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).

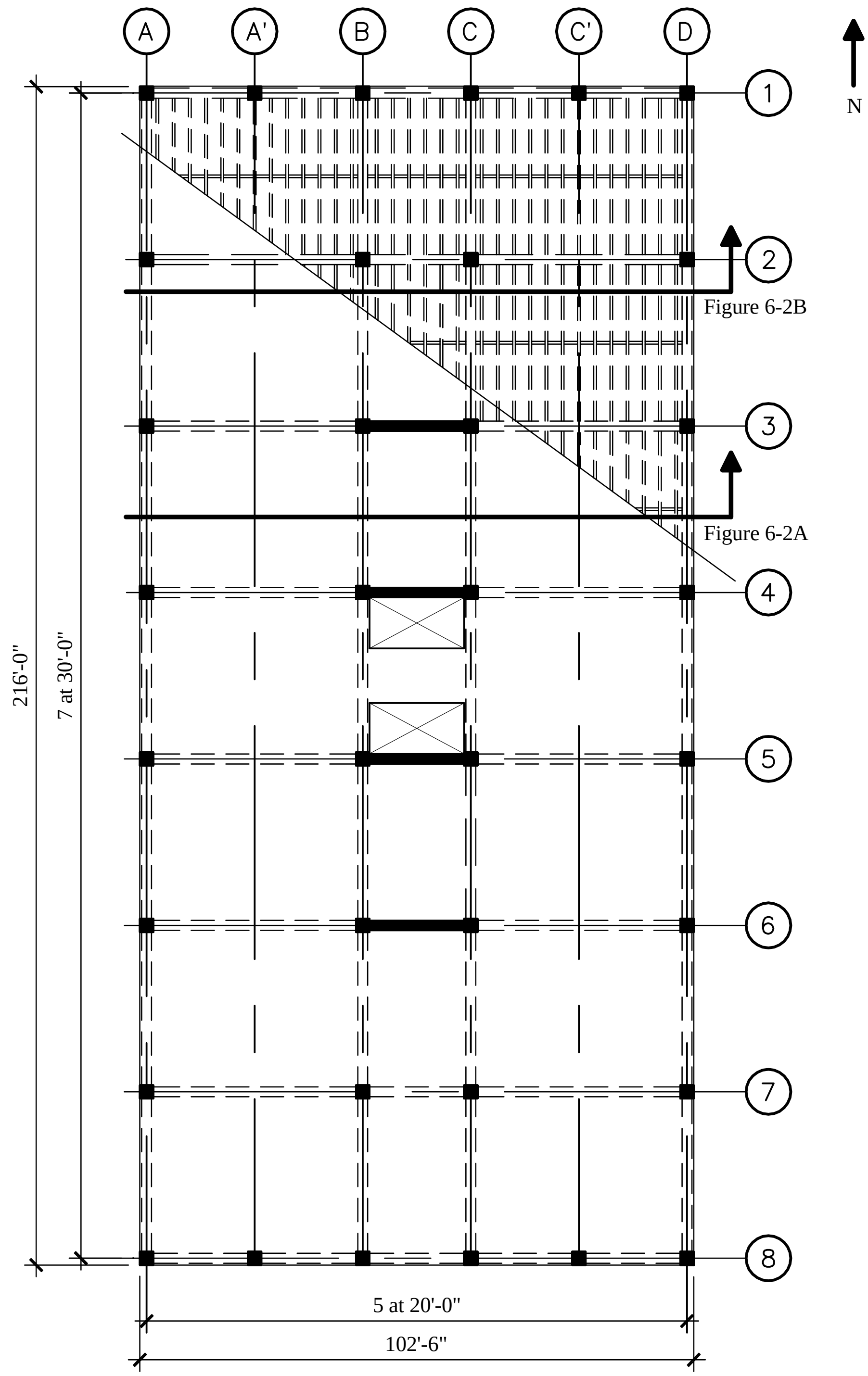
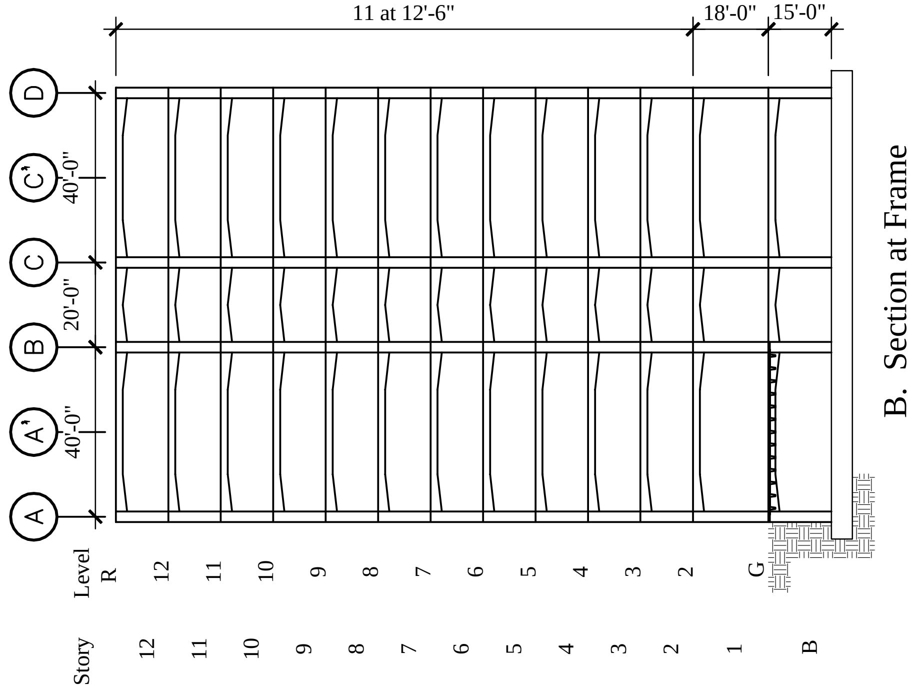


Figure 6-1 Typical floor plan of the Berkeley building. The Honolulu building is similar but without structural walls (1.0 ft = 0.3048 m).



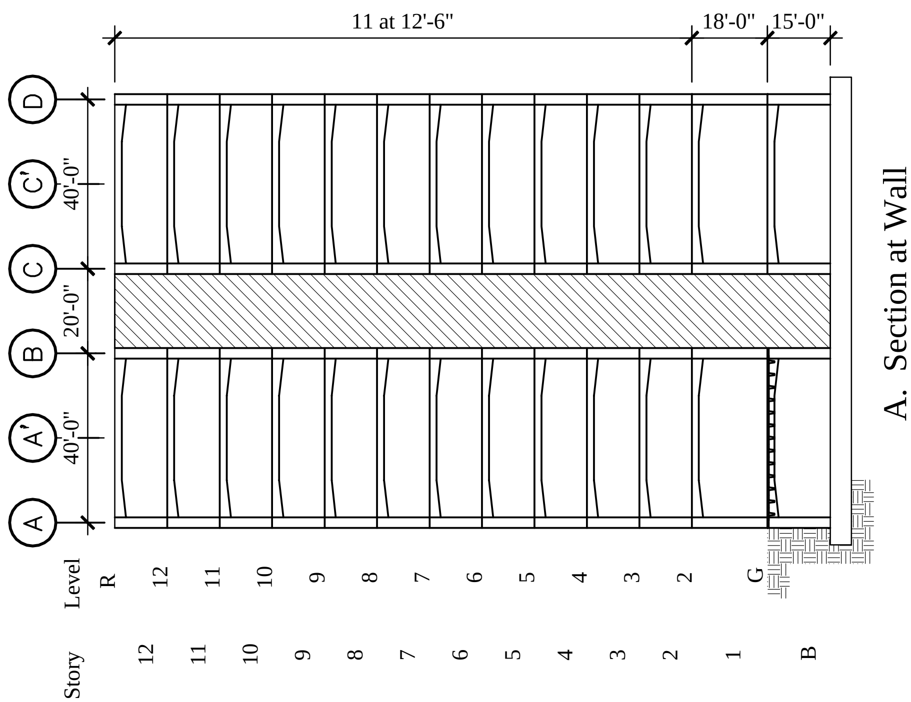


Figure 6-2 Typical elevations of the Berkeley building; the Honolulu building is similar but without structural walls (1.0 ft = 0.3048 m).

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 x 1.65 = 1.65
$S_{M1} = F_v S_1$ = 1.3 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 x 0.61 = 0.708
$S_{M1} = F_v S_1$ = 1.62 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** N-S	E-W	Description
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** N-S	E-W	Description
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 N-S	Berkeley E-W	Honolulu N-S	Honolulu 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.

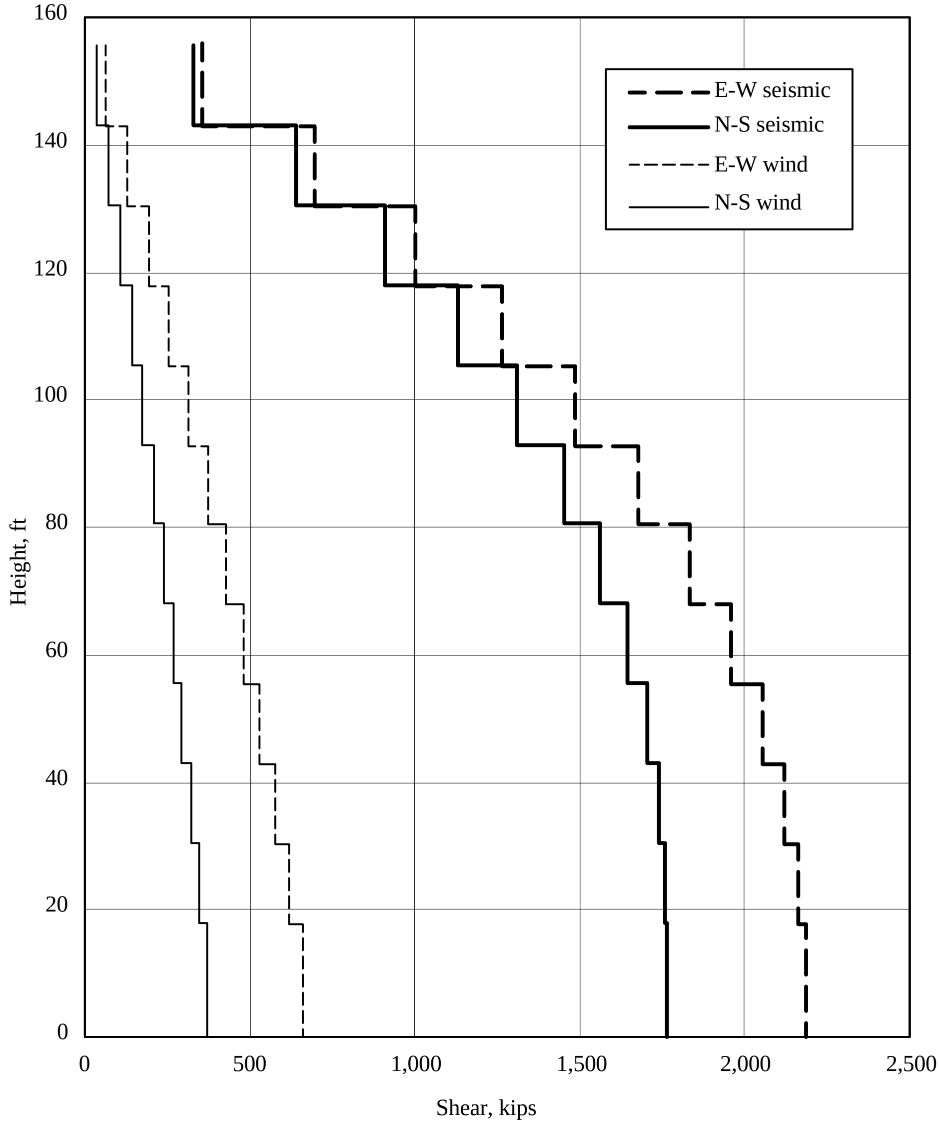


Figure 6-3 Comparison of wind and seismic story shears for the Berkeley building (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

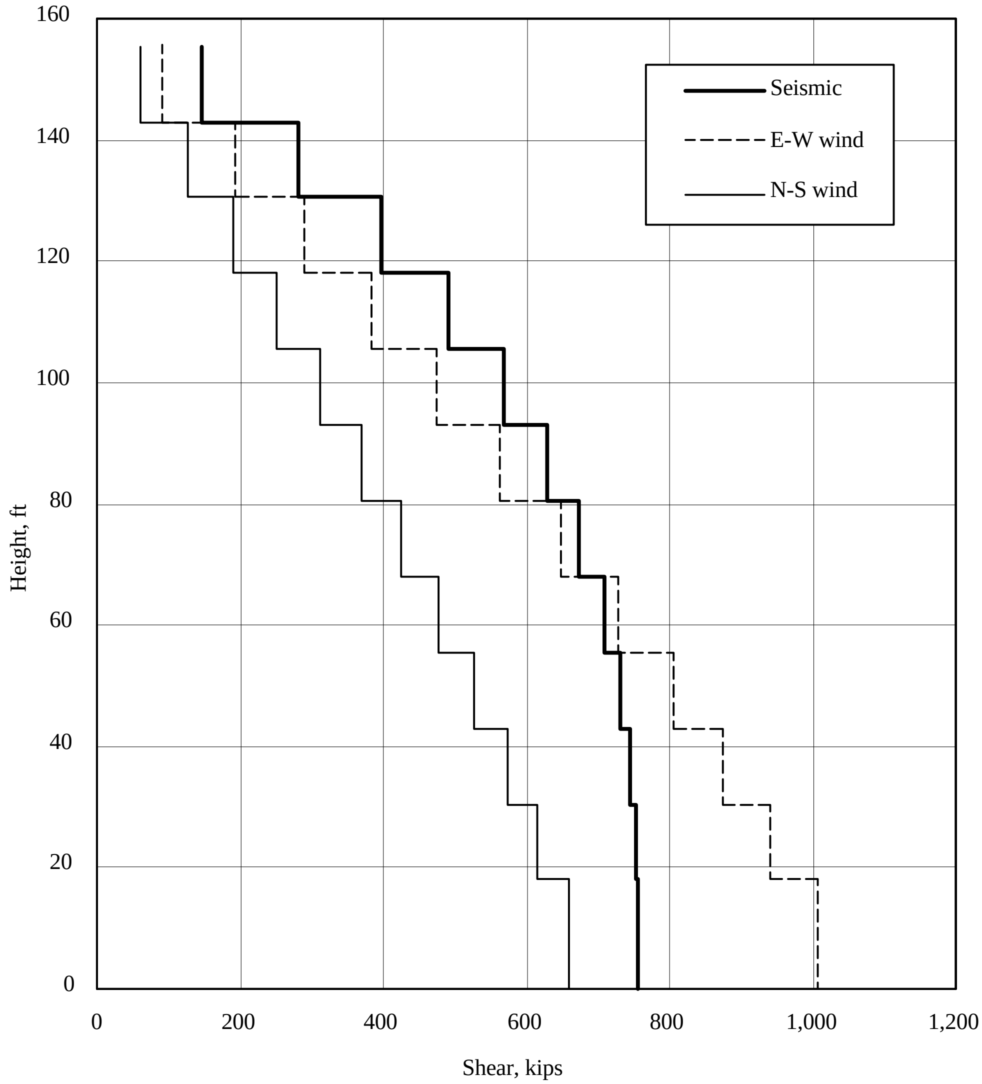


Figure 6-4 Comparison of wind and seismic story shears for the Honolulu building (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

