9

MASONRY

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This chapter illustrates application of the 2000 *NEHRP Recommended Provisions* (herein after the *Provisions),* to the design of a variety of reinforced masonry structures in regions with different levels of seismicity. Example 9.1 features a single-story masonry warehouse building with tall, slender walls; Example 9.2 presents a five-story masonry hotel building with a bearing wall system designed in areas with different seismicities; and Example 9.3 covers a twelve-story masonry building having the same plan as the hotel but located in a region of high seismicity. Selected portions of each building are designed to demonstrate specific aspects of the design provisions.

Masonry is a discontinuous and heterogeneous material. The design philosophy of reinforced grouted masonry approaches that of reinforced concrete; however, there are significant differences between masonry and concrete in terms of restrictions on the placement of reinforcement and the effects of the joints. These physical differences create significant differences in the design criteria.

All structures were analyzed using two-dimensional (2-D) static methods. Examples 9.2 and 9.3 use dynamic analyses to determine the structural periods. Example 9.2 employs the SAP 2000 program, V6.11 (Computers and Structures, Berkeley, California); Example 9.3 employs the RISA 2D program, V.5.5 (Risa Technologies, Foothill Ranch, California).

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

The most significant change to the masonry chapter in the 2003 *Provisions* is the incorporation by reference of ACI 530-02 for strength design in masonry. A significant portion of 2003 *Provisions* Chapter 11 has been replaced by a reference to this standard as well as a limited number of modifications to the standard, similar to other materials chapters. This updated chapter, however, does not result in significant technical changes as ACI 530-02 is in substantial agreement with the strength design methodology contained in the 2000 *Provisions*.

Another change to the provisions for masonry structures is the addition of a new lateral system, prestressed masonry shear walls. This system is not covered in this volume of design examples.

Some general technical changes in the 2003 *Provisions* that relate to the calculations and/or design in this chapter include updated seismic hazard maps, changes to Seismic Design Category classification for short period structures, revisions to the redundancy requirements, revisions to the wall anchorage design requirement for flexible diaphragms, and a new "Simplified Design Procedure" that could be applicable to some of the examples in this chapter.

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

In addition to the *Provisions*, the following documents are referenced in this chapter:

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

ACI 530 — American Concrete Institute. 1999 [2002]. *Building Code Requirements for Masonry Structures*, ACI 530/ASCE 5/TMS 402.

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

Amrhein — Amrhein, J, and D. Lee. 1994. *Tall Slender Walls, 2nd Ed.* Masonry Institute of America.

Drysdale — Drysdale R., A. Hamid, and L. Baker. 1999. *Masonry Structures, Behavior and Design.* Boulder Colorado: The Boulder Masonry Society.

IBC — International Code Council. 2000. *International Building Code*.

UBC — International Conference of Building Officials. 1997. *Uniform Building Code*.

NCMA — National Concrete Masonry Association. *A Manual of Facts on Concrete Masonry*, NCMA-TEK is an information series from the National Concrete Masonry Association, various dates.

SEAOC — Seismology Committee, Structural Engineers Association of California. 1999. *Recommended Lateral Force Requirements and Commentary*, 7th Ed.

The short form designations for each citation are used throughout. The citation to the IBC exists for two reasons. One of the designs employees a tall, slender wall that is partially governed by wind loads and the IBC provisions are used for that design. Also, the *R* factors for masonry walls are significantly different in the IBC than in the *Provisions*; this is not true for other structural systems.

9.1 WAREHOUSE WITH MASONRY WALLS AND WOOD ROOF, LOS ANGELES, CALIFORNIA

This example features a one-story building with reinforced masonry bearing walls and shear walls.

9.1.1 Building Description

This simple rectangular warehouse is 100 ft by 200 ft in plan (Figure 9.1-1). The masonry walls are 30 ft high on all sides, with the upper 2 ft being a parapet. The wood roof structure slopes slightly higher towards the center of the building for drainage. The walls are 8 in. thick on the long side of the building, for which the slender wall design method is adopted, and 12 in. thick on both ends. The masonry is grouted in the cells containing reinforcement, but it is not grouted solid. The assumed strength of masonry is 2,000 psi. Normal weight concrete masonry units (CMU) with type S mortar are assumed.

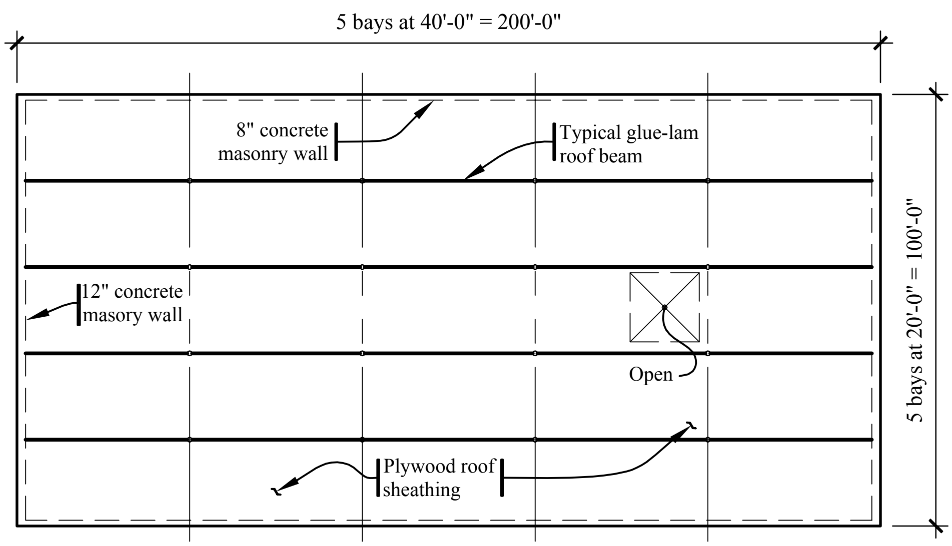


Figure 9.1-1 Roof plan (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

The long side walls are solid (no openings). The end walls are penetrated by several large doors, which results in more highly stressed piers between the doors (Figure 9.1-2); thus, the greater thickness for the end walls.

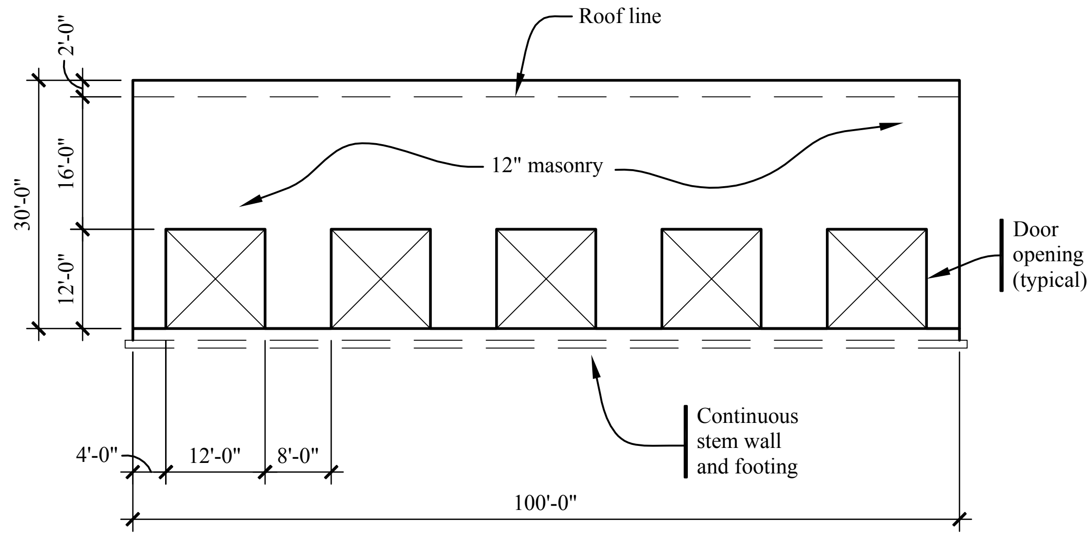


Figure 9.1-2 End wall elevation (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

The floor is concrete slab-on-grade construction. Conventional spread footings are used to support the interior steel columns. The soil at the site is a dense, gravelly sand.

The roof structure is wood and acts as a diaphragm to carry lateral loads in its plane from and to the exterior walls. The roofing is ballasted, yielding a total roof dead load of 20 psf. There are no interior walls for seismic resistance. This design results in a highly stressed diaphragm with large calculated deflections. The design of the wood roof diaphragm and the masonry wall-to-diaphragm connections is illustrated in Sec. 10.2.

In this example, the following aspects of the structural design are considered:

1. Design of reinforced masonry walls for seismic loads and
2. Computation of P-delta effects.

9.1.2 Design Requirements

[Note that the new "Simplified Design Procedure" contained in the 2003 *Provisions* Simplified Alternate Chapter 4 as referenced by the 2003 *Provisions* Sec. 4.1.1 is likely to be applicable to this example, subject to the limitations specified in 2003 *Provisions* Sec. Alt. 4.1.1.]

9.1.2.1 *Provisions* Parameters

Site Class (*Provisions* Sec. 4.1.2.1 [Sec. 3.5])	= C
S_S (*Provisions* Map 5 [Figure 3.3-3])	= 1.50
S_1 (*Provisions* Map 6 [Figure 3.3-4])	= 0.60
Seismic Use Group (*Provisions* Sec. 1.3[Sec. 1.2])	= I

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps , and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

The remaining basic parameters depend on the ground motion adjusted for site conditions.

9.1.2.2 Response Parameter Determination

The mapped spectral response factors must be adjusted for site class in accordance with *Provisions* Sec. 4.1.2.4 [3.3.2]. The adjusted spectral response acceleration parameters are computed according to *Provisions* Eq. 4.1.2.4-1 [3.3-1] and 4.1.2.4-2 [3.3-2] for the short period and one-second period, respectively, as follows:

$S_{MS} = F_a S_S = 1.0(1.50) = 1.50$
$S_{M1} = F_v S_1 = 1.3(0.60) = 0.78$

Where F_a and F_v are site coefficients defined in *Provisions* Tables 4.1.2.4a [3.3-1] and 4.1.2.4b [3.3-2], respectively. The design spectral response acceleration parameters (*Provisions* Sec. 4.1.2.5 [Sec. 3.3.3]) are determined in accordance with *Provisions* Eq. 4.1.2.5-1 [Eq. 3.3-3] and 4.1.2.5-2 [3.3-4] for the short-period and one-second period, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.50) = 1.00$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.78) = 0.52$$

The Seismic Design Category may be determined by the design spectral acceleration parameters combined with the Seismic Use Group. For buildings assigned to Seismic Design Category D, masonry shear walls must satisfy the requirements for special reinforced masonry shear walls in accordance with *Provisions* Sec. 11.3.8.2 [ACI 530 Sec. 1.13.6.4]. A summary of the seismic design parameters follows:

Seismic Design Category (*Provisions* Sec. 4.2.1 [1.4])	= D
Seismic Force Resisting System (*Provisions* Table 5.2.2 [4.3-1])	= Special Reinforced Masonry Shear Wall
Response Modification Factor, R (*Provisions* Table 5.2.2 [4.3-1])	= 3.5
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [4.3-1])	= 3.5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [4.3-1])	= 2.5
Reliability Factor, ρ (*Provisions* Sec. 5.2.4.2 [Sec. 4.3.3])	= 1.0

(Determination of ρ is discussed in Sec. 9.1.3 below [see Sec. 9.1.3.1 for changes to the reliability factor in the 2003 *Provisions*].)

Note that the R factor for this system in the IBC and in ASCE 7 is 4.5. [5.0 in the 2003 IBC and ASCE 7-02] This difference would have a substantial effect on the seismic design; however, the vertical reinforcement in the tall 8-in. walls is controlled by wind loads so it would not change.

9.1.2.3 Structural Design Considerations

With respect to the load path, the roof diaphragm supports the upper 16 ft of the masonry walls (half the clear span plus the parapet) in the out-of-plane direction, transferring the lateral force to in-plane masonry shear walls.

Soil structure interaction is not considered.

The building is of bearing wall construction.

Other than the opening in the roof, the building is symmetric about both principal axes, and the vertical elements of the seismic resisting system are arrayed entirely at the perimeter. The opening is not large enough to be considered an irregularity (per *Provisions* Table 5.2.3.2[Table 4.3-2]); thus, the building is regular, both horizontally and vertically. *Provisions* Table 5.2.5.1[Table 4.4-1], permits several analytical procedures to be used; the equivalent lateral force (ELF) procedure (*Provisions* Sec. 5.4) is selected for used in this example. The orthogonality requirements of *Provisions* Sec. 5.2.5.2 Sec. 4.4.2 are potentially significant for the piers between the door openings at the end walls. Thus, those walls will be designed for 100 percent of the forces in one direction plus 30 percent of the forces in the perpendicular direction.

There will be no inherent torsion because the building is symmetric. The effects of accidental torsion, and its potential amplification, need not be included because the roof diaphragm is flexible. This is the authors' interpretation of what amounts to a conflict between *Provisions* Table 5.2.3.2[Table 4.3-2], Item 1, and *Provisions* Sec. 5.4.4.2[Sec. 5.24.2] and Sec. 5.4.4.3[Sec. 5.2.4.3].

The masonry bearing walls also must be designed for forces perpendicular to their plane (*Provisions* Sec. 5.2.6.2.7)[Sec. 4.6.1.3].

For in-plane loading, the walls will be treated as cantilevered shear walls. For out-of-plane loading, the walls will be treated as pinned at the bottom and simply supported at the top. The assumption of a pinned connection at the base is deemed appropriate because the foundation is shallow and narrow which permits rotation near the base of the wall.

9.1.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [Sec. 4.2.2]) are the same as specified in ASCE 7 (and similar to the IBC). The seismic load effect, E, is defined by *Provisions* Eq. 5.2.7-1 [4.2-1] and Eq. 5.2.7-2 [4.2-2] as:

$$E = \rho Q_E \pm 0.2 S_{DS} D = (1.0) Q_E \pm 0.2(1.00) D = Q_E \pm 0.2D$$

This assumes $\rho = 1.0$ as will be confirmed in the following section.

9.1.3.1 Reliability Factor

In accordance with *Provisions* Sec. 5.2.4.2[4.3.3], the reliability factor, ρ, applies to the in-plane load direction.

For the long direction of building:

$$r_{max_x} = \left(\frac{V_{wall}}{V_{story}}\right)\left(\frac{10}{l_w}\right)$$

$$r_{max_x} = (0.5)\left(\frac{10}{200}\right) = 0.025$$

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{20,000}} = 2 - \frac{20}{0.025\sqrt{20,000}} = -3.66$$

ρ = -3.66 < 1.0 = ρ_{min}, so use ρ = 1.0.

For the short direction of the building:

$$r_{max_x} = \left(\frac{V_{wall}}{V_{story}}\right)\left(\frac{10}{l_w}\right) = \left(\frac{(V_{wall})(0.23)}{V_{story}}\right)\left(\frac{10}{8}\right) = (0.5)(0.23)(1) = 0.115$$

Although the calculation is not shown here, note that a single 8-ft-long pier carries approximately 23 percent (determined by considering the relative rigidities of the piers) of the in-plane load for each end wall.

Also, 1.0 was used for the $10/l_w$ term even though 10/8 ft > 1.0. According to *Provisions* 5.2.4.2, the $10/l_w$ term need not exceed 1.0 *only* for walls of light frame construction. This example was created based on a draft version of the 2000 *Provisions*, which limited the value of the $10/l_w$ term to 1.0 for all shear walls, a requirement that was later changed for the published edition. Thus, this calculation is not strictly correct. Using the correct value of r_{max_x} would result in ρ = 1.02 rather than the 0.77 computed below. This would result in a slight change in the factor on Q_E, 1.02 vs. 1.00, which has not been carried through the remainder of this example.

(When the redundancy factor was developed by the Structural Engineers Association of California in the wake of the 1994 Northridge earthquake, the upper bound of 1.0 for $10/l_w$ was simply not mentioned. The 1997 *Provisions*, the UBC, and the IBC were published with no upper bound on $10/l_w$. However, the original authors of the concept published their intent with the SEAOC document in 1999 with the upper bound of 1.0 on $10/l_w$ for all types of shear walls. The same change was adopted within BSSC for the 2000 *Provisions.* A subsequent change to the 2000 *Provisions* limited the upper bound of 1.0 to apply only to light frame walls.)

Therefore,

$$r_{max_x} = 0.12$$

$$\rho = 2 - \frac{20}{0.115\sqrt{20,000}} = 0.77$$

ρ = 0.77 < 1.0 = ρ_{min}, so use ρ = 1.0.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Therefore, the redundancy factor would have to be investigated only in the transverse direction where the aspect ratios of the piers between door openings are greater than 1.0. In the longitudinal direct, where the aspect ratio is (significantly) less than 1.0, ρ = 1.0 by default.]

9.1.3.2 Combination of Load Effects

Load combinations for the in-plane loading direction from ASCE 7 are:

$$1.2D + 1.0E + 0.5L + 0.2S$$

and

$$0.9D + 1.0E + 1.6H.$$

L, S, H do not apply for this example so the load combinations become:

$$1.2D + 1.0E$$

and

$$0.9D + 1.0E.$$

When the effect of the earthquake determined above, $1.2D + 1.0(Q_E \pm 0.2D)$, is inserted in each of the load combinations:

$$1.4D + 1.0\,Q_E$$
$$1.0D - 1.0\,Q_E$$

and

$$0.9D + 1.0(Q_E \pm 0.2D)$$

which results in:

$$1.1D + 1.0\,Q_E$$

and

$$0.7D - 1.0\,Q_E$$

Thus, the controlling cases from all of the above are:

$$1.4D + 1.0\,Q_E$$

when gravity and seismic are additive and

$$0.7D - 1.0\,Q_E$$

when gravity and seismic counteract.

These load combinations are for the in-plane direction of loading. Load combinations for the out-of-plane direction of loading are similar except that the reliability coefficient (ρ) is not applicable. Thus, for this example (where $\rho = 1.0$), the load combinations for both the in-plane and the out-of-plane directions are:

$$1.4D + 1.0\,Q_E$$

and

$0.7D - 1.0\ Q_E$.

The combination of earthquake motion (and corresponding loading) in two orthogonal directions must be considered (*Provisions* Sec. 5.2.5.2.3) [Sec. 4.4.2.3].

9.1.4 Seismic Forces

9.1.4.1 Base Shear

Base shear is computed using the parameters determined previously. The *Provisions* does not recognize the effect of long, flexible diaphragms on the fundamental period of vibration. The approximate period equations, which limit the computed period, are based only on the height. Since the structure is relatively short and stiff, short-period response will govern the design equations. According to *Provisions* Sec. 5.4.1 [Sec. 5.2.1.1] and Eq. 5.4.1.1-1 [Eq. 5.2-3] (for short-period structures):

$$V = C_s W = \left[\frac{S_{DS}}{R/I}\right] W = \left[\frac{1.0}{3.5/1}\right] W = 0.286\ W$$

The seismic weight for forces in the long direction is:

Roof = 20 psf (100)200	= 400 kips
End walls = 103 psf (2 walls)[(30 ft)(100 ft) - 5(12 ft)(12 ft)](17.8 ft/28 ft)	= 299 kips
Side walls = 65 psf (30ft)(200ft)(2 walls)	= 780 kips
Total	= 1,479 kips

Note that the centroid of the end walls is determined to be 17.8 ft above the base, so the portion of the weight distributed to the roof is approximately the total weight multiplied by 17.8 ft/28 ft (weights and section properties of the walls are described subsequently).

Therefore, the base shear to each of the long walls is:

V = (0.286)(1,479 kips)/2 = 211 kips.

The seismic weight for forces in the short direction is:

Roof = 20 psf (100)200	= 400 kips
Side walls = 65 psf (2 walls)(30ft)(200ft)(15ft/28ft)	= 418 kips
End walls = 103 psf (2 walls)[(30ft)(100ft)-5(12ft)(12ft)]	= 470 kips
Total	= 1,288 kips

The base shear to each of the short walls is:

V = (0.286)(1,288 kips)/2 = 184 kips.

9.1.4.2 Diaphragm Force

See Sec. 10.2 for diaphragm forces and design.

9.1.4.3 Wall Forces

because the diaphragm is flexible with respect to the walls, shear is distributed to the walls on the basis of beam theory ignoring walls perpendicular to the motion (this is the "tributary" basis).

The building is symmetric. Given the previously explained assumption that accidental torsion need not be applied, the force to each wall becomes half the force on the diaphragm.

All exterior walls are bearing walls and, according to *Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3], must be designed for a normal (out-of-plane) force of $0.4S_{DS}W_c$. The out-of-plane design is shown in Sec. 9.1.5.3 below.

9.1.5 Longitudinal Walls

The total base shear is the design force. *Provisions* Sec. 11.7 [Sec. 11.2] is the reference for design strengths. The compressive strength of the masonry (f_m') is 2,000 psi. *Provisions* Sec. 11.3.10.2 gives $E_m = 750 f_m' = (750)(2 \text{ ksi}) = 1{,}500$ ksi.

[2003 *Provisions* Sec. 11.2 adopts ACI 530 as a design basis for strength design masonry and provides some modifications to ACI 530. In general, the adoption of ACI 530 as a reference does not have a significant effect on this design example. Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1, eliminating the conflict discussed below.]

Be careful to use values consistent with the *Provisions*. Different standards call for different values. To illustrate this point, the values of E_m from different standards are shown in Table 9.1-1.

Table 9.1-1 Comparison of E_m

Standard	E_m	E_m for this example
Provisions	$750 f_m'$	1,500 ksi
IBC	$900 f_m'$	1,800 ksi
ACI 530	$900 f_m'$	1,800 ksi

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

For 8-inch thick CMU with vertical cells grouted at 24 in. o.c. and horizontal bond beams at 48 inch o.c., the weight is conservatively taken as 65 psf (recall the CMU are normal weight) and the net bedded area is 51.3 in.^{2}/ft based on tabulations in NCMA-TEK 141.

9.1.5.1 Horizontal Reinforcement

As determined in Sec. 9.1.4.1, the design base shear tributary to each longitudinal wall is 211 kips. Based on *Provisions* Sec. 11.7.2.2 [ACI 530, Sec. 3.1.3], the design shear strength must exceed either the shear corresponding to the development of 1.25 times the nominal flexure strength of the wall, which is very unlikely in this example due to the length of wall, or 2.5 times V_u = 2.5(211) = 528 kips.

From *Provisions* Eq. 11.7.3.2[ACI 530, Eq. 3-21] , the masonry component of the shear strength capacity for reinforced masonry is:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f_m'} + 0.25\,P\ .$$

Conservatively treating M/Vd as equal to 1.0 for the long walls and conservatively treating P as the weight of the wall only without considering the roof weight contribution:

$$V_m = [4.0 - 1.75(1.0)](51.3)(200)\sqrt{2000} + 0.25(390) = 1130 \text{ kips}$$

and

ϕV_m = 0.8(1,130) = 904 kips > 528 kips OK

where ϕ = 0.8 is the resistance factor for shear from *Provisions* Table11.5.3[ACI 530, sec. 3.1.4] .

Horizontal reinforcement therefore is not required for shear but is required if the wall is to qualify as a "Special Reinforced Masonry Wall."

According to *Provisions* Sec. 11.3.8.3[ACI 530, Sec. 1.13.6.3] , minimum reinforcement is (0.0007)(7.625 in.)(8 in.) = 0.043 in.^{2} per course, but it may be wise to use more horizontal reinforcement for shrinkage in this very long wall and then use minimum reinforcement in the vertical direction (this concept applies even though this wall requires far more than the minimum reinforcement in the vertical direction due to its large height-to-thickness ratio). Two #5 bars at 48 in. on center provides 0.103 in.^{2} per course. This amounts to 0.4 percent of the area of masonry plus the grout in the bond beams. The actual shrinkage properties of the masonry and the grout and local experience should be considered in deciding how much reinforcement to provide. For long walls that have no control joints, as in this example, providing more than minimum horizontal reinforcement is appropriate.

9.1.5.2 Vertical Reinforcement

Steps for verifying a trial design are noted in the sections that follow.

9.1.5.3 Out-of Plane Flexure

As indicated previously, the design demand for seismic out-of-plane flexure is $0.4S_{DS}W_c$. For a wall weight of 65 psf for the 8-in.-thick CMU side walls, this demand is 0.4(1.00)(65 psf) = 26 psf.

Calculations for out-of-plane flexure become somewhat involved and include the following:

1. Select a trial design.

2. Investigate to ensure ductility.

3. Make sure the trial design is suitable for wind (or other nonseismic) lateral loadings using the IBC.

   Note that many section properties determined in accordance with the IBC are different from those indicated in the *Provisions* so section properties will have to be determined multiple times. The IBC portion of the calculation is not included in this example.

[2003 *Provisions* and the 2003 IBC both adopt ACI 530-02 by reference, so the section properties should be the same for both documents.]

4. Calculate midheight deflection due to wind by the IBC. (While the *Provisions* have story drift requirements, they do not impose a midheight deflection limit for walls).

5. Calculate seismic demand.

6. Determine seismic resistance and compare to demand determined in Step 5.

Proceed with these steps as follows:

9.1.5.3.1 Trial design

A trial design of #7 bars at 24 in. on center is selected. See Figure 9.1-3.

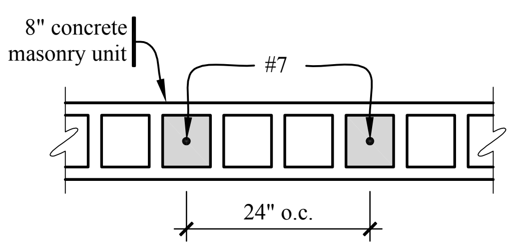


Figure 9.1-3 Trial design for 8-in.-thick CMU wall (1.0 in = 25.4 mm).

9.1.5.3.2 Investigate to ensure ductility

The critical strain condition corresponds to a strain in the extreme tension reinforcement (which is a single #7 centered in the wall in this example) equal to 1.3 times the strain at yield stress.

Based on *Provisions* Sec. 11.6.2.2[ACI 530, Sec. 3.2.3.5.1] for this case:

t = 7.63 in.
$d = t/2$ = 3.81 in.
$\varepsilon_m = 0.0025$
$\varepsilon_s = 1.3\varepsilon_y = 1.3(f_y/E_s)$ = 1.3(60 ksi /29,000 ksi) = 0.0027

$$c = \left[\frac{\varepsilon_m}{(\varepsilon_m + \varepsilon_s)}\right] d = 1.83 \text{ in.}$$

$a = 0.8c$ = 1.46 in.

The Whitney compression stress block, a = 1.46 in. for this strain distribution, is greater than the 1.25 in. face shell width. Thus, the compression stress block is broken into two components: one for full compression against solid masonry (the face shell) and another for compression against the webs and grouted cells, but accounting for the open cells. These are shown as C_1 and C_2 in Figure 9.1-4:

$C_1 = 0.80 f_m'$ (1.25 in.)b = (0.80)(2 ksi)(1.25)(24) = 48 kips (for a 24-in. length)
$C_2 = 0.80 f_m'$ (a-1.25 in.)(8 in.) = (0.80)(2 ksi)(1.46-1.25)(8) = 2.69 kips (for a 24-in. length)

The 8-in. dimension in the C_2 calculation is for combined width of grouted cell and adjacent mortared webs over a 24-in. length of wall. The actual width of one cell plus the two adjacent webs will vary with various block manufacturers, and may be larger or smaller than 8 in. The 8-in. value has the benefit of simplicity and is correct when considering solidly grouted walls.

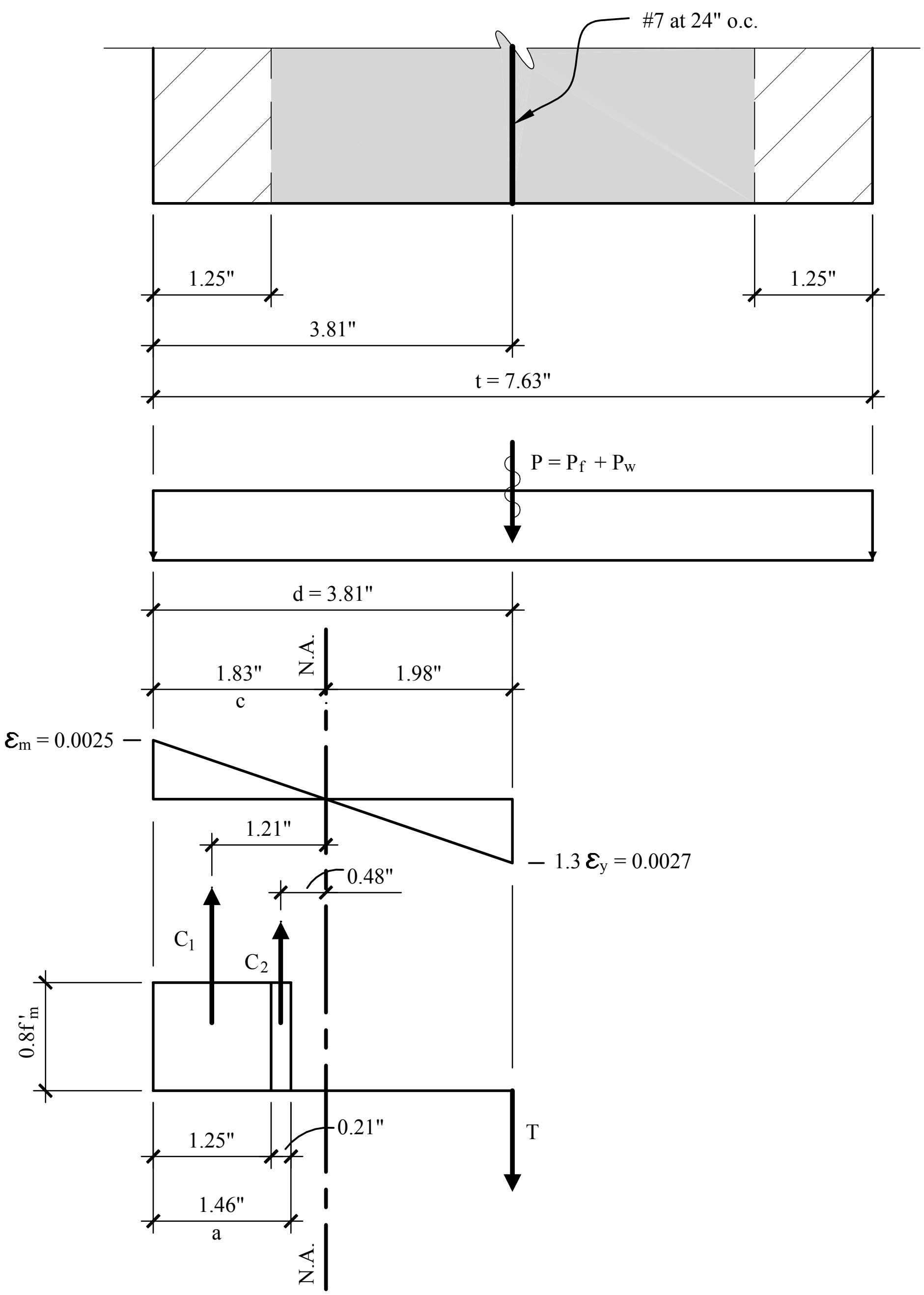


Figure 9.1-4 Investigation of out-of-plane ductility for the 8-in.-thick CMU side walls (1.0 in = 25.4 mm).

T is based on 1.25 F_y (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]:

$T = 1.25F_yA_s$ = (1.25)(60 ksi)(0.60 in.^{2}) = 45 kips (for a 24-in. length)

Use unfactored P (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]:

$P = (P_f + P_w)$ = (20 psf (10 ft.) + 65 psf (16 ft.)) = 1.24 klf = 2.48 kips (for a 24-in. length)

Check $C_1 + C_2 > T + P$:

$T + P$ = 47.5 kips
$C_1 + C_2$ = 50.7 kips > 47.5 kips. OK

The compression capacity is greater than the tension capacity; therefore, the ductile failure mode criterion is satisfied.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (1.5 times) and axial force to consider when performing the ductility check (factored loads).]

9.1.5.3.3 Check for wind load using the IBC

Load factors and section properties are not the same in the IBC and the *Provisions* (The wind design check is beyond the scope of this seismic example so it is not presented here.) Both strength and deflection need to be ascertained in accordance with IBC.

Note that, for comparison, selected properties for the *Provisions* (and IBC) ductility check, IBC wind strength check, and *Provisions* seismic strength check are tabulated below. Keeping track of which version of a given parameter is used for each of the calculations can get confusing; be careful to apply the correct property for each analysis.

Table 9.1-2 Comparison of Variables (explanations in the following text)

Parameter	*Provisions* Ductility Calculation	*Provisions* Strength Calculation	*IBC* Wind Calculation
P	1.24 klf	0.87 & 1.74 klf	1.12 klf
E_m	NA	1,500,000 psi	1,800,000 psi
f_r	NA	80 psi	112 psi
w	NA	26 psf	19 psf (service)
ε_s	0.0027	NA	NA
d	3.82 in.	3.82 in.	3.82 in.
c	1.83 in.	1.25 in.	1.25 in.
a	1.46 in.	1.00 in.	1.00 in.
$C_{res} = C_1+C_2$	50.1 kips	52.1 kips	56.4 kips
$n = E_s/E_m$	NA	19.33	16.11
I_g	NA	355 in.^{4}	355 in.^{4}
S_g	NA	93.2 in.^{3}	93.2 in.^{3}
A_{se}	NA	0.32 in.^{2}/ft	0.32 in.^{2}/ft
I_{cr}	NA	48.4 in.^{4}/ft	48.4 in.^{4}/ft
$M_{cr} = f_rS$	NA	7.46 in.-kips	10.44 in.-kips
δ_{allow}	NA	NA	2.32 in.

NA = not applicable, 1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 in = 25.4 mm, 1.0 ksi = 6.98 MPa, 1.0 in.-kip = 0.113 kN-m.

9.1.5.3.4 Calculate midheight deflection due to wind by the IBC

The actual calculation is not presented here. For this example the midheight deflection was calculated using IBC Eq. 21-41[ACI 530, Eq. 3-31] with I_{cr} = 47.3 in.^{4} per ft. Using IBC Eq. 21.41[ACI 530, Eq. 3-31], the calculated deflection is 2.32 in., which is less than 2.35 in. = 0.007*h* (IBC Eq. 21-39[ACI 530, Eq. 3-29]).

9.1.5.3.5 Calculate seismic demand

For this case, the two load factors for dead load apply: 0.7*D* and 1.4*D*. Conventional wisdom holds that the lower dead load will result in lower moment-resisting capacity of the wall so the 0.7*D* load factor would be expected to govern. However, the lower dead load also results in lower P-delta so both cases should be checked. (As it turns out, the higher factor of 1.4*D* governs).

Check moment capacity for 0.7*D*:

$P_u = 0.7(P_f + P_w)$.

For this example, the iterative procedure for addressing P-delta from Amrhein will be used, not *Provisions* Eq. 11.5.4.3[ACI 530, Commentary Sec. 3.1.5.3] which is intended for in-plane deflections:

Roof load, P_f = 0.7(0.2 klf) = 0.14 klf
Eccentricity, e = 7.32 in. (distance from wall centerline to roof reaction centerline)
Modulus of elasticity (*Provisions* Eq. 11.3.10.2 [ACI 530, 1.8.2.2]), $E_m = 750 f_m'$ = 1,500,000 psi

[Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1.]

Modular ratio, $n = \dfrac{E_s}{E_m} = 19.3$

The modulus of rupture (f_r) is found in *Provisions* Table 11.3.10.5.1[ACI 530, Sec. 3.1.7.2.1]. The values given in the table are for either hollow CMU or fully grouted CMU. Values for partially grouted CMU are not given; Footnote a indicates that interpolation between these values must be performed. As illustrated in Figure 9.1-6, the interpolated value for this example is 80 psi:

(f_r - 50 psi)/(103 in.^{2} - 60 in.^{2}) = (136 psi - 50 psi)/(183 in.^{2} - 60 in.^{2})
f_r = 80 psi
I_g = 355 in.^{4}/ft
S_g = 93.2 in.^{3}/ft
$M_{cr} = f_r S_g$ = 7460 in-lb/ft

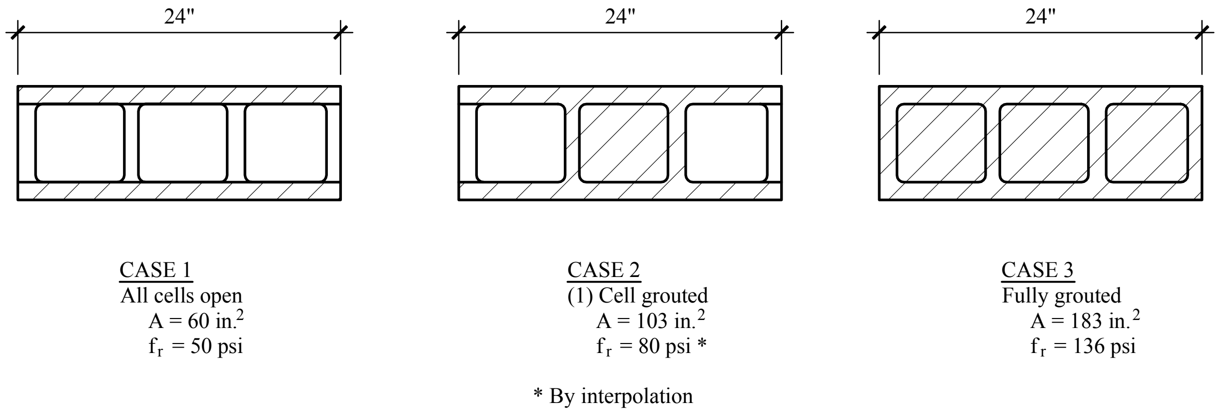


Figure 9.1-6 Basis for interpolation of modulus of rupture, f_r (1.0 in = 25.4 mm, 1.0 psi = 6.89 kPa).

Refer to Figure 9.1-7 for determining I_{cr}. The neutral axis shown on the figure is not the conventional neutral axis by linear analysis; instead it is the plastic centroid, which is simpler to locate, especially when the neutral axis position results in a T beam cross-section. (For this wall, the neutral axis does not produce a T section, but for the other wall in this building, a T section does result.) Cracked moments of inertia computed by this procedure are less than those computed by linear analysis but generally not so much less that the difference is significant. (This is the method used for computing the cracked section moment of inertia for slender walls in the standard for concrete structures, ACI 318.) Axial load does enter the computation of the plastic neutral axis and the effective area of reinforcement. Thus:

P = 1.24 klf
T = ((0.60 in.^{2})/(2 ft.))(60 ksi) = 18.0 klf
$C = T + P$ = 19.24 klf
$a = C/(0.8 f'_m b)$ = (19.24 klf)/(0.8(2.0 ksi)(12 in./ft.)) = 1.002 in.
$c = a/0.8$ = 1.253 in.
$I_{cr} = nA_{se}(d-c)^2 + bc^3/3$ = 19.33(0.30 in.^{2} + (1.24 klf)/60 ksi)(3.81 in. - 1.25 in.)^2 +
(12 in./ft)(1.25 in.)^3/3 = 4.84 in.^{4}/ft

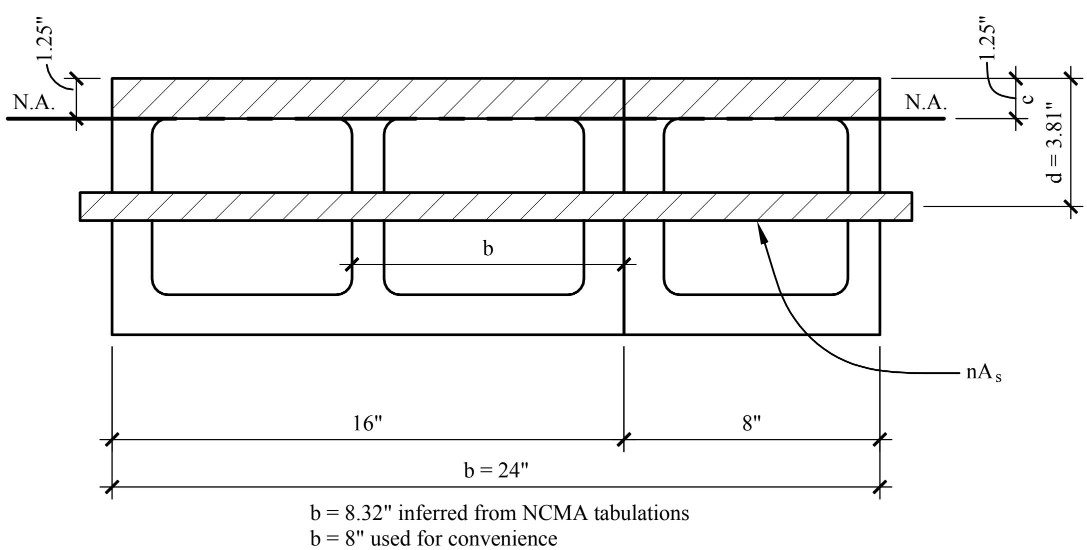


Figure 9.1-7 Cracked moment of inertia (I_{cr}) for 8-in.-thick CMU side walls (1.0 in = 25.4 mm).

Note that I_{cr} could be recomputed for $P = 0.7D$ and $P = 1.4D$ but that refinement is not pursued in this example.

The standard technique is to compute the secondary moment in an iterative fashion as shown below:

Axial load

$$P_u = 0.7(P_f + P_w) = 0.7(0.2 \text{ klf} + 1.04 \text{ klf}) = 0.868 \text{ klf}$$

First iteration

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{0.868 + (0.60)(60)}{60} = 0.614 \text{ in.}^2 / 2 \text{ ft.} = 0.312 \text{ in.}^2 / \text{ft}$$

$$M_{u1} = w_u h^2 / 8 + P_0 e + (P_0 + P_w)\Delta$$

$$M_{u1} = \frac{(26 \text{ psf/12})(336 \text{ in.})^2}{8} + (140 \text{ plf})\left(\frac{7.32 \text{ in.}}{2}\right) + (140\text{plf} + 728 \text{ plf})(0)$$

$$M_{u1} = 31{,}088 \text{ in.-lb/lf} > M_{cr} = 7460$$

$$\Delta_{s1} = \frac{5(7460)(336)^2}{48(1{,}500{,}000)(355)} + \frac{5(31{,}088 - 7460)(336)^2}{48(1{,}500{,}000)(48.4)} = 0.165 + 3.827 = 3.99 \text{ in.}$$

Second iteration

$$M_{u2} = 30{,}576 + 512 + (140 + 728)(3.99) = 34{,}551 \text{ in.-lb}$$

$$\Delta_{s2} = 0.165 + \frac{5(34{,}551 - 7460)(336)^2}{48(1{,}500{,}000)(48.4)} = 0.165 + 4.388 = 4.55 \text{ in.}$$

Third iteration

$$M_{u3} = 30{,}576 + 512 + (140 + 728)(4.55) = 35{,}040 \text{ in.-lb/lf}$$

$$\Delta_{s3} = 0.165 + \frac{5(35{,}040 - 7460)(336)^2}{48(1{,}500{,}000)(48.4)} = 0.165 + 4.467 = 4.63 \text{ in.}$$

Convergence check

$$\frac{4.63 - 4.55}{4.55} = 1.8\% < 5\%$$

$M_u = 35{,}040$ in.-lb (for the $0.7D$ load case)

Using the same procedure, find M_u for the $1.4D$ load case. The results are summarized below:

First iteration

$P = 7360$ plf
$M_{u1} = 31{,}601$ in.-lb/ft

Δ_{u1} = 4.08 in.

Second iteration

M_{u2} = 38,684 in.-lb/ft
Δ_{u2} = 5.22 in.

Third iteration

M_{u3} = 40,667 in.-lb/ft
Δ_3 = 5.54 in.

Fourth iteration

M_{u4} = 41,225 in.-lb/ft
Δ_{u4} = 5.63 in.

Check convergence

$$\frac{5.63-5.54}{5.54}=1.7\%<5\%$$

M_u = 41,225 in.-lb (for the 1.4D load case)

9.1.5.3.6 Determine flexural strength of wall

Refer to Figure 9.1-8. As in the case for the ductility check, a strain diagram is drawn. Unlike the ductility check, the strain in the steel is not predetermined. Instead, as in conventional strength design of reinforced concrete, a rectangular stress block is computed first and then the flexural capacity is checked.

$T = A_s f_y$ = (0.30 in.^{2}/ft.)60 ksi = 18.0 klf

The results for the two axial load cases are tabulated below.

