2)	=	3
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2)	=	4½
Height limit (*Provisions* Table 14.5.1.1)	=	50 ft

If the structure was classified as a building, an intermediate reinforced concrete moment frame would not be permitted in Seismic Design Category D.

[Changes in the 2003 *Provisions* would affect this example significantly. Table 14.2-2 would be used to determine design coefficients and corresponding levels of detailing. For structures of this height using an intermediate concrete moment frame system, R = 3, Ω_0 = 2, and C_d = 2.5.]

12.5.3 Design of the System

12.5.3.1 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.167}{5/1.0} = 0.233$$

From analysis, T = 0.596 seconds. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed:

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.60}{0.596\left(5/1.0\right)} = 0.201$$

Using *Provisions* Eq. 5.4.1.1-3 , C_s shall not be less than:

$C_s = 0.044IS_{DS} = 0.044(1.0)(1.167) = 0.0513$

[Under the 2003 *Provisions* no additional minimum base shear must be considered since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed.]

Provisions Eq. 5.4.1.1-2 [5.2-3] controls; C_s = 0.201.

12.5.3.2 Seismic Weight

In accordance with Provisions Sec. 5.3 [5.2.1] and 14.6.6 [14.2.6], calculate the dead load due to the deck, beams, and support piers, as follows:

$$W_{Deck} = 1.0 \text{ ft}(43 \text{ ft})(21 \text{ ft})(0.150 \text{ kip/ft}^3) = 135.5 \text{ kips}$$

$$W_{Beam} = 4(2 \text{ ft})(2 \text{ ft})(21 \text{ ft})(0.150 \text{ kip/ft}^3) = 50.4 \text{ kips}$$

$$W_{Pier} = 8[\pi(1.25 \text{ ft})^2][(10 \text{ ft} - 3 \text{ ft}) + (20 \text{ ft})/2](0.150 \text{ kip/ft}^3) = 100.1 \text{ kips}$$

$$W_{DL} = W_{Deck} + W_{Beams} + W_{Piers} = 135.5 + 50.4 + 100.1 = 286.0 \text{ kips}$$

Calculate 25 percent of the storage live load

$$W_{1/4\,LL} = 0.25(1000 \text{ psf})(43 \text{ ft})(21 \text{ ft}) = 225.8 \text{ kips}$$

Calculate the weight of the displaced water (*Provisions* Sec. 14.6.6 [14.3.3.1])

$$W_{Disp.\,water} = 8[\pi(1.25 \text{ ft})^2](20 \text{ ft})(64 \text{ pcf}) = 50.27 \text{ kips}$$

Therefore, the total seismic weight is

$$W = W_{DL} + W_{1/4LL} + W_{Disp.\,water} = 286.0 + 225.8 + 50.27 = 562.1 \text{ kips}$$

12.5.3.3 Base Shear

Using *Provisions* Eq. 5.3.2 [5.2-1]:

$$V = C_sW = 0.201(562.1 \text{ kips}) = 113.0 \text{ kips}$$

12.5.3.4 Redundancy Factor

This structure is small in area and has a large number of piles. Following the method described in Sec. 12.2.3.6, yields ρ = 1.0.

12.6 TANKS AND VESSELS, EVERETT, WASHINGTON

The seismic response of tanks and vessels can be significantly different from that of buildings. For a structure composed of interconnected solid elements, it is not difficult to recognize how ground motions accelerate the structure and cause inert forces within the structure. Tanks and vessels, when empty, respond in a similar manner.

When there is liquid in the tank, the response is much more complicated. As earthquake ground motions accelerate the tank shell, the shell applies lateral forces to the liquid. The liquid, which responds to those lateral forces. The liquid response may be amplified significantly if the period content of the earthquake ground motion is similar to the natural sloshing period of the liquid.

Earthquake-induced impulsive fluid forces are those calculated assuming that the liquid is a solid mass. The convective fluid forces are those that result from sloshing in the tank. It is important to account for the convective forces on columns and appurtenances inside the tank, because they are affected by sloshing in the same way that waves affect a pier in the ocean.

The freeboard considerations are critical. Often times, the roof acts as a structural diaphragm. If a tank does not have sufficient freeboard, the sloshing wave can rip the roof from the wall of the tank. This could result in the failure of the wall and loss of the liquid within.

The nature of seismic design for liquid containing tanks and vessels is complicated. The fluid mass that is effective for impulsive and convective seismic forces is discussed in the literature referenced in the NEHRP *Provisions* and *Commentary*.

12.6.1 Flat-Bottom Water Storage Tank

12.6.1.1 Description

This example illustrates the calculation of the design base shear using the equivalent lateral force (ELF) procedure for a steel water storage tank used to store potable water for a process within a chemical plant (Figure 12-6).

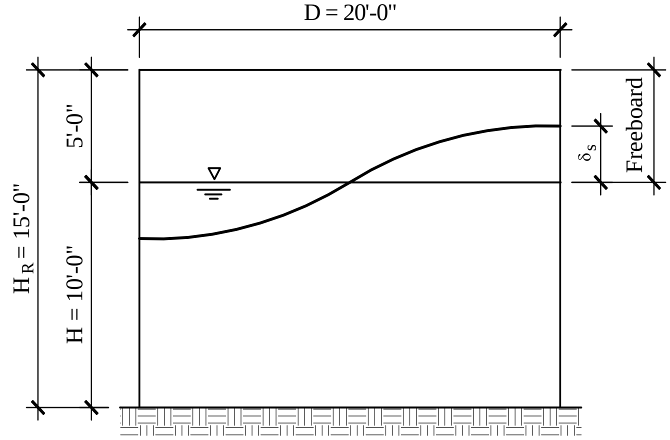


Figure 12-6 Storage tank section (1.0 ft = 0.3048 m).

The tank is located away from personnel working within the facility.

The weight of the tank shell, roof, and equipment is 15,400 lb.

12.6.1.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	=	I
Importance Factor, I (*Provisions* Sec. 14.5.1.2 [14.2.1])	=	1.0
Site Coordinates	=	48.000° N, 122.250° W
Short Period Response, S_S	=	1.236

One Second Period Response, S_1	=	0.406
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	C (per geotech)
Acceleration-based Site Coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.0
Velocity-based Site Coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.39

Design spectral acceleration response parameters

$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.0)(1.236)$	=	0.824
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.39)(0.406)$	=	0.376

Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-3])	=	Flat-bottom, ground-supported, anchored, bolted steel tank
Response Modification Coefficient, R (*Provisions* Table 14.5.1.1 [14.2-3])	=	3
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [14.2-3])	=	2
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [14.2-3])	=	2½

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

12.6.1.3 Calculations for Impulsive Response

12.6.1.3.1 Natural Period for the First Mode of Vibration

Based on analysis, the period for impulsive response of the tank and its contents is T_i = 0.14 sec.

12.6.1.3.2 Spectral Acceleration

Based on *Provisions* Figure 14.7.3.6-1 [14.4-1]:

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.376}{0.824} = 0.456 \text{ seconds}$$

Using *Provisions* Sec. 14.7.3.6.1 [114.4.7.5.1] with $T_i < T_s$,:

$S_{ai} = S_{DS} = 0.824$

12.6.1.3.3 Seismic (Impulsive) Weight

W_{tank} = 15.4 kips

W_{1water} = π(10 ft)^2(10 ft)(0.0624 kip/ft^3) (W_1/W_T)= 196.0 (0.75) kips = 147 kips

The ratio W_1/W_T (= 0.75) was determined from AWWA D100 (it depends on the ratio of height to diameter)

$W_i = W_{tank} + W_{1water}$ = 15.4 + 147 = 162.4 kips

12.6.1.3.4 Base Shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_i = \frac{S_{ai}W_i}{R} = \frac{0.824(162.4\text{ kips})}{3} = 44.6\text{ kips}$$

12.6.1.4 Calculations for Convective Response Natural Period for the First Mode of Sloshing

12.6.1.4.1 Natural Period for the First Mode of Sloshing

Using *Provisions* Section 14.7.3.6.1 [14.4.7.5.1]:

$$T_c = 2\pi\sqrt{\frac{D}{3.68g\tanh\left(\dfrac{3.68H}{D}\right)}} = 2\pi\sqrt{\frac{20\text{ ft}}{3.68\left(32.174\,\frac{\text{ft}}{\text{s}^2}\right)\tanh\left(\dfrac{3.68(10\text{ ft})}{10\text{ ft}}\right)}} = 2.58\text{ s}$$

12.6.1.4.2 Spectral Acceleration

Using *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1] with T_c < 4 seconds:

$$S_{ac} = \frac{1.5S_{D1}}{T_c} = \frac{1.5(0.376)}{2.58} = 0.219$$

12.6.1.4.3 Seismic (Convective) Weight

$W_c = W_{water}$ (W_2/W_T) = 196 (0.30) = 58.8 kips

The ratio W_2/W_T (= 0.30) was determined from AWWA D100.

12.6.1.4.4 Base Shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_c = \frac{S_{ac}W_c}{R} = \frac{0.219(58.8\text{ kips})}{3} = 4.29\text{ kips}$$

12.6.1.5 Design Base Shear

Although Item b of *Provisions* Sec. 14.7.3.2 [14.4.7.1] indicates that impulsive and convective components may, in general, be combined using the SRSS method, *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1] requires that the direct sum be used for ground-supported storage tanks for liquids. Using *Provisions* Eq. 14.7.3.6.1 [14.4-1]:

$V = V_i + V_c = 44.6 + 4.29 = 48.9$ kips

[In the 2003 *Provisions*, use of the SRSS method is also permitted for ground-supported storage tanks for liquids.]

12.6.2 FLAT-BOTTOM GASOLINE TANK

12.6.2.1 Description

This example illustrates the calculation of the base shear and the required freeboard using the ELF procedure for a petro-chemical storage tank in a refinery tank farm near a populated city neighborhood. An impoundment dike is not provided to control liquid spills.

The tank is a flat-bottom, ground-supported, anchored, bolted steel tank constructed in accordance with API 650. The weight of the tank shell, roof, and equipment is 15,400 lb.

12.6.2.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	=	III
(The tank is used for storage of hazardous material.)		
Importance Factor, I (*Provisions* Sec. 14.5.1.2 [14.2.1])	=	1.5
Site Coordinates	=	48.000° N, 122.250° W
Short Period Response, S_S	=	1.236
One Second Period Response, S_1	=	0.406
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	C (per geotech)
Acceleration-based Site Coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.0
Velocity-based Site Coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.39

Design spectral acceleration response parameters

$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.0)(1.236)$	=	0.824
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.39)(0.406)$	=	0.376

Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-3])	=	Flat-bottom, ground-supported, anchored, bolted steel tank
Response Modification Coefficient, R (*Provisions* Table 14.5.1.1 [14.2-3])	=	3
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [14.2-3])	=	2
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [14.2-3])	=	2½

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

12.6.2.3 Calculations for Impulsive Response

12.6.2.3.1 Natural Period for the First Mode of Vibration

Based on analysis, the period for impulsive response of the tank and its contents is T_i = 0.14 sec.

12.6.2.3.2 Spectral Acceleration

Based on *Provisions* Figure 14.7.3.6-1 [14.4-1]:

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.376}{0.824} = 0.456 \text{ seconds}$$

Using *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1] with $T_i < T_s$,:

$S_{ai} = S_{DS} = 0.824$

12.6.2.3.3 Seismic (Impulsive) Weight

W_{tank} = 15.4 kips

$W_{Gas} = \pi(10\text{ ft})^2(10\text{ ft})(0.046\text{ kip/ft}^3)(W_1/W_T)$ = 144.5 kips (0.75) = 108.4 kips

Note: The ratio W_1/W_T was determined from AWWA D100, but API 650 should be used.

$W_i = W_{tank} + W_{Gas}$ = 15.4 + 108.4 = 123.8 kips

12.6.2.3.4 Base Shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_i = \frac{S_{ai} I W_i}{R} = \frac{0.824(1.5)(123.8\text{ kips})}{3} = 51.0\text{ kips}$$

12.6.2.4 Calculations for Convective Response

12.6.2.4.1 Natural Period for the First Mode of Sloshing

The dimensions are the same as those used for the water tank in Sec. 12.6.1; therefore, T_c = 2.58 sec.

12.6.2.4.2 Spectral Acceleration

Likewise, S_{ac} = 0.219.

12.6.2.4.3 Seismic (Convective) Weight

$$W_c = W_{LNG}\,(W_2/W_T) = 144.5\,(0.30) = 43.4 \text{ kips}$$

The ratio W_2/W_T was determined from AWWA D100.

12.6.2.4.4 Base shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_c = \frac{S_{ac} I W_c}{R} = \frac{0.824(1.5)(43.5 \text{ kips})}{3} = 17.9 \text{ kips}$$

12.6.2.5 Design Base Shear

Using *Provisions* Eq. 14.7.3.6.1 [14.4-1]:

$$V = V_i + V_c = 51.0 + 17.9 = 68.9 \text{ kips}$$

12.6.2.6 Minimum Freeboard

Provisions Table 14.7.3.6.1.2 [14.4-2] indicates that a minimum freeboard equal to δ_s is required for this tank. Using *Provisions* Eq. 14.7.3.6.1.2 [14.4-9]:

$$\delta_s = 0.5DIS_{ac} = 0.5(20 \text{ ft})(1.5)(0.219) = 3.29 \text{ ft}$$

The 5 ft freeboard provided is adequate.