6.3 DRIFT AND P-DELTA EFFECTS

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

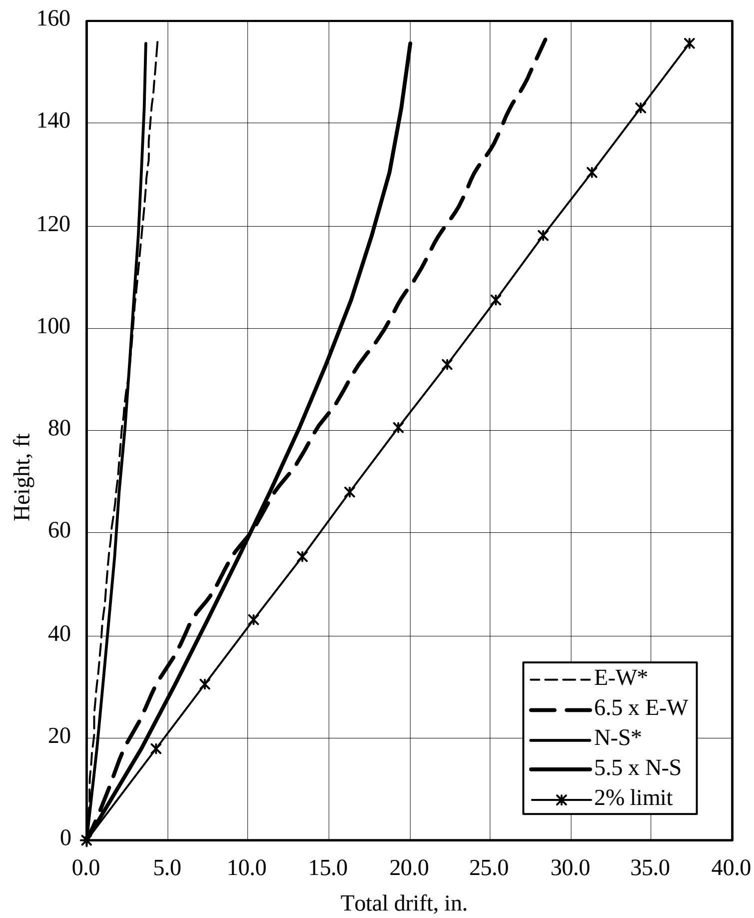
Drift and P-delta effects are checked according to *Provisions* Sec. 5.2.8 [5.2.6.1] and 5.4.6 [5.2.6.2], respectively. According to *Provisions* Table 5.2.8 [4.5-1], the story drift limit for this Seismic Use Group I building is $0.020h_{sx}$ where h_{sx} is the height of story x. This limit may be 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:

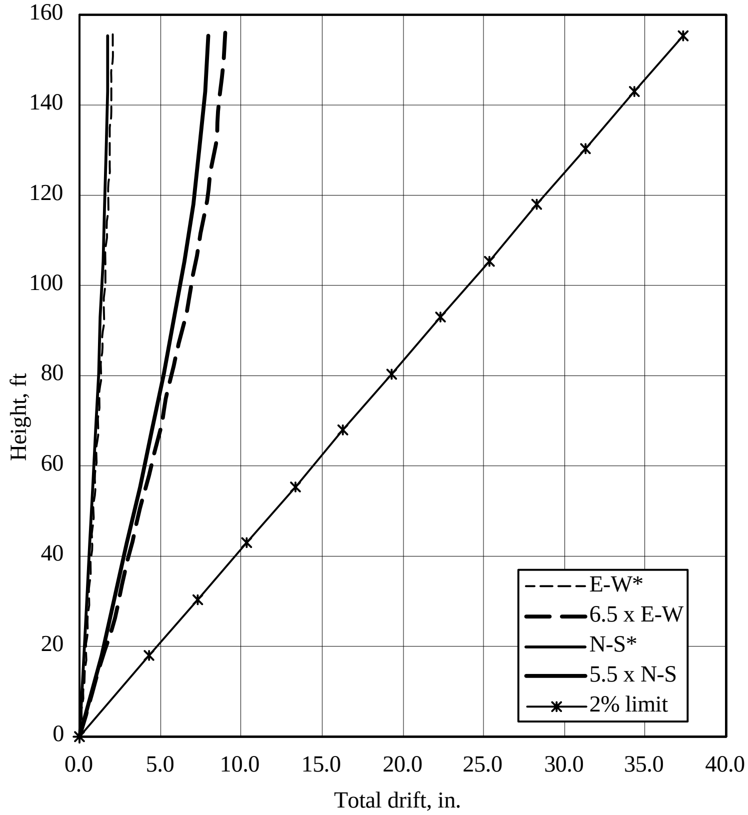
Total displacement at center of mass = δ_{avg} = 4.360 in. (see Table 6-9b)
Rotation at center of mass = 0.000648 radians
Maximum displacement at corner of floor plate = d_{max} = 4.360 + 0.000648(216)(12)/2 = 5.200 in.
Ratio d_{max}/d_{avg} = 5.200/4.360 = 1.19 < 1.20, so no torsional irregularity exists.

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 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 $\varphi = 0.85$]

6.4.3 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.

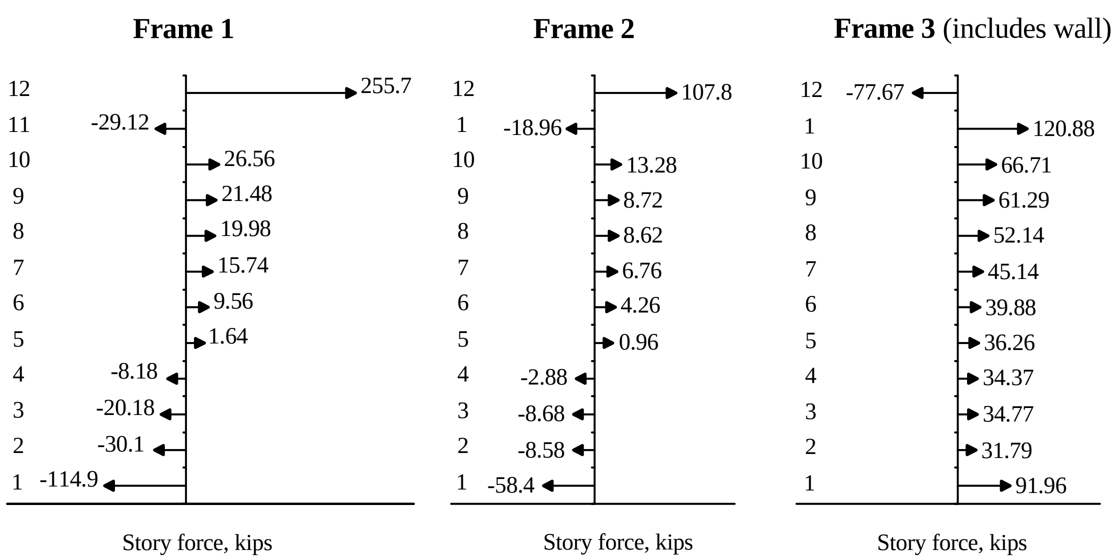


Figure 6-7 Story forces in the E-W direction (1.0 kip = 4.45 kN).

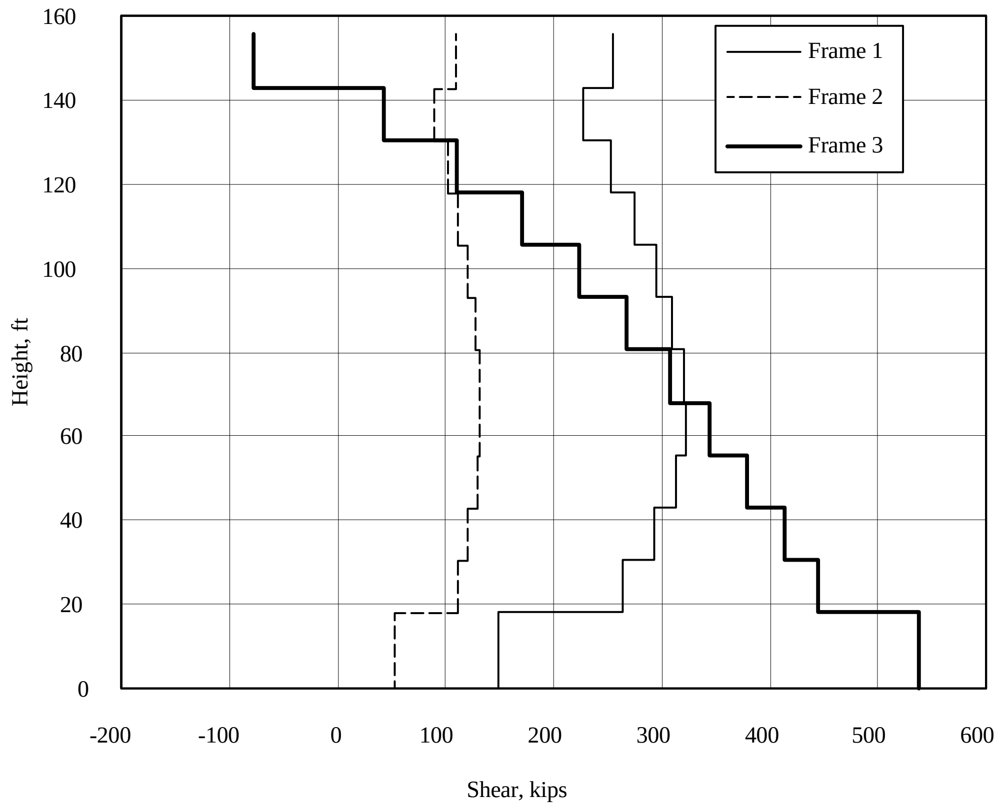


Figure 6-8 Story shears in the E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

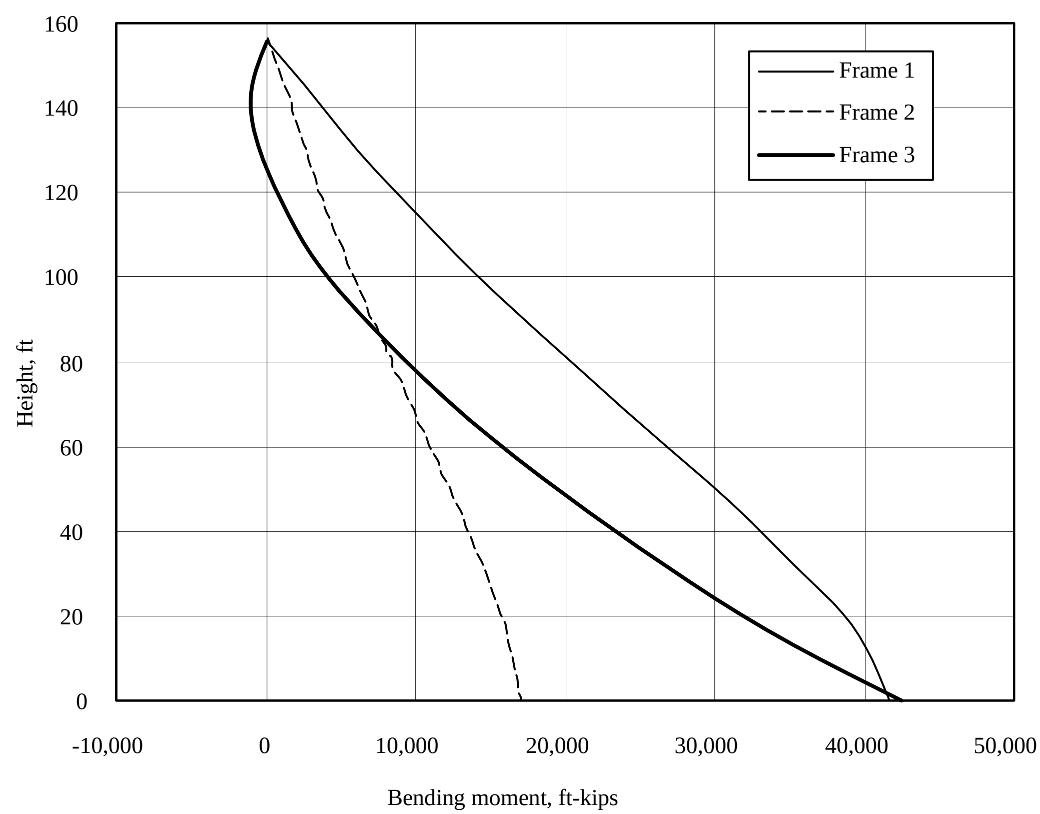


Figure 6-9 Story overturning moments in the E-W direction (1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m).

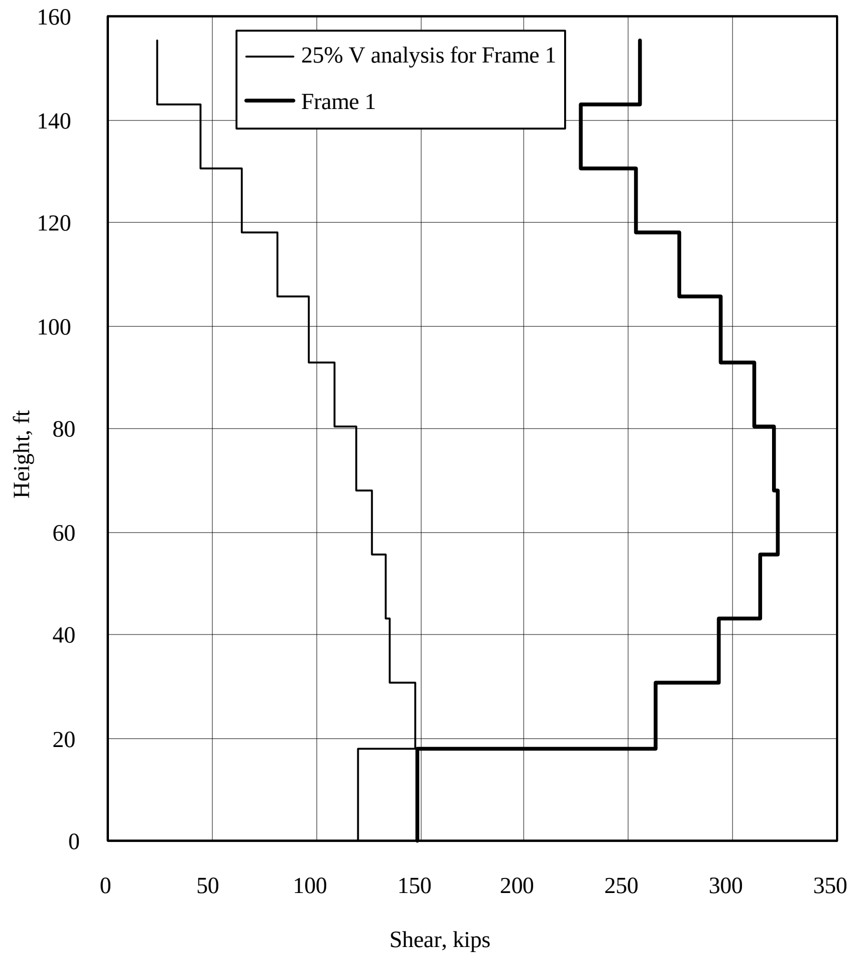


Figure 6-10 25 percent story shears, Frame 1 E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