Load Case	0.7D + E	1.4D + E
Factored P, klf	0.87	1.74
$T + P = C$, klf	18.87	19.74
$a = C / (0.8f'_m b)$, in.	0.981	1.028
$M_N = C (d - a/2)$, in.-kip/ft.	62.6	65.1
$\varphi M_N = 0.85M_N$, in.-kip/ft.	53.2	55.3
M_U, in.-kip/ft.	35.0	41.2
Acceptance	OK	OK

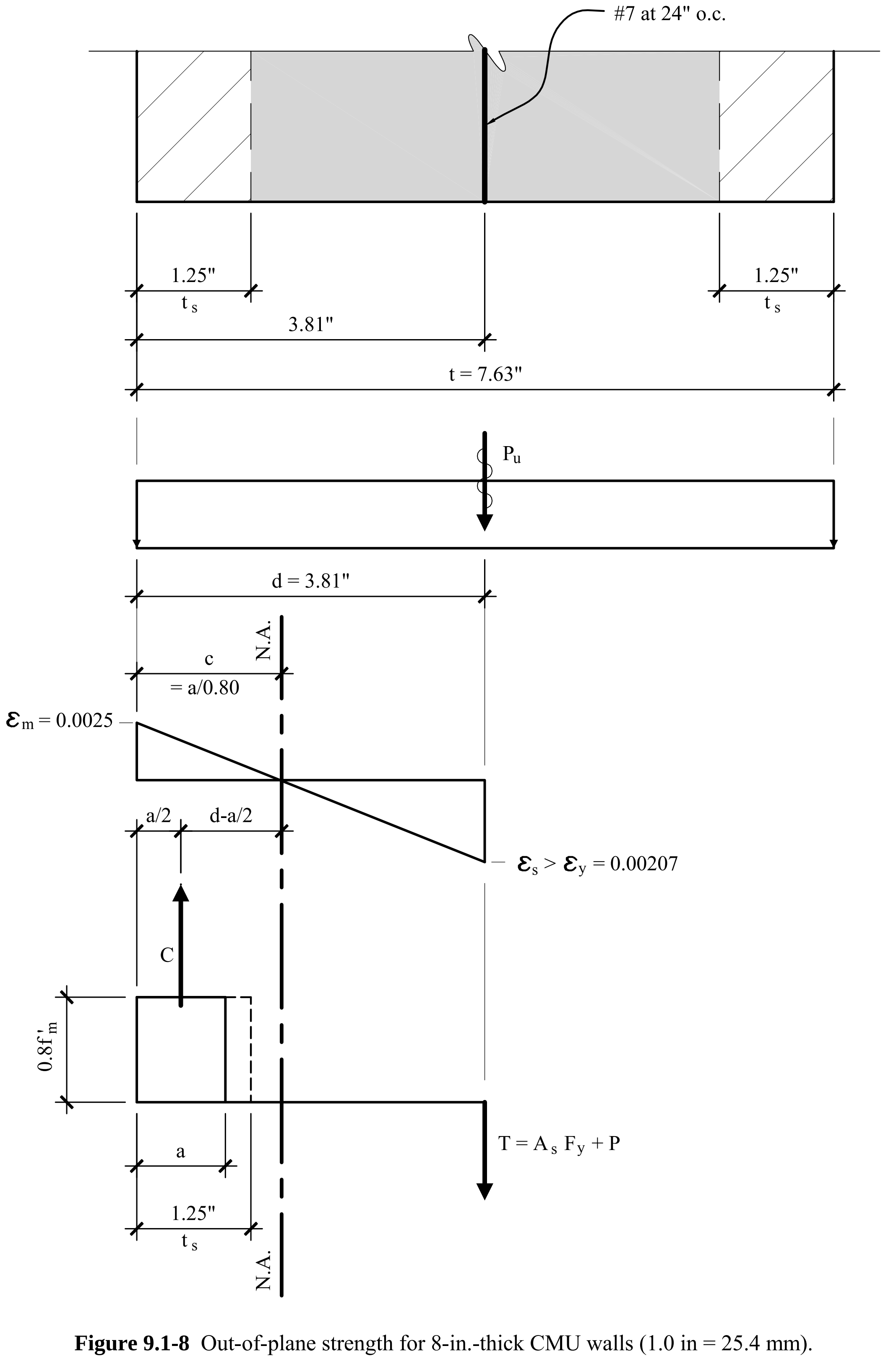


Figure 9.1-8 Out-of-plane strength for 8-in.-thick CMU walls (1.0 in = 25.4 mm).

Note that wind actually controls the stiffness and strength out-of-plane and that this is only a "tentative" acceptance for seismic. The *Provisions* requires a check of the combined orthogonal loads in accordance with *Provisions* Sec. 5.2.5.2, Item a [Sec. 4.4.2.3]. However, as discussed below, a combined orthogonal load check was deemed unnecessary for this example.

9.1.5.4 In-Plane Flexure

In-plane calculations for flexure in masonry walls include two items per the *Provisions*:

1. Ductility check and
2. Strength check.

It is recognized that this wall is very strong and stiff in the in-plane direction. In fact, most engineers would not even consider these checks necessary in ordinary design. The ductility check is illustrated here for two reasons: to show a method of implementing the requirement and to point out an unexpected result. (In the authors' opinion, the *Provisions* should reconsider the application of the ductility check where the M/Vd_v ratio is substantially less than 1.0.)

9.1.5.4.1 Ductility check

Provisions Sec. 11.6.2.2 [ACI 530, 3.2.3.5.1] requires that the critical strain condition correspond to a strain in the extreme tension reinforcement equal to 5 times the strain associated with F_y. This calculation uses unfactored gravity loads. (See Figure 9.1-9.)

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

$$P = P_w + P_f = (0.065 \text{ ksf } (30 \text{ ft.}) + 0.02 \text{ ksf } (10 \text{ ft.}))(200 \text{ ft.}) = 430 \text{ kips}$$

P is at the base of the wall rather than at the midheight.

$$c = \left(\frac{\varepsilon_m}{\varepsilon_m + \varepsilon_s}\right) d = \left(\frac{0.0025}{0.0025 + 0.0103}\right) 200 \text{ ft} = 38.94 \text{ ft}$$

$$a = 0.8c = 31.15 \text{ ft} = 373.8 \text{ in.}$$

$$C_m = 0.8 f_m' a b_{avg} = 2{,}560 \text{ kips}$$

Where b_{avg} is taken from the average area used earlier, 51.3 in.^{2}/ft.; see Figure 9.1-9 for locations of tension steel and compression steel (the rebar in the compression zone will act as compression steel). From this it can be seen that:

$$T_{s1} = (1.25 f_y)\left(\frac{40.27}{(2)(2 \text{ ft o.c.})}\right)(0.60) = 453 \text{ kips}$$

$$T_{s2} = (1.25 f_y)\left(\frac{120.79}{2}\right)(0.60) = 2{,}718 \text{ kips}$$

$$C_{s1} = f_y \left(\frac{6.73}{2 \text{ ft. o.c.}}\right)(0.60) = 121 \text{ kips}$$

$$C_{s2} = (f_y)\left(\frac{32.21}{(2)(2)}\right)(0.60) = 290 \text{ kips}$$

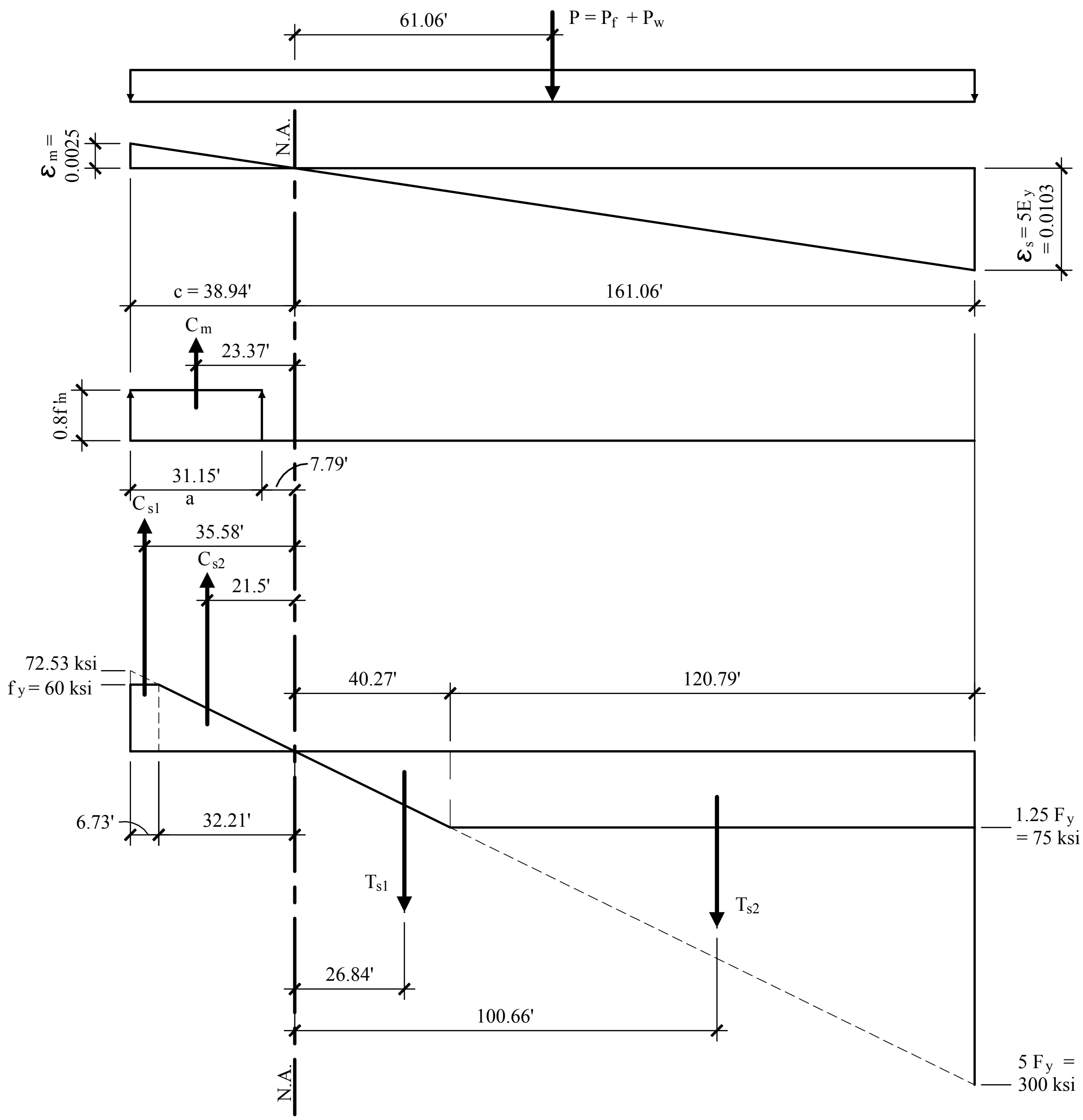


Figure 9.1-9 In-plane ductility check for side walls (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

Note that some authorities would not consider the compression resistance of reinforcing steel that is not enclosed within ties. The *Provisions* clearly allows inclusion of compression in the reinforcement.

$\Sigma C > \Sigma T$

$C_m + C_{s1} + C_{s2} > P + T_{s1} + T_{s2}$

$2{,}560 + 121 + 290 = 2{,}971 < 3{,}601 = 430 + 453 + 2{,}718$

Therefore, there is not enough compression capacity to ensure ductile failure.

In order to ensure ductile failure with #7 bars at 24 in. on center, one of the following revisions must be made: either add (3,601 kips - 2,971 kips) = 630 kips to C_m or reduce T by reducing A_s. Since this amount of reinforcement is needed for out-of-plane flexure, A_s cannot be reduced.

Try filling all cells for 10 ft - 0 in. from each end of the wall. As shown in Figure 9.1-10, this results in 10 additional grouted cells.

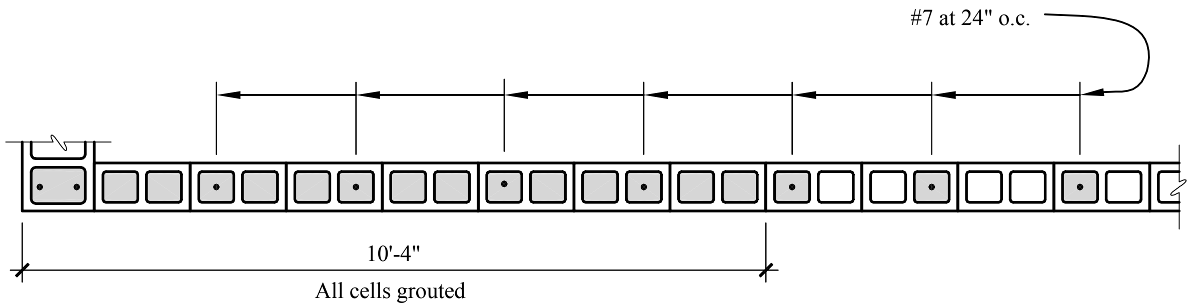


Figure 9.1-10 Grout cells solid within 10 ft of each end of side walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

Area of one grouted cell:	(8 in.)(5.13 in.) = 41 in.^{2}	
Volume of grout for one cell:	(6 in.)(5.13 in.)(30 ft.)/(144 in.^{2}/ft.^{2}) = 6.41 ft.^{3}	
Weight of grout for one cell:	(0.140 kcf)(6.41) = 0.90 kips/cell	
Additional P:	(10 additional cells)(0.9) = 9.0 kips	
Additional C_m:	$0.8 f_m'$ (41 in.^{2})(10 cells) = 656 kips	
Additional C_m - additional P:	656 kips - 9 kips = 647 kips	
Net additional C_m:	647 kips > 630 kips	OK

or, as expressed in terms of the above equation:

$\Sigma C > \Sigma T$

2,971 kips + 656 kips = 3,627 kips > 3,610 kips = 3,601 kips + 9 kips OK

Since C > T, the ductile criterion is satisfied.

This particular check is somewhat controversial. In the opinion of the authors, flexural yield is feasible for walls with *M/Vd* in excess of 1.0; this criterion limits the compressive strain in the masonry, which leads to good performance in strong ground shaking. For walls with *M/Vd* substantially less than 1.0, the wall will fail in shear before a flexural yield is possible. Therefore, the criterion does not affect performance. Well distributed and well developed reinforcement to control the shear cracks is the most important ductility attribute for such walls.

9.1.5.4.2 Strength check

The wall is so long with respect to its height that in-plane strength for flexure is acceptable by inspection.

9.1.5.5 Combined Loads

Combined loads are not calculated here because the in-plane strength is obviously very high. Out-of-plane resistance governs the flexural design.

9.1.5.6 Shear in Longitudinal Walls (Side Walls)

Compute out-of-plane shear at base of wall in accordance with *Provisions* Sec. 5.2.6.2.7[Sec. 4.6.1.3]:

$F_p = 0.4S_{DS}W_c$ = (0.4)(1.00)(65 psf)(28 ft/2) = 364 plf.

Information from the flexural design from Sec. 9.1.5.3 is needed to determine the required shear strength based upon development of the flexural capacity. The ratio of φM_N to M_U is the largest for the load case 0.7D + E. The load that would develop the flexural capacity is approximated by ratio (a second P-delta analysis does not seem justified for this check):

$$w' = w \times \frac{\phi M_N / \phi}{M_U} = 26\text{ psf} \times \frac{53.2 / 0.85}{35.0} = 46.5\text{ psf}$$

1.25 times this results in a load for shear design of 58 psf. Thus V_U = (58 psf)(28 ft. /2) = 818 plf. The capacity of computed per *Provisions* Eq.11.7.3.2[ACI 530, Eq. 3.2.1] :

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25\,P$$

M/Vd need not be taken larger than 1.0. A_n is taken as $b_w d$ = 8.32(3.81) = 31.7 in.^{2} per cell from Figure 9.17. Because this shear exists at both the bottom and the top of the wall, conservatively neglect the effect of P:

$$V_m = [4.0 - 1.75(1.0)](51.3\text{ in.}^2 / 2\text{ ft.})\sqrt{2,000} + 0 = 1.595\text{ klf}$$

$$\phi V_m = (0.8)(1.595) = 1.28\text{ klf} > 0.81\text{ klf}$$

As indicated in Sec. 9.1.4.1 and Sec. 9.1.5.1, the in-plane demand at the base of the wall, V_u = 2.5(211 kips) = 528 kips, and the shear capacity, ϕV_m is larger than 904 kips.

For the purpose of understanding likely behavior of the building somewhat better, V_n is estimated more accurately for these long walls:

$M/Vd = h/l$ = 28/200 = 0.14
P = 0.7D = 0.7(430) = 301 kip
V_m = [4.0 - 1.75(0.14)][200(51.3) + 2(10)91.5-51.3)](0.045) + 0.25(301) = 1870 + 75 = 1945 kip
$V_s = 0.5(A_v/s)f_y d$ = 0.5(0.62/4.0)(60)(200) = 930 kip
V_n = 1945 + 930 = 2875 kip
Maximum $V_n = 6\sqrt{f'_m}A$ = 6(0.045 ksi)(9234 in.^{2}) = 2493 < 2875 kip
φV_n = 0.8(2493) = 1994 kip
V_E = 211kip
V_n/V_E = 11.8 >> R used in design

In other words, it is unlikely that the long masonry walls will yield in either in-plane shear or flexure at the design seismic ground motion. The walls will likely yield in out-of-plane response and the roof diaphragm may also yield. The roof diaphragm for this building is illustrated in Sec. 10.2.

The combined loads for shear (orthogonal loading, per *Provisions* Sec. 5.2.5.2.2, Item a)[Sec. 4.4.2.3] are shown in Table 9.1-3.

Table 9.1-3 Combined Loads for Shear in Side Wall

	Out-of-Plane	In-Plane	Total
Case 1	1.00(810/1,280)+	0.30(528/1994)=	0.71 < 1.00 OK
Case 2	0.30(810/1,280)+	1.00(528/1994)=	0.45 < 1.00 OK

Values are in kips; 1.0 kip = 4.45 kN.

9.1.6 Transverse Walls

The transverse walls will be designed in a manner similar to the longitudinal walls. Complicating the design of the transverse walls are the door openings, which leave a series of masonry piers between the doors.

9.1.6.1 Horizontal Reinforcement

The minimum reinforcement, per *Provisions* Sec. 11.3.8.3[ACI 530, Sec.1.13.6.3] , is (0.0007)(11.625 in.)(8 in.) = 0.065 in.^{2} per course. The maximum spacing of horizontal reinforcement is 48 in., for which the minimum reinforcement is 0.39 in.^{2}. Two #4 in bond beams at 48 in. on center would satisfy the requirement. The large amount of vertical reinforcement would combine to satisfy the minimum total reinforcement requirement. However, given the 100-ft length of the wall, a larger amount is desired for control of restrained shrinkage as discussed in Sec. 9.1.5.1. Two #5 at 48 in. on center will be used.

9.1.6.2 Vertical Reinforcement

The area for each bay subject to out-of-plane wind is 20 ft wide by 30 ft high because wind load applied to the doors is transferred to the masonry piers. However, the area per bay subject to both in-plane and out-of-plane seismic is reduced by the area of the doors. This is because the doors are relatively light compared to the masonry. See Figures 9.1-12 and 9.1-13.

9.1.6.3 Out-of-Plane Flexure

Out-of-plane flexure will be considered in a manner similar to that illustrated in Sec. 9.1.5.3 . The design of this wall must account for the effect of door openings between a row of piers. The steps are the same as identified previously and are summarized here for convenience:

1. Select a trial design,
2. Investigate to ensure ductility,
3. Make sure the trial design is suitable for wind (or other non-seismic) lateral loadings using IBC,
4. Calculate midheight deflection due to wind by IBC,
5. Calculate the seismic demand, and
6. Determine the seismic resistance and compare to the demand determined in Step 5.

9.1.6.3.1 Trial design

A trial design of 12-in.-thick CMU reinforced with two #6 bars at 24 in. on center is selected. The self-weight of the wall, accounting for horizontal bond beams at 4ft on center, is conservatively taken as 103 psf. Adjacent to each door jamb, the vertical reinforcement will be placed into two cells. See Figure 9.1-11.

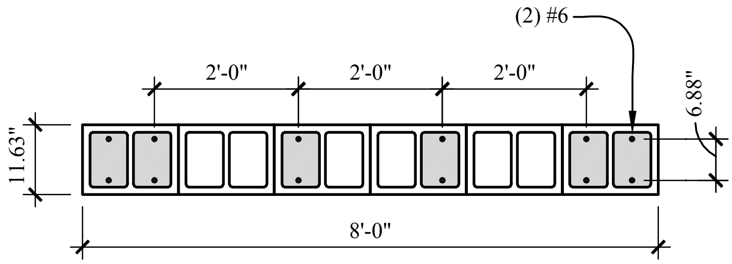


Figure 9.1-11 Trial design for piers on end walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

Next, determine the design loads. The centroid for seismic loads, accounting for the door openings, is determined to be 17.8 ft above the base. See Figures 9.1-12 and 9.1-13.

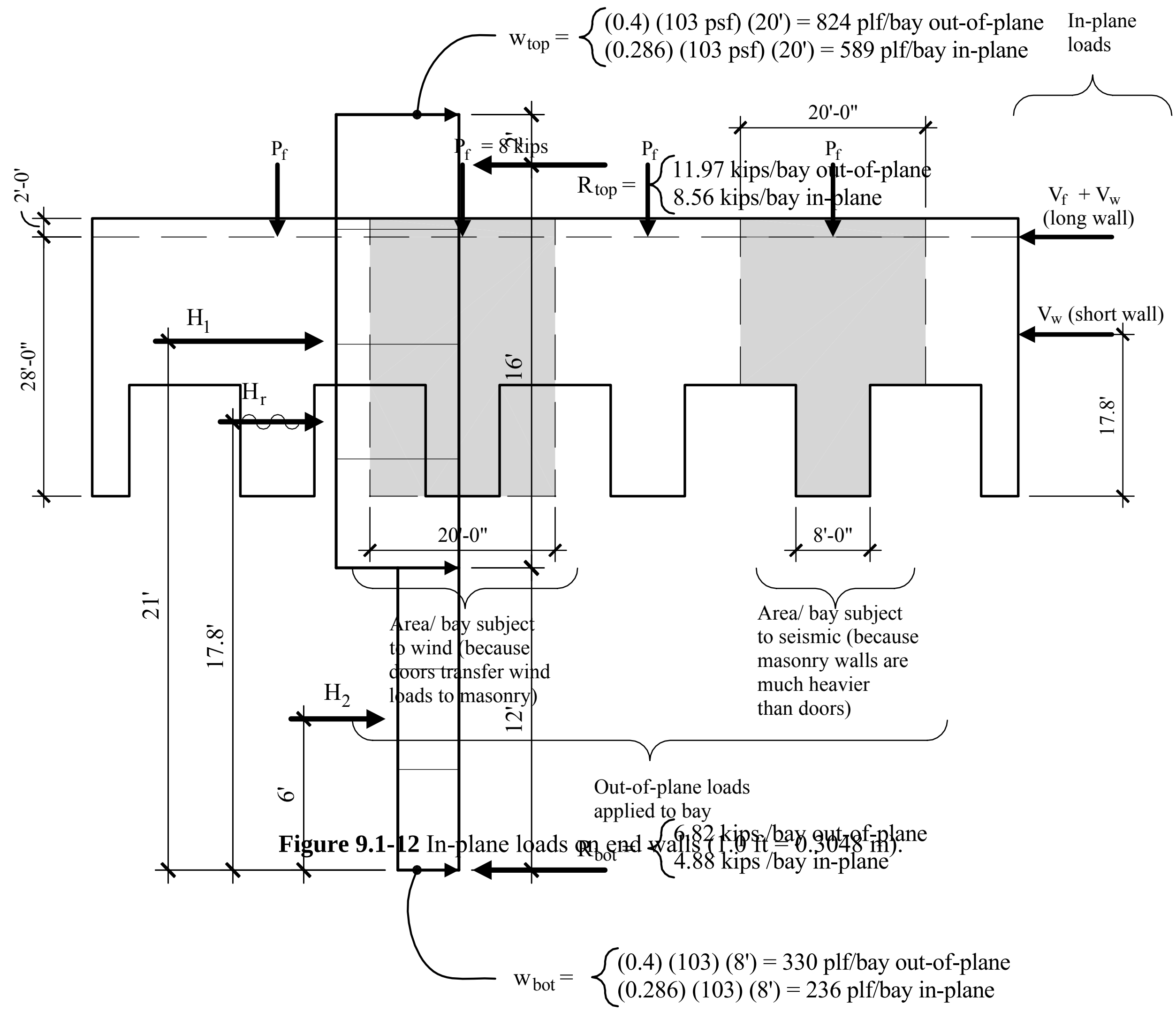


Figure 9.1-12 In-plane loads on end walls (1.0 ft = 0.3048 m).

Figure 9.1-13 Out-of-plane load diagram and resultant of lateral loads (1.0 ft = 0.3048 m, 1.0 lb = 4.45 N, 1.0 kip = 4.45 kN).

9.1.6.3.2 Investigate to ensure ductility

The critical strain condition is corresponds to a strain in the extreme tension reinforcement (which is a pair of #6 bars in the end cell in this example) equal to 1.3 times the strain at yield stress. See Figures 9.1-11 and 9.1-14.

For this case:

t = 11.63 in.
d = 11.63 - 2.38 = 9.25 in.
ε_m = 0.0025 (*Provisions* Sec. 11.6.2.1.b)[ACI 530, Sec. 3.2.2]
$\varepsilon_s = 1.3\ \varepsilon_y = 1.3\ (f_y/E_s)$ = 1.3 (60 ksi /29,000 ksi) = 0.0027 (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]

$$c = \left[\frac{\varepsilon_m}{(\varepsilon_m + \varepsilon_s)}\right] d = 4.45 \text{ in.}$$

a = 0.8c = 3.56 in. (*Provisions* Sec. 11.6.2.2)

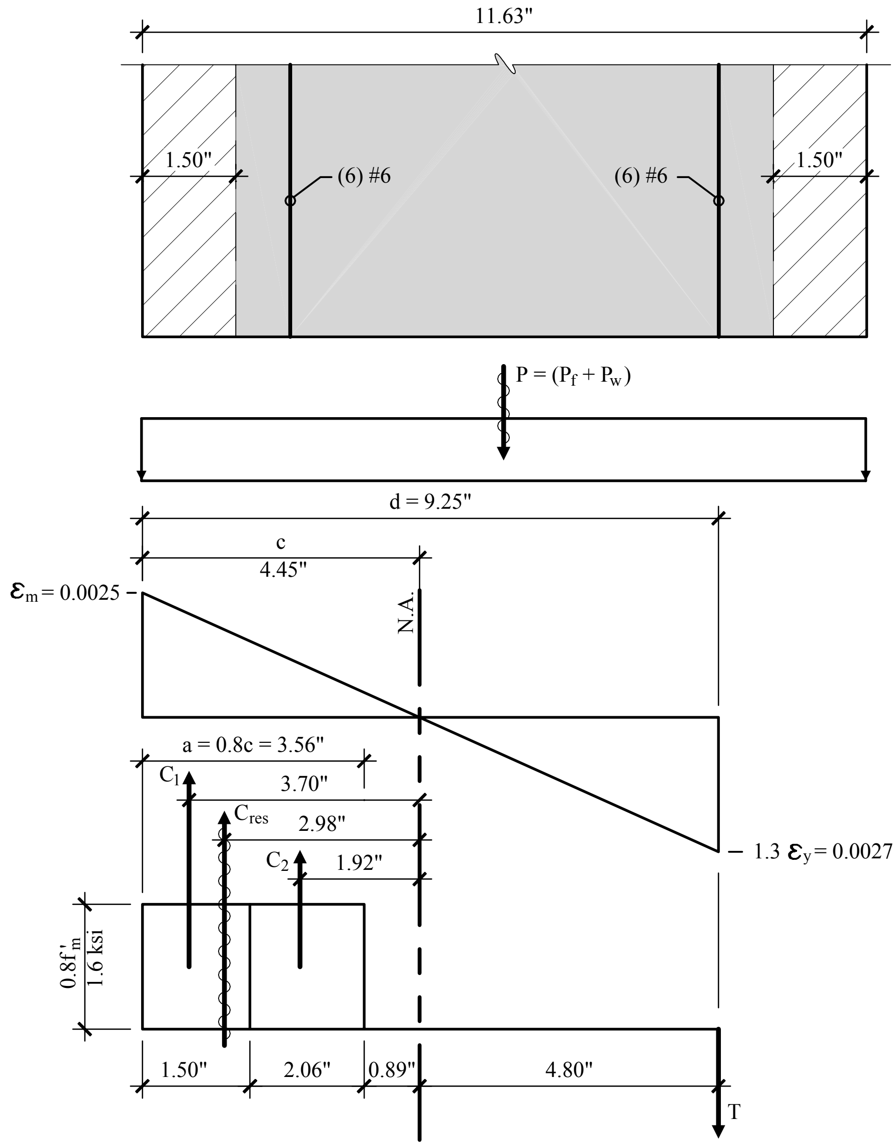


Figure 9.1-14 Investigation of out-of-plane ductility for end wall (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

Note that the Whitney compression stress block, a = 3.56 in. deep, is greater than the 1.50-in. face shell thickness. Thus, the compression stress block is broken into two components: one for full compression against solid masonry (the face shell) and another for compression against the webs and grouted cells but accounting for the open cells. These are shown as C_1 and C_2 on Figure 9.1-15. The values are computed using *Provisions* Sec. 11.6.2.1e:[ACI 530, 3.2.2.e];

C_1 = $0.80f_m'$ (1.50 in.)b = (0.80)(2 ksi)(1.50)(96) = 230 kips (for full length of pier)
C_2 = $0.80f_m'$ (a - 1.50 in.)(6(8 in.)) = (0.80)(2 ksi)(3.56 - 1.50)(48) = 158 kips

The 48 in. dimension in the C_2 calculation is the combined width of grouted cell and adjacent mortared webs over the 96-in. length of the pier.

T is based on $1.25F_y$ (*Provisions* Sec. 11.6.2.2)[ACI 530, Sec. 3.2.3.5.1]:

$T = 1.25F_yA_s = (1.25)(60 \text{ ksi})(6 \times 0.44 \text{ in.}^2) = 198$ kips/pier
$P = (P_f + P_w) = 8.0 \text{ k} + (0.103 \text{ ksf})(18 \text{ ft.})(20 \text{ ft.}) = 45.1$ kips/pier

P is computed at the head of the doors:

$C_1 + C_2 > P + T$
388 kip > 243 kips

Since the compression capacity is greater than the tension capacity, the ductility criterion is satisfied.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (1.5 times) and axial force to consider when performing the ductility check (factored loads).]

9.1.6.3.3 Check for wind loading using IBC

Note that load factors and section properties are different in the IBC and the *Provisions*. Note also that wind per bay is over the full 20 ft wide by 30 ft high bay as discussed above. (The calculations are not presented here.)

9.1.6.3.4 Calculate midheight deflection due to wind by IBC

Although the calculations are not presented here, note that in Figure 9.1-15 the neutral axis position and partial grouting results in a T beam cross section for the cracked moment of inertia. Use of the plastic neutral axis is a simplification for computation of the cracked moment of inertia. For this example, midheight out-of-plane deflection is 1.27 in. < 2.35 in. = 0.007*h*, which is acceptable.

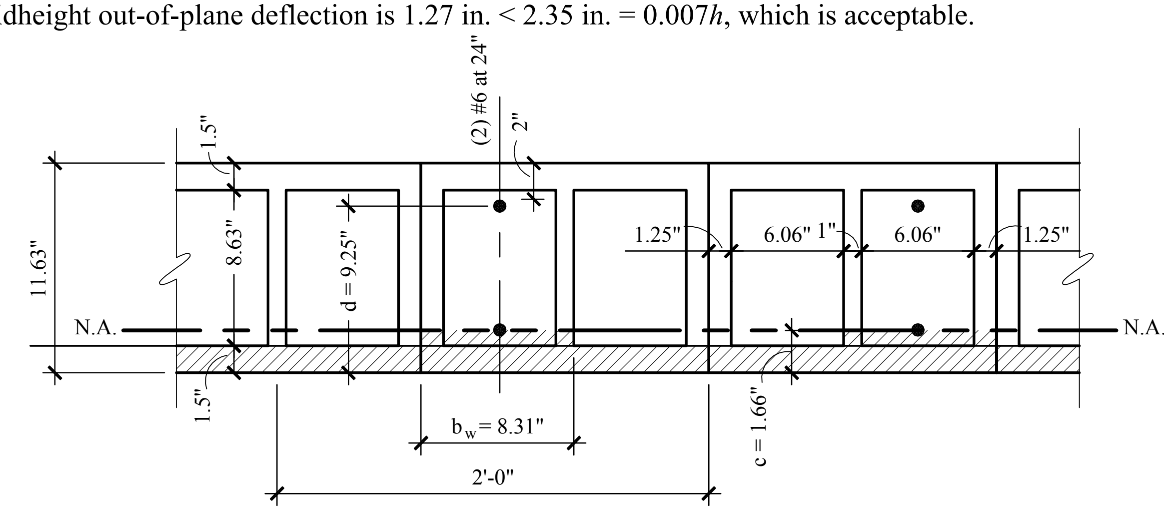


Figure 9.1-15 Cracked moment of inertia (I_{cr}) for end walls (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m). Dimension "c" depends on calculations shown for Figure 9.1-16.

9.1.6.3.5 Calculate Seismic Demand

For this example, the load combination with 0.7*D* has been used and, for this calculation, forces and moments over a single pier (width = 96 in.) are used. This does not violate the "$b > 6t$" rule (ACI 530 Sec. 7.3.3)[ACI 530, Sec. 3.2.4.3.3] because the pier is reinforced at 24 in. o.c. The use of the full width of the pier instead of a 24 in. width is simply for calculation convenience.

For this example, a P-delta analysis using RISA-2D was run. This resulted in:

Maximum moment, M_u = 66.22 ft-kips/bay = 66.22/20 ft = 3.31 klf (does not govern)
Moment at top of pier, M_u = 62.12 ft-kips/pier = 62.12 / 8 ft = 7.77 klf (governs)
Shear at bottom of pier, V_u = 6.72 kips/pier
Reaction at roof, V_u = 12.07 kips/bay
Axial force at base, R_u = 54.97 kips/pier

The shears do not agree with the reactions shown in Figure 9.1-13; because the results in Figure 9.1-13 do not include the P-delta consideration.

9.1.6.3.6 Determine moment resistance at the top of the pier

See Figure 9.1-16.

A_s = 6-#6 = 2.64 in.^{2}
d = 9.25 in.
T = 2.64(60) = 158.4 kip
$C = T + P$ = 203.5 kip
$a = C / (0.8f'_m b)$ = 203.5 / [0.8(2)96] = 1.32 in.

Because a is less than the face shell thickness (1.50 in.), compute as for a rectangular beam. Moments are computed about the centerline of the wall.

$M_N = C\,(t/2 - a/2) + P\,(0) + T\,(d - t/2)$
$= 203.5(5.81 - 1.32/2) + 158.4(9.25-1.32/2) = 1593$ in.-kip = 132.7 ft.-kip
$\varphi M_N = 0.85(132.7) = 112.8$ ft.-kip

Because moment capacity at the top of the pier, ϕM_n = 112.8 ft-kips, exceeds the maximum moment demand at top of pier, M_u = 62.1 ft-kips, the condition is acceptable but note that this is only tentative acceptance.

The *Provisions* requires a check of the combined loads in accordance with *Provisions* 5.2.5.2, Item a [Sec. 4.4.2.3]. See Sec. 9.1.6.5 for the combined loads check.

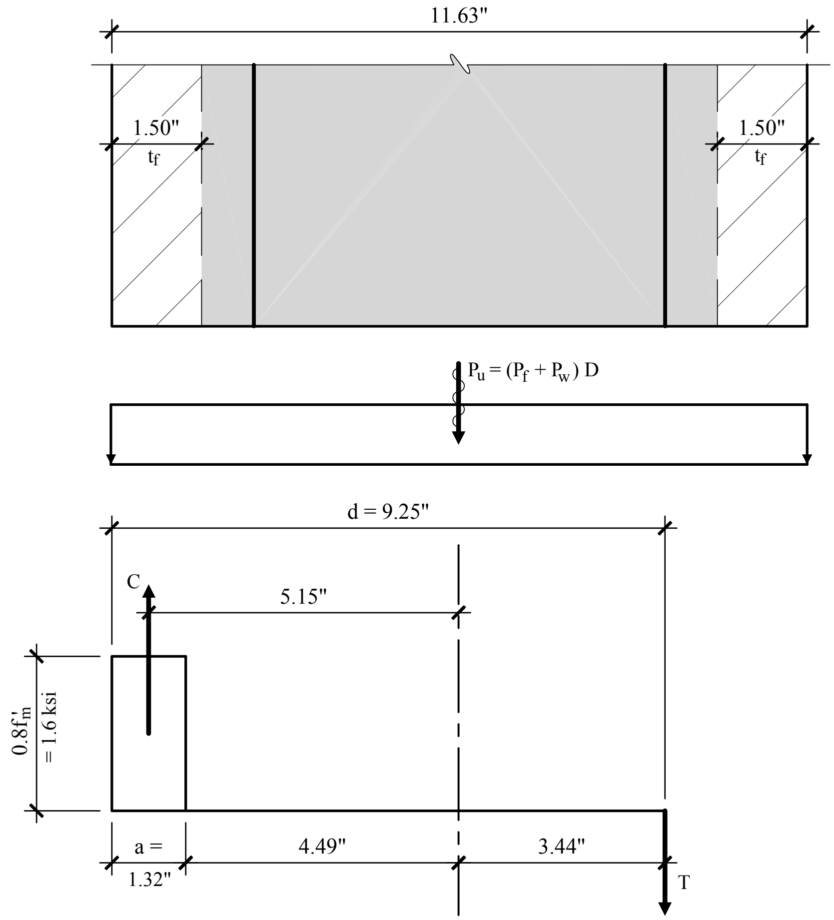


Figure 9.1-16 Out-of-plane seismic strength of pier on end wall (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

9.1.6.4 In-Plane Flexure

There are several possible methods to compute the shears and moments in the individual piers of the end wall. For this example, the end wall was modeled using RISA-2D. The horizontal beam was modeled at the top of the opening, rather than at its midheight. The in-plane lateral loads (from Figure 9.1-12) were applied at the 12-ft elevation and combined with joint moments representing transfer of the horizontal forces from their point of action down to the 12-ft elevation. Vertical load due to roof beams and the self-weight of the end wall were included. The input loads are shown on Figure 9.1-17. For this example:

w = (18 ft.)(103 psf) + (20 ft.)(20 psf) = 2.254 klf
H = (184 kip)/5 = 36.8 kip
M = 0.286((400 + 418)(28 - 12) + 470(17.8 - 12)) = 452 ft-kip

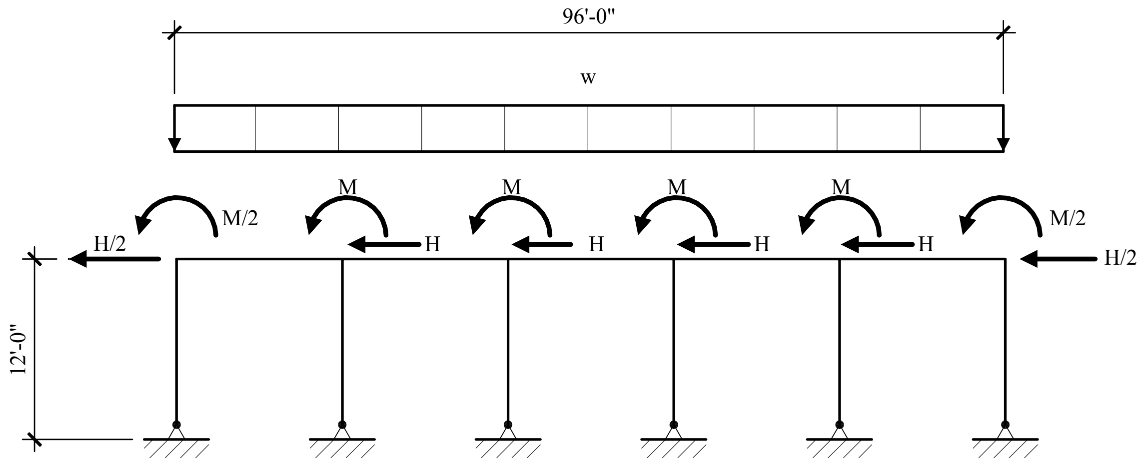


Figure 9.1-17 Input loads for in-plane end wall analysis (1.0 ft = 0.3048 m).

The input forces at the end wall are distributed over all the piers to simulate actual conditions. The RISA-2D frame analysis accounts for the relative stiffnesses of the 4-ft-and 8-ft-wide piers. The final distribution of forces, shears, and moments for an interior pier is shown on Figure 9.1-18.

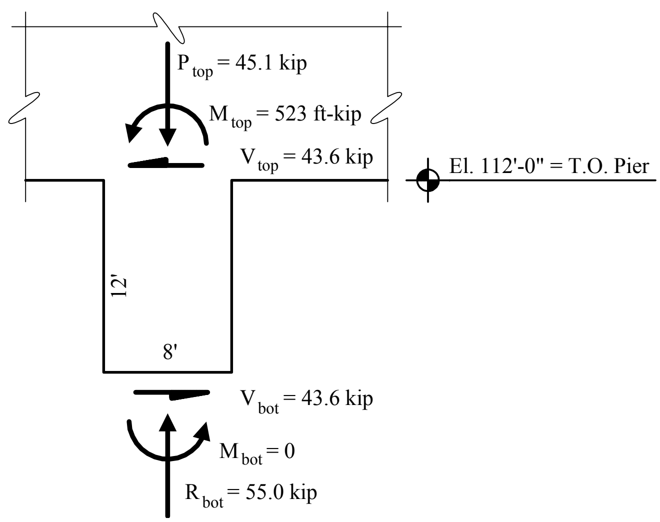


Figure 9.1-18 In-plane design condition for 8-ft-wide pier (1.0 ft = 0.3048 m).

As a trial design for in-plane pier design, use two #6 bars at 24 in. on center supplemented by adding two #6 bars in the cells adjacent to the door jambs (see Figure 9.1-19).

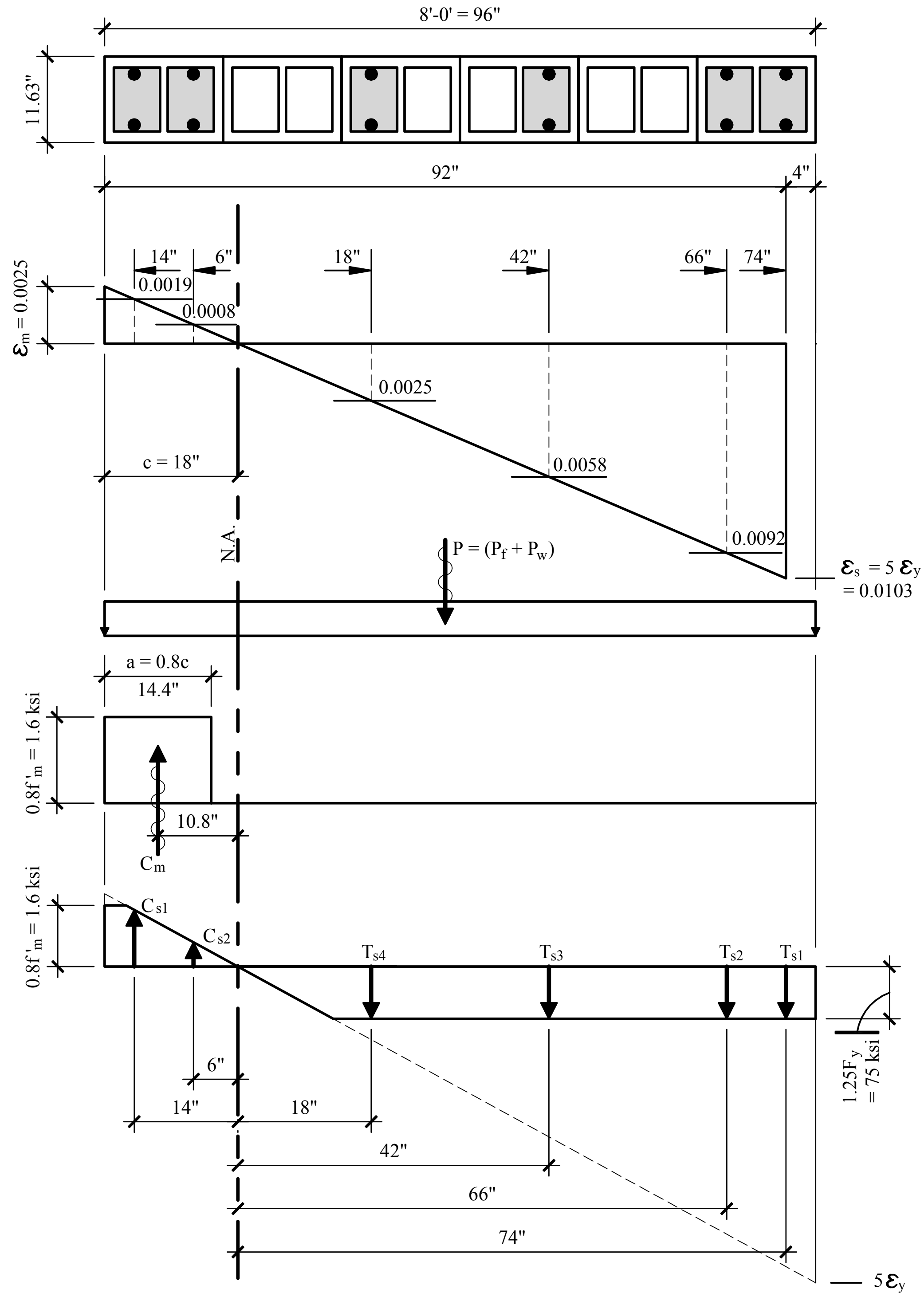


Figure 9.1-19 In-plane ductility check for 8-ft-wide pier (1.0 in = 25.4 mm, 1.0 ksi = 6.89 MPa).

The design values for in-plane design at the top of the pier are:

	Unfactored	$0.7D + 1.0E$	$1.4D + 1.0E$
Axia	$P = 45.1$ kips	$P_u = 31.6$ kips	$P_u = 63.2$ kips
Shea	$V = 43.6$ kips	$V_u = 43.6$ kips	$V_u = 43.6$ kips
Mom	$M = 523$ ft-kips	$M_u = 523$ ft-kips	$M_u = 523$ ft-kips

The ductility check is illustrated in Figure 9.1-19:

$\varepsilon_m = 0.0025$
$\varepsilon_s = 5\varepsilon_y = (5)(60/29{,}000) = 0.0103$
$d = 92$ in.