12.7 EMERGENCY ELECTRIC POWER SUBSTATION STRUCTURE, ASHPORT, TENNESSEE

The main section addressing electrical transmission, substation, and distribution structures is in the appendix to Chapter 14 of the *Provisions*. The information is in an appendix so that designers can take time to evaluate and comment on the seismic design procedures before they are included in the main text of the *Provisions*.

[In the 2003 *Provisions* Sections A14.2.1 and A14.2.2 were removed because the appropriate industry standards had been updated to include seismic design criteria and earthquake ground motions consistent with the *Provisions*. Therefore, all references to the *Provisions* in Sec. 12.7 of this chapter are obsolete.]

12.7.1 Description

This example illustrates the calculation of the base shear using the ELF procedure for a braced frame that supports a large transformer (Figure 12-7). The substation is intended to provide emergency electric power to the emergency control center for the fire and police departments of a community. There is only one center designed for this purpose.

The weight of the transformer equipment is 17,300 lb.

The weight of the support structure is 12,400 lb.

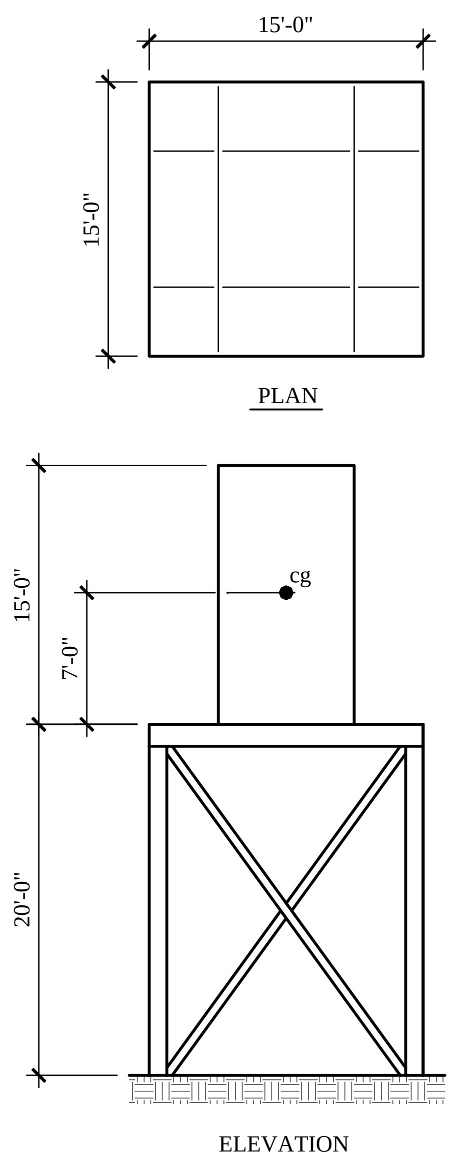


Figure 12-7 Platform for elevated transformer (1.0 ft = 0.3048 m).

The period of the structure is T = 0.240 sec.

Although the ratio of the supported structure over the total weight is greater than 25 percent, experience indicates that the transformer will behave as a lumped rigid mass.

12.7.2 *Provisions* Parameters

12.7.2.1 Ground Motion

The design response spectral accelerations are defined as

S_{DS} = 1.86
S_{D1} = 0.79

12.7.2.2 Seismic Use Group and Importance Factor

The structure is for emergency electric power for a Seismic Use Group III facility. Therefore, the platform is assigned to Seismic Use Group III, as required by *Provisions* Sec. 1.3 [1.2]. Using *Provisions* Table 14.5.1.2 [14.2-1], the Importance Factor, *I*, is equal to 1.5.

12.7.2.3 Response Modification Coefficient

From *Provisions* Table 14A.2.1, *R* is 3.

12.7.3 Design of the System

12.7.3.1 Seismic Response Coefficient

Provisions Sec. 14A.2.2 defines C_s in a manner that is not consistent with the rest of the *Provisions*. This inconsistency will be eliminated in future editions of the *Provisions*. In this example, the equations are applied in a manner that is consistent with Chapters 5 [4 and 5] and 14 – that is, *R* is applied in the calculation of C_s rather than in the calculation of *V*.

Using *Provisions* Section 14A.2.2:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.86}{3/1.5} = 0.93$$

but C_s need not be larger than:

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.79}{0.24\left(3/1.5\right)} = 1.646$$

Therefore, C_s = 0.93.

12.7.3.2 Seismic Weight

$W = W_{Transformer} + W_{Support\ structure}$ = 17.3 + 12.4 = 29.7 kips

12.7.3.3 Base Shear

Using *Provisions* Section 14A.2.2: $V = C_sW$ = 0.93(29.7 kips) = 27.6 kips

13

DESIGN FOR NONSTRUCTURAL COMPONENTS

Robert Bachman, P.E. and Richard Drake, P.E.

Chapter 6 of the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereinafter, the *Provisions* and *Commentary*) addresses architectural, mechanical, and electrical components of buildings. Two examples are presented here to illustrate many of the requirements and procedures. Design and anchorage are illustrated for exterior precast concrete cladding, and for a roof-mounted HVAC unit. The rooftop unit is examined in two common installations: directly attached, and isolated with snubbers. This chapter also contains an explanation of the fundamental aspects of the *Provisions*, and an explanation of how piping, designed according to the ASME Power Piping code, is checked for the force and displacement requirements of the *Provisions*.

A large variety of materials and industries are involved with nonstructural components is large, and numerous documents define and describe methods of design, construction, manufacture, installation, attachment, etc. Some of the documents address seismic issues but many do not. *Provisions* Sec. 6.1.1 [6.1.2] contains a listing of approved standards for various nonstructural components.

Although the *Guide* is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

A few noteworthy changes were made to the nonstructural components requirements of the 2003 *Provisions*. These include explicit definition of load effects (including vertical seismic forces) within the chapter and revised classification of nonductile anchors (based on demonstrated ductility or prequalification rather than embedment-length-to-diameter ratio).

In addition to changes *Provisions* Chapter 6, the basic earthquake hazard maps were updated and the concrete design reference was updated to ACI 318-02 (with a significant resulting changes to the calculations for anchors in concrete).

Where they affect the design examples in this chapter of the *Guide*, significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the *Provisions*, the following are referenced in this chapter:

ACI 318	American Concrete Institute. 1999 [2002]. *Building Code Requirements and Commentary for Reinforced Concrete*.
ASCE 7	American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures*.
ASHRAE APP IP	American Society of Heating, Refrigeration, and Air-Conditioning Engineers (ASHRAE). 1999. *Seismic and Wind Restraint Design*, Chapter 53.
ASME B31.1	American Society of Mechanical Engineers. *Power Piping Code*.
IBC	International Code Council. 2000. *International Building Code*.

The symbols used in this chapter are drawn from Chapter 2 of the *Provisions* or reflect common engineering usage. The examples are presented in U.S. customary units.

[In the 2003 *Provisions*, definitions and symbols specific to nonstructural components appear in Sec. 6.1.3 and 6.1.4, respectively.]

13.1 DEVELOPMENT AND BACKGROUND OF THE PROVISIONS FOR NONSTRUCTURAL COMPONENTS

13.1.1 Approach to Nonstructural Components

The *Provisions* requires that nonstructural components be checked for two fundamentally different demands placed upon them by the response of the structure to earthquake ground motion: resistance to inertial forces and accommodation of imposed displacements. Building codes have long had requirements for resistance to inertial forces. Most such requirements apply to the component mass an acceleration that vary with the basic ground motion parameter and a few broad categories of components. The broad categories are intended distinguish between components whose dynamic response couples with that of the supporting structure in such a fashion as to cause the component response accelerations to be amplified above the accelerations of the structure and those components that are rigid enough with respect to the structure so that the component response is not amplified over the structural response. In recent years, a coefficient based on the function of the building or of the component have been introduced as another multiplier for components important to life safety or essential facilities.

The *Provisions* includes an equation to compute the inertial force that involves two additional concepts: variation of the acceleration with relative height within the structure, and reduction in design force based upon available ductility in the component, or its attachment. The *Provisions* also includes a quantitative measure for the deformation imposed upon nonstructural components. The inertial force demands tend to control the seismic design for isolated or heavy components, whereas, the imposed deformations are important for the seismic design for elements that are continuous through multiple levels of a structure, or across expansion joints between adjacent structures, such as cladding or piping.

The remaining portions of this section describe the sequence of steps and decisions prescribed by the *Provisions* to check these two seismic demands on nonstructural components.

13.1.2 Force Equations

The following seismic force equations are prescribed for nonstructural components:(*Provisions* Eq. 6.1.3-1 [6.2-1], 6.1.3-2 [6.2-3], and 6.1.3-3 [6.2-4]):

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right)$$

$$F_{p_{max}} = 1.6 S_{DS} I_p W_p$$

$$F_{p_{min}} = 0.3 S_{DS} I_p W_p$$

where:

F_p = horizontal equivalent static seismic design force centered at the component's center of gravity and distributed relative to the component's mass distribution.

a_p = component amplification factor (either 1.0 or 2.5) as tabulated in *Provisions* Table 6.2.2 [6.3-1] for architectural components and *Provisions* Table 6.3.2 [6.4-1] for mechanical and electrical components (Alternatively, may be computed by dynamic analysis)

S_{DS} = five percent damped spectral response acceleration parameter at short period as defined in *Provisions* Sec. 4.1.2 [3.3.3]

W_p = component operating weight

R_p = component response modification factor (varies from 1.0 to 5.0) as tabulated in *Provisions* Table 6.2.2 [6.3-1]for architectural components and *Provisions* Table 6.3.2 [6.4-1] for mechanical and electrical components

I_p = component importance factor (either 1.0 or 1.5) as indicated in *Provisions* Sec. 6.1.5 [6.2-2]

z = elevation in structure of component point of attachment relative to the base

h = roof elevation of the structure or elevation of highest point of the seismic-force-resisting system of the structure relative to the base

The seismic design force, F_p, is to be applied independently in the longitudinal, and transverse directions. The effects of these loads on the component are combined with the effects of static loads. *Provisions* Eq. 6.1.3-2 [6.2-3 and 6.2-4] and 6.1.3-3, provide maximum and minimum limits for the seismic design force.

For each point of attachment, a force, F_p, should be determined based on *Provisions* Eq. 6.1.3-1 [6.2-1]. The minima and maxima determined from *Provisions* Eq. 6.1.3-2 and 6.1.3-3 [6.2-1] must be considered in determining each F_p. The weight, W_p, used to determine each F_p should be based on the tributary weight of the component associated with the point of attachment. For designing the component, the attachment force, F_p, should be distributed relative to the component's mass distribution over the area used to establish the tributary weight. With the exception of the bearing walls, which are covered by *Provisions* Sec. 5.2.6.2.7 [4.6.1.3], and out-of-plane wall anchorage to flexible diaphragms, which is covered by *Provisions* Sec. 5.2.6.3.2 [4.6.2.1], each anchorage force should be based on simple statics determined by using all the distributed loads applied to the complete component. Cantilever parapets that are part of a continuous element, should be separately checked for parapet forces.

13.1.3 Load Combinations and Acceptance Criteria

13.1.3.1 Seismic Load Effects

When the effects of vertical gravity loads and horizontal earthquake loads are additive, *Provisions* Eq. 5.2.7.1-1 [4.2-1] is used:

$$E = \rho Q_E + 0.2 S_{DS} D$$

When the effects of vertical gravity load counteract those of horizontal earthquake loads, *Provisions* Eq. 5.2.7.1-2 [4.2-2] is used:

$$E = \rho Q_E - 0.2 S_{DS} D$$

where:

E = effect of horizontal and vertical earthquake-induced forces

ρ = redundancy factor (= 1.0 for nonstructural components)

Q_E = effect of horizontal seismic forces (due to application of F_p for nonstructural components)

D = effect of dead load

$0.2S_{DS}D$ = effect of vertical seismic forces

13.1.3.2 Strength Load Combinations

Provisions Sec. 5.2.7 [4.2.2] requires the use of ASCE 7 factored load combinations. The combinations from ASCE 7 Sec. 2.3.2 that include earthquake effects are:

$$U = 1.2D + 1.0E + 0.5L + 0.2S$$

$$U = 0.9D + 1.0E + 1.6H$$

13.1.4 Component Amplification Factor

The component amplification factor, a_p, found in *Provisions* Eq. 6.1.3-1 [6.2-1] represents the dynamic amplification of the component relative to the maximum acceleration of the component support point(s). Typically, this amplification is a function of the fundamental period of the component, T_p, and the fundamental period of the support structure, T. It is recognized that at the time the components are designed or selected, the effective fundamental period of the structure, T, is not always available. It is also recognized that for a majority of nonstructural components, the component fundamental period, T_p, can be accurately obtained only by expensive shake-table or pullback tests. As a result, the determination of a component's fundamental period by dynamic analysis, considering T/T_p ratios, is not always practicable. For this reason, acceptable values of a_p have been provided in the *Provisions* tables. Therefore, component amplification factors from either these tables or a dynamic analysis may be used. Values for a_p are tabulated for each component based on the expectation that the component will behave in either a rigid or a flexible manner. For simplicity, a step function increase based on input motion amplifications is provided to help distinguish between rigid and flexible behavior. If the fundamental period of the component is less than 0.06 seconds, no dynamic amplification is expected and a_p may be taken to equal 1.00. If the fundamental period of the component is greater than 0.06 seconds, dynamic amplification is expected, and a_p is taken to equal 2.50. In addition, a rational analysis determination of

a_p is permitted if reasonable values of both T and T_p are available. Acceptable procedures for determining a_p are provided in *Commentary* Chapter 6.

