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

1. Requires the system (even if the precast carries only gravity loads) to satisfy one of the following two sets of provisions:
   a. Resist amplified chord forces in diaphragms and, if moment-resisting frames are used as the vertical system, provide a minimum degree of redundancy measured as a fraction of available bays, or
   b. Provide a moment-resisting connection at all beam-to-column joints with positive lateral support for columns and with special considerations for bearing lengths.

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

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

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

4. Defines that monolithic emulation may be achieved through the use of either:
   a. Ductile connections, in which the nonlinear response occurs at a connection between a precast unit and another structural element, precast or not, or
   b. Strong connections, in which the nonlinear response occurs in reinforced concrete sections (generally precast) away from connections that are strong enough to avoid yield even as the forces at the nonlinear response location increase with strain hardening.

5. Defines both ductile and strong connections can be either:
   a. Wet connections where reinforcement is spliced with mechanical couplers, welds, or lap splices (observing the restrictions regarding the location of splices given for monolithic concrete) and the connection is completed with grout, or
b. Dry connections, which are defined as any connection that is not a wet connection.

6. Requires that ductile connections be either:

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

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

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

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

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

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

ACI 318-99  American Concrete Institute. 1999. *Building Code Requirements and Commentary for Structural Concrete.*

ACI 318-02  American Concrete Institute. 2002. *Building Code Requirements and Commentary for Structural Concrete.*


SEAA Hollow Core  Structural Engineers Association of Arizona, Central Chapter. *Design and Detailing of Untopped Hollow-Core Slab Systems for Diaphragm Shear.*

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

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

The most significant change related to precast concrete in the 2003 Provisions is that precast shear wall systems are now recognized separately from cast-in-place systems. The 2003 Provisions recognizes ordinary and intermediate precast concrete shear walls. The design of ordinary precast shear walls is based on ACI 318-02 excluding Chapter 21 and the design of intermediate shear walls is based on ACI 318-02 Sec. 21.13 (with limited modifications in Chapter 9 of the 2003 Provisions). The 2003 Provisions does not distinguish between precast and cast-in-place concrete for special shear walls. Special precast shear walls either need to satisfy the design requirements for special cast-in-place concrete shear walls.
(ACI 318-02 Sec. 21.7) or most be substantiated using experimental evidence and analysis (2003 Provisions Sec. 9.2.2.4 and 9.6). Many of the design provisions for precast shear walls in the 2000 Provisions have been removed, and the requirements in ACI 318-02 are in some ways less specific. Where this occurs, the 2000 Provisions references in this chapter are simply annotated as “[not applicable in the 2003 Provisions].” Commentary on how the specific design provision was incorporated into ACI 318-02 is included where appropriate.

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

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

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

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

7.1.1.1 General Design Requirements

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

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

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

7.1.1.2 General In-Plane Seismic Design Forces for Untopped Diaphragms

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

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

\[ \text{Note that this equation is incorrectly numbered as 5.2.5.4 in the first printing of the 2000 Provisions.} \]
Categories D and higher; it is intended in the *Provisions* Appendix to Chapter 9 that the same equation be used for untopped diaphragms in Seismic Design Categories B and C.

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

\( \Omega_0 \) is the overstrength factor (*Provisions* Table 5.2.2 [4.3-1])

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

\( w_{ps} \) is the weight tributary to the diaphragm at Level \( x \)

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

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

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

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

\[
F'_{ps} = 1.25KF_{ps}^* \quad \text{where}
\]

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

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

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

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

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

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

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Birmingham 1</th>
<th>New York City</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$\Omega_o$</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>$C_s$</td>
<td>0.12</td>
<td>0.156</td>
</tr>
<tr>
<td>$w_i$ (roof)</td>
<td>861 kips</td>
<td>869 kips</td>
</tr>
<tr>
<td>$w_i$ (floor)</td>
<td>963 kips</td>
<td>978 kips</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>0.24</td>
<td>0.39</td>
</tr>
<tr>
<td>$I$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

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

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

The factor $K$ is large primarily due to consideration of axial load. The strength for design is controlled by minimum axial load, whereas $K$ is maximum for the maximum axial load, which includes some live load and a vertical acceleration on dead load.
Table 7.1-2 Shear Wall Overstrength

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

7.1.1.3 Diaphragm Forces for Birmingham Building 1

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

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

Roof

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

Floors

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

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

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

Calculations for $F_{px}$ are provided in Table 7.1-3.
Table 7.1-3  Birmingham 1 $F_{px}$ Calculations

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

1.0 kip = 4.45 kN.

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

The minimum value of $F_{px} = 0.2SDS_i w_{px}$

$= 0.2(0.24)1.0(807 \text{ kips}) = 38.7 \text{ kips (at the roof)}$

$= 0.2(0.24)1.0(855 \text{ kips}) = 41.0 \text{ kips (at floors)}$

The maximum value of $F_{px} = 0.4SDS_i w_{px}$

$= 2(38.7 \text{ kips}) = 77.5 \text{ kips (at the roof)}$

$= 2(41.0 \text{ kips}) = 82.1 \text{ kips (at floors)}$

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

$$C_s w_{px} = 0.12(807 \text{ kips}) = 96.8 \text{ kips (at the roof)}$$

$$C_s w_{px} = 0.12(855 \text{ kips}) = 103 \text{ kips (at the floors)}$$

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

$$F'_{px} = \rho \Omega_s C_s w_{px} = 1.0(2.5)(96.8 \text{ kips}) = 242 \text{ kips (at the roof)}$$

$$F'_{px} = \rho \Omega_s C_s w_{px} = 1.0(2.5)(103 \text{ kips}) = 256 \text{ kips (at the floors)}$$

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

$$F'_{px} = 1.25KF_{px} = 1.25(1.85)164 \text{ kips} = 379 \text{ kips (at the roof)}$$

$$F'_{px} = 1.25KF_{px} = 1.25(1.85)155 \text{ kips} = 358 \text{ kips (at the floors)}$$

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

7.1.1.4 Diaphragm Forces for New York Building

The weight tributary to the roof and floor diaphragms ($w_{px}$) is the total story weight ($w_i$) at Level $i$ minus the weight of the walls parallel to the force.
Compute diaphragm weight \( w_{px} \) for the roof and floor as follows:

**Roof**

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

**Floors**

- Total weight = 978 kips
- Walls parallel to force = (48 psf)(277 ft)(8.67 ft) = 863 kips

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

\[
C_s w_{px} = 0.156(812 \text{ kips}) = 127 \text{ kips (at the roof)}
\]

\[
C_s w_{px} = 0.156(863 \text{ kips}) = 135 \text{ kips (at the floors)}
\]

The diaphragm seismic design forces are:

\[
F'_{px} = \rho Q_0 C_s w_{px} = 1.0(2.5)(127 \text{ kips}) = 318 \text{ kips (at the roof)}
\]

\[
F''_{px} = \rho Q_0 C_s w_{px} = 1.0(2.5)(135 \text{ kips}) = 337 \text{ kips (at the floors)}
\]

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

\[
1.25KF_{px}^* = 1.25(1.58)214 \text{ kips} = 423 \text{ kips (at the roof)}
\]

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

**7.1.1.5 Static Analysis of Diaphragms**

The balance of this example will use the controlling diaphragm seismic design force of 423 kips for the New York building. In the transverse direction, the loads will be distributed as shown in Figure 7.1-1.
Assuming the four shear walls have the same stiffness and ignoring torsion, the diaphragm reactions at the transverse shear walls ($F$ as shown in Figure 7.1-1) are computed as follows:

$$F = \frac{423 \text{ kips}}{4} = 105.8 \text{ kips}$$

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

- $W_1 = (67 \text{ psf}(72 \text{ ft}) + 48 \text{ psf}(8.67 \text{ ft})4)(423 \text{ kips} / 863 \text{ kips}) = 3,180 \text{ lb/ft}$
- $W_2 = 67 \text{ psf}(72 \text{ ft})(423 \text{ kips} / 863 \text{ kips}) = 2,364 \text{ lb/ft}$

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

- Joint 1 – maximum transverse shear parallel to the panels at panel-to-panel joints
- Joint 2 – maximum transverse shear parallel to the panels at the panel-to-wall joint
- Joint 3 – maximum transverse moment and chord force
- Joint 4 – maximum longitudinal shear perpendicular to the panels at the panel-to-wall connection (exterior longitudinal walls) and anchorage of exterior masonry wall to the diaphragm for out-of-plane forces
- Joint 5 – collector element and shear for the interior longitudinal walls
Provisions Sec. 9.1.1.4 [not applicable in 2003 Provisions] defines a chord amplification factor for diaphragms in structures having precast gravity-load systems. [The chord amplification factor has been dropped in the 2003 Provisions and does not occur in ASC 318-02. See the initial section of this chapter for additional discussion on changes for the 2003 Provisions.] This amplification factor appears to apply to buildings with vertical seismic-force-resisting members constructed of precast or monolithic concrete. Because these masonry wall buildings are similar to buildings with concrete walls, this amplification factor has been included in calculating the chord forces. The amplification factor is:

\[
\frac{1 + 0.4 \left( \frac{L_{\text{eff}}}{b_d} \right)^2}{12h_s} \geq 1.0
\]

where

- \( L_{\text{eff}} \) = length of the diaphragm between inflection points. Since the diaphragms have no inflection points, twice the length of the 40-ft-long cantilevers is used for \( L_{\text{eff}} = 80 \) ft
- \( h_s \) = story height = 8.67 ft
- \( b_d \) = diaphragm width = 72 ft

The amplification factor = \( \left( 72 \right) \frac{1 + 0.4 \left( \frac{80}{72} \right)^2}{12 \left( 8.67 \right)} = 1.03 \)
Joint forces are:

Joint 1 – Transverse forces

Shear, \( V_{u1} = 3.18 \text{ kips/ft (36 ft)} \) = 114.5 kips  
Moment, \( M_{u1} = 114.5 \text{ kips (36 ft/2)} \) = 2,061 ft-kips  
Chord tension force, \( T_{u1} = 
\frac{M}{d} = 1.03(2,061 \text{ ft-kips}/71 \text{ ft}) \) = 29.9 kips

Joint 2 – Transverse forces

Shear, \( V_{u2} = 3.18 \text{ kips/ft (40 ft)} \) = 127 kips  
Moment, \( M_{u2} = 127 \text{ kips (40 ft/2)} \) = 2,540 ft-kips  
Chord tension force, \( T_{u2} = 
\frac{M}{d} = 1.03(2,540 \text{ ft-kips}/71 \text{ ft}) \) = 36.9 kips

Joint 3 – Transverse forces

Shear, \( V_{u3} = 127 \text{ kips} + 2.36 \text{ kips/ft (24 ft)} - 105.8 \text{ kips} \) = 78.1 kips  
Moment, \( M_{u3} = 127 \text{ kips (44 ft)} + 56.7 \text{ kips (12 ft)} - 105.8 \text{ kips (24 ft)} \) = 3,738 ft-kips  
Chord tension force, \( T_{u3} = 
\frac{M}{d} = 1.03(3,738 \text{ ft-kips}/71 \text{ ft}) \) = 54.2 kips

Joint 4 – Longitudinal forces

Wall Force, \( F = 423 \text{ kips}/8 \) = 52.9 kips  
Wall shear along wall length, \( V_{u4} = 52.9 \text{ kips (36 ft)}/(152 \text{ ft}/2) \) = 25.0 kips  
Collector force at wall end, \( T_{u4} = C_{u4} = 52.9 \text{ kips} - 25.0 \text{ kips} \) = 27.9 kips

Joint 4 – Out-of-plane forces

The Provisions have several requirements for out-of-plane forces. None are unique to precast diaphragms and all are less than the requirements in ACI 318 for precast construction regardless of seismic considerations. Assuming the planks are similar to beams and comply with the minimum requirements of Provisions Sec. 5.2.6.1.1 [4.6.1.1] (Seismic Design Category A and greater) [In the 2003 Provisions, all requirements for Seismic Design Category A are in Sec. 1.5 but they generally are the same as those in the 2000 Provisions. The design and detailing requirements in 2003 Provisions Sec. 4.6 apply to Seismic Design Category B and greater], the required out-of-plane horizontal force is:

\[ 0.05(D + L)_{\text{plank}} = 0.05(67 \text{ psf} + 40 \text{ psf})(24 \text{ ft}/2) = 64.2 \text{ plf} \]

According to Provisions Sec. 5.2.6.1.2 [4.6.1.2] (Seismic Design Category A and greater), the minimum anchorage for masonry walls is:

\[ F_p = 400(S_{DS})I = 400(0.39)1.0 = 156 \text{ plf} \]

According to Provisions Sec. 5.2.6.2.7 [4.6.1.3] (Seismic Design Category B and greater), bearing wall anchorage shall be designed for a force computed as:

\[ 0.4(S_{DS})W_{\text{wall}} = 0.4(0.39)(48 \text{ psf})(8.67 \text{ ft}) = 64.9 \text{ plf} \]

Provisions Sec. 5.2.6.3.2 [4.6.2.1] (Seismic Design Category C and greater) requires masonry wall anchorage to flexible diaphragms to be designed for a larger force. This diaphragm is
considered rigid with respect to the walls, and considering that it is designed to avoid yield under the loads that will yield the walls, this is a reasonable assumption.

\[ F_p = 1.2(S_{Dv})I_{wp} = 1.2(0.39)1.0[(48 \text{ psf})(8.67 \text{ ft})] = 195 \text{ plf} \]

[In the 2003 Provisions, Eq. 4.6-1 in Sec. 4.6.2.1 has been changed to 0.85S_{Dv}I_{wp}.]

The force requirements in ACI 318 Sec. 16.5 will be described later.