From the strain diagram, the strains at the rebar locations are:

$\varepsilon_{66} = 0.0092$
$\varepsilon_{42} = 0.0058$
$\varepsilon_{18} = 0.0025$
$\varepsilon_{6} = 0.0008$
$\varepsilon_{14} = 0.0019$

To check ductility, use unfactored loads:

$P = P_f + P_w = (0.020 \text{ ksf})(20 \text{ ft})(20 \text{ ft}) + (0.103 \text{ ksf})(18 \text{ ft})(20 \text{ ft})$
$P = 8 \text{ kips} + 37.1 \text{ kips} = 45.1 \text{ kips}$

$a = 0.8c = 14.4$ in.
$C_m = (0.8f_m{}')\text{ab} = 1.6 \text{ ksi})(14.4 \text{ in.})(11.63 \text{ in.}) = 268.0 \text{ kips}$
$T_{s1} = T_{s2} = T_{s3} = T_{s4} = (1.25F_y)(A_s) = (1.25)(60 \text{ ksi})(2 \times 0.44 \text{ in.2}) = 66 \text{ kips}$
$C_{s1} = F_yA_s(\varepsilon_{14}/\varepsilon_y) = (60 \text{ ksi})(2 \times 0.44 \text{ in.}^2)(0.0019/0.00207) = 48.5 \text{ kips}$
$C_{s2} = F_yA_s(\varepsilon_6/\varepsilon_y) = (60 \text{ ksi})(2 \times 0.44 \text{ in.}^2)(0.0008/0.00207) = 20.4 \text{ kips}$

$\Sigma C > \Sigma T + P$
$C_m + C_{s1} + C_{s2} > T_{s1} + T_{s2} + T_{s3} + T_{s4} + P$
268 + 48.5 + 20.4 > 66 + +66 + 66 + 66 + 45.1
336.9 kips > 309.1 kips

Since compression capacity exceeds tension capacity, ductile failure is ensured. Note that $1.25F_y$ is used for tension calculations per *Provisions* Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5-1] .

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

For the strength check, see Figure 9.1-20.

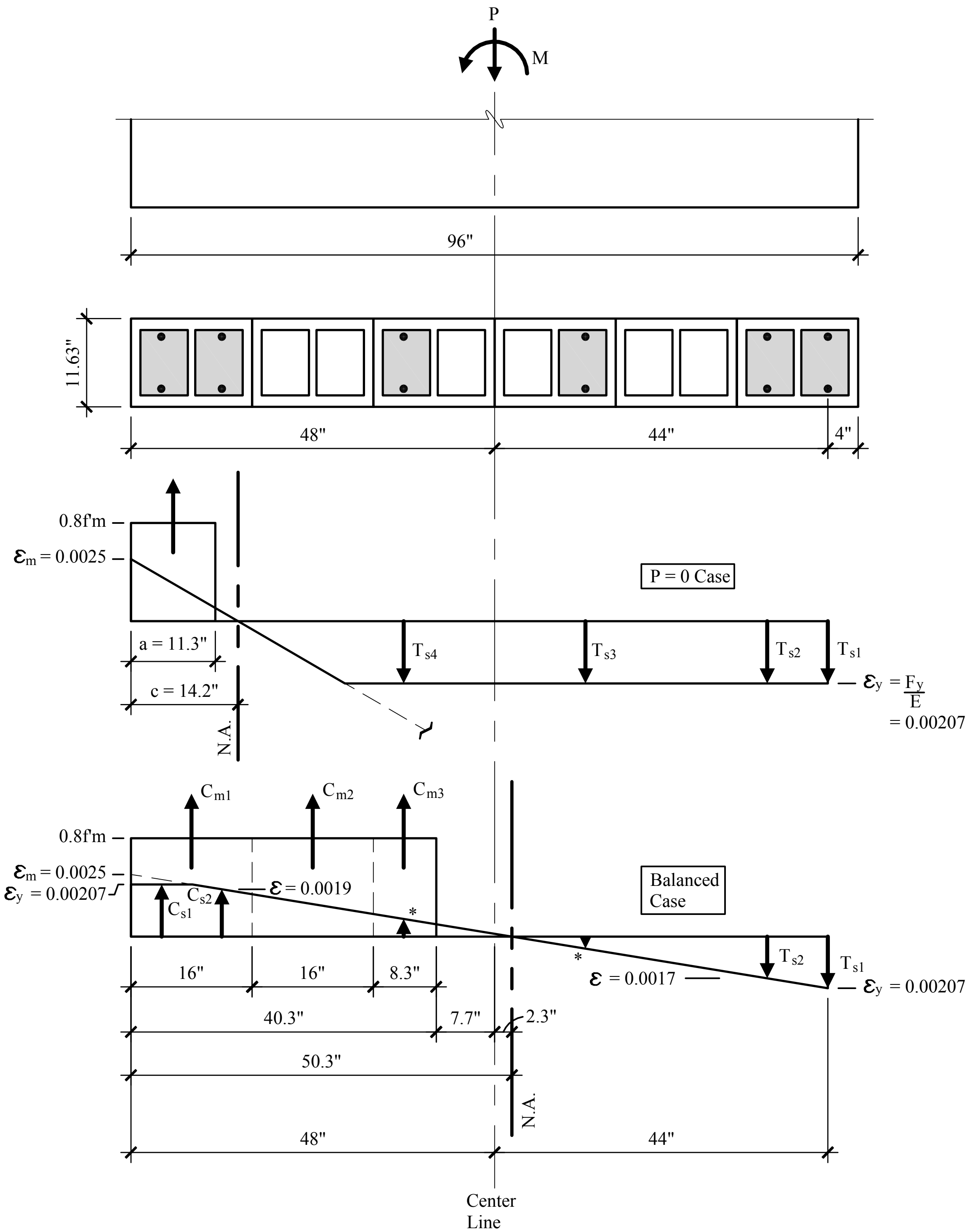


Figure 9.1-20 In-plane seismic strength of pier (1.0 in = 25.4 mm). Strain diagram superimposed on strength diagram for both cases. Note that low force in reinforcement is neglected in calculations.

To ascertain the strength of the pier, a ϕP_n - ϕM_n curve will be developed. Only the portion below the "balance point" will be examined as that portion is sufficient for the purposes of this example. Ductile failures occur only at points on the curve that are below the balance point so this is consistent with the overall approach).

For the $P = 0$ case, assume all bars in tension reach their yield stress and neglect compression steel (a conservative assumption):

$T_{s1} = T_{s2} = T_{s3} = T_{s4}$ = (2)(0.44 in.2)(60 ksi) = 52.8 kips
C_m = ΣT_s = (4)(52.8) = 211.2 kips
C_m = $0.8f'_m ab$ = (0.8)(2 ksi)a(11.63 in.) = 18.6a

Thus, a = 11.3 in. and $c = a/\phi$ = 11.3 / 0.8 = 14.2 in.

$\Sigma M_{cl} = 0$:
M_n = 42.35 $C_m + 44T_{s1} + 36T_{s2} + 12T_{s3} - 12T_{s4}$ = 13,168 in.-kips
ϕM_n = (0.85)(13,168) = 11,193 in.-kips = 933 ft-kips

For the balanced case:

d = 92 in.
ε = 0.0025
ε_y = 60/29,000 = 0.00207

$$c = \left(\frac{\varepsilon_m}{\varepsilon_m + \varepsilon_y}\right) d = 50.3 \text{ in.}$$

$a = 0.8c$ = 40.3 in.

Compression values are determined from the Whitney compression block adjusted for fully grouted cells or nongrouted cells:

C_{m1} = (1.6 ksi)(16 in.)(11.63 in.) = 297.8 kips
C_{m2} = (1.6 ksi)(16 in.)(2 × 1.50 in.) = 76.8 kips
C_{m3} = (1.6 ksi)(8.3 in.)(11.63 in.) = 154.4 kips
C_{s1} = (0.88 in.2)(60 ksi) = 52.8 kips
C_{s2} = (0.88 in.2)(60 ksi)(0.0019 / 0.00207) = 48.5 kips
T_{s2} = (0.88 in.2)(60 ksi) = 52.8 kips
T_{s2} = (0.88 in.2)(60 ksi)(0.0017 / 0/00207) = 43.4 kips

$\Sigma F_y = 0$:
$P_n = \Sigma C - \Sigma T$ = 297.8 + 76.8 + 154.4 + 52.8 + 48.5 -52.8 - 43.4 = 534 kips
ϕP_n = (0.85)(534) = 454 kips

$\Sigma M_{cl} = 0$:
$M_n = 40C_{m1} + 24C_{m2} + 11.85C_{m3} + 44C_{s1} + 36C_{s2} + 44T_{s1} + 36T_{s2}$ = 23,540 in.-kips
ϕM_n = (0.85)(23,540) = 20,009 in. - kips = 1,667 ft-kips

The two cases are plotted in Figure 9.1-21 to develop the ϕP_n - ϕM_n curve on the pier. The demand (P_u, M_u) also is plotted. As can be seen, the pier design is acceptable because the demand is within the ϕP_n - ϕM_n curve. (See the Birmingham 1 example in Sec. 9.2 for additional discussion of ϕP_n - ϕM_n curves.) By linear interpolation, φM_n at the minimum axial load is 968 kip.

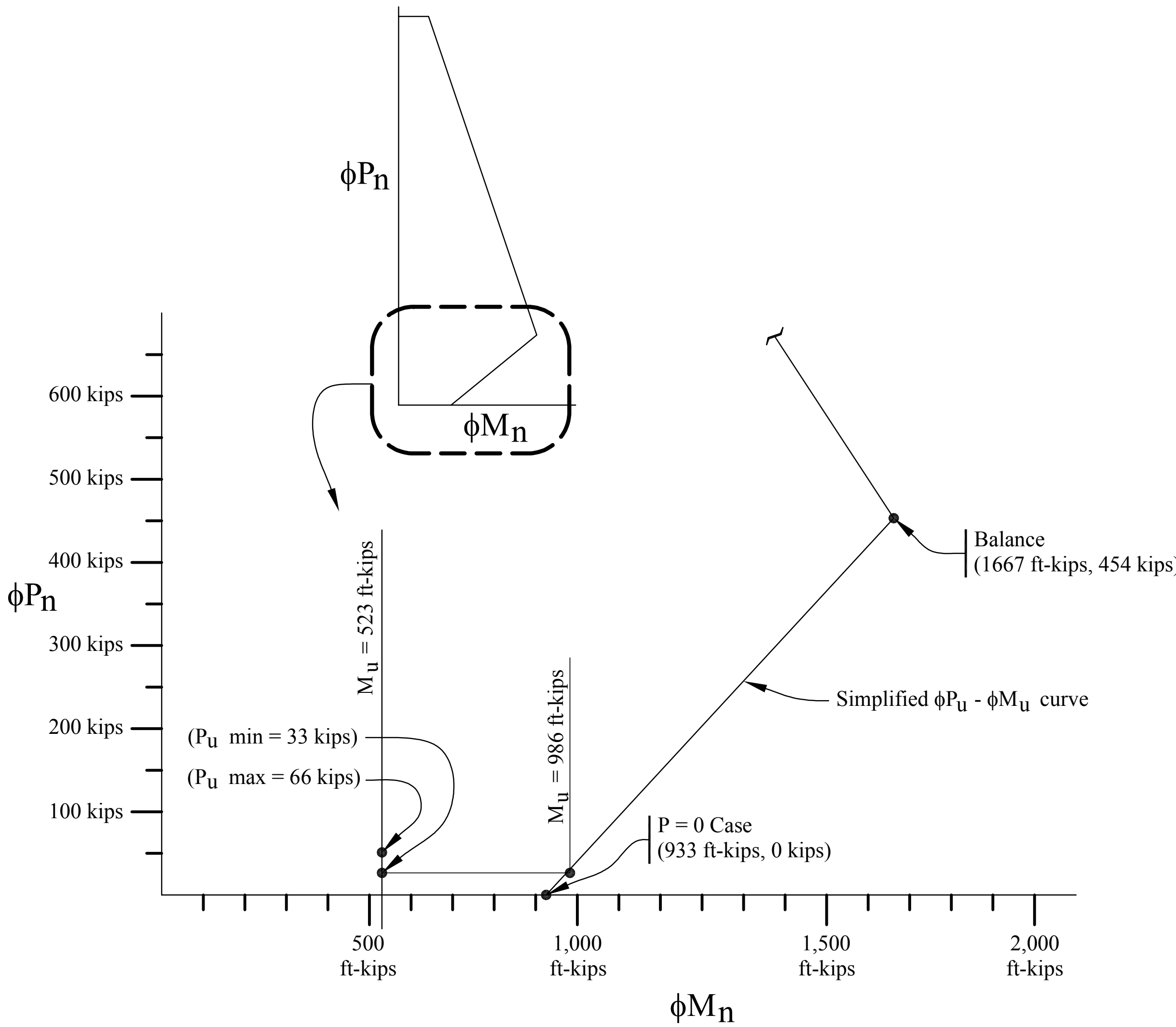


Figure 9.1-21 In-plane φP_{11} - φM_{11} diagram for pier (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.1.6.5 Combined Loads

Combined loads for in-plane and out-of-plane moments in piers at end walls, per *Provisions* Sec. 5.2.5.2.2, Item a, are shown in Table 9.1-4.

Table 9.1-4 Combined Loads for Flexure in End Pier

0.7*D*	Out-of-Plane		In-Plane		Total	
Case 1	1.0(62.12/112.8)	+	0.3(523/986)	=	0.71 < 1.00	OK
Case 2	0.3(62.12/112.8)	+	1.0(523/986)	=	0.70 < 1.00	OK

Values are in kips; 1.0 kip = 4.45 kN.

9.1.6.6 Shear at Transverse Walls (End Walls)

The shear at the base of the pier is 43.6 kips/bay. At the head of the opening where the moment demand is highest, the in-plane shear is slightly less (based on the weight of the pier). There, V = 43.6 kips - 0.286(8 ft)(12 ft)(0.103 ksf) = 40.8 kips. (This refinement in shear is not shown in Figure 9.1-18 although the difference in axial load at the two locations is shown.) The capacity for shear must exceed 2.5 times the demand or the shear associated with 125 percent of the flexural capacity. Using the results in Table 9.1-4, the 125 percent implies a factor on shear by analysis of:

$$1.25\left(\frac{1}{\text{Demand to capacity ratio}}\right)\left(\frac{1}{\phi}\right) = 1.25\left(\frac{1}{0.7}\right)\left(\frac{1}{0.85}\right) = 2.10$$

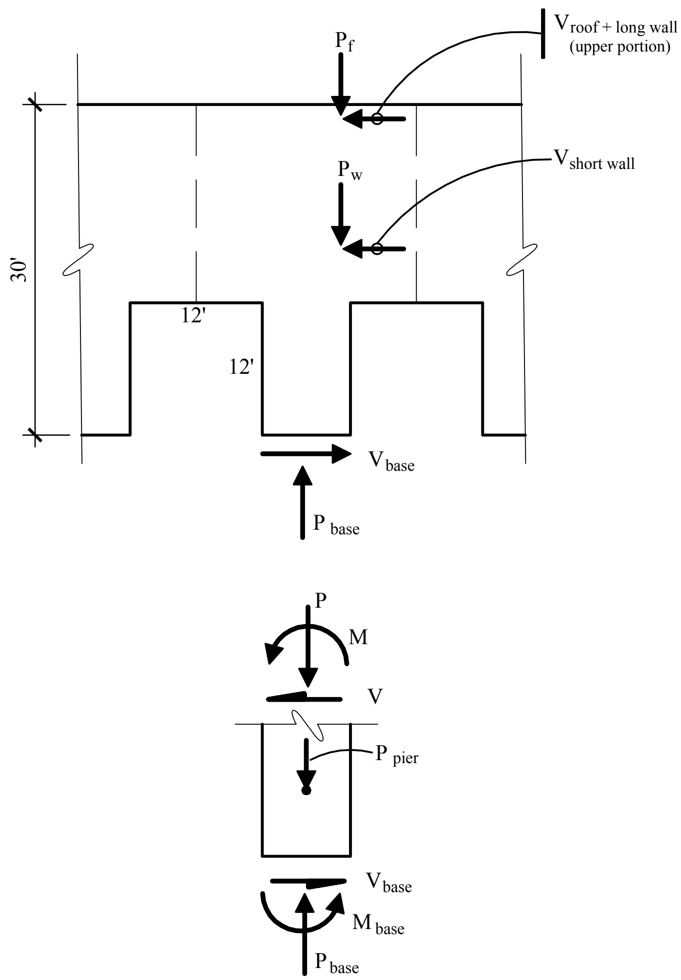


Figure 9.1-22 In-plane shear on end wall and pier (1.0 ft = 0.3048 m).

Therefore, the required shear capacities at the head and base of the pier are 91.6 kips and 85.7 kips, respectively.

The in-plane shear capacity is computed as follows where the net area, A_n, of the pier is the area of face shells plus the area of grouted cells and adjacent webs:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25P$$

A_n = (96 in. × 1.50 in.× 2) + (6 cells × 8 in. × 8.63 in.) = 702 in.^{2}

$$V_S = 0.5\left(\frac{A_V}{s}\right) f_y d_V$$

$$= 0.5\left(\frac{0.62 \text{ in.}^2}{48 \text{ in.}}\right)(60 \text{ ksi})(96 \text{ in.})$$

$$= 37.2 \text{ kips / bay}$$

At the head of the opening:

V_m = [4.0 - 1.75(1.0)](702 in.^{2})(0.0447 ksi) + (0.25)(0.7)(45.1 kips) = 78.5 kips/bay

ϕV_N = (0.8)(78.5 + 37.2) = 92.6 kips/bay

At the base:

V_m = [4.0 - 1.75(0)](702 in.^{2})(0.0447 ksi) + (0.25)(0.7)(55.0 kips) = 135.2 kips/bay

ϕV_N = (0.8)(135.2 + 37.2) = 137.9 kips/bay

As discussed previously, *M/Vd* need not exceed 1.0 in the above equation.

For out-of-plane shear, see Figure 9.1-13. Shear at the top of wall is 12.07 kips/bay and shear at the base of the pier is 6.72 kips/bay. From the values in the figure, the shear at the head of the opening is computed as 6.72 kips - (12 ft)(0.33 kip/ft) = 2.76 kips. The same multiplier of 2.10 for development of 125 percent of flexural capacity will be applied to out-of-plane shear resulting in 25.3 kips at the top of the wall, 5.80 kips at the head of the opening, and 14.11 kips at the base.

Out-of-plane shear capacity is computed using the same equation. $\Sigma b_w d$ is taken as the net area A_n. Note that *M/Vd* is zero at the support because the moment is assumed to be zero; however, a few inches into the span, *M/Vd* will exceed 1.0 so the limiting value of 1.0 is used here. This is typically the case when considering out-of-plane loads on a wall.

For computing shear capacity at the top of the wall:

A_n = $b_w d$ =((8 in./2 ft.) × 20 ft)(9.25 in.) = 740 in.^{2}
V_m = [4.0 - 1.75(1)](740 in.^{2})(0.0447 ksi) + (0.25)(8.0) = 76.9 kips/bay
ϕV_m = (0.8)(76.9) = 61.5 kips/bay

For computing shear capacity in the pier:

A_n =(8 in./cell)(6 cells)(9.25 in.) = 444 in.^{2}
V_m = [4.0 - 1.75(1)](444 in.^{2})(0.0447 ksi) + (0.25)(41.67) = 55.4 kips/bay
ϕV_m = (0.8)(55.4) = 44.3 kips/bay

The combined loads for shear at the end pier (per *Provisions* 5.2.5.2.2, Item a [Sec. 4.4-23]) are shown in

Table 9.1-5.

Table 9.1-5 Combined Loads for Shear in End Wall

	In-Plane		Out-of-Plane		Total
Case 1 Pier base	1.0(91.6/137.9)	+	0.3(14.11/44.3)	=	0.76 < 1.00 OK
Case 2 Pier base	0.3(91.6/137.9)	+	1.0(14.11/44.3)	=	0.52< 1.00 OK
Case 1 Pier head	1.0(85.7/92.6)	+	0.3(5.80/44.3)	=	0.96 > 1.00 OK
Case 2 Pier head	0.3(85.7/92.6)	+	1.0(5.80/44.3)	=	0.41 < 1.00 OK

Values are in kips; 1.0 kip = 4.45 kN.

9.1.7 Bond Beam

Reinforcement for the bond beam located at the elevation of the roof diaphragm can be used for the diaphragm chord. The uniform lateral load for the design of the chord is the lateral load from the long wall plus the lateral load from the roof and is equal to 0.87 klf. The maximum tension in rebar is equal the maximum moment divided by the diaphragm depth:

M/d = 4,350 ft-kips/100 ft = 43.5 kips

The seismic load factor is 1.0. The required reinforcement is:

$A_{reqd} = T/\phi F_y$ = 43.5/(0.85)(60) = 0.85 in.^{2}

This will be satisfied by two #6 bars, A_s = (2 × 0.44 in.^{2}) = 0.88 in.^{2}

In Sec. 10.2, the diaphragm chord is designed as a wood member utilizing the wood ledger member. Using either the wood ledger or the bond beam is considered acceptable.

9.1.8 In-Plane Deflection

Deflection of the end wall (short wall) has two components as illustrated in Figure 9.1-23.

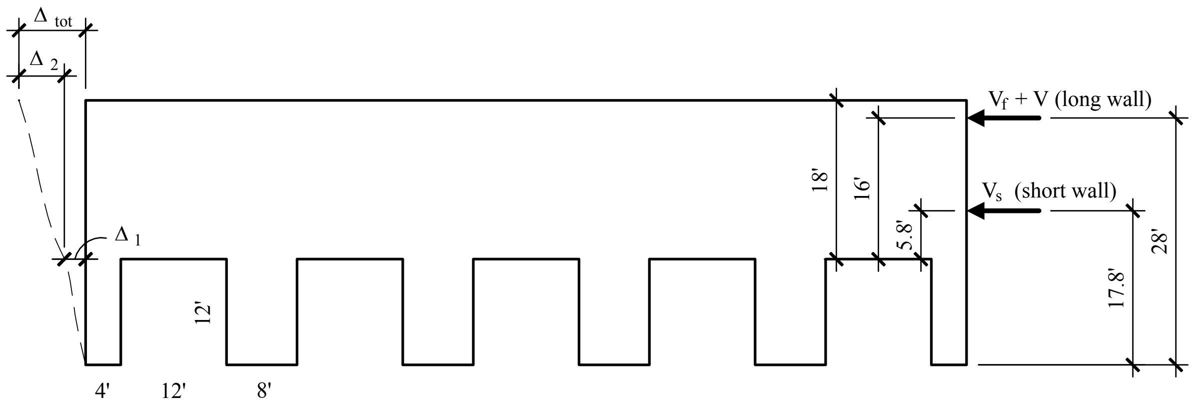


Figure 9.1-23 In-plane deflection of end wall (1.0 ft = 0.3048 m).

As obtained from the RISA 2D analysis of the piers, Δ_1 = 0.047 in.:

$$\Delta_2 = \sum \frac{\alpha VL}{AG}$$

where α is the form factor equal to 6/5 and

$G = E_m/2(1 + \mu) = 1500$ ksi / 2(1 + 0.15) = 652 ksi
$A = A_n$ = Area of face shells + area of grouted cells
= (100 ft × 12 in./ft × 2 × 1.50 in.^{2}) +(50)(8 in.)(8.63 in.) = 7,050 in.^{2}

Therefore:

$$\Delta_2 = \left(\frac{6}{5}\right)\frac{(67.15)(5.8\times12)}{(7{,}050)(652)} + \left(\frac{6}{5}\right)\frac{(116.9)(16\times12)}{(7{,}050)(652)} = 0.0013 + 0.0059 = 0.007 \text{ in.}$$

and,

$\Delta_{total} = C_d(0.047 + 0.007) = 3.5(0.054$ in.) = 0.19 in. < 3.36 in.
(3.36 = $0.01h_n = 0.01h_{sx}$) (*Provisions* Sec. 11.5.4)

Note that the drift limits for masonry structures are smaller than for other types of structure. It is possible to interpret *Provisions* Table 5.2.8 [Table 4.5-1] to give a limit of $0.007h_n$ for this structure but that limit also is easily satisfied. The real displacement in this structure is in the roof diaphragm; see Sec. 10.2.

9.2 FIVE-STORY MASONRY RESIDENTIAL BUILDINGS IN BIRMINGHAM, ALABAMA; NEW YORK, NEW YORK; AND LOS ANGELES, CALIFORNIA

9.2.1 Building Description

In plan, this five-story residential building has bearing walls at 24 ft on center (see Figures 9.2-1 and 9.2-2). All structural walls are of 8-in.-thick concrete masonry units (CMU). The floor is of 8-in.-thick hollow core precast, prestressed concrete planks. To demonstrate the incremental seismic requirements for masonry structures, the building is partially designed for four locations: two sites in Birmingham, Alabama; a site in New York, New York; and a site in Los Angeles, California. The two sites in Birmingham have been selected to illustrate the influence of different soil profiles at the same location. The building is designed for Site Classes C and E in Birmingham. The building falls in Seismic Design Categories B and D in these locations, respectively. For Site Class D soils, the building falls in Seismic Design Categories C and D for New York and Los Angeles, respectively.

[Note that the method for assigning seismic design category for short period buildings has been revised in the 2003 *Provisions*. If the fundamental period, T_a, is less than $0.8T_s$, the period used to determine drift is less than T_s, and the base shear is computed using 2003 *Provisions* Eq 5.2-2, then seismic design category is assigned using just 2003 *Provisions* Table 1.4-1 (rather than the greater of 2003 *Provisions* Tables 1.4-1 and 1.4-2). This change results in the Birmingham Site Class E building being assigned to Seismic Design Category C instead of D. The changes to this example based on the revised seismic design category are not noted in the remainder of the example. The New York building provides an example of what the Seismic Design Category C requirements would be for the Birmingham Site Class E building.]

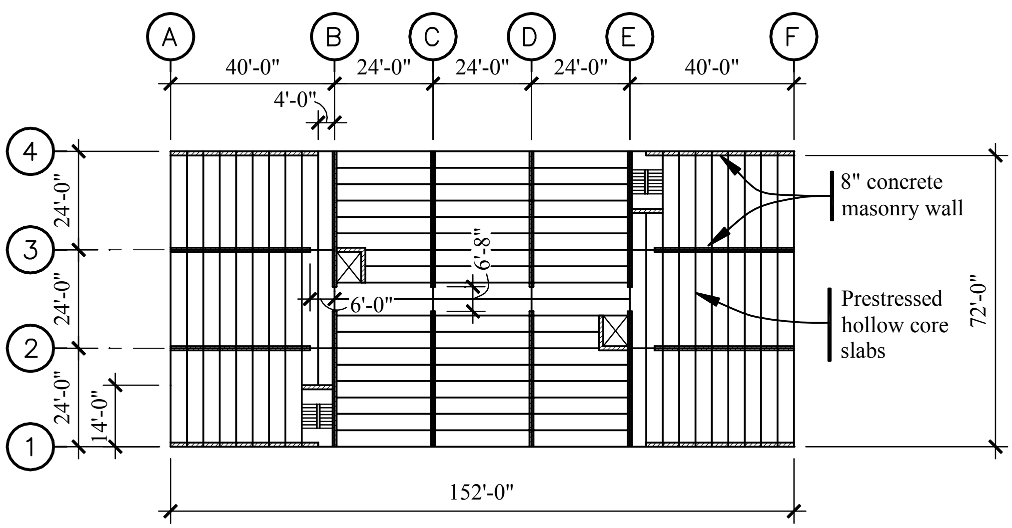


Figure 9.2-1 Typical floor plan (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

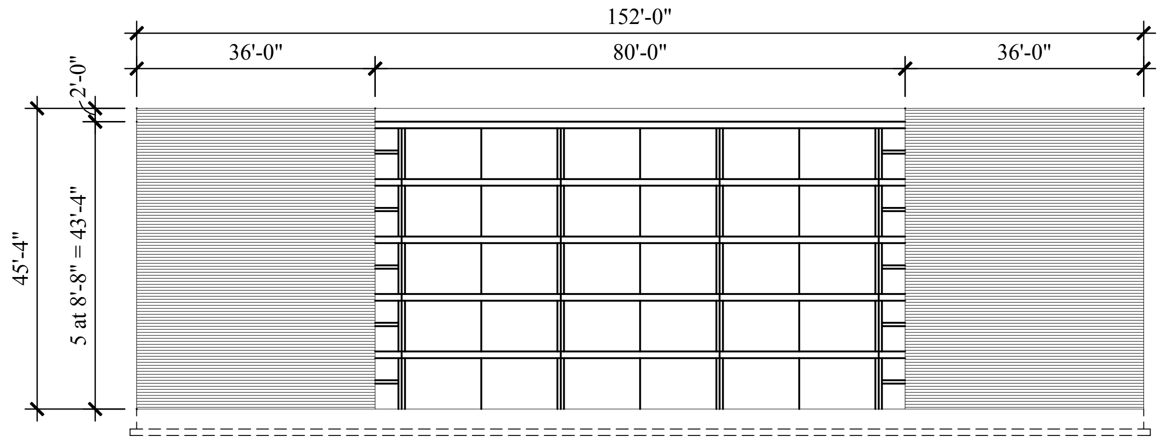


Figure 9.2-2 Building elevation (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

For the New York and both Birmingham sites, it is assumed that shear friction reinforcement in the joints of the diaphragm planks is sufficient to resist seismic forces, so no topping is used. For the Los Angeles site, a cast-in-place 2 ½-in.-thick reinforced lightweight concrete topping is applied to all floors. The structure is free of irregularities both in plan and elevation. The *Provisions*, by reference to ACI 318, requires reinforced cast-in-place toppings as diaphragms in Seismic Design Category D and higher. Thus, the Birmingham example in Site Class E would require a topping, although that is not included in this example.

Provisions Chapter 9 has an appendix (intended for trial use and feedback) for the design of untopped precast units as diaphragms. The design of an untopped diaphragm for Seismic Design Categories A, B, and C is not explicitly addressed in ACI 318. The designs of both untopped and topped diaphragms for these buildings are described in Chapter 7 of this volume using ACI 318 for the topped diaphragm in the Los Angeles building and using the appendix to *Provisions* Chapter 9 for untopped diaphragms in the New York building. It is assumed here that the diaphragm for the Birmingham 2 example would be similar to the New York example, and the extra weight of the Birmingham 2 topping is not included in the illustration here.

No foundations are designed in this example. However, for the purpose of determining the site class coefficient (*Provisions* Sec. 4.1.2.1 [Sec. 3.5]), a stiff soil profile with standard penetration test results of $15 < N < 50$ is assumed for Los Angeles and New York sites resulting in a Site Class D for these two locations. For Birmingham, however, one site has soft rock with $N > 50$ and the other has soft clay with $N < 15$, which results in Site Classes C and E, respectively. The foundation systems are assumed to be able to carry the superstructure loads including the overturning moments.

The masonry walls in two perpendicular directions act as bearing and shear walls with different levels of axial loads. The geometry of the building in plan and elevation results in nearly equal lateral resistance in both directions. The walls are constructed of CMU and are typically minimally reinforced in all locations. The walls are assumed to act as columns in their planes. Figure 9.2-3 illustrates the wall layout.

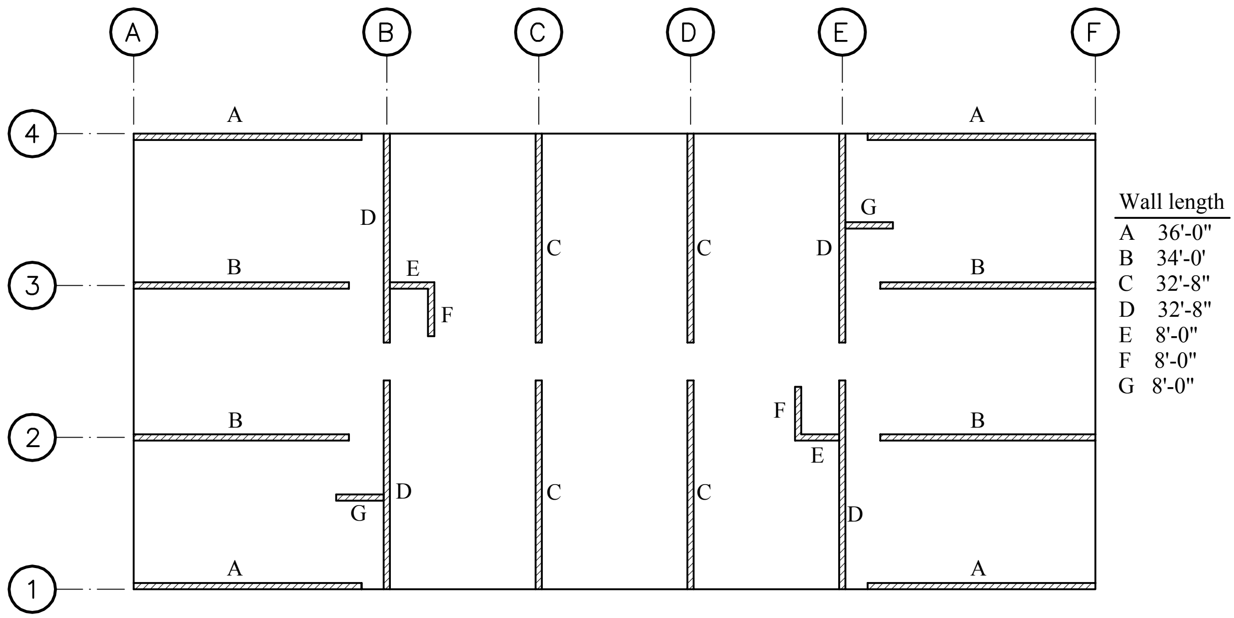


Figure 9.2-3 Plan of walls (1.0 ft = 0.3048 m).

The floors serve as horizontal diaphragms distributing the seismic forces to the walls and are assumed to be stiff enough to be considered rigid. There is little information about the stiffness of untopped precast diaphragms. The design procedure in the appendix to *Provisions* Chapter 9 results in a diaphragm intended to remain below the elastic limit until the walls reach an upper bound estimate of strength, therefore it appears that the assumption is reasonable.

Material properties are as follows:

The compressive strength of masonry, f'_m, is taken as 2,000 psi and the steel reinforcement has a yield limit of 60 ksi.

The design snow load (on an exposed flat roof) is taken as 20 psf for New York; design for snow does not control the roof design in the other locations.

This example covers the following aspects of a seismic design:

1. Determining the equivalent lateral forces,
2. Design of selected masonry shear walls for their in-plane loads, and
3. Computation of drifts.

See Chapter 7 of this volume for the design and detailing of untopped and topped precast diaphragms.

9.2.2 Design Requirements

9.2.2.1 *Provisions* Parameters

The basic parameters affecting the design and detailing of the buildings are shown in Table 9.2-1.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

9.2.2.2 Structural Design Considerations

The floors act as horizontal diaphragms and the walls parallel to the motion act as shear walls for all four buildings

The system is categorized as a bearing wall system (*Provisions* Sec. 5.2.2[Sec. 4.3]). For Seismic Design Category D, the bearing wall system has a height limit of 160 ft and must comply with the requirements for special reinforced masonry shear walls (*Provisions* Sec. 11.11.5[Sec. 11.2.1.5]). Note that the structural system is one of uncoupled shear walls. Crossing beams over the interior doorways (their design is not included in this example) will need to continue to support the gravity loads from the deck slabs above during the earthquake, but are not designed to provide coupling between the shear walls.

The building is symmetric and appears to be regular both in plan and elevation. It will be shown, however, that the building is actually torsionally irregular. *Provisions* Table 5.2.5 [Table 4.4-1] permits use of the equivalent lateral force (ELF) procedure in accordance with *Provisions* Sec. 5.4 [Sec. 5.2] for Birmingham 1 and New York City (Seismic Design Categories B and C). By the same table, the Category D buildings must use a dynamic analysis for design. For this particular building arrangement, the modal response spectrum analysis does not identify any particular effect of the torsional irregularity, as will be illustrated.

Table 9.2-1 Design Parameters

Design Parameter	Value for Birmingham 1	Value for Birmingham 2	Value for New York	Value for Los Angeles
S_s (Map 1) [Figure 3.3-1]	0.3	0.3	0.4	1.5
S_1 (Map 2) [Figure 3.3-2]	0.12	0.12	0.09	0.6
Site Class	C	E	D	D
F_a	1.2	2.34	1.48	1
F_v	1.68	3.44	2.4	1.5
$S_{MS} = F_a S_s$	0.36	0.7	0.59	1.5
$S_{M1} = F_v S_1$	0.2	0.41	0.22	0.9
$S_{DS} = 2/3\ S_{MS}$	0.24	0.47	0.39	1
$S_{D1} = 2/3\ S_{M1}$	0.13	0.28	0.14	0.6
Seismic Design Category	B	D	C	D
Masonry Wall Type	Ordinary Reinforced	Special Reinforced	Intermediate Reinforced	Special Reinforced
Provisions Design Coefficients (Table 5.2.2 [4.3-1])				
R	2.0	3.5	2.5	3.5

Design Parameter	Value for Birmingham 1	Value for Birmingham 2	Value for New York	Value for Los Angeles
Ω_0	2.5	2.5	2.5	2.5
C_d	1.75	3.5	2.25	3.5
IBC Design Coefficients (presented for comparison with *Provisions* coefficients)				
R	2.5	5.0	3.5	5.0
Ω_0	2.5	2.5	2.5	2.5
C_d	1.75	3.5	2.25	3.5

The orthogonal effect (*Provisions* Sec. 5.2.5.2, Item a [Sec. 4.4.2]) applies to structures assigned to Seismic Design Categories C and D (all of the example buildings except for Birmingham 1). However, the arrangement of this building is not particularly susceptible to orthogonal effects. This is because the stresses developed under out-of-plane loading for short-height walls (story clear height is 8 ft) are low and, their contribution to orthogonal effects is minimal.

The walls are all solid and there are no significant discontinuities, as defined by *Provisions* Sec. 5.2.6.2.3 [Sec. 4.3.2.3], in the vertical elements of the seismic-force-resisting system.

Ignoring the short walls at stairs and elevators, there are eight shear walls in each direction, therefore, the system appears to have adequate redundancy (*Provisions* Sec. 5.2.6.2.4 [Sec. 4.3.3]). The reliability factor, however, will be computed. [See Sec. 9.2.3.1 for changes to the reliability factor.]

Tie and continuity requirements (*Provisions* Sec. 5.2.6.1.2 [Sec. 4.6]) must be addressed when detailing connections between floors and walls (see Chapter 7 of this volume).

Nonstructural elements (*Provisions* Chapter 14 [Chapter 6]) are not considered in this example.

Collector elements are required in the diaphragm for longitudinal response (*Provisions* Sec. 5.2.6.2.5 [Sec. 4.6]). Rebar in the longitudinal direction, spliced into bond beams, will be used for this purpose (see Chapter 7 of this volume).

Diaphragms must be designed for the required forces (*Provisions* Sec. 5.2.6.2.6 [Sec. 4.6]).

The bearing walls must be designed for the required force perpendicular to their plane (*Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3]).

Each wall is a vertical cantilever; there are no coupling beams. The walls are classified as masonry cantilever shear wall structures in *Provisions* Table 5.2.8 [Table 4.5-1], which limits interstory drift to 0.01 times the story height. *Provisions* Sec.11.5.4.1.1 also limits drift to 0.01 times the wall height for such a structure.

[The deflection limits have been removed from Chapter 11 of the 2003 *Provisions* because they were redundant with the general deflection limits. Based on ACI 530 Sec. 1.13.3.2, the maximum drift for all masonry structures is 0.007 times the story height. Thus, there appears to be a conflict between ACI 530 and 2003 *Provisions* Table 4.5-1.]

Vertical accelerations must be considered for the prestressed slabs in Seismic Design Category D (*Provisions* Sec. 5.2.6.4.3 [Sec. 4.6.3.1]); refer to Chapter 7 of this volume. The evaluation of such components involves the earthquake effect determined using *Provisions* Eq. 5.2.7-1 [4.2-1] and 5.2.7-2 [4.2-1]. The important load is the vertical effect ($-0.2S_{DS}D$), which reduces the effect of dead loads. Because the system is prestressed, application of this load might lead to tension where there would otherwise be no reinforcement. The reinforcement within the topping will control this effect. Refer to Sec. 7.1 of this volume for the design of precast, prestressed slabs and topping.

Design, detailing, and structural component effects are presented in the chapters of the *Provisions* that are relevant to the materials used.

9.2.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [Sec. 4.2.2]) are the same as those in ASCE 7 (and are similar to those in the IBC). The seismic load effect, E, is defined by *Provisions* Eq. 5.2.7-1 [4.2-1] and 5.2.7-2 [4.2-2] as:

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

9.2.3.1 Reliability Factor

Note that ρ is a multiplier on design force effects and applies only to the in-plane direction of the shear walls. For structures in Seismic Design Categories A, B and C, $\rho = 1.0$ (*Provisions* Sec. 5.2.4.1 [Sec. 4.3.3.1]). For structures in Seismic Design Category D, ρ is determined per *Provisions* Sec. 5.2.4.2 [Sec. 4.3.3.2].

For the transverse direction, ignoring accidental torsion:

$$r_{max_x} = \left(\frac{V_{wall}}{V_{story}}\right)\left(\frac{10}{l_w}\right) \cong \left(\frac{1}{8}\right)\left(\frac{10}{33}\right) = 0.038$$

and,

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}} = 2 - \frac{20}{0.038\sqrt{10,944}} = -3.03$$

Since the computed $\rho < 1.0$ use $\rho = 1.0$ for the transverse direction. Accidental torsion does not change r_{max_x} enough to change this conclusion.

Based on similar calculations for the longitudinal direction, ρ is determined to be 1.0.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For structures assigned to Seismic Design Categories B and C, $\rho = 1.0$ in all cases. For a shear wall building assigned to Seismic Design Category D, $\rho = 1.0$ as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. The intent is that the aspect ratio is based on story height, not total height. Therefore, the redundancy factor would not have to be investigated ($\rho = 1.0$) for the structure(s) assigned to Seismic Design Category D.]

9.2.3.2 Combination of Load Effects

The seismic load effect, E, determined for each of the buildings is:

Birmingham 1	$E = (1.0)Q_E \pm (0.2)(0.24)D = Q_E \pm 0.05D$
Birmingham 2	$E = (1.0)Q_E \pm (0.2)(0.47)D = Q_E \pm 0.09D$
New York	$E = (1.0)Q_E \pm (0.2)(0.39)D = Q_E \pm 0.08D$
Los Angeles	$E = (1.0)Q_E \pm (0.2)(1.00)D = Q_E \pm 0.20D$

The applicable load combinations from ASCE 7 are:

$$1.2D + 1.0E + 0.5L + 0.2S$$

when the effects of gravity and seismic loads are additive and

$$0.9D + 1.0E + 1.6H$$

when the effects of gravity and seismic loads are counteractive. (H is the effect of lateral pressures of soil and water in soil.)

Load effect H does not apply for this design, and the snow load effect, S, exceeds the minimum roof live load only at the building in New York. However, even for New York, the snow load effect is only used for combinations of gravity loading. Consideration of snow loads is not required in the effective seismic weight, W, of the structure when the design snow load does not exceed 30 psf (*Provisions* Sec. 5.3 [Sec. 5.2.1]).

The basic load combinations are combined with E as determined above, and the load combinations representing the extreme cases are:

Birmingham 1	$1.25D + Q_E + 0.5L$ $0.85D - Q_E$
Birmingham 2	$1.29D + Q_E + 0.5L$ $0.81D - Q_E$
New York	$1.28D + Q_E + 0.5L + 0.2S$ $0.82D - Q_E$
Los Angeles	$1.40D + Q_E + 0.5L$ $0.70D - Q_E$

These combinations are for the in-plane direction. Load combinations for the out-of-plane direction are similar except that the reliability coefficient (1.0 in all cases for in-plane loading) is not applicable.

It is worth noting that there is an inconsistency in the treatment of snow loads combined with seismic loads. IBC Sec. 1605.3 clearly deletes the snow term from the ASD combinations where the design snow load does not exceed 30 psf. There is no similar provision for the strength load combinations in the IBC for reference standard, ASCE 7.

[The strength design load combinations in the 2003 IBC do have a similar exemption for snow loads, but ASCE 7-02 load combinations do not.]

9.2.4 Seismic Design for Birmingham 1

9.2.4.1 Birmingham 1 Weights

Use 67 psf for 8-in.-thick, normal weight hollow core plank plus the nonmasonry partitions. This site is assigned to Seismic Design Category B, and the walls will be designed as ordinary reinforced masonry shear walls (*Provisions* Sec. 11.11.3 [Sec. 4.2.1.3]), which do not require prescriptive seismic reinforcement. However, both ACI 530 and IBC 2106.1.1.2 stipulate that ordinary reinforced masonry shear walls have a minimum of vertical #4 bars at 120 in. on center. [By reference to ACI 530, the 2003 *Provisions* (and 2003 IBC) do have prescriptive seismic reinforcement requirements for ordinary reinforced masonry shear walls. Refer to ACI 530 Sec. 1.13.2.2.3.] Given the length of the walls, vertical reinforcement of #4 bars at 8 ft on center works well for detailing reasons and will be used here. For this example, 45 psf will be assumed for the 8-in.-thick lightweight CMU walls. The 45 psf value includes grouted cells and bond beams in the course just below the floor planks.

Story weight, w_i, is computed as follows:

For the roof:

Roof slab (plus roofing) = (67 psf) (152 ft)(72 ft) = 733 kips
Walls = (45 psf)(589 ft)(8.67 ft/2) + (45 psf)(4)(36 ft)(2 ft) = <u>128 kips</u>
Total = 861 kips

Note that there is a 2-ft-high masonry parapet on four walls and the total length of masonry wall, including the short walls, is 589 ft.

For a typical floor:

Slab (plus partitions) = 733 kips
Walls = (45 psf)(589 ft)(8.67 ft) = <u>230 kips</u>
Total = 963 kips

Total effective seismic weight, W = 861 + (4)(963) = 4,713 kips

This total excludes the lower half of the first story walls, which do not contribute to seismic loads that are imposed on CMU shear walls.

9.2.4.2 Birmingham 1 Base Shear Calculation

The seismic response coefficient, C_s, is computed using *Provisions* Sec. 5.4.1.1 [Sec. 5.2.1.1].
Per *Provisions* Eq. 5.4.1.1-1 [Eq. 5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.24}{2/1} = 0.12$$

The value of C_s need not be greater than *Provisions* Eq. 5.4.1.1-2 [Eq. 5.2-3]:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.13}{0.338(2/1)} = 0.192$$

T is the fundamental period of the building approximated per *Provisions* Eq. 5.4.2.1-1[Eq. 5.2-6] as:

$$T_a = C_r h_n^x = (0.02)(43.33^{0.75}) = 0.338 \text{ sec}$$

where C_r = 0.02 and x = 0.75 are from *Provisions* Table 5.4.2.1 [Table 5.2-2].