13.1.5 Seismic Coefficient at Grade

The short period design spectral acceleration, S_{DS}, considers the site seismicity and local soil conditions. The site seismicity is obtained from the design value maps (or CD-ROM), and S_{DS} is determined in accordance with *Provisions* Sec. 4.1.2.5 [3.3.3]. The coefficient S_{DS} is the used to design the structure. The *Provisions* approximates the effective peak ground acceleration as $0.4S_{DS}$, which is why 0.4 appears in *Provisions* Eq. 6.1.3-1 [6.2-1].

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

13.1.6 Relative Location Factor

The relative location factor, $\left(1+2\frac{z}{h}\right)$, scales the seismic coefficient at grade, resulting in values linearly varying from 1.0 at grade to 3.0 at roof level. This factor approximates the dynamic amplification of ground acceleration by the supporting structure.

13.1.7 Component Response Modification Factor

The component response modification factor, R_p, represents the energy absorption capability of the component's construction and attachments. In the absence of applicable research, these factors are based on judgment with respect to the following benchmark values:

1. R_p = 1.0 or 1.5, brittle or buckling failure mode is expected
2. R_p = 2.5, some minimal level of energy dissipation capacity
3. R_p = 3.5 or 5.0, highly ductile materials and detailing

13.1.8 Component Importance Factor

The component importance factor, I_p, represents the greater of the life safety importance and/or the hazard exposure importance of the component. The factor indirectly accounts for the functionality of the component or structure by requiring design for a lesser amount of inelastic behavior (or higher force level). It is assumed that a lesser amount of inelastic behavior will result in a component that will have a higher likelihood of functioning after a major earthquake.

13.1.9 Accommodation of Seismic Relative Displacements

The *Provisions* requires that seismic relative displacements, D_p, be determined in accordance with several equations. For two connection points on Structure A (or on the same structural system), one at Level x and the other at Level y, D_p is determined from *Provisions* Eq. 6.1.4-1 [6.2-5] as:

$$D_p = \delta_{xA} - \delta_{yA}$$

Because the computed displacements are frequently not available to the designer of nonstructural components, one may use the maximum permissible structural displacements per *Provisions* Eq. 6.1.4-2 [6.2-6]:

$$D_{p_{max}} = (X - Y)\frac{\Delta_{aA}}{h_{sx}}$$

For two connection points on Structures A and B (or on two separate structural systems), one at Level *x*, and the other at Level *y*, D_P and D_{Pmax} are determined from *Provisions* Eq. 6.1.4-3 [6.2-7] and 6.1.4-4 [6.2-8], respectively, as:

$$D_p = |\delta_{xA}| + |\delta_{yB}|$$

$$D_{p_{max}} = \frac{X\Delta_{aA}}{h_{sx}} + \frac{Y\Delta_{aB}}{h_{sx}}$$

where:

D_p = seismic relative displacement that the component must be designed to accommodate

δ_{xA} = deflection of building Level *x* of Structure A, determined by an elastic analysis as defined in *Provisions* Sec. 5.4.6.1 or 5.5.5 [5.2.6, 5.3.5, or 5.4.3] and multiplied by the C_d factor

δ_{yA} = deflection of building Level *y* of Structure A, determined in the same fashion as δ_{xA}

X = height of upper support attachment at Level *x* as measured from the base

Y = height of lower support attachment at Level *y* as measured from the base

Δ_{aA} = allowable story drift for Structure A as defined in *Provisions* Table 5.2.8 [4.5-1]

h_{sx} = story height used in the definition of the allowable drift, Δ_a, in *Provisions* Table 5.2.8 [4.5-1]

δ_{yB} = deflection of building Level *y* of Structure B, determined in the same fashion as δ_{xA}

Δ_{aB} = allowable story drift for Structure B as defined in *Provisions* Table 5.2.8 [4.5-1]

The effects of seismic relative displacements must be considered in combination with displacements caused by other loads as appropriate. Specific methods for evaluating seismic relative displacement effects of components and associated acceptance criteria are not specified in the *Provisions*. However, the intention is to satisfy the purpose of the *Provisions*. Therefore, for nonessential facilities, nonstructural components can experience serious damage during the design level earthquake provided they do not constitute a serious life safety hazard. For essential facilities, nonstructural components can experience some damage or inelastic deformation during the design level earthquake provided they do not significantly impair the function of the facility.

13.1.10 Component Anchorage Factors and Acceptance Criteria

Design seismic forces in the connected parts, F_p, are prescribed in *Provisions* Sec. 6.1.3 [6.2.6].

When component anchorage is provided by expansion anchors or shallow anchors, a value of R_p = 1.5 is used. Shallow anchors are defined as those with embedment length-to-diameter ratios of less than 8. Anchors embedded in concrete or masonry are proportioned to carry the least of the following:

1. The design strength of the connected part

2. 1.3 times the prescribed seismic design force or
3. The maximum force that can be transferred to the connected part by the component structural system

Determination of design seismic forces in anchors must consider installation eccentricities, prying effects, multiple anchor effects, and the stiffness of the connected system.

Use of powder-driven fasteners is not permitted for seismic design tension forces in Seismic Design Categories D, E, and F unless approved for such loading.

[In the 2003 *Provisions* reference is made to "power-actuated" fasteners so as to cover a broader range of fastener types than is implied by "powder-driven."]

The design strength of anchors in concrete is determined in accordance with, *Provisions* Chapter 9, which is basically the same as IBC Sec. 1913, and has been updated somewhat in Appendix D of ACI 318-2002. (These rules for anchors in concrete will probably be deleted from the next edition of the *Provisions* in favor of a reference to ACI 318.)

[The 2003 *Provisions* refer to Appendix D of ACI 318-02 rather than providing specific, detailed requirements.]

13.1.11 Construction Documents

Construction documents must be prepared by a registered design professional and must include sufficient detail for use by the owner, building officials, contractors, and special inspectors; *Provisions* Table 6.1.7 [6.2-1] includes specific requirements.

13.2 ARCHITECTURAL CONCRETE WALL PANEL

13.2.1 Example Description

In this example, the architectural components are a 4.5-in.-thick precast normal weight concrete spandrel panel and a column cover supported by the structural steel frame of a five-story building as shown in Figures 13.2-1 and 13.2-2.

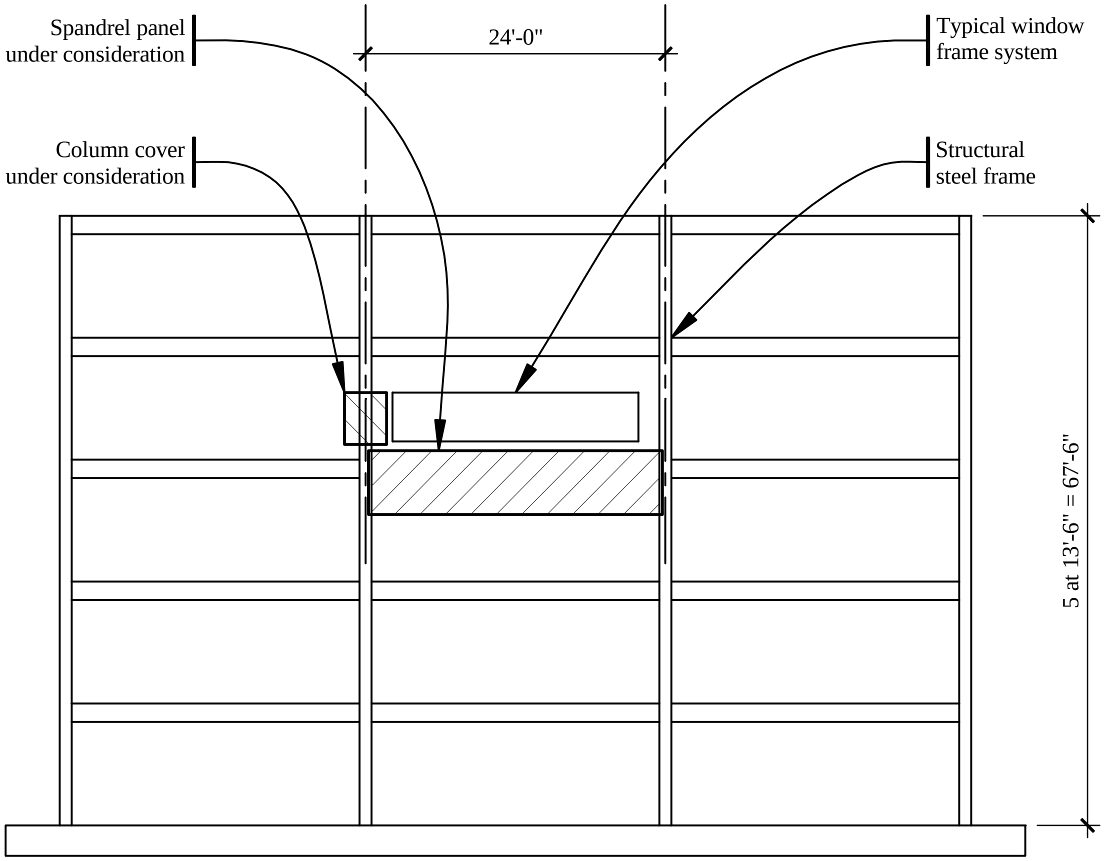


Figure 13.2-1 Five-story building elevation showing panel location (1.0 ft = 0.3048 m)

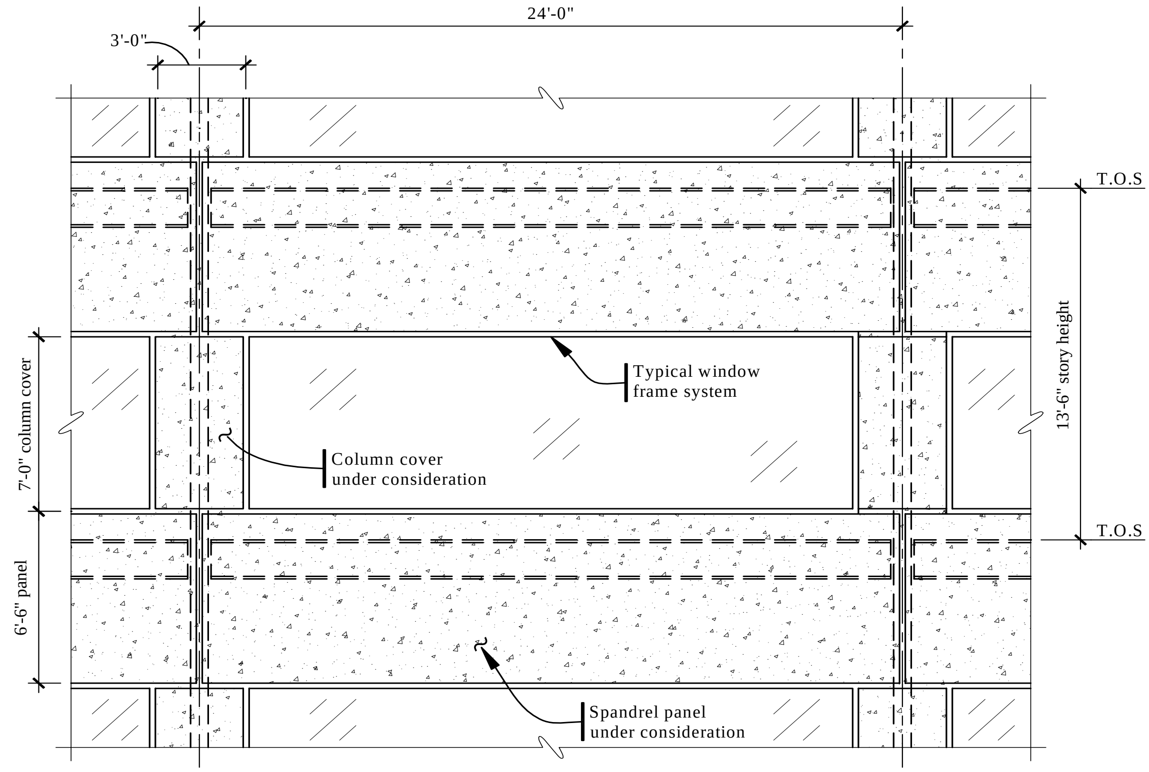


Figure 13.2-2 Detailed building elevation (1.0 ft = 0.3048 m).

The columns, at the third level of the five-story office building, support the spandrel panel under consideration. The columns between the third and fourth levels of the building support the column cover under consideration. The building, located near a significant active fault in Los Angeles, California, is assigned to Seismic Use Group I. Wind pressures normal to the building are 17 psf determined in accordance with ASCE 7. The spandrel panel supports glass windows weighing 10 psf.

This example develops prescribed seismic forces for the selected spandrel panel and prescribed seismic displacements for the selected column cover.

It should be noted that details of precast connections vary according to the preferences and local practices of the precast panel supplier. In addition, some connections may involve patented designs. As a result, this example will concentrate on quantifying the prescribed seismic forces and displacements. After the prescribed seismic forces and displacements are determined, the connections can be detailed and designed according to the appropriate AISC and ACI codes and PCI (Precast/Prestressed Concrete Institute) recommendations.

13.2.2 Design Requirements

13.2.2.1 *Provisions* Parameters and Coefficients

a_p = 1.0 for wall panels (*Provisions* Table 6.2.2 [6.3-1])

a_p = 1.25 for connection fasteners (*Provisions* Table 6.2.2 [6.3-1])

S_{DS} = 1.487 (Design Values CD-ROM for the selected location and site class)

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

Seismic Design Category = D (*Provisions* Table 4.2.1a [1.4-1] for S_1 < 0.75)

[In the footnote to 2003 *Provisions* Table 1.4-1, the value of S_1 used to trigger assignment to Seismic Design Category E or F was changed from 0.75 to 0.6.]