Joint 5 – Longitudinal forces

- Wall force, \( F = 423 \text{ kips}/8 = 52.9 \text{ kips} \)
- Wall shear along each side of wall, \( V_u = 52.9 \text{ kips} \)
- Collector force at wall end, \( T_a = C_a = 52.9 \text{ kips} - 25.0 \text{ kips} = 27.9 \text{ kips} \)

ACI 318 Sec. 16.5 also has minimum connection force requirements for structural integrity of precast concrete bearing wall construction. For buildings over two stories there are force requirements for horizontal and vertical members. This building has no vertical precast members. However, ACI 318 Sec. 16.5.1 specifies that the strengths “. . . for structural integrity shall apply to all precast concrete structures.” This is interpreted to apply to the precast elements of this masonry bearing wall structure. The horizontal tie force requirements are:

1. 1,500 lb/ft parallel and perpendicular to the span of the floor members. The maximum spacing of ties parallel to the span is 10 ft. The maximum spacing of ties perpendicular to the span is the distance between supporting walls or beams.

2. 16,000 lb parallel to the perimeter of a floor or roof located within 4 ft of the edge at all edges.

ACI’s tie forces are far greater than the minimum tie forces given in the Provisions for beam supports and anchorage for of masonry walls. They do control some of the reinforcement provided, but most of the reinforcement is controlled by the computed connections for diaphragm action.

7.1.1.6 Diaphragm Design and Details

Before beginning the proportioning of reinforcement, a note about ACI’s \( \phi \) factors is necessary. The Provisions cites ASCE 7 for combination of seismic load effects with the effects of other loads. Both ASCE 7 and the Provisions make it clear that the appropriate \( \phi \) factors within ACI 318 are those contained within Appendix C of ACI 318-99. These factors are about 10% less than the comparable factors within the main body of the standard. The 2002 edition of ACI 318 has placed the ASCE 7 load combinations within the main body of the standard and revised the \( \phi \) factors accordingly. This example uses the \( \phi \) factors given in the 2002 edition of ACI 318, which are the same as those given in Appendix C of the 1999 edition with one exception. Thus, the \( \phi \) factors used here are:

- Tension control (bending and ties) \( \phi = 0.90 \)
- Shear \( \phi = 0.75 \)
- Compression control in tied members \( \phi = 0.65 \).

The minimum tie force requirements given in ACI 318 Sec. 16.5 are specified as nominal values, meaning that \( \phi = 1.00 \) for those forces.
7.1.1.6.1 Design and Detailing at Joint 3

Joint 3 is designed first to check the requirements of Provisions Sec. 9A.3.9 [A9.2.4], which references ACI 318 Sec. 21.7.8.3 [21.9.8.3], which then refers to ACI 318 Sec. 21.7.5.3 [21.9.5.3]. This section provides requirements for transverse reinforcement in the chords of the diaphragm. The compressive stress in the chord is computed using the ultimate moment based on a linear elastic model and gross section properties. To determine the in-plane section modulus (S) of the diaphragm, an equivalent thickness (t) based on the cross sectional area is used for the hollow core precast units as follows.

\[ t = \text{area/width} = 215/48 = 4.5 \text{ in.} \]

\[ S = t d / 6 \]

Chord compressive stress is computed as:

\[ M_u / S = 6M_{u3}/td^2 = 6(3,738 \times 12)/(4.5)(72 \times 12)^2 = 80.1 \text{ psi} \]

The design 28-day compressive strength of the grout is 4,000 psi. Since the chord compressive stress is less than 0.2 \( f'c = 0.2(4,000) = 800 \text{ psi} \), the transverse reinforcement indicated in ACI 318 Sec. 21.4.4.1 through 21.4.4.3 is not required.

Compute the required amount of chord reinforcement as:

Chord reinforcement, \( A_{cs} = T_u / \phi f_y = (54.2 \text{ kips})/[0.9(60 \text{ ksi})] = 1.00 \text{ in.}^2 \)

Use two #7 bars, \( A_s = 2(0.60) = 1.20 \text{ in.}^2 \) along the exterior edges (top and bottom of the plan in Figure 7.1-2). Require cover for chord bars and spacing between bars at splices and anchorage zones by ACI 318 Sec. 21.7.8.3 [21.9.8.3].

Minimum cover = 2.5(7/8) = 2.19 in., but not less than 2.0 in.
Minimum spacing = 3(7/8) = 2.63 in., but not less than 1-1/2 in.

Figure 7.1-3 shows the chord element at the exterior edges of the diaphragm. The chord bars extend along the length of the exterior longitudinal walls and act as collectors for these walls in the longitudinal direction (see Joint 4 collector reinforcement and Figure 7.1-7).
Joint 3 must also be checked for the minimum ACI tie forces. The chord reinforcement obviously exceeds the 16 kip perimeter force requirement. The 1.5 kips per foot requirement requires a 6 kip tie at each joint between the planks, which is satisfied with a #3 bar in each joint (0.11 in.² at 60 ksi = 6.6 kips). This bar is required at all bearing walls and is shown in subsequent details.

7.1.1.6.2 Joint 1 Design and Detailing

The design must provide sufficient reinforcement for chord forces as well as shear friction connection forces as follows:

Chord reinforcement, \( A_{c,1} = \frac{T_{u,1}}{\phi f_y} = \frac{29.9 \text{ kips}}{0.9(60 \text{ ksi})} = 0.55 \text{ in.}^2 \) (collector force from Joint 4 calculations at 27.9 kips is not directly additive).

Shear friction reinforcement, \( A_{vf,1} = \frac{V_{u,1}}{\phi \mu f_y} = \frac{114.5 \text{ kips}}{(0.75)(1.0)(60 \text{ ksi})} = 2.54 \text{ in.}^2 \)

Total reinforcement required = \( 2(0.55 \text{ in.}^2) + 2.54 \text{ in.}^2 = 3.65 \text{ in.}^2 \)

ACI tie force = \( (3 \text{ kips/ft})(72 \text{ ft}) = 216 \text{ kips} \); reinforcement = \( 216 \text{ kips}/(60 \text{ ksi}) = 3.60 \text{ in.}^2 \)

Provide four #7 bars (two at each of the outside edges) plus four #6 bars (two each at the interior joint at the ends of the plank) for a total area of reinforcement of \( 4(0.60 \text{ in.}^2) + 4(0.44 \text{ in.}^2) = 4.16 \text{ in.}^2 \)

Because the interior joint reinforcement acts as the collector reinforcement in the longitudinal direction for the interior longitudinal walls, the cover and spacing of the two #6 bars in the interior joints will be provided to meet the requirements of ACI 318 Sec. 21.7.8.3 [21.9.8.3]:

Minimum cover = \( 2.5(6/8) = 1.88 \text{ in.} \), but not less than 2.0 in.

Minimum spacing = \( 3(6/8) = 2.25 \text{ in.} \), but not less than 1-1/2 in.

Figure 7.1-4 shows the reinforcement in the interior joints at the ends of the plank, which is also the collector reinforcement for the interior longitudinal walls (Joint 5). The two #6 bars extend along the length of the interior longitudinal walls as shown in Figure 7.1-8.

![Figure 7.1-4 Interior joint reinforcement at the ends of plank and the collector reinforcement at the end of the interior longitudinal walls - Joints 1 and 5 (1.0 in. = 25.4 mm).](image-url)
Figure 7.1-5 shows the extension of the two #6 bars of Figure 7.1-4 into the region where the plank is parallel to the bars. The bars will need to be extended the full length of the diaphragm unless supplemental plank reinforcement is provided. This detail makes use of this supplement plank reinforcement (two #6 bars or an equal area of strand per ACI 318-99 Sec. 21.7.5.2 [21.9.5.2]) and shows the bars anchored at each end of the plank. The anchorage length of the #6 bars is calculated using ACI 318-99 Sec. 21.7.5.4 [21.9.5.4] which references ACI 318 Sec. 21.5.4:

\[ l_d = 1.6(2.5) \left( \frac{f_y d_b}{65 \sqrt{f_c'}} \right) = 1.6(2.5) \left[ \frac{60,000(d_b)}{65 \sqrt{4,000}} \right] = 58.2d_b \]

The 2.5 factor is for the difference between straight and hooked bars, and the 1.6 factor applies when the development length is not within a confined core. Using #6 bars, the required \( l_d = 58.2(0.75 \text{ in.}) = 43.7 \text{ in.} \). Therefore, use \( l_d = 4 \text{ ft}, \) which is the width of the plank.

7.1.1.6.3 Joint 2 Design and Detailing

The chord design is similar to the previous calculations:

Chord reinforcement, \( A_{ch2} = T_{u2} / \phi f_y = (36.9 \text{ kips}) / [0.9(60 \text{ ksi})] = 0.68 \text{ in.}^2 \)

The shear force may be reduced along Joint 2 by the shear friction resistance provided by the supplemental chord reinforcement \( (2A_{chord} - A_{ch2}) \) and by the four #6 bars projecting from the interior longitudinal walls across this joint. The supplemental chord bars, which are located at the end of the walls, are conservatively excluded here. The shear force along the outer joint of the wall where the plank is parallel to the wall is modified as:

\[ V_{u2}^{\text{Mod}} = V_{u2} - \left[ \phi f_y \mu (A_{chord}) \right] = 127 - \left[ 0.75(60)(1.0)(4 \times 0.44) \right] = 47.8 \text{ kips} \]

This force must be transferred from the planks to the wall. Using the arrangement shown in Figure 7.1-6, the required shear friction reinforcement \( (A_{v2}) \) is computed as:

\[ A_{v2} = \frac{V_{u2}^{\text{Mod}}}{\phi f_y (\mu \sin \alpha_f + \cos \alpha_f)} = \frac{47.8}{0.75(60)(1.0 \sin 26.6^\circ + \cos 26.6^\circ)} = 0.79 \text{ in.}^2 \]
Use two #3 bars placed at 26.6 degrees (2-to-1 slope) across the joint at 4 ft from the ends of the plank and at 8 ft on center (three sets per plank). The angle ($\alpha_f$) used above provides development of the #3 bars while limiting the grouting to the outside core of the plank. The total shear reinforcement provided is $9(0.11 \text{ in.}^2) = 0.99 \text{ in.}^2$

The shear force between the other face of this wall and the diaphragm is:

$$V_{u2} - F = 127 - 106 = 21 \text{ kips}$$

The shear friction resistance provided by #3 bars in the grout key between each plank (provided for the 1.5 klf requirement of the ACI) is computed as:

$$\phi A_f f_{yf} \mu = (0.75)(10 \text{ bars})(0.11 \text{ in.}^2)(60 \text{ ksi})(1.0) = 49.5 \text{ kips}$$

The development length of the #3 and #4 bars will now be checked. For the 180 degree standard hook use ACI 318 Sec. 12.5, $l_{dh} = l_{hb}$ times the factors of ACI 318 Sec. 12.5.3, but not less than $8d_h$ or 6 in. Side cover exceeds 2-1/2 in. and cover on the bar extension beyond the hook is provided by the grout and the planks, which is close enough to 2 in. to apply the 0.7 factor of ACI 318 Sec. 12.5.3.2. The continuous #5 provides transverse reinforcement, but it is not arranged to take advantage of ACI 318’s 0.8 factor. For the #3 hook:

$$l_{dh} = \frac{0.7(1,200)d_h}{\sqrt{f_c'}} = \frac{0.7(1,200)(0.375)}{\sqrt{4,000}} = 4.95 \text{ in.} \quad (6" \text{ minimum})$$

The available distance for the perpendicular hook is about 5-1/2 in. The bar will not be fully developed at the end of the plank because of the 6 in. minimum requirement. The full strength is not required for shear transfer. By inspection, the diagonal #3 hook will be developed in the wall as required for the computed diaphragm-to-shear-wall transfer. The straight end of the #3 will now be checked. The standard development length of ACI 318 Sec. 12.2 is used for $l_d$.

$$l_d = \frac{f_y f_{hd}}{25\sqrt{f_c'}} = \frac{60,000(0.375)}{25\sqrt{4,000}} = 14.2 \text{ in.}$$

Figure 7.1-6 shows the reinforcement along each side of the wall on Joint 2.
7.1.6.4 Joint 4 Design and Detailing

The required shear friction reinforcement along the wall length is computed as:

\[ A_{qf} = \frac{V_{ul}}{\phi f_y} = \frac{25.0 \text{ kips}}{(0.75)(1.0)(60 \text{ ksi})} = 0.56 \text{ in.}^2 \]

Based upon the ACI tie requirement, provide #3 bars at each plank-to-plank joint. For eight bars total, the area of reinforcement is \(8(0.11) = 0.88 \text{ in.}^2\), which is more than sufficient even considering the marginal development length, which is less favorable at Joint 2. The bars are extended 2 ft into the grout key, which is more than the development length and equal to half the width of the plank.

The required collector reinforcement is computed as:

\[ A_{cd} = \frac{T_{ul}}{\phi f_y} = \frac{27.9 \text{ kips}}{0.9(60 \text{ ksi})} = 0.52 \text{ in.}^2 \]

The two #7 bars, which are an extension of the transverse chord reinforcement, provide an area of reinforcement of 1.20 in.².

The reinforcement required by the Provisions for out-of-plane force is (195 plf) is far less than the ACI 318 requirement.

Figure 7.1-7 shows this joint along the wall.
7.1.1.6.5 Joint 5 Design and Detailing

The required shear friction reinforcement along the wall length is computed as:

\[ A_{vf} = \frac{V_{u}}{\phi f_{y}} = \frac{12.5 \text{ kips}}{0.75 \times 1.0 \times 60 \text{ ksi}} = 0.28 \text{ in.}^2 \]

Provide #3 bars at each plank-to-plank joint for a total of 8 bars.

The required collector reinforcement is computed as:

\[ A_{sc} = \frac{T_{u}}{\phi f_{y}} = \frac{27.9 \text{ kips}}{0.9 \times 60 \text{ ksi}} = 0.52 \text{ in.}^2 \]

Two #6 bars specified for the design of Joint 1 above provide an area of reinforcement of 0.88 in.\(^2\). Figure 7.1-8 shows this joint along the wall.
7.1.2 Topped Precast Concrete Units for Five-Story Masonry Building, Los Angeles, California (see Sec. 9.2)

This design shows the floor and roof diaphragms using topped precast units in the five-story masonry building in Los Angeles, California. The topping thickness exceeds the minimum thickness of 2 in. as required for composite topping slabs by ACI 318 Sec. 21.7.4 [21.9.4]. The topping shall be lightweight concrete (weight = 115 pcf) with a 28-day compressive strength ($f'_{c}$) of 4,000 psi and is to act compositely with the 8-in.-thick hollow-core precast, prestressed concrete plank. Design parameters are provided in Sec. 9.2. Figure 9.2-1 shows the typical floor and roof plan.