The value for C_s is taken as 0.12 (the lesser of the two computed values). This value is still larger than the minimum specified in *Provisions* Eq. 5.4.1.1-3:

$$C_s = 0.044IS_{DS} = (0.044)(1.0)(0.24) = 0\ 0.0106$$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4.1 [Eq. 5.2-1]as:

$$V = C_sW = (0.12)(4{,}713) = 566 \text{ kips}$$

9.2.4.3 Birmingham 1 Vertical Distribution of Seismic Forces

Provisions Sec. 5.4.4 [Sec. 5.2.3] stipulates the procedure for determining the portion of the total seismic load assigned to each floor level. The story force, F_x, is calculated using *Provisions* Eq. 5.4.3-1 [Eq. 5.2-10] and 5.4.3.-2 [Eq. 5.2-11], respectively, as:

$$F_x = C_{vx}V$$

and

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

For T = 0.338 sec < 0.5 sec, k = 1.0.

The seismic design shear in any story is determined from *Provisions* Eq. 5.4.4[Eq. 5.2-12]:

$$V_x = \sum_{i=x}^{n} F_i$$

The story overturning moment is computed from *Provisions* Eq. 5.4.5[Eq. 5.2-14]:

$$M_x = \sum_{i=x}^{n} F_i(h_i - h_x)$$

The application of these equations for this building is shown in Table 9.2-2.

Table 9.2-2 Birmingham 1 Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	861	43.34	37,310	0.3089	175	1.8e+14	1,515
4	963	34.67	33,384	0.2764	156		4,385
3	963	26.00	25,038	0.2073	117		8,272
2	963	17.33	16,692	0.1382	78		12,836
1	963	8.67	8,346	0.0691	39		17,739
∑	4,715		120,770	1.0000	566		

1.0 kips = 4.45 kN, 1.0 ft = 0.3048 m.

A note regarding locations of V and M: the vertical weight at the roof (5th level), which includes the upper half of the wall above the 5th floor (4th level), produces the shear V applied at the 5th level. That shear in turn produces the moment applied at the top of the 4th level. Resisting this moment is the rebar in the wall combined with the wall weight above the 4th level. Note that the story overturning moment is applied to the level below the level thatreceives the story shear. This is illustrated in Figure 9.2-4.

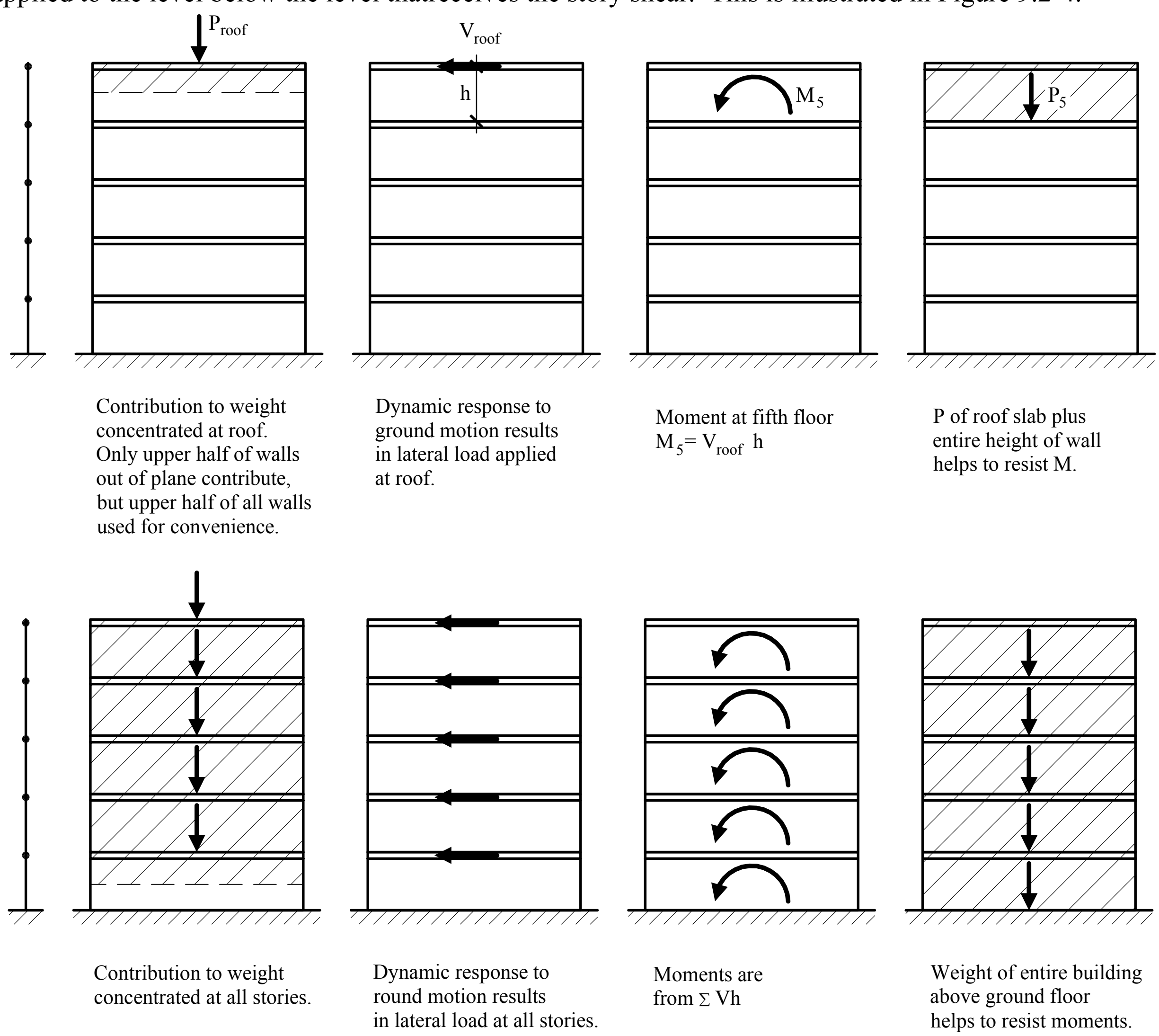


Figure 9.2-4 Location of moments due to story shears.

9.2.4.4 Birmingham 1 Horizontal Distribution of Forces

The wall lengths are shown in Figure 9.2-3. The initial grouting pattern is basically the same for walls A, B, and C. Because of a low relative stiffness, the effects Walls D, E, and F are ignored in this analysis. Walls A, B, and C are so nearly the same length that their stiffnesses will be assumed to be the same for this example.

Torsion is considered according to *Provisions* Sec. 5.4.4[Sec. 5.2.4]. For a symmetric plan, as in this example, the only torsion to be considered is the accidental torsion, M_{ta}, caused by an assumed eccentricity of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied loads.

Dynamic amplification of the torsion need not be considered for Seismic Design Category B per *Provisions* Sec. 5.4.4.3 [Sec. 5.2.4.3].

For this example, the building will be analyzed in the transverse direction only. The evaluation of Wall D is selected for this example. The rigid diaphragm distributes the lateral forces into walls in both directions. Two components of force must be considered: direct shear and shear induced by torsion.

The direct shear force carried by Wall D is one-eighth of the total story shear (eight equal walls). The torsional moment per *Provisions* Sec. 5.4.4.2 [Sec. 5.2.4.2] is:

$$M_{ta} = 0.05bV_x = (0.05)(152 \text{ ft})V_x = 7.6V_x$$

The torsional force per wall, V_t, is:

$$V_t = \frac{M_t K d}{\sum K d^2}$$

where K is the stiffness (rigidity) of each wall.

Because all the walls in this example are assumed to be equally stiff:

$$V_t = M_t \left[\frac{d}{\sum d^2} \right]$$

where d is the distance from each wall to the center of twisting.

$$\sum d^2 = 4(36)^2 + 4(12)^2 + 4(36)^2 + 4(12)^2 = 11{,}520$$

The maximum torsional shear force in Wall D, therefore is:

$$V_t = 7.6\ V(36/11{,}520) = 0.0238V$$

Total shear in Wall D is:

$$V_{tot} = 0.125V + 0.0238V = 0.149V$$

The total story shear and overturning moment may now be distributed to Wall D and the wall proportions checked. The wall capacity will be checked before considering deflections.

9.2.4.5 Birmingham 1 Transverse Wall (Wall D)

The strength or limit state design concept is used in the *Provisions*. This method was introduced in the 2002 edition of ACI 530, the basic reference standard for masonry design. Because strength design was not in prior editions of ACI 530, strength design of masonry as defined in the *Provisions* is illustrated here.

[The 2003 *Provisions* adopts by reference the ACI 530-02 provisions for strength design in masonry, and the previous strength design section has been removed. This adoption does not result in significant technical changes, and the references to the corresponding sections in ACI 530 are noted in the following sections.]

9.2.4.5.1 Birmingham 1 Shear Strength

Provisions Sec. 11.7.2 [ACI 530, Sec. 3.1.3] states that the ultimate shear loads must be compared to the design shear strength per *Provisions* Eq. 11.7.2.1:

$$V_u \le \phi V_n$$

The strength reduction factor, ϕ, is 0.8 (*Provisions* Table 11.5.3, ACI 530 [See 3.1.4.3]). The design shear strength, ϕV_n, must exceed the shear corresponding to the development of 1.25 times the nominal flexural strength of the member but need not exceed 2.5 times V_u (*Provisions* Sec. 11.7.2.2 [ACI 530, Sec. 3.1.3]). The nominal shear strength, V_n, is (*Provisions* Eq. 11.7.3.1-1 [ACI 530, Eq. 3-18]):

$$V_n = V_m + V_s$$

The shear strength provided by masonry is (*Provisions* Eq. 11.7.3.2 [ACI 530, Eq. 3-21]):

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right]A_n\sqrt{f'_m} + 0.25\,P$$

For grouted cells at 8 ft on center:

$$A_n = (2 \times 1.25 \text{ in.} \times 32.67 \text{ ft x } 12 \text{ in.}) + (8 \times 5.13 \text{ in.}^2 \times 5 \text{ cells}) = 1{,}185 \text{ in.}^2$$

The shear strength provided by reinforcement is given by *Provisions* Eq. 11.7.3.3 [ACI 530, Sec. 3.2.4.1.2.2] as:

$$V_s = 0.5\left(\frac{A_v}{s}\right)F_y d_v$$

The wall will have a bond beam with two #4 bars at each story to bear the precast floor planks and wire joint reinforcement at alternating courses. Common joint reinforcement with 9 gauge wires at each face shell will be used; each wire has a cross-sectional area of 0.017 in.^{2} With six courses of joint reinforcement and two #4 bars, the total area per story is 0.60 in.^{2} or 0.07 in.^{2}/ft.

$$V_s = 0.5(0.07 \text{ in.2/ft.})(60 \text{ ksi})(32.67 \text{ ft.}) = 68.3 \text{ kips}$$

The maximum nominal shear strength of the member (Wall D in this case) for $M/Vd_v > 1.00$ (the *Provisions* has a typographical error for the inequality sign) is given by *Provisions* Eq. 11.7.3.1-3 [ACI 530, Eq. 3-22]:

$$V_n(\max) = 4\sqrt{f'_m}A_n$$

The coefficient 4 becomes 6 for $M/Vd_v < 0.25$. Interpolation between yields the following:

$$V_N\ (\max) = (6.67 - 2.67\left(\frac{M}{Vd_V}\right))\sqrt{f'_m}A_n$$

The shear strength of Wall D, based on the equations listed above, is summarized in Table 9.2-3. Note that V_x and M_x in this table are values from Table 9.2-2 multiplied by 0.149 (which represents the portion of direct and torsional shear assigned to Wall D). P is the dead load of the roof or floor times the tributary area for Wall D. (Note that there is a small load from the floor plank parallel to the wall.)

Table 9.2-3 Shear Strength Calculations for Birmingham 1 Wall D

Story	V_x (kips)	M_x (ft-kips)	M_x/V_xd	2.5 V_x (kips)	P (kips)	φV_m (kips)	φV_s (kips)	φV_n (kips)	φV_n *max* (kips)
5	26	225	0.265	65.0	41	158.1	54.6	212.7	252.7
4	49.3	652	0.405	123.3	89	157.3	54.6	211.9	236.9
3	66.7	1230	0.564	166.8	137	155.1	54.6	209.7	218.8
2	78.4	1910	0.746	196.0	184	151.0	54.6	205.6	198.3
1	84.2	2640	0.960	210.5	232	144.8	54.6	199.4	174.1

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

V_U exceeds both φV_n and φV_n *max* at the first story. It would be feasible to add grouted cells in the first story to remedy the deficiency. However, it will be shown following the flexural design that the shear to develop 1.25 times the flexural capacity is 1.94(84.2 kips) = 163 kips, which is OK.

9.2.4.5.2 Birmingham 1 Axial and Flexural Strength

All the walls in this example are bearing shear walls since they support vertical loads as well as lateral forces. In-plane calculations include:

1. Strength check and
2. Ductility check

9.2.4.5.2.1 Strength check

The wall demands, using the load combinations determined previously, are presented in Table 9.2-4 for Wall D. In the table, Load Combination 1 is $1.25D + Q_E + 0.5L$ and Load Combination 2 is $0.85D + Q_E$.

Table 9.2-4 Demands for Birmingham 1 Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
54321	4.2e+12	8172534	5.111518e+13	2.2565e+17	3.576116e+12	2.256521e+17

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

Strength at the bottom story (where P, V, and M are the greatest) will be examined. (For a real design, all levels should be examined). The strength design will consider Load Combination 2 from Table 9.2-4 to be the governing case because it has the same lateral load as Load Combination 1 but with lower values of axial force.

For the base of the shear walls:

$P_{u_{min}}$ = 197 kips plus factored weight of lower ½ of 1st story wall = 197 + (0.85)(6.4) = 202 kips
$P_{u_{max}}$ = 307 + (1.25)(6.4) = 315 kips
M_u = 2,640 ft-kips

Try one #4 bars in each end cell and a #4 bar at 8 ft on center for the interior cells. A ϕP_n - ϕM_n curve, representing the wall strength envelope, will be developed and used to evaluate P_u and M_u determined above. Three cases will be analyzed and their results will be used in plotting the ϕP_n - ϕM_n curve.

In accordance with *Provisions* Sec. 11.6.2.1 [ACI 530, Sec. 3.2.2], the strength of the section is reached as the compressive strains in masonry reach their maximum usable value of 0.0025 for CMU. The force equilibrium in the section is attained by assuming an equivalent rectangular stress block of $0.8f'_m$ over an effective depth of 0.8*c*, where *c* is the distance of the neutral axis from the fibers of maximum compressive strain. Stress in all steel bars is taken into account. The strains in the bars are proportional to their distance from the neutral axis. For strains above yield, the stress is independent of strain and is taken as equal to the specified yield strength F_y. See to Figure 9.2-5 for strains and stresses for all three cases selected.

Case 1 (P = 0)

Assume all tension bars yield (which can be verified later):

T_{s1} = (0.20 in.^{2})(60 ksi) = 12.0 kips
T_{s2} = (0.20 in.^{2})(60 ksi) = 12.0 kips each

Because the neutral axis is close to the compression end of the wall, compression steel, C_{s1}, will be neglected (it would make little difference anyway) for Case 1:

$\Sigma F_y = 0$:
$C_m = \Sigma T$
C_m =(4)(12.0) = 48.0 kips

The compression block will be entirely within the first grouted cell:

$C_m = 0.8 f'_m ab$
48.0 = (0.8)(2.0 ksi)a(7.625 in)
a = 3.9 in. = 0.33 ft
$c = a/0.8$ = 0.33/0.8 = 0.41 ft

Thus, the neutral axis is determined to be 0.41 ft from the compression end on the wall, which is within the first grouted cell:

$\Sigma M_{cl} = 0$: (The math will be a little easier if moments are taken about the wall centerline.)
M_n = (16.33-0.33/2 ft)C_m + (16.00 ft) T_{s1} + (0.00 ft)ΣT_{s2} + (0.00 ft)P_n
M_n = (16.17)(48.0) + (16.00)(12) + 0 + 0 = 968 ft-kips
ϕM_n = (0.85)(968) = 823 ft-kips

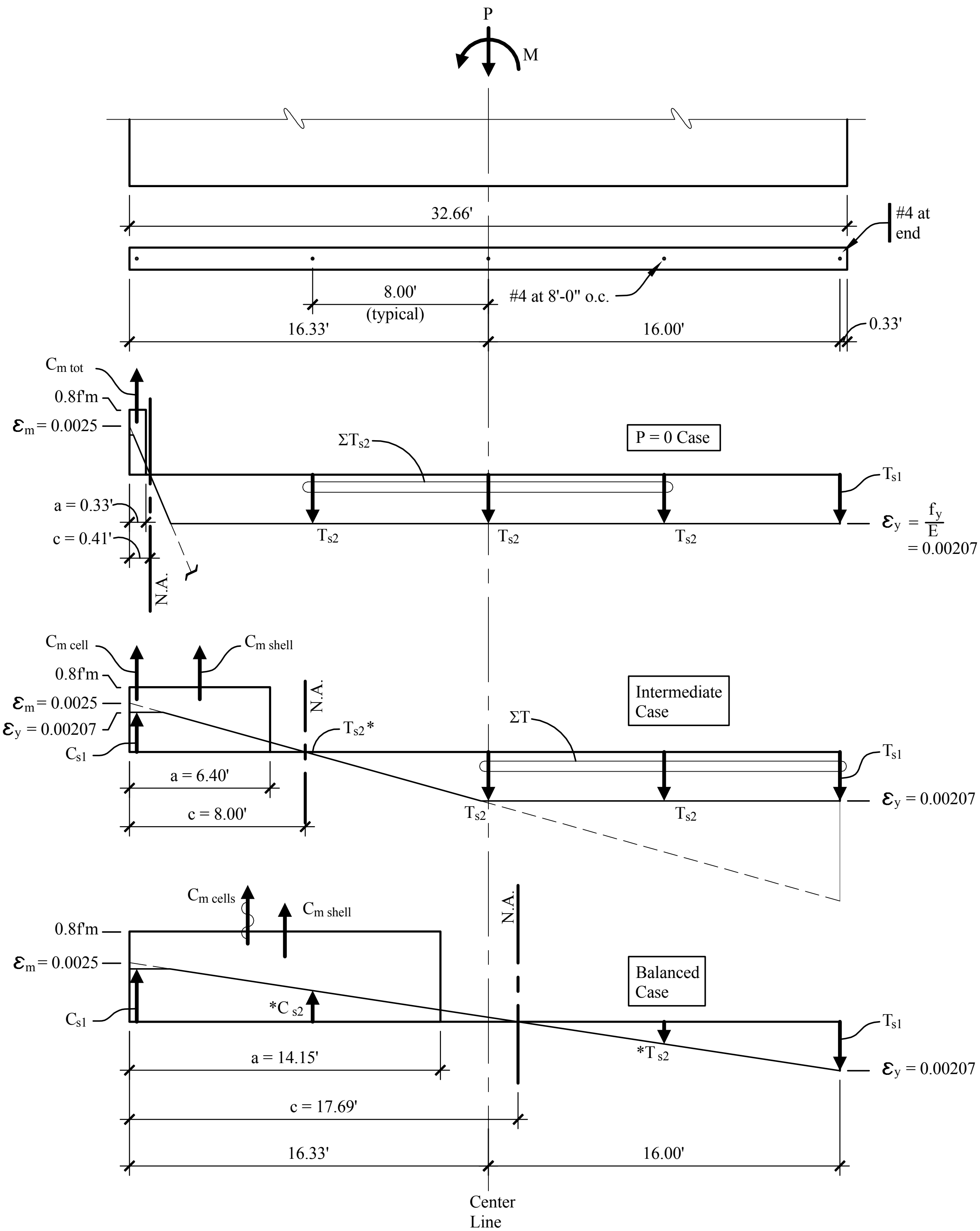


Figure 9.2-5 Strength of Birmingham 1 Wall D (1.0 ft = 0.3048 m). Strain diagram superimposed on strength diagram for the three cases. The low force in the reinforcement is neglected in the calculations.

To summarize, Case 1:

ϕP_n = 0 kips
ϕM_n = 823 ft-kips

<u>Case 2 (Intermediate case between P = 0 and P_{bal})</u>

Let c = 8.00 ft.(this is an arbitrary selection). Thus, the neutral axis is defined at 8 ft from the compression end of the wall:

$a = 0.8c = (0.8)(8.00) = 6.40$ ft

$C_{m\ shells} = 0.8f'_m$(2 shells)(1.25 in. / shell)(6.40 ft. (12 in./ft) = 307.2 kips
$C_{m\ cells} = 0.8\,f'_m$(41 in.^{2}) = 65.6 kips
$C_{m\ tot} = C_{m\ shells} + C_{m\ cells}$ = 307.2 + 65.6 = 373 kips

C_{s1} = (0.20 in.^{2})(60 ksi) = 12 kips (Compression steel is included in this case)
T_{s1} = (0.20 in.^{2})(60 ksi) = 12 kips
T_{s2} = (0.20 in.^{2})(60 ksi) = 12 kips each

Some authorities would not consider the compression resistance of reinforcing steel that is not enclosed within ties. The *Provisions* clearly allows inclusion of compression in the reinforcement.

$\Sigma F_y = 0$:
$C_{m\ tot} + C_{s1} = P_n + T_{s1} + \Sigma T_{s2}$
$373 + 12 = P_n + (3)(12.0)$
P_n = 349 kips
ϕP_n = (0.85)(349) = 297 kips

$\Sigma M_{cl} = 0$:
$M_n = (13.13\text{ ft})C_{m\ shell} + (16.00\text{ ft})(C_{m\ cell} + C_{s1}) + (16.00\text{ ft})T_{s1} + (8.00\text{ ft})T_{s2}$
M_n = (13.13)(307.2) + (16.00)(65.6 + 12) + (16.00)(12.0) + (8.00 ft)(12.0) = 5,563 ft-kips
ϕM_n = (0.85)(5,563) = 4,729 ft-kips

To summarize Case 2:

ϕP_n = 297 kips
ϕM_n = 4,729 ft-kips

<u>Case 3 (Balanced case)</u>

In this case, T_{s1} just reaches its yield stress:

$$c = \left[\frac{0.0025}{(0.0025 + 0.00207)}\right](32.33\text{ ft}) = 17.69\text{ ft}$$

$a = 0.8c = (0.8)(17.69) = 14.15$ ft
$C_{m\ shells} = 0.8f'_m$(2 shells)(1.25 in. / shell)(14.15 ft.) (12 in./ft) = 679.2 kips
$C_{m\ cells} = 0.8f'_m$(2 cells)(41 in.^{2}/cell) = 131.2 kips
$C_{m\ tot} = C_{m\ shells} + C_{m\ cells}$ = 810.4 kips

C_{s1} = (0.20 in.^{2})(60 ksi) = 12.0 kips

T_{s1} = (0.20 in.^{2})(60 ksi) = 12.0 kips

C_{s2} and T_{s2} are neglected because they are small, constituting less than 2 percent of the total P_n.

$\Sigma F_y = 0$:
$P_n = \Sigma C - \Sigma T$
$P_n = C_{m\ tot} + C_{s1} - T_{s1}$ = 810.4 + 12.0 - 12.0 = 810.4 kips
ϕP_n = (0.85)(810.4) = 689 kips

$\Sigma M_{cl} = 0$:
$M_n = 9.26\ C_{m\ shells} + ((16 + 8)/2)\ C_{m\ cells} + 16\ C_{s1} + 8\ T_{s2} + 16\ T_{s1}$
M_n = (9.26)(679.2) + (12.0)(131.2) + (16.00)(12.0) + (ignore small T_{s2}) + (16.0)(12.0) = 8,248 kips
ϕM_n = (0.85)(8,248) = 7,011 ft-kips

To summarize Case 3:

ϕP_n = 689 kips
ϕM_n = 7,011 ft-kips

Using the results from the three cases above, the ϕP_n - ϕM_n curve shown in Figure 9.2-6 is plotted. Although the portion of the ϕP_n - ϕM_n curve above the balanced failure point could be determined, it is not necessary here. Thus, only the portion of the curve below the balance point will be examined. This is the region of high moment capacity.

Similar to reinforced concrete beam-columns, in-plane compression failure of the cantilevered shear wall will occur if $P_u > P_{bal}$, and tension failure will occur if $P_u < P_{bal}$. A ductile failure mode is essential to the design, so the portion of the curve above the "balance point" is not useable.

As can be seen, the points for $P_{u\ min}$, M_u and $P_{u\ max}$, are within the ϕP_n - ϕM_n envelope; thus, the strength design is acceptable with the minimum reinforcement. Figure 9.2-6 shows two schemes for determining the design flexural resistance for a given axial load. One interpolates along the straight line between pure bending and the balanced load. The second makes use of intermediate points for interpolation. This particular example illustrates that there can be a significant difference in the interpolated moment capacity between the two schemes for axial loads midway between the balanced load and pure bending.

For the purpose of shear design, the value of φM_N at the design axial load is necessary. Interpolating between the intermediate point and the P = 0 point for P = 202 kips yields φM_N = 3,480 ft-kip. Thus, the factor on shear to represent development of 125 percent of flexural capacity is:

$$1.25\frac{\phi M_N/\phi}{M_U} = 1.25\frac{3480/0.85}{2640} = 1.94$$

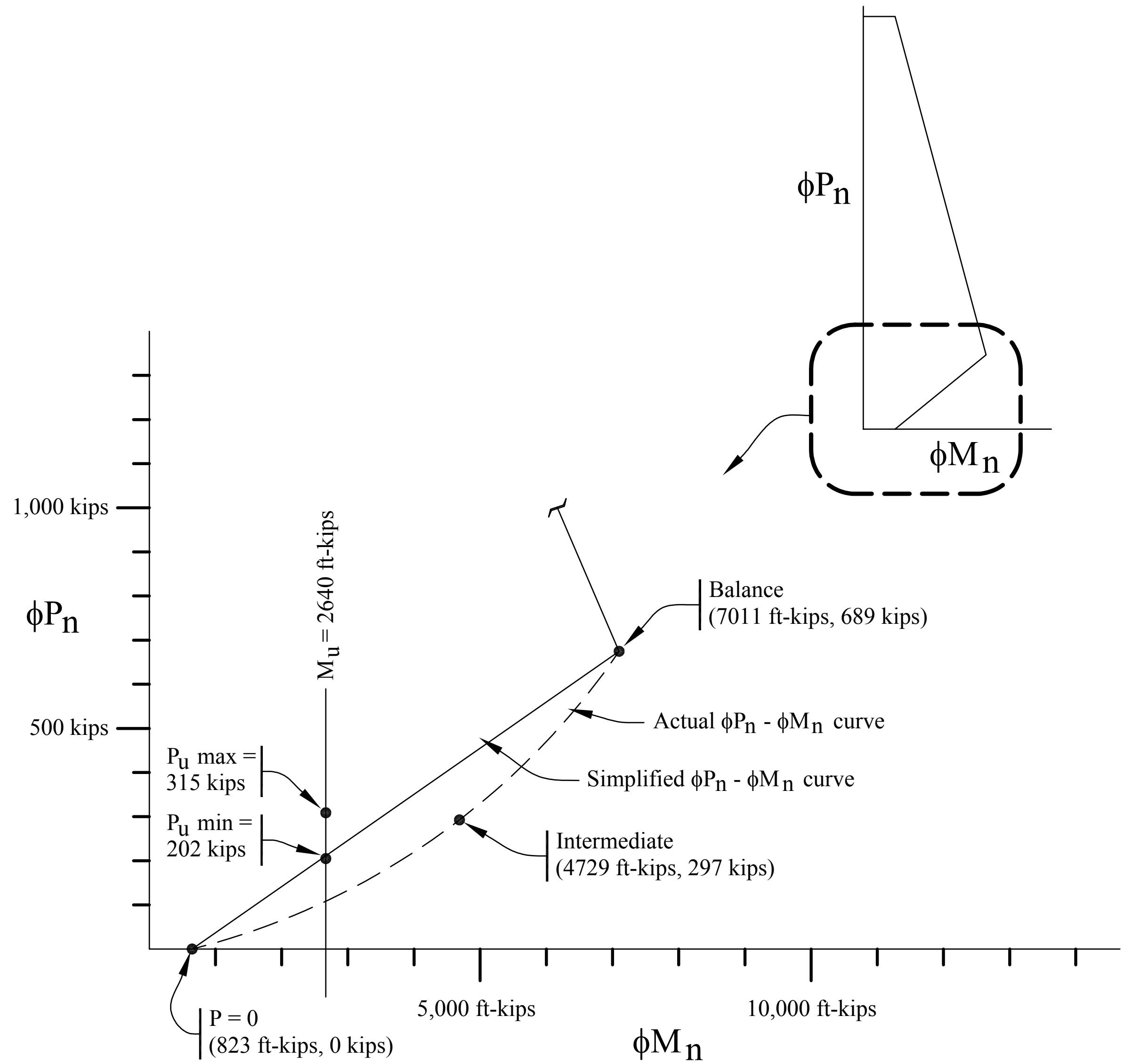


Figure 9.2-6 φP_{11} - φM_{11} diagram for Birmingham 1 Wall D (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.2.4.5.2.2 Ductility check

Provisions Sec.11.6.2.2 [ACI 530, Sec. 3.2.3.5] requires that the critical strain condition correspond to a strain in the extreme tension reinforcement equal to 5 times the strain associated with F_y. Note that this calculation uses unfactored gravity axial loads (*Provisions* Sec.11.6.2.2 [ACI 530, Sec. 3.2.3.5]). Refer to Figure 9.2-5 and the following calculations which illustrate this using loads at the bottom story (highest axial loads). Calculations for other stories are not presented in this example.

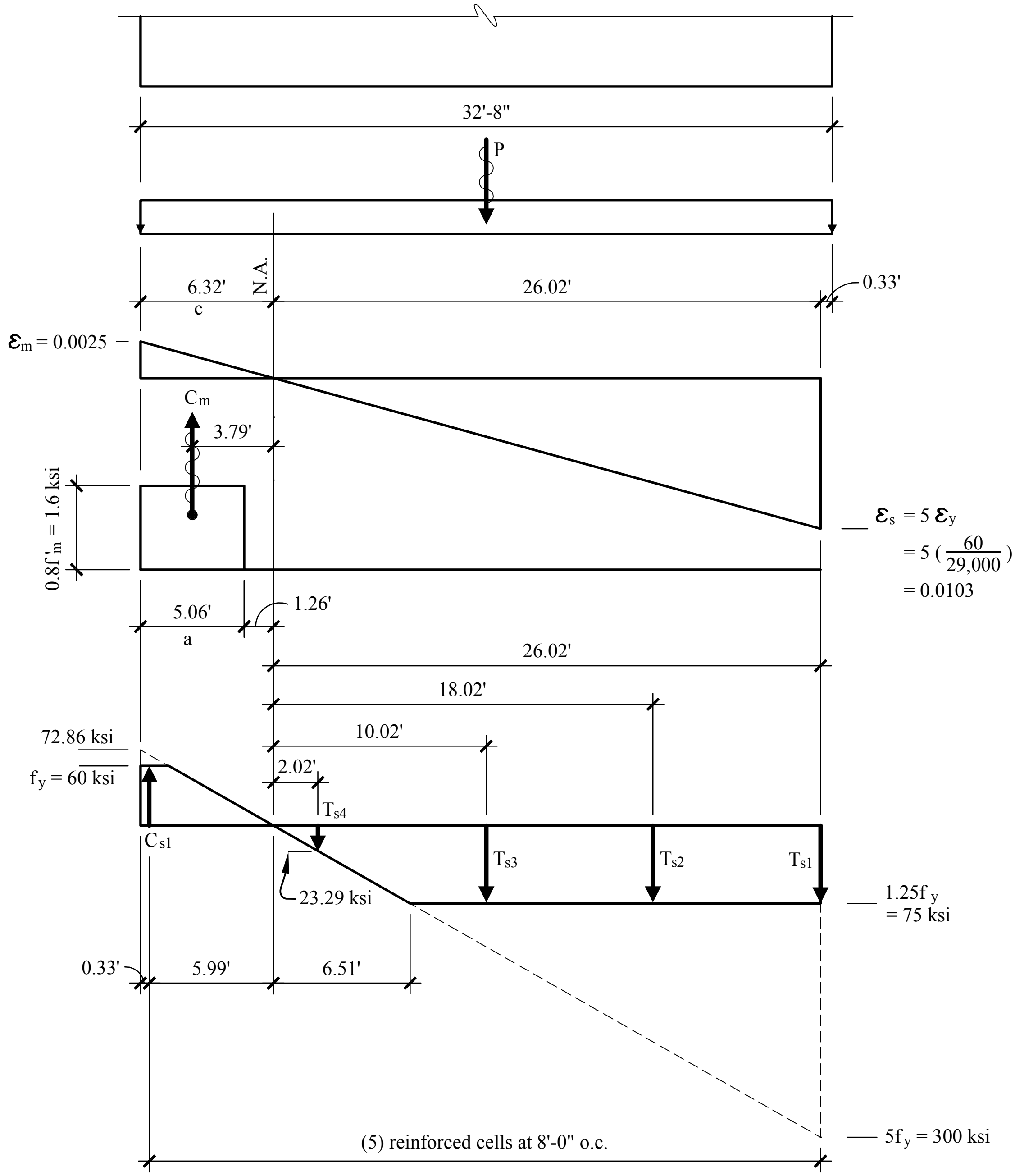


Figure 9.2-7 Ductility check for Birmingham 1 Wall D (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa).

For Level 1 (bottom story), the unfactored axial loads are:

P = 232 kips + weight of half of first story wall = 232 + 6.4 = 238.4 kips

Refer to Figure 9.2-7:

$C_m = 0.8 f'_m(ab + A_{cell})$ = (1.6 ksi)[(5.06 ft. x 12 in./ft.)(2.5 in.) + 41 in.^{2}] = 308.5 kips (same as above)
$C_{s1} = F_yA_s$ = (60 ksi)(0.20 in.^{2}) = 12.0 kips
$T_{s1} = T_{s2} = T_{s3}$ = (1.25 × 60 ksi)(0.20 in.^{2}) = 15 kips

T_{s4} = (23.29 ksi)(0.20 in.^{2}) = 4.6 kips

$\sum C > \sum P + T$
$C_m + C_{s1} > P + T_{s1} + T_{s2} + T_{s3} + T_{s4}$
308.5 + 12.0 > 238.4 + 15 + 15 + 15 + 4.6
320.5 kips > 288 kips OK

There is more compression capacity than required so ductile failure condition governs.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

9.2.4.6 Birmingham 1 Deflections

The calculations for deflection involve many variables and assumptions, and it must be recognized that any calculation of deflection is approximate at best.

Deflections are to be calculated and compared with the prescribed limits set forth by *Provisions* Table 5.2.8. Deformation requirements for masonry structures are given in *Provisions* Sec. 11.5.4 [Table 4.5-1].

The following procedure will be used for calculating deflections:

1. For each story, compare M_x (from Table 9.2-3) to $M_{cr} = S(f_r + P_{u\ min} / A)$ to determine if wall will crack.
2. If $M_{cr} < M_x$, then use cracked moment of inertia and *Provisions* Eq. 11.5.4.3.
3. If $M_{cr} > M_x$, then use $I_n = I_g$ for moment of inertia of wall.
4. Compute deflection for each level.

Other approximations can be used such as the cubic interpolation formula given in *Provisions* 11.5.4.3, but that equation was derived for reinforced concrete members acting as single span beams, not cantilevers. In the authors' opinion, all these approximations pale in comparison to the approximation of nonlinear deformation using C_d.

For the Birmingham 1 building:

b_e = effective masonry wall width
b_e = [(2 × 1.25 in.)(32.67 ft × 12) + (5 cells)(41 in.^{2}/cell)]/(32.67 ft × 12) = 3.02 in.
$S = b_e l^2/6$ = (3.02)(32.67 × 12)^2/6 = 77,434 in.^{3}
f_r = 0.250 ksi
$A = b_e l$ = (3.02 in.)(32.67 ft × 12) = 1,185 in.^{2}

P_u is calculated using 1.00*D* (see Table 9.2-4). 1.00*D* is considered to be a reasonable value for axial load for this admittedly approximate analysis. If greater conservatism is desired, P_u could be calculated using 0.85*D*.

The results are shown in Table 9.2-5.

Table 9.2-5 Birmingham 1 Cracked Wall Determination

Level	$P_{u_{min}}$ (kips)	M_{cr} (ft-kips)	M_u (ft-kips)	Status
5	41	1836	225	uncracked
4	89	2098	652	uncracked
3	137	2359	1230	uncracked
2	185	2621	1910	uncracked
1	232	2877	2640	uncracked

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

For uncracked walls:

$$I_n = I_g = bl^3/12 = (3.02 \text{ in.})(32.67 \times 12)^3 /12 = 1.52 \text{ x } 10^7 \text{ in.}^4$$

The calculation of δ will consider flexural and shear deflections. For the final determination of deflection, a RISA-2D analysis was made. The result is summarized Table 9.2-6 below. Figure 9.2-8 illustrates the deflected shape of the wall.

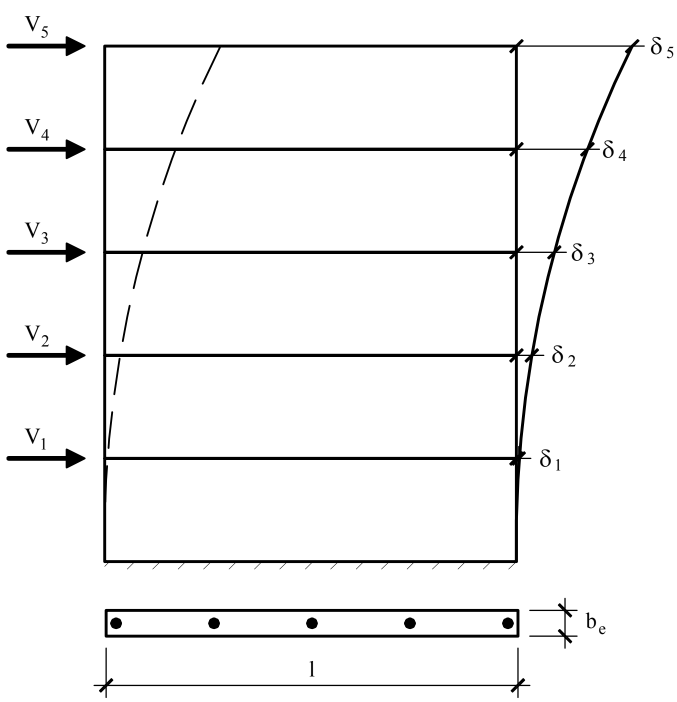


Figure 9.2-8 Shear wall deflections.

Table 9.2-6 Deflections, Birmingham 1

Level	F (kips)	I_{eff} (in.^{4})	$\delta_{flexural}$ (in.)	δ_{shear} (in.)	δ_{total} (in.)	$C_d \delta_{total}$ (in.)	Δ (in.)
54321	26.0	1.52×10^7	0.108	0.054	0.162	0.284	0.061
	23.2	1.52×10^7	0.078	0.049	0.128	0.223	0.066
	17.4	1.52×10^7	0.049	0.041	0.090	0.157	0.066
	11.7	1.52×10^7	0.024	0.028	0.052	0.091	0.054
	5.8	1.52×10^7	0.007	0.015	0.021	0.037	0.037

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

The maximum story drift occurs at Level 4 (*Provisions* Table 5.2.8 [Table 4.5-1]):

[The specific procedures for computing deflection of shear walls have been removed from the 2003 Provisions. ACI 530 does not contain the corresponding provisions in the text, however, the commentary contains a discussion and equations that are similar to the procedures in the 2000 *Provisions*. However, as indicated previously, there is a potential conflict between the drift limits in 2003 *Provisions* Table 4.5-1 and ACI 530 Sec. 1.13.3.2.]

Δ = 0.066 in. < 1.04 in. = $0.01h_n$ OK

9.2.4.7 Birmingham 1 Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4-6.1.3] requires that the bearing walls be designed for out-of-plane loads determined as follows:

$w = 0.40 S_{DS} W_c \geq 0.1 W_c$
$w = (0.40)(0.24)(45 \text{ psf}) = 4.3 \text{ psf} < 4.5 \text{ psf} = 0.1 W_c$

The calculated seismic load, w = 4.5 psf, is much less than wind pressure for exterior walls and is also less than the 5 psf required by IBC Sec. 1607.13 for interior walls. Thus, seismic loads do not govern the design of any of the walls for loading in the out-of-plane direction.

9.2.4.8 Birmingham 1 Orthogonal Effects

Orthogonal effects do not have to be considered for Seismic Design Category B (*Provisions* Sec. 5.2.5.2.1 [Sec. 4.4.2.1]).

This completes the design of Transverse Wall D.

9.2.4.9 Summary of Design for Birmingham 1 Wall D

8 in. CMU
f'_m = 2,000 psi

Reinforcement:

One vertical #4 bar at wall end cells
Vertical #4 bars at 8 ft on center at intermediate cells throughout
Bond beam with two - #4 bars at each story just below the floor and roof slabs

Horizontal joint reinforcement at 16 inches

Grout at cells with reinforcement and at bond beams.

9.2.5 Seismic Design for New York City

This example focuses on differences from the design for the Birmingham 1 site.

9.2.5.1 New York City Weights

As before, use 67 psf for 8-in.-thick normal weight hollow core plank plus the nonmasonry partitions. This site is assigned to Seismic Design Category C, and the walls will be designed as intermediate reinforced masonry shear walls (*Provisions* Sec. 11.11.4 [Sec. 11.2.1.4] and Sec. 11.3.7 [Sec. 11.2.1.4]), which requires prescriptive seismic reinforcement (*Provisions* Sec. 11.3.7.3 [ACI 530, Sec. 1.13.2.2.4]). Intermediate reinforced masonry shear walls have a minimum of #4 bars at 4 ft on center. For this example, 48 psf will be assumed for the 8-in. CMU walls. The 48 psf value includes grouted cells and bond beams in the course just below the floor planks. In Seismic Design Category C, more of the regularity requirement must be checked. It will be shown that this symmetric building with a seemingly well distributed lateral force system is torsionally irregular by the *Provisions*.

Story weight, w_i:

Roof

Roof slab (plus roofing) = (67 psf) (152 ft)(72 ft) = 733 kips
Walls = (48 psf)(589 ft)(8.67 ft/2) + (48 psf)(4)(36 ft)(2 ft) = 136 kips
Total = 869 kips

There is a 2-ft high masonry parapet on four walls and the total length of masonry wall is 589 ft.

Typical floor

Slab (plus partitions) = 733 kips
Walls = (48 psf)(589 ft)(8.67 ft) = 245 kips
Total = 978 kips

Total effective seismic weight, W = 869 + (4)(978) = 4781 kips

This total excludes the lower half of the first story walls, which do not contribute to seismic loads that are imposed on CMU shear walls.

9.2.5.2 New York City Base Shear Calculation

The seismic response coefficient, C_s, is computed from *Provisions* Sec. 5.4.1.1 [Sec. 5.2-1.1]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.39}{2.5/1} = 0.156$$

The value of C_s need not be greater than:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.14}{0.338(2.5/1)} = 0.166$$

where T is the same as found in Sec. 9.2.4.2.

The value for C_s is taken as 0.156 (the lesser of the two computed values). This value is still larger than the minimum specified in *Provisions* Eq. 5.3.2.1-3. Using *Provisions* Eq. 5.4.1.1-3:

$$C_s = 0.044 S_{D1} I = (0.044)(0.14)(1) = 0.00616$$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4.1 [Eq. 5.2-1]:

$$V = C_s W = (0.156)(4{,}781) = 746 \text{ kips}$$

9.2.5.3 New York City Vertical Distribution of Seismic Forces

The vertical distribution of seismic forces is determined in accordance with *Provisions* Sec. 5.4.4 [Sec. 5.2.3], which was described in Sec. 9.2.4.3. Note that for *Provisions* Eq. 5.4.3-2 [Eq. 5.2-11], $k = 1.0$ since $T = 0.338$ sec (similar to the Birmingham 1 building).

The application of the *Provisions* equations for this building is shown in Table 9.2-7:

Table 9.2-7 New York City Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_X (kips)	M_x (ft-kips)
5	869	43.34	37,657	0.3076	229	2.3e+14	1,985
4	978	34.67	33,904	0.2770	207		5,765
3	978	26.00	25,428	0.2077	155		10,889
2	978	17.33	16,949	0.1385	103		16,907
1	978	8.67	8,476	0.0692	52		23,370
$\sum$	4,781		122,414	1.000	746		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m.

9.2.5.4 New York City Horizontal Distribution of Forces

The initial distribution is the same as Birmingham 1. See Sec. 9.2.4.4 and Figure 9.2-3 for wall designations.

Total shear in Wall Type D:

$$V_{tot} = 0.125V + 0.0238V = 0.149V$$

Provisions Sec.5.4.4.3 [Sec. 4.3.2.2] requires a check of torsional irregularity using the ratio of maximum displacement at the end of the structure, including accidental torsion, to the average displacement of the two ends of the building. For this simple and symmetric structure, the actual displacements do not have to be computed to find the ratio. Relying on symmetry and the assumption of rigid diaphragm behavior used to distribute the forces, the ratio of the maximum displacement of Wall D to the average displacement of the floor will be the same as the ratio of the wall shears with and without accidental torsion:

$$\frac{F_{max}}{F_{ave}} = \frac{0.149V}{0.125V} = 1.190$$

This can be extrapolated to the end of the rigid diaphragm therefore:

$$\frac{\delta_{max}}{\delta_{ave}} = 1 + 0.190\left(\frac{152/2}{36}\right) = 1.402$$

Provisions Table 5.2.3.2 [Table 4.3-2] defines a building as having a "Torsional Irregularity" if this ratio exceeds 1.2 and as having an "Extreme Torsional Irregularity" if this ratio exceeds 1.4. Thus, an important result of the Seismic Design Category C classification is that the total torsion must be amplified by the factor:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{ave}}\right)^2 = \left(\frac{1.402}{1.2}\right)^2 = 1.365$$

Therefore, the portion of the base shear for design of Wall D is now:

$$V_D = 0.125V + 1.365(0.0238V) = 0.158V$$

which is a 5.8 percent increase from the fraction before considering torsional irregularity.