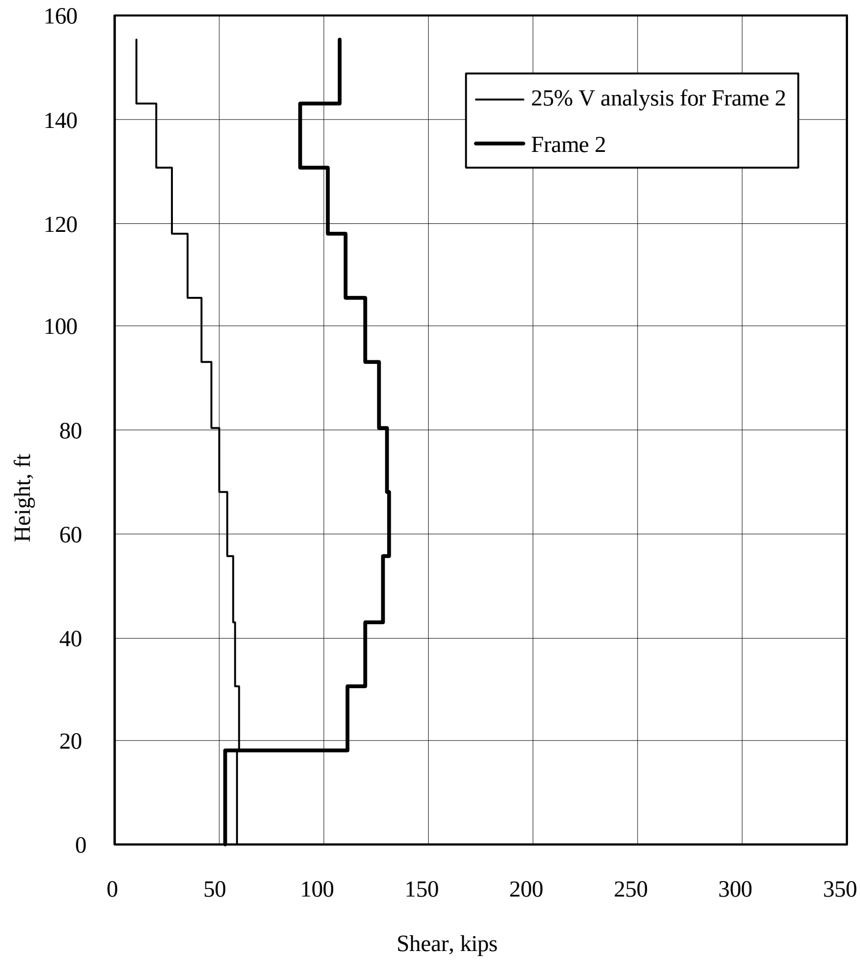


Figure 6-11 25 percent story shears, Frame 2 E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN).

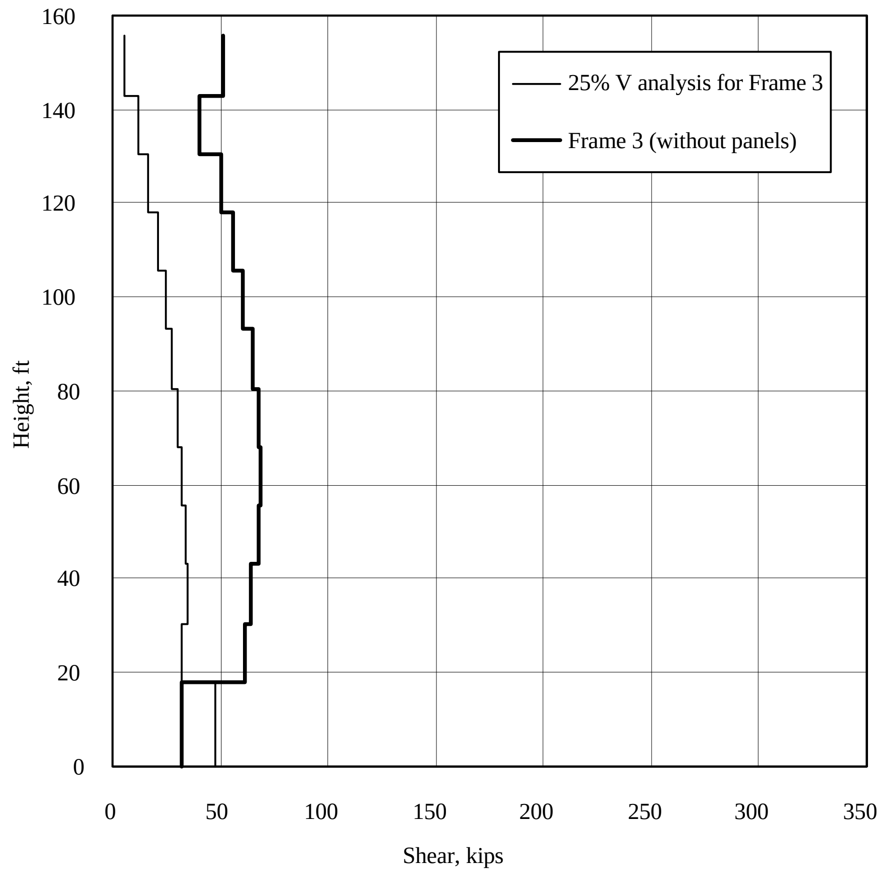


Figure 6-12 25 percent story shear, Frame 3 E-W direction (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN)..

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.

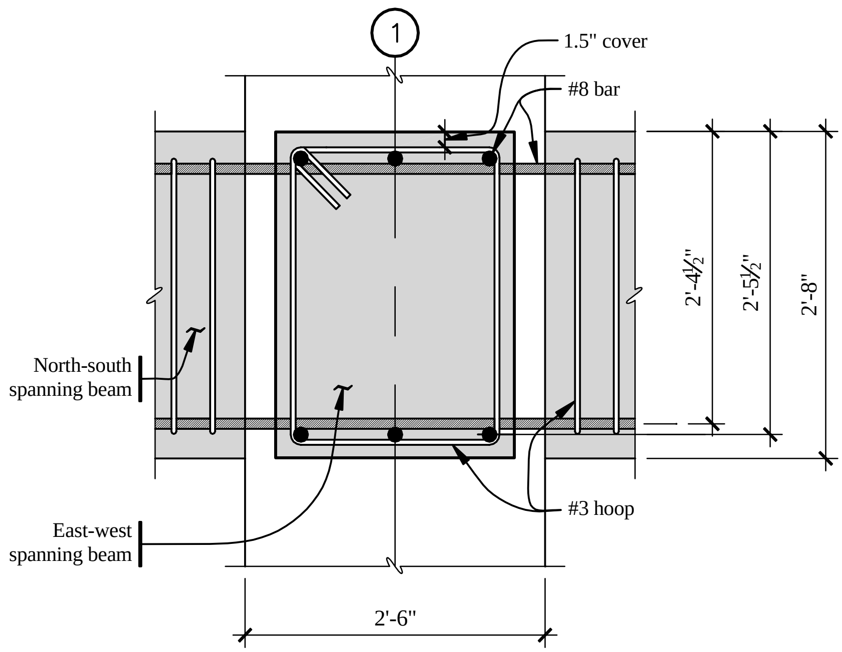


Figure 6-13 Layout for beam reinforcement (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm).

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.

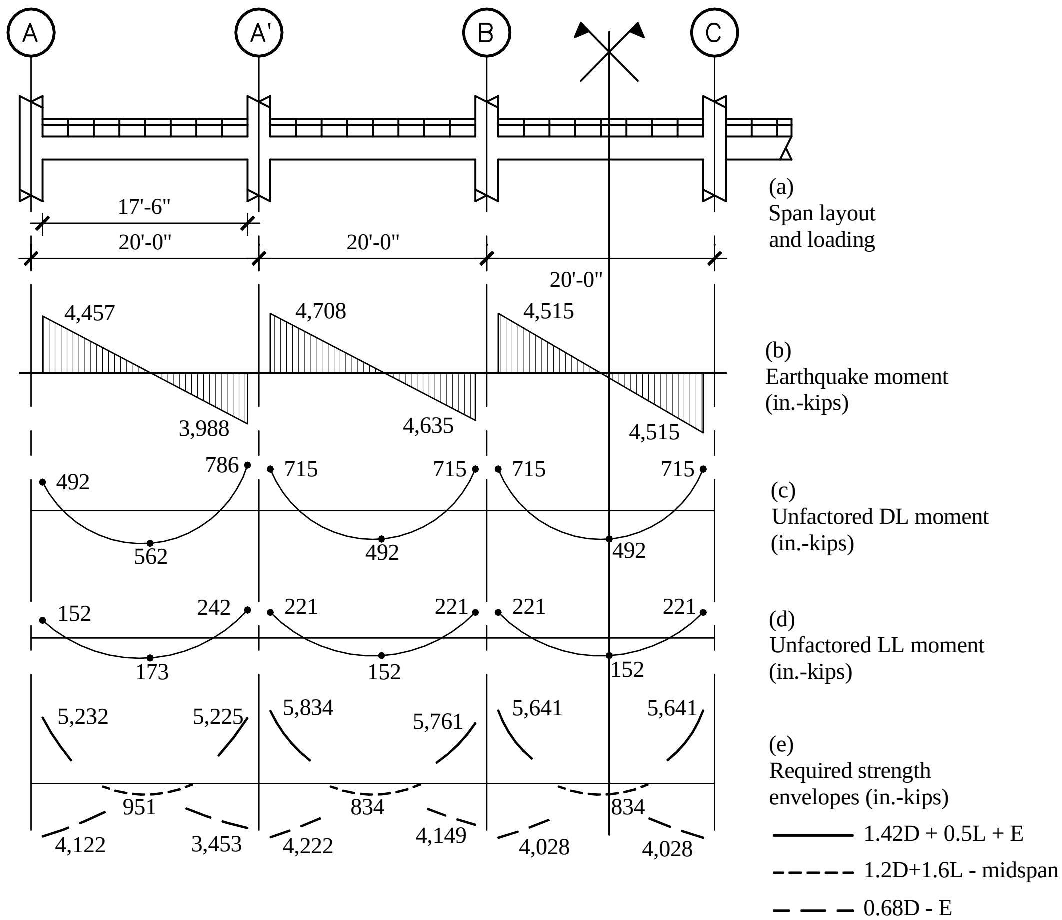


Figure 6-14 Bending moments for Frame 1 (1.0 ft = 0.3048 m, 1.0 in.-kip = 0.113 kN-m).

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.

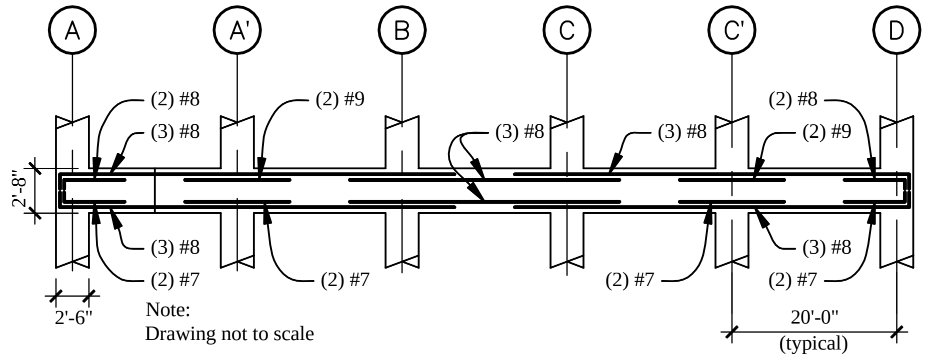


Figure 6-15 Preliminary rebar layout for Frame 1 (1.0 ft = 03.048 m).

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

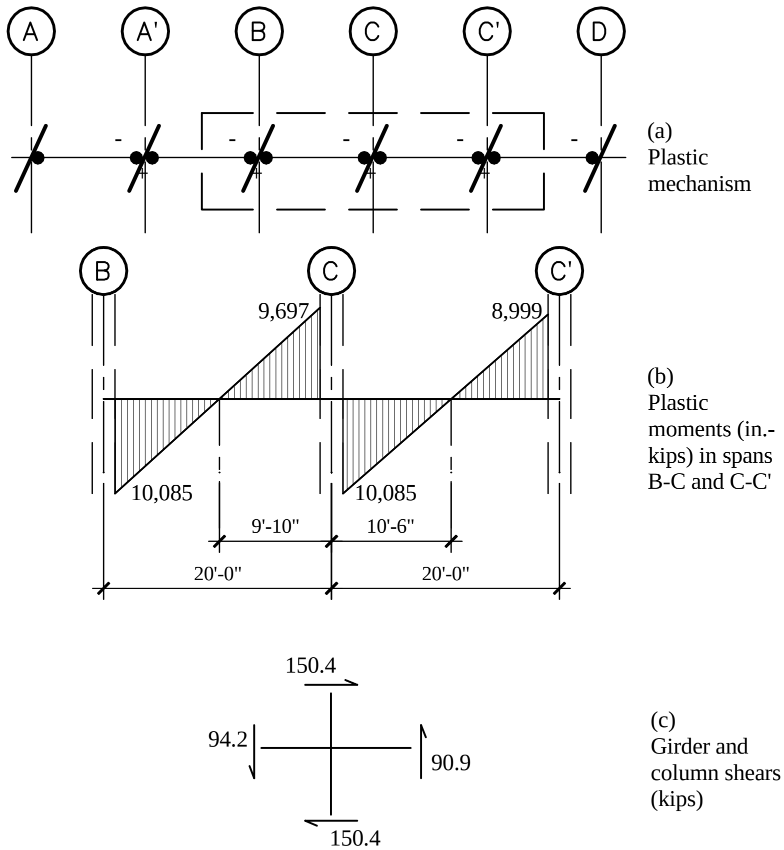


Figure 6-16 Diagram for computing column shears (1.0 ft = 0.3048 m, 1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

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:

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

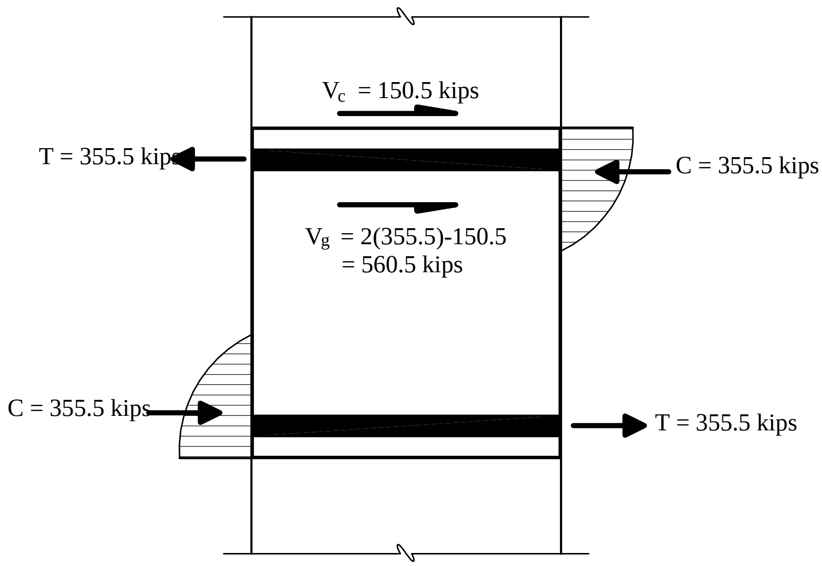


Figure 6-17 Computing joint shear stress (1.0 kip = 4.45kN).

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.