7.1.2.1 General Design Requirements

Topped diaphragms may be used in any Seismic Design Category. ACI 318 Sec. 21.7 [21.9] provides design provisions for topped precast concrete diaphragms. Provisions Sec. 5.2.6 [4.6] specifies the forces to be used in designing the diaphragms. The amplification factor of Provisions Sec. 9.1.1.4 [not applicable in the 2003 Provisions] is 1.03, the same as previously computed for the untopped diaphragm.
As noted above, the chord amplification factor has been dropped for the 2003 Provisions and does not occur in ASC 318-02.]

7.1.2.2 General In-Plane Seismic Design Forces for Topped Diaphragms

The in-plane diaphragm seismic design force \( F_{px} \) is calculated using Provisions Eq. 5.2.6.4.4 [4.6-2] but must not be less than \( 0.2 S_{DS} I w_{px} \) and need not be more than \( 0.4 S_{DS} I w_{px} \). \( V_x \) must be added to \( F_{px} \) calculated using Eq. 5.2.6.4.4 [4.6-2] where:

\[
\begin{align*}
  w_{px} & = \text{the weight tributary to the diaphragm at Level } x \\
  S_{DS} & = \text{the spectral response acceleration parameter at short periods (Provisions Sec. 4.1.2 [3.3.5])} \\
  I & = \text{occupancy importance factor (Provisions Sec. 1.4 [1.3])} \\
  V_x & = \text{the portion of the seismic shear force required to be transferred to the components of the vertical seismic-force-resisting system due to offsets or changes in stiffness of the vertical resisting member at the diaphragm being designed.}
\end{align*}
\]

For Seismic Design Category C and higher, Provisions Sec. 5.2.6.3.1 [4.6.2.2] requires that collector elements, collector splices, and collector connections to the vertical seismic-force-resisting members be designed in accordance with Provisions Sec. 5.2.7.1 [4.2.2.2], which combines the diaphragm forces times the overstrength factor \( \Omega_0 \) and the effects of gravity forces. The parameters from example in Sec. 9.2 used to calculate in-plane seismic design forces for the diaphragms are provided in Table 7.1-4.

<table>
<thead>
<tr>
<th>Table 7.1-4 Design Parameters from Sec. 9.2</th>
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<tbody>
<tr>
<td>Design Parameter</td>
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<tr>
<td>( \Omega_0 )</td>
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<tr>
<td>( w_i ) (roof)</td>
</tr>
<tr>
<td>( w_i ) (floor)</td>
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<tr>
<td>( S_{DS} )</td>
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<tr>
<td>( I )</td>
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<tr>
<td>Seismic Design Category</td>
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</tbody>
</table>

1.0 kip = 4.45 kN.

7.1.2.3 Diaphragm Forces

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

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

**Roof**

- Total weight = 1,166 kips
- Walls parallel to force = (60 psf)(277 ft)(8.67 ft/2) = 1,094 kips
- \( w_{px} \) = 72 kips
Floors

Total weight = 1,302 kips
Walls parallel to force = (60 psf)(277 ft)(8.67 ft) = -144 kips
wpx = 1,158 kips

Compute diaphragm demands in accordance with Provisions Eq. 5.2.6.4.4 [4.6-2]:

\[
F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i}
\]

Calculations for \( F_{px} \) are provided in Table 7.1-5. The values for \( F_i \) and \( V_i \) are listed in Table 9.2-17.

<table>
<thead>
<tr>
<th>Table 7.1-5 ( F_{px} ) Calculations from Sec. 9.2</th>
</tr>
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<tbody>
<tr>
<td>Level</td>
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<tr>
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<tr>
<td>Roof</td>
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<td>4</td>
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</table>

1.0 kip = 4.45 kN.

The minimum value of \( F_{px} = 0.2S_{DS}w_{px} \) = 0.2(1.0)1.0(1,094 kips) = 219 kips (at the roof)
= 0.2(1.0)1.0(1,158 kips) = 232 kips (at floors)

The maximum value of \( F_{px} = 0.4S_{DS}w_{px} \) = 2(219 kips) = 438 kips (at the roof)
= 2(232 kips) = 463 kips (at floors)

The value of \( F_{px} \) used for design of the diaphragms is 463 kips, except for collector elements where forces will be computed below.

7.1.2.4 Static Analysis of Diaphragms

The seismic design force of 463 kips is distributed as in Sec. 7.1.1.6 (Figure 7.1-1 shows the distribution). The force is only 9.5 percent higher than that used to design the untopped diaphragm for the New York design due to the intent to prevent yielding in the untopped diaphragm. Figure 7.1-2 shows critical regions of the diaphragm to be considered in this design. Collector elements will be designed for 2.5 times the diaphragm force based on the overstrength factor (\( \Omega_o \)).

Joint forces taken from Sec. 7.1.1.5 times 1.095 are as:
Joint 1 – Transverse forces

Shear, \( V_{u1} = 114.5 \text{ kips} \times 1.095 \) = 125 kips
Moment, \( M_{u1} = 2,061 \text{ ft-kips} \times 1.095 \) = 2,250 ft-kips
Chord tension force, \( T_{u1} = \frac{M}{d} = 1.03 \times 2,250 \text{ ft-kips} / 71 \text{ ft} \) = 32.6 kips

Joint 2 – Transverse forces

Shear, \( V_{u2} = 127 \text{ kips} \times 1.095 \) = 139 kips
Moment, \( M_{u2} = 2,540 \text{ ft-kips} \times 1.095 \) = 2,780 ft-kips
Chord tension force, \( T_{u2} = \frac{M}{d} = 1.03 \times 2,780 \text{ ft-kips} / 71 \text{ ft} \) = 39.3 kips

Joint 3 – Transverse forces

Shear, \( V_{u3} = 78.1 \text{ kips} \times 1.095 \) = 85.5 kips
Moment, \( M_{u3} = 3,738 \text{ ft-kips} \times 1.095 \) = 4,090 ft-kips
Chord tension force, \( T_{u3} = \frac{M}{d} = 1.03 \times 4,090 \text{ ft-kips} / 71 \text{ ft} \) = 59.3 kips

Joint 4 – Longitudinal forces

Wall Force, \( F = 52.9 \text{ kips} \times 1.095 \) = 57.9 kips
Wall shear along wall length, \( V_{w4} = 25 \text{ kips} \times 1.095 \) = 27.4 kips
Collector force at wall end, \( \Omega_0T_{w4} = 2.5(27.9 \text{ kips})(1.095) \) = 76.4 kips

Out-of-Plane forces

Just as with the untopped diaphragm, the out-of-plane forces are controlled by ACI 318 Sec. 16.5, which requires horizontal ties of 1.5 kips per foot from floor to walls.

Joint 5 – Longitudinal forces

Wall Force, \( F = 463 \text{ kips} / 8 \text{ walls} \) = 57.9 kips
Wall shear along each side of wall, \( V_{w4} = 12.5 \text{ kips} \times 1.095 \) = 13.7 kips
Collector force at wall end, \( \Omega_0T_{w4} = 2.5(27.9 \text{ kips})(1.095) \) = 76.4 kips

7.1.2.5 Diaphragm Design and Details

7.1.2.5.1 Minimum Reinforcement for 2.5 in. Topping

ACI 318 Sec. 21.7.5.1 [21.9.5.1] references ACI 318 Sec. 7.12, which requires a minimum \( A_s = 0.0018bd \) for welded wire fabric. For a 2.5 in. topping, the required \( A_s = 0.054 \text{ in.}^2/\text{ft} \). WWF 10×10 - W4.5×W4.5 provides 0.054 in.\(^2\)/ft. The minimum spacing of wires is 10 in. and the maximum spacing is 18 in. Note that the ACI 318 Sec. 7.12 limit on spacing of five times thickness is interpreted such that the topping thickness is not the pertinent thickness.

7.1.2.5.2 Boundary Members

Joint 3 has the maximum bending moment and is used to determine the boundary member reinforcement of the chord along the exterior edge. The need for transverse boundary member reinforcement is reviewed using ACI 318 Sec. 21.7.5.3 [21.9.5.3]. Calculate the compressive stress in the chord with the ultimate moment using a linear elastic model and gross section properties of the topping. It is
conservative to ignore the precast units, but not necessary. As developed previously, the chord compressive stress is:

\[ 6M_{ud}/td^2 = 6(4,090 \times 12)/(2.5)(72 \times 12)^2 = 158 \text{ psi} \]

The chord compressive stress is less than \(0.2f'_c = 0.2(4,000) = 800\text{ psi}\). Transverse reinforcement in the boundary member is not required.

The required chord reinforcement is:

\[ A_{s3} = T_{u3}/\phi f_y = (59.3 \text{ kips})/[0.9(60 \text{ ksi})] = 1.10 \text{ in.}^2 \]

7.1.2.5.3 Collectors

The design for Joint 4 collector reinforcement at the end of the exterior longitudinal walls and for Joint 5 at the interior longitudinal walls is the same.

\[ A_{s4} = A_{s5} = \Omega_{0} T_{u4}/\phi f_y = (76.4 \text{ kips})/[0.9(60 \text{ ksi})] = 1.41 \text{ in.}^2 \]

Use two #8 bars \((A_s = 2 \times 0.79 = 1.58 \text{ in.}^2)\) along the exterior edges, along the length of the exterior longitudinal walls, and along the length of the interior longitudinal walls. Provide cover for chord and collector bars and spacing between bars per ACI 318 Sec. 21.7.8.3 [21.9.8.3].

Minimum cover = 2.5(8/8) = 2.5 in., but not less than 2.0 in.
Minimum spacing = 3(8/8) = 3.0 in., but not less than 1-1/2 in.

Figure 7.1-9 shows the diaphragm plan and section cuts of the details and Figure 7.1-10, the boundary member and chord/collector reinforcement along the edge. Given the close margin on cover, the transverse reinforcement at lap splices also is shown.
Figure 7.1-10 Boundary member, and chord and collector reinforcement (1.0 in. = 25.4 mm).

Figure 7.1-11 shows the collector reinforcement for the interior longitudinal walls. The side cover of 2-1/2 in. is provided by casting the topping into the cores and by the stems of the plank. A minimum space of 1 in. is provided between the plank stems and the sides of the bars.

Figure 7.1-11 Collector reinforcement at the end of the interior longitudinal walls - Joint 5 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

7.1.2.5.4 Shear Resistance
Thin composite and noncomposite topping slabs on precast floor and roof members may not have reliable shear strength provided by the concrete. In accordance with ACI 318 Sec. 21.7.7.2 [21.9.7.2], all of the shear resistance must be provided by the reinforcement (that is, $V_c = 0$).

$$\phi V_n = \phi A_v \rho f_y = 0.75(0.054 \text{ in.}^2/\text{ft})60 \text{ ksi} = 2.43 \text{ kips/ft}$$

The shear resistance in the transverse direction is:

$$2.43 \text{ kips/ft (72 ft) = 175 kips}$$

which is greater than the Joint 2 shear (maximum transverse shear) of 139 kips. No. 3 dowels are used to make the welded wire fabric continuous across the masonry walls. The topping is to be cast into the masonry walls as shown in Figure 7.1-12, and the spacing of the No. 3 bars is set to be modular with the CMU.

![Figure 7.1-12 Wall-to-diaphragm reinforcement along interior longitudinal walls - Joint 5 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).](image)

The required shear reinforcement along the exterior longitudinal wall (Joint 4) is:

$$A_{vr4} = \frac{V_{u4}}{\phi \mu f_y} = (27.4 \text{ kips})/(0.75)(1.0)(60 \text{ ksi}) = 0.61 \text{ in.}^2$$

7.1.2.5.5 Check Out-of-Plane Forces

At Joint 4 with bars at 2 ft on center, $F_p = 624 \text{ plf} = 2 \text{ ft}(624 \text{ plf}) = 1.25 \text{ kips}$. The required reinforcement, $A_v = 1.25/(0.9)(60 \text{ ksi}) = 0.023 \text{ in.}^2$. Provide #3 bars at 2 ft on center, which provides a nominal strength of $0.11 \times 60 / 2 = 3.3 \text{ klf}$. The detail provides more than required by ACI 318 Sec. 16.5 for the 1.5 klf tie force. The development length was checked in the prior example. Using #3 bars at 2 ft on center will be adequate, and the detail is shown in Figure 7.1-13. The detail at joint 2 is similar.
Figure 7.1-13 Exterior longitudinal wall-to-diaphragm reinforcement and out-of-plane anchorage - Joint 4 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).
7.2 THREE-STORY OFFICE BUILDING WITH PRECAST CONCRETE SHEAR WALLS

This example illustrates the seismic design of ordinary precast concrete shear walls that may be used in regions of low to moderate seismicity. The Provisions has one requirement for detailing such walls: connections that resist overturning shall be Type Y or Z. ACI 318-02 has incorporated a less specific requirement, renamed the system as intermediate precast structural walls, and removed some of the detail. This example shows an interpretation of the intent of the Provisions for precast shear wall systems in regions of moderate and low seismicity, which should also meet the cited ACI 318-02 requirements.

[As indicated at the beginning of this chapter, the requirements for precast shear wall systems in the 2003 Provisions have been revised – primarily to point to ACI 318-02 by reference. See also Sec. 7.2.2.1 for more discussion of system requirements.]

7.2.1 Building Description

This precast concrete building is a three-story office building (Seismic Use Group I) in southern New England on Site Class D soils. The structure utilizes 10-ft-wide by 18-in.-deep prestressed double tees (DTs) spanning 40 ft to prestressed inverted tee beams for the floors and the roof. The DTs are to be constructed using lightweight concrete. Each of the above-grade floors and the roof are covered with a 2-in.-thick (minimum), normal weight cast-in-place concrete topping. The vertical seismic-force-resisting system is to be constructed entirely of precast concrete walls located around the stairs and elevator/mechanical shafts. The only features illustrated in this example are the rational selection of the seismic design parameters and the design of the reinforcement and connections of the precast concrete shear walls. The diaphragm design is not illustrated.