The total story shear and overturning moment may now be distributed to Wall D and the wall proportions checked. The wall capacity will be checked before considering deflections.

9.2.5.5 New York City Transverse Wall D

The strength or limit state design concept is used in the *Provisions*.

9.2.5.5.1 New York City Shear Strength

Similar to the design for Birmingham 1, the shear wall design is governed by:

$$V_u \le \phi V_n$$

$$V_n = V_m + V_s$$

$$V_n \text{ max} = 4 \text{ to } 6\sqrt{f'_m}A_n \quad \text{depending on } M/Vd$$

$$V_m = \left[4 - 1.75\left(\frac{M}{Vd}\right)\right]A_n\sqrt{f'_m} + 0.25\,P$$

$$V_s = 0.5\left(\frac{A_v}{s}\right)f_y d_v$$

where

A_n = (2 × 1.25 in. × 32.67 ft × 12 in.) + (41 in.^{2} × 9 cells) = 1,349 in.^{2}

The shear strength of each Wall D, based on the aforementioned formulas and the strength reduction factor of ϕ = 0.8 for shear from *Provisions* Table 11.5.3 [ACI 530, Sec. 3.1.4.3], is summarized in Table 9.2-8. Note that V_x and M_x in this table are values from Table 9.2-7 multiplied by 0.158 (representing the portion of direct and indirect shear assigned to Wall D), and *P* is the dead load of the roof or floor times the tributary area for Wall D.

Table 9.2-8 New York City Shear Strength Calculation for Wall D

Story	V_x (kips)	M_x (ft-kips)	M_x/V_xd	2.5 V_x (kips)	P (kips)	φV_m (kips)	φV_s (kips)	φV_n (kips)	φV_n *max* (kips)
5	36.1	313	0.265	90.3	42	179.0	54.6	233.6	287.6
4	68.7	908	0.405	171.8	90	176.9	54.6	231.5	269.7
3	93.1	1715	0.564	232.8	139	173.2	54.6	227.8	249.2
2	109.3	2663	0.746	273.3	188	167.7	54.6	222.3	225.8
1	117.5	3680	0.959	293.8	236	159.3	54.6	213.9	198.4

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

V_u exceeds φV_n at the lower three stories. As will be shown at the conclusion of the design for flexure, the factor to achieve 125 percent of the nominal flexural capacity is 1.58. This results in V_u being less than φV_n at all stories. If that were not the case, it would be necessary to grout more cells to increase A_n or to increase f'_m.

9.2.5.5.2 New York City Axial and Flexural Strength

The walls in this example are all load-bearing shear walls because they support vertical loads as well as lateral forces. In-plane calculations include:

1. Strength check and
2. Ductility check.

9.2.5.5.2.1 Strength check

Wall demands, using load combinations determined previously, are presented in Table 9.2-9 for Wall D. In the table, Load Combination 1 is 1.28D + Q_E + 0.5L and Load Combination 2 is 0.82D + Q_E.

Table 9.2-9 Demands for New York City Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
5	42	0	54	313	34	313
4	90	8	119	908	74	908
3	139	17	186	1715	114	1715
2	188	25	253	2663	154	2663
1	236	34	319	3680	194	3680

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

As in Sec. 9.2.4.5.2, strength at the bottom story (where P, V, and M are the greatest) will be examined. The strength design will consider Load Combination 2 from Table 9.2-9 to be the governing case because it has the same lateral load as Load Combination 1 but with lower values of axial force. Refer to Fig. 9.2-9 for notation and dimensions.

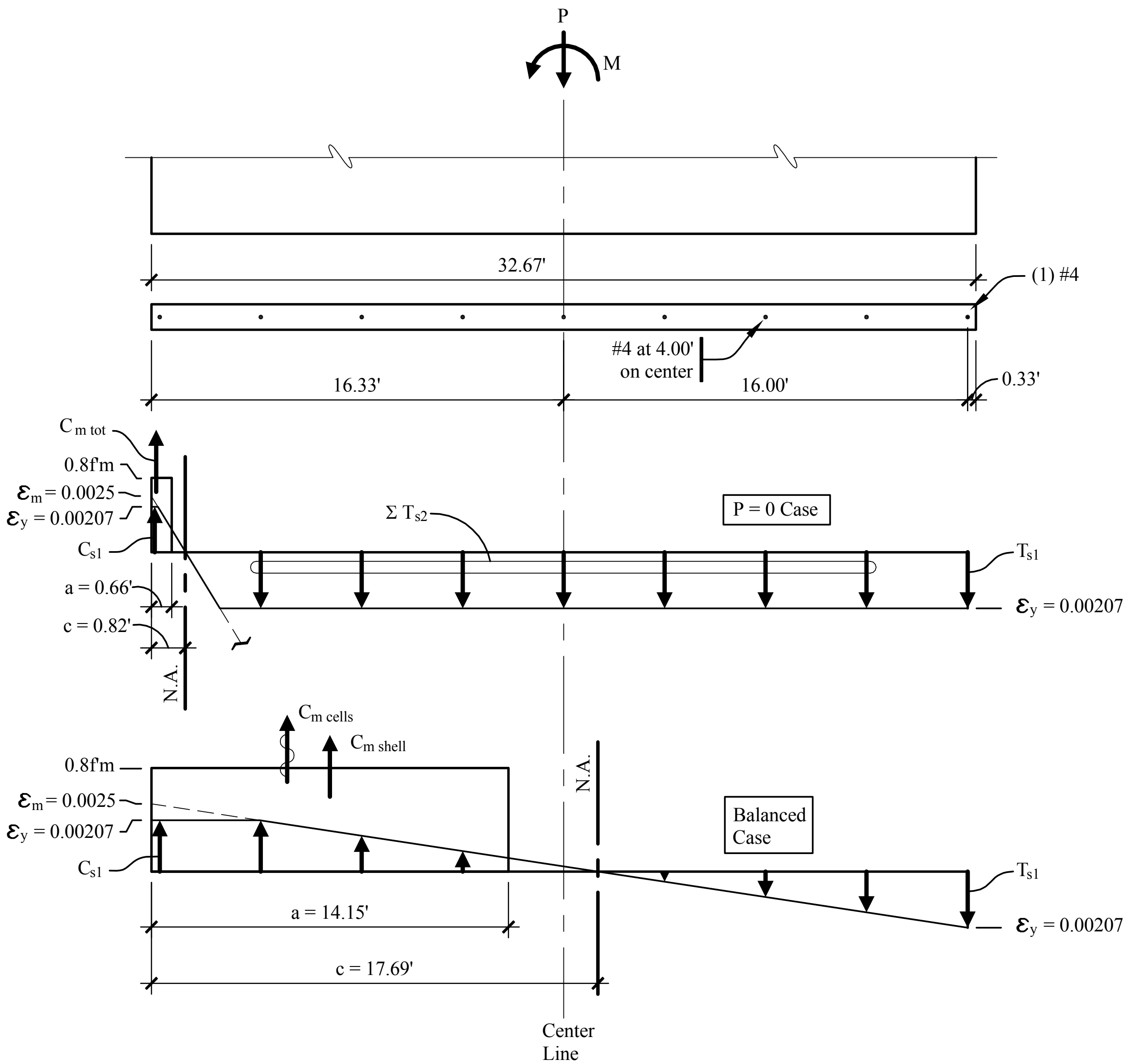


Figure 9.2-9 Strength of New York City and Birmingham 2 Wall D. Strength diagrams are superimposed over the strain diagrams for the two cases (intermediate case is not shown) (1.0 ft = 0.3048 m).

Examine the strength of Wall D at Level 1:

$$P_{u_{min}} = 0.82\,D = 0.82\,(236 + \text{factored weight of lower half of first story wall})$$

$$= 0.82(236 + 6.4) = 199 \text{ kips}$$

$$P_{u_{max}} = 1.28\,D + 0.5\,L\;319 = 1.28(236 + 6.4) + 0.5(34) = 327 \text{ kips}$$

$$M_u = 3{,}680 \text{ ft-kips}$$

Because intermediate reinforced masonry shear walls are used (Seismic Design Category C), vertical reinforcement at is required at 4 ft on center in accordance with *Provisions* Sec. 11.3.7.3 [ACI 530, Sec. 1.13.2.2.4]. Therefore, try one #4 bar in each end cell and #4 bars at 4 ft on center at all intermediate cells.

The calculation procedure is similar to that for the Birmingham 1 building presented in Sec. 9.2.4.5.2. The results of the calculations (not shown) for the New York building are summarized below.

<u>P = 0 case</u>

$\phi P_n = 0$
ϕM_n = 1,475 ft-kips

<u>Intermediate case</u>

c = 8.0 ft
ϕP_n = 330 kips
ϕM_n = 5,600 ft-kips

<u>Balanced case</u>

$\phi P_n = 807$
ϕM_n = 8,214 ft-kips

With the intermediate case, it is simple to use the three points to make two straight lines on the interaction diagram. Use the simplified ϕP_n - ϕM_n curve shown in Figure 9.2-10. The straight line from pure bending to the balanced point is conservative and can easily be used where the design is not as close to the criterion. It is the nature of lightly reinforced and lightly loaded masonry walls that the intermediate point is frequently useful.

Use one #4 bar in each end cell and one #4 bar at 4 ft on center throughout the remainder of the wall.

As shown in the design for Birmingham 1,for the purpose of shear design, the value of φM_N at the design axial load is necessary. Interpolating between the intermediate point and the P = 0 point for P = 199 kips yields φM_N = 3,960 ft-kip. Thus, the factor on shear to represent development of 125 percent of flexural capacity is:

$$1.25\frac{\phi M_N / \phi}{M_U} = 1.25\frac{3960 / 0.85}{3680} = 1.58$$

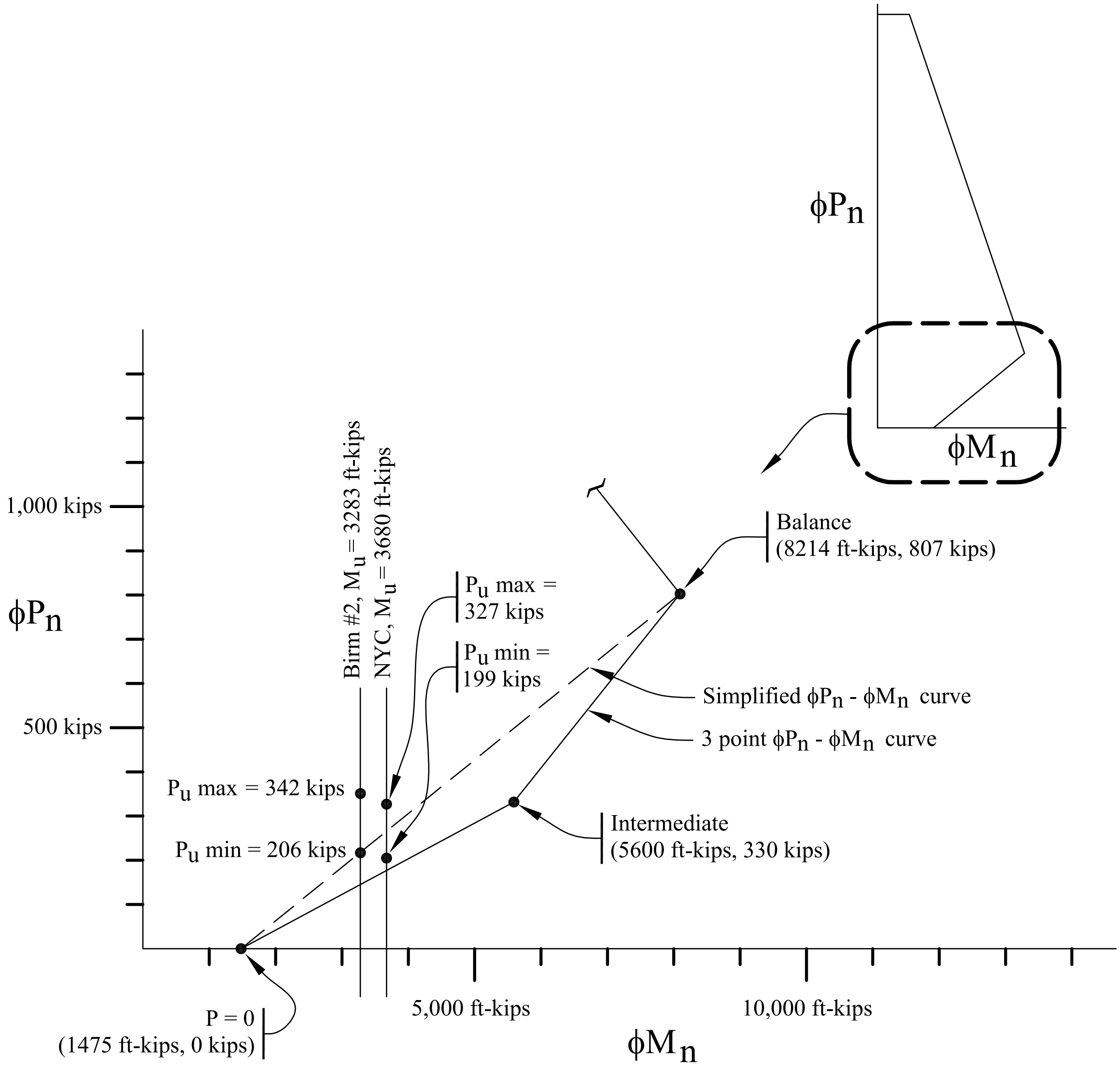


Figure 9.2-10 φP_{11} - φM_{11} Diagram for New York City and Birmingham 2 Wall D (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.2.5.5.2.2 Ductility check

Refer to Sec. 9.2.4.5.2, Item 2, for explanation [see Sec. 9.2.4.5.2 for discussion of revisions to the ductility requirements in the 2003 *Provisions*.]. For Level 1 (bottom story), the unfactored loads are:

P = 236 + weight of lower ½ of first story wall = 236 + 6.4 = 242.4 kips
M = 3,483 ft-kips

$C_m = 0.8 f_m' \ [(a)(b) + A_{cells}]$
where b = face shells = (2 × 1.25 in.) and A_{cell} = 41 in.^{2}

C_m = (1.6 ksi)[(5.03 ft × 12)(2.5 in.) + (2)(41)] = 372.6 kips

$C_{s1} = F_y A_s$ = (60 ksi)(0.20 in.^{2}) = 12 kips
C_{s2} = (22.6 ksi)(0.20 in.^{2}) = 4.5 kips
$T_{s1} = T_{s2} = T_{s3} = T_{s4} = T_{s5}$ = (75 ksi)(0.20 in.^{2}) = 15 kips
T_{s6} = (69.6 ksi)(0.20 in.^{2}) = 13.9 kips
T_{s7}= (23.5 ksi)(0.20 sq. in.) = 4.7 kips

$\sum C > \sum P + T$
$C_m + C_{s1} + C_{s2} > P + T_{s1} + T_{s2} + T_{s3} + T_{s4} + T_{s5} + T_{s6} + T_{s7}$
372.6 + 12.0 + 4.5 > 242.5 + 5(15) + 13.9 + 4.7
389 kips > 336 kips OK

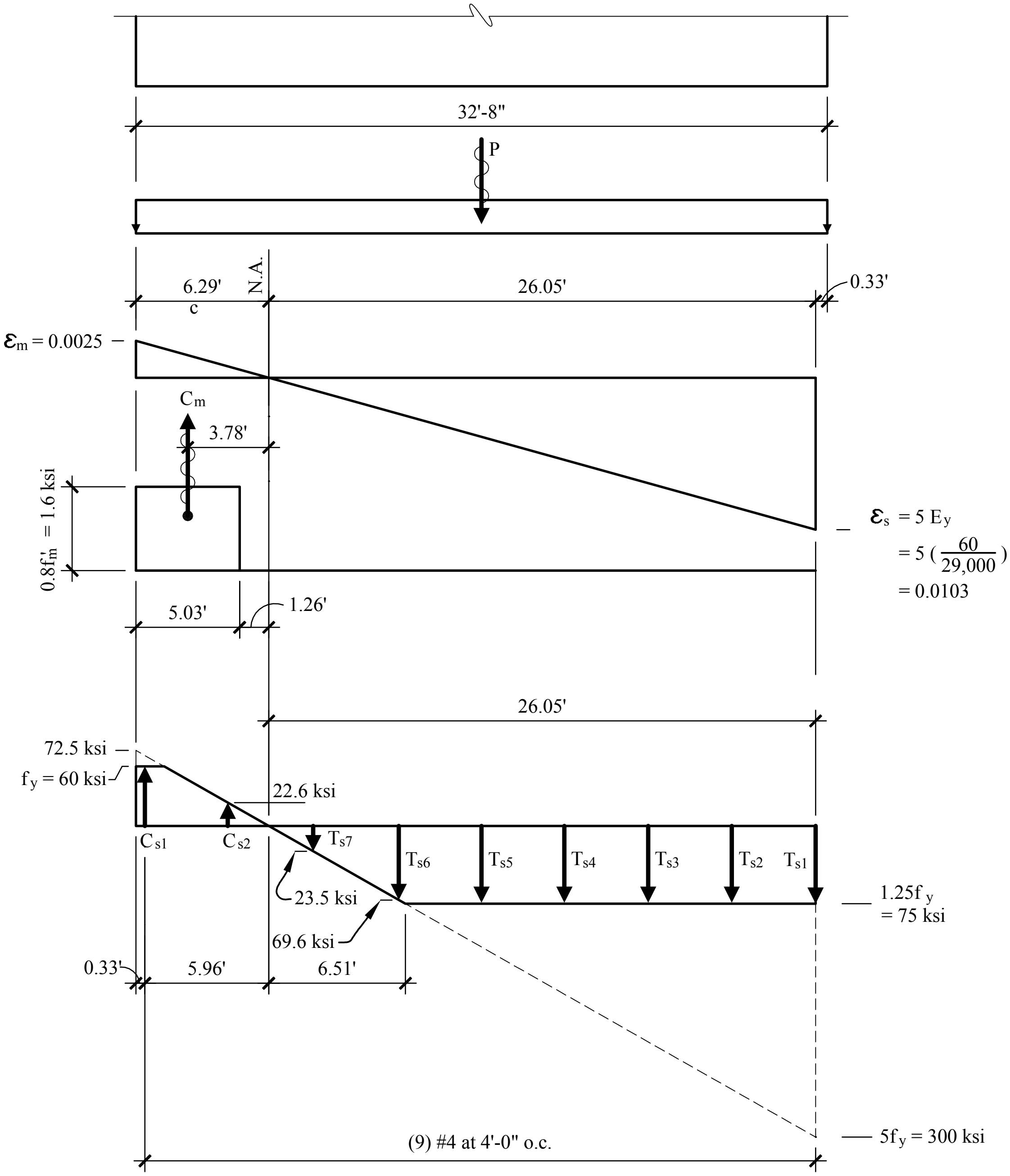


Figure 9.2-11 Ductility check for New York City and Birmingham 2 Wall D (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa).

9.2.5.6 New York Deflections

Refer to 9.2.4.6 for more explanation [see Sec. 9.2.4.6 for discussion of revisions to the deflection computations and requirements in the 2003 *Provisions*, as well as the potentially conflicting drift limits]. For the New York City building, the determination of whether the walls will be cracked is:

b_e = effective masonry wall width

$b_e = [(2 \times 1.25 \text{ in.})(32.67 \text{ ft} \times 12) + (9 \text{ cells})(41 \text{ in.}^2/\text{cell})]/(32.67 \text{ ft} \times 12) = 3.44 \text{ in.}$

$A = b_e\, l = (3.44 \text{ in.})(32.67 \times 12) = 1{,}349 \text{ in.}^2$

$S = b_e\, l^2/6 = (3.44)(32.67 \times 12)^2/6 = 88{,}100$ in.^{3}
$f_r = 0.250$ ksi

P_u is calculated using 1.00D (see Table 9.2-8 for values and refer to Sec. 9.2.4.6 for discussion). Table 9.2-10 a summarizes of these calculations.

Table 9.2-10 New York City Cracked Wall Determination

Level	P_u (kips)	M_{cr} (ft-kips)	M_x (ft-kips)	Status
5	42	2064	313	uncracked
4	90	2325	908	uncracked
3	139	2592	1715	uncracked
2	188	2860	2663	uncracked
1	236	3120	3680	cracked

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

For the uncracked walls:

$I_n = I_g = bl^3/12 = (3.44\text{ in.})(32.67 \times 12)^3/12 = 1.73 \times 10^7$ in.^{4}

For the cracked wall, observe that the intermediate point on the interaction diagram is relatively close to the design point. Therefore, as a different type of approximation, compute a cracked moment of inertia using the depth to the neutral axis of 8.0 ft:

$I_{cr} = b_e c^3/3 + \sum n A_s d^2$

$I_{cr} = (3.44\text{ in.})(8.0\text{ ft} \times 12)^3/3 + 19.3(0.2)(4.33^2 + 8.33^2 + 12.33^2 + 16.33^2 + 20.33^2 + 24.33^2)144 =$
$= 1.01 \times 10^6 + 0.84 \times 10^6 = 1.85 \times 10^6$ in.^{4}

Per *Provisions* Eq. 11.5.4.3:

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n$$

$I_{eff} = 1.13 \times 10^7$ in.^{4}

Provisions 11.5.4.3 would imply that I_{eff} would be used for the full height. Another reasonable option is to use I_{cr} at the first story and I_g above that. The calculation of δ should consider shear deflections in addition to the flexural deflections. For this example I_{eff} will be used over the full height for the final determination of deflection (a RISA 2D analysis was made). The result is summarized in Table 9.2-11.

Table 9.2-11 New York City Deflections

Level	F (kips)	I_{eff} (in.^{4})	$\delta_{flexural}$ (in.)	δ_{shear} (in.)	δ_{total} (in.)	$C_d\,\delta_{total}$ (in.)	Δ (in.)
5	34.1	1.13×10^7	0.256	0.080	0.336	0.757	0.163
4	30.9	1.13×10^7	0.189	0.075	0.264	0.593	0.171
3	23.1	1.13×10^7	0.124	0.064	0.188	0.422	0.163
2	15.3	1.13×10^7	0.065	0.050	0.115	0.259	0.141
1	7.7	1.13×10^6	0.020	0.033	0.053	0.118	0.118

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

The maximum story drift occurs at Level 4:

Δ_4 = 0.171 in. < 1.04 in. = 0.01 h_4 (*Provisions* Table 5.2.8 [Table 4.5-1]) OK

The total displacement at the top of the wall is

Δ = 0.757 in. < 5.2 in. = 0.01 h_n (*Provisions* 11.5.4.1.1) OK

9.2.5.7 New York City Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4.4.2.2] requires that the bearing walls be designed for out-of-plane loads determined as

$$w = 0.40\, S_{DS}\, W_c \geq 0.1 W_c$$

With $S_{DS} = 0.39$, $w = 0.156W_c > 0.1W_c$, so w = (0.156)(48 psf) = 7.5 psf, which is much less than wind pressure for exterior walls. Even though Wall D is not an exterior wall, the lateral pressure is sufficiently low that it is considered acceptable by inspection, without further calculation. Seismic loads do not govern the design of Wall D for loading in the out-of-plane direction.

9.2.5.8 New York City Orthogonal Effects

According to *Provisions* Sec. 5.2.5.2.2, orthogonal interaction effects have to be considered for Seismic Design Category C when the equivalent lateral force (ELF) procedure is used (as it is here). However, the out-of-plane component of only 30 percent of 7.5 psf on the wall will not produce a significant effect when combined with the in-plane direction of loads, so no further calculation will be made.

This completes the design of the transverse Wall D for the New York building.

9.2.5.9 Summary of New York City Wall D Design

8 in. CMU
f'_m = 2,000 psi

Reinforcement:

Vertical #4 bars at 4 ft on center throughout the wall
Bond beam with two #4 at each story just below the floor or roof slabs
Horizontal joint reinforcement at alternate courses

9.2.6 Birmingham 2 Seismic Design

The emphasis here is on differences from the previous two locations for the same building. Per *Provisions* Table 5.2.5.1 [Table 4.4-1], the torsional irregularity requires that the design of a Seismic Design Category D building be based on a dynamic analysis. Although not explicitly stated, the implication is that the analytical model should be three-dimensional in order to capture the torsional response. This example will compare both the equivalent lateral force procedure and the modal response spectrum analysis procedure and will demonstrate that, as long as the torsional effects are accounted for, the static analysis could be considered adequate for design.

9.2.6.1 Birmingham 2 Weights

The floor weight for this examples will use the same 67 psf for 8-in.-thick, normal weight hollow core plank plus roofing and the nonmasonry partitions as used in the prior examples (see Sec. 9.2.1). This site is assigned to Seismic Design Category D, and the walls will be designed as special reinforced masonry shear walls (*Provisions* Sec. 11.11.5 and Sec. 11.3.8[ACI 530, Sec. 1.13.2.2.5), which requires prescriptive seismic reinforcement (*Provisions* Sec. 11.3.7.3). Special reinforced masonry shear walls have a maximum spacing of rebar at 4 ft on center both horizontally and vertically. Also, the total area of horizontal and vertical reinforcement must exceed 0.0020 times the gross area of the wall, and neither direction may have a ratio of less than 0.0007. The vertical #4 bars at 48 in. used for the New York City design yields a ratio of 0.00055, so it must be increased. Two viable options are #5 bars at 48 in. (yielding 0.00085) and #4 bars at alternating spaces of 32 in. and 40 in. (12 bars in the wall), which yields 0.0080. The latter is chosen in order to avoid unnecessarily increasing the shear demand. Therefore, the horizontal reinforcement must be (0.0020 - 0.0008)(7.625 in.)(12 in./ft.) = 0.11 in.^{2}/ft. or 0.95 in.^{2} per story. Two #5 bars in bond beams at 48 in. on center will be adequate. For this example, 56 psf weight for the 8-in.-thick CMU walls will be assumed. The 56 psf value includes grouted cells and bond beams.

Story weight, w_i:

Roof:

Roof slab (plus roofing) = (67 psf) (152 ft)(72 ft)	= 733 kips
Walls = (56 psf)(589 ft)(8.67 ft/2) + (56 psf)(4)(36 ft)(2 ft)	= 159 kips
Total	= 892 kips

There is a 2-ft-high masonry parapet on four walls and the total length of masonry wall is 589 ft.

Typical floor:

Slab (plus partitions)	= 733 kips
Walls = (56 psf)(589 ft)(8.67 ft)	= 286 kips
Total	= 1,019 kips

Total effective seismic weight, W = 892 + (4)(1,019) = 4,968 kips

This total excludes the lower half of the first story walls which do not contribute to seismic loads that are imposed on CMU shear walls.

9.2.6.2 Birmingham 2 Base Shear Calculation

The ELF analysis proceeds as described for the other locations. The seismic response coefficient, C_s, is computed using *Provisions* Eq. 5.4.1.1-1 [Eq. 5.2-2] and 5.4.1.1-2 [Eq.5.2-3]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.47}{3.5/1} = 0.134 \qquad \text{(Controls)}$$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.28}{0.338(3.5/1)} = 0.237$$

This is somewhat less than the 746 kips computed for the New York City design due to the larger *R* factor.

The fundamental period of the building, based on *Provisions* Eq. 5.4.2.1-1 [Eq.5.2-6], is 0.338 sec as computed previously (the approximate period, based on building system and building height, will be the same for all locations). The value for C_s is taken as 0.134 (the lesser of the two values). This value is still larger than the minimum specified in *Provisions* Eq. 5.3.2.1-3 which is:

$$C_s = 0.044 S_{D1} I = (0.044)(0.28)(1) = 0.012$$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4.1 [Eq.5.2-1] as:

$$V = C_s W = (0.134)(4{,}968) = 666 \text{ kips}$$

A three-dimensional (3D) model was created in SAP 2000 for the modal response spectrum analysis. The masonry walls were modeled as shell bending elements and the floors were modeled as an assembly of beams and shell membrane elements. The beams have very little mass and a large flexural moment of inertia to avoid consideration of models of vertical vibration of the floors. The flexural stiffness of the beams was released at the bearing walls in order to avoid a wall slab frame that would inadvertently increase the torsional resistance. The mass of the floors was captured by the shell membrane elements. Table 9.2-12 shows data on the modes of vibration used in the analysis.

Provisions Sec. 4.1.2.6 [Sec. 3.3.4] was used to create the response spectrum for the modal analysis. The key points that define the spectrum are:

$T_S = S_{D1}/S_{DS} = 0.28/0.47 = 0.60$ sec
$T_0 = 0.2\ T_S = 0.12$ sec
at $T = 0$, $S_a = 0.4\ S_{DS}/R = 0.0537\ g$
from $T = T_0$ to T_S, $S_a = S_{DS}/R = 0.1343\ g$
for $T > T_S$, $S_a = S_D 1/(RT) = 0.080/T$

The computed fundamental period is less than the approximate period. The transverse direction base shear from the SRSS combination of the modes is 457.6 kips, which is considerably less than that obtained using the ELF method.

Provisions Sec. 5.5.7 [Sec. 5.3.7] requires that the modal base shear be compared with the ELF base shear computed using a period somewhat larger than the approximate fundamental period (C_uT_a). Per Sec. 9.2.4.2, T_a = 0.338 sec. and per *Provisions* Table 5.4.2 [Table 5.2-1] C_u = 1.4. Thus, C_uT_a = 0.48 sec., which is less that S_{D1}/S_{DS}. Therefore, the ELF base shear for comparison is 666 kips as just computed. Because 85 percent of 666 kips = 566 kips, *Provisions* Sec. 5.5.7 [Sec. 5.3.7] dictates that all the results of the modal analysis be factored by:

$$\frac{0.85V_{ELF}}{V_{Modal}} = \frac{566}{458} = 1.24$$

Both analyses will be carried forward as discussed in the subsequent sections.

Table 9.2-12 Birmington 2 Periods, Mass Participation Factors, and Modal Base Shears in the Transverse Direction for Modes Used in Analysis

Mode	Period,	Individual mode (percent)			Cumulative sum (percent)			Trans.
number	(seconds)	Long.	Trans.	Vert.	Long.	Trans.	Vert.	base shear
1	0.2467	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	0.1919	0.00	70.18	0.00	0.00	70.18	0.00	451.1
3	0.1915	70.55	0.00	0.00	70.55	70.18	0.00	0.0
4	0.0579	0.00	18.20	0.00	70.55	88.39	0.00	73.9
5	0.0574	17.86	0.00	0.00	88.41	88.39	0.00	0.0
6	0.0535	0.00	4.09	0.00	88.41	92.48	0.00	16.1
7	0.0532	4.17	0.00	0.00	92.58	92.48	0.00	0.0
8	0.0413	0.00	0.01	0.00	92.58	92.48	0.00	0.0
9	0.0332	1.50	0.24	0.00	94.08	92.72	0.00	0.8
10	0.0329	0.30	2.07	0.00	94.38	94.79	0.00	7.1
11	0.0310	1.28	0.22	0.00	95.66	95.01	0.00	0.8
12	0.0295	0.22	1.13	0.00	95.89	96.14	0.00	3.8
13	0.0253	1.97	0.53	0.00	97.86	96.67	0.00	1.7
14	0.0244	0.53	1.85	0.00	98.39	98.52	0.00	5.9
15	0.0190	1.05	0.36	0.00	99.44	98.89	0.00	1.1
16	0.0179	0.33	0.94	0.00	99.77	99.82	0.00	2.8
17	0.0128	0.19	0.07	0.00	99.95	99.90	0.00	0.2
18	0.0105	0.03	0.10	0.00	99.99	99.99	0.00	0.3

1 kip = 4.45 kN.

9.2.6.3 Birmingham 2 Vertical Distribution of Seismic Forces

The dynamic analysis will be revisited for the horizontal distribution of forces in the next section but as demonstrated there, the ELF procedure is considered adequate to account for the torsional behavior in this example. The dynamic analysis can certainly be used to deduce the vertical distribution of forces. This analysis was constructed to study amplification of accidental torsion. It would be necessary to integrate the shell forces to find specific story forces, and it is not necessary to complete the design. Therefore, the vertical distribution of seismic forces for the ELF analysis is determined in accordance with *Provisions* Sec. 5.4.4 [Sec. 5.2.3], which was described in Sec. 9.2.4.3. For *Provisions* Eq. 5.4.3-2 [Sec. 5.2-11], k = 1.0 since T = 0.338 sec (similar to the Birmingham 1 and New York City buildings). It should be noted that the response spectrum analysis may result in moments that are less than those calculated using the ELF method; however, because of its relative simplicity, the ELF is used in this example.

Application of the *Provisions* equations for this building is shown in Table 9.2-13:

Table 9.2-13 Birmingham 2 Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	w_xh_x (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	892	43.34	38,659	0.3045	203	203	1,760
4	1,019	34.67	35,329	0.2782	185	388	5,124
3	1,019	26.00	26,494	0.2086	139	527	9,693
2	1,019	17.33	17,659	0.1391	93	620	15,068
1	1,019	8.67	8,835	0.0695	46	666	20,843
∑	4,968		126,976	1.000	666		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m.

9.2.6.4 Birmingham 2 Horizontal Distribution of Forces

For the ELF analysis, this is the same as that for New York City location; see Sec. 9.2.5.4.

Total shear in wall type D:

$$V_{tot} = 0.125V + 1.365(0.0238)V = 0.158V = 104.9\text{kips}$$

The dynamic analysis shows that the fundamental mode is a pure torsional mode. The fact that the fundamental mode is torsional does confirm, to an extent, that the structure is torsionally sensitive. This modal analysis does not show any significant effect of the torsion, however. The pure symmetry of this structure is somewhat idealistic. Real structures usually have some real eccentricity between mass and stiffness, and dynamic analysis then yields coupled modes, which contribute to computed forces.

The *Provisions* does not require that the accidental eccentricity be analyzed dynamically. For illustration, however, this was done by adjusting the mass of the floor elements to generate an eccentricity of 5 percent of the 152-ft length of the building. Table 9.2-14 shows the results of such an analysis. (Accidental torsion could also be considered using a linear combination of the dynamic results and a statically applied moment equal to the accidental torsional moment.)

The transverse direction base shear from the SRSS combination of the modes is 403.8 kips, significantly less than the 457.6 kips for the symmetric model. The amplification factor for this base shear is 566/404 = 1.4. This smaller base shear from modal analysis of a model with an artificially introduced eccentricity is normal. For two primary reasons. First, the mass participates in more modes. The participation in the largest mode is generally less, and the combined result is dominated by the largest single mode. Second, the period for the fundamental mode generally increases, which will reduce the spectral response except for structures with short periods (such as this one).

The base shear in Wall D was computed by adding the in-plane reactions. For the symmetric model the result was 57 kips, which is 12.5 percent of the total of 458 kips, as would be expected. Amplifying this by the 1.24 factor yields 71 kips The application of a static horizontal torsion equal to the 5 percent eccentricity times a base shear of 566 kips (the "floor") adds 13 kips, for a total of 84 kips. If the static horizontal torsion is amplified by 1.365, as found in the analysis for the New York location, the total becomes 89 kips, which is less than the 99 kips and 105 kips computed in the ELF analysis without and with, respectively, the amplification of accidental torsion. The Wall D base shear from the eccentric model was 66 kips; with the amplification of base shear = 1.4, this becomes 92 kips. Note that this value is less than the direct shear from the symmetric model plus the amplified static torsion. The obvious conclusion is that more careful consideration of torsional instability than actually required by the

Provisions does not indicate any more penalty than already given by the procedures for the ELF in the *Provisions*. Therefore the remainder of the example designs for this building are completed using the ELF.

Table 9.2-14 Birmingham periods, Mass Participation Factors, and Modal Base Shears in the Transverse Direction for Modes Used in Analysis

Mode Number	Period (sec)	Individual mode (percent) Long.	Trans.	Vert.	Cumulative sum (percent) Long.	Trans.	Vert.	Trans. Base Shear
1	0.2507	0.0	8.8	0.0	0.0	8.8	0.0	56.3
2	0.1915	70.5	0.0	0.1	70.5	8.8	0.1	0.0
3	0.1867	0.0	61.4	0.0	70.5	70.2	0.1	394.9
4	0.0698	0.0	2.9	0.0	70.5	73.1	0.1	12.7
5	0.0613	1.1	0.0	23.0	71.6	73.1	23.1	0.0
6	0.0575	19.2	0.0	0.0	90.9	73.1	23.2	0.0
7	0.0570	0.0	13.7	0.0	90.9	86.8	23.2	55.5
8	0.0533	0.0	5.6	0.0	90.9	92.4	23.2	22.0
9	0.0480	1.2	0.0	12.8	92.0	92.4	35.9	0.0
10	0.0380	1.4	0.0	0.0	93.5	92.4	35.9	0.0
11	0.0374	0.0	0.4	0.0	93.5	92.8	35.9	1.3
12	0.0327	1.7	0.0	0.2	95.2	92.8	36.1	0.0
13	0.0322	0.0	3.1	0.0	95.2	95.9	36.1	10.4
14	0.0263	2.8	0.0	0.1	98.0	95.9	36.2	0.0
15	0.0243	0.0	3.0	0.0	98.0	98.8	36.2	9.5
16	0.0201	1.6	0.0	0.1	99.6	98.8	36.3	0.0
17	0.0164	0.0	1.1	0.0	99.6	100.0	36.3	3.4
18	0.0141	0.4	0.0	0.1	100.0	100.0	36.3	0

The total story shear and overturning moment (from the ELF analysis) may now be distributed to Wall D and the wall proportions checked. The wall capacity will be checked before considering deflections.

The "extreme torsional irregularity" has an additional consequence for Seismic Design Category D: *Provisions* 5.6.2.4.2 [Sec. 4.6.3.2] requires that the design forces for connections between diaphragms, collectors, and vertical elements (walls) be increased by 25 percent above the diaphragm forces given in *Provisions* 5.4.1 [Sec. 4.6.3.4]. For this example, the diaphragm of precast elements is designed using the different requirements of the appendix to *Provisions* Chapter 9 (see Chapter 7 of this volume).

9.2.6.5 Birmingham 2 Transverse Wall (Wall D)

The design demands are slightly smaller than for the New York City design, yet there is more reinforcement, both vertical and horizontal in the walls. This illustration will focus on those items where the additional reinforcement has special significance.

9.2.6.5.1 Birmingham 2 Shear Strength

Refer to Sec. 9.2.5.5.1 for most quantities. The additional horizontal reinforcement raises V_s and the additional grouted cells raises A_n and, therefore both V_m and V_n *max*.

A_v/s = (4)(0.31 in.^{2})/(8.67 ft.) = 0.1431 in.^{2}/ft
V_s = 0.5(0.1431)(60 ksi)(32.67 ft) = 140.2 kips
A_n = (2 × 1.25 in. × 32.67 ft x 12 in.) + (41 in.^{2} × 12 cells) = 1,472 in.^{2}

The shear strength of Wall D is summarized in Table 9.2-15 below. (Note that V_x and M_x in this table are values from Table 9.2-13 multiplied by 0.158, the portion of direct and torsional shear assigned to the wall). Clearly, the dynamic analysis would make it possible to design this wall for smaller forces, but the minimum configuration suffices. The 1.96 multiplier on V_x to determine V_u is explained in the subsequent section on flexural design.

Table 9.2-15 Shear Strength Calculations for Wall D, Birmingham 2

Level (x)	V_x (kips)	M_x (ft-kips)	M_x/V_xd	$1.98V_x$ (kips)	P (kips)	ϕV_m (kips)	ϕV_s (kips)	ϕV_n (kips)	ϕV_n *max* (kips)
5	32.0	277	0.265	63.4	42	194.6	112.2	306.8	313.9
4	61.1	907	0.454	121	90	186.8	112.2	299	287.3
3	83.0	1527	0.563	164.3	139	186.6	112.2	298.8	272.0
2	97.7	2373	0.743	193.4	188	179.7	112.2	291.9	246.7
1	104.9	3283	0.958	207.7	236	169.6	112.2	281.8	216.6

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

Note that V_n *max* is less than V_n at all levels except the top story. The capacity is greater than the demand at all stories, therefore, the design is satisfactory for shear.

9.2.6.5.2 Birmingham 2 Axial and Flexural Strength

Once again, the similarities to the design for the New York City location will be exploited. Normally, the in-plane calculations include:

1. Strength check
2. Ductility check

9.2.6.5.2.1 Strength check

The wall demands, using the load combinations determined previously, are presented in Table 9.2-16 for Wall D. In the table, Load Combination 1 is $1.29D + Q_E + 0.5L$ and Load Combination 2 is $0.81D + Q_E$.

Table 9.2-16 Birmingham 2 Demands for Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
5	43	0	55	277	36	277
4	94	8	125	807	76	807
3	145	17	196	1527	117	1527
2	196	25	265	2373	159	2373
1	247	34	336	3283	200	3283

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

Strength at the bottom story (where P, V, and M are the greatest) are less than required for the New York City design. The demands are plotted on Figure 9.2-10, showing that the design for New York City has sufficient axial and flexural capacity for this Birmingham 2 location. For this design, the interaction capacity line will be shifted to the right, due to the presence of additional reinforcing bars. The only calculation here will be an estimate of the factor to develop the flexural capacity at the design axial load.

The flexural capacity for lightly load walls is approximately proportional to the sum of axial load plus the yield of the reinforcing steel:

$$\frac{\text{Birmingham \#2 capacity}}{\text{NewYorkCapacity}} = \frac{200 \text{ kips } + \ 12 \times 0.20 \text{ in.}^2 \times 60 \text{ ksi}}{199 \text{ kips } + \ 9 \times 0.20 \text{ in.}^2 \times 60 \text{ ksi}} = \frac{344}{307} = 1.12$$

Therefore the factor by which the walls shears must be multiplied to represent 125 percent of flexural capacity, given that the factor was 1.58 for the New York design is:

$$1.58 \times 1.12 \frac{\text{New York base shear}}{\text{Birmingham \#2 base shear}} = 1.77 \times \frac{746}{666} = 1.98$$

9.2.6.5.2.2 Ductility check

The *Provisions* requirements for ductility are described in Sec. 9.2.4.5.2 and 9.2.5.5.2. Since the wall reinforcement and loads are so similar to those for the New York City building, the computations are not repeated here.

[Refer to Sec. 9.2.4.5.2 for discussion of revisions to the ductility requirements in the 2003 *Provisions*.]

9.2.6.6 Birmingham 2 Deflections

The calculations for deflection would be very similar to that for the New York City location. Ironically, that procedure will indicate that the wall is not cracked at the design load. The C_d factor is larger, 3.5 vs. 2.25. However, the calculation is not repeated here; refer to Sec. 9.2.4.6 and Sec. 9.2.5.6.

[Refer to Sec. 9.2.4.6 for discussion of revisions to the deflection computations and requirements in the 2003 *Provisions*, as well as the potentially conflicting drift limits.]

9.2.6.7 Birmingham 2 Out-of-Plane Forces

Provisions Sec. 5.2.6.2.7 [Sec. 4.6.1.3] requires that the bearing walls be designed for out-of-plane loads determined:

$w = 0.40\, S_{DS}\, W_c \geq 0.1 W_c$
$w = (0.40)(0.47)(56 \text{ psf}) = 10.5 \text{ psf} \geq 0.1 W_c$

The calculated seismic load, w = 10.5 psf, is less than wind pressure for exterior walls. Even though Wall D is not an exterior wall, the lateral pressure is sufficiently low that it is considered acceptable by inspection without further calculation. Seismic loads do not govern the design of Wall D for loading in the out-of-plane direction.

9.2.6.8 Birmingham 2 Orthogonal Effects

According to *Provisions* Sec. 5.2.5.2.2 [Sec. 4.4.2.3], orthogonal interaction effects have to be considered for Seismic Design Category D when the ELF procedure is used (as it is here). However, the out-of-plane component of only 30 percent of 10.5 psf on the wall will not produce a significant effect when combined with the in-plane direction of loads so no further calculation will be made.

This completes the design of the Transverse Wall D.

9.2.6.9 Birmingham 2 Summary of Wall Design for Wall D

8-in. CMU
f'_m = 2,000 psi

Reinforcement:

12 vertical #4 bars per wall (spaces alternate at 32 and 40 in. on center)
Two bond beams with 2 - #5 at each story, at bearing for the planks, and at 4 ft above each floor.
Horizontal joint reinforcement at alternate courses is recommended, but not required.

9.2.7 Seismic Design for Los Angeles

Once again, the differences from the designs for the other locations will be emphasized. As explained for the Birmingham 2 building, the *Provisions* would require a dynamic analysis for design of this building. For the reasons explained in Sec. 9.2.6.4, this design is illustrated using the ELF procedure.

9.2.7.1 Los Angeles Weights

Use 91 psf for 8-in.-thick, normal weight hollow core plank, 2.5 in. lightweight concrete topping (115 pcf), plus the nonmasonry partitions. This building is Seismic Design Category D, and the walls will be designed as special reinforced masonry shear walls (*Provisions* Sec. 11.11.5 and Sec. 11.3.8 [Sec.11.2.1.5]), which requires prescriptive seismic reinforcement (*Provisions* Sec. 11.3.8.3 [ACI 530, Sec. 1.13.2.2.5]). Special reinforced masonry shear walls have a minimum spacing of vertical reinforcement of 4 ft on center. For this example, 60 psf weight for the 8-in. CMU walls will be assumed. The 60 psf value includes grouted cells and bond beams in the course just below the floor planks and in the course 4 ft above the floors. A typical wall section is shown in Figure 9.2-12.