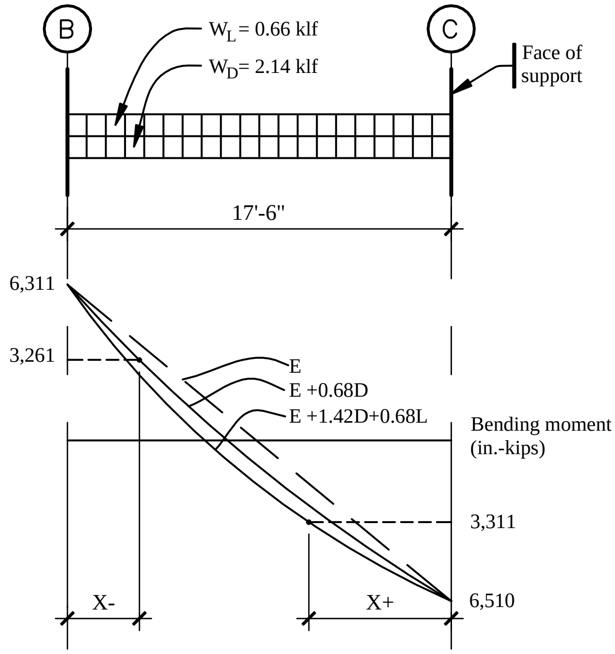


Figure 6-18 Loading for determination of rebar cutoffs (1.0 ft = 0.3048 m, 1.0 klf = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

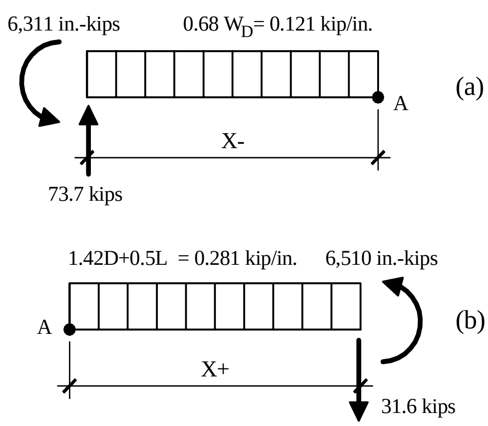


Figure 6-19 Free body diagrams (1.0 kip = 4.45kN, 1.0 klf = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

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:

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

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:

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

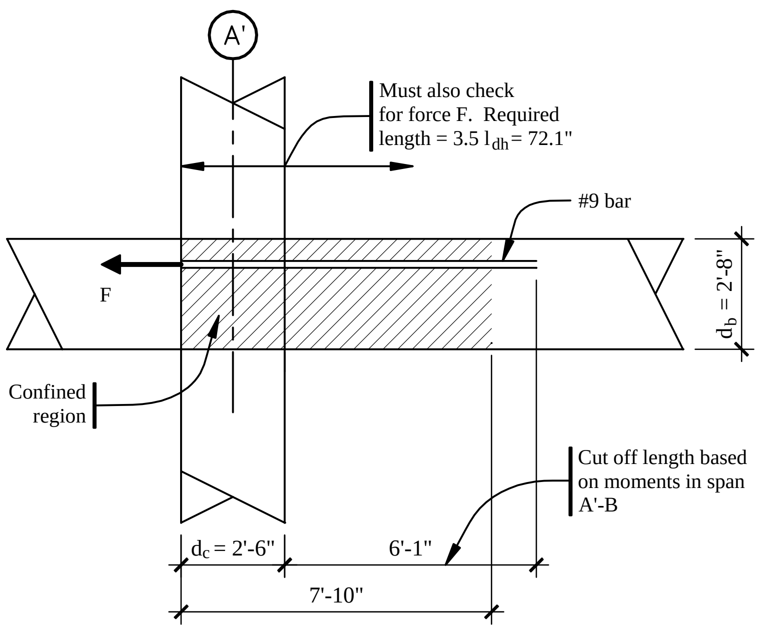


Figure 6-20 Development length for top bars (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm).

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.

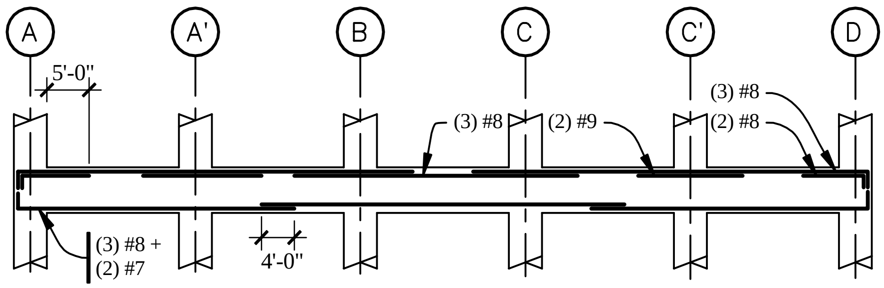


Figure 6-21 Final bar arrangement (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm).

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.

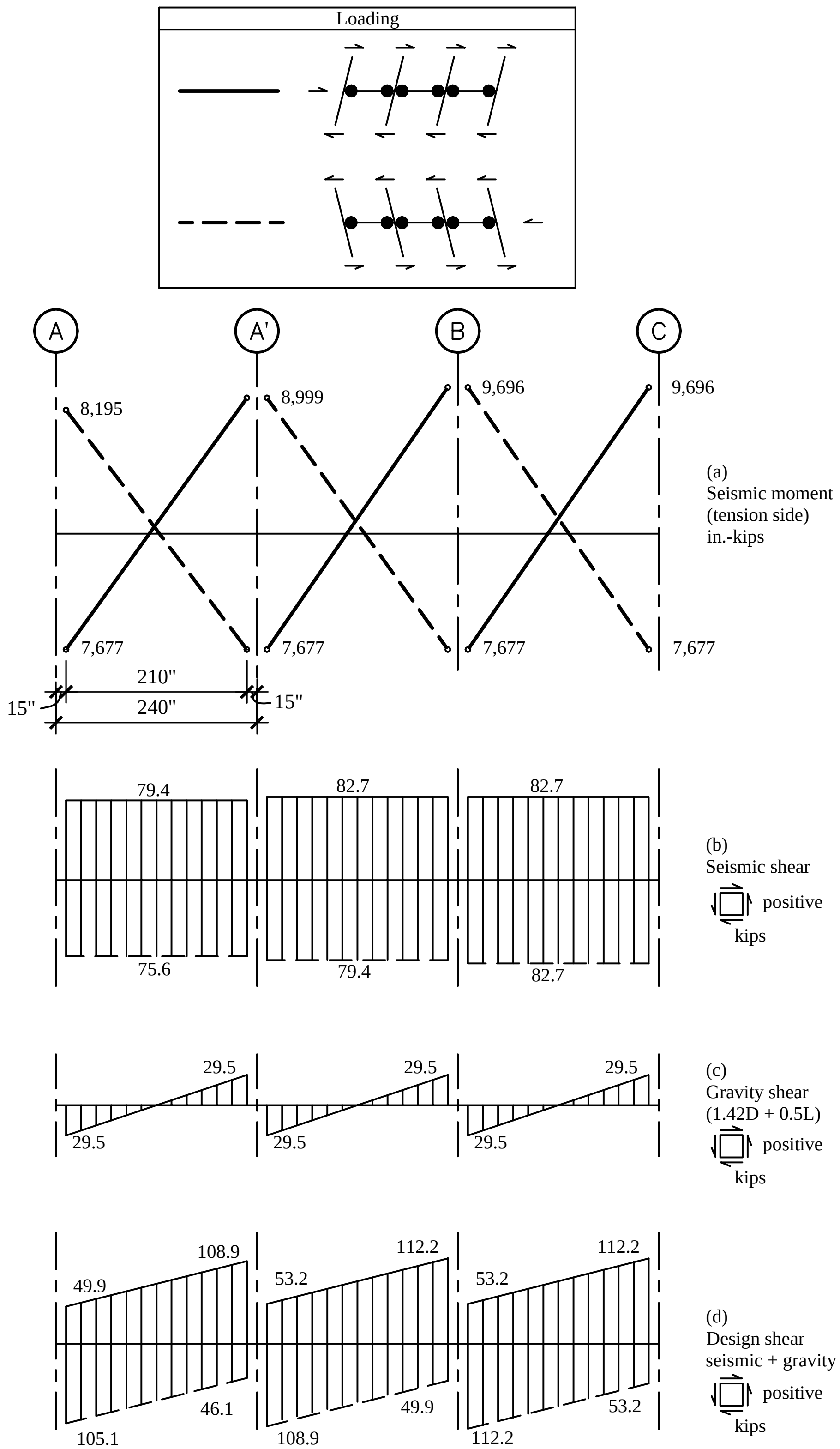


Figure 6-22 Shear forces for transverse reinforcement (1.0 in = 25.4 mm, 1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

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.

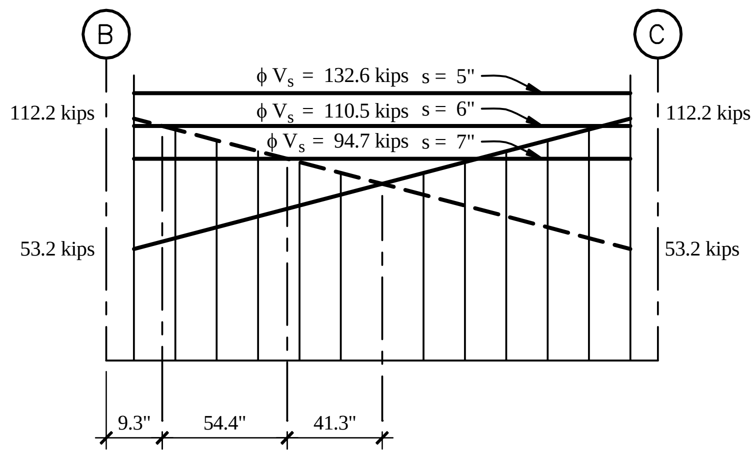


Figure 6-23 Detailed shear force envelope in Span B-C (1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

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.

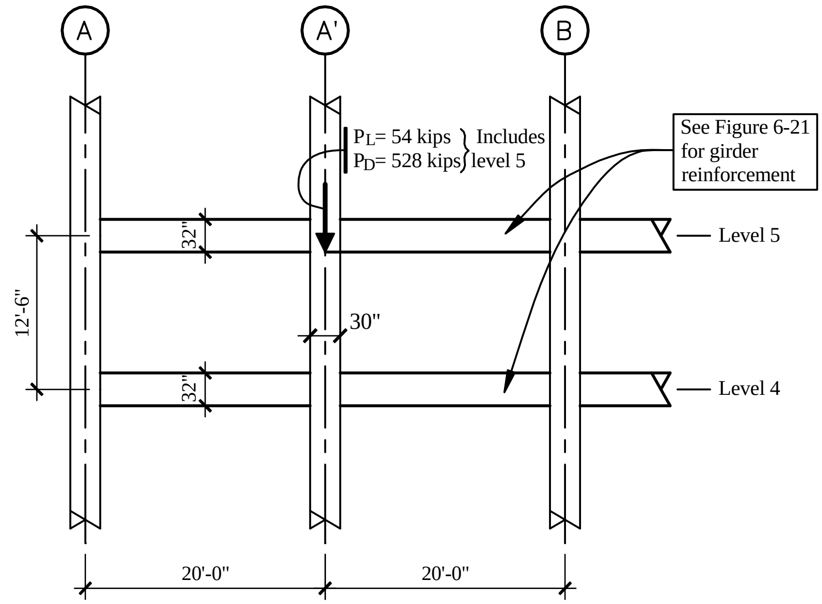


Figure 6-24 Layout and loads on column of Frame A (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

6.4.5.3.1 Longitudinal Reinforcement

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

1.42D + 0.5L + 1.0E
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.

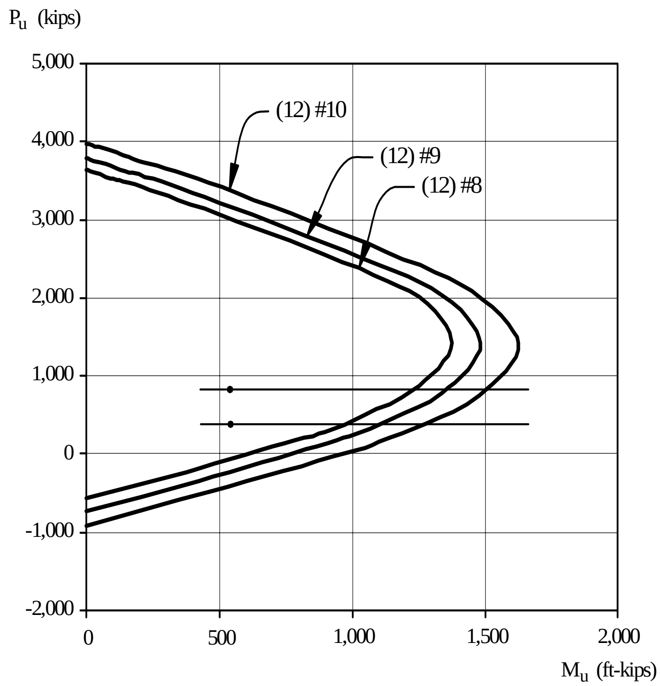


Figure 6-25 Design interaction diagram for column on Gridline A' (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

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:

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

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

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

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.

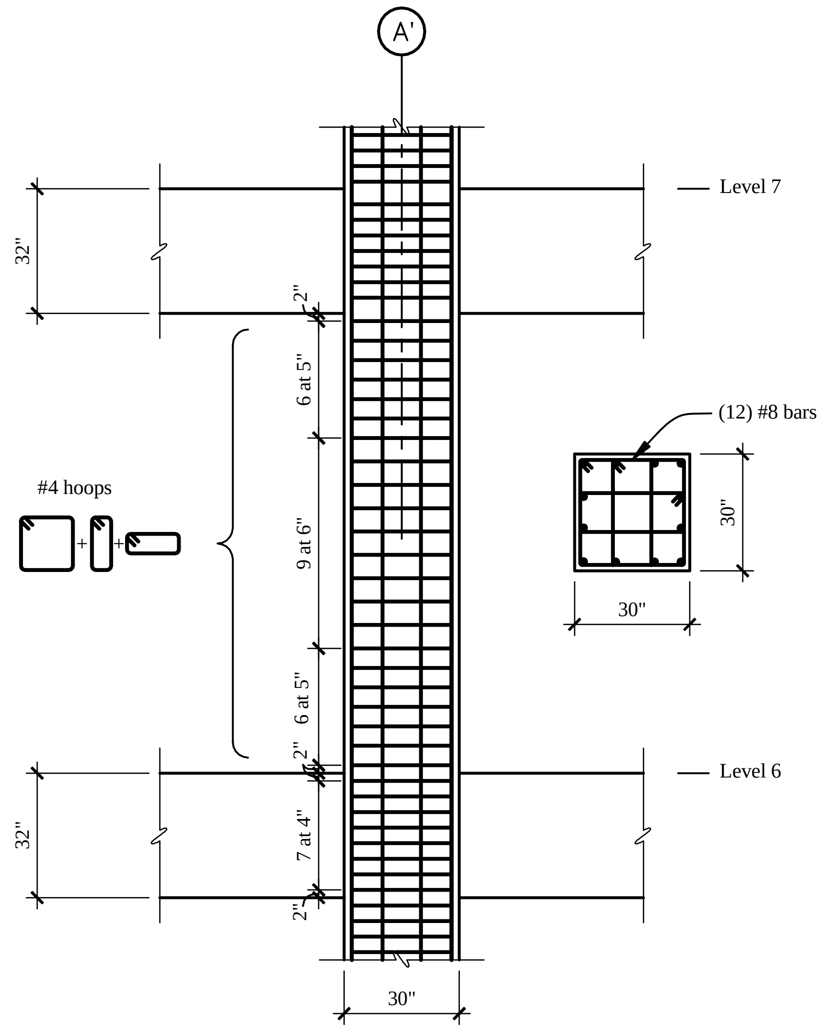


Figure 6-26 Details of reinforcement for column (1.0 in = 25.4 mm).