As shown in Figure 7.2-1, the building has a regular plan. The precast shear walls are continuous from the ground level to 12 ft above the roof. Walls of the elevator/mechanical pits are cast-in-place below grade. The building has no vertical irregularities. The story-to-story height is 12 ft.
The precast walls are estimated to be 8 in. thick for building mass calculations. These walls are normal weight concrete with a 28-day compressive strength, \( f'_{c} = 5,000 \text{ psi} \). Reinforcing bars used at the ends of the walls and in welded connectors are ASTM A706 (60 ksi yield strength). The concrete for the foundations and below-grade walls has a 28-day compressive strength, \( f'_{c} = 4,000 \text{ psi} \).

### 7.2.2 Design Requirements

#### 7.2.2.1 Seismic Parameters of the Provisions

The basic parameters affecting the design and detailing of the building are shown in Table 7.2-1.
### Table 7.2-1  Design Parameters

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Use Group I</td>
<td>$I = 1.0$</td>
</tr>
<tr>
<td>$S_s$ (Map 1 [Figure 3.3-1])</td>
<td>0.266</td>
</tr>
<tr>
<td>$S_i$ (Map 2 [Figure 3.3-2])</td>
<td>0.08</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>$F_a$</td>
<td>1.59</td>
</tr>
<tr>
<td>$F_v$</td>
<td>2.4</td>
</tr>
<tr>
<td>$S_{MS} = F_a S_s$</td>
<td>0.425</td>
</tr>
<tr>
<td>$S_{MI} = F_v S_i$</td>
<td>0.192</td>
</tr>
<tr>
<td>$S_{DS} = 2/3 S_{MS}$</td>
<td>0.283</td>
</tr>
<tr>
<td>$S_{DI} = 2/3 S_{MI}$</td>
<td>0.128</td>
</tr>
</tbody>
</table>

### Seismic Design Category

<table>
<thead>
<tr>
<th>Basic Seismic-Force-Resisting System</th>
<th>Bearing Wall System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Type *</td>
<td>Ordinary Reinforced Concrete Shear Walls</td>
</tr>
<tr>
<td>$R$</td>
<td>4</td>
</tr>
<tr>
<td>$\Omega_0$</td>
<td>2.5</td>
</tr>
<tr>
<td>$C_d$</td>
<td>4</td>
</tr>
</tbody>
</table>

* Provisions Sec. 9.1.1.3 [9.2.2.1.3] provides for the use of ordinary reinforced concrete shear walls in Seismic Design Category B, which does not require adherence to the special seismic design provisions of ACI 318 Chapter 21.

[The 2003 Provisions have adopted the 2002 U.S. Geological Survey probabilistic seismic hazard maps and the maps have been added to the body of the 2003 Provisions as figures in Chapter 3. These figures replace the previously used separate map package.]

[Ordinary precast concrete shear walls is recognized as a system in Table 4.3-1 of the 2003 Provisions. Consistent with the philosophy that precast systems are not expected to perform as well as cast-in-place systems, the design factors for the ordinary precast concrete shear walls per 2003 Provisions Table 4.3-1 are: $R = 3$, $\Omega_0 = 2.5$, and $C_d = 3$. Note that while this system is permitted in Seismic Design Category B, unlike ordinary reinforced concrete shear walls, it is not permitted in Seismic Design Category C. Alternatively, as this example indicates conceptually, this building could be designed incorporating intermediate precast concrete shear walls with the following design values per 2003 Provisions Table 4.3-1: $R = 4$, $\Omega_0 = 2.5$, and $C_d = 4$.]

### 7.2.2.2 Structural Design Considerations

#### 7.2.2.2.1 Precast Shear Wall System

This system is designed to yield in bending at the base of the precast shear walls without shear slippage at any of the joints. Although not a stated design requirement of the Provisions or ACI 318-02 for this Seismic Design Category, shear slip could kink the vertical rebar at the connection and sabotage the intended performance, which counts on an $R$ factor of 4. The flexural connections at the ends of the
walls, which are highly stressed by seismic forces, are designed to be the Type Y connection specified in the Provisions. See Provisions Sec. 9.1.1.2 [9.2.2.1.1] (ACI Sec. 21.1 [21.1]) for the definitions of ordinary precast concrete structural walls and Provisions Sec. 9.1.1.12 [not applicable for the 2003 Provisions] (ACI Sec. 21.11.6) for the connections. The remainder of the connections (shear connectors) are then made strong enough to ensure that the inelastic straining is forced to the intended location.

[Per 2003 Provisions Sec. 9.2.2.1.1 (ACI 318-02 Sec. 21.1), ordinary precast concrete shear walls need only satisfy the requirements of ACI 318-02 Chapters 1-18 (with Chapter 16 superceding Chapter 14). Therefore, the connections are to be designed in accordance with ACI 318-02 Sec. 16.6.]

Although it would be desirable to force yielding to occur in a significant portion of the connections, it frequently is not possible to do so with common configurations of precast elements and connections. The connections are often unavoidable weak links. Careful attention to detail is required to assure adequate ductility in the location of first yield and that no other connections yield prematurely. For this particular example, the vertical bars at the ends of the shear walls act as flexural reinforcement for the walls and are selected as the location of first yield. The yielding will not propagate far into the wall vertically due to the unavoidable increase in flexural strength provided by unspliced reinforcement within the panel. The issue of most significant concern is the performance of the shear connections at the same joint. The connections are designed to provide the necessary shear resistance and avoid slip without unwittingly increasing the flexural capacity of the connection because such an increase would also increase the maximum shear force on the joint. At the base of the panel, welded steel angles are designed to be flexible for uplift but stiff for in-plane shear.

7.2.2.2.2 Building System

No height limitations are imposed (Provisions Table 5.2.2 [4.3-1]).

For structural design, the floors are assumed to act as rigid horizontal diaphragms to distribute seismic inertial forces to the walls parallel to the motion. The building is regular both in plan and elevation, for which, according to Provisions Table 5.2.5.1 [4.4-4], use of the ELF procedure (Provisions Sec. 5.4 [5.2]) is permitted.

Orthogonal load combinations are not required for this building (Provisions Sec. 5.2.5.2.1 [4.4.2.1]).

Ties, continuity, and anchorage (Provisions Sec. 5.2.6.1 and 5.2.6.2 [4.6.1.1 and 4.6.1.2]) must be explicitly considered when detailing connections between the floors and roof, and the walls and columns.

This example does not include consideration of nonstructural elements.

Collector elements are required due to the short length of shear walls as compared to the diaphragm dimensions, but are not designed in this example.

Diaphragms need to be designed for the required forces (Provisions Sec. 5.2.6.2.6 [4.6.1.9]), but that design is not illustrated here.

The bearing walls must be designed for a force perpendicular to their plane (Provisions Sec. 5.2.6.2.7 [4.6.1.3]), but this requirement is of no real consequence for this building.

The drift limit is 0.025\(h_{sx}\) (Provisions Table 5.2.8 [4.5-1]), but drift is not computed here.
ACI 318 Sec. 16.5 requires minimum strengths for connections between elements of precast building structures. The horizontal forces were described in Sec. 7.1; the vertical forces will be described in this example.

### 7.2.3 Load Combinations

The basic load combinations (Provisions Sec. 5.2.7 [4.2.2]) require that seismic forces and gravity loads be combined in accordance with the factored load combinations presented in ASCE 7 except that the factors for seismic loads ($E$) are defined by Provisions Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2]:

$$E = \rho Q_E \pm 0.2 S_{ds} D = (1.0) Q_E \pm (0.2)(0.283)D = Q_E \pm 0.0567D$$

According to Provisions Sec. 5.2.4.1 [4.3.3.1], $\rho = 1.0$ for structures in Seismic Design Categories A, B, and C, even though this seismic resisting system is not particularly redundant.

The relevant load combinations from ASCE 7 are:

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

Into each of these load combinations, substitute $E$ as determined above:

- $1.26D + Q_E + 0.5L$
- $1.14D - Q_E + 0.5L$ (will not control)
- $0.96D + Q_E$ (will not control)
- $0.843D - Q_E$

These load combinations are for loading in the plane of the shear walls.

### 7.2.4 Seismic Force Analysis

#### 7.2.4.1 Weight Calculations

For the roof and two floors

- 18 in. double tees (32 psf) + 2 in. topping (24 psf) = 56.0 psf
- Precast beams at 40 ft = 12.5 psf
- 16 in. square columns = 4.5 psf
- Ceiling, mechanical, miscellaneous = 4.0 psf
- Exterior cladding (per floor area) = 5.0 psf
- Partitions = 10.0 psf
- Total = 92.0 psf

The weight of each floor including the precast shear walls is:

$$120 \text{ ft}(150 \text{ ft})(92 \text{ psf}/1,000) + [15 \text{ ft}(4) + 25 \text{ ft}(2)](12 \text{ ft})(0.10 \text{ ksf}) = 1,790 \text{ kips}$$

Considering the roof to be the same weight as a floor, the total building weight is $W = 3(1,790 \text{ kips}) = 5,360 \text{ kips}$.
The seismic response coefficient \( C_s \) is computed using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:
\[
C_s = \frac{S_{ds}}{R/I} = \frac{0.283}{4/1} = 0.0708
\]

except that it need not exceed the value from *Provisions* Eq. 5.4.1.1-2 [5.2-3] computed as:
\[
C_s = \frac{S_{di}}{T(R/I)} = \frac{0.128}{0.29(4/1)} = 0.110
\]

where \( T \) is the fundamental period of the building computed using the approximate method of *Provisions* Eq. 5.4.2.1-1 [5.2-6]:
\[
T_a = C_r h_n^x = (0.02)(36)^{0.75} = 0.29 \text{ sec}
\]

Therefore, use \( C_s = 0.0708 \), which is larger than the minimum specified in *Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*]:
\[
C_s = 0.044 IS_{DS} = (0.044)(0.283) = 0.0125
\]

[The minimum \( C_s \) has been changed to 0.01 in the 2003 *Provisions*.]

The total seismic base shear is then calculated using *Provisions* Eq. 5.4-1 [5.2-1] as:
\[
V = C_s W = (0.0708)(5370) = 380 \text{ kips}
\]

Note that this force is substantially larger than a design wind would be. If a nominal 20 psf were applied to the long face and then amplified by a load factor of 1.6, the result would be less than half this seismic force already reduced by an \( R \) factor of 4.

### 7.2.4.3 Vertical Distribution of Seismic Forces

The seismic lateral force \( F_x \) at any level is determined in accordance with *Provisions* Sec. 5.4.3 [5.2.3]:
\[
F_x = C_{vx} V
\]

where
\[
C_{vx} = \frac{w_i h_i^k}{\sum_{i=1}^{n} w_i h_i^k}
\]

Since the period, \( T < 0.5 \text{ sec} \), \( k = 1 \) in both building directions. With equal weights at each floor level, the resulting values of \( C_{vx} \) and \( F_x \) are as follows:

<table>
<thead>
<tr>
<th>Level</th>
<th>( C_{vx} )</th>
<th>( F_x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>0.50</td>
<td>190 kips</td>
</tr>
<tr>
<td>Third Floor</td>
<td>0.33</td>
<td>127 kips</td>
</tr>
<tr>
<td>Second Floor</td>
<td>0.17</td>
<td>63.0 kips</td>
</tr>
</tbody>
</table>
7.2.4.4 Horizontal Shear Distribution and Torsion

7.2.4.4.1 Longitudinal Direction

Design each of the 25-ft-long walls at the elevator/mechanical shafts for half the total shear. Since the longitudinal walls are very close to the center of rigidity, assume that torsion will be resisted by the 15-ft-long stairwell walls in the transverse direction. The forces for each of the longitudinal walls are shown in Figure 7.2-2.

![Figure 7.2-2 Forces on the longitudinal walls](image)

**Figure 7.2-2** Forces on the longitudinal walls (1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m).

7.2.4.4.2 Transverse Direction

Design the four 15-ft-long stairwell walls for the total shear including 5 percent accidental torsion (*Provisions* Sec. 5.4.4.2 [5.2.4.2]). A rough approximation is used in place of a more rigorous analysis considering all of the walls. The maximum force on the walls is computed as:

\[
V = \frac{380}{4} + \frac{380(0.05)(150)}{(100 \text{ ft moment arm}) \times (2 \text{ walls in each set})} = 109 \text{ kips}
\]

Thus

\[
F_r = 109(0.50) = 54.5 \text{ kips}
\]

\[
F_1 = 109(0.33) = 36.3 \text{ kips}
\]

\[
F_2 = 109(0.167) = 18.2 \text{ kips}
\]

Seismic forces on the transverse walls of the stairwells are shown in Figure 7.2-3.
7.2.5 PROPORTIONING AND DETAILING

The strength of members and components is determined using the strengths permitted and required in ACI 318 excluding Chapter 21 (see Provisions Sec. 9.1.1.3 [9.2.2.1.3]).

7.2.5.1 Overturning Moment and End Reinforcement

Design shear panels to resist overturning by means of reinforcing bars at each end with a direct tension coupler at the joints. A commonly used alternative is a threaded post-tensioning bar inserted through the stack of panels, but the behavior is different, and the application of the rules for a Type Y connection to such a design is not clear.