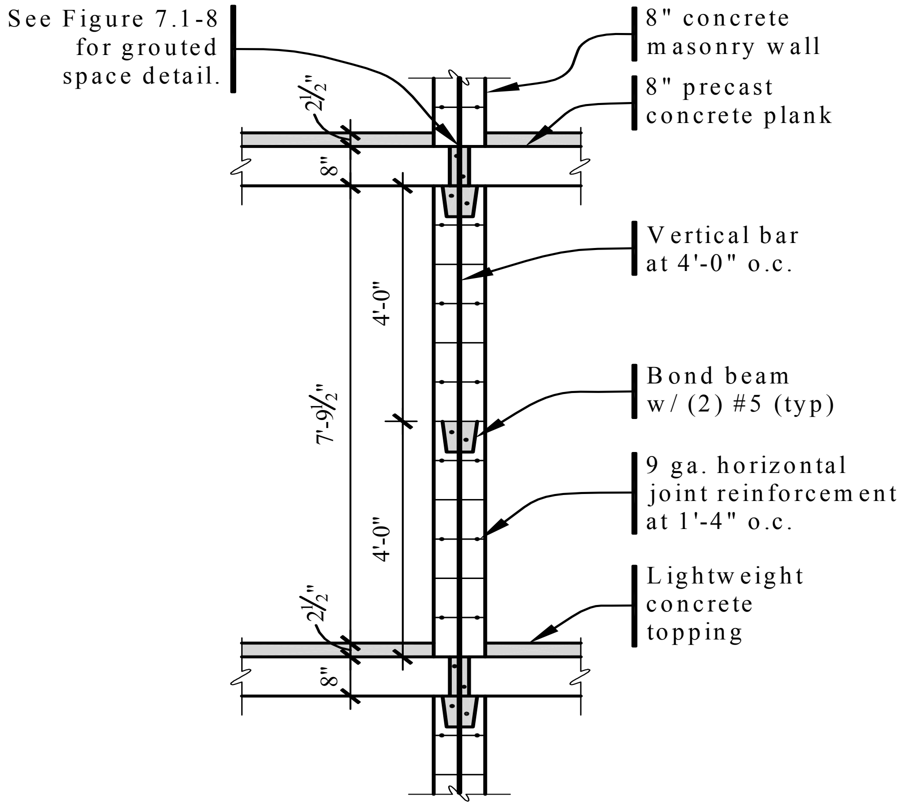


Figure 9.2-12 Typical wall section for the Los Angeles location (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

Story weight ,w_i:

Roof weight:

Roof slab (plus roofing) = (91 psf) (152 ft)(72 ft)	= 996 kips
Walls = (60 psf)(589 ft)(8.67 ft/2) + (60 psf)(4)(36 ft)(2 ft)	= 170 kips
Total	= 1,166 kips

There is a 2-ft-high masonry parapet on four walls and the total length of masonry wall is 589 ft.

Typical floor:

Slab (plus partitions)	= 996 kips
Walls = (60 psf)(589 ft)(8.67 ft)	= 306 kips
Total	= 1,302 kips

Total effective seismic weight, W = 1,166 + (4)(1,302) = 6,374 kips

This total excludes the lower half of the first story walls, which do not contribute to seismic loads that are not imposed on the CMU shear walls.

9.2.7.2 Los Angeles Base Shear Calculation

The seismic response coefficient, C_s, is computed using *Provisions* Eq. 5.4.1.1-1 [Eq. 5.2-2] and 5.4.1.1-2 [Eq. 5.2-3]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.00}{3.5/1} = 0.286 \qquad \text{Controls}$$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.60}{0.338(3.5/1)} = 0.507$$

where T is the fundamental period of the building, which is 0.338 sec as computed previously (the approximate period, based on building system and building height, will be the same for all locations). The value for C_s is taken as 0.286 (the lesser of these two). This value is still larger than the minimum specified in *Provisions* Eq. 5.3.2.1-3 which is:

$C_s = 0.044 S_{D1} I = (0.044)(0.60)(1) = 0.026$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated *Provisions* Eq. 5.4.1 [Eq.5.2-1]:

$V = C_s W = (0.286)(6{,}374) = 1{,}823$ kips

9.2.7.3 Los Angeles Vertical Distribution of Seismic Forces

The vertical distribution of seismic forces is determined in accordance with *Provisions* Sec. 5.4.4 [Sec. 5.2.3], which as described in Sec. 9.2.4.3. Note that for *Provisions* Eq. 5.4.3-2 [Eq. 5.2-11], $k = 1.0$ since $T = 0.338$ sec (similar to the previous example buildings).

The application of the *Provisions* equations for this building is shown in Table 9.2-17:

Table 9.2-17 Los Angeles Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (ft)	$w_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	1,166	43.34	50,534	0.309	564	564	4,890
4	1,302	34.67	45,140	0.276	504	1,608	14,150
3	1,302	26.00	33,852	0.207	378	1,446	26,686
2	1,302	17.33	22,564	0.138	252	1,698	41,409
1	1,302	8.67	11,288	0.069	126	1,824	57,222
∑	6,374		163,378	1.000	1,824		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m

9.2.7.4 Los Angeles Horizontal Distribution of Forces

This is the same as for the Birmingham 2 design; see Sec. 9.2.6.4.

Total shear in Wall Type D:

$$V_{tot} = 0.125V + 1.365(0.0238)V = 0.158V$$

The total story shear and overturning moment may now be distributed to each wall and the wall proportions checked. The wall capacity will be checked before considering deflections.

9.2.7.5 Los Angeles Transverse Wall D

The strength or limit state design concept is used in the *Provisions*.

9.2.7.5.1 Los Angeles Shear Strength

The equations are the same as for the prior locations for this example building. Looking forward to the design for flexural and axial load, the amplification factor on the shear is computed as:

$$1.25\frac{M_n}{M_u} = 1.25\frac{9156/0.85}{9012} = 1.49 \quad \text{(which is less than the 2.5 upper bound)}$$

Therefore, the demand shear is 1.49 times the value from analysis. (This design continues to illustrate the ELF analysis and; as explained for the Birmingham 2 design, smaller demands could be derived from the dynamic analysis.) All other parameters are similar to those for Birmingham 2 except that:

$$A_n = (2 \times 1.25 \text{ in.} \times 32.67 \text{ ft x } 12 \text{ in.}) + (41 \text{ in.}^2 \times 15 \text{ cells}) = 1{,}595 \text{ in.}^2$$

The shear strength of each Wall D, based on the aforementioned formulas and data, are summarized in Table 9.2-18.

Table 9.2-18 Los Angeles Shear Strength Calculations for Wall D

Story	V_x (kips)	M_x (ft-kips)	M_x/V_xd	$1.49V_x$ (kips)	P (kips)	ϕV_m (kips)	ϕV_s (kips)	ϕV_n (kips)	ϕV_n *max* (kips)
5	88.8	770	0.265	132.3	42	210.1	112.2	322.3	340
4	168.2	2229	0.406	250.6	90	205.7	112.2	317.9	318.7
3	227.7	4203	0.565	339.3	139	199.6	112.2	311.8	294.5
2	267.4	6522	0.747	398.4	188	191.3	112.2	303.8	266.8
1	287.2	9012	0.960	427.9	236	179.5	112.2	291.7	234.3

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

Just as for the Birmingham 2 design, the maximum on V_n controls over the sum of V_m and V_s at all stories except the top. Unlike the prior design, the shear capacity is inadequate in the lower three stories. The solution is to add grout. At the first story, solid grouting is necessary:

$$A_n = (7.625 \text{ in.})(32.67 \text{ ft.})(12 \text{ in./ft.}) = 2989 \text{ in.}^2$$
$$\varphi V_n \, max = 0.8(4.11)(0.0447 \text{ ksi})(2989 \text{ in.2}) = 439 \text{ kips} > 428 \text{ kips} \qquad \text{OK}$$

At the third story, six additional cells are necessary, and at the second story, approximately two out of three cells must be grouted. The additional weight adds somewhat to the demand but only about 2 percent. If the entire building were grouted solid (which would be common practice in the hypothetical location), the weight would increase enough that the shear strength criterion might be violated.

9.2.7.5.2 Los Angeles Axial and Flexural Strength

The basics of the flexural design have been demonstrated for the previous locations. The demand is much higher at this location, however, which introduces issues about the amount and distribution of reinforcement in excess of the minimum requirements. Therefore, the strength and ductility checks will both be examined.

9.2.7.5.2.1 Strength check

Load combinations, using factored loads, are presented in Table 9.2-19 for Wall D. In the table, Load Combination 1 is 1.4D + Q_E + 0.5L, and Load Combination 2 is 0.7D + Q_E.

Table 9.2-19 Los Angeles Load Combinations for Wall D

			Load Combination 1		Load Combination 2	
Level (x)	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
5	63	0	88	770	44	770
4	126	8	180	2229	88	2229
3	189	17	273	4203	132	4203
2	251	25	364	6522	176	6522
1	314	34	456	9012	220	9012

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

Strength at the bottom story (where P, V, and M are the greatest) is examined. This example considers Load Combination 2 from Table 9.2.19 to be the governing case, because it has the same lateral load as Load Combination 1 but lower values of axial force.

Refer to Figure 9.2-13 for notation and dimensions.

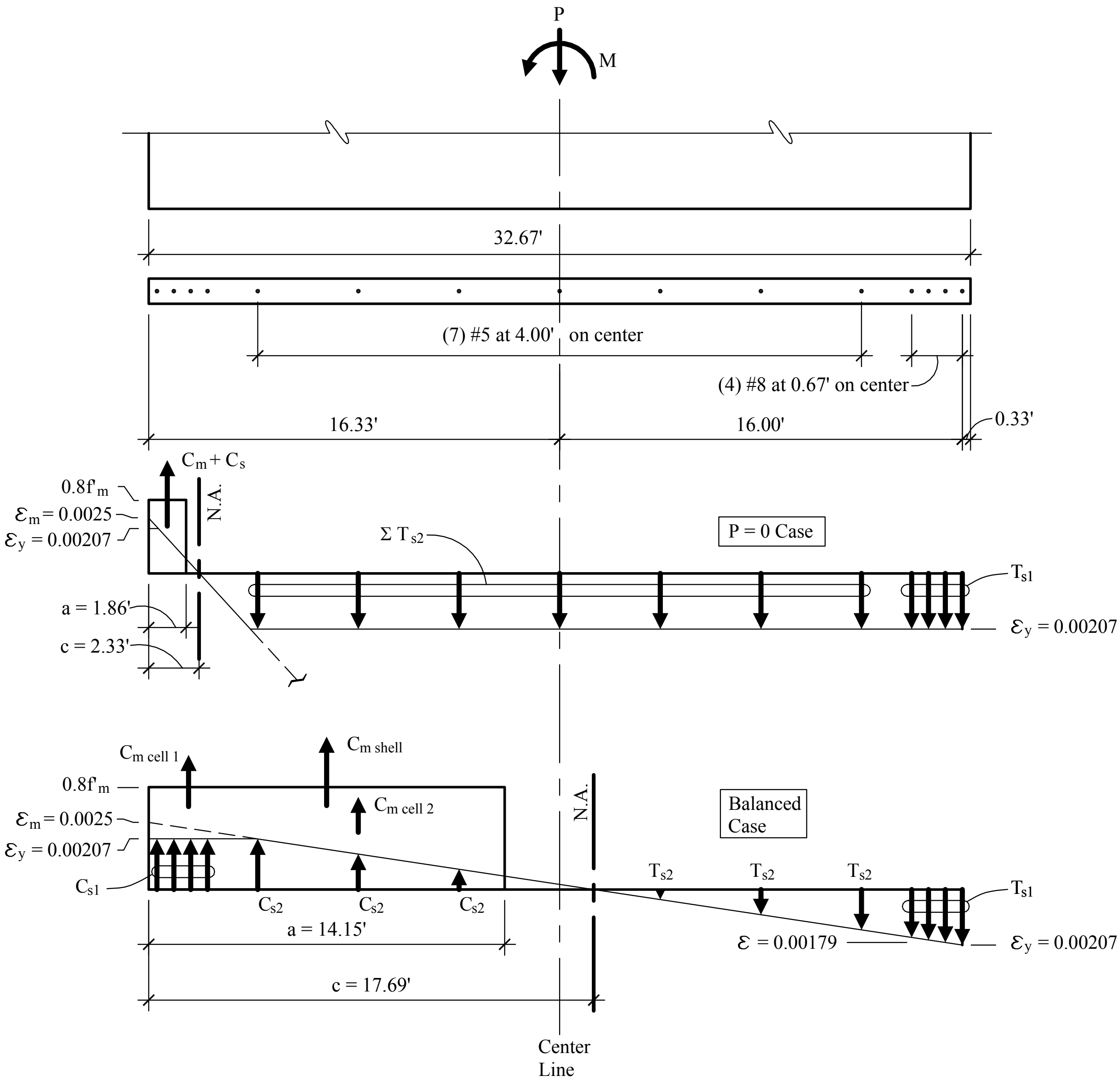


Figure 9.2-13 Los Angeles: Strength of wall D (1.0 ft = 0.3048 m).
Strength diagrams superimposed on strain diagrams for the two cases.

Examine the strength of Wall D at Level 1:

$$P_{u_{min}} = 220 \text{ kips}$$

$$P_{u_{max}} = 456 \text{ kips}$$

$$M_u = 9{,}012 \text{ ft-kips}$$

Because special reinforced masonry shear walls are used (Seismic Design Category D), vertical reinforcement at 4 ft. on center and horizontal bond beams at 4 ft on center are prescribed (*Provisions* Sec. 11.3.7.3 [ACI 530, Sec. 1.13.2.2.5]). (Note that the wall is 43.33 ft high, not 8 ft high, for purposes of determining the maximum spacing of vertical and horizontal reinforcement.)

For this bending moment, the minimum vertical reinforcement will not suffice. For reinforcement uniformly distributed, a first approximation could be taken from a simple model using an effective internal moment arm of 80 percent of the overall length of the wall:

$$A_s = \frac{M - 0.8Pl/2}{60(\text{ksi})(0.8l/2)} = \frac{9012 - 0.8 \times 220 \times 32.67/2}{60 \times 0.8 \times 32.67/2} = 7.8 \text{ in}^2 = 0.24 \text{ in}^2\text{/ft.}$$

The minimum vertical steel is 0.0007 times the gross area, which is 0.064 in.^2/ft. At the maximum spacing of 4 ft, a #5 bar is slightly above the minimum. Experimental evidence indicates that uniformly distributed reinforcement will deliver good performance. This could be accomplished with a #9 bar at 48 in. or a #8 bar at 40 in. This design will work well in a wall that is solidly grouted; however, for walls that are grouted only at cells containing reinforcement, it will be found that this wall fails the ductility check (which can be remedied by placing several extra grouted cells near each end of the wall as was shown in Sec. 9.1.5.4). The flexural design was completed before the shear design (described in the previous section) discovered the need for solid grout in the first story. The remainder of this flexural design check is carried out without consideration of the added grout. (It is unlikely that the interaction line will be affected near the design points, but the balanced point will definitely change.)

It has long been common engineering practice to concentrate flexural reinforcement near the ends of the wall. (This a normal result of walls that intersect to form flanges with reinforcement in both web and flange.) For this design, if one uses the minimum #5 bar at 48 inches, then the extra steel at the ends of the walls is approximately:

$$A_{s\,end} = (7.8 - 7 \times 0.31)/2 = 2.8 \text{ in}^2$$

Try #8 bars in each of the first four end cells and #5 bars at 4 ft on center at all intermediate cells.

The calculation procedure is similar to that presented in Sec. 9.2.4.5.2. The strain and stress diagrams are shown in Figure 9.2-13 for the Birmingham 1 building and the results are as follows:

<u>P = 0 case</u>

ϕP_n = 0
ϕM_n = 6,636 ft-kips

<u>Intermediate case, setting c = 4.0 ft</u>

ϕP_n = 223 kips
ϕM_n = 9,190 ft-kips

<u>Balanced case</u>

ϕP_n = 1049 kips
ϕM_n = 14,436 ft-kips

The simplified ϕP_n - ϕM_n curve is shown in Figure 9.2-14 and indicates the design with #8 bars in the first four end cells and #5 bars at 4 ft on center throughout the remainder of the wall is satisfactory.

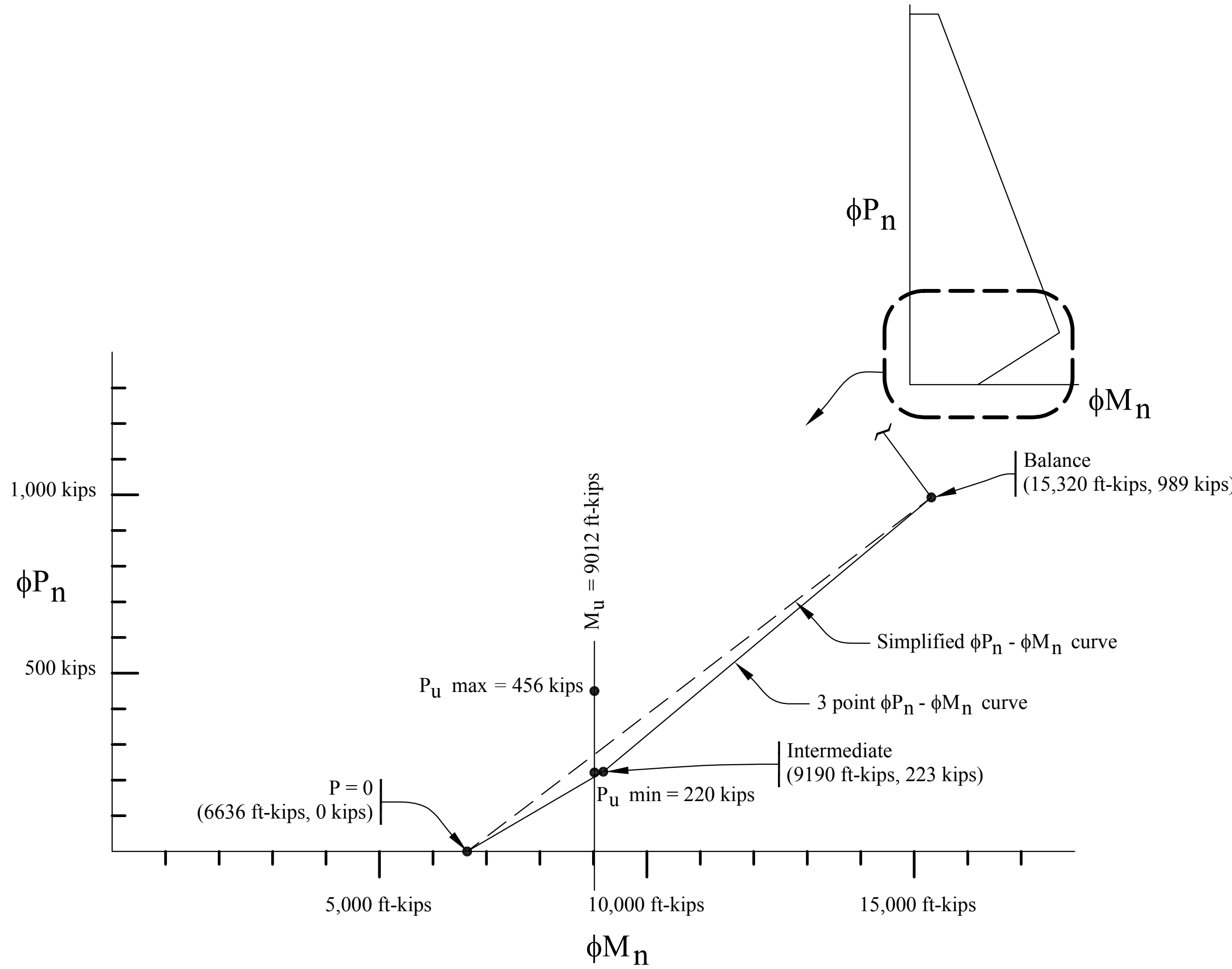


Figure 9.2-14 φP_{11} - φM_{11} diagram for Los Angeles Wall D (1.0 kip = 4.45 kN, 1.0 kip-ft = 1.36 kN-m).

9.2.7.5.2.2 Ductility check

Provisions Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5] has been illustrated in the prior designs. Recall that this calculation uses unfactored gravity axial loads (*Provisions* Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5]). Refer to Figure 9.2-15 and the following calculations which illustrate this using loads at the bottom story (highest axial loads). The extra grout required for shear is also ignored here. More grout gives higher compression capacity, which is conservative.

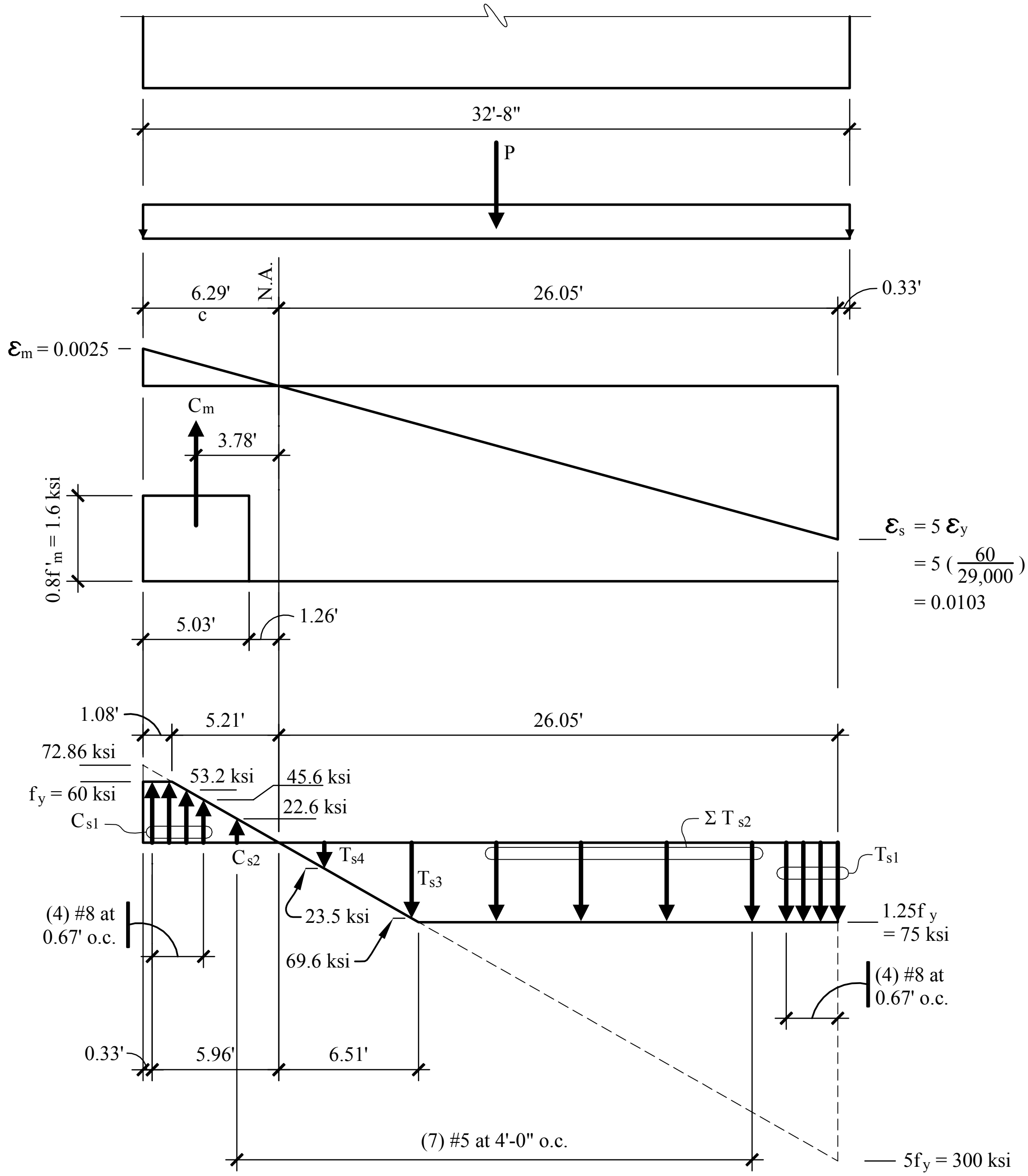


Figure 9.2-15 Ductility check for Los Angeles Wall D (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa)

For Level 1 (bottom story), the unfactored loads are:

P = 314 kips
$C_m = 0.8f'_m[(a)(b) + A_{cells}]$

where b = flange width = (2 × 1.25 = 2.5 in.) and A_{cells} = 41 in.^{2}

C_m = (1.6 ksi)[(5.03 ft × 12)(2.5 in.) + (5 cells)(41)] = 569.4 kips
C_{s1} = 0.79(2 x 60 + 53.2 + 45.6) = 172.9 kips
C_{s2} = (22.6 ksi)(0.31 in.^{2}) = 7.0 kips

$\sum C$ = 749 kips
$\sum T_{s1}$ = (4 × 0.79 in.^{2})(75 ksi) = 237 kips
$\sum T_{s2}$ = (4 × 0.31 in.^{2})(75 ksi) = 93.0 kips
T_{s3} = (0.31 in.^{2})(69.6 ksi) = 21.6 kips
T_{s4} = (0.31 in.^{2})(23.5 ksi) = 7.3 kips
$\sum T$ = 359 kips

$\sum C > \sum P + T$
749 kips > 673 kips

If a solution with fully distributed reinforcement were used, the tension from reinforcement would increase while the compression from grout at the end of the wall, as well as compression of steel at the compression would also decrease. The criterion would not be satisfied. Adding grout would be required.

[Refer to Sec. 9.2.4.5.2 for discussion of revisions to the ductility requirements in the 2003 *Provisions*.]

9.2.7.6 Los Angeles Deflections

Recall the assertion that the calculations for deflection involve many variables and assumptions and that any calculation of deflection is approximate at best. The requirements and procedures for computing deflection are provided in Sec. 9.2.4.6. [Refer to Sec. 9.2.4.6 for discussion of revisions to the deflection computations and requirements in the 2003 *Provisions*, as well as the potentially conflicting drift limits.]

For the Los Angeles building, the determination of whether the walls will be cracked is as follows:

b_e = effective masonry wall width
b_e = [(2 × 1.25 in.)(32.67 ft × 12) + (15 cells)(41 in.^{2}/cell)]/32.67 ft × 12) = 4.07 in.
$A = b_e\, l$ = (4.07 in.)(32.67 × 12) = 1595 in.^{2}
$S = b_e\, l^2/6$ = (7.07)(32.67 × 12)^2/6 = 104,207 in.^{3}
f_r = 0.250 ksi

P_u is calculated using 1.00D (See Table 9.2-18 for values, and refer to Sec. 9.2.4.6 for discussion). Table 9.2-20 provides a summary of these calculations. (The extra grout required for shear strength is also not considered here; the revision would slightly reduce the computed deflections by raising the cracking moment.)

Table 9.2-20 Los Angeles Cracked Wall Determination

Level	$P_{u_{min}}$ (kips)	M_{cr} (ft-kips)	M_x (ft-kips)	Status
5	63	2514	770	uncracked
4	126	2857	2229	uncracked
3	189	3200	4203	cracked
2	251	3538	6522	cracked
1	314	3880	9012	cracked

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

For the uncracked walls (Levels 4 and 5):

$I_n = I_g = b_e l^3/12$ = (4.07 in.)(32.67 × 12)^3/12 = 2.04 × 10^7 in.^{4}

For the cracked walls, the transformed cross section will computed by classic methods. Assuming the neutral axis to be about 10 ft in from the compression face gives five #5 bars in tension. The tension reinforcement totals:

$A_s = 4(0.79) + 5(0.31) = 3.16 + 1.55 = 4.71$ in.^{2}

The axial compression stiffens the wall. The effect is approximated with an equivalent area of tension reinforcement equal to half the compression. Thus, the total reinforcement becomes:

$A_{se} = 4.71 + 0.5(314)/60 = 4.71 + 2.62 = 7.33$ in.^{2}

The centroid of this equivalent reinforcement is 29.5 ft from the compression face. Following the classic method for transformed cracked cross sections and with $n = 19.3$:

$\rho = 7.33/(4.04 \times 29.5 \times 12) = 0.0051$
$\rho n = 0.0051(19.3) = 0.099$
$k = \mathrm{sqrt}(\rho n^2 + 2\rho n) - \rho n = 0.36$
$kd = c = 10.5$ feet (which is close enough to the assumed 10 feet)

$I_{cr} = bc^3/3 + \sum nA_s d^2 = 4.04(29.5 \times 12)^3 + 19.3(7.33)(29.5 - 10.5)^2(144) = 1.01 \times 10^7$ in.^{4}

The *Provisions* encourages the use of the cubic interpolation formula illustrated for the previous locations. For the values here, this yields $I_{eff} = 1.09 \times 10^7$ in.^{4}, which is about half the gross moment of inertia (which in itself is not a bad approximation for a cracked and well reinforced cross section). For this example, the deflection computation will instead use the cracked moment of inertia in the lower three stories and the gross moment of inertia in the upper two stories. The results from a RISA 2D analysis are shown in Table 9.2-21, and are about 5 percent higher than use of I_{eff} over the full height.

Table 9.2-21 Los Angeles Deflections

Level	F (kips)	I_{eff} (in.^{4})	$\delta_{flexural}$ (in.)	δ_{shear} (in.)	δ_{total} (in.)	$C_d\,\delta_{total}$ (in.)	Δ (in.)
5	84.0	2.04×10^7	0.491	0.127	0.618	2.163	0.455
4	75.1	2.04×10^7	0.370	0.118	0.488	1.708	0.543
3	56.3	1.01 × 107	0.237	0.096	0.333	1.166	0.515
2	37.6	1.01×10^7	0.118	0.068	0.186	0.651	0.413
1	18.8	1.01×10^7	0.033	0.035	0.068	0.238	0.238

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

The maximum drift occurs at Level 4 per *Provisions* Table 5.2.8 is:

$\Delta = 0.543$ in. < 1.04 in. $= 0.01\, h_n$ (*Provisions* Table 5.2.8 [Table 4.5-1]) OK

9.2.7.7 Los Angeles Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4.6.1.3] requires that the bearing walls be designed for out-of-plane loads determined as follows:

$w = 0.40\, S_{DS}\, W_c \geq 0.1W_c$
$w = (0.40)(1.00)(60 \text{ psf}) = 24\text{psf} \geq 0.1W_c$

The out-of-plane bending moment, using the strength design method for masonry, for a pressure, w = 24 psf and considering the P-delta effect, is computed to be 2,232 in.-lb/ft. This compares to a computed strength of the wall of 14,378 in.-lb/ft, considering only the #5 bars at 4 ft on center. Thus, the wall is loaded to about 16 percent of its capacity in flexure in the out-of-plane direction. (See Sec. 9.1 for a more detailed discussion of strength design of masonry walls, including the P-delta effect.)

9.2.7.8 Los Angeles Orthogonal Effects

According to *Provisions* Sec. 5.2.5.2.2 [Sec. 4.4.2.3], orthogonal interaction effects have to be considered for Seismic Design Category D when the ELF procedure is used (as it is here).

The out-of-plane effect is 16 percent of capacity, as discussed in Sec. 9.2.7.7 above. When considering the 0.3 combination factor, the out-of-plane action adds about 5 percent overall to the interaction effect. For the lowest story of the wall, this could conceivably require a slight increase in capacity for in-plane actions. In the authors' opinion, this is on the fringe of requiring real consideration (in contrast to the end walls of Example 9.1).

This completes the design of the transverse Wall D.

9.2.7.9 Los Angeles Summary of Wall Design for Wall D

8-in. CMU
f'_m = 2,000 psi

Reinforcement:

Four vertical #8 bars, one bar in each cell for the four end cells
Vertical #5 bars at 4 ft on center at intermediate cells
Two bond beams with two #5 bars at each story, at floor bearing and at 4 ft above each floor
Horizontal joint reinforcement at alternate courses recommended, but not required

Grout:

All cells with reinforcement and bond beams, plus solid grout at first story, at two out of three cells in the second story, and at six extra cells in the third story

Table 9.2-22 compares the reinforcement and grout for Wall D designed for each of the four locations.

Table 9.2-22 Variation in Reinforcement and Grout by Location

	Birmingham 1	New York City	Birmingham 2	Los Angeles
Vertical bars	5 - #4	9 - #4	12 - #4	8 - #8 + 7 - #5
Horizontal bars	10 - -#4 + jt. reinf	10 - #4 + jt. reinf	20 - #5	20 - #5
Grout (cu. ft.)	91	122	189	295

1 cu. ft. = 0.0283 m^3.

9.3 TWELVE-STORY RESIDENTIAL BUILDING IN LOS ANGELES, CALIFORNIA

9.3.1 Building Description

This 12-story residential building has a plan form similar to that of the five-story masonry building described in Sec. 9.2. The floor plan and building elevation are illustrated in Figures 9.3-1 and 9.3-2, respectively. The floors are composed of 14-in.-deep open web steel joists spaced at 30 in. that support a 3-in. concrete slab on steel form deck. A fire-rated ceiling is included at the bottom chord of the joists. Partitions, including the shaft openings, are gypsum board on metal studs, and the exterior nonstructural curtain walls are glass and aluminum.

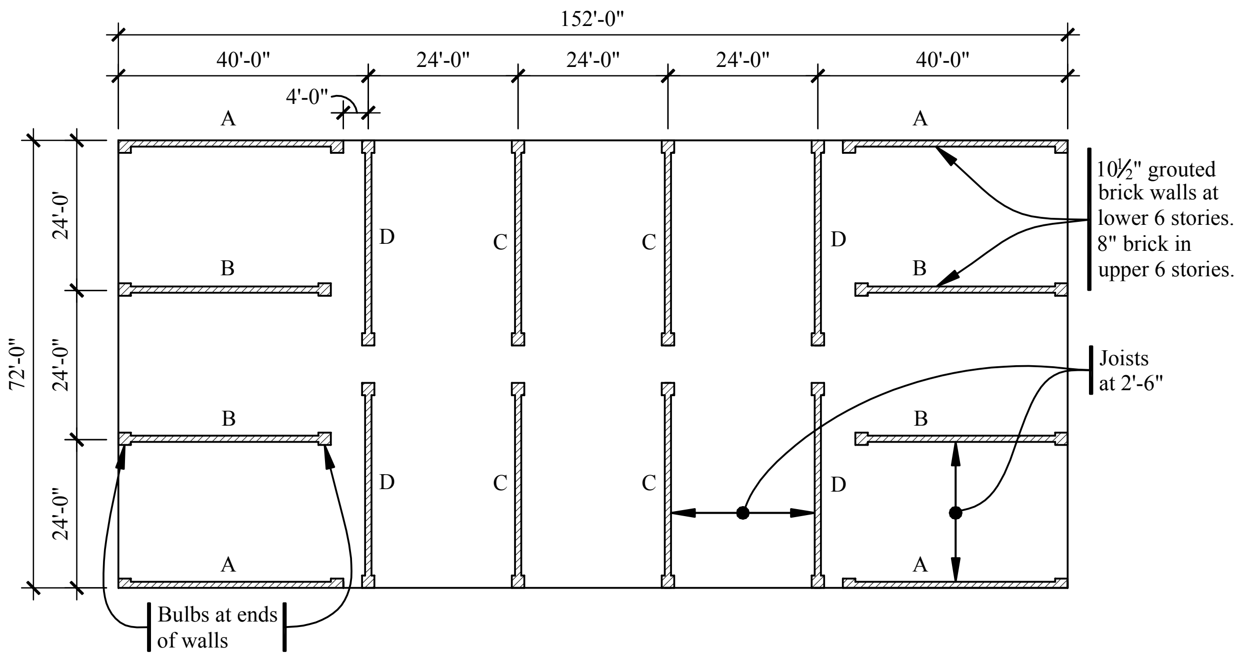


Figure 9.3-1 Floor plan (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

All structural walls are of grouted brick. For purposes of illustration, two styles of wall are included. The lower six stories have 10-1/2in.-thick walls consisting of two wythes of 4-in. (nominal) brick and a 3-1/4-in. grout space. The upper six stories have 8-in. (nominal) brick, hollow unit style, with the vertical reinforcing in the cells and the horizontal reinforcing in bond beams. (In actual construction, however, a single style wall might be used throughout: either a two-wythe grouted wall or a through-the-wall unit of an appropriate thickness). The walls are subject to high overturning moments and have a reinforced masonry column at each end. The column concentrates the flexural reinforcement and increases resistance to overturning. (Similar concentration and strength could be obtained with transverse masonry walls serving as flanges for the shear walls had the architectural arrangement been conducive to this approach.) Although there is experimental evidence of improved performance of walls with all vertical reinforcement uniformly distributed, concentration at the ends is common in engineering practice and the flexural demands are such for this tall masonry building that the concentration of masonry and reinforcement at the ends is simply much more economical.

The compressive strength of masonry, f'_m, used in this design is 2,500 psi for Levels 1 through 6 and 3,000 psi for Levels 7 through 12.

This example illustrates the following aspects of the seismic design of the structure:

1. Development of equivalent lateral forces
2. Reinforced masonry shear wall design
3. Check for building deflection and story drift
4. Check of diaphragm strength.

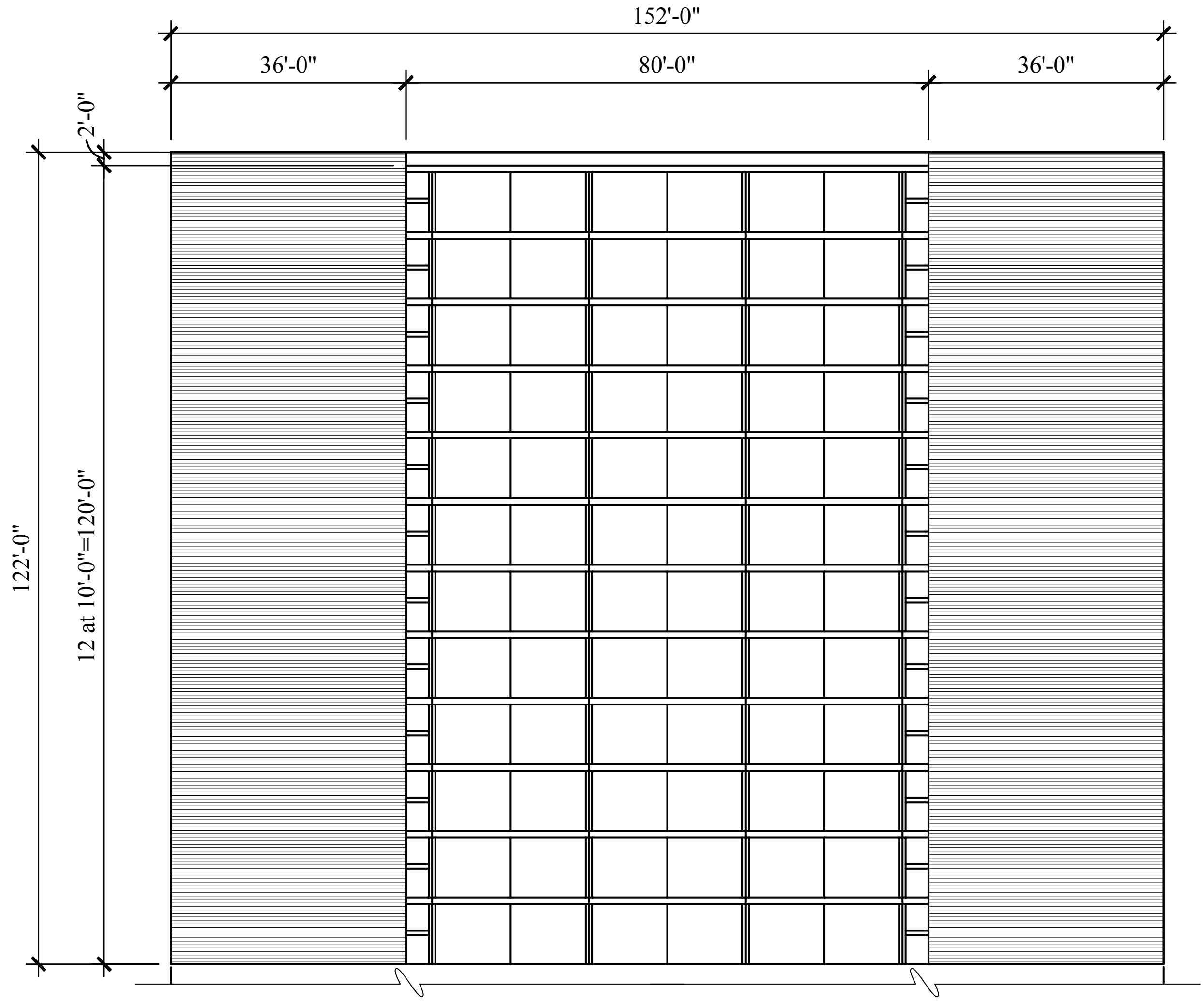


Figure 9.3-2 Elevation (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

9.3.2 Design Requirements

9.3.2.1 *Provisions* Design Parameters

Table 9.3-1 shows the design parameters for building design.

Table 9.3-1 Design Parameters

Design Parameter	Value
S_s (Map 1 [Figure 3.3-3])	1.5
S_1 (Map 2 [Figure 3.3-4])	0.6
Site Class	C
F_a	1
F_v	1.3
$S_{MS} = F_a S_s$	1.5
$S_{M1} = F_v S_1$	0.78
$S_{DS} = 2/3\ S_{MS}$	1
$S_{D1} = 2/3\ S_{M1}$	0.52
Seismic Design Category	D
Masonry Wall Type	Special Reinforced
R	3.5
Ω_0	2.5
C_d	3.5

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

9.3.2.2 Structural Design Requirements

The load path consists of the floors acting as horizontal diaphragms and the walls parallel to the motion acting as shear walls.

Soil-structure interaction is not considered.

The building is a bearing wall system (*Provisions* Table 5.2.2 [4.3-1]) .

Deformational compatibility must be assured (*Provisions* Sec. 5.2.2.4.3 [Sec. 4.5.3]). The structural system is one of non-coupled shear walls. Crossing beams over the halls (their design is not included in this example) will need to continue to support the gravity loads from the floors and roof during an earthquake but will not provide coupling between the shear walls.

The building is symmetric in plan but has the same torsional irregularity described in Sec. 9.2.5.4. The vertical configuration is regular except for the change in wall type between the sixth and seventh stories, which produces a significant discontinuity in stiffness and strength, both for shear and flexure (*Provisions* Sec. 5.2.3.3 [Sec. 4.3.2.3] and 5.2.6.2.3 [Sec. 4.6.1.6]). There is no weak story because the strength does not increase as one goes upward. The stiffness discontinuity will be shown to qualify as regular.

Provisions Table 5.2.5.1 [Table 4.4-1] would not permit the use of the ELF procedure of *Provisions* Sec. 5.4 [Sec. 5.2]; instead a dynamic analysis of some type is required. As will be illustrated, this particular building does not really benefit from this requirement.

The design and detailing must comply with the requirements of *Provisions* Sec. 5.2.6 [Sec. 4.6].

The walls must resist forces normal to their plane (*Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3]). These forces will be used when addressing the orthogonal effects (*Provisions* Sec.5.2.5.2.2 [Sec. 4.4.2.3]).

With eight walls in each direction, the system is expected to be redundant.

Tie and continuity requirements for anchorage of masonry walls must be considered when detailing the connections between floors and walls (*Provisions* Sec. 5.2.6.1.2 [Sec. 4.6.2.1] and 5.2.6.1.3).

Openings in walls and diaphragms need to be reinforced (*Provisions* Sec. 5.2.6.2.2 [Sec. 4.6.1.4]).

Diaphragms need to be designed to comply with *Provisions* Sec. 5.2.6.2.6 [Sec. 4.6.3.4].

The story drift limit is $0.01h_{sx}$ (*Provisions* Sec. 5.2.8 [Sec. 4.5.1]) and the overall drift limit is $0.01h_{sn}$ (*Provisions* Sec. 11.5.4.1). For this structure the difference between these two is significant, as will be shown.

[The deflection limits have been removed from Chapter 11 of the 2003 *Provisions* because they were redundant with the general deflection limits. Based on ACI 530 Sec. 1.13.3.2, the maximum drift for all masonry structures is 0.007 times the story height. Thus, there appears to be a conflict between ACI 530 and 2003 *Provisions* Table 4.5-1.]

9.3.2.3 Load Combinations

The basic load combinations (*Provisions* Sec. 5.2.7 [Sec. 4.2.2]) are the same as those in ASCE 7 except that the seismic load effect, E, is defined by *Provisions* Eq. 5.2.7-1 [Eq. 4.2-1] and 5.2.7-2 [Eq. 4.2-2] as:

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

Based on the configuration of the shear walls and the results presented in Sec. 9.2, the reliability factor, ρ, is treated as equal to 1.0 for both directions of loading. Refer to Sec. 9.2.3.1 for additional information.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. The intent is that the aspect ratio is based on story height, not total height. Therefore, the redundancy factor would not have to be investigated (ρ = 1.0) for this building.]

The discussion on load combinations for the Los Angeles site in Sec.9.2 is equally applicable to this example. Refer to Sec. 9.2.3.2 for determination of load combinations.

The load combinations representing the extreme cases are:

$$1.4D + Q_E + 0.5L$$
$$0.7D + Q_E$$

9.3.3 Seismic Force Analysis

The analysis is performed using the ELF procedure of *Provisions* Sec. 5.4 [Sec. 5.2] and checked with a modal response spectrum (MRS) analysis in conformance with *Provisions* Sec. 5.5 [Sec. 5.3]. This example illustrates an analysis for earthquake motions acting in the transverse direction only. Earthquake motions in all directions will need to be addressed for an actual project.

9.3.3.1 Building Weights

For the ELF analysis, the masses are considered to be concentrated at each floor level whereas, for the MRS analysis, it is distributed on both wall and floor elements. Note that the term "level" corresponds to the slab above each story. Thus Level 1 is the second floor; Level 12 is the roof.