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.

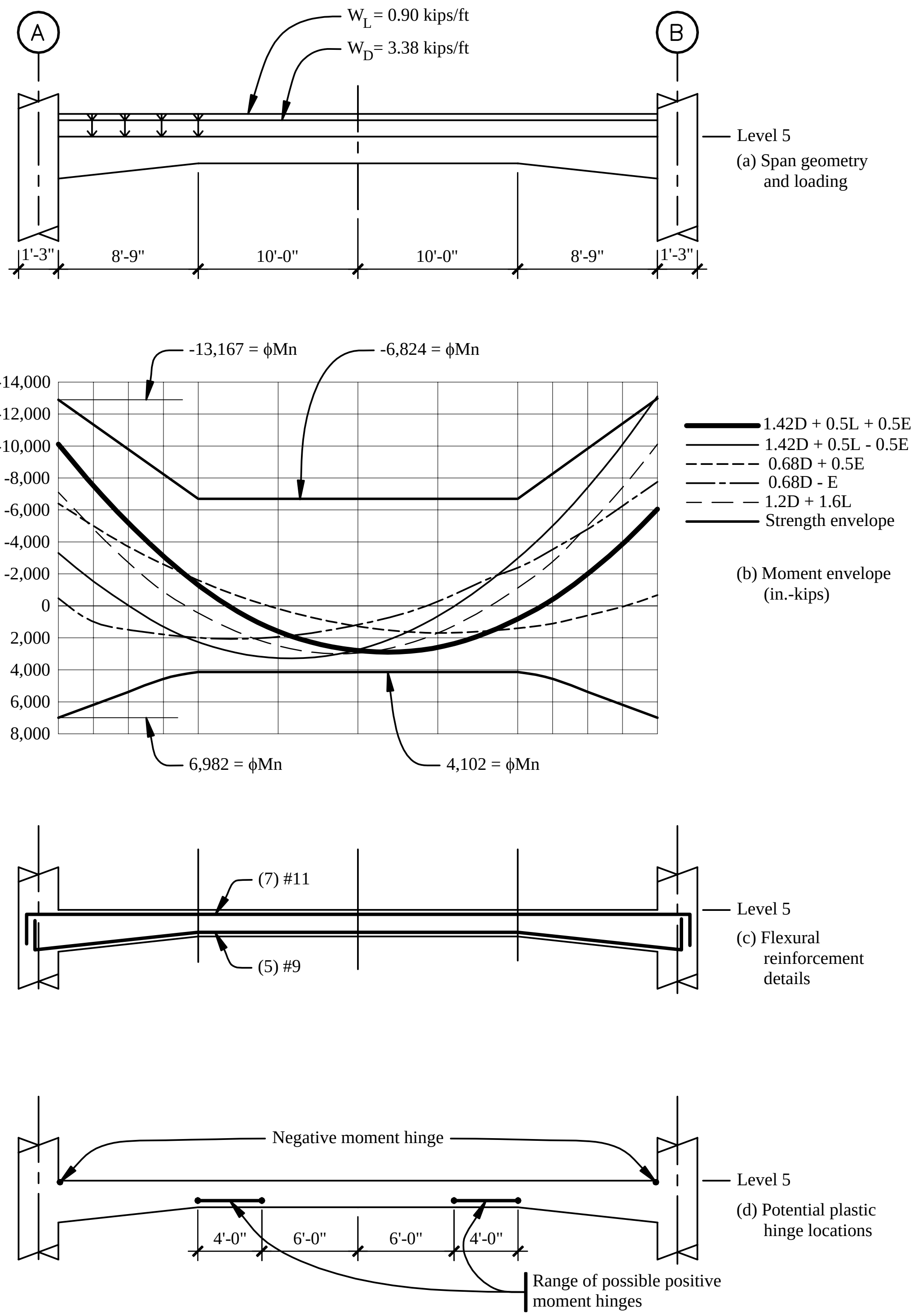


Figure 6-27 Design forces and detailing of haunched girder (1.0 ft = 0.3048 m, 1.0 k/ft = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

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 × 1.54 = 10.92 in.^{2}
d = 32- 1.5 - 3/8 - 1.41/2 = 29.4 in.
ρ = 10.92/(29.4 × 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, ϕM_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.
ρ = 5.00/(29.6 × 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.
ϕM_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.
ρ =5.00/(17.6 × 22.5) = 0.0126
b = 86.5 in.
a = [5.00 (60)]/[0.85 (4) 86.5] = 1.02 in.
ϕM_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 (ρ = 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 ρ = 0.028, which is a violation of ACI 21.3.2.1 (ρ_{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.

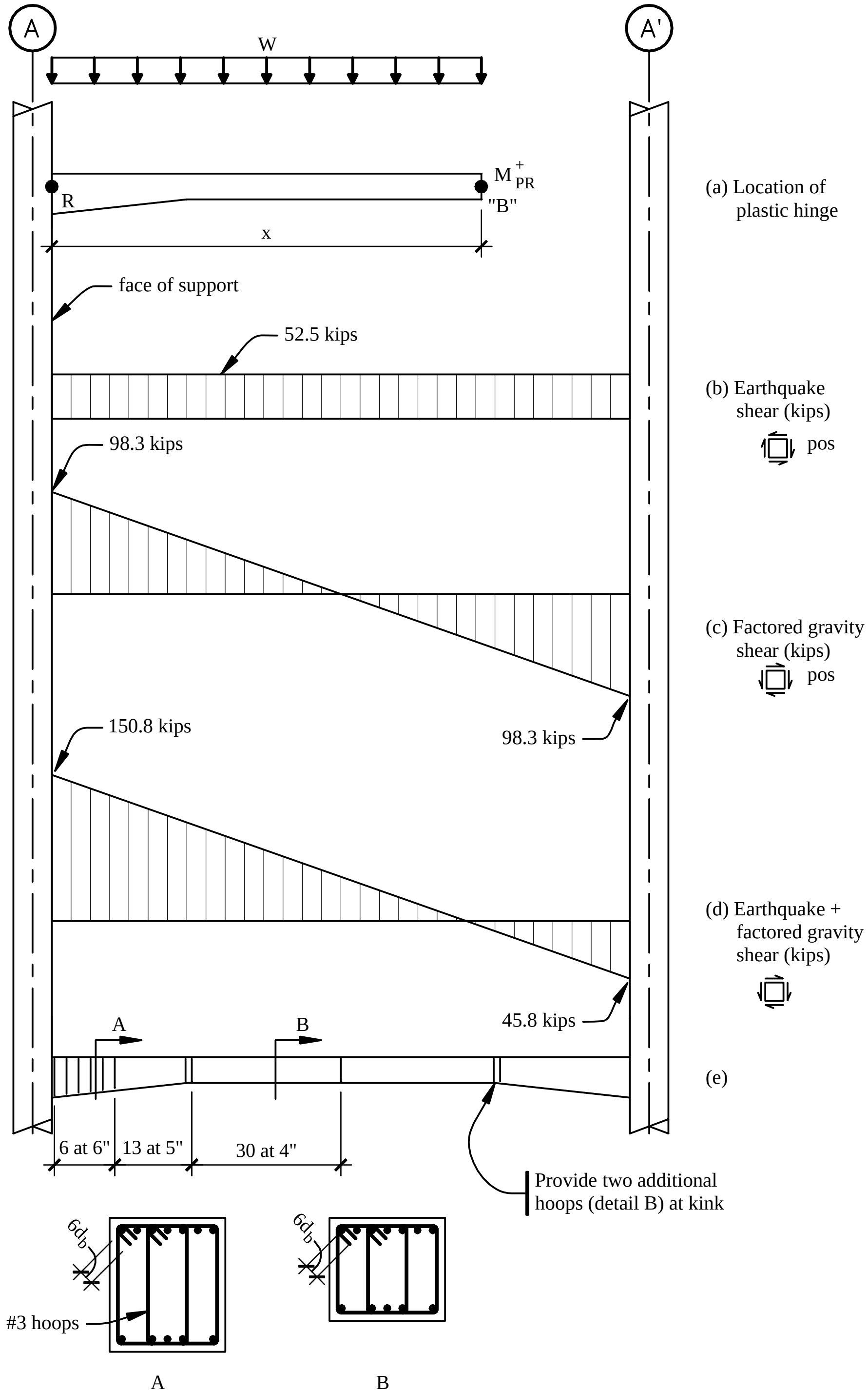


Figure 6-28 Computing shear in haunched girder (1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

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	kips
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

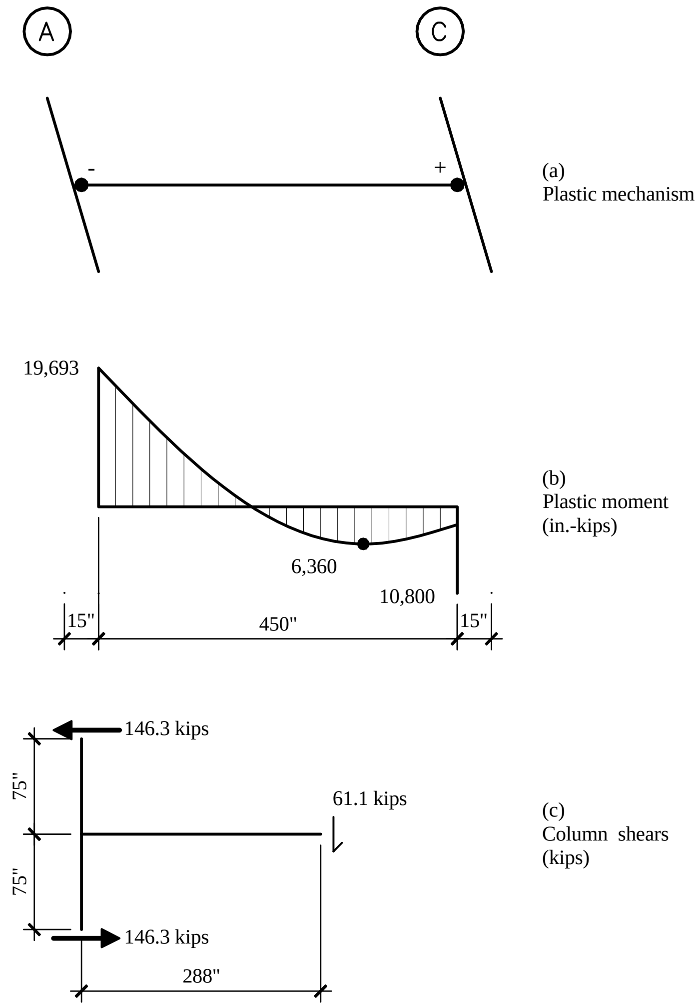


Figure 6-29 Computation of column shears for use in joint design (1.0 in = 25.4 mm, 1.0 kip = 4.45 kN).

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.

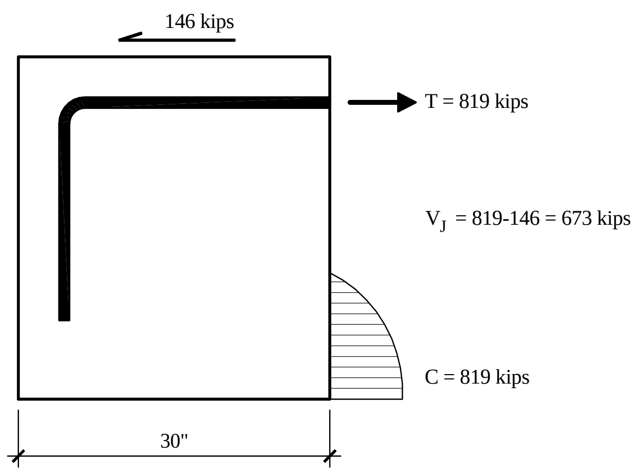


Figure 6-30 Computing joint shear force (1.0 kip = 4.45kN).

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:

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

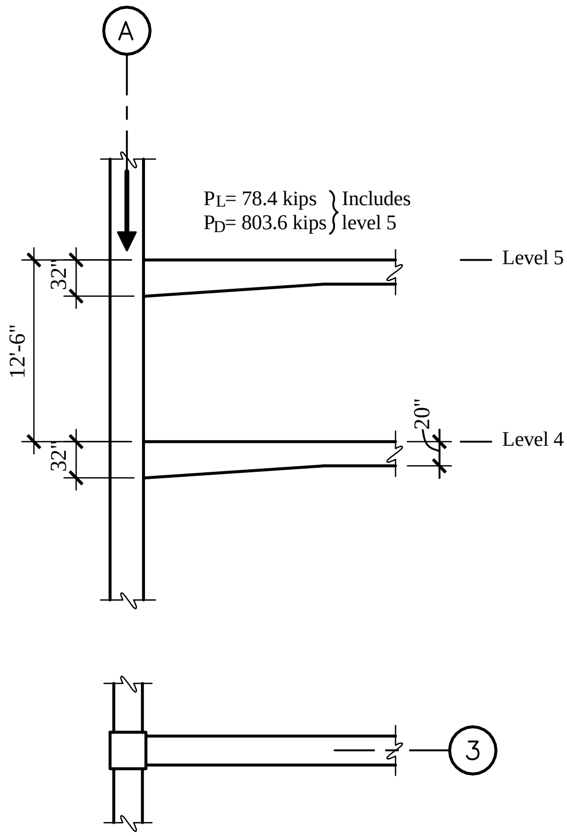


Figure 6-31 Column loading (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

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.

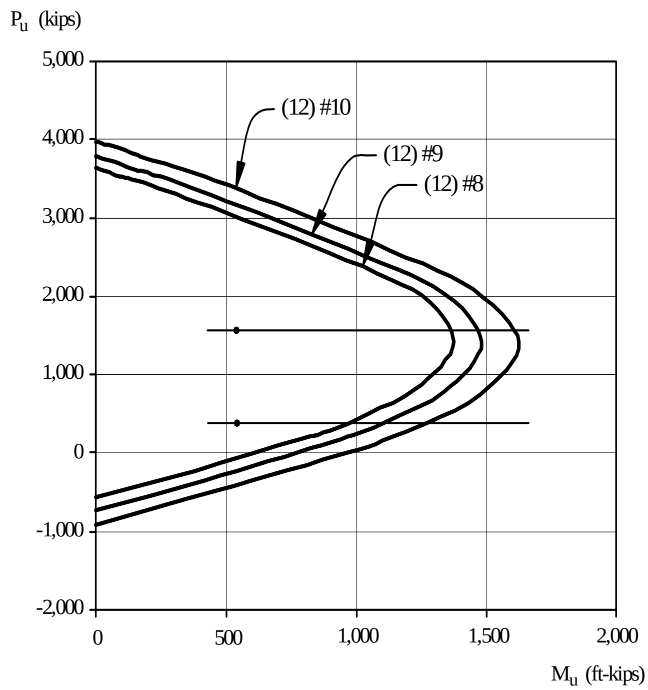


Figure 6-32 Interaction diagram and column design forces (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

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.