7.2.5.1.1 Longitudinal Direction

The free-body diagram for the longitudinal walls is shown in Figure 7.2-4. The tension connection at the base of the precast panel to the below grade wall is governed by the seismic overturning moment and the dead loads of the panel and supported floors and roof. In this example, the weights for an elevator penthouse, with a floor and equipment at 180 psf between the shafts and a roof at 20 psf, are included. The weight for the floors includes double tees, ceiling and partition (total of 70 psf), but not beams and columns. Floor live load is 50 psf, except 100 psf is used in the elevator lobby. Roof snow load is 30 psf. (The elevator penthouse is so small that it was ignored in computing the gross seismic forces on the building, but it is not ignored in the following calculations.)
At the base

\[ M_E = (95 \text{ kips})(36 \text{ ft}) + (63.5 \text{ kips})(24 \text{ ft}) + (31.5 \text{ kips})(12 \text{ ft}) = 5,520 \text{ ft-kips} \]

\[ \sum D = \text{wall + exterior floors (\& roof) + lobby floors + penthouse floor + penthouse roof} \]
\[ = (25 \text{ ft})(48 \text{ ft})(0.1 \text{ ksf}) + (25 \text{ ft})(48 \text{ ft}/2)(0.070 \text{ ksf})(3) + (25 \text{ ft})(8 \text{ ft}/2)(0.070 \text{ ksf})(2) + \\
(25 \text{ ft})(8 \text{ ft}/2)(0.18 \text{ ksf}) + (25 \text{ ft})(24 \text{ ft}/2)(0.02 \text{ ksf}) \]
\[ = 120 + 126 + 14 + 18 + 6 = 284 \text{ kips} \]

\[ \sum L = (25 \text{ ft})(48 \text{ ft}/2)(0.05 \text{ ksf})(2) + (25 \text{ ft})(8 \text{ ft}/2)(0.1 \text{ ksf}) = 60 + 10 = 70 \text{ kips} \]

\[ \sum S = (25\text{ft})(48 \text{ ft} + 24 \text{ ft})(0.03 \text{ ksf})/2 = 27 \text{ kips} \]

Using the load combinations described above, the vertical loads for combining with the overturning moment are computed as:

\[ P_{\text{max}} = 1.26 D + 0.5 L + 0.2 S = 397 \text{ kips} \]
\[ P_{\text{min}} = 0.843 D = 239 \text{ kips} \]

The axial load is quite small for the wall panel. The average compression \( P_{\text{max}}/A_g = 0.165 \text{ ksi} \) (3.3 percent of \( f'_c \)). Therefore, the tension reinforcement can easily be found from the simple couple shown on Figure 7.2-4.

The effective moment arm is:

\[ jd = 25 - 1.5 = 23.5 \text{ ft} \]
and the net tension on the uplift side is:

\[
T_u = \frac{M}{jd} - \frac{P_{\text{min}}}{2} = \frac{5320}{23.5} - \frac{239}{2} = 107 \text{ kips}
\]

The required reinforcement is:

\[
A_s = T_u / f_y = (107 \text{ kips})/[0.9(60 \text{ ksi})] = 1.98 \text{ in.}^2
\]

Use two #9 bars \((A_s = 2.0 \text{ in.}^2)\) at each end with direct tension couplers for each bar at each panel joint. Since the flexural reinforcement must extend a minimum distance \(d\) (the flexural depth) beyond where it is no longer required, use both #9 bars at each end of the panel at all three levels for simplicity.

At this point a check of ACI 318 Sec. 16.5 will be made. Bearing walls must have vertical ties with a nominal strength exceeding 3 kips/ft, and there must be at least two ties per panel. With one tie at each end of a 25 ft panel, the demand on the tie is:

\[
T_n = (3 \text{ kip/ft})(25 \text{ ft})/2 = 37.5 \text{ kip}
\]

The two #9 bars are more than adequate for the ACI requirement.

Although no check for confinement of the compression boundary is required for ordinary precast concrete shear walls, it is shown here for interest. Using the check from ACI 318-99 Sec. 21.6.6.2 [21.7.6.2], the depth to the neutral axis is:

Total compression force = \(A_s f_y + P_{\text{max}} = (2.0)(60) + 397 = 517 \text{ kips}\)

Compression block \(a = (517 \text{ kips})/[(0.85)(5 \text{ ksi})(8 \text{ in. width})] = 15.2 \text{ in.}\)

Neutral axis depth \(c = a/(0.80) = 19.0 \text{ in.}\)

The maximum depth \((c)\) with no boundary member per ACI 318-99 Eq. 21-8 [21-8] is:

\[
c \leq \frac{l}{600(\delta_u / h_w)}
\]

where the term \((\delta_u / h_w)\) shall not be taken less than 0.007. Once the base joint yields, it is unlikely that there will be any flexural cracking in the wall more than a few feet above the base. An analysis of the wall for the design lateral forces using 50% of the gross moment of inertia, ignoring the effect of axial loads, and applying the \(C_d\) factor of 4 to the results gives a ratio \((\delta_u / h_w)\) far less than 0.007. Therefore, applying the 0.007 in the equation results in a distance \(c\) of 71 in., far in excess of the 19 in. required. Thus, ACI 318-99 would not require transverse reinforcement of the boundary even if this wall were designed as a special reinforced concrete shear wall. For those used to checking the compression stress as an index:

\[
\sigma = \frac{P}{A} + \frac{M}{S} = \frac{389}{8(25)12} + \frac{6(5,520)}{8(25)^2 (12)} = 742 \text{ psi}
\]

The limiting stress is \(0.2f_c'\), which is 1000 psi, so no transverse reinforcement is required at the ends of the longitudinal walls.

7.2.5.1.2 Transverse Direction
The free-body diagram of the transverse walls is shown in Figure 7.2-5. The weight of the precast concrete stairs is 100 psf and the roof over the stairs is 70 psf.

The transverse wall is similar to the longitudinal wall.

At the base

\[
M_E = (54.5 \text{ kips})(36 \text{ ft}) + (36.3 \text{ kips})(24 \text{ ft}) + (18.2 \text{ kips})(12 \text{ ft}) = 3,052 \text{ ft-kips}
\]

\[
\sum D = (15 \text{ ft})(48 \text{ ft})(0.1 \text{ ksf}) + 2(12.5 \text{ ft/2})(10 \text{ ft/2})(0.07 \text{ ksf})(3) + (15 \text{ ft})(8 \text{ ft/2})[(0.1 \text{ ksf})(3) + (0.07 \text{ ksf})] = 72 + 13 + 18 + 4 = 107 \text{ kips}
\]

\[
\sum L = 2(12.5 \text{ ft/2})(10 \text{ ft/2})(0.05 \text{ ksf})(2) + (15 \text{ ft})(8 \text{ ft/2})(0.1 \text{ ksf})(3) = 6 + 18 = 24 \text{ kips}
\]

\[
\sum S = [2(12.5 \text{ ft/2})(10 \text{ ft/2}) + (15 \text{ ft})(8 \text{ ft/2})](0.03 \text{ ksf}) = 3.7 \text{ kips}
\]

\[
P_{\text{max}} = 1.26(107) + 0.5(24) + 0.2(4) = 148 \text{ kips}
\]

\[
P_{\text{min}} = 0.843(107) = 90.5 \text{ kips}
\]

\[
jd = 15 - 1.5 = 13.5 \text{ ft}
\]

\[
T_u = (M_{\text{net}}/jd) - P_{\text{null}}/2 = (3,052/13.5) - 90.5/2 = 181 \text{ kips}
\]

\[
A_s = T_u/\phi_y = (181 \text{ kips})/[0.9(60 \text{ ksi})] = 3.35 \text{ in.}^2
\]
Use two #10 and one #9 bars \((A_s = 3.54 \text{ in.}^2)\) at each end of each wall with a direct tension coupler at each bar for each panel joint. All three bars at each end of the panel will also extend up through all three levels for simplicity. Following the same method for boundary member check as on the longitudinal walls:

\[
\text{Total compression force} = A_s f_y + P_{max} = (3.54)(60) + 148 = 360 \text{ kips}
\]

Compression block \(a = (360 \text{ kips})/[(0.85)(5 \text{ ksi})(8 \text{ in. width})] = 10.6 \text{ in.}\)

Neutral axis depth \(c = a/(0.80) = 13.3 \text{ in.}\)

Even though this wall is more flexible and the lateral loads will induce more flexural cracking, the computed deflections are still small and the minimum value of 0.007 is used for the ratio \((\delta_u/h_w)\). This yields a maximum value of \(c = 42.9 \text{ in.}\), thus confinement of the boundary would not be required. The check of compression stress as an index gives:

\[
\sigma = \frac{P}{A} + \frac{M}{S} = \frac{140}{8(15)12} + \frac{6(2,930)}{8(15)^2(12)} = 951 \text{ psi}
\]

Since \(\sigma < 1,000 \text{ psi}\), no transverse reinforcement is required at the ends of the transverse walls. Note how much closer to the criterion this transverse wall is by the compression stress check.

The overturning reinforcement and connection are shown in Figures 7.2-6. Provisions Sec. 9.1.1.12 [not applicable in the 2003 Provisions] (ACI 21.11.6.4) requires that this Type Y connection develop a probable strength of 125% of the nominal strength and that the anchorage on either side of the connection develop 130% of the defined probable strength. [As already noted, the connection requirements for ordinary precast concrete shear walls have been removed in the 2003 Provisions and the ACI 318-02 requirements are less specific.] The 125% requirement applies to the grouted mechanical splice, and the requirement that a mechanical coupler develop 125% of specified yield strength of the bar is identical to the Type 1 coupler defined by ACI 318 Sec. 21.2.6.1. Some of the grouted splices on the market can qualify as the Type 2 coupler defined by ACI, which must develop the specified tensile strength of the bar. The development length, \(l_{dh}\), for the spliced bars is multiplied by both the 1.25 and the 1.3 factors to satisfy the Provisions requirement. The bar in the panel is made continuous to the roof, therefore no calculation of development length is necessary in the panel. The dowel from the foundation will be hooked, otherwise the depth of the foundation would be more than required for structural reasons. The size of the foundation will provide adequate cover to allow the 0.7 factor on ACI’s standard development length for hooked bars. For the #9 bar:

\[
1.3(1.25)l_{dh} = \frac{(1.6.25)0.7(1200)d_h}{\sqrt{f'_c}} = \frac{1365(1.128)}{\sqrt{4000}} = 24.3 \text{ in.}
\]

Similarly, for the #10 bar, the length is 27.4 in.

Like many shear wall designs, this design does concentrate a demand for overturning resistance on the foundation. In this instance the resistance may be provided by a large footing (on the order of 20 ft by 28 ft by 3 ft thick) under the entire stairwell, or by deep piers or piles with an appropriate cap for load transfer. Refer to Chapter 4 for examples of design of each type of foundation, although not for this particular example.
7.2.5.2 Shear Connections and Reinforcement

Panel joints often are designed to resist the shear force by means of shear friction but that technique is not used for this example because the joint at the foundation will open due to flexural yielding. This opening would concentrate the shear stress on the small area of the dry-packed joint that remains in compression. This distribution can be affected by the shims used in construction. Tests have shown that this often leads to slip of the joint, which could lead to a kink in the principal tension reinforcement at or near its splice and destroy the integrity of the system. Therefore, the joint will be designed with direct shear connectors that will prevent slip along the joint. This is the authors’ interpretation of the Provisions text indicating that “Type Y connections shall develop under flexural, shear, and axial load actions, as required, a probable strength. . . .” based upon 125 percent of the specified yield in the connection. It would not be required by the ACI 318-02 rules for intermediate precast walls.
7.2.5.2.1 Longitudinal Direction

The shear amplification factor is determined as:

\[
\frac{M_{\text{capacity}}}{M_{\text{demand}}} = \frac{A_j (1.25) f_y j d + P_{\text{max}} j d / 2}{M_u} = \frac{(2.0 \text{ in.}^2)(1.25)(60 \text{ ksi})(23.5 \text{ ft}) + (397 \text{ kip})(23.5 \text{ ft} / 2)}{5320 \text{ ft-kip}}
\]

= 1.54

Therefore, the design shear \((V_u)\) at the base is 1.54(190 kips) = 292 kips

The base shear connection is shown in Figure 7.2-7 and is to be flexible vertically but stiff horizontally in the plane of the panel. The vertical flexibility is intended to minimize the contribution of these connections to overturning resistance, which would simply increase the shear demand.

![Diagram of the shear connection](image)

**Figure 7.2-7** Shear connection at base (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m).

In the panel, provide an assembly with two face plates 3/8 in. × 4 in. × 12 in. connected by a C8x18.75 and with diagonal #5 bars as shown in the figure. In the foundation provide an embedded plate 1/2 × 12 × 1'-6" with six 3/4 in. diameter headed anchor studs. In the field, weld an L 4 × 3 × 5/16 × 0'-8", long leg horizontal, on each face. The shear capacity of this connection is checked:
Shear in the two loose angles

\[ \phi V_n = \phi(0.6F_u)l(2) = (0.75)(0.6)(58\text{ ksi})(0.3125\text{ in.})(8\text{ in.})(2) = 130.5\text{ kip} \]

Weld at toe of loose angles

\[ \phi V_n = \phi(0.6F_u)tl(2) = (0.75)(0.6)(70\text{ ksi})(0.25\text{ in.})/(\sqrt{2})(8\text{ in.})(2) = 89.1\text{ kip} \]

Weld at face plates, using Table 8-9 in AISC Manual (3rd edition; same table is 8-42 in 2nd edition)

\[ \phi V_n = CC_1Dl(2 \text{ sides}) \]
\[ C_1 = 1.0 \text{ for E70 electrodes} \]
\[ l = 8\text{ in.} \]
\[ D = 4 \text{ (sixteenths of an inch)} \]
\[ k = 2\text{ in.} / 8\text{ in.} = 0.25 \]
\[ a = \text{eccentricity, summed vectorially: horizontal component is } 4\text{ in.; vertical component is } 2.67\text{ in.; thus, } a_l = 4.80\text{ in. and } a = 4.8\text{ in.}/8\text{ in.} = 0.6 \text{ from the table. By interpolation, } C = 1.29 \]
\[ \phi V_n = (1.29)(1.0)(4)(8)(2) = 82.6\text{ kip} \]

Weld from channel to plate has at least as much capacity, but less demand.