Lower Levels:

Slab, joists, partitions, ceiling, mechanical/electrical (M/E), curtain wall at 53 psf
(0.053 ksf)(152 ft)(72 ft) = 580 kips/story

Walls: 10.5 in. at 114 psf (brick at 73 psf + grout at 41 psf)
(0.114 ksf)(10 ft)[(8)(29 ft) + (4)(30 ft) + (4)(32 ft)] = 547 kips/story

Bulbs at ends of walls: (24 in. × 24 in. bulb)
Brick: 2.01 ft^2 /bulb; Grout: 1.99 ft^2/bulb
[(2.01 ft^2)(0.120 kcf) + (1.99 ft^2)(0.150 kcf)] (10 ft)(32 bulbs) = 173 kips/story

Upper Levels:

Slab, joists, partitions, ceiling, M/E, curtain wall at 53 psf
(0.053 ksf)(152 ft)(72 ft) = 580 kips/story

Walls: 8 in. Partially grouted brick at 48 psf
(0.048 ksf)(10 ft)[(8)(29.67 ft) + (4)(30.67 ft) + (4)(32.67 ft)] = 236 kips/story

Bulbs at ends of walls (grouted 20 in. × 20 in. brick bulb)
(0.315 klf/bulb)(10 ft)(32 bulbs) = 101 kips/story

Roof:

Slab, roofing, joists,, ceiling, M&E, curtain wall at 53 psf :
(0.053 ksf)(152 ft)(72 ft) = 580 kips

Walls
(238 kips/story + 101 kips/story)/2 = 170 kips

Parapet
(4 parapets)(2 ft)[(0.048 kips/lf)(33 ft) +(3.15 kips/bulb)(2 bulbs)/(10 ft)] = 18 kips

Preliminary design indicates a 10-1/2-in. wall with bulbs for the six lower stories and an 8-in. wall with bulbs for the six upper stories. Therefore, effective seismic weight, W, is computed as follows:

Levels 1-5	(5)(580 + 547 + 173)	= (5)(1,300 kips/level)	= 6,500 kips
Level 6	580 + (547 + 236)/2 + (173 + 101)/2	= 1,109 kips	= 1,109 kips
Levels 7-11	(5)(580 + 236 + 101)	= (5)(917 kips/level)	= 4,585 kips
Level 12 (roof)	(580 + 170 + 18)	= 768 kips	= 768 kips
Total			= 12,962 kips

The weight of the lower half of walls for the first story are not included with the walls for Level 1 because the walls do not contribute to the seismic loads.

9.3.3.2 Base Shear

The seismic coefficient, C_s, for the ELF analysis is computed as:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.00}{3.5/1.0} = 0.286$$

The value of C_s need not be greater than:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.52}{(0.75)(3.5/1)} = 0.198$$

The value of the fundamental period, T, was determined from a dynamic analysis of the building modeled as a cantilevered shear wall. RISA 2D was used for this analysis, with cracked sections taken into account. From this analysis, a period of $T = 0.75$ sec was determined. See Sec. 9.3.4. This value is also obtained from the 3D dynamic analysis (described subsequently) for the first translational mode in the transverse direction when using a reduced modulus of elasticity to account for cracking in the masonry (approximately 60 percent of the nominal value for E). *Provisions* Sec. 5.4.2 [Sec. 5.2-2] requires that the fundamental period, T, established in a properly substantiated analysis be no larger than the approximate period, T_a, multiplied by C_u, determined from *Provisions* Table 5.4.2 [Table 5.2-1]. The approximate period of the building, T_a, is calculated based as:

$$T_a = C_r h_n^{3/4} = (0.02)(120)^{0.75} = 0.725 \text{ sec}$$

where $C_r = 0.02$ from *Provisions* Table 5.4.2.1 [Table 5.2-2], and $h_n = 120$ ft

$$T_a C_u = (0.725)(1.4) = 1.015 \text{ sec} > 0.75 \text{ sec} = T$$

(Note that $T = 0.75$ sec will be verified later when deflections are examined).

The value for C_s is taken to be 0.198 (the minimum of the two values computed above). This value is still larger than the minimum specified:

$$C_s = 0.044 I S_{D1} = (0.044)(1.0)(0.60) = 0.0264$$

[This minimum Cs value has been removed in the 2003 *Provisions*. In its place is a minimum Cs value for long-period structures, which is not applicable to this example.]

The total seismic base shear is then calculated by *Provisions* Eq. 5.4.1 [Eq. 5.2-1]:

$$V = C_s W = (0.198)(12{,}962 \text{ kips}) = 2{,}568 \text{ kips}$$

A 3-D model was created in SAP 2000 for the MRS analysis. Just as for the five-story building described in Sec. 9.2, the masonry walls were modeled as shell bending elements and the floors were modeled as an assembly of beams and shell membrane elements. See Sec. 9.2.6.2 for further description. The difference in f'_m between upper and lower stories was not modeled; the value of E_m used was 1,100 ksi, which is 59 percent of the value from *Provisions* Eq. 11.3.10.2 for the lower stories. [Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900 f'_m$ per ACI 530 Sec. 1.8.2.2.1.] As mentioned, this value was selected as an approximation of the effects of flexural cracking. Unlike the five-story building, the difference in length between the longitudinal and transverse walls was modeled. However, to simplify construction of the model, wall types A and B are the same length. Because this example illustrates design in the transverse direction, this liberty has little effect. Table 9.3-2 shows data on the modes of vibration used in the analysis.

Table 9.3-2 Periods, mass participation ratios, and modal base shears in the transverse direction for modes used in analysis

Mode number	Period (seconds)	Individual mode (percent)			Cumulative sum (percent)			Trans. base shear
		Long.	Trans.	Vert.	Long.	Trans.	Vert.	
1	0.9471	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	0.7469	0.00	59.12	0.00	0.00	59.12	0.00	1528.0
3	0.6941	59.16	0.00	0.00	59.16	59.12	0.00	0.0
4	0.2247	0.00	0.00	0.00	59.16	59.12	0.00	0.0
5	0.1763	0.00	24.38	0.00	59.16	83.50	0.00	896.2
6	0.1669	24.57	0.00	0.00	83.73	83.50	0.00	0.0
7	0.1070	0.00	0.01	0.00	83.73	83.51	0.00	0.5
8	0.1059	0.00	0.00	0.28	83.74	83.51	0.28	0.0
9	0.1050	0.00	0.00	29.48	83.74	83.51	29.76	0.0
10	0.0953	0.00	0.00	0.00	83.74	83.51	29.76	0.0
11	0.0900	0.00	0.00	1.51	83.74	83.51	31.27	0.0
12	0.0858	0.00	0.03	0.01	83.74	83.54	31.28	1.1
13	0.0832	0.00	7.25	0.00	83.74	90.79	31.28	234.4
14	0.0795	7.11	0.00	0.00	90.85	90.79	31.28	0.0
15	0.0778	0.04	0.00	0.19	90.88	90.79	31.48	0.0
16	0.0545	0.00	4.47	0.00	90.88	95.26	31.48	117.5
17	0.0526	4.44	0.00	0.00	95.32	95.26	31.48	0.0
18	0.0413	0.01	1.24	0.00	95.33	96.51	31.48	29.1
19	0.0392	1.66	0.05	0.00	96.99	96.55	31.48	1.1
20	0.0358	0.07	0.87	0.00	97.06	97.43	31.48	19.5
21	0.0288	1.59	0.33	0.01	98.66	97.76	31.49	7.0
22	0.0278	0.33	1.40	0.00	98.99	99.16	31.49	28.9
23	0.0191	0.76	0.23	0.01	99.75	99.39	31.50	4.3
24	0.0186	0.23	0.60	0.00	99.98	99.98	31.50	11.1

1 kip = 4.45kN.

The combined modal base shear is 1,791 kips

The fundamental mode captures no translation of mass; it is a pure torsional response. This is a confirmation of the intent of the torsional irregularity provision. The first translational mode has a period of 0.75 sec, confirming the earlier statements. Also note that the base shear is only about 70 percent of the ELF base shear (2,568 kips) even though the fundamental period is the same. The ELF analysis assumes that all the mass participates in the fundamental mode whereas the dynamic analysis does not. The absolute sum of modal base shears is higher than the ELF but the statistical sum is not. *Provisions* Sec. 5.5.7 requires that the modal base shear be compared with 85 percent of the ELF base shear. The comparison value is 0.85(2,568 kips), which is 2,183 kips. Because this is greater than the value from the modal analysis, the modal analysis results would have to be factored upwards by the ratio 2,183/1,791 = 1.22. The period used for this comparison cannot exceed C_uT_a, which is 1.015 sec as described previously. Note that the period used is from Mode 2, because Mode 1 is a purely torsional mode. The 1.22 factor is very close to the factor for the five story building computed in Sec. 9.2.6.2; an additional comparison will follow.

9.3.3.3 Vertical Distribution of Seismic Forces

Carrying forward with the ELF analysis, *Provisions* Sec. 5.4.3 [Sec. 5.2.3] provides the procedure for determining the portion of the total seismic loads assigned to each floor level. The story force, F_x, is calculated as:

$$F_x = C_{vx} V$$

and

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

For T = 0.75 sec, which is between 0.5 sec and 2.5 sec, the value of k is determined to be 1.125 based on interpolation (*Provisions* Sec. 5.4.3 [Sec. 5.2.3]).

The seismic design shear in any story shall be determined from:

$$V_x = \sum_{i=x}^{n} F_i$$

The story overturning moment is computed from:

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

Table 9.3-3 shows the application of these equations for this building.

Table 9.3-3 Seismic Forces and Moments by Level

Level (x)	w_x (kips)	h_x (kips)	$w_x h_x^{1.125}$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (kips)
12	768	120	167,700	0.128	329	329	3,300
11	917	110	181,500	0.139	357	686	10,200
10	917	100	163,100	0.125	320	1,006	20,200
9	917	90	144,800	0.111	284	1,291	33,100
8	917	80	126,900	0.097	249	1,540	48,500
7	917	70	109,200	0.084	214	1,754	66,000
6	1,109	60	111,000	0.085	218	1,972	85,800
5	1,300	50	106,000	0.081	208	2,181	107,600
4	1,300	40	82,500	0.063	162	2,342	131,000
3	1,300	30	59,700	0.046	117	2,460	155,600
2	1,300	20	37,800	0.029	74	2.534	181,000
1	1,300	10	17,300	0.013	34	2,568	206,600
			1,307,400	1.00	2,568		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m.

The dynamic modal analysis does give a direct output for the gross overturning moment, of about 66 percent of the moment from the ELF analysis. Because the model is built with shell elements, there is no direct value for the variation of moment with height.

9.3.3.4 Horizontal Distribution

For the ELF analysis, the approach is essentially the same as used for the five-story masonry building described in Sec. 9.2.4.4:

Direct shear: All transverse walls have the same properties, except axial load. Axial load affects cracking but, each wall considered has the same stiffness. Therefore, each will resist an equivalent amount in direct shear:

$V = V/8 = 0.125V_x$

Torsion: The center of mass corresponds with the center of resistance; therefore, the only torsion is due to the 5 percent accidental eccentricity in accordance with *Provisions* Sec. 5.4.4.2 [Sec. 5.2.4.2]:

$M_{ta} = 0.05\ bV = (0.05)(152 \text{ ft})V = 7.6V$

The longitudinal walls are slightly longer than the transverse walls. Unlike the example in Sec. 9.2, this difference will be illustrated here in a simple fashion. The diaphragm is assumed to be rigid. When the walls are not identical, a measure of the actual stiffness is necessary; for masonry walls, this involves both flexural and shear deformations. The conventional technique is an application of the following equation for deformation of a simple cantilever wall without bulbs or flanges at the ends:

$$\Delta_{wall} = \frac{Vh^3}{3E_m I} + \frac{6Vh}{5G_m A}$$

Considering $G_m = 0.4\ E_m$, $A = Lt$, and $I = L^3t/12$, this can be simplified to :

$$\Delta_{wall} = \frac{V}{Et}\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]$$

Rigidity, K, is inversely proportional to deflection. Considering E and t as equal for all walls:

$$K = \frac{1}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$$

Figure 9.3-3 identifies the walls.

For a multistory building, the quantity h is not easy to pin down. For this example, the authors suggest the following approach: use h= 10 ft (one story) to evaluate the shear in the wall at the base and also use h = 80 ft (two thirds of total height) to evaluate the moments in the walls. Table 9.3-4 shows some of the intermediate steps for these two assumptions.

(d, as used here, is the distance of the wall to the centroid of the building, not the length of the wall, as used elsewhere)

Table 9.3-4 Relative Rigidities

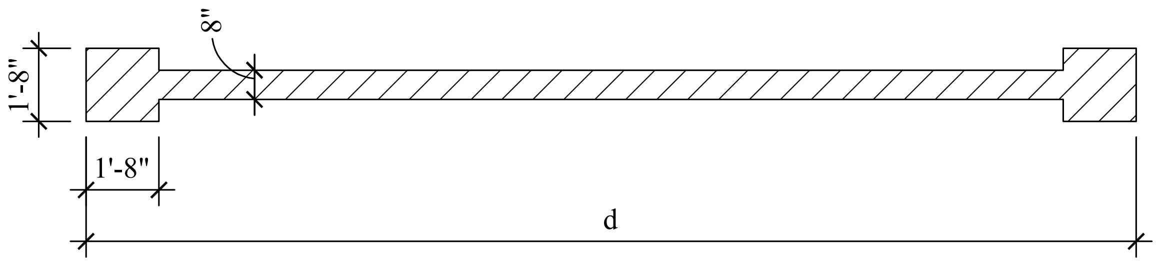
Wall	Length (ft)	Arm, d (ft)	for shear, h = 10 ft			for moment, h = 80 ft		
			h/d	K	Kd^2 (ft^2)	h/d	K	Kd^2 (ft^2)
A	36	36	0.278	1.088	1,410	2.22	0.01978	25.63
B	34	12	0.294	1.017	146	2.35	0.01690	2.43
C	33	12	0.303	0.980	141	2.42	0.01556	2.24
D	33	36	0.303	0.980	1,270	2.42	0.01556	20.17
					2,967			50.47

1.0 ft = 03.048 m

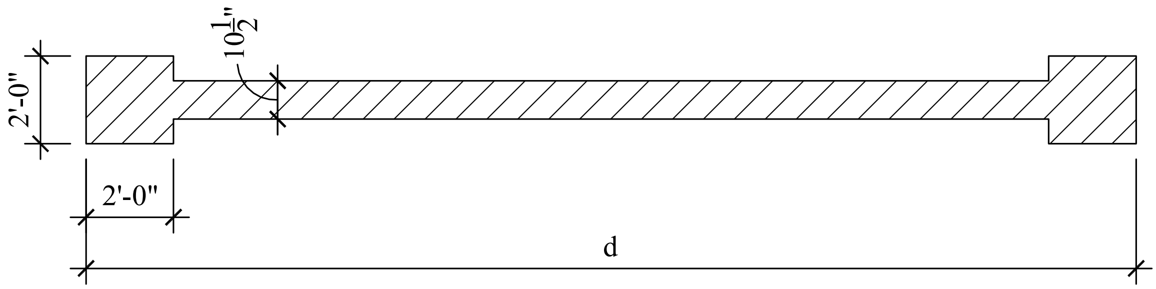
The total torsional rigidity is four times the amount in Table 9.3-4, since there are four walls of each type. When considering torsion, Wall D is the critical member (shortest length, greatest d).

For shear due to accidental torsion:

$$V_t = \frac{MKd}{\Sigma Kd^2} = 7.6V\left[\frac{(0.980)(36)}{4(2,967)}\right] = 0.0226V \text{ or } 7.6V\left[\frac{0.01556(36)}{4(50.47)}\right] = 0.0211V$$



Wall at Stories 7 to 12



Wall at Stories 1 to 6

Wall length	
Wall	d
A	36'-0"
B	34'-0"
C	33'-0"
D	33'-0"

Figure 9.3-3 Wall dimensions (1.0 in. = 25.4 mm, 1.0 ft = 0.0348 m).

When considering the approximations involved, the remainder of the ELF example will simply use $0.0226V$ for V_t. Because a 3D analytical model exists, a simplistic load case with a static horizontal torsion at each level was defined. The couple varied directly with height, so the variation of mass with height was ignored. Examining the base reactions for Wall D yields a torsional shear equal to $0.0221V$ and an overturning moment corresponding to $0.0191V$. Therefore, the hand computations illustrated are somewhat conservative.

Total shear for Wall D is equal to the direct shear plus shear due to accidental torsion, which is computed as:

$$0.125V + 0.0226V = 0.148V$$

The resulting shears and overturning moments for Wall D are shown in Table 9.3-5.

Table 9.3-5 Shear for Wall D

Level	Story Shear (kips)	Wall Shear (kips)	Story Moment (ft-kips)	Wall Moment (ft-kips)
12	329	49	3,300	500
11	686	102	10,200	1,500
10	1,006	149	20,200	3,000
9	1,291	191	33,100	4,900
8	1,540	228	48,500	7,200
7	1,754	260	66,000	9,800
6	1,972	292	85,800	12,700
5	2,181	323	107,600	15,900
4	2,342	347	131,000	19,400
3	2,460	364	155,600	23,000
2	2,534	375	181,000	26,800
1	2,568	380	206,600	30,60

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

When considering accidental torsion, a check for torsional irregularity must be made. First consider the case used for design: a direct shear of $0.125V$ and a torsional shear of $0.0226V$. The ratio of extreme displacement to average displacement can be found from these values and the dimensions, considering symmetry:

Average displacement is proportional to $0.125V$
Torsional displacement at Wall D is proportional to $0.0226V$
Torsional displacement at the corner is proportional to $(0.0226V)((152 \text{ ft.}/2)/36 \text{ ft.} = 0.0447V$
Ratio of corner to average displacement = (0.125 + 0.0447)/0.125 = 1.38

If the lower value of torsional shear, $0.0191V$, found from the 3D computer analysis for the static torsion is used, the ratio becomes 1.32. In either case, the result is a torsional irregularity (ratio exceeds 1.2) but not an extreme torsional irregularity (ratio does not exceed 1.4). The reason for the difference from the five-story building, in which the ratio exceeded 1.4, is that the longer walls in the longitudinal directions. For the ELF analysis, *Provisions* Sec. 5.4.4.3 [Sec. 5.2.4.3] requires the accidental torsion to be amplified:

$$A_x = \left(\frac{Max\ displacement}{1.2\ Ave\ displacement} \right)^2 \leq 3.0$$

If one uses the ratio of 1.32 based on the 3D computer analysis, the amplifier is 1.21 and the torsional shear becomes $1.32(0.0191V) = 0.0231V$. This is close enough to the unamplified $0.0226V$ that the ELF analysis will simply proceed with a torsional shear of $0.0226V$.

As described in Sec. 9.2.6.4 for the five-story building, the 3D analytical model was altered to offset the center of mass from the center of rigidity. The modal periods, mass participation ratios, and base shears are given in Table 9.3-6. The total base shear is 1620 kips, down from the 1,791 kips found without the eccentricity. The 1,620 kips still slightly exceeds the minimum for design of 1,613 kips described earlier. Thus, MRS analysis can be used directly in the load combinations and can be considered to include the amplified accidental torsion.

Table 9.3-6 Periods, Mass Participation Ratios, and Modal Base Shears in the Transverse Direction for Modes Used in Analysis of Building with Deliberate Eccentricity

Mode number	Period (seconds)	Individual mode (percent) Long.	Trans.	Vert.	Cumulative sum (percent) Long.	Trans.	Vert.	Trans. base shear
1	0.965	0.0	8.5	0.0	0.0	8.5	0.0	169.4
2	0.723	0.0	50.6	0.0	0.0	59.1	0.0	1352.7
3	0.694	59.2	0.0	0.0	59.2	59.1	0.0	0.0
4	0.229	0.0	3.3	0.0	59.2	62.5	0.0	122.7
5	0.171	0.0	21.0	0.0	59.2	83.5	0.0	772.7
6	0.167	24.6	0.0	0.0	83.7	83.5	0.0	0.0
7	0.120	0.0	0.0	20.3	83.7	83.5	20.3	0.0
8	0.108	0.0	1.0	0.0	83.7	84.4	20.3	35.3
9	0.105	0.0	0.0	0.0	83.7	84.4	20.3	0.1
10	0.097	0.0	0.0	0.0	83.7	84.4	20.3	0.1
11	0.090	0.0	0.0	11.4	83.8	84.4	31.7	0.0
12	0.081	0.0	6.2	0.0	83.8	90.7	31.7	198.6
13	0.079	7.1	0.0	0.1	90.9	90.7	31.8	0.0
14	0.074	0.0	0.0	3.1	90.9	90.7	34.8	0.1
15	0.072	0.0	0.6	0.1	90.9	91.3	34.9	17.5
16	0.061	0.0	0.5	0.1	90.9	91.7	34.9	12.8
17	0.053	4.3	0.0	0.0	95.2	91.7	34.9	0.0
18	0.052	0.0	4.1	0.0	95.2	95.8	34.9	104.8
19	0.043	0.7	0.0	0.0	95.9	95.8	34.9	0.0
20	0.037	1.5	0.0	0.0	97.4	95.8	35.0	0.1
21	0.035	0.0	2.5	0.0	97.4	98.3	35.0	54.7
22	0.027	1.8	0.0	0.0	99.2	98.3	35.0	0.0
23	0.022	0.0	1.7	0.0	99.2	100.0	35.0	32.8
24	0.018	0.8	0.0	0.0	100.0	100.0	35.0	0

1 kip = 4.45 kN

The combined modal base shear is 1620 kips.

Mode 1 now includes a translational component, and the comparison to an ELF base shear would be performed with its period. For T = 0.965 sec, the ELF base shear becomes 1,996 kips, and the comparison value is 0.85 (1996) = 1,696 kips. This is 78 percent of the value for the symmetric model and illustrates one of the problems in handling accidental torsion in a consistent fashion.

Without factoring the modal results up to achieve a base shear of 1,696 kips (a factor of 1.047), the reactions indicate that the base shear for wall D is 266.5 kips, or 0.1645 times the total base shear. If one takes the direct shear as one-eighth (0.125V), that leaves 0.0395V for the dynamically amplified shear due to accidental torsion, which could be interpreted to be an amplification of 1.79 over the shear of 0.0221V found for the static torsion. Thus, it is clear that the amplification value of 1.21 from the equation given for the ELF analysis underestimates the dynamic amplification of accidental torsion. The bottom line is that the Wall D shear of 266.5 kips from the dynamic analysis is significantly less than the shear of 380 kips found in the ELF analysis without amplification of accidental torsion. The example will proceed based upon the shear of 380 kips.

9.3.3.5 Transverse Wall (Wall D)

The strength or limit state design concept is used in the *Provisions*.

[The 2003 *Provisions* adopts by reference the ACI 530-02 provisions for strength design in masonry, and the previous strength design section has been removed. This adoption does not result in significant technical changes, and the references to the corresponding sections in ACI 530 are noted in the following sections.]

9.3.3.5.1 Axial and Flexural Strength General

The walls in this example are all bearing shear walls since they support vertical loads as well as lateral forces.

The demands for the representative design example, Wall D, are presented in this section. The design of the lower and upper portions of Wall D is presented in the next two sections. For both locations, in-plane calculations include:

1. Strength check and
2. Ductility check.

The axial and flexural demands for Wall D, using the load combinations identified in Sec. 9.3.2.3, are presented in Table 9.3-7. In the table, Load Combination 1 represents 1.4D + 1.0 Q_E + 0.5L, and Load Combination 2 represents 0.7D + 1.0 Q_E.

Table 9.3-7 Load Combinations for Wall D

			Load Combination 1		Load Combination 2	
Level	P_D (kips)	P_L (kips)	P_u (kips)	M_u (ft-kips)	P_u (kips)	M_u (ft-kips)
12	37	0	51	500	26	500
11	80	8	117	1,500	56	1,500
10	124	17	182	3,000	87	3,000
9	168	25	247	4,900	117	4,900
8	212	34	313	7,200	148	7,200
7	255	43	379	9,800	179	9,800
6	308	50	457	12,700	216	12,700
5	370	59	548	15,900	259	15,900
4	432	67	639	19,400	303	19,400
3	494	76	730	23,000	346	23,000
2	556	84	821	26,800	389	26,800
1	618	92	912	30,600	433	30,60

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

Strength at the lowest story (where P, V, and M are the greatest) for both the lower wall (Level 1) and the upper wall (Level 7) constructions will be examined. The design for both locations is based on the values for Load Combination 2 in Table 9.3-7.

9.3.3.5.2 Axial and Flexural Strength Lower Levels

Examine the strength of Wall D at Level 1:

9.3.3.5.2.1 Strength Check (Level 1)

$P_{u_{min}}$ = 433 kips + factored weight of half of 1st story wall = 433 + (0.7)(21.9) = 448 kips

M_u = 30,600 ft-kips

For this Seismic Design Category D building, the special reinforced masonry shear walls must have vertical and horizontal reinforcement spaced at no more than 4 ft on center. The minimum in either direction is 0.0007(10.5 in.) = 0.0074 in.^{2}/in. (vertical #5 at 42 in. on center). That will be used as the vertical reinforcement (although some of the subsequent calculations of flexural resistance are based upon a spacing of 48 in. on center); the shear strength demands for horizontal reinforcement will be greater and will satisfy the total amount of 0.0020(10.5 in.) = 0.021 in.^{2}/in.(horizontal #5 at 22 in. on center will suffice).

Try 16 #9 bars in each bulb. Refer to Figure 9.3-4 for the placement of the reinforcement in the bulb. In some of the strength calculations, the #5 bars in the wall will be neglected as a conservative simplification.

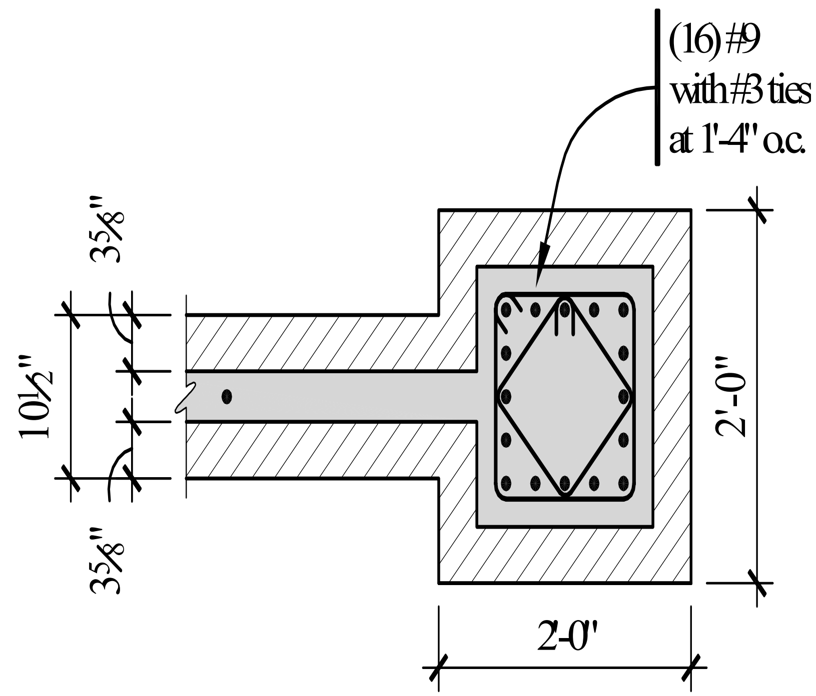


Figure 9.3-4 Bulb reinforcement at lower levels
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

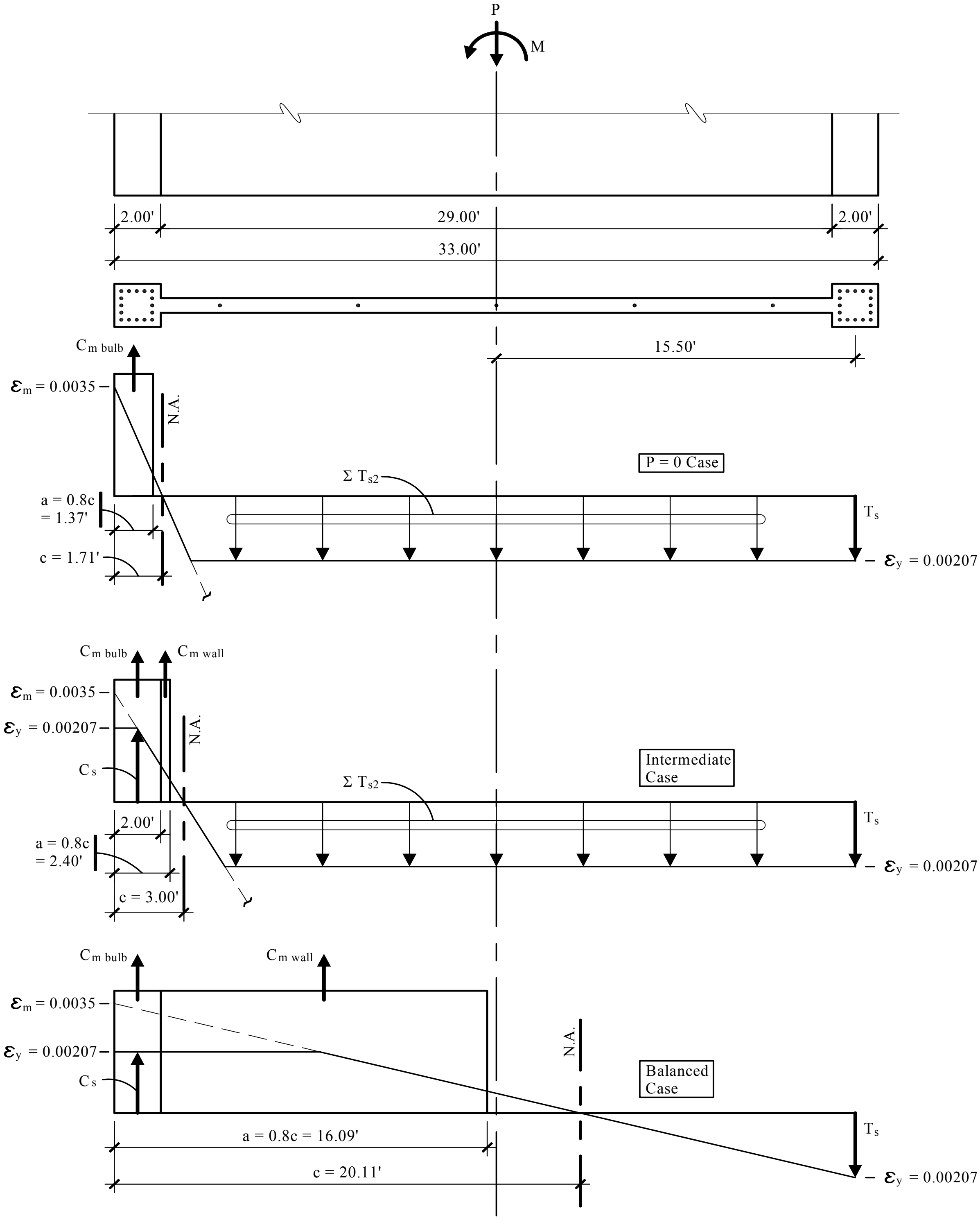


Figure 9.3-5 Strength of Wall D, Level 1 (1.0 ft = 0.3048 m)

For evaluating the capacity of the wall, a ϕP_n - ϕM_n curve will be developed to represent the wall strength envelope. The demands (P_u and M_u determined above) will then be compared to this curve. Several cases will be analyzed and their results used in plotting the ϕP_n - ϕM_n curve. Refer to Figure 9.3-5 for notation and dimensions.

Case 1 (P = 0)

The neutral axis will be within the compression bulb, so assume that only the bars closest to the compression face are effective in compression. (Also recall that the *Provisions* clearly endorses the use of compression reinforcement in strength computations.)

T_{s1} = (16 bars)(1.00 in.^{2})(60 ksi) = 960 kips
T_{s2} = (7 bars)(0.31 in.^{2})(60 ksi) = 130 kips
C_s = (5 bars)(1.00 in.^{2})(60 ksi) = 300 kips

$\Sigma C = \Sigma T + P$
$C_m + C_s = T_{s1} + T_{s2} + P$
C_m = 960 + 130 +0 - 300 = 790 kips

C_m = 790 kips = $\phi f'_m$(24 in.)a = (0.8)(2.5 ksi)(24 in.)a
a = 16.46 in. = 1.37 ft
$c = a/0.8$ = 1.37/0.8 = 1.71 ft = 20.7 in.

Check strain in compression steel

ε_s = 0.0035(20.7 in. - 6 in.)/(20.7 in.) = 0.0025 > yield; assumption OK

$\Sigma M_{cl} = 0$
M_n = (790 kips)(16.5 ft- 1.37 ft/2) + (300 + 960 kips)(15.5 ft) + (130 kips)(0 ft.) = 32,170 ft-kips
ϕM_n = (0.85)(32,170) = 27,340 ft-kips

Case 2 (Intermediate case between P = 0 and balanced case):

Select an intermediate value of c. Let c = 3.0 ft, and determine P_n and M_n for this case.

$a = 0.8c$ = 2.4 ft
$C_{m\ bulb}$ = (0.8)(2.5 ksi)(24 in.)^2 = 1152 kips
$C_{m\ wall}$ = (0.8)(2.5 ksi)(10.5 in.)(0.4 ft. × 12) = 101 kips
C_s = (16 bars)(1.00 in.^{2})(60 ksi) = 960 kips (approximate; not all bars reach full yield)
ΣC = (1152 + 101 + 960) = 2213 kips
ΣT = 960 +130 kips = 1090 kips

$\Sigma F_y = 0$
$P_n = \Sigma C - \Sigma T$ = 2213 - 1090 = 1123 kips
ϕP_n = (0.85)(1123) = 955 kips

$\Sigma M_{cl} = 0$
M_u = (1152 +960 kips)(15.5 ft) + (101 kips)(14.1 ft) + (960 kips)(15.5 ft) = 49,040 ft-kips
ϕM_n = (0.85)(49,040) = 41,680 ft-kips

Case 3 (Balanced case):

$$c = \left[\frac{0.0035}{0.0035 + 0.00207}\right](32.00\text{ ft}) = 20.11\text{ ft}$$

$a = (0.8)c$ = 16.09 ft

Ignore the distributed #5 bars for this case.

$C_{m\ bulb}$ = 1152 kips
$C_{m\ wall}$ = (0.8)(2.5 ksi)(10.5 in.)(14.09 ft. × 12) = 3,550 kips
C_s = 960 kips
ΣC = (1152 + 3550 + 960) = 5,662 kips
T_s = 960 kips

$\Sigma F_y = 0$
$P_n = \Sigma C - \Sigma T$ = 5662 - 960 = 4,702 kips
φP_n = (0.85)(4,702) = 3,997 kips

$\Sigma M_{cl} = 0$
M_u = (1152 +960 kips)(15.5 ft) + (3550 kips)(7.46 ft) + (960 kips)(15.5 ft) = 74,100 ft-kips
ϕM_n = (0.85)(74,100) = 62,980 ft-kips

The actual design strength, ϕM_n, at the level of minimum axial load can be found by interpolation to be 33,840 ft.-kip.

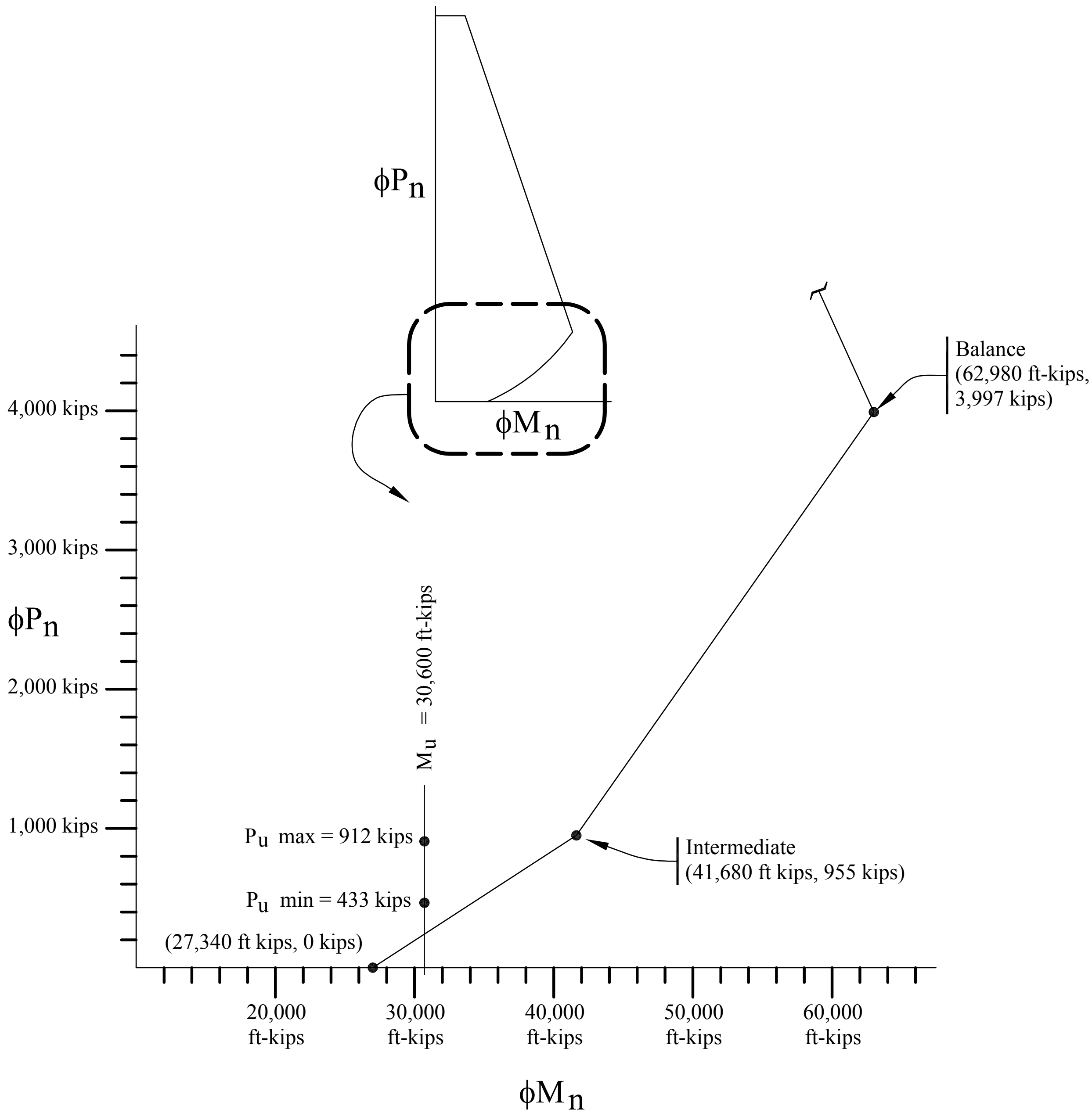


Figure 9.3-6 φP_{11} - φM_{11} Diagram for Level 1 (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

9.3.3.5.2.2 Ductility check (Level 1)

Provisions Sec. 11.6.2.2 [ACI 530, Sec. 3.2.3.5] requires that the critical strain condition correspond to a strain in the extreme tension reinforcement equal to 5 times the strain associated with F_y. Note that this calculation uses unfactored gravity axial loads (*Provisions* 11.6.2.2 [ACI 530, Sec. 3.2.3.5]). See Figure 9.3-7 and the following calculations.

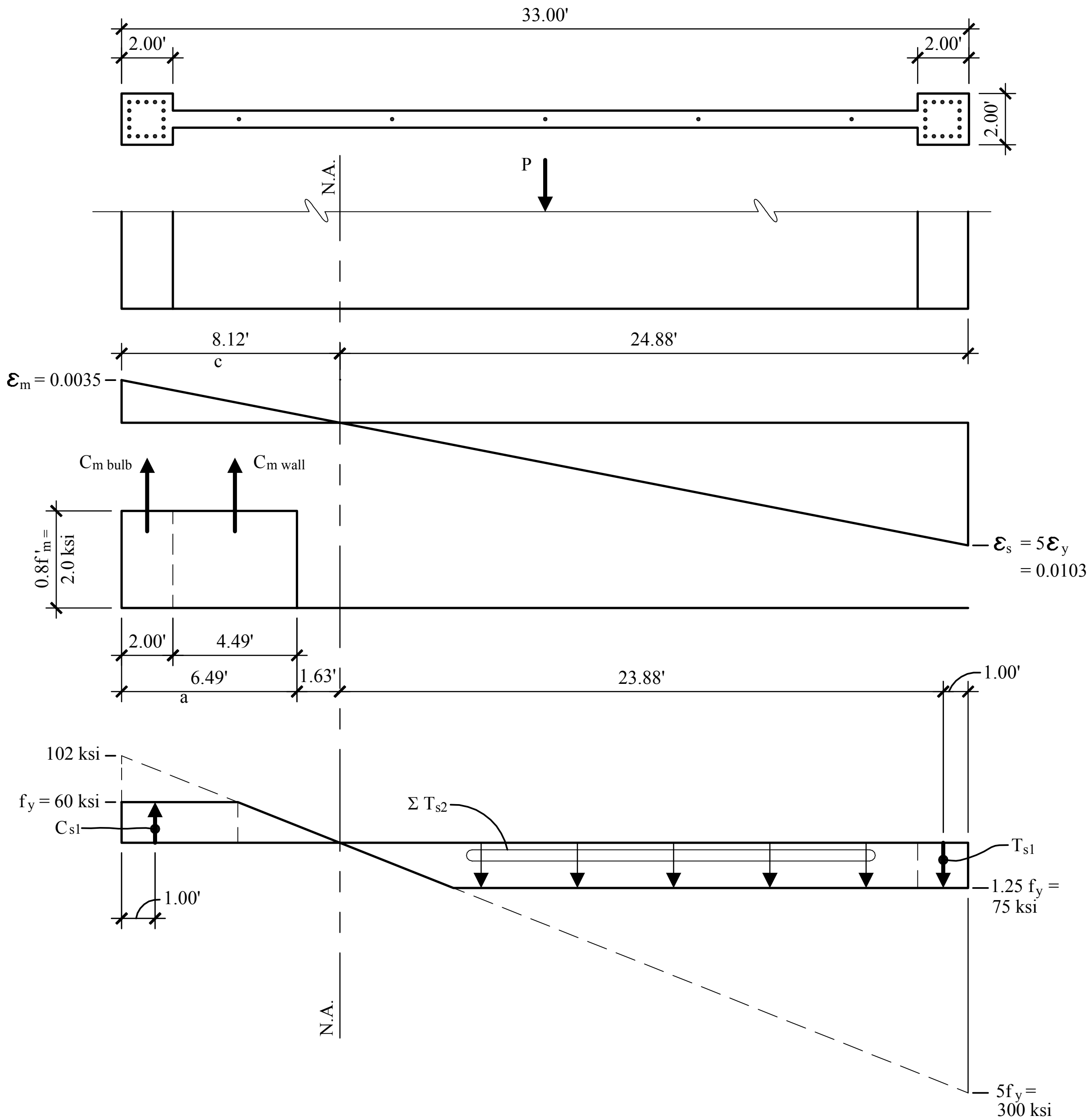


Figure 9.3-7 Ductility check for Wall D, Level 1 (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa)**.**

$$c = \left[\frac{0.0035}{(0.0035+0.0103)}\right](32.00 \text{ ft}) = 8.12 \text{ ft}$$

$a = 0.8\,c = 6.49$ ft

For Level 1, the unfactored loads are:

$P = 618$ kips
$M = 30{,}600$ ft-kips
$C_{m_{bulb}} = 0.8 f'_m A_{bulb} = 1{,}152$ kips
$C_{m_{wall}} = 0.8 f'_m (10.5 \text{ in.})(4.49 \text{ ft} \times 12) = 1131$ kips

The distributed wall rebar that is near the neutral axis is divided between tension and compression, and therefore it will not have much effect on the result of this check, so it will be neglected.

C_{s1} = (60 ksi)(16 × 1.00 in.^{2}) = 960 kips
T_{s1} = (16 × 1.00 in.^{2})(75 ksi) = 1,200 kips
T_{s2} = (4 × 0.31 in.^{2})(75 ksi) = 93 kips

P = 618 kips (unfactored dead load)

$\sum C > \sum P + \sum T$
1152 +1131 + 960 > 618 + 1200 + 93
3,243 kips > 1,818 kips

There is more compression capacity than tension capacity, so a ductile failure condition governs.

[The ductility (maximum reinforcement) requirements in ACI 530 are similar to those in the 2000 *Provisions*. However, the 2003 *Provisions* also modify some of the ACI 530 requirements, including critical strain in extreme tensile reinforcement (4 times yield) and axial force to consider when performing the ductility check (factored loads).]

9.3.3.5.3 Axial and Flexural Strength Upper Levels

Examine the strength of Wall D at Level 7.

$P_{u_{min}}$ = 179 kips + factored weight of ½ of 7th story wall = 179 + (0.7)(11.4) =190 kips
M_u = 9,800 ft-kips

This is a point, however, where some of the reinforcement in the lower wall will be terminated. Although not required by the *Provisions*, most design standards require the longitudinal reinforcement to be extended a distance d beyond the point where it could theoretically be terminated. (The ASD chapter of ACI 530 has such a requirement.) Therefore, the reinforcement at level 6 (7th floor) should be capable of resisting the moment d below. d is approximately three stories for this wall, therefore, take M_u = 19,400 ft-kip (and P = 303 kip) from Level 3.

Try eight #9 in each bulb and vertical #5 bars at 4 ft on center in the wall. Refer to Figure 9.3-8 for the placement of the bulb reinforcement.

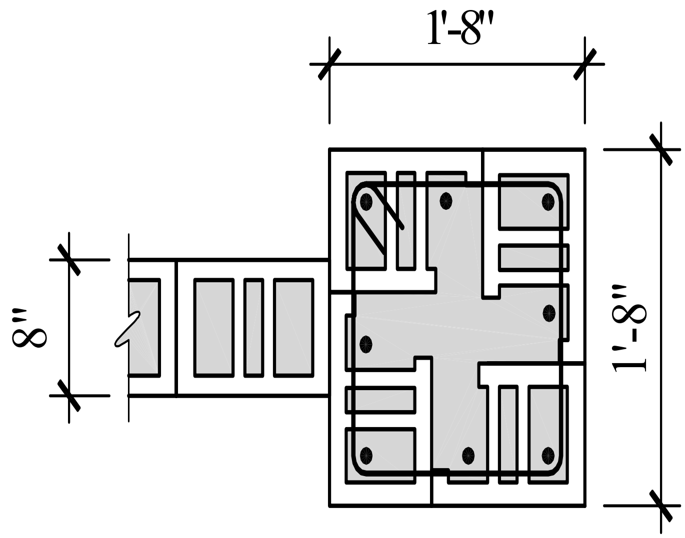


Figure 9.3-8 Bulb reinforcement at upper levels
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

For evaluating the capacity of the wall, a ϕP_n - ϕM_n curve will be developed to represent the wall strength envelope for Level 7. The demands (P_u and M_u determined above) will then be compared to this curve. Several cases will be analyzed and their results used in plotting the ϕP_n - ϕM_n curve. Refer to Figure. 9.3-9 for notation and dimensions.