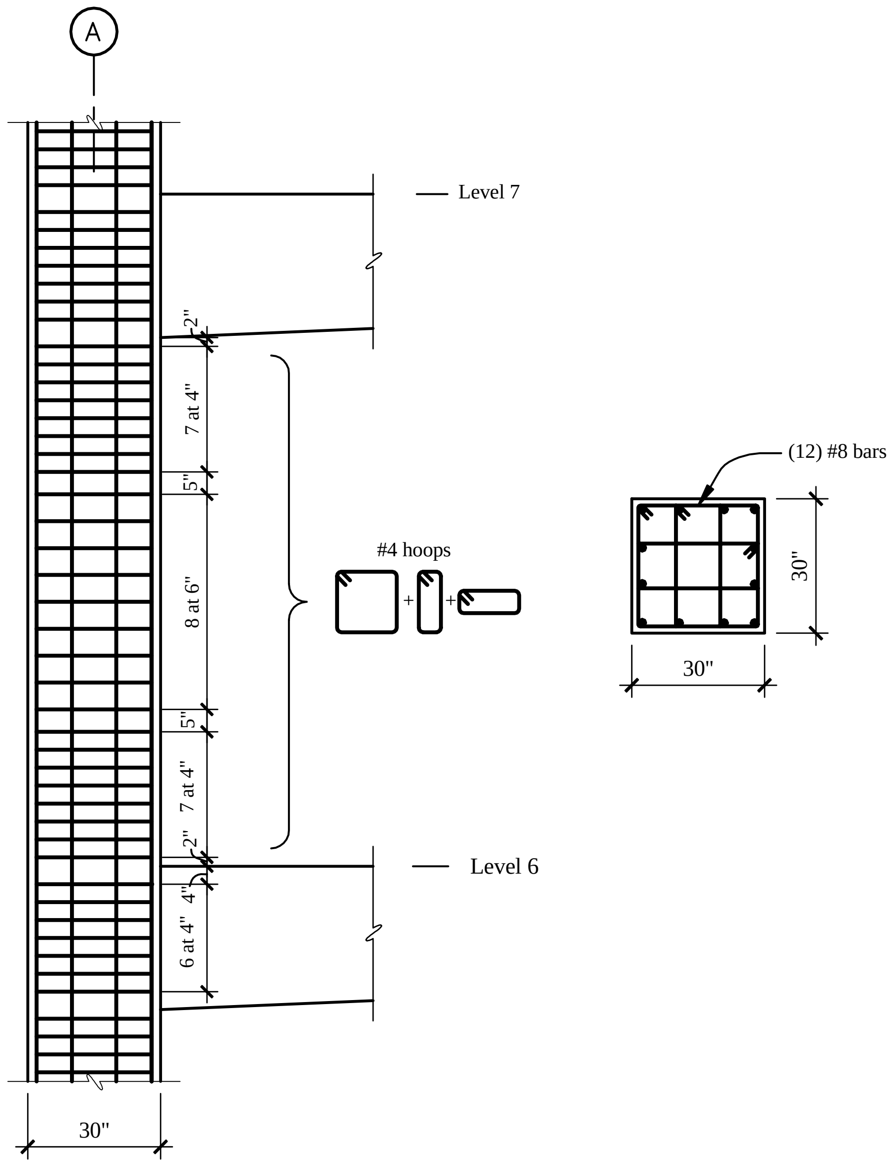


Figure 6-33 Column detail (1.0 in = 25.4 mm).

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:

$P_u = 1.42D + 0.5L = 1{,}800((1.42 \times 0.16) + 0.50\text{x}0.02)) = 427$ kips for maximum compression
$P_u = 0.68D = 1{,}800(0.68 \times 0.16) = 196$ 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.

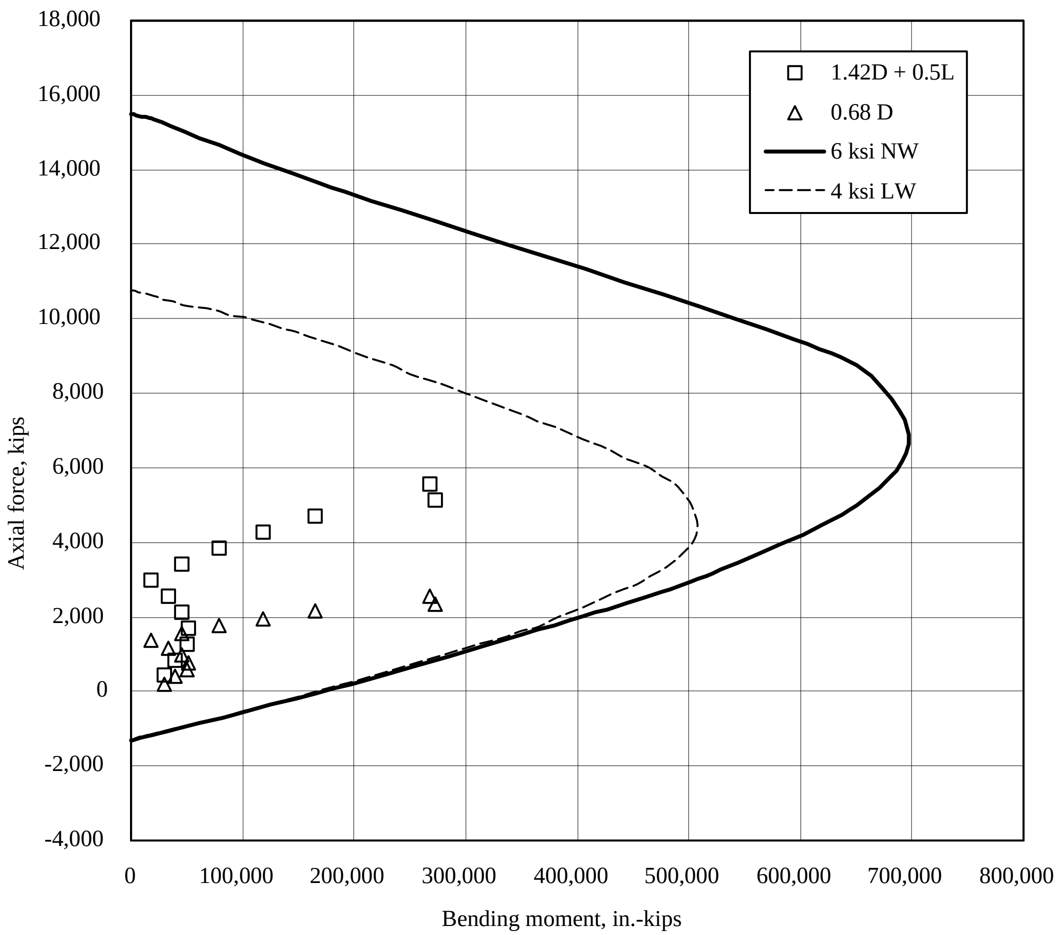


Figure 6-34 Interaction diagram for structural wall (1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

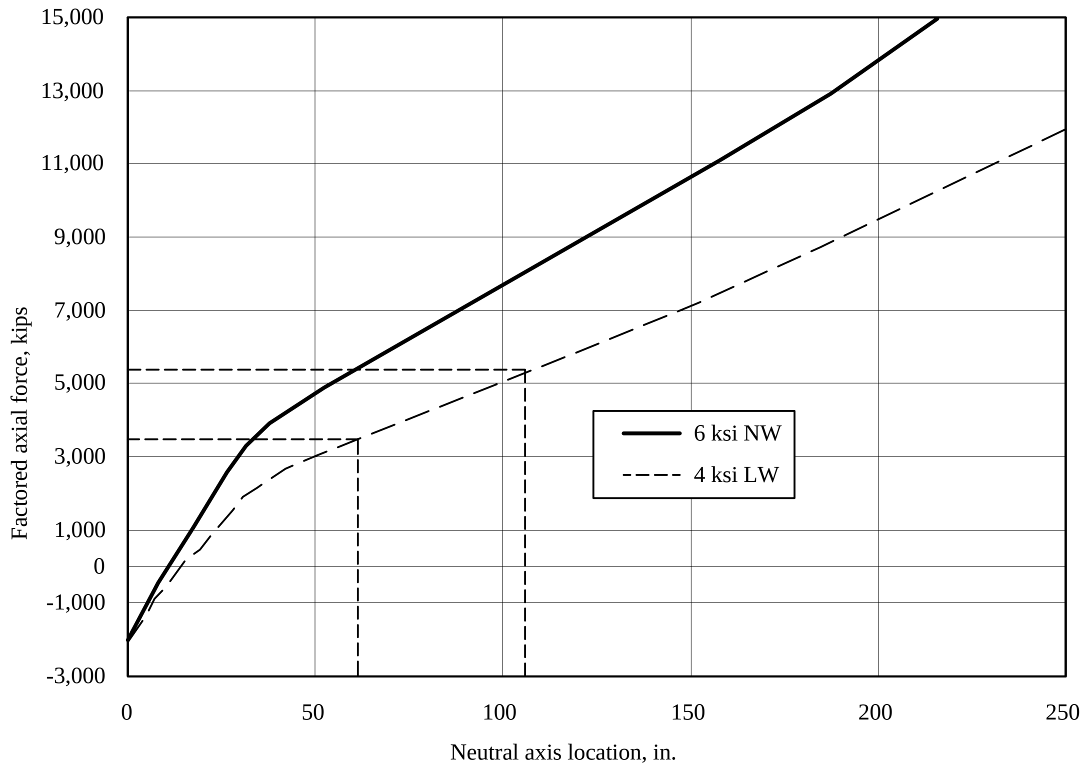


Figure 6-35 Variation of neutral axis depth with compressive force (1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

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.

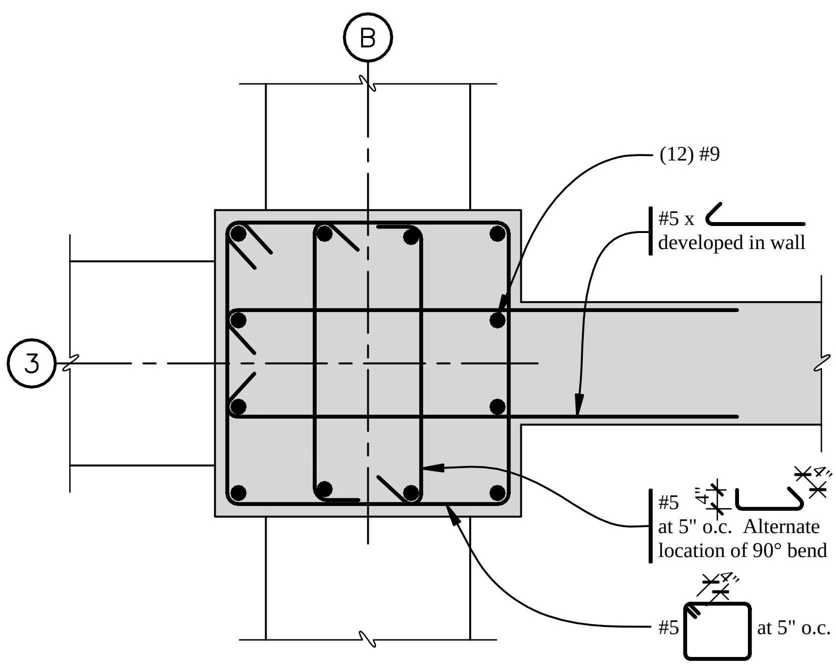


Figure 6-36 Details of structural wall boundary element (1.0 in = 25.4 mm).

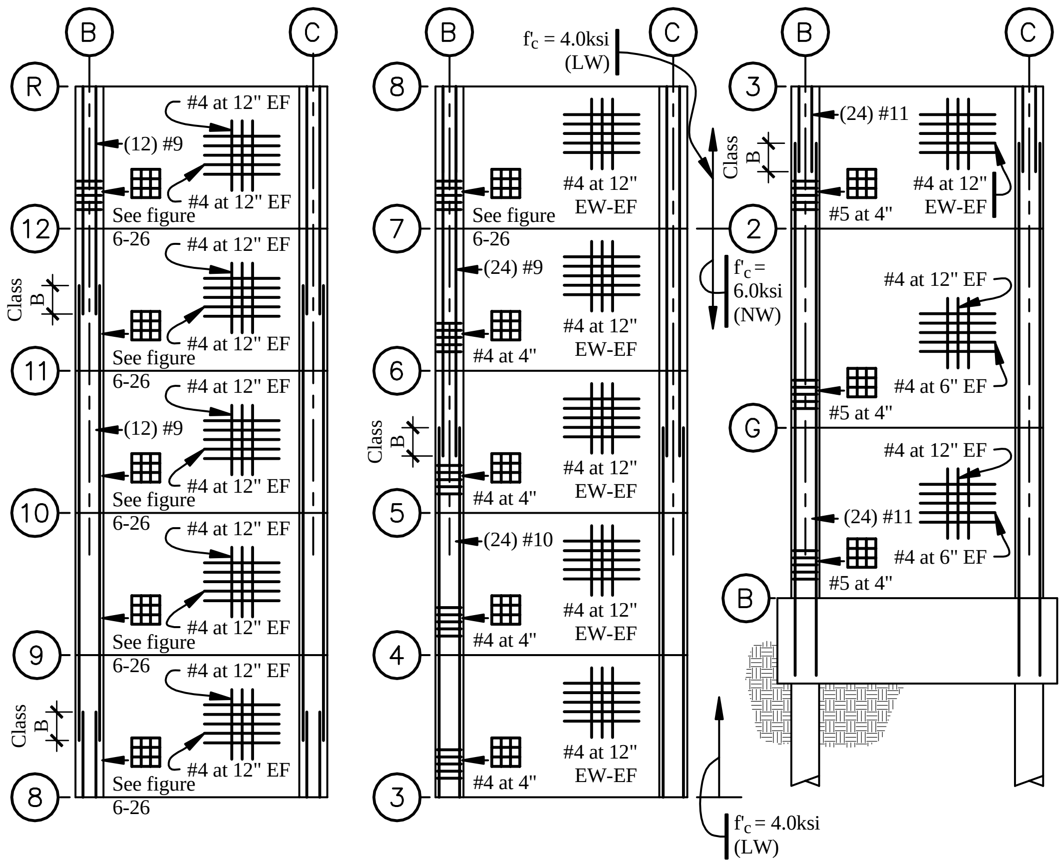


Figure 6-37 Overall details of structural wall (1.0 in = 25.4 mm).

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.

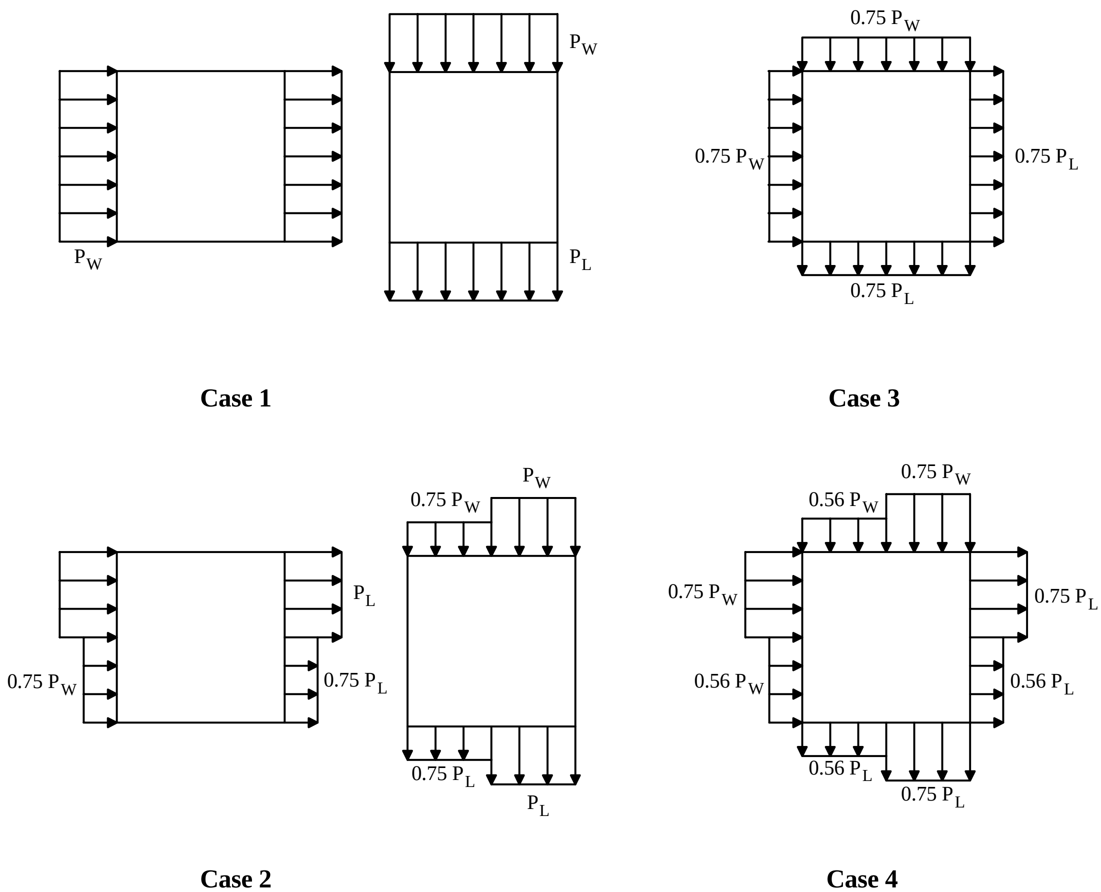


Figure 6-38 Wind loading requirements from ASCE 7.