Bearing of concrete at steel channel

\[ f_c = \phi(0.85f' c) = 0.65(0.85)(5\text{ ksi}) = 2.76\text{ ksi} \]

The C8 has the following properties:

\[ t_w = 0.487\text{ in.} \]
\[ b_f = 2.53\text{ in.} \]
\[ t_f = 0.39\text{ in.} \text{ (average)} \]

The bearing will be controlled by bending in the web (because of the tapered flange, the critical flange thickness is greater than the web thickness). Conservatively ignoring the concrete’s resistance to vertical deformation of the flange, compute the width \( b \) of flange loaded at 2.76 ksi that develops the plastic moment in the web:

\[ M_p = \phi F_y t_w^2/4 = (0.9)(50\text{ ksi})(0.487^2\text{ in.}^2)/4 = 2.67\text{ in.-kip/in.} \]
\[ M_u = f_c[(b-t_f)^2/2 - (t_f)^2/2] = 2.76[(b - 0.243\text{ in.})^2 - (0.243\text{ in.})^2]/2 \]

setting the two equal results in \( b = 1.65\text{ in.} \)

Therefore bearing on the channel is

\[ \phi V_c = f_c(2 - t_u)(l) = (2.76\text{ ksi})[(2(1.65) - 0.487\text{ in.})(6\text{ in.}) = 46.6\text{ kip} \]

To the bearing capacity on the channel is added the 4 - #5 diagonal bars, which are effective in tension and compression; \( \phi = 0.75 \) for shear is used here:

\[ \phi V_s = \phi f_A \cos \alpha = (0.75)(60\text{ ksi})(4)(0.31\text{ in.}^2)(\cos 45^\circ) = 39.5\text{ kip} \]

Thus, the total capacity for transfer to concrete is:

\[ \phi V_n = \phi V_c + \phi V_s = 46.6 + 39.6 = 86.1\text{ kip} \]
The capacity of the plate in the foundation is governed by the headed anchor studs. The Provisions contain the new anchorage to concrete provisions that are in ACI 318-02 Appendix D. [In the 2003 Provisions, the anchorage to concrete provisions have been removed and replaced by the reference to ACI 318-02.] Capacity in shear for anchors located far from an edge of concrete, such as these, and with sufficient embedment to avoid the pryout failure mode is governed by the capacity of the steel:

\[ \phi V_s = \phi n A_{se} f_{ut} = (0.65)(6\text{ studs})(0.44 \text{ in.}^2\text{ per stud})(60 \text{ ksi}) = 103 \text{ kip} \]

Provisions Sec 9.2.3.3.2 (ACI 318-02 Sec. D.3.3.3) specifies an additional factor of 0.75 to derate anchors in structures assigned to Seismic Design Categories C and higher.

In summary the various shear capacities of the connection are:

- Shear in the two loose angles: 130.5 kip
- Weld at toe of loose angles: 89.1 kip
- Weld at face plates: 82.6 kip
- Transfer to concrete: 86.1 kip
- Headed anchor studs at foundation: 103 kip

The number of embedded plates (\( n \)) required for a panel is:

\[ n = \frac{292}{82.6} = 3.5 \]

Use four connection assemblies, equally spaced along each side (5'-0" on center works well to avoid the end reinforcement). The plates are recessed to position the #5 bars within the thickness of the panel and within the reinforcement of the panel.

It is instructive to consider how much moment capacity is added by the resistance of these connections to vertical lift at the joint. The vertical force at the tip of the angle that will create the plastic moment in the leg of the angle is:

\[ T = \frac{M_p}{x} = \frac{F_y l_t^2}{4(1-k)} = \frac{36 \text{ ksi}(8 \text{ in})(0.3125^2 \text{ in.}^2)/4}{(4 \text{ in.} - 0.69 \text{ in.})} = 2.12 \text{ kips} \]

There are four assemblies with two loose angles each, giving a total vertical force of 17 kips. The moment resistance is this force times half the length of the panel, which yields 212 ft-kips. The total demand moment, for which the entire system is proportioned, is 5320 ft - kips. Thus, these connections will add about 4% to the resistance and ignoring this contribution is reasonable. If a straight plate 1/4 in. x 8 in., which would be sufficient, were used and if the welds and foundation embedment did not fail first, the tensile capacity would be 72 kips each, a factor of 42 increase over the angles, and the shear connections would have the unintended effect of more than doubling the flexural resistance, which could easily cause failures in the system.

Using ACI 318 Sec. 11.10, check the shear strength of the precast panel at the first floor:

\[ \phi V_c = \phi 2A_{c_v} \sqrt{f_{ct}^c} \cdot h d = 0.85(2)\sqrt{5,000(8)(23.5)(12)} = 271 \text{ kips} \]

Because \( \phi V_c > V_u = 190 \text{ kips} \), the wall is adequate for shear without even considering the reinforcement. Note that the shear strength of wall itself is not governed by the overstrength required for the connection. However, since \( V_u > \phi V_c/2 = 136 \text{ kips} \), ACI Sec. 11.10.8 requires minimum wall reinforcement in accordance with ACI 318 Sec. 11.10.9.4 rather than Chapter 14 or 16. For the minimum required \( \rho_h = 0.0025 \), the required reinforcement is:
\[ A_v = 0.0025(8)(12) = 0.24 \text{ in.}^2/\text{ft} \]

As before, use two layers of welded wire fabric, WWF 4\times4 - W4.0\times W4.0, one on each face. Shear reinforcement provided, \[ A_v = 0.12(2) = 0.24 \text{ in.}^2/\text{ft} \]

Next, compute the shear strength at Level 2. Since the end reinforcement at the base extends to the top of the shear wall, bending is not a concern. Yield of the vertical bars will not occur, the second floor joint will not open (unlike at the base) and, therefore, shear friction could rationally be used to design the connections at this level and above. Shear keys in the surface of both panels would be advisable. Also, because of the lack of flexural yield at the joint, it is not necessary to make the shear connection be flexible with respect to vertical movement. To be consistent with the seismic force increase from yielding at the base, the shear at this level will be increased using the same amplification factor as calculated for the first story.

The design shear, \[ V_{u2} = 1.54(95 + 63.5) = 244 \text{ kips}. \]

Using the same recessed embedded plate assemblies in the panel as at the base, but welded with a straight plate, the number of plates, \[ n = 244/82.6 = 2.96. \] Use three plates, equally spaced along each side.

Figure 7.2-8 shows the shear connection at the second and third floors of the longitudinal precast concrete shear wall panels.

\textbf{Figure 7.2-8} Shear connections on each side of the wall at the second and third floors (1.0 in = 25.4 mm).
7.2.5.2.2 Transverse Direction

Use the same procedure as for the longitudinal walls:

\[
\frac{M_{\text{capacity}}}{M_{\text{demand}}} = \frac{A_y(1.25)f_yj'd + P_{\text{max}}j'd/2}{M_u} = \frac{(3.54 \text{ in.}^2)(1.25)(60 \text{ ksi})(13.5 \text{ ft}) + (148 \text{ kip})(23.5 \text{ ft} / 2)}{3052 \text{ ft-kip}} = 1.50
\]

Design shear, \( V_u \) at base is 1.50(105 kips) = 157.5 kips.

Use the same shear connections as at the base of the longitudinal walls (Figure 7.2-7). The connection capacity is 82.6 kips and the number of connections required is \( n = 157.5 / 82.6 = 1.9 \). Provide two connections on each panel.

Check the shear strength of the first floor panel as described previously:

\[
\phi V_c = \phi 2 \sqrt{f_y}hd = 0.85(2) \sqrt{5,000}(8)(13.5)(12) = 156 \text{ kips}
\]

Similar to the longitudinal direction, \( \phi V_c \geq V_u = 142 \text{ kips} \), but \( V_u \geq \phi V_c / 2 \) so provide two layers of welded wire fabric, WWF 4×4 - W4.0×W4.0, one on each face as in the longitudinal walls.

Compute the shear demand at the second floor level joint as indicated below.

The design shear, \( V_u = 1.50(52.3 + 34.9) = 130.8 \text{ kips} \).

Use the same plates as in the longitudinal walls. The number of plates, \( n = 130.8 / 82.6 = 1.6 \). Use two plates, equally spaced. Use the same shear connections for the transverse walls as for the longitudinal walls as shown in Figures 7.2-7 and 7.2-8.
7.3 ONE-STORY PRECAST SHEAR WALL BUILDING

This example illustrates precast shear wall seismic design using monolithic emulation as defined in the Provisions Sec. 9.1.1.12 [not applicable in the 2003 Provisions] (ACI Sec. 21.11.3) for a single-story building in a region of high seismicity. For buildings in Seismic Design Category D, Provisions Sec. 9.1.1.12 [not applicable in the 2003 Provisions] (ACI Sec. 21.11.2.1) requires that the precast seismic-force-resisting system emulate the behavior of monolithic reinforced concrete construction or that the system’s cyclic capacity be demonstrated by testing. This example presents an interpretation of monolithic emulation design with ductile connections. Here the connections in tension at the base of the wall panels yield by bending steel angles out-of-plane. The same connections at the bottom of the panel are detailed and designed to be very strong in shear and to resist the nominal shear strength of the concrete panel.

[Many of the provisions for precast concrete shear walls in areas of high seismicity have been moved out of the 2003 Provisions and into ACI 318-02. For structures assigned to Seismic Design Category D, 2003 Provisions Sec. 9.2.2.1.3 (ACI 318-02 Sec. 21.21.1.4) permits special precast concrete shear walls (ACI 318-02 Sec. 21.8) or intermediate precast concrete shear walls (ACI 318-02 Sec. 21.13). The 2003 Provisions does not differentiate between precast or cast-in-place concrete for special shear walls. This is because ACI 318-02 Sec. 21.8 essentially requires special precast concrete shear walls to satisfy the same design requirements as special reinforced concrete shear walls (ACI 318-02 Sec. 21.7). Alternatively, special precast concrete shear walls are permitted if they satisfy experimental and analytical requirements contained in 2003 Provisions Sec. 9.2.2.4 and 9.6.]

7.3.1 Building Description

The precast concrete building is a single-story industrial warehouse building (Seismic Use Group I) located in the Los Angeles area on Site Class C soils. The structure has 8-ft-wide by 12-1/2-in.-deep prestressed double tee (DT) wall panels. The roof is light gage metal decking spanning to bar joists that are spaced at 4 ft on center to match the location of the DT legs. The center supports for the joists are joist girders spanning 40 ft to steel tube columns. The vertical seismic-force-resisting system is the precast/prestressed DT wall panels located around the perimeter of the building. The average roof height is 20 ft, and there is a 3 ft parapet. The building is located in the Los Angeles area on Site Class C soils. Figure 7.3-1 shows the plan of the building, which is regular.
The precast wall panels used in this building are typical DT wall panels commonly found in many locations but not normally used in Southern California. For these wall panels, an extra 1/2 in. has been added to the thickness of the deck (flange). This extra thickness is intended to reduce cracking of the flanges and provide cover for the bars used in the deck at the base. The use of thicker flanges is addressed later.

Provisions Sec. 9.1.1.5 [9.2.2.1.5.4] (ACI Sec. 21.2.5.1 [21.2.5.1]) limits the grade and type of reinforcement in boundary elements of shear walls and excludes the use of bonded prestressing tendons (strand) due to seismic loads. ACI 318-99 Sec. 21.7.5.2 [21.9.5.2] permits the use of strand in boundary elements of diaphragms provided the stress is limited to 60,000 psi. This design example uses the strand as the reinforcement based on that analogy. The rationale for this is that the primary reinforcement of the DT, the strand, is not working as the ductile element of the wall panel and is not expected to yield in an earthquake.

The wall panels are normal-weight concrete with a 28-day compressive strength, $f'_{c} = 5,000$ psi. Reinforcing bars used in the welded connections of the panels and footings are ASTM A706 (60 ksi). The concrete for the foundations has a 28-day compressive strength, $f'_{c} = 4,000$ psi.
7.3.2  Design Requirements

7.3.2.1  Seismic Parameters of the Provisions

The basic parameters affecting the design and detailing of the building are shown in Table 7.3-1.

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Use Group I</td>
<td>$I = 1.0$</td>
</tr>
<tr>
<td>$S_S$ (Map 1 [Figure 3.3-1])</td>
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</tr>
<tr>
<td>$S_I$ (Map 2 [Figure 3.3-2])</td>
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</tr>
<tr>
<td>Site Class</td>
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</tr>
<tr>
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<td>1.0</td>
</tr>
<tr>
<td>$F_v$</td>
<td>1.3</td>
</tr>
<tr>
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</tr>
<tr>
<td>$S_{MI} = F_v S_I$</td>
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</tr>
<tr>
<td>$S_{DS} = 2/3 S_{MS}$</td>
<td>1.0</td>
</tr>
<tr>
<td>$S_{DI} = 2/3 S_{MI}$</td>
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</tr>
<tr>
<td>Seismic Design Category</td>
<td>D</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basic Seismic-Force-Resisting System</th>
<th>Bearing Walls System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Type *</td>
<td>Special Reinforced Concrete Shear Wall</td>
</tr>
<tr>
<td>$R$</td>
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</tr>
<tr>
<td>$\Omega_0$</td>
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</tr>
<tr>
<td>$C_d$</td>
<td>5</td>
</tr>
</tbody>
</table>

* Provisions Sec. 9.7.1.2 [9.2.2.1.3] requires special reinforced concrete shear walls in Seismic Design Category D and requires adherence to the special seismic design provisions of ACI 318 Chapter 21.

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

7.3.2.2  Structural Design Considerations

7.3.2.2.1  Precast Shear Wall System

The criteria for the design is to provide yielding in a dry connection for bending at the base of each precast shear wall panel while maintaining significant shear resistance in the connection. The flexural connection for a wall panel at the base is located in one DT leg while the connection at the other leg is used for compression. Per Provisions Sec. 9.1.1.12 (ACI Sec. 21.11.3.1) [not applicable in the 2003 Provisions], these connections resist the shear force equal to the nominal shear strength of the panel and have a nominal strength equal to twice the shear that exists when the actual moment is equal to $M_{Pr}$.
(which ACI defines as \( \phi = 1.0 \) and a steel stress equal to 125% of specified yield). Yielding will develop in the dry connection at the base by bending the horizontal leg of the steel angle welded between the embedded plates of the DT and footing. The horizontal leg of this angle is designed in a manner to resist the seismic tension of the shear wall due to overturning and then yield and deform inelastically. The connections on the two legs of the DT are each designed to resist 50 percent of the shear. The anchorage of the connection into the concrete is designed to satisfy the Type Z requirements in Provisions Sec. 9.1.1.12 (ACI Sec. 21.11.6.5) [not applicable in the 2003 Provisions]. Careful attention to structural details of these connections is required to ensure tension ductility and resistance to large shear forces that are applied to the embedded plates in the DT and footing.