Spandrel Panel W_p = (150 lb/ft^3)(24 ft)(6.5 ft)(0.375 ft) = 8775 lb

Glass W_p = (10 lb/ft^2)(21 ft)(7 ft) = 1470 lb (supported by spandrel panel)

Column Cover W_p = (150 lb/ft^3)(3 ft)(7 ft)(0.375 ft) = 1181 lb

R_p = 2.5 for wall panels (*Provisions* Table 6.3.2 [6.3-1])

R_p = 1.0 for connection fasteners ((*Provisions* Sec. 6.1.6.1 [6.2.8.1])

[In the 2003 *Provisions* component anchorage is designed using R_p = 1.5 unless specific ductility or prequalification requirements are satisfied.]

I_p = 1.0 (*Provisions* Sec. 6.1.5 [6.2.2])

$$\frac{z}{h} = \frac{40.5 \text{ ft}}{67.5 \text{ ft}} = 0.6$$ (at third floor)

ρ = 1.0 (*Provisions* Sec. 6.1.3)

[The 2003 *Provisions* indicate that the redundancy factor does not apply to the design of nonstructural components. Although the effect is similar to stating that ρ = 1, there is a real difference since load effects for such components and their supports and attachments are now defined in Chapter 6 rather than by reference to Chapter 4.]

13.2.2.2 Performance Criteria

Component failure should not cause failure of an essential architectural, mechanical, or electrical component (*Provisions* Sec. 6.1 [6.2.3]).

Component seismic attachments must be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (*Provisions* Sec. 6.1.2 [6.2.5]).

The effects of seismic relative displacements must be considered in combination with displacements caused by other loads as appropriate (*Provisions* Sec. 6.1.4 [6.2.7]).

Exterior nonstructural wall panels that are attached to or enclose the structure shall be designed to resist the forces in accordance with *Provisions* Eq. 6.1.3-1 or 6.1.3-2 [6.2.6] and must be able to accommodate movements of the structure resulting from response to the design basis ground motion, D_p, or temperature changes (*Provisions* Sec. 6.2.4 [6.3.2]).

13.2.3 Spandrel Panel

13.2.3.1 Connection Details

Figure 13.2-3 shows the types and locations of connections that support one spandrel panel.

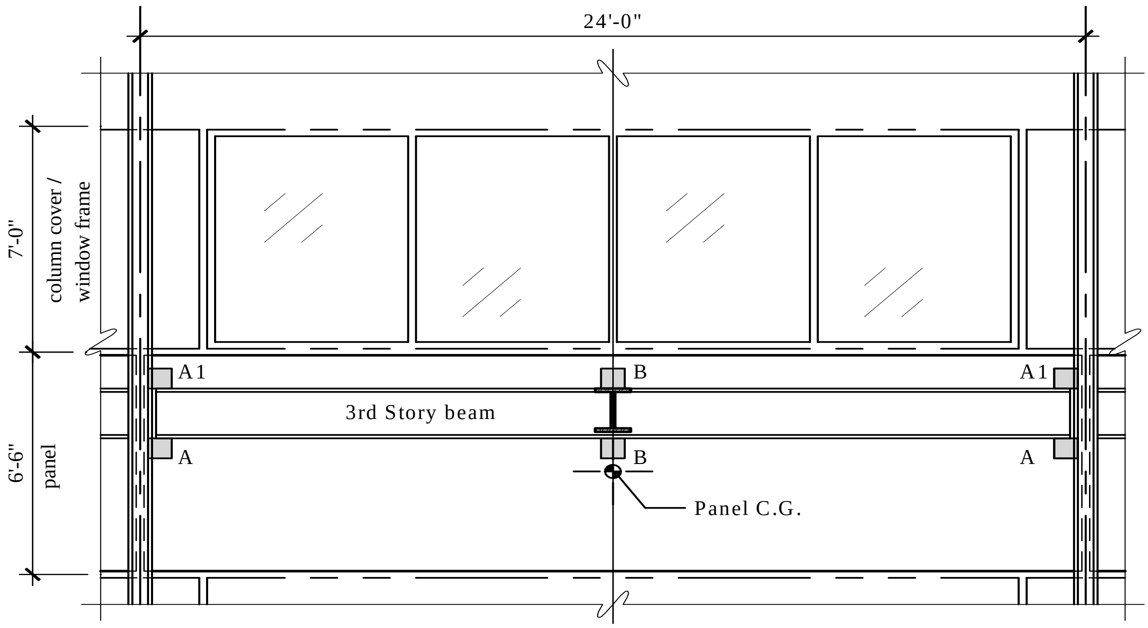


Figure 13.2-3 Spandrel panel connection layout from interior (1.0 ft = 0.3048 m).

The connection system must resist the weight of the panel and supported construction including the eccentricity between that load and the supports as well as inertial forces generated by response to the seismic motions in all three dimensions. Furthermore, the connection system must not create undue interaction between the structural frame and the panel, such as restraint of the natural shrinkage of the panel or the transfer of floor live load from the beam to the panel. The panels are usually very stiff compared to the frame, and this requires careful release of potential constraints at connections. PCI's *Architectural Precast Concrete* (2nd Ed. 1989), provides an extended discussion of important design concepts for such panels.

For this example, the basic gravity load, and vertical accelerations are resisted at Points A, which provides the recommended simple and statically determinant system for the main gravity weight. The eccentricity of vertical loads is resisted by a force couple at the two pairs of A1 and A connections. Horizontal loads parallel to the panel are resisted by the A connections. Horizontal loads perpendicular to the panel are resisted by all six connections. The A connections, therefore, restrain movement in three dimensions while the A1 and B connections restrain movement in only one dimension, perpendicular to the panel. Connection components can be designed to resolve some eccentricities by bending of the element; for example, the eccentricity of the horizontal in-plane force with the structural frame can be resisted by bending the A connection.

The practice of resisting the horizontal in-plane force at two points varies with seismic demand and local industry practice. The option is to resist all of the in-plane horizontal force at one connection in order to avoid restraint of panel shrinkage. The choice made here depends on local experience indicating that precast panels of this length have been restrained at the two ends without undue shrinkage restraint problems.

The A and A1 connections are often designed to take the loads directly to the columns, particularly on steel moment frames where attachments to the flexural hinging regions of beams are difficult to accomplish. The lower B connection often require an intersecting beam to provide sufficient stiffness and strength to resist the loads.

The column cover is supported both vertically and horizontally by the column, transfers no loads to the spandrel panel, and provides no support for the window frame.

The window frame is supported both vertically and horizontally along the length of the spandrel panel and transfers no loads to the column covers.

13.2.3.2 Prescribed Seismic Forces

Lateral forces on the wall panels and connection fasteners include seismic loads in accordance with the *Provisions* and wind loads in accordance with ASCE 7 as indicated in the problem statement. Wind forces are not illustrated here.

13.2.3.2.1 Panels

$D = W_p$ = 8775 lb + 1470 lb = 10245 lb (vertical gravity effect)

$$F_p = \frac{0.4(1.0)(1.487)(10245\text{ lb})}{\left(2.5/1.0\right)}\left(1+2(0.6)\right) = 5362\text{ lb}$$ (*Provisions* Eq. 6.1.3-1 [6.2-1])

$F_{p_{max}}$ = 1.6(1.487)(1.0)(10245 lb) = 24375 lb (*Provisions* Eq. 6.1.3-2 [6.2-3])

$F_{p_{min}}$ = 0.3(1.487)(1.0)(10245 lb) = 4570 lb (*Provisions* Eq. 6.1.3-3 [6.2-4])

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following five steps would be cast differently; the result is the same.]

Q_E (due to application of F_p) = 5362 lb (*Provisions* Sec. 6.1.3)

ρQ_E = (1.0)(5362 lb) = 5362 lb (horizontal earthquake effect)

$0.2S_{DS}D$ = (0.2)(1.487)(10245 lb) = 3047 lb (vertical earthquake effect)

$E = \rho Q_E + 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-2 [4.2-2])

13.2.3.2.2 Connection Fasteners

The *Provisions* specifies a reduced R_P, and an increased a_P for "Fasteners," which is intended to prevent premature failure in those elements of connections that are inherently brittle, such as embedments that depend on concrete breakout strength, or are simply too small to adequately dissipate energy inelastically, such as welds or bolts. The net effect more than triples the design seismic force.

$$F_p = \frac{0.4(1.25)(1.487)(10245\text{ lb})}{\left(1.0/1.0\right)}\left(1+2(0.6)\right) = 16757\text{ lb}$$ (*Provisions* Eq. 6.1.3-1 [6.2-1])

$F_{p_{max}} = 1.6(1.487)(1.0)(10245 \text{ lb}) = 24375 \text{ lb}$ (*Provisions* Eq. 6.1.3-2 [6.2-3])

$F_{p_{min}} = 0.3(1.487)(1.0)(10245 \text{ lb}) = 4570 \text{ lb}$ (*Provisions* Eq. 6.1.3-3 [6.2-4])

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following five steps would be cast differently; the result is the same.]

Q_E (due to application of F_p) = 16757 lb (*Provisions* Sec. 6.1.3)

$\rho Q_E = (1.0)(16757 \text{ lb}) = 16757 \text{ lb}$ (horizontal earthquake effect)

$0.2S_{DS}D = (0.2)(1.487)(10245 \text{ lb}) = 3047 \text{ lb}$ (vertical earthquake effect)

$E = \rho Q_E + 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-2 [4.2-2])

13.2.3.3 Proportioning and Design

13.2.3.3.1 Panels

The wall panels should be designed for the following loads in accordance with ACI 318. The design of the reinforced concrete panel is standard and is not illustrated in this example. Spandrel panel moments are shown in Figure 13.2-4. Reaction shears (V_u), forces (H_u), and moments (M_u) are calculated for applicable strength load combinations.

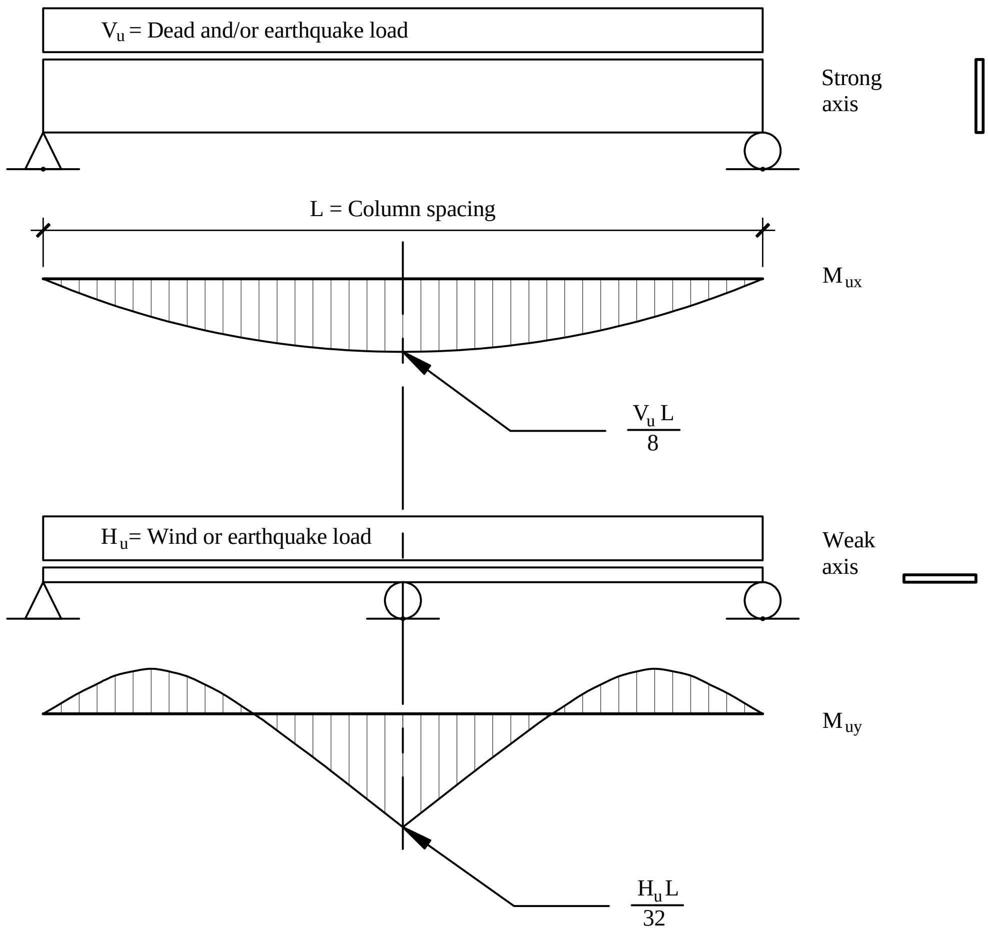


Figure 13.2-4 Spandrel panel moments.