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.

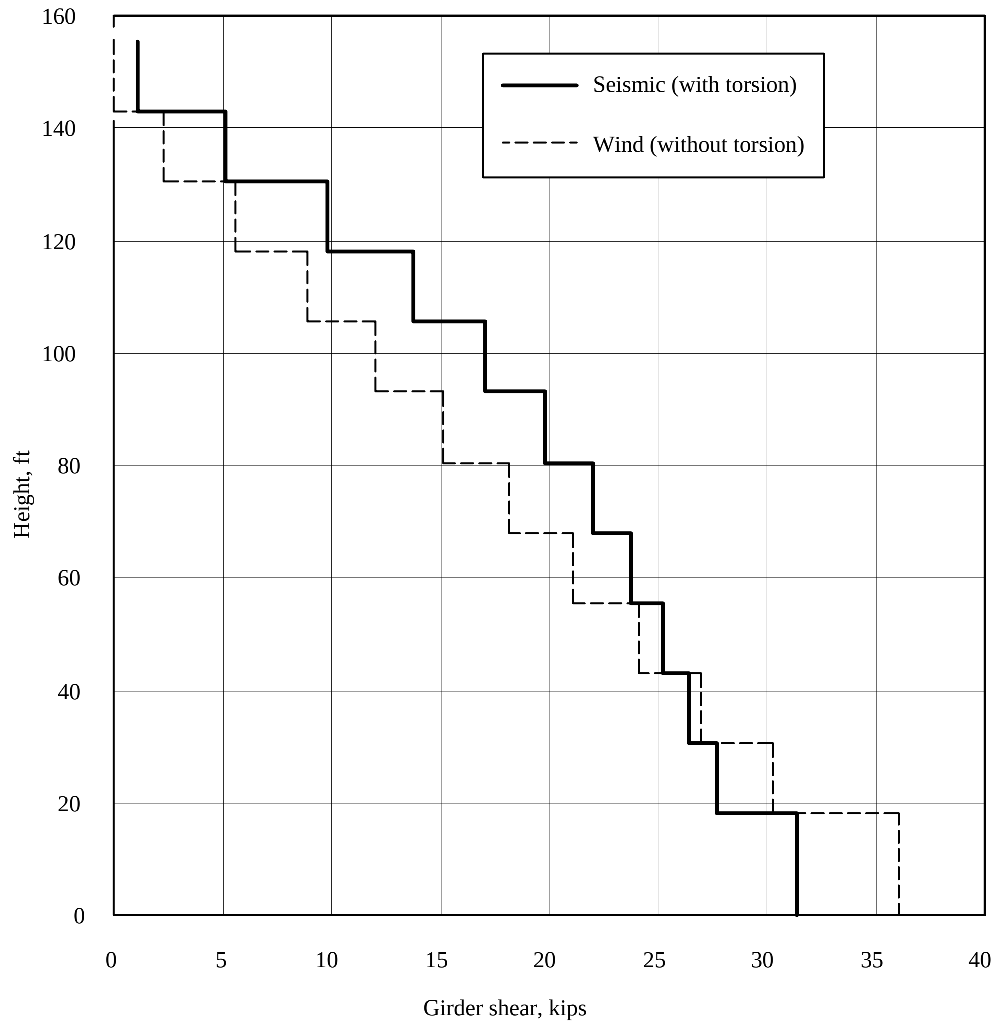


Figure 6-39 Wind vs. seismic shears in exterior bay of Frame 1 (1.0 ft = 0.3048 m, 1.0 kip = 4.45kN).

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].

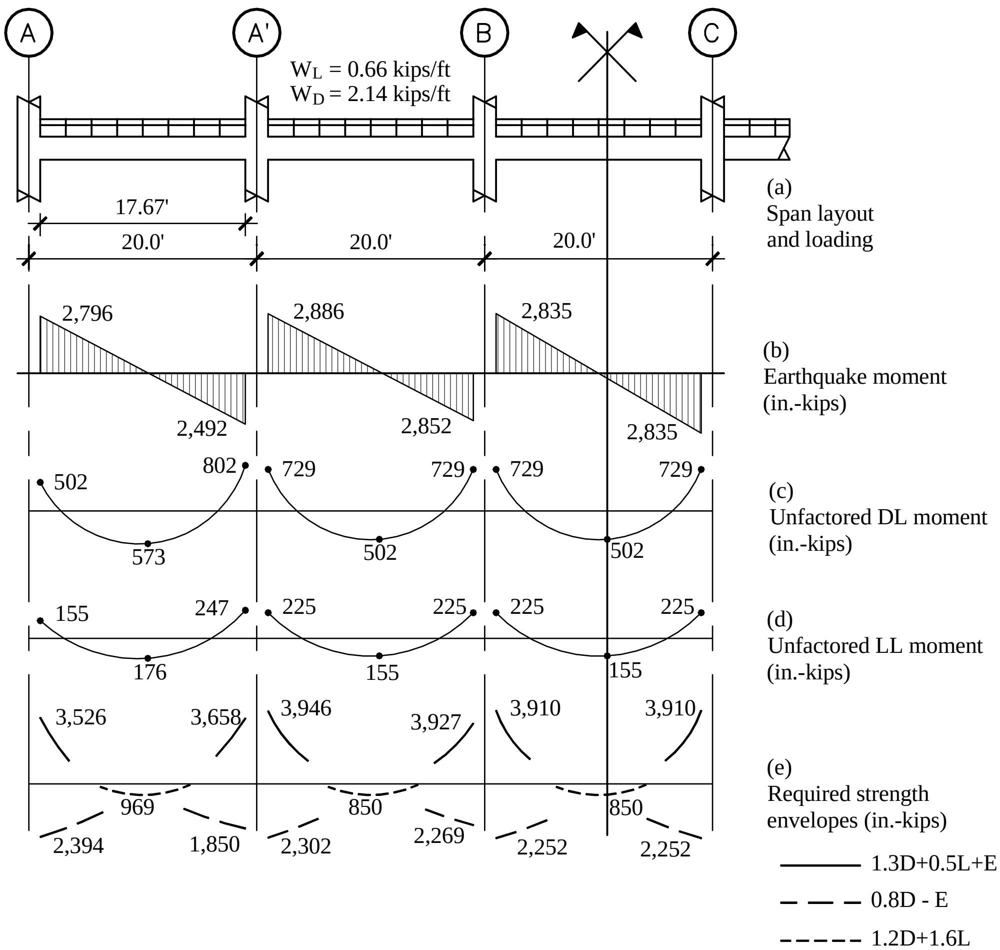


Figure 6-40 Bending moment envelopes at Level 5 of Frame 1 (1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

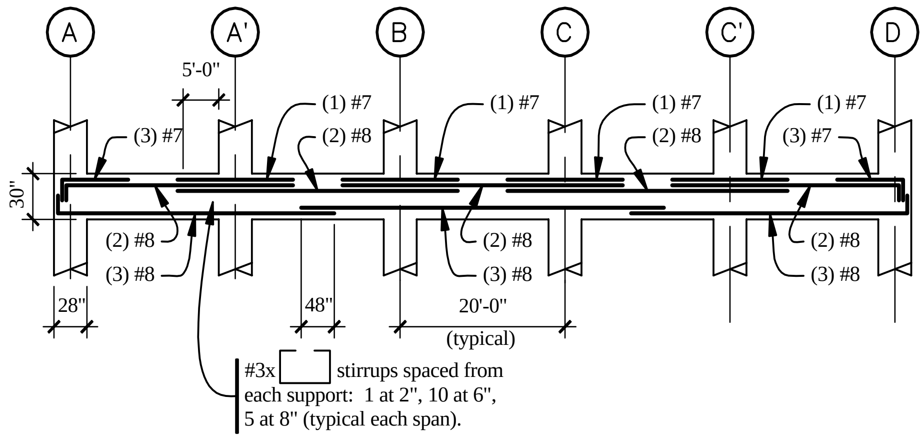


Figure 6-41 Preliminary reinforcement layout for Level 5 of Frame 1 (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

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:

$$V_u \le \phi V_c + \phi V_s$$

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.

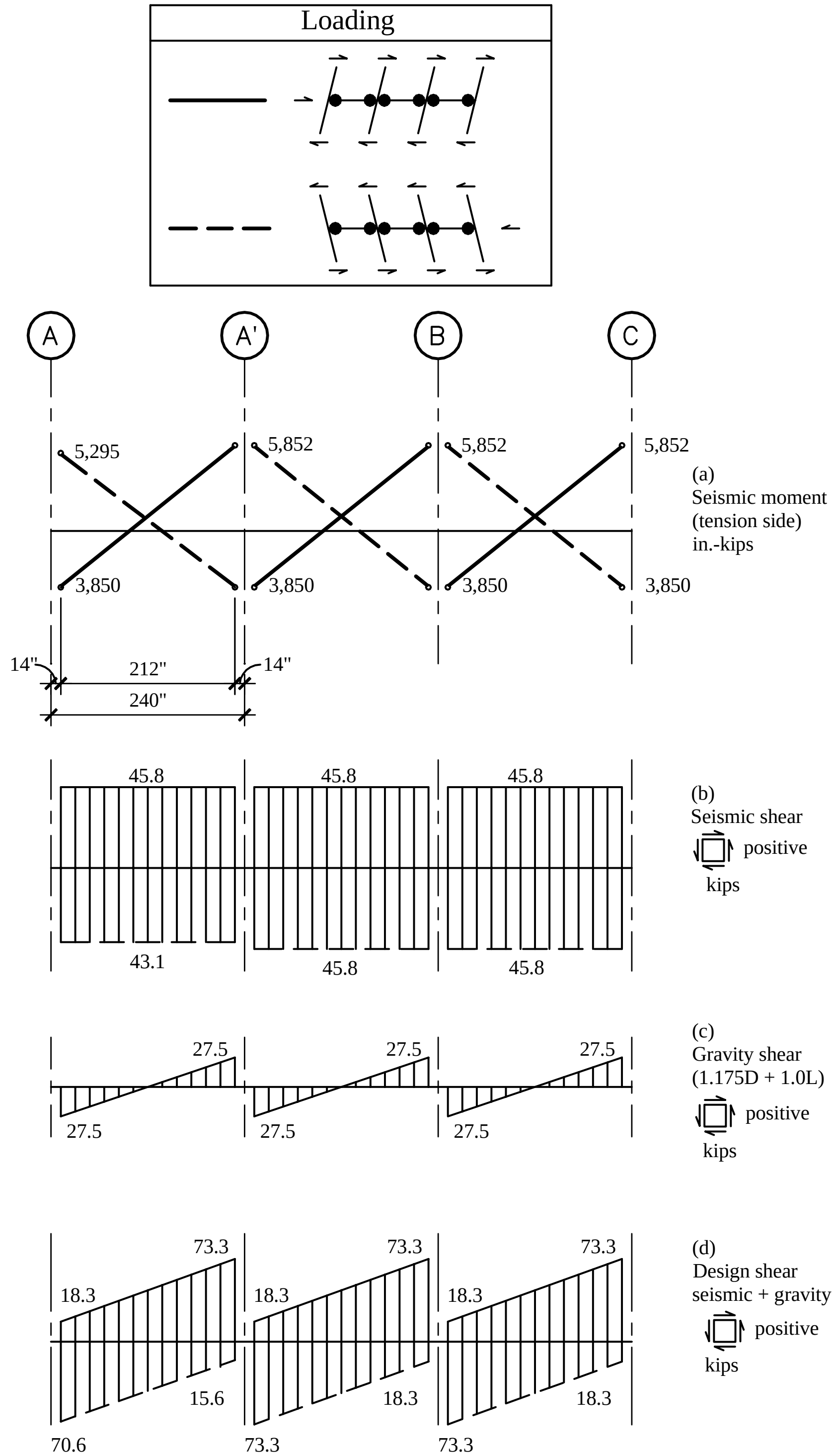


Figure 6-42 Shear strength envelopes for Span A'-B of Frame 1 (1.0 in = 25.4 mm, 1.0 kip = 4.45kN, 1.0 in.-kip = 0.113 kN-m).

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.

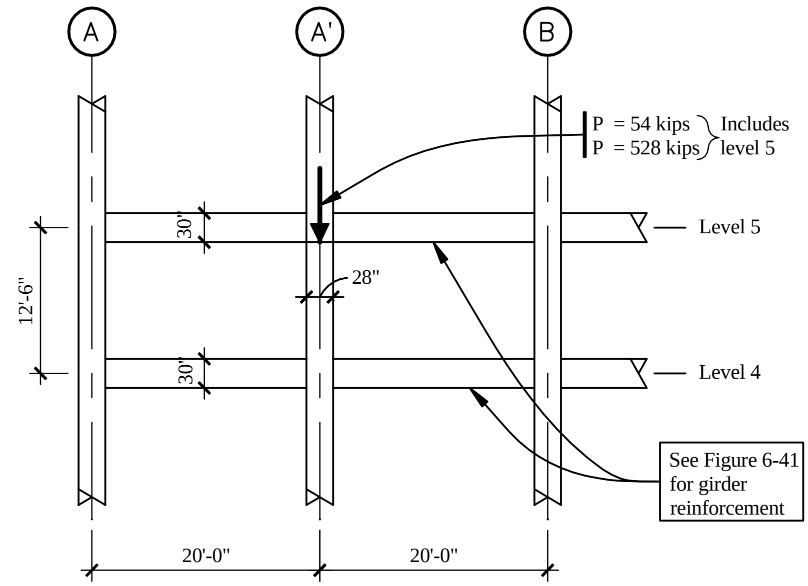


Figure 6-43 Isolated view of column A' (1.0 ft = 0.3048 m, 1.0 kip = 4.45kN).

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 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 kips

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

P_u = 0.8(528) - 33.2 = 389.2 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:

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

Using the 1.2 and 1.6 load factors on gravity load:

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

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

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

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).