[Based on the 2003 Provisions, unless the design of special precast shear walls is substantiated by experimental evidence and analysis per 2003 Provisions Sec. 9.2.2.4 (ACI 318-02 Sec. 21.8.2), the design must satisfy ACI 318-02 Sec. 21.7 requirements for special structural walls as referenced by ACI 318-02 Sec. 21.8.1. The connection requirements are not as clearly defined as in the 2000 Provisions.]

7.3.2.2 Building System

Height limit is 160 ft (Provisions Table 5.2.2 [4.3-1]).

The metal deck roof acts as a flexible horizontal diaphragm to distribute seismic inertia forces to the walls parallel to the earthquake motion (Provisions Sec. 5.2.3.1 [4.3.2.1]).

The building is regular both in plan and elevation.

The reliability factor, \( \rho \) is computed in accordance with Provisions Sec. 5.2.4.2 [4.3.3]. The maximum \( \rho \) value is given when \( r_{\text{max}} \) is the largest value. \( r_{\text{max}} \) is the ratio of design story shear resisted by the single element carrying the most shear force to the total story shear. All shear wall elements (8-ft-wide panels) have the same stiffness. Therefore, the shear in each element is the total shear along a side divided by the number of elements (wall panels). The largest \( r_{\text{max}} \) value is along the side with the least number of panels. Along the side with 11 panels, \( r_{\text{max}} \) is computed as:

\[
r_{\text{max}} = \frac{1/2/11}{1.0} = 0.0455
\]

\[
A_x = 96 \text{ ft} \times 120 \text{ ft} = 11,520 \text{ ft}^2
\]

\[
\rho_x = 2 - \frac{20}{r_{\text{max}} \sqrt{A_x}} = 2 - \frac{20}{0.0455 \sqrt{11,520}} = -2.10
\]

Therefore, use \( \rho = 1.0 \).

[The redundancy requirements have been substantially changed for the 2003 Provisions. 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 single shear wall with an aspect ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Based on the design procedures for the walls, each individual panel should be considered a separate wall with an aspect ratio greater than 1.0. Alternatively, if the structure is regular in plan and there are at least two bays of perimeter framing on each side of the structure in each orthogonal direction, the exception in 2003 Provisions Sec. 4.3.3.2]
permits the use of \( D, \rho = 1.0 \). This exception could be interpreted as applying to this example, which is regular and has more than two wall panels (bays) in both directions.]

The structural analysis to be used is the ELF procedure (Provisions Sec. 5.4 [5.2]) as permitted by Provisions Table 5.2.5 [4.4-1].

Orthogonal load combinations are not required for flexible diaphragms in Seismic Design Category D (Provisions Sec. 5.2.5.2.3 [4.4.2.3]).

This example does not include design of the foundation system, the metal deck diaphragm, or the nonstructural elements.

Ties, continuity, and anchorage (Provisions 5.2.6.1 through 5.2.6.4 [4.6]) must be explicitly considered when detailing connections between the roof and the wall panels. This example does not include the design of these connections, but sketches of details are provided to guide the design engineer.

There are no drift limitations for single-story buildings as long as they are designed to accommodate predicted lateral displacements (Provisions Table 5.2.8, footnote b [4.5-1, footnote c]).

**7.3.3 Load Combinations**

The basic load combinations (Provisions Sec. 5.2.7) require that seismic forces and gravity loads be combined in accordance with the factored load combinations as presented in ASCE 7, except that the load factor for earthquake effects \( E \) is defined by Provisions Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2]:

\[
E = \rho Q_E \pm 0.2SDSD = (1.0)Q_E \pm (0.2)(1.0)D = Q_E \pm 0.2D
\]

The relevant load combinations from ASCE 7 are:

\[
\begin{align*}
1.2D &\pm 1.0E + 0.5L \\
0.9D &\pm 1.0E
\end{align*}
\]

Note that roof live load need not be combined with seismic loads, so the live load term, \( L \), can be omitted from the equation.

Into each of these load combinations, substitute \( E \) as determined above:

\[
\begin{align*}
1.4D &+ Q_E \\
1.0D - Q_E & \quad \text{(will not control)} \\
1.1D + Q_E & \quad \text{(will not control)} \\
0.7D - Q_E
\end{align*}
\]

These load combinations are for the in-plane direction of the shear walls.
7.3.4 Seismic Force Analysis

7.3.4.1 Weight Calculations

Compute the weight tributary to the roof diaphragm

Roofing = 2.0 psf  
Metal decking = 1.8 psf  
Insulation = 1.5 psf  
Lights, mechanical, sprinkler system etc. = 3.2 psf  
Bar joists = 2.7 psf  
Joist girder and columns = 0.8 psf  
Total = 12.0 psf

The total weight of the roof is computed as:

\[(120 \text{ ft} \times 96 \text{ ft})(12 \text{ psf}/1,000) = 138 \text{ kips}\]

The exterior double tee wall weight tributary to the roof is:

\[(20 \text{ ft}/2 + 3 \text{ ft}][42 \text{ psf}/1,000](120 \text{ ft} + 96 \text{ ft})^2 = 236 \text{ kips}\]

Total building weight for seismic lateral load, \(W = 138 + 236 = 374 \text{ kips}\)

7.3.4.2 Base Shear

The seismic response coefficient \((C_s)\) is computed using Provisions Eq. 5.4.1.1-1 [5.2-2] as:

\[C_s = \frac{S_{DO}}{R/I} = \frac{1.0}{5/1} = 0.20\]

except that it need not exceed the value from Provisions Eq. 5.4.1.1-2 [5.2-3] as follows:

\[C_s = \frac{S_{DI}}{T(R/I)} = \frac{0.52}{0.189(5/1)} = 0.55\]

where \(T\) is the fundamental period of the building computed using the approximate method of Provisions Eq. 5.4.2.1-1 [5.2-6]:

\[T_p = C_s h_n^x = (0.02)(20.0)^{0.75} = 0.189 \text{ sec}\]

Therefore, use \(C_s = 0.20\), which is larger than the minimum specified in Provisions Eq. 5.4.1.1-3 [not applicable in the 2003 Provisions]:

\[C_s = 0.044IS_{DO} = (0.044)(1.0)(1.0) = 0.044\]

[The minimum \(C_s\) value has been changed to 0.01 in. the 2003 Provisions.

The total seismic base shear is then calculated using Provisions Eq. 5.4-1 [5.2-1] as:
7.3.4.3 Horizontal Shear Distribution and Torsion

Torsion is not considered in the shear distribution in buildings with flexible diaphragms. The shear along each side of the building will be equal, based on a tributary area force distribution.

7.3.4.3.1 Longitudinal Direction

The total shear along each side of the building is \( V/2 = 37.4 \) kips. The maximum shear on longitudinal panels (at the side with the openings) is:

\[
V_{lu} = \frac{37.4}{11} = 3.4 \text{ kips}
\]

On each side, each longitudinal wall panel resists the same shear force as shown in the free-body diagram of Figure 7.3-2, where \( D_1 \) represents roof joist reactions and \( D_2 \) is the panel weight.

\[ V = C_s W = (0.20)(374) = 74.8 \text{ kips} \]
7.3.4.3.2 Transverse Direction

Seismic forces on the transverse wall panels are all equal and are:

\[ V_{tu} = \frac{37.4}{12} = 3.12 \text{ kips} \]

Figure 7.3-3 shows the transverse wall panel free-body diagram.

Note the assumption of uniform distribution to the wall panels in a line requires that the roof diaphragm be provided with a collector element along its edge. The chord designed for diaphragm action in the perpendicular direction will normally be capable of fulfilling this function, but an explicit check should be made in the design.

Figure 7.3-3 Free-body diagram of a panel in the transverse direction (1.0 ft = 0.3048 m).

7.3.5 Proportioning and Detailing

The strength of members and components is determined using the strengths permitted and required in ACI 318 including Chapter 21.
7.3.5.1 Tension and Shear Forces at the Panel Base

Design each precast shear panel to resist the seismic overturning moment by means of a ductile tension connector at the base of the panel. A steel angle connector will be provided at the connection of each leg of the DT panel to the concrete footing. The horizontal leg of the angle is designed to yield in bending as needed in an earthquake. *Provisions* Sec. 9.1.1.12 [not applicable in the 2003 *Provisions*] requires that dry connections at locations of nonlinear action comply with applicable requirements of monolithic concrete construction and satisfy the following:

1. Where the moment action on the connection is assumed equal to $M_{pr}$, the co-existing shear on the connection shall be no greater than $0.5S_{nConnection}$ and

2. The nominal shear strength for the connection shall not be less than the shear strengths of the members immediately adjacent to that connection.

Precisely how ductile dry connections emulate monolithic construction is not clearly explained. The dry connections used here do meet the definition of a yielding steel element at a connection contained in ACI 318-02. For the purposes of this example, these two additional requirements are interpreted as:

1. When tension from the seismic overturning moment causes 1.25 times the yield moment in the angle, the horizontal shear on this connection shall not exceed one-half the nominal shear strength of the connection. For this design, one-half the total shear will be resisted by the angle at the DT leg in tension and the remainder by the angle at the DT leg in compression.

2. The nominal shear strength of the connections at the legs need to be designed to exceed the in-plane shear strength of the DT.

Determine the forces for design of the DT connection at the base.

7.3.5.1.1 Longitudinal Direction

Use the free-body diagram shown in Figure 7.3-2. The maximum tension for the connection at the base of the precast panel to the concrete footing is governed by the seismic overturning moment and the dead loads of the panel and the roof. The weight for the roof is 11.2 psf, which excludes the joist girders and columns.

At the base

$$M_E = (3.4 \text{ kips})(20 \text{ ft}) = 68.0 \text{ ft-kips}$$

Dead loads

$$D_1 = \left(11.2/1,000\right)\left(\frac{48}{2}\right)4 = 1.08 \text{ kips}$$

$$D_2 = 0.042(23)(8) = 7.73 \text{ kips}$$

$$\Sigma D = 2(1.08) + 7.73 = 9.89 \text{ kips}$$

$$1.4D = 13.8 \text{ kips}$$

$$0.7D = 6.92 \text{ kips}$$

Compute the tension force due to net overturning based on an effective moment arm, $d = 4.0 \text{ ft}$ (distance between the DT legs). The maximum is found when combined with $0.7D$: 
\[ T_u = M_d d - 0.7D/2 = 68.0/4 - 6.92/2 = 13.5 \text{ kips} \]

### 7.3.5.1.2 Transverse Direction

For the transverse direction, use the free-body diagram of Figure 7.3-3. The maximum tension for connection at the base of the precast panel to the concrete footing is governed by the seismic overturning moment and the dead loads of just the panel. No load from the roof is included, since it is negligible.

At the base
\[ M_E = (3.12 \text{ kips})(20 \text{ ft}) = 62.4 \text{ ft-kips} \]

The dead load of the panel (as computed above) is \( D_2 = 7.73 \text{ kips} \), and \( 0.7D = 5.41 \).

The tension force is computed as above for \( d = 4.0 \text{ ft} \) (distance between the DT legs):
\[ T_u = 62.4/4 - 5.41/2 = 12.9 \text{ kips} \]

This tension force is less than that at the longitudinal wall panels. Use the tension force of the longitudinal wall panels for the design of the angle connections.

### 7.3.5.2 Panel Reinforcement

Check the maximum compressive stress in the DT leg for the requirement of transverse boundary element reinforcement per ACI 318 Sec. 21.6.6.3 [21.7.6.3]. Figure 7.3-4 shows the cross section used. The section is limited by the area of dry-pack under the DT at the footing.

The reason to limit the area of dry-pack at the footing is to locate the boundary elements in the legs of the DT, at least at the bottom of the panel. The flange between the legs of the DT is not as susceptible to cracking during transportation as are the corners of DT flanges outside the confines of the legs. The compressive stress due to the overturning moment at the top of the footing and dead load is:

\[ \sigma = \frac{P}{A} + \frac{M_E}{S} = \frac{13,800}{227} + \frac{12(68,000)}{3,240} = 313 \text{ psi} \]

Roof live loads need not be included as a factored axial load in the compressive stress check, but the force from the prestress steel will be added to the compression stress above because the prestress force will be effective a few feet above the base and will add compression to the DT leg. Each leg of the DT will be reinforced with one 1/2-in. diameter and one 3/8-in. diameter strand. Figure 7.3-5 shows the location of these prestressed strands.
Next, compute the compressive stress resulting from these strands. Note the moment at the height of strand development above the footing, about 26 in. for the effective stress \( f_{se} \), is less than at the top of footing. This reduces the compressive stress by:

\[
\frac{(3.4)(26)}{3,240} \times 1000 = 27 \text{ psi}
\]

In each leg, use

\[
P = 0.58f_{pu}A_{ps} = 0.58(270 \text{ ksi})(0.153 + 0.085) = 37.3 \text{ kips}
\]

\[
A = 168 \text{ in.}^2
\]

\[
e = y_h - CG_{strand} = 9.48 - 8.57 = 0.91 \text{ in.}
\]

\[
S_b = 189 \text{ in.}^3
\]

\[
\sigma = \frac{P}{A} + \frac{Pe}{S} = \frac{37,300}{168} + \frac{0.91(37,300)}{189} = 402 \text{ psi}
\]

Therefore, the total compressive stress is approximately 313 + 402 - 27 = 688 psi.

The limiting stress is 0.2 \( f_{'c} \), which is 1000 psi, so no special boundary elements are required in the longitudinal wall panels.

Reinforcement in the DT for tension is checked at 26 in. above the footing. The strand reinforcement of the DT leg resisting tension is limited to 60,000 psi. The rationale for using this stress is discussed at the beginning of this example.

\[
D_2 = (0.042)(20.83)(8) = 7.0 \text{ kips}
\]
\[ P_{\text{min}} = 0.7(7.0 + 2(1.08)) = 6.41 \text{ kips} \]
\[ M_e = (3.4)(17.83) = 60.6 \text{ ft-kips} \]
\[ T_u = M_{net}/d - P_{\text{min}}/2 = 12.0 \text{ kips} \]

The area of tension reinforcement required is:
\[ A_s = T_u/\phi_{fy} = (12.0 \text{ kips})/[0.9(60 \text{ ksi})] = 0.22 \text{ in.}^2 \]

The area of one ½ in. diameter and one 3/8 in. diameter strand is 0.153 in.\(^2\) + 0.085 in.\(^2\) = 0.236 in.\(^2\). The mesh in the legs is available for tension resistance, but not required in this check.