U = 1.4D

$V_u = 1.4(10245 \text{ lb}) = 14343 \text{ lb}$ (vertical load downward)

$$M_{ux} = \frac{(14343 \text{ lb})(24 \text{ ft})}{8} = 43029 \text{ ft-lb}$$ (strong axis moment)

U = 1.2D + 1.0E

$V_{u_{max}} = 1.2(10245 \text{ lb}) + 1.0(3047 \text{ lb}) = 15341 \text{ lb}$ (vertical load downward)

$\Leftrightarrow H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load parallel to panel)

$\perp H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load perpendicular to panel)

$$M_{ux_{max}} = \frac{(15341 \text{ lb})(24 \text{ ft})}{8} = 46023 \text{ ft-lb}$$ (strong axis moment)

$$M_{uy} = \frac{(5362 \text{ lb})(24 \text{ ft})}{32} = 4022 \text{ ft-lb}$$ (weak axis moment)

U = 0.9D + 1.0E

$V_{u_{min}} = 0.9(10245 \text{ lb}) - 1.0(3047 \text{ lb}) = 6174 \text{ lb}$ (vertical load downward)

$\Leftrightarrow H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load parallel to panel)

$\perp H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load perpendicular to panel)

$$M_{ux_{min}} = \frac{(6174 \text{ lb})(24 \text{ ft})}{8} = 18522 \text{ ft-lb}$$ (strong axis moment)

$$M_{uy} = \frac{(5362 \text{ lb})(24 \text{ ft})}{32} = 4022 \text{ ft-lb}$$ (weak axis moment)

13.2.3.3.2 Connection Fasteners

The connection fasteners should be designed for the following loads in accordance with ACI 318-2002 (Appendix D) and the AISC specification. There are special reduction factors for anchorage in high seismic demand locations, and the parameters for this project would invoke those reduction factors. The design of the connection fasteners is not illustrated in this example. Spandrel panel connection forces are shown in Figure 13.2-5. Reaction shears (V_u), forces (H_u), and moments (M_u) are calculated for applicable strength load combinations.

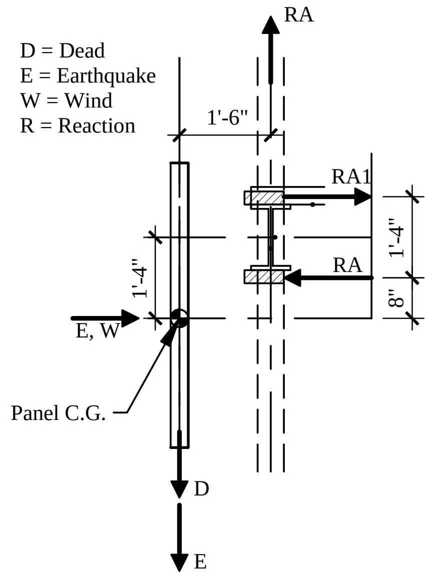


Figure 13.2-5 Spandrel panel connection forces.

U = 1.4D

$$V_{uA} = \frac{1.4(10245\ \text{lb})}{2} = 7172\ \text{lb}$$ (vertical load downward at Point A and A1)

M_{uA} = (7172 lb)(1.5 ft) = 10758 ft-lb (moment resisted by paired Points A and A1)

Horizontal couple from moment at A and A1 = 10758 / 1.33 = 8071 lb

U = 1.2D + 1.0E

$$V_{uA_{max}} = \frac{1.2(10245\ \text{lb}) + 1.0(3047\ \text{lb})}{2} = 7671\ \text{lb}$$ (vertical load downward at Point A)

$$\perp H_{uA} = 1.0(16757\ \text{lb})\frac{3}{16} = 3142\ \text{lb}$$ (horizontal load perpendicular to panel at Points A and A1)

H_{Ain} = (7671 lb)(1.5 ft) / (1.33 ft) + (3142 lb)(2.0 ft) / (1.33 ft) = 13366 lb (inward force at Point A)

H_{A1out} = (7671 lb)(1.5 ft) / (1.33 ft) + (3142 lb)(0.67) / (1.33 ft) = 10222 lb(outward force at Point A1)

$$\Leftrightarrow H_{uA} = \frac{1.0(16757\ \text{lb})}{2} = 8378\ \text{lb}$$ (horizontal load parallel to panel at Point A)

M_{u2A} = (8378 lb)(1.5 ft) = 12568 ft-lb (flexural moment at Point A)

$$\perp H_{uB} = 1.0(16757\ \text{lb})\frac{5}{8} = 10473\ \text{lb}$$ (horizontal load perpendicular to panel at Points B and B1)

H_B = (10743 lb)(2.0 ft) / (1.33 ft) = 15714 lb (inward or outward force at B)

H_{B1} = (10473 lb)(0.67 ft) / (1.33 ft) = 5237 lb (inward or outward force at B1)

U = 0.9D + 1.0E

$$V_{uA_{min}} = \frac{0.9(10245 \text{ lb}) - 1.0(3047 \text{ lb})}{2} = 3086 \text{ lb}$$ (vertical load downward at Point A)

Horizontal forces are the same as combination 1.2 D + 1.0 E. No uplift occurs; the net reaction at A is downward. Maximum forces are controlled by prior combination. It is important to realize that inward and outward acting horizontal forces generate different demands when the connections are eccentric to the center of mass as it is in this example. Only the maximum reactions are computed above.

13.2.3.4 Prescribed Seismic Displacements

Prescribed seismic displacements are not applicable to the building panel because all connections are essentially at the same elevation.

13.2.4 Column Cover

13.2.4.1 Connection Details

Figure 13.2-6 shows the key to the types of forces resisted at each column cover connection.

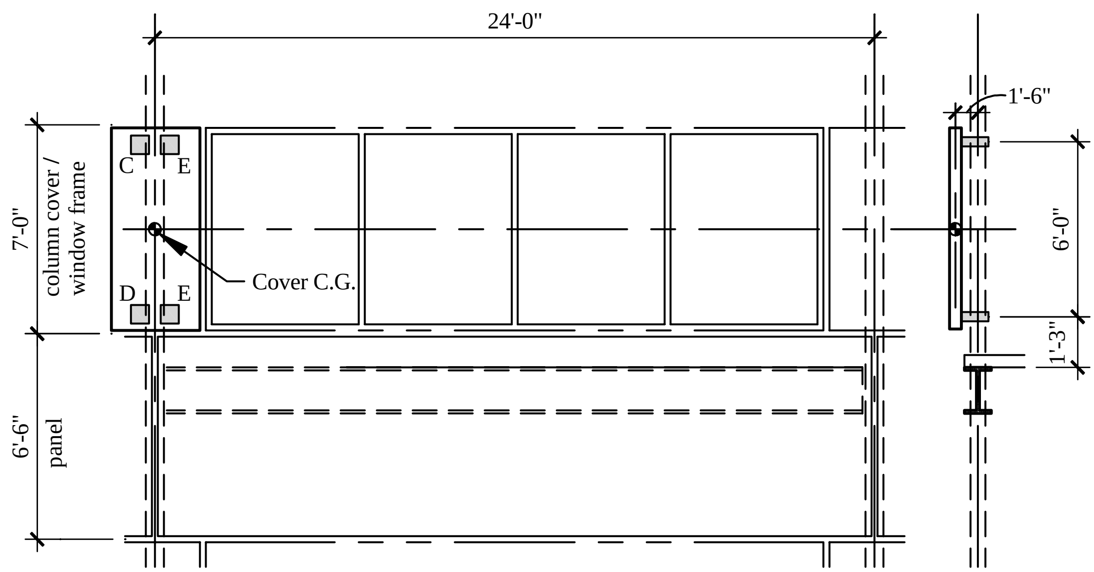


Figure 13.2-6 Column cover connection layout (1.0 ft = 0.3048 m).

Vertical loads, horizontal loads parallel to the panel, and horizontal loads perpendicular to the panel are resisted at Point C. The eccentricity of vertical loads is resisted by a force couple at Points C and D. The horizontal load parallel to the panel eccentricity between the panel and the support is resisted in flexure of the connection. The connection is designed to take the loads directly to the columns.

Horizontal loads parallel to the panel and horizontal loads perpendicular to the panel are resisted at Point D. The vertical load eccentricity between the panel and the support is resisted by a force couple of Points

C and D. The eccentricity of horizontal loads parallel to the panel is resisted by flexure at the connection. The connection must not restrict vertical movement of the panel due to thermal effects or seismic input. The connection is designed to take the loads directly to the columns.

Horizontal loads perpendicular to the panel are resisted at Points E. The connection is designed to take the loads directly to the columns.

There is no load eccentricity associated with the horizontal loads perpendicular to the panel.

In this example, all connections are made to the sides of the column because there usually is not enough room between the outside face of the column and the inside face of the cover to allow a feasible load-carrying connection.

13.2.4.2 Prescribed Seismic Forces

Calculation of prescribed seismic forces for the column cover are not shown in this example. They should be determined in the same manner as illustrated for the spandrel panels.

13.2.4.3 Prescribed Seismic Displacements

The results of an elastic analysis of the building structure are not usually available in time for use in the design of the precast cladding system. As a result, prescribed seismic displacements are usually calculated based on allowable story drift requirements:

h_{sx} = story height = 13 ft 6 in

X = height of upper support attachment = 47 ft 9 in

Y = height of lower support attachment = 41 ft 9 in

$\Delta_a = 0.020h_{sx}$ (*Provisions* Table 5.2.8 [4.5-1])

$$D_{p_{max}} = (X - Y)\frac{\Delta_a}{h_{sx}} = (72\text{ in.})\frac{0.020h_{sx}}{h_{sx}} = 1.44\text{ in.}$$ (*Provisions* Eq. 6.1.4-2 [6.2-6])

The joints at the top and bottom of the column cover must be designed to accommodate an in-plane relative displacement of 1.44 inches. The column cover will rotate somewhat as these displacements occur, depending on the nature of the connections to the column. If the supports at one level are “fixed” to the columns while the other level is designed to “float,” then the rotation will be that of the column at the point of attachment.

13.2.5 Additional Design Considerations

13.2.5.1 Window Frame System

The window frame system is supported by the spandrel panels above and below. Assuming that the spandrel panels move rigidly in-plane with each floor level, the window frame system must accommodate a prescribed seismic displacement based on the full story height.

$$D_{p_{max}} = (X - Y)\frac{\Delta_a}{h_{sx}} = (162\text{ in.})\frac{0.020h_{sx}}{h_{sx}} = 3.24\text{ in.}$$ (*Provisions* Eq. 6.1.4-2 [6.2-6])

The window frame system must be designed to accommodate an in-plane relative displacement of 3.24 in. between the supports. This is normally accommodated by a clearance between the glass and the frame. *Provisions* Sec. 6.2.10.1 [6.3.7], prescribes a method of checking such a clearance. It requires that the clearance be large enough so that the glass panel will not fall out of the frame unless the relative seismic displacement at the top and bottom of the panel exceeds 125 percent of the value predicted amplified by the building importance factor. If h_p and b_p are the respective height and width of individual panes and if the horizontal and vertical clearances are designated c_1 and c_2, respectively, then the following expression applies:

$$D_{clear} = 2c_1\left(1+\frac{h_p c_2}{b_p c_1}\right) \geq 1.25 D_p$$

For h_p = 7 ft, b_p = 5 ft, and D_p = 3.24 in., and setting $c_1 = c_2$, the required clearance is 0.84 in.

13.2.5.2 Building Corners

Some thought needs to be given to seismic behavior at external building corners. The preferred approach is to detail the corners with two separate panel pieces, mitered at a 45 degree angle, with high grade sealant between the sections. An alternative choice of detailing L-shaped corner pieces, would introduce more seismic mass and load eccentricity into connections on both sides of the corner column.

13.2.5.3 Dimensional Coordination

It is important to coordinate dimensions with the architect and structural engineer. Precast concrete panels must be located a sufficient distance from the building structural frame to allow room for the design of efficient load transfer connection pieces. However, distances must not be so large as to unnecessarily increase the load eccentricities between the panels and the frame.

13.3 HVAC FAN UNIT SUPPORT

13.3.1 Example Description

In this example, the mechanical component is a 4-ft-high, 5-ft-wide, 8-ft-long, 3000-lb HVAC fan unit that is supported on the two long sides near each corner (Figure 13.3-1). The component is located at the roof level of a five-story office building, near a significant active fault in Los Angeles, California. The building is assigned to Seismic Use Group I. Two methods of attaching the component to the 4,000 psi, normal-weight roof slab, are considered as follows:

1. Direct attachment to the structure with 36 ksi, carbon steel, cast-in-place anchors and
2. Support on vibration isolation springs, that are attached to the slab with 36 ksi carbon steel post-installed expansion anchors.

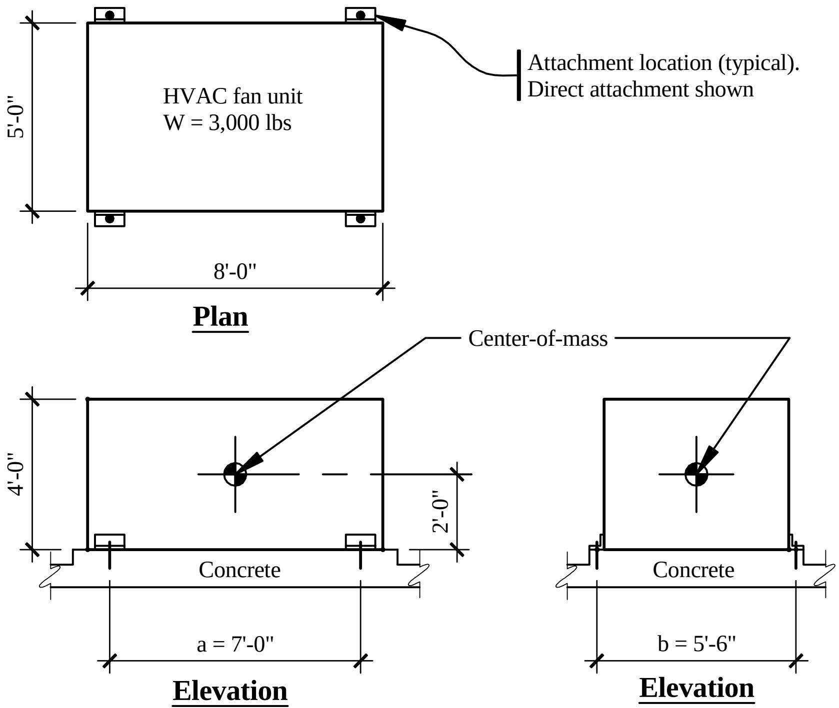


Figure 13.3-1 Air handling fan unit (1.0 ft = 0.3048 m, 1.0 lb = 4.45 N).