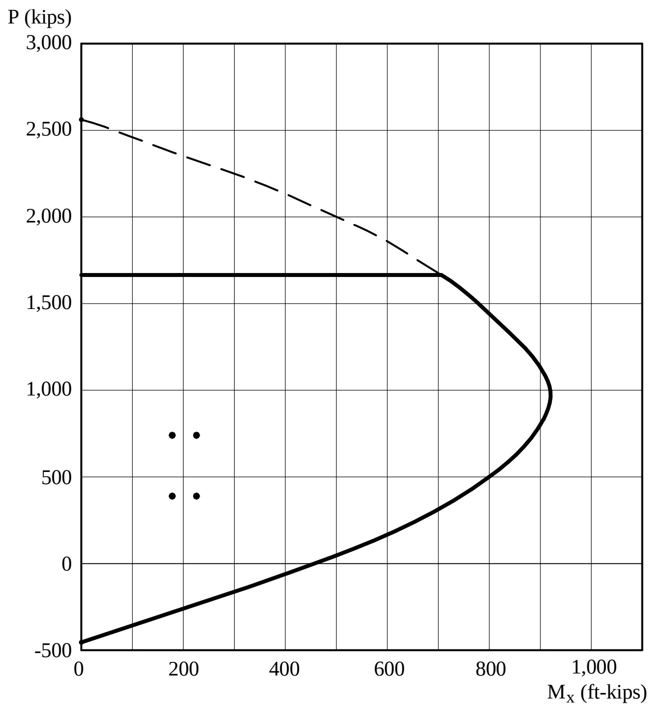


Figure 6-44 Interaction diagram for column (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

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.

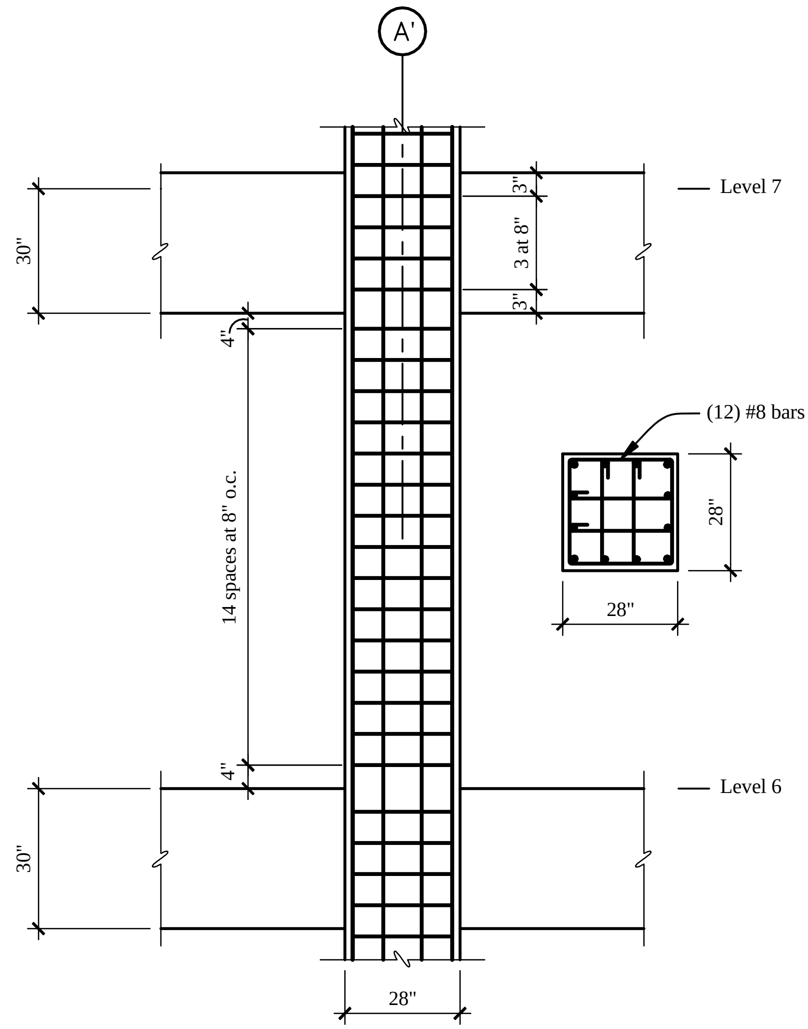


Figure 6-45 Column reinforcement (1.0 in = 25.4 mm).

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.

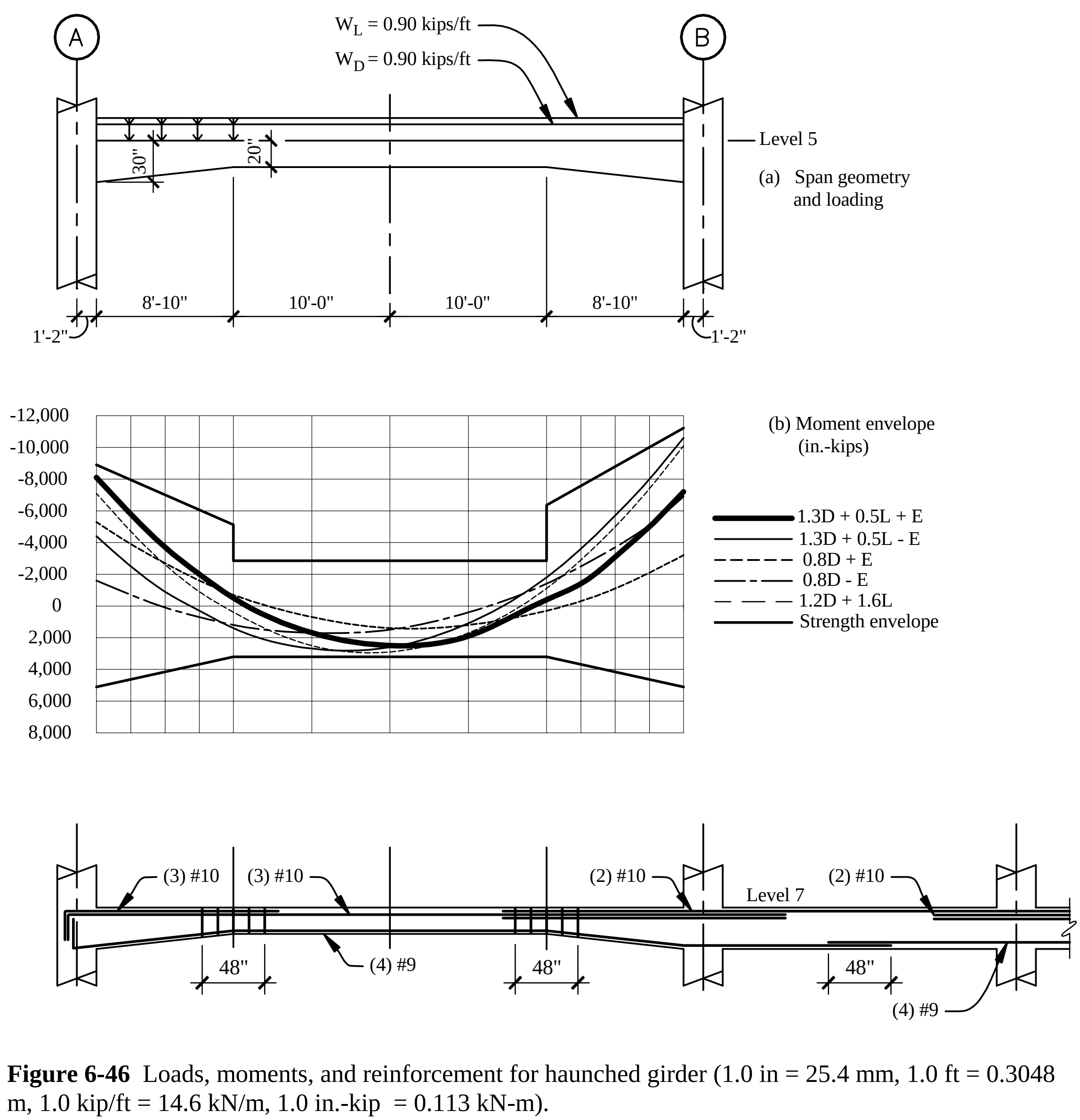


Figure 6-46 Loads, moments, and reinforcement for haunched girder (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m, 1.0 in.-kip = 0.113 kN-m).

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 $V_s = A_v f_y d/s$:

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

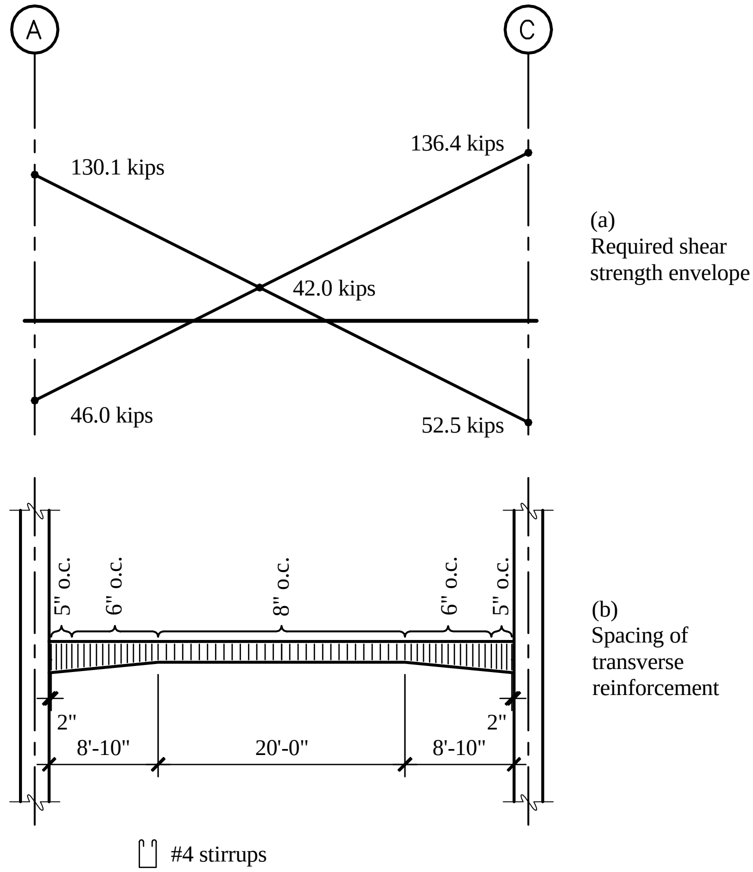


Figure 6-47 Shear force envelope for haunched girder (1.0 ft = 0.3048m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

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 2*h* 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

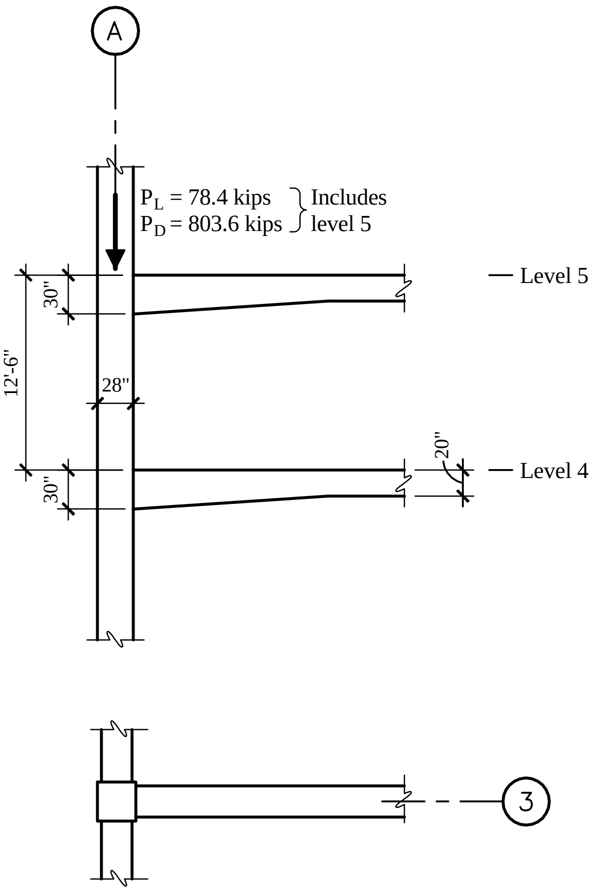


Figure 6-48 Loading for Column A, Frame 3 (1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 kip = 4.45kN).

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.

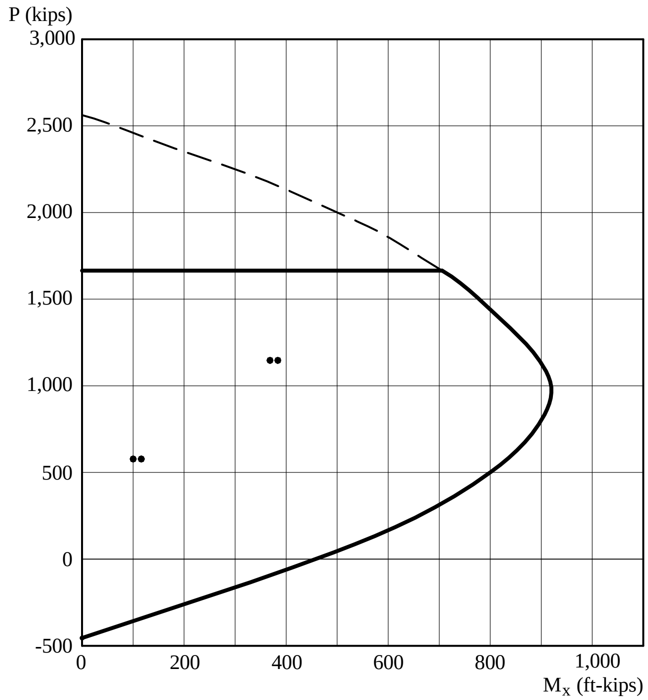


Figure 6-49 Interaction diagram for Column A, Frame 3 (1.0 kip = 4.45kN, 1.0 ft-kip = 1.36 kN-m).

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.

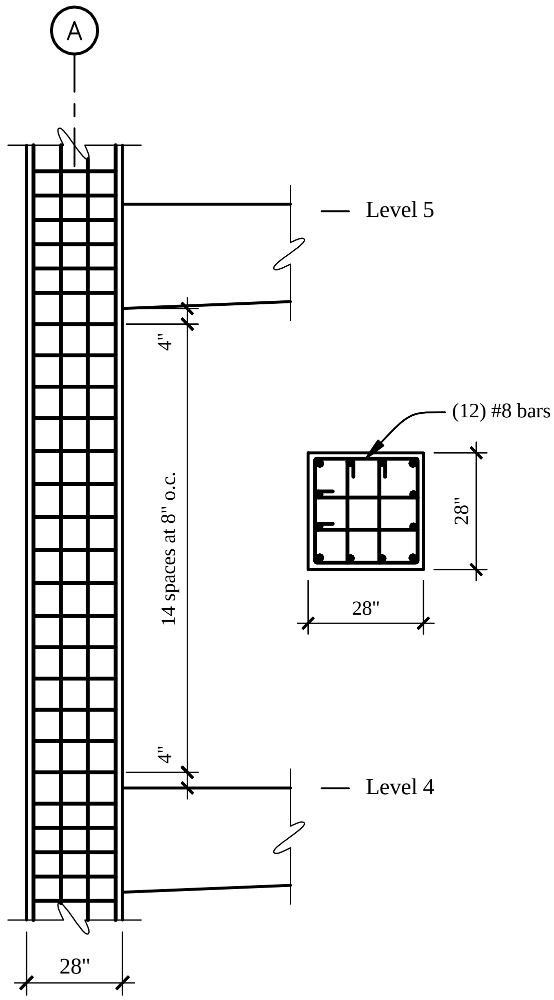


Figure 6-50 Details for Column A, Frame 3 (1.0 in = 25.4 mm).