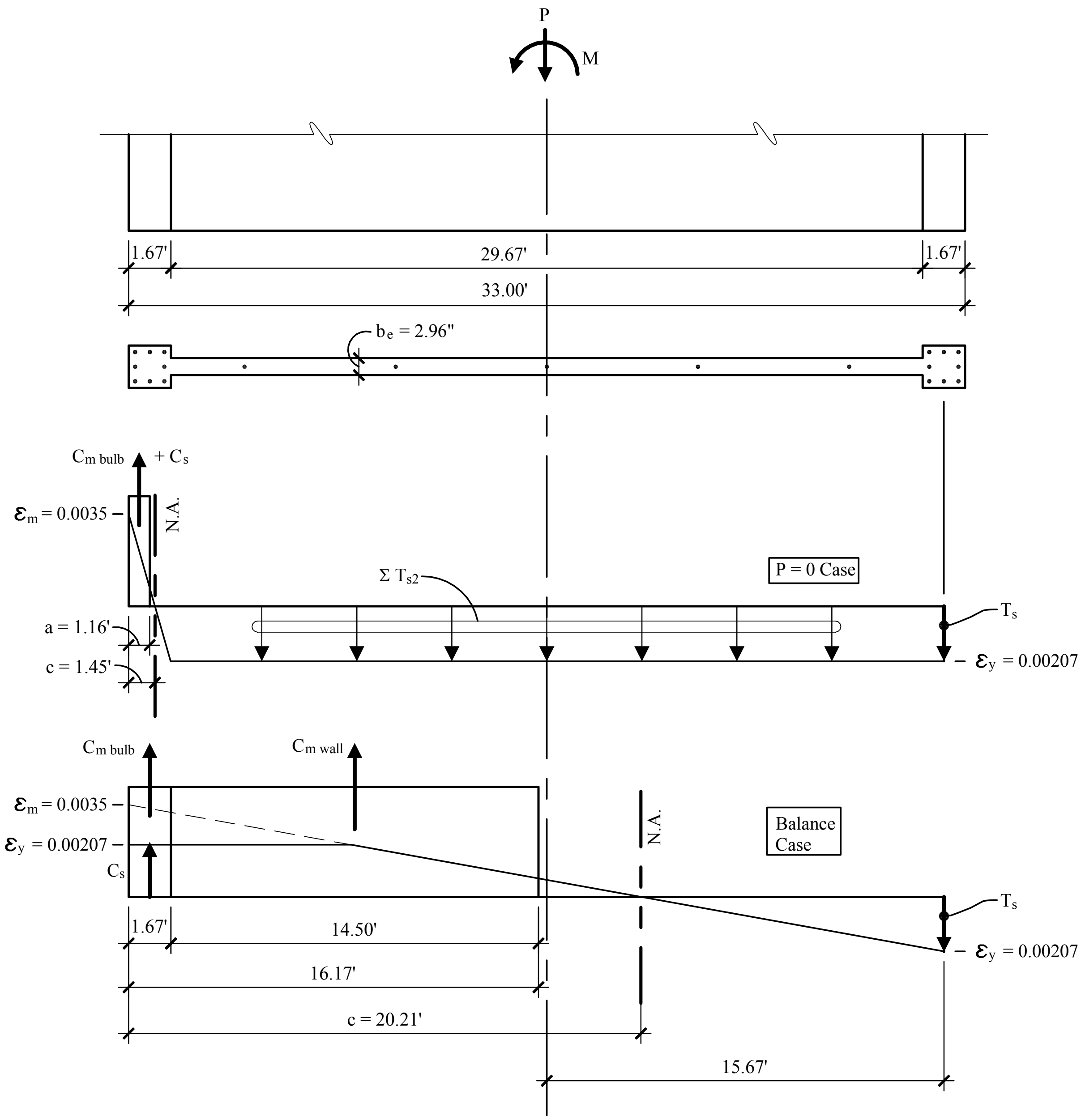


Figure 9.3-9 Strength of Wall D at Level 7 (1.0 ft = 0.3048 m)

Case 1 ($P = 0$)

Tension forces:

T_{s1} = (8 bars)(1.00 in.^{2})(60 ksi) = 480 kips
T_{s2} = (7 bars)(0.31 in.^{2})(60 ksi) = 130 kips

Equilibrium:

$$\Sigma C = \Sigma T + P$$

ΣC = 480 + 130 + 0 = 610 kips

Assume bars closest to compression face yield:

$\Sigma C = C_s + C_m$
C_s = (3 bars)(1.00 in.^{2})(60 ksi) = 180 kips
C_m = 610 - 180 = 430 kips

Locate equivalent stress block and neutral axis:

430 = $\phi f'_m$(20 in.)a = (0.8)(3 ksi)(20 in.)a
a = 8.96 in. = 0.75 ft
$c = a$/0.8 = 0.75/0.8 = 0.93 ft = 11.2 in.

Verify strain in compression steel:

At the outside layer, ε = (0.0035)(7.2 in. / 11.2 in.) = 0.0023 > yield,
At the central layer, ε = (0.0035)(1.2 in. / 11.2 in.) = 0.0004 => f_s = 11 ksi

Resultant moment:

$\Sigma M_{cl} = 0$:
M_n = (430 kips)(16.5 ft- 0.75 ft/2) + (180 kips)(16.5 - 0.33 ft) + (480 kips)(16.5 - 0.83 ft) + (130 kips)(0 ft.) = 17,370 ft-kips
ϕM_n = (0.85)(17,280) = 14,760 ft-kips

<u>Case 2 (Intermediate)</u>

Assume the neutral axis at the face of the bulb, c = 1.67 ft

$a = 0.8c$ = 1.33 ft. = 16 in.
C_m = (2.4 ksi)(16 in.)(20 in.) = 768 kip

For the compression steel, it is necessary to compute the strains:

At the outside layer, ε = (0.0035)(16 in. / 20 in.) = 0.0028 > yield
At the central layer, ε = (0.0035)(10 in. / 20 in.) = 0.00175 => f_s = 50 ksi
At the inside layer, ε = (0.0035)(4 in. / 20 in.) = 0.0007 => f_s = 20 ksi
C_s = (3.0 x 60 ksi + 2.0 x 50 ksi + 3.0 x 20 ksi) = 340 kips

T_{s1} = (8 bars)(1.00 in.^{2})(60 ksi) = 480 kips
T_{s2} = (7 bars)(0.31 in.^{2})(60 ksi) = 130 kips

P_n = 768 + 340 - 480 -130 = 498 kips
ϕP_n = (0.85)(498) = 423 kips

M_n = (768 kips)(16.5 ft- 1.33 ft/2) + (340 + 480 kips)(16.5 - 1.67/2 ft) + (130 kips)(0 ft)
= 25,010 ft-kips
ϕM_n = (0.85)(25,010) = 21,260 ft-kips

At P = 303 kips, ϕM_n = 19,990 ft.-kips by interpolation (exceeds 19,400 ft.-kips, OK)

Case 3 (Balanced Case):

$$c = \left[\frac{0.0035}{0.0035+0.00207}\right](32.17\ \text{ft}) = 20.21\ \text{ft}$$

$a = (0.8)c = 16.17$ ft

b_e = effective width of wall
$b_e = [(2)(1.3125\text{ in.})(29.67\text{ ft x }12) + (8\text{ cells})(15\text{ in.}^2/\text{cell})] = 2.96$ in./ft
$C_{m\ bulb} = 960$ kips
$C_{m\ wall} = (0.8)(3\text{ ksi})(2.96\text{ in.})(14.17\text{ ft x }12) = 101$ kips
$C_s = 480$ kips
$\Sigma C = (960 + 101 + 480) = 1{,}541$ kips
$T_s = 480$ kips, ignoring the distributed #5 bars

$\Sigma F_y = 0$
$P_n = \Sigma C - \Sigma T = 1{,}541 - 480 = 1{,}061$ kips
$\phi P_n = (0.85)(1{,}061) = 902$ kips

$\Sigma M_{cl} = 0$
$M_u = (480 + 960\text{ kips})(15.67\text{ ft}) + (101\text{ kips})(7.42\text{ ft}) + (480\text{ kips})(15.67\text{ ft}) = 30{,}830$ ft-kips
$\phi M_n = (0.85)(30{,}830) = 26{,}210$ ft-kips

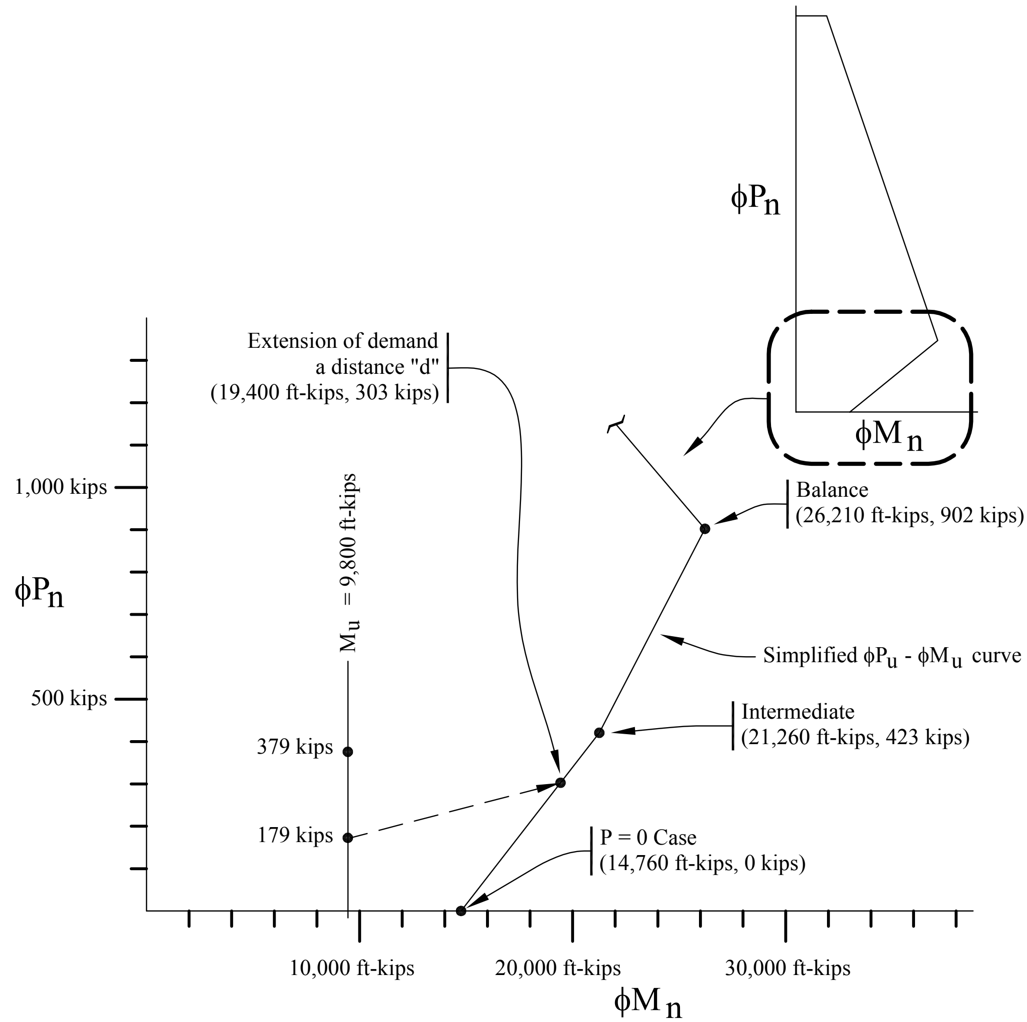


Figure 9.3-10 φP_{11} - φM_{11} Diagram for Level 7 (1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m).

The ductility check is performed similar to that for the wall at Level 1. See Figure 9.3-11 and the following calculations.

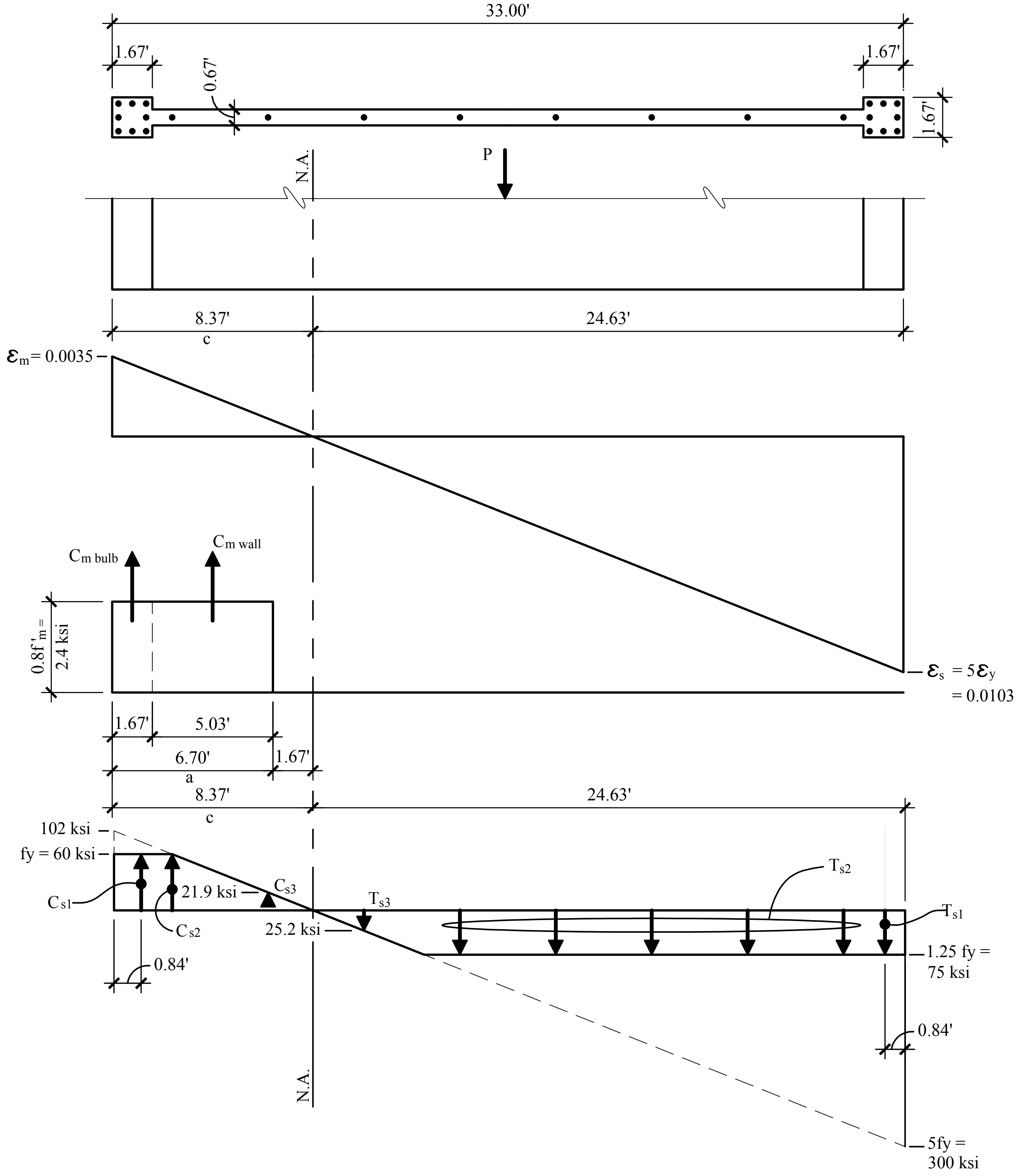


Figure 9.3-11 Ductility check for Wall D, Level 7 (1.0 ft = 0.3048 m, 1.0 ksi = 6.89 MPa)

For Level 7, the unfactored loads are:

P = 255 kips
M = 11,600 ft-kips

$C_{mb} = 0.8f'_m A_b$ = 0.8(3.0 ksi)(400 in.2) = 960 kips
$C_{mw} = 0.8f'_m A_w$ = 0.8(3.0 ksi)[2(1.3125 in.)(5.03 ft.)(12 in/ft) + (1 cell)(25.6 in.^{2}/cell)]
= 442 kips

C_{s1} = (60 ksi)(8× 1.00 in.^{2}) = 480 kips
C_{s2} = (60 ksi)(0.31 in.^{2}) = 19 kips

T_{s1} = (8 × 1.00 in.^{2})(75 ksi) = 600 kips
T_{s2} = (4 × 0.31 in.^{2})(75 ksi) = 116 kips
T_{s3} = (0.31 in.^{2})(25.2 ksi) = 7 kips

P = 255 kips (unfactored dead load)

$\Sigma C > P + \Sigma T$
960 + 442 + 480 + 19 > 255 + 600 + 116 + 7 ?
1901 kips > 978 kips OK

There is more compression capacity than tension capacity, so a ductile failure condition governs.

9.3.3.5.4 Shear Strength

The first step is to determine the net area, A_n, for Wall D. The definition of A_n in the *Provisions,* however, does not explicitly address bulbs or flanges at the ends of walls. Following an analogy with reinforced concrete design, the area is taken as the thickness of the web of the wall times the overall length. In partially grouted walls this is not extended to the point of ignoring the grouted cores because the implication is that grouted cores are intended to be included. (However, if the spacing of bond beams greatly exceeds the spacing of grouted cores, even that assumption might be questionable.)

For Levels 1 through 6:

A_n = (10.5 in.(33 ft × 12))= 4,158 in.^{2}

For Levels 7 through 12 (using 8 x 8 x 12 clay brick units):

A_n = (2)(1.3125 in.)(12 in.)(33 ft) + (7 + 2 × 3 cells)(25.6 in.^{2} /cell with adjacent webs)
= 1,372 in.^{2}

Shear strength is determined as described in Sec. 9.2 using *Provisions* Eq. 11.7.2.1 [ACI 520, Sec. 3.13] and *Provisions* Eq. 11.7.3.1-1 [ACI 530, Eq. 3-18], respectively:

$$V_u \leq \phi V_n$$

$$V_n = V_m + V_s$$

For Levels 1 through 6 using *Provisions* Eq. 11.7.3.1-3 where $\dfrac{M_x}{V_x d} > 1.0$:

$$V_n(\max) = 4\sqrt{f'_m}A_n = (4)(\sqrt{2,500})(4,806) = 961 \text{ kips} \quad (P3)$$

For Levels 7 through 12 where $\dfrac{M_x}{V_x d}$ varies from 0.30 to 1.14:

V_n (max) varies from $5.87\sqrt{f'_m}A_n$ to $4\sqrt{f'_m}A_n$

Therefore, V_n (max) varies from $5.87\sqrt{3{,}000}(1836) = 590 \text{ kips}$ to $4\sqrt{3{,}000}(1836) = 402 \text{ kips}$

depending on the value of $\dfrac{M_x}{V_x d}$. The masonry shear strength is computed as:

$$V_m = \left[4 - 1.75\left(\frac{M}{Vd}\right)\right]A_n\sqrt{f'_m} + 0.25P$$

The shear strength of Wall D, based on the aforementioned formulas and the strength reduction factor of $\phi = 0.8$ for shear from *Provisions* Table 11.5.3 [ACI 530, Sec. 3.1.4.3] , is summarized in Table 9.3-8. V_x and M_x in this table are values from Table 9.3-2 multiplied by 0.148 (the portion of direct and torsional shear assigned to Wall D). P is the dead load of the roof or floor multiplied by the tributary area for Wall D, and d is the wall length, not height (d = 32.67 ft for Wall D).

The demand shear, Vu, is found by amplifying the loads to a level that produces a moment of 125 percent of the nominal flexural strength at the base of the wall. Given the basic flexural demand of 30,600 ft-kips, a design resistance of 33,840 ft-kips, $\varphi = 0.85$, and the 1.25 factor, the overall amplification of design load is 1.63.

Table 9.3-8 Shear Strength for Wall D

Level	V_x/wall (kips)	M_x/wall (ft-kips)	M_x/V_xd	$1.63V_x$	ϕV_n *max* (kips)	ϕV_m (kips)	P (kips)	ϕV_m (kips)	Req'd ϕV_s (kips)
12	49	500	0.309	80	351.2	OK	37	215	-
11	102	1500	0.446	166	329.3	OK	80	210	-
10	149	3000	0.610	243	303.0	OK	124	201	42
9	191	4900	0.777	311	276.2	NG	168	192	119
8	228	7200	0.957	372	247.4	NG	212	182	190
7	260	9800	1.142	424	240.5	NG	255	186	238
6	292	12700	1.318	476	665.3	OK	308	436	40
5	323	15900	1.492	526	665.3	OK	370	448	78
4	347	19400	1.694	566	665.3	OK	432	461	106
3	364	23000	1.915	593	665.3	OK	494	473	120
2	375	26800	2.166	611	665.3	OK	556	485	126
1	380	30600	2.440	619	665.3	OK	618	498	121

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

Note that 1.63 V_x exceeds $V_{n\,max}$ at Levels 7, 8, and 9. The next most economical solution appears to be to add grout to increase A_n and, therefore, both V_m and $V_{n\,max}$. Check Level 7 using solid grout:

A_n = (7.5 in.)(33 ft)(12 in./ft) = 2970 in.^{2}

$V_n(\text{max}) = 4\sqrt{f'_m}A_n = (4)(\sqrt{2{,}500})(2{,}970) = 650$ kips

$\varphi V_n\,max$ = 0.8(650) = 520 kips > 424 kips OK

$$\phi V_m = (0.80)\left[\left(4.0 - 1.75(1.0)\right)2{,}970\sqrt{3000}/1000 + 0.25(255)\right] = 344 \text{ kips}$$

$$\varphi V_s = V_u - \varphi V_m = 424 - 344 = 80 \text{ kips}$$

Check minimum reinforcement for capacity. With vertical #5 at 48 in., a reinforcement ratio of 0.00086 is provided. Thus the horizontal reinforcement must exceed (0.0020 - 0.00086)(7.5 in.)(12 in.) = 0.1025 in.^{2}/ft. With story heights of 10 ft., bond beams at 40 in. on center are convenient, which would require 0.34 in.^{2} Therefore, for 2 - #4 at 40 in. on center:

$$\varphi V_s = 0.80(0.5)(0.4 / 40)\ (60 \text{ ksi})(33 \text{ ft.})(12 \text{ in./ft.}) = 95 \text{ kips} > 80 \text{ kips} \qquad \text{OK}$$

The largest demand for φV_x in the lower levels is 126 kips at level 2. As explained in the design of the lower level walls for flexural and axial loads (Sec. 9.3.3.5.1), horizontal #5 at 22 in. are required to satisfy minimum reinforcement. Given the story height, check for horizontal #5 at 20 in.:

$$\varphi V_s = 0.8(0.5)(0.31 / 20)\ (60 \text{ ksi})(33 \text{ ft.})(12 \text{ in./ft.}) = 147 \text{ kips} > 126 \text{ kips} \qquad \text{OK}$$

In summary, for shear it is necessary to grout the hollow units at story 7 solid, and to add some grout at stories 8 and 9. Horizontal reinforcement is 2 - #4 in bond beams at 40 in. on center in the upper stories and one #5 at 20 in. on center in the grouted cavity of the lower stories.

9.3.4 Deflections

The calculations for deflection involve many variables and assumptions, and it must be recognized that any calculation of deflection is approximate at best.

Deflections are to be calculated and compared with the prescribed limits set forth by *Provisions* Table 5.2.8 [Table 4.5-1]. Deformation requirements for masonry structures are discussed in *Provisions* Sec. 11.5.4.

The following procedure will be used for calculating deflections:

1. Determine if the wall at each story will crack by comparing M_x (see Table 9.3-6) to M_{cr} where
   $$M_{cr} = S\left(f_r + P_{u_{min}}/A\right)$$
2. If $M_{cr} < M_x$, then use cracked moment of inertia and *Provisions* Eq. 11.5.4.3.
3. If $M_{cr} > M_x$, then use $I_n = I_g$ for moment of inertia of wall.
4. Compute deflection for each level.
5. $\delta_{max} = \sum$ story drift

[The specific procedures for computing deflection of shear walls have been removed from the 2003 *Provisions*. ACI 530 does not contain the corresponding provisions in the text, however, the commentary contains a discussion and equations that are similar to the procedures in the 2000 *Provisions*. Based on ACI 530 Sec. 1.13.3.2, the maximum drift for all masonry structures is 0.007 times the story height. Thus, there appears to be a conflict between ACI 530 and 2003 *Provisions* Table 4.5-1.]

For the upper levels (the additional grout required for shear strength is not considered here):

b_e = effective masonry wall width

$b_e = [(2 \times 1.3125 \text{ in.})(356) + (7 \text{ cells})(15 \text{ in.}^2/\text{cell})]/(356) = 2.92 \text{ in.}$

$A = A_{wall} + 2A_{bulb} = (2.92 \text{ in.})(356 \text{ in.}) + (2)(400 \text{ in.}^2) = 1{,}840 \text{ in.}^2$

[Note that by adopting ACI 530 in the 2003 *Provisions*, $E_m = 900f'_m$ per ACI 530 Sec. 1.8.2.2.1]

Per *Provisions* Eq. 11.3.10.2 [ACI 530, 1.8.2.2]:

$E = 750\,f'_m$ = 2,250 ksi (n = 12.89)

$I_g = I_{wall} + I_{bulb}$

$$I_g = \frac{(2.96)(356)^3}{12} + (2\text{ bulbs})(20\times 20)\left(\frac{376}{2}\right)^2 = 39.4\times 10^6\text{ in.}^4$$

$S = I_g/c = 39.4 \times 10^6 /(198) = 199{,}000\text{ in.}^3$
f_r = 0.250 ksi
$P_{u_{min}}$ = 1.00D (see Table 9.3-6.)

For the lower levels:

$A = A_{wall} + 2A_{bulb}$ = (10.5 in.)(348 in.) + (2)(576 in.^{2}) = 4,806 in.^{2}
$E = 750\,f'_m$ = 1,875 ksi (n = 15.47)
$I = I_{wall} + I_{bulb}$

$$I_g = \frac{(10.5)(348)^3}{12} + (2\text{ bulbs})(24\times 24)\left(\frac{372}{2}\right)^2 = 76.7\times 10^6\text{ in.}^4$$

$S = I_g / c = 76.7 \times 10^6 /(198) = 387{,}000\text{ in.}^3$
f_r = 0.250 ksi
$P_{u_{min}}$ = 1.00D (see Table 9.3-6.)

Table 9.3-9 provides a summary of these calculations.

Table 9.3-9 Cracked Wall Determination

Level	$P_{u_{min}}$ (kips)	M_{cr} (ft-kips)	M_x (ft-kips)	Status
12	37	7,620	500	uncracked
11	80	8,820	1,500	uncracked
10	124	8,950	3,000	uncracked
9	168	9,620	4,900	uncracked
8	212	10,300	7,200	uncracked
7	255	11,000	9,800	uncracked
6	308	15,400	12,700	uncracked
5	370	16,000	15,900	uncracked
4	432	16,700	19,400	cracked
3	494	17,300	23,000	cracked
2	556	18,000	26,800	cracked
1	618	18,600	30,600	cracked

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

For the uncracked walls at the upper levels:

$I_n = I_g = 39.4 \times 10^6$ in.^{4}

For the uncracked walls at the lower levels:

$I_n = I_g = 76.7 \times 10^6$ in.^{4}

For the cracked walls at the lower levels, the determination of I_{cr} will be for the load combination of 1.0D + 0.5L. The 0.5L represents an average condition of live load. Making reference to Figure 9.3-6, it can be observed that at this level of P_u, the point on the ϕP_n - ϕM_n curve is near the "intermediate point" previously determined. This is where c = 3.0 ft. (The actual c dimension will be very close to 3.0 ft). For this case, and referring to Figure 9.3-5, the cracked moment of inertia is:

$$I_{cr} = I_{bulb} + I_{wall} + I_{nAs}$$
$$= [24^4 + (24 \times 24)(24)^2] + [10.5 \times 12^3/3] + [(15.47 \times 16 \text{ in.}^2)(29 \text{ ft} \times 12)^2]$$
$$= 30.3 \times 10^6 \text{ in}^4.$$

Note that 98.9 percent of the value comes from one term: the reinforcement in the tension bulb. If the distributed #5 bars is added to this computation, the value becomes 31.5×10^6 in.^{4}

With the other masonry examples, the interpolation between gross and cracked section properties was used. The application of that is less clear here, where the properties step at midheight, so two analyses are performed. First, each story is considered to be cracked or uncracked. Second is the author's interpretation of the effective moment of inertia equation as:

For all the cracked walls (*Provisions* Eq. 11.5.4.3 [ACI 530, Commentary Sec. 3.1.5.3]):

$$I_{eff} = I_n \left(\frac{M_{cr}}{M_a} \right)^3 + I_{cr} \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] \le I_n$$

$$I_{eff} = \left(76.7 \times 10^6\right) \left(\frac{18,600}{30,600} \right)^3 + \left(30.6 \times 10^6\right) \left[1 - \left(\frac{18,600}{30,600} \right)^3 \right] = 41.0 \times 10^6 \text{ in.}^4$$

The entire 12-story Wall D will be treated as a stepped, vertical masonry cantilever shear wall. For the lower step (Levels 1-6), $I_{eff} = 41.0 \times 10^6$ in.^{4} Even though the upper walls are uncracked, I_{eff} of the upper step (Levels 7-12), will be I_n reduced in the same proportion as the lower levels:

$$I_{eff} = (39.4 \times 10^6) \left(\frac{41.0 \times 10^6}{76.7 \times 10^6} \right) = 21.1 \times 10^6 \qquad \text{(upper levels)}$$

Both the deflections and the fundamental period can now be found. Two RISA 2D analyses were run, and the deflections shown in Table 9.3-10 were obtained. The deflection from the RISA 2D analysis at each level is multiplied by C_d (= 3.5) to determine the inelastic deflection at each level. From these, the story drift, Δ, at each level can be found.

The periods shown in the table validate the period of T = 0.75 sec previously used to determine the base shear in Sec. 9.3.3.2.

Table 9.3-10 Deflections for ELF Analysis (inches)

	Using gross and cracked properties,	Using effective moment of inertia

	story by story (T = 0.798 sec)				(T = 0.755 sec)			
Level	Elastic	Total	Drift	Ratio	Elastic	Total	Drift	Ratio
12	3.40	11.90			3.14	10.99		
11	3.04	10.65	1.25		2.77	9.71	1.28	
10	2.68	9.38	1.27	1.01	2.40	8.41	**1.30**	1.01
9	2.32	8.10	1.27	1.00	2.04	7.13	1.28	0.99
8	1.95	6.84	1.26	0.99	1.68	5.87	1.26	0.98
7	1.60	5.60	1.24	0.98	1.34	4.68	1.19	0.95
6	1.26	4.41	1.19	0.96	1.02	3.58	1.10	0.92
5	0.95	3.34	1.07	0.90	0.76	2.65	0.93	0.85
4	0.66	2.31	1.03	0.96	0.52	1.82	0.83	0.90
3	0.40	1.40	0.91	0.88	0.32	1.11	0.71	0.85
2	0.20	0.68	0.71	0.78	0.16	0.55	0.56	0.79
1	0.06	0.20	0.48	0.67	0.05	0.17	0.38	0.68
0	0	0.00	0.20	0.42	0	0.00	0.17	0.44

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

The two methods give comparable results. The maximum building deflection is compared to the maximum deflection permitted by *Provisions* Sec. 11.5.4.1.1 as follows:

$C_d\delta_{max}$ = 11.90 in. < 14.4 in. = $0.1h_n$ OK

The maximum story drift occurs at Story 11 and is compared to the maximum story drift permitted by *Provisions* Table 5.2.8 [Table 4.5-1] as follows:

Δ = 1.30 in. > 1.20 in. = $0.01h_{sx}$ NG

Although this indicates a failure to satisfy the *Provisions*, in the author's opinion the drift is satisfactory for two reasons. First the MRS analysis shows smaller drifts (Table 9.3-11) that are within the criteria. On a more fundamental level, however, the authors believe the basic check for drift of a masonry wall is performed according to *Provisions* Sec. 11.5.4.1.1, which applies only to the total displacement at the top of the wall, and that the story drift for any particular story is more properly related to the values for non-masonry buildings. That limit is 0.020 h_{sx}, or 2.4 in. per story. For a building with a torsional irregularity, *Provisions* Sec. 5.4.6.1 [Sec. 4.5.1] requires that the story drift be checked at the plan location with the largest drift, which would be a corner for this building. That limit is satisfied for this building, both by ELF and MRS analyses.

Table 9.3-11 Displacements from Modal Analysis, inches

Level	At corner of floor plate with maximum displacements. Story drift would be pertinent, although not at 0.010			At wall with maximum in-plane displacement. Roof limit for masonry would be pertinent.		
	Elastic	Total	Approx. Drift	Elastic	Total	Approx. Drift
12	2.48	**8.67**		1.91	**6.70**	
11	2.20	7.69	0.98	1.69	5.90	0.79
10	1.91	6.70	0.99	1.48	5.17	0.74
9	1.63	5.70	**1.00**	1.26	4.40	**0.77**
8	1.35	4.72	0.98	1.04	3.64	0.76
7	1.08	3.77	0.95	0.83	2.91	0.73
6	0.83	2.90	0.88	0.64	2.23	0.68
5	0.62	2.16	0.74	0.48	1.67	0.57
4	0.43	1.50	0.66	0.33	1.16	0.51
3	0.27	0.93	0.57	0.21	0.72	0.44
2	0.14	0.48	0.46	0.11	0.37	0.35
1	0.05	0.16	0.32	0.04	0.12	0.25
0	0.00	0.00	0.16	0.00	0.00	0.12

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

The drifts in Table 9.3-11 are not the true modal drifts. The values are computed from the modal sum maximum displacements, rather than being a modal sum of drifts in each mode. The values in the table are less than the true value.

Both tables also confirm that the change in stiffness at midheight does not produce a stiffness irregularity. *Provisions* Sec. 5.2.3.3, Exception 1 [Sec. 4.3.2.3, Exception 1], clarifies that if the drift in a story never exceeds 130 percent of the drift in the story above, then there is no vertical stiffness irregularity. Note that the inverse does not apply; even though the drift in Story 2 is more than double that in Story 1, it does not constitute a stiffness irregularity.

9.3.5 Out-of-Plane Forces

Provisions Sec 5.2.6.2.7 [Sec. 4.6.1.3] states that the bearing walls shall be designed for out-of-plane loads equal to:

$$w = 0.40\, S_{DS}\, W_c \geq 0.1 W_c$$
$$w = (0.40)(1.00)(114 \text{ psf}) = 45.6 \text{ psf} \geq 0.1 W_c$$

Therefore, w = 45.6 psf. Out-of-plane bending, using the strength design method for masonry, for a load of 45.6 psf acting on a 10 ft story height is approximated as 456 ft.-lb. per linear ft of wall. This compares to a computed strength of the wall of 1,600 in.-lb per linear foot of wall, considering only the #5 bars at 4 ft on center. Thus the wall is loaded to 28.5 percent of its capacity in flexure in the out-of-plane direction. The upper wall has the same reinforcement, about 42 percent of the load and about 71 percent of the thickness. Therefore, it will be loaded to a smaller fraction of its capacity. (Refer to Example 9.1 for a more detailed discussion of strength design of masonry walls, including the P-delta effect.)

9.3.6 Orthogonal Effects

In accordance with *Provisions* Sec. 5.2.5.2.2 [Sec. 4.4.2.3], orthogonal interaction effects must be considered for buildings in Seismic Design Category D when the ELF procedure is used. Any out-of-plane effect on the heavily reinforced bulbs is negligible compared to the in-plane effect, so orthogonal effects on the bulbs need not be considered further. Considering only the #5 bars and the load combination of 100 percent of in-plane load plus 30 percent of the out-of-plane load, yields a result that 0.3(0.285), or 8.6 percent of the capacity of the #5 bars is not available for in-plane resistance. Given that the #5 bars contribute about 12 percent to the tension resistance (130 kips, vs 960 kips for the bulb reinforcement), the overall effect is a change of about 1 percent in in-plane resistance, which is negligible.

This completes the design of the transverse Wall D.

9.3.7 Wall Anchorage

The anchorage for the bearing walls must be designed for the force, F_p, determined in accordance with *Provisions* Sec. 5.2.6.2.7 [Sec. 4.6.1.3] as:

$F_p = 0.4S_{DS}W_c$ = (0.4)(1.00)(10 ft)(114 psf) = 456 lb/ft
Minimum force = $0.10W_c$ = (0.10)(10 ft)(114 psf) = 114 lb/ft

Provisions Sec. 5.2.6.3.2 [Sec. 4.6.2.1] references *Provisions* Sec. 6.1.3 [Sec. 6.2.2] for anchorage of walls where diaphragms are not flexible. For the lower wall:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2z/h\right) = \frac{0.4(1.0)(1.0)(10\,\text{ft.}\times 114\,\text{psf})}{2.5/1.0}\left(1+2(0.5)\right) = 364 \text{ lb./ft.}$$

Therefore, design for 456 lb/ft. For a 2 ft-6 in. joist spacing, the anchorage force at each joist is F_p = 1,140 lb.

Refer to Figure 9.3-12 for the connection detail. A 3/16-in. fillet, weld 2 in. long on each side of the joist seat to its bearing plate will be more than sufficient. Two1/2 in.-diameter headed anchor studs on the bottom of the bearing plate also will be more than sufficient to transfer 4,560 lb into the wall.

9.3.8 Diaphragm Strength

See Example 7.1 for a more detailed discussion on the design of horizontal diaphragms.

To compute the story force associated with the diaphragm on each level, use *Provisions* Eq. 5.2.6.4-4 [Eq. 4.6-2]:

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

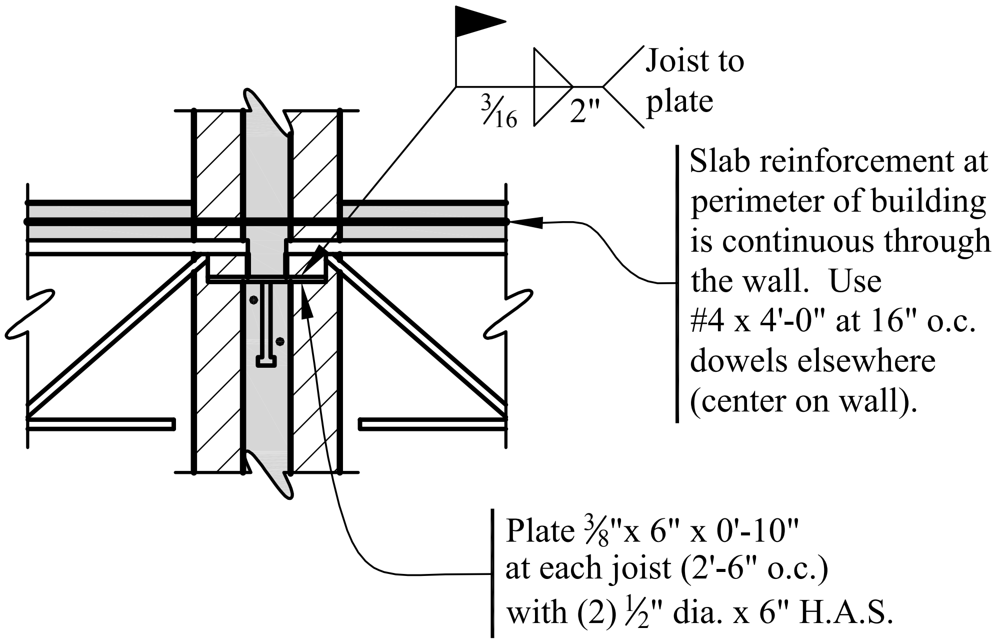


Figure 9.3-12 Floor anchorage to wall (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The results are shown in Table 9.3-12. Note that w_i is approximately the same as w_{px} for this case, the only difference being the weight of the walls perpendicular to the force direction, so the w_i values were used for both.

Table 9.3-12 Diaphragm Seismic Forces

Level	w_i (kips)	F_i (kips)	F_{px} (kips)
12	768	329	329
11	917	357	373
10	917	320	355
9	917	284	336
8	917	249	318
7	917	214	301
6	1109	218	338
5	1300	208	365
4	1300	162	336
3	1300	117	309
2	1300	74	282
1	1300	34	258

1.0 kip = 4.45 kN

The maximum story force is 373 kips. Therefore, use 373 kips/152 ft = 2.45 kips/ft in the transverse direction. The shear in the diaphragm is shown in Figure 9.3-13b. The reaction, *R*, at each wall pair is 373/4 = 93.25 kips. The diaphragm force at each wall pair is 93.25 kips/(2 × 33 ft) = 1.41 kips/ft.

The maximum diaphragm shear stress is $v = V/td$ = 1410 plf/(2.5 in.)(12 in.) = 47 psi. This compares to an allowable shear of

$$\phi v_c = (0.85)(2)\sqrt{f_c'} = (0.85)(2)\sqrt{3{,}000} = 93 \text{ psi}$$

for 3,000 psi concrete. Thus, no shear reinforcing is necessary. Provide ρ = 0.0018 as minimum reinforcement, so A_s = 0.054 in.^{2}/ft. Use WWF 6 × 6-2.9/2.9, which has A_s = 0.058 in.^{2}/ft.

The moment in the diaphragm is shown in Figure 9.3-13c. The maximum moment is 2,460 ft-kips.

Perimeter reinforcement in the diaphragm is determined from:

$T = M/d$ = (2,460 ft-kips)/(72 ft) = 34.1 kips

$A_s = T/\phi F_y$ = 34.1 kips / (0.85)(60 ksi) = 0.67 in.^{2}

Boundary elements of diaphragms may also serve as collectors. The collector force is not usually the same as the chord force. *Provisions* Sec. 5.2.6.4.1 [Sec. 4.6.2.2] requires that collector forces be amplified by Ω_0. Collector elements are required in this diaphragm for the longitudinal direction. A similar design problem is illustrated in Chapter 7 of this volume. Where reinforcing steel withing a topping slab is used for chords or collectors, ACI 318, Sec. 21.9.8 (2002 edition) imposes special spacing and cover requirements. Given the thin slab in this building, the chord reinforcement will have to be limited to bars with couplers at the splices or a thickened edge will be required.

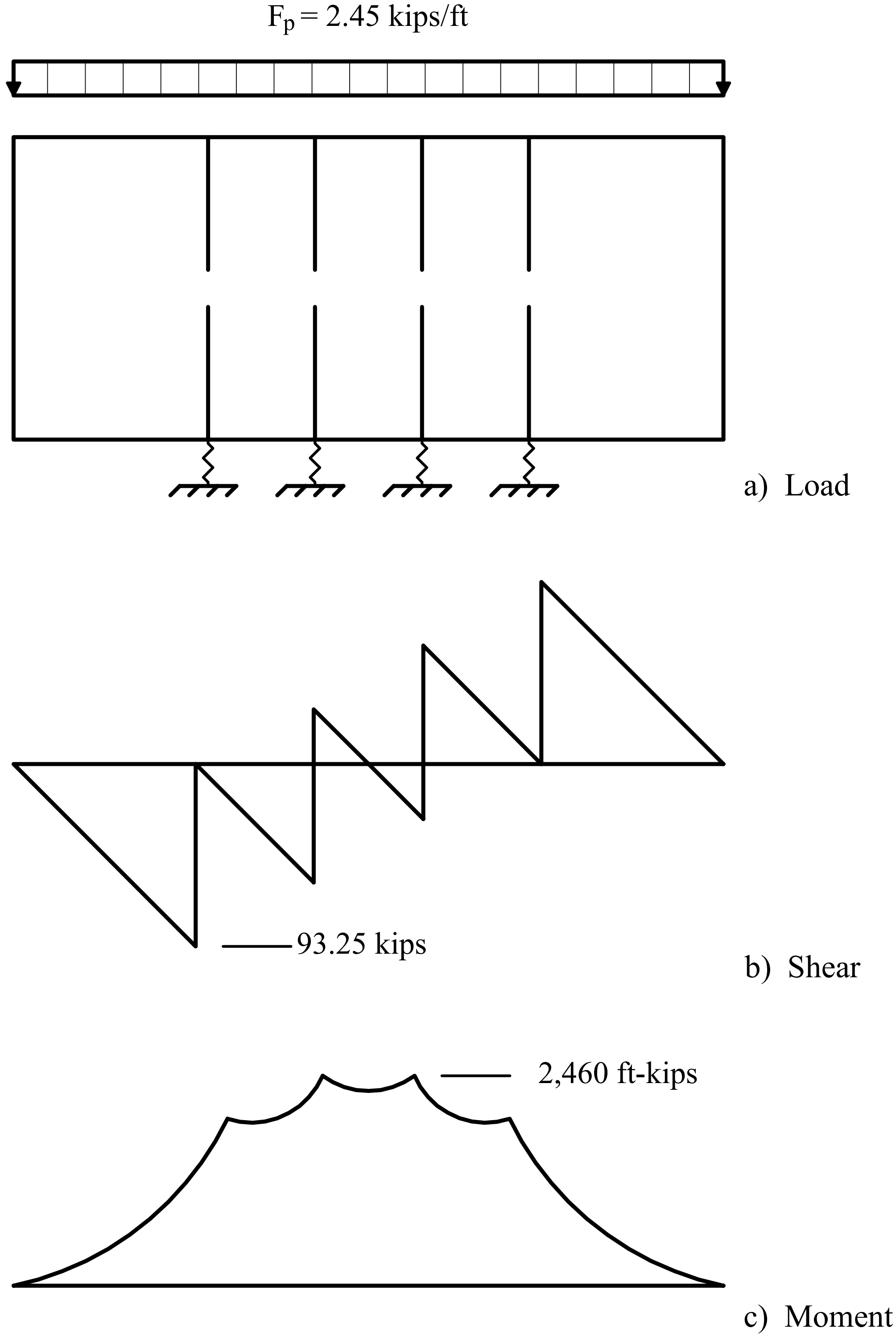


Figure 9.3-13.

Figure 9.3-13 Shears and moments for diaphragm
(1.0 kip/ft = 14.6 kN/m, 1.0 kip = 4.45 kN, 1.0 kip-ft = 1.36kN-m)