13.3.2 Design Requirements

13.3.2.1 *Provisions* Parameters and Coefficients

a_p = 1.0 for direct attachment (*Provisions* Table 6.3.2 [6.4-1])

a_p = 2.5 for vibration isolated (*Provisions* Table 6.3.2 [6.4-1])

S_{DS} = 1.487 (Design Values CD-ROM)

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

Seismic Design Category = D (*Provisions* Table 4.2.1a [14.4-1])

W_p = 3000 lb (given)

R_p = 2.5 for HVAC system equipment (*Provisions* Table 6.3.2 [6.4-1])

R_p = 1.5 for expansion anchors and shallow, cast-in-place anchors* ((*Provisions* Sec. 6.1.6.1 [6.2.8.1])

Shallow anchors are defined by *Provisions* Sec. 2.1 as anchors having embedment-to-diameter ratios of less than 8.

[In the 2003 *Provisions* component anchorage is treated differently. Rather than making distinctions based on an anchor being “shallow,” component anchorage is designed using R_p = 1.5 unless specific ductility or prequalification requirements are satisfied.]

$I_p = 1.0$ (*Provisions* Sec. 6.1.5 [6.2.2])

$z/h = 1.0$ (for roof mounted equipment)

$\rho = 1.0$ (*Provisions* Sec. 6.1.3 [6.2.6])

[The 2003 *Provisions* indicate that the redundancy factor does not apply to the design of nonstructural components. Although the effect is similar to stating that $\rho = 1$, there is a real difference since load effects for such components and their supports and attachments are now defined in Chapter 6 rather than by reference to Chapter 4.]

13.3.2.2 Performance Criteria

Component failure should not cause failure of an essential architectural, mechanical, or electrical component (*Provisions* Sec. 6.1 [6.2.3]).

Component seismic attachments must be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity (*Provisions* Sec. 6.1.2 [6.2.5]).

Anchors embedded in concrete or masonry must be proportioned to carry the least of: (a) the design strength of the connected part, (b) 1.3 times the force in the connected part due to the prescribed forces, or (c) the maximum force that can be transferred to the connected part by the component structural system (*Provisions* Sec. 6.1.6.2 [6.2.8.2]).

Attachments and supports transferring seismic loads must be constructed of materials suitable for the application and must be designed and constructed in accordance with a nationally recognized structural standard (*Provisions* Sec. 6.3.13.2.a [6.4.4, item 6]).

Components mounted on vibration isolation systems must have a bumper restraint or snubber in each horizontal direction. Vertical restraints must be provided where required to resist overturning. Isolator housings and restraints must also be constructed of ductile materials. A viscoelastic pad, or similar material of appropriate thickness, must be used between the bumper and equipment item to limit the impact load (*Provisions* Sec. 6.3.13.2.e). Such components must also resist an amplified design force.

13.3.3 Direct Attachment to Structure

This section illustrates design for cast-in-place concrete anchors with embedment-length-to-diameters ratios of 8 or greater; thus, the use of R_p = 2.5 is permitted. [In 2003 *Provisions* Sec. 6.2.8.1, the value of R_p used in designing component anchorage is no longer based on the embedment depth-to-diameter ratio. Instead R_p = 1.5 unless specific ductility or prequalification requirements are satisfied.]

13.3.3.1 Prescribed Seismic Forces

See Figure 13.3-2 for freebody diagram for seismic force analysis.

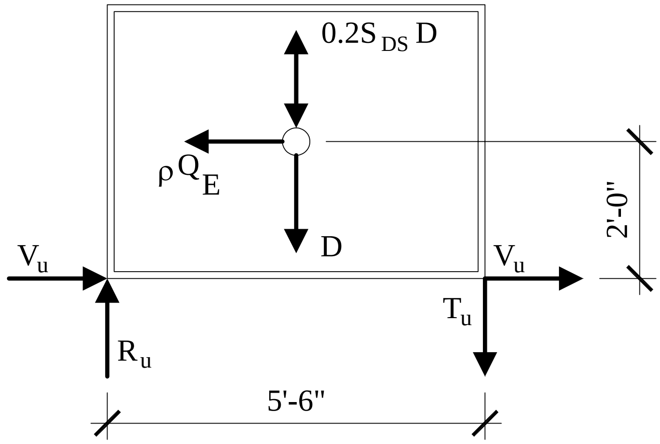


Figure 13.3-2 Free-body diagram for seismic force analysis (1.0 ft = 0.348 m).

$$F_p = \frac{0.4(1.0)(1.487)(3000 \text{ lb})}{\left(2.5/1.0\right)}\left(1+2(1)\right) = 2141 \text{ lb}$$ (*Provisions* Eq. 6.1.3-1 [6.2-1])

$$F_{p_{max}} = 1.6(1.487)(1.0)(3000 \text{ lb}) = 7138 \text{ lb}$$ (*Provisions* Eq. 6.1.3-2 [6.2-3])

$$F_{p_{min}} = 0.3(1.487)(1.0)(3000 \text{ lb}) = 1338 \text{ lb}$$ (*Provisions* Eq. 6.1.3-3 [6.2-4])

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following six steps would be cast differently; the result is the same.]

Q_E (due to application of F_p) = 2141 lb (*Provisions* Sec. 6.1.3)

ρQ_E = (1.0)(2141 lb) = 2141 lb (horizontal earthquake effect)

$0.2S_{DS}D$ = (0.2)(1.487)(3000 lb) = 892 lb (vertical earthquake effect)

$D = W_p$ = 3000 lb (vertical gravity effect)

$E = \rho Q_E + 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-2 [4.2-2])

U = 1.2D + 1.0E + 0.5L + 0.2S

$$V_u = \frac{1.0(2141 \text{ lb})}{4 \text{ bolts}} = 535 \text{ lb/bolt}$$

$$T_u = \frac{-1.2(3000 \text{ lb})(2.75 \text{ ft}) + 1.0(2141 \text{ lb})(2 \text{ ft}) + 1.0(892 \text{ lb})(2.75 \text{ ft})}{(5.5 \text{ ft})(2 \text{ bolts})} = -288 \text{ lb/bolt}$$ no tension

U = 0.9D - (1.3W or 1.0E)

$$V_u = \frac{1.0(2141 \text{ lb})}{4 \text{ bolts}} = 535 \text{ lb/bolt}$$

$$T_u = \frac{-0.9(3000 \text{ lb})(2.75 \text{ ft}) + 1.0(2141 \text{ lb})(2 \text{ ft}) + 1.0(892 \text{ lb})(2.75 \text{ ft})}{(5.5 \text{ ft})(2 \text{ bolts})} = -63 \text{ lb/bolt}$$ <u>no tension</u>

13.3.3.2 Proportioning and Design

See Figure 13.3-3 for anchor for direct attachment to structure.

Check one ¼-in.-diameter cast-in-place anchor embedded 2 in. into the concrete slab with no transverse reinforcing engaging the anchor and extending through the failure surface. Although there is no required tension strength on these anchors, design strengths and tension/shear interaction acceptance relationships are calculated to demonstrate the use of the *Provisions* equations.

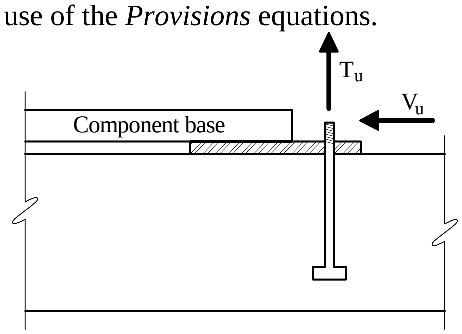


Figure 13.3-3 Anchor for direct attachment to structure.

5.3.3.2.1 Design Tension Strength on Isolated Anchor in Slab, Away from Edge, Loaded Concentrically

[The 2003 *Provisions* refer to Appendix D of ACI 318-02 rather than providing specific, detailed requirements. Note also that some of the resistance factors, ϕ, are different.]

Following *Provisions* Sec. 9.2, for a headed bolt or a rod with a nut at the bottom:

$$\frac{L}{d} = \frac{2 \text{ in.}}{0.25 \text{ in.}} = 8.0$$ (not shallow, R_p = 2.5 is permitted)

[In 2003 *Provisions* selection of R_p is no longer based on the L/d ratio; see Sec. 6.2.8.1.]

Tension capacity of steel, ϕ = 0.80:

$N_s = A_{se}F_y$ = (0.049 in.^{2})(36000 psi) = 1764 lb (*Provisions* Sec. 9.2.5.1.2 [ACI 318-02 Eq. D-3])

[In Appendix D of ACI 318-02 this capacity calculation is based on f_{ut} rather than F_y, since "the large majority of anchor materials do not exhibit a well-defined yield point."]

Tension capacity of concrete, ϕ = 0.70, with no eccentricity, no pullthrough, and no edge or group effect:

$$N_c = k\sqrt{f_c'}h_{ef}^{1.5} = 24\sqrt{4000 \text{ psi}}(8^{1.5}) = 4293 \text{ lb}$$ (*Provisions* Eq. 9.2.5.2.2-1 [ACI 318-02 Eq. D-7])

[In Appendix D of ACI 318-02 this item is defined as N_b rather than N_c.]

The steel controls but, with no tension demand, the point is moot.

5.3.3.2.2 Design Shear Strength on Isolated Anchor, Away from Edge

Shear capacity of steel, $\phi = 0.80$:

$$V_s = A_{se}F_y = (0.049\text{ in.}^2)(36000\text{ psi}) = 1764\text{ lb} \quad (\textit{Provisions}\text{ Eq. 9.2.6.1.2-1 [ACI 318-02 Eq. D-17)}$$

[In Appendix D of ACI 318-02 this capacity calculation is based on f_{ut} rather than F_y, since "the large majority of anchor materials do not exhibit a well-defined yield point."]

Shear capacity of concrete, far from edge, limited to pryout, $\phi = 0.70$:

$$V_{cp} = 2N_c = 2(4293) = 8493\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 9.2.6.3.1 [ACI 318-02 Eq. D-28)}$$

The steel controls, with $\phi V_N = 0.8(1764) = 1411$ lb

Per *Provisions* Sec. 6.1.6.2 [6.2.8.2], anchors embedded in concrete or masonry are to be proportioned to carry at least 1.3 times the force in the connected part due to the prescribed forces. Thus, $V_u = 1.3(535) = 696$ lb and the anchor is clearly adequate.

5.3.3.2.3 Combined Tension and Shear

The *Provisions* gives a new equation (Eq. 9.2.7.3 [ACI 318-02 Eq. D-29]) for the interaction of tension and shear on an anchor or a group of anchors:

$$\frac{N_u}{\phi N_N} + \frac{V_u}{\phi V_N} \le 1.2\text{, which applies when either term exceeds 0.2}$$

5.3.3.2.3 Summary

At each corner of the component, provide one ¼-in.-diameter cast-in-place anchor embedded 2 in. into the concrete slab. Transverse reinforcement engaging the anchor and extending through the failure surface is not necessary.

13.3.4 Support on Vibration Isolation Springs

13.3.4.1 Prescribed Seismic Forces

Design forces for vibration isolation springs are determined by an analysis of earthquake forces applied in a diagonal horizontal direction as shown in Figure 13.3-4. Terminology and concept are taken from ASHRAE APP IP.