To determine the nominal shear strength of the concrete for the connection design, complete the shear calculation for the panel in accordance with ACI Sec. 21.6 [21.7]. The demand on each panel is:
\[ V_u = V_{lu} = 3.4 \text{ kips} \]

Only the deck between the DT legs is used to resist the in-plane shear (the legs act like flanges, meaning that the area effective for shear is the deck between the legs). First, determine the minimum required shear reinforcement based on ACI Sec. 21.6.2.1 [21.7.2]. Since
\[ A_{sh} \sqrt{f_c'} = 2.5(48) \sqrt{5,000} = 8.49 \text{ kips} \]

exceeds \( V_u = 3.4 \text{ kips} \), the reinforcement of the deck is per ACI 318 Sec. 16.4.2. Using welded wire fabric, the required areas of reinforcement are:
\[ A_{sh} = (0.001)(2.5)(12) = 0.03 \text{ in.}^2/\text{ft} \]

Provide 6 × 6 - W2.5 × W2.0 welded wire fabric.
\[ A_{sh} = 0.05 \text{ in.}^2/\text{ft} \]
\[ A_{sw} = 0.04 \text{ in.}^2/\text{ft} \]

The nominal shear strength of the wall panel by ACI 318 Sec. 21.6.4.1 is:
\[ V_n = A_{cv} \left( \alpha_c \sqrt{f_c'} + \rho_n f_y \right) = (2.5)(48) \frac{2 \sqrt{5,000}}{1,000} + 0.05(4)(60) = 29.0 \text{ kips} \]

where \( \alpha_c \) is 2.0 for \( h_w/l_w = 23/4 = 5.75 \), which is greater than 2.0. Given that the connections will be designed for a shear of 29 kips, it is obvious that half the nominal shear strength will exceed the seismic shear demand, which is 3.4 kips.

The prestress force and the area of the DT legs are excluded from the calculation of the nominal shear strength of the DT wall panel. The prestress force is not effective at the base, where the connection is, and the legs are like the flanges of a channel, which are not effective in shear.

7.3.5.3 Size the Yielding Angle

The angle, which is the ductile element of the connection, is welded between the plates embedded in the DT leg and the footing. This angle is a L5 × 3-1/2 × 3/4 × 0 ft-5 in. with the long leg vertical. The steel for the angle and embedded plates will be ASTM A572, Grade 50. The horizontal leg of the angle needs to be long enough to provide significant displacement at the roof, although this is not stated as a
requirement in either the *Provisions* or ACI 318. This will be examined briefly here. The angle and its welds are shown in Figure 7.3-6.

![Figure 7.3-6](image)

Figure 7.3-6 Free-body of the angle and the fillet weld connecting the embedded plates in the DT and the footing (elevation and section) (1.0 in = 25.4 mm).

The bending moment at a distance \( k \) from the heel of the angle (location of the plastic hinge in the angle) is:

\[
M_u = T_u (3.5 - k) = 13.5(3.5 - 1.25) = 30.4 \text{ in.-kips}
\]

Providing a stronger angle (e.g., a shorter horizontal leg) will simply increase the demands on the remainder of the assembly. Using *Provisions* Sec. 9.1.1.12 (ACI Sec. 21.11.6.5) [not applicable in the 2003 *Provisions*], the tension force for the remainder of this connection other than the angle is based upon a probable strength equal to 140% of the nominal strength. Thus

\[
T''_u = \frac{M_n (1.4)}{3.5 - k} = \frac{(50)(5)(0.75)^2}{3.5 - 1.25} \times 1.4 = 21.9 \text{ kips}
\]

Check the welds for the tension force of 21.9 kips and a shear force \((V'_u)\) of 29.0/2 = 14.5 kips, or the shear associated with \(T'_u\), whichever is greater. The bearing panel, with its larger vertical load, will give a larger shear.
1.4D = 13.8 kips, and 

\[ V = \frac{[T_u(4) + 1.4D(2)]}{20} = \frac{[21.9/4 + 13.8(2)]}{20} = 5.76 \text{ kips}. \]

\( V_u \) for the panel obviously controls.

But before checking the welds, consider the deformability of the system as controlled by the yielding angle. Ignore all sources of deformation except the angle. (This is not a bad assumption regarding the double tee itself, but other aspects of the connections, particularly the plate and reinforcement embedded in the DT, will contribute to the overall deformation. Also, the diaphragm deformation will overwhelm all other aspects of deformation, but this is not the place to address flexible diaphragm issues.) The angle deformation will be idealized as a cantilever with a length from the tip to the center of the corner, then upward to the level of the bottom of the DT, which amounts to:

\[ L = 3.5 \text{ in.} - t/2 + 1 \text{ in.} - t/2 = 3.75 \text{ in.} \]

Using an elastic-plastic idealization, the vertical deformation at the design moment in the leg is

\[ \delta_v = TL^3/3EI = (13.5 \text{ kips})(3.75 \text{ in.})^3/[3(29000 \text{ ksi})(5 \text{ in.})(0.75 \text{ in.})^3/12] = 0.047 \text{ in.} \]

This translates into a horizontal motion at the roof of 0.24 in. (20 ft to the roof, divided by the 4 ft from leg to leg at the base of the DT.) With \( C_d \) of 4, the predicted total displacement is 0.96 in. These displacements are not very large, but now compare with the expectations of the Provisions. The approximate period predicted for a 20-ft-tall shear wall building is 0.19 sec. Given a weight of 374 kips, as computed previously, this would imply a stiffness from the fundamental equation of dynamics:

\[ T = 2\pi \sqrt{\frac{W/g}{K}} \Rightarrow K = 4\pi^2 W/(gT) = 4\pi^2 374/(386 \times 0.19) = 201 \text{ kip/in.} \]

Now, given the design seismic base shear of 74.8 kips, this would imply an elastic displacement of

\[ \delta_h = 74.8 \text{ kip} / (201 \text{ kip/in.}) = 0.37 \text{ in.} \]

This is about 50% larger than the simplistic calculation considering only the angle. The bending of angle legs about their weak axis has a long history of providing ductility and, thus, it appears that this dry connection will provide enough deformability to be in the range of expectation of the Provisions.

### 7.3.5.4 Welds to Connection Angle

Welds will be fillet welds using E70 electrodes.

For the base metal, \( \phi R_u = \phi(F_y)A_{BM} \).

For which the limiting stress is \( \phi F_y = 0.9(50) = 45.0 \text{ ksi.} \)

For the weld metal, \( \phi R_w = \phi(F_y)A_w = 0.75(0.6)70(0.707)A_w \).

For which limiting stress is 22.3 ksi.

Size a fillet weld, 5 in. long at the angle to embedded plate in the footing:

Using an elastic approach

\[ \text{Resultant force} = \sqrt{V^2 + T^2} = \sqrt{14.5^2 + 21.9^2} = 26.3 \text{ kips} \]
\[ A_w = \frac{26.3}{22.3} = 1.18 \text{ in.}^2 \]
\[ t = \frac{A_w}{l} = \frac{1.18 \text{ in.}^2}{5 \text{ in.}} = 0.24 \text{ in.} \]

For a 3/4 in. angle leg, use a 5/16 in. fillet weld. Given the importance of this weld, increasing the size to 3/8 in. would be a reasonable step. With ordinary quality control to avoid flaws, increasing the strength of this weld by such an amount should not have a detrimental effect elsewhere in the connection.

Now size the weld to the plate in the DT. Continue to use the conservative elastic method to calculate weld stresses. Try a fillet weld 5 in. long across the top and 4 in. long on each vertical leg of the angle. Using the free-body diagram of Figure 7.3-6 for tension and Figure 7.3-7 for shear, the weld moments and stresses are:

\[ M_x = T_y (3.5) = 21.9 (3.5) = 76.7 \text{ in.-kips} \]
\[ M_y = V_x (3.5) = (14.5)(3.5) = 50.8 \text{ in.-kips} \]
\[ M_z = V_x (y_b + 1.0) = 14.5 (2.77 + 1.0) = 54.7 \text{ in.-kips} \]

For the weld between the angle and the embedded plate in the DT as shown in Figure 7.3-7 the section properties for a weld leg (t) are:

\[ A = 13t \text{ in.}^2 \]
\[ I_x = 23.0t \text{ in.}^4 \]
\[ I_y = 60.4t \text{ in.}^4 \]
\[ I_p = I_x + I_y = 83.4\text{ in.}^4 \]
\[ y_b = 2.77\text{ in.} \]
\[ x_L = 2.5\text{ in.} \]

To check the weld, stresses are computed at all four ends (and corners). The maximum stress is at the lower right end of the inverted U shown in Figure 7.3-6.

\[ \sigma_x = \frac{V_u'}{A} + \frac{M_z y_b}{I_p} = \frac{14.5}{13t} + \frac{(54)(2.77)}{83.4t} = \left( \frac{2.93}{t} \right) \text{ksi} \]

\[ \sigma_y = -\frac{T_u'}{A} + \frac{M_z x_L}{I_p} = -\frac{21.9}{13t} + \frac{(54.7)(2.5)}{83.4t} = \left( \frac{0.045}{t} \right) \text{ksi} \]

\[ \sigma_z = -\frac{M_x x_L}{I_y} + \frac{M_x y_b}{I_x} = \frac{(50.8)(2.5)}{60.4t} + \frac{(76.7)(2.77)}{23.0t} = \left( \frac{11.3}{t} \right) \text{ksi} \]

\[ \sigma_R = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2} = \frac{1}{t} \sqrt{(2.93)^2 + (0.045)^2 + (11.3)^2} = \left( \frac{11.67}{t} \right) \text{ksi} \]

Thus, \( t = 11.67/22.3 = 0.52\text{ in.} \), say 9/16 in. Field welds are conservatively sized with the elastic method for simplicity and to minimize construction issues.

### 7.3.5.5 Tension and Shear at the Footing Embedment

Reinforcement to anchor the embedded plates is sized for the same tension and shear, and the development lengths are lengthened by an additional 30%, per Provisions Sec. 9.1.1.12 (ACI Sec. 21.11.6.5) [not applicable in the 2003 Provisions]. Reinforcement in the DT leg and in the footing will be welded to embedded plates as shown in Figure 7.3-8.

The welded reinforcement is sloped to provide concrete cover and to embed the bars in the central region of the DT leg and footing. The tension reinforcement area required in the footing is:

\[ A_{s,\text{Sloped}} = \frac{T_u'}{\phi f_y' \cos \theta} = \frac{21.9}{0.9(60)(\cos 26.5^\circ)} = 0.45\text{ in.}^2 \]

Use two #5 bars \((A_s = 0.62\text{ in.}^2)\) at each embedded plate in the footing.

The shear bars in the footing will be two #4 placed on an angle of two (plus)-to-one. The resultant shear resistance is:

\[ \phi V_n = 0.75(0.2)(2)(60)(\cos 26.5^\circ) = 16.1\text{ kips} \]
Figure 7.3-8 Section at the connection of the precast/prestressed shear wall panel and the footing (1.0 in = 25.4 mm).

### 7.3.5.6 Tension and Shear at the DT Embedment

The area of reinforcement for the welded bars of the embedded plate in the DT, which develop tension as the angle bends through cycles is:

\[
A_s = \frac{T'_u}{\phi f_y \cos \theta} = \frac{21.9}{0.9(60) \cos 6.3^\circ} = 0.408 \text{ in.}^2
\]

Two #4 bars are adequate. Note that the bars in the DT leg are required to extend upward 1.3 times the development length, which would be 22 in. In this case they will be extended 22 in. past the point of development of the effective stress in the strand, which totals about 48 in.

The same embedded plate used for tension will also be used to resist one-half the nominal shear. This shear force is 14.5 kips. The transfer of direct shear to the concrete is easily accomplished with bearing on the sides of the reinforcing bars welded to the plate. Two #5 and two #4 bars (explained later) are welded to the plate. The available bearing area is approximately \( A_{br} = 4(0.5 \text{ in.})(5 \text{ in.}(\text{available})) = 10 \text{ in.}^2 \) and the bearing capacity of the concrete is \( \phi V_n = (0.65)(0.85)(5 \text{ ksi})(10 \text{ in.}^2) = 27.6 \text{ kips} > 14.5 \text{ kip demand} \).
The weld of these bars to the plate must develop both the tensile demand and this shear force. The weld is a flare bevel weld, with an effective throat of 0.2 times the bar diameter along each side of the bar. (Refer to the PCI Handbook.) For the #4 bar, the weld capacity is

$$\phi V_n = (0.75)(0.6)(70 \text{ ksi})(0.2)(0.5 \text{ in.})(2) = 6.3 \text{ kips/in.}$$

The shear demand is prorated among the four bars as (14.5 kip)/4 = 3.5 kip. The tension demand is the larger of 1.25 $f_y$ on the bar (15 kip) or $T_u/2$ (11.0 kip). The vectorial sum of shear and tension demand is 15.4 kip. Thus, the minimum length of weld is $15.4 / 6.3 = 2.4$ in.

### 7.3.5.7 Resolution of Eccentricities at the DT Embedment

Check the twisting of the embedded plate in the DT for $M_z$.

Use $M_z = 54.7$ in.-kips.

$$A_s = \frac{M_z}{\phi f_y (jd)} = \frac{54.7}{0.9(60)(9.0)} = 0.11 \text{ in.}^2$$

Use one #4 bar on each side of the vertical embedded plate in the DT as shown in Figure 7.3-9. This is the same bar used to transfer direct shear in bearing.

Check the DT embedded plate for $M_y$ (50.8 in.-kips) and $M_x$ (76.7 in.-kips) using the two #4 bars welded to the back side of the plate near the corners of the weld on the loose angle and the two #3 bars welded to the back side of the plate near the bottom of the DT leg (as shown in Figure 7.3-9). It is relatively straightforward to compute the resultant moment magnitude and direction, assume a triangular shaped compression block in the concrete, and then compute the resisting moment. It is quicker to make a reasonable assumption as to