Angle of diagonal loading:

$$\theta = \tan^{-1}\left(\frac{b}{a}\right) \qquad \text{(ASHRAE APP IP. Eq. 17)}$$

Tension per isolator:

$$T_u = \frac{W_p - F_{pv}}{4} - \frac{F_p h}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right) \qquad \text{(ASHRAE APP IP. Eq. 18)}$$

Compression per isolator:

$$C_u = \frac{W_p + F_{pv}}{4} + \frac{F_p h}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right) \qquad \text{(ASHRAE APP IP. Eq. 19)}$$

Shear per isolator:

$$V_u = \frac{F_p}{4} \qquad \text{(ASHRAE APP IP. Eq. 20)}$$

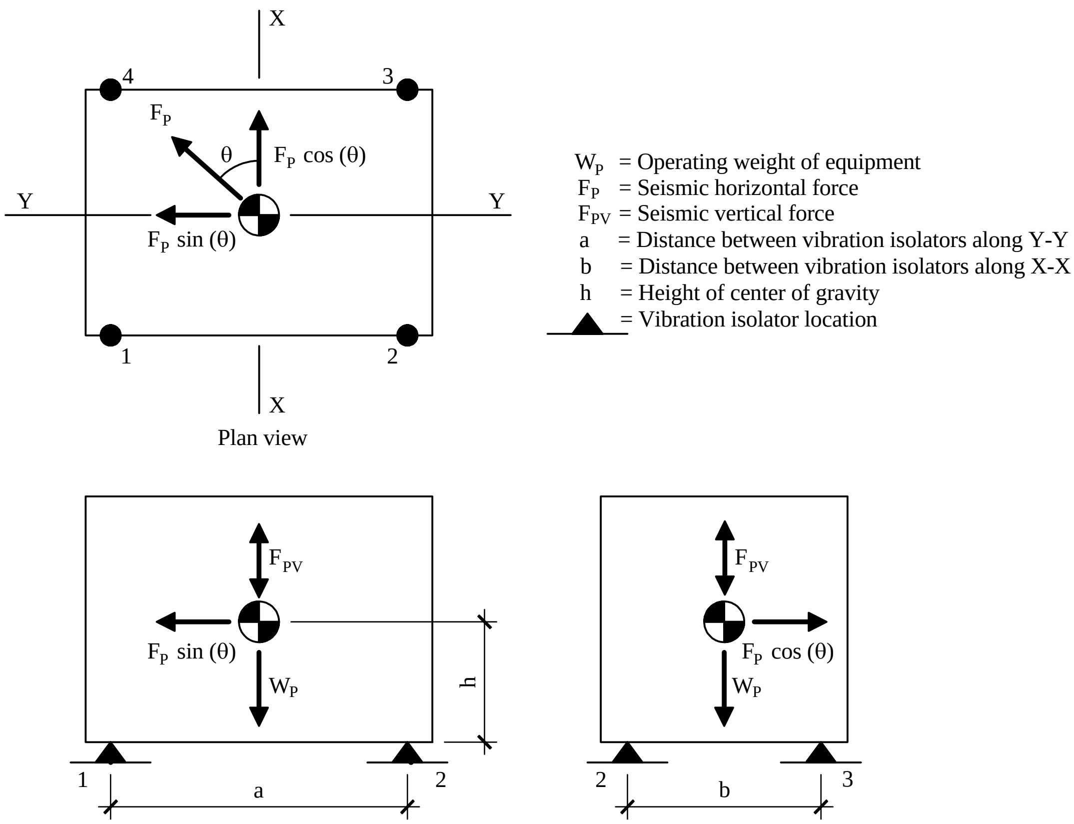


Figure 13.3-4 ASHRAE diagonal seismic force analysis for vibration isolation springs.

Select worst case assumption: Design for post-installed expansion anchors, requiring the use of R_p = 1.5.

$$F_p = \frac{0.4(2.5)(1.487)(3000\text{ lb})}{\left(1.5/1.0\right)}\left(1+2(1)\right) = 8922\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 6.1.3-1 [6.2-1]})$$

$$F_{p_{max}} = 1.6(1.487)(1.0)(3000\text{ lb}) = 7138\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 6.1.3-2 [6.2-3]})$$

$$F_{p_{min}} = 0.3(1.487)(1.0)(3000\text{ lb}) = 1338\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 6.1.3-3 [6.2-4]})$$

Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. Per Provisions Table 6.3.2 [6.4-1], Footnote B, the design force shall be taken as $2F_p$.

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following steps would be cast differently; the result is the same.]

$Q_E = F_p = 2(7138 \text{ lb}) = 14276 \text{ lb}$ (*Provisions* Sec. 6.1.3)

$\rho Q_E = (1.0)(14276 \text{ lb}) = 14276 \text{ lb}$ (horizontal earthquake effect)

$F_{pv(\text{ASHRAE})} = 0.2S_{DS}D = (0.2)(1.487)(3000 \text{ lb}) = 892 \text{ lb}$ (vertical earthquake effect)

$D = W_p = 3000 \text{ lb}$ (vertical gravity effect)

$E = \rho Q_E + 0.2S_{DS}D$ (Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (Eq. 5.2.7.1-2 [4.2-2])

<u>U = 1.2D + 1.0E + 0.5L + 0.2S</u>

$$\theta = \tan^{-1}\left(\frac{7 \text{ ft}}{5.5 \text{ ft}}\right) = 51.8^{\circ}$$

$$T_u = \frac{1.2(3000 \text{ lb}) - (892 \text{ lb})}{4} - \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8°)}{7 \text{ ft}} + \frac{\sin(51.8°)}{5.5 \text{ ft}}\right) = -2624 \text{ lb}$$

$$C_u = \frac{1.2(3000 \text{ lb}) + (892 \text{ lb})}{4} + \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8°)}{7 \text{ ft}} + \frac{\sin(51.8°)}{5.5 \text{ ft}}\right) = 4424 \text{ lb}$$

$$V_u = \frac{14276 \text{ lb}}{4} = 3569 \text{ lb}$$

<u>U = 0.9D + 1.0E</u>

$$\theta = \tan^{-1}\left(\frac{7 \text{ ft}}{5.5 \text{ ft}}\right) = 51.8^{\circ}$$

$$T_u = \frac{0.9(3000 \text{ lb}) - (892 \text{ lb})}{4} - \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8°)}{7 \text{ ft}} + \frac{\sin(51.8°)}{5.5 \text{ ft}}\right) = -2849 \text{ lb}$$

$$C_u = \frac{0.9(3000 \text{ lb}) + (892 \text{ lb})}{4} + \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8°)}{7 \text{ ft}} + \frac{\sin(51.8°)}{5.5 \text{ ft}}\right) = 4199 \text{ lb}$$

$$V_u = \frac{14276 \text{ lb}}{4} = 3569 \text{ lb}$$

13.3.4.2 Proportioning and Details

Anchor and snubber loads for support on vibration isolation springs are shown in Figure 13.3-5.

Check vibration isolation system within housing anchored with two 1-in.-diameter post-installed expansion anchors embedded 9 inches into the concrete.

The *Provisions* does not provide a basis for determining the design strength of post-installed expansion anchors. Although manufacturers provide ultimate strength shear and tension loads for their products, the *Provisions* does not provide resistance or quality values, ϕ, to allow determination of design shear and tension strengths. The 2002 edition of ACI 318 contains provisions for post-installed anchors which depend on anchor testing per the ACI 355.2-01 testing standard. Prior to the adoption of this method, the best course available was to use allowable stress loads published in evaluation reports prepared by model building code agencies. These allowable stress loads were then multiplied by 1.4 to convert to design strengths. This technique will be illustrated in the following.

[The 2003 *Provisions* refer to Appendix D of ACI 318-02.]

Allowable stress combinations are included in the 2000 *IBC*. The 1.4 strength conversion factor is unnecessary when using allowable stress loads published in evaluation reports prepared by model building code agencies.

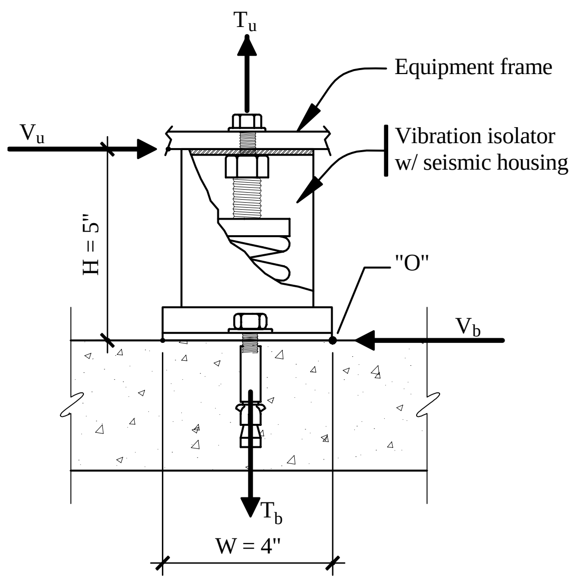


Figure 13.3-5 Anchor and snubber loads for support on vibration isolation springs (1.0 in. = 25.4 mm).

13.3.4.2.1 Design Tension Strength on Isolated Anchor in Slab, Away from Edge

Allowable stress tension values are obtained from ICBO Evaluation Services, Inc., ER-4627 for Hilti Kwik Bolt II concrete anchors. Similar certified allowable values are expected with anchors from other manufacturers.

anchor diameter = 1 in.

anchor depth = 9 in.

$f_c{}' = 4000$ psi

with special inspection

T_{allow} = 8800 lb

$P_s = \phi P_c$ = 1.4(8800 lb) = 12320 lb

13.3.4.2.2 Design Shear Strength on Isolated Anchor in Slab, Away from Edge

Allowable stress shear values are obtained from ICBO Evaluation Services, Inc., ER-4627 for Hilti Kwik Bolt II concrete anchors. Similar certified allowable values are expected with anchors from other manufacturers.

anchor diameter = 1 in.

anchor depth = 9 in.

f_c' = 4000 psi

V_{allow} = 8055 lb

$V_s = \phi V_c$ = 1.4(8055 lb) = 11277 lb

13.3.4.2.3 Combined Tension and Shear

Per *Provisions* Sec. 6.1.6.2 [6.2.8.2], anchors embedded in concrete or masonry shall be proportioned to carry at least 1.3 times the force in the connected part due to the prescribed forces.

In the *Provisions* and in the 2000 *IBC*, the factor of 2.0 is reduced to 1.3. This will greatly reduce the prescribed seismic forces.

Interaction relationships for combined shear and tension loads are obtained from ICBO Evaluation Services, Inc. ER-4627 for Hilti Kwik Bolt II concrete anchors. Similar results are expected using other anchors.

As stated in ICBO ES evaluation report:

$$\left(\frac{P_s}{P_t}\right)^{5/3} + \left(\frac{V_s}{V_t}\right)^{5/3} \leq 1$$

Using *Provisions* terminology:

$$\left(\frac{T_b}{\phi P_c}\right)^{5/3} + \left(\frac{V_b}{\phi V_c}\right)^{5/3} \leq 1$$

$$\left(\frac{2 \times 4462 \text{ lb}}{11277 \text{ lb}}\right)^{5/3} + \left(\frac{2 \times 1785 \text{ lb}}{12075 \text{ lb}}\right)^{5/3} = 0.68 + 0.13 = 0.81 < 1 \qquad \text{OK}$$

13.3.4.2.4 Summary

At each corner of the HVAC fan unit, provide a vibration isolation system within a housing anchored with two 1-in.-diameter post-installed expansion anchors embedded 9 in. into the concrete slab. Special inspection is required. A raised concrete pad is probably required to allow proper embedment of the post-installed expansion anchors.

Other post-installed anchors, such as chemical (adhesive) or undercut post-installed anchors also could be investigated. These anchors may require more involved installation procedures, but they may allow the use of R_p = 2.5 if they have an embedment depth-to-diameter ratio of at least 8 (i.e., are not shallow anchors). The higher R_p value will result in much smaller prescribed seismic forces and, therefore, a much reduced embedment depth.

[Again note that in 2003 *Provisions* selection of R_p is no longer based on the *L*/*d* ratio; see Sec. 6.2.8.1.]

13.3.5 Additional Considerations for Support on Vibration Isolators

Vibration isolation springs are provided for equipment to prevent vibration from being transmitted to the building structure. However, they provide virtually no resistance to horizontal seismic forces. In such cases, some type of restraint is required to resist the seismic forces. Figure 13.3-6 illustrates one concept where a bolt attached to the equipment base is allowed to slide a controlled distance (gap) in either direction along its longitudinal axis before it contacts resilient impact material.

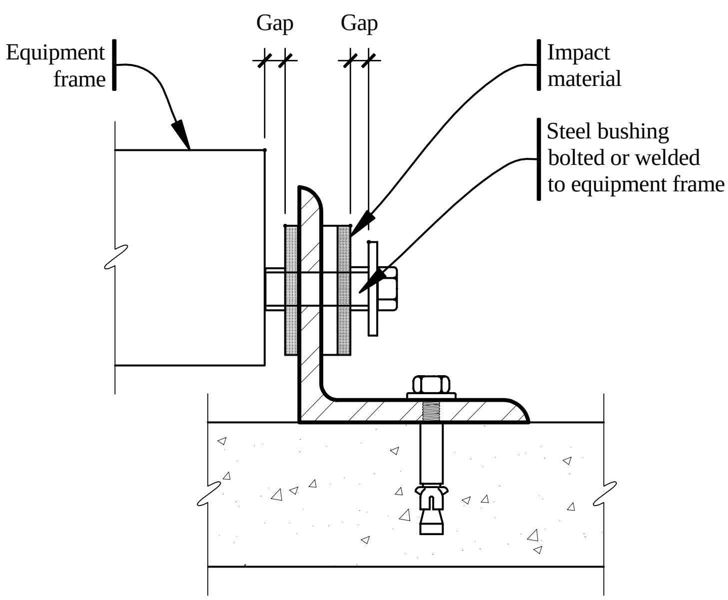


Figure 13.3-6 Lateral restraint required to resist seismic forces.

Design of restraints for vibration-isolated equipment varies for different applications and for different manufacturers. In most cases, restraint design incorporates all directional capability with an air gap, a soft impact material, and a ductile restraint or housing.

Restraints should have all-directional restraint capability to resist both horizontal and vertical motion. Vibration isolators have little or no resistance to overturning forces. Therefore, if there is a difference in height between the equipment center of gravity and the support points of the springs, rocking is inevitable and vertical restraint is required.

An air gap between the restraint device and the equipment prevents vibration from transmitting to the structure during normal operation of the equipment. Air gaps are generally no greater than ¼ in. Dynamic tests indicate a significant increase in acceleration for air gaps larger than ¼ in.

A soft impact material often an elastomer such as bridge bearing neoprene reduces accelerations and impact loads by preventing steel-to-steel contact. The thickness of the elastomer can significantly reduce accelerations to both the equipment and the restraint device and should be specifically addressed for life safety applications.

A ductile restraint or housing is critical to prevent catastrophic failure. Unfortunately, housed isolators made of brittle materials such as cast iron often are assumed to be capable of resisting seismic loads and continue to be installed in seismic zones.

Overturning calculations for vibration- isolated equipment must consider a worst case scenario as illustrated in *Guide* Sec. 13.3.4.1. However, important variations in calculation procedures merit further discussion. For equipment that is usually directly attached to the structure, or mounted on housed vibration isolators, the weight can be used as a restoring force since the equipment will not transfer a tension load to the anchors until the entire equipment weight is overcome at any corner. For equipment installed on any other vibration isolated system (such as the separate spring and snubber arrangement shown in Figure 13.3-5), the weight cannot be used as a restoring force in the overturning calculations.

As the foregoing illustrates, design of restraints for resiliently mounted equipment is a specialized topic. The *Provisions* sets out only a few of the governing criteria. Some suppliers of vibration isolators in the highest seismic zones are familiar with the appropriate criteria and procedures. Consultation with these suppliers may be beneficial.

13.4 ANALYSIS OF PIPING SYSTEMS

13.4.1 ASME Code Allowable Stress Approach

Piping systems are typically designed to satisfy national standards such as ASME B31.1. Piping required to be designed to other ASME piping codes use similar approaches with similar definition of terms.

13.4.1.1 Earthquake Design Requirements

ASME B31.1 Sec. 101.5.3 requires that the effects of earthquakes, where applicable, be considered in the design of piping, piping supports, and restraints using data for the site as a guide in assessing the forces involved. However, earthquakes need not be considered as acting concurrently with wind.

13.4.1.2 Stresses Due to Sustained Loads

The effects of pressure, weight, and other sustained loads must meet the requirements of ASME B31.1 Eq. 11A:

$$S_L = \frac{PD_o}{4t_n} + \frac{0.75iM_A}{Z} \le 1.0S_h$$

where:

S_L = sum of the longitudinal stresses due to pressure, weight, and other sustained loads

P = internal design pressure, psig

D_o = outside diameter of pipe, in.

t_n = nominal pipe wall thickness, in.

i = stress intensification factor from ASME Piping Code Appendix D, unitless
= 1.0 for straight pipe
≥ 1.0 for fittings and connections

M_A = resultant moment loading on cross section due to weight and other sustained loads, in.-lb

Z = section modulus, in.^{3}

S_h = basic material allowable stress at maximum (hot) temperature from ASME Piping Code Appendix A

For example, ASTM A53 seamless pipe and tube, Grade B: S_h = 15.0 ksi for -20 to 650 degrees F.

13.4.1.3 Stresses Due to Occasional Loads

The effects of pressure, weight, and other sustained loads, and occasional loads including earthquake must meet the requirements of ASME B31.1 Eq. 12A:

$$\frac{PD_o}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75iM_B}{Z} \le kS_h$$

where:

M_B = resultant moment loading on cross-section due to occasional loads, such as from thrust loads, pressure and flow transients, and earthquake. Use one-half the earthquake moment range. Effects of earthquake anchor displacements may be excluded if they are considered in Eq. 13A, in.-lb

k = duration factor, unitless

= 1.15 for occasional loads acting less than 10% of any 24 hour operating period

= 1.20 for occasional loads acting less than 1% of any 24 hour operating period

= 2.00 for rarely occurring earthquake loads resulting from both inertial forces and anchor movements (per ASME interpretation)

13.4.1.4 Thermal Expansion Stress Range

The effects of thermal expansion must meet the requirements of ASME B31.1 Eq. 13A:

$$S_E = \frac{iM_C}{Z} \le S_A + f\left(S_h - S_L\right)$$

where:

S_E = sum of the longitudinal stresses due to thermal expansion, ksi

M_C = range of resultant moments due to thermal expansion. Also includes the effects of earthquake anchor displacements if not considered in Eq. 12A, in.-lb

S_A = allowable stress range, ksi (per ASME B31.1 Eq. 1, $S_A = f\left(1.25S_c + 0.25S_h\right)$

f = stress range reduction factor for cyclic conditions from the ASME Piping Code Table 102.3.2.

S_c = basic material allowable stress at minimum (cold) temperature from the ASME Piping Code Appendix A

13.4.1.5 Summary

In the ASME B31.1 allowable stress approach, the earthquake's effects only appear in the M_B and M_C terms.

Earthquake inertial effects ⇒ M_B term

Earthquake displacement effects ⇒ M_C term

13.4.2 Allowable Stress Load Combinations

ASME B31.1 utilizes an allowable stress approach; therefore, allowable stress force levels and allowable stress load combinations should be used. While the *Provisions* are based on strength design, the IBC provides the following two sets of allowable stress loads and load combinations. The IBC load combinations are appropriate for use for piping systems when considering earthquake effects. When earthquake effects are not considered, load combinations should be taken from the appropriate piping system design code.

13.4.2.1 IBC Basic Allowable Stress Load Combinations

No increases in allowable stress are permitted for the following set of load combinations:

D (IBC Eq. 16-7)

$D + L + (L_r \text{ or } S \text{ or } R)$ (IBC Eq. 16-9)

$D + (W \text{ or } 0.7E)$ (IBC Eq. 16-10)

$0.6D - 0.7E$ (IBC Eq. 16-12)

13.4.2.2 IBC Alternate Basic Allowable Stress Load Combinations

Increases in allowable stress (typically 1/3) are permitted for the following alternate set of load combinations that include W or E:

$D + L + (L_r \text{ or } S \text{ or } R)$ (IBC Eq. 16-13)

$D + L + S + E/1.4$ (IBC Eq. 16-17)

$0.9D + E/1.4$ (IBC Eq. 16-18)

13.4.2.3 Modified IBC Allowable Stress Load Combinations

It is convenient to define separate earthquake load terms to represent the separate inertial and displacement effects.

E_I = Earthquake inertial effects ⇒ M_B term

E_Δ = Earthquake displacement effects ⇒ M_C term

It is also convenient to use the IBC Alternate Basic Allowable Stress Load Combinations modified to use ASME Piping Code terminology, deleting roof load effects (L_r or S or R) and multiplying by 0.75 to account for the 1.33 allowable stress increase when W or E is included. Only modified IBC Eq. 16-17 and 16-18 will be considered in the discussion that follows.

$0.75[D + L + S + (E_I + E_\Delta)/1.4]$ (modified IBC Eq. 16-17)

$0.75[0.9D + (E_I + E_\Delta)/1.4]$ (modified IBC Eq. 16-18)

13.4.3 Application of the *Provisions*

13.4.3.1 Overview

Provisions Sec. 6.3.11 [6.4.2, item 4] requires that, in addition to their attachments and supports, piping systems assigned an I_p greater than 1.0 must themselves be designed to meet the force and displacement requirements of *Provisions* Sec. 6.1.3 and 6.1.4 [6.2.6 and 6.2.7] and the additional requirements of this section.

13.4.3.1 Forces

Provisions Sec. 6.1.3 [6.2.6] provides specific guidance regarding the equivalent static forces that must be considered. In computing the earthquake forces for piping systems, the inertial portion of the forces (noted as E_I in this example) are computed using *Provisions* Eq. 6.1.3-1, 6.1.3-2, and 6.1.3-3 [6.2-1, 6.2-3, and 6.2-4] for F_p with a_p = 1 and R_p = 3.5. For anchor points with different elevations, the average value of the F_p may be used with minimum and maximums observed. In addition, when computing the inertial forces, the vertical seismic effects (± $0.2S_{DS}W_p$) should be considered.

The term E_I can be expressed in terms of the forces defined in the *Provisions* converted to an allowable stress basis by the 1.4 factor:

$$E_I = \left(\frac{F_p}{1.4}\right)_{horizontal} \pm \left(\frac{0.2S_{DS}W_p}{1.4}\right)_{vertical}$$

It is convenient to designate the term $\left(1 \pm \frac{0.2S_{DS}}{1.4}\right)$ by the variable β.

The vertical component of E_I can now be defined as βM_a and applied to all load combinations that include E_I.

M_B can now be defined as the resultant moment induced by the design force $F_p/1.4$ where F_p is as defined by *Provisions* Eq. 6.1.3-1, 6.1.3-2, or 6.1.3-3.

13.4.3.3 Displacements

Provisions Sec. 6.1.4 [6.2.7] provides specific guidance regarding the relative displacements that must be considered. Typically piping systems are designed considering forces and displacements using elastic analysis and allowable stresses for code prescribed wind and seismic equivalent static forces in combination with operational loads.

However, no specific guidance is provided in the *Provisions* except to say that the relative displacements should be accommodated. The intent of the word "accommodate" was not to require that a piping system remain elastic. Indeed, many types of piping systems typically are very ductile and can accommodate large amounts of inelastic strain while still functioning quite satisfactorily. What was intended was that the relative displacements between anchor and constraining points that displace significantly relative to one another be demonstrated to be accommodated by some rational means. This accommodation can be made by demonstrating that the pipe has enough flexibility and/or inelastic strain capacity to accommodate the displacement by providing loops in the pipeline to permit the displacement or by adding flex lines or articulating couplings which provide free movement to accommodate the displacement. Sufficient flexibility may not exist where branch lines, may be forced to move with a ceiling or other structural system are connected to main lines. Often this “accommodation” is done by using engineering judgment, without calculations. However, if relative displacement calculations were required for a piping system, a flexibility analysis would be required. A flexibility analysis is one in which a pipe is modeled as a finite element system with commercial pipe stress analysis programs (such as Autopipe or CAESAR II) and the points of attachment are displaced by the prescribed relative displacements. The allowable stress for such a condition may be significantly greater than the normal allowable stress for the pipe.

The internal moments resulting from support displacement may be computed by means of elastic analysis programs using the maximum computed relative displacements as described earlier and then adjusted. As with elastic inertial forces, the internal moments caused by relative displacement can be divided by R_p to account for reserve capacity. A reduction factor of 1.4 may be used to convert them for use with allowable stress equations. Therefore, M_C can now be defined as the resultant moment induced by the design relative seismic displacement $D_p/1.4R_p$ where D_p is defined by *Provisions* Eq. 6.1.4-1, 6.1.4-2, 6.1.4-3, or 6.1.4-4 [6.2-5, 6.2-6, 6.2-7, or 6.2-8].

13.4.3.4 Load Combinations

Combining ASME B31.1 Eq. 12A and 13A with modified IBC Eq. 1605.3.2.5 and 1605.3.2.6 yields the following:

For modified *IBC* Eq. 16-17

$$\frac{PD_o}{4t_n} + \beta\left(\frac{0.75iM_A}{Z}\right) \pm \frac{0.75iM_B}{Z} \le kS_h$$

$$\frac{iM_C}{Z} \le S_A + f\left(S_h - S_L\right)$$

and for modified *IBC* Eq. 16-18

$$\frac{PD_o}{4t_n} + 0.9\beta\left(\frac{0.75iM_A}{Z}\right) - \frac{0.75iM_B}{Z} \le kS_h$$

$$\frac{iM_C}{Z} \le S_A + f\left(S_h - S_L\right)$$

where:

$$\beta = \left(1 \pm \frac{0.2S_{DS}}{1.4}\right)$$

M_A = the resultant moment due to weight

M_B = the resultant moment induced by the design force $F_p/1.4$ where F_p is as defined by *Provisions* Eq. 6.1.3-1, 6.1.3-2, or 6.1.3-3 [6.2-1, 6.2-3, or 6.2-4].

M_C = the resultant moment induced by the design relative seismic displacement $D_p/1.4R_p$ where D_p is as defined by *Provisions* Eq. 6.1.4-1, 6.1.4-2, 6.1.4-3, or 6.1.4-4 [6.2-5, 6.2-6, 6.2-7, or 6.2-8].

S_{DS}, W_p, and R_p are as defined in the *Provisions*.

P, D_o, t_n, I, Z, k, S_h, S_A, f, and S_L are as defined in ASME B31.1.

Appendix A

THE BUILDING SEISMIC SAFETY COUNCIL

The purpose of the Building Seismic Safety Council is to enhance the public's safety by providing a national forum to foster improved seismic safety provisions for use by the building community. For the purposes of the Council, the building community is taken to include all those involved in the planning, design, construction, regulation, and utilization of buildings.

To achieve its purposes, the Council shall conduct activities and provide the leadership needed to:

1. Promote development of seismic safety provisions suitable for use throughout the United States;
2. Recommend, encourage, and promote adoption of appropriate seismic safety provisions in voluntary standards and model codes;
3. Assess implementation progress by federal, state, and local regulatory and construction agencies;
4. Identify opportunities for the improvement of seismic regulations and practices and encourage public and private organizations to effect such improvements;
5. Promote the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, elected officials, industry representatives, other members of the building community and the public.
6. Provide advice to governmental bodies on their programs of research, development, and implementation; and
7. Periodically review and evaluate research findings, practice, and experience and make recommendations for incorporation into seismic design practices.

The scope of the Council's activities encompasses seismic safety of structures with explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations.

Achievement of the Council's purpose is important to all in the public and private sectors. Council activities will provide an opportunity for participation by those at interest, including local, State, and Federal Government, voluntary organizations, business, industry, the design professions, the construction industry, the research community and the public. Regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach adaptable to the relative risk, resources and capabilities of each community. The Council recognizes that appropriate earthquake hazard reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated into their legislation, regulations, practices, rules, codes, relief procedures and loan requirements, whenever possible, so that these measures and initiatives become part of established activities rather than being superposed as separate and additional.

The Council is established as a voluntary advisory, facilitative council of the National Institute of Building Sciences, a nonprofit corporation incorporated in the District of Columbia, under the authority given the Institute by the Housing and Community Development Act of 1974, (Public Law 93-383), Title VIII, in furtherance of the objectives of the Earthquake Hazards Reduction Act of 1977 (Public Law 95-124) and in support of the President's National Earthquake Hazards Reduction Program, June 22, 1978.

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(August 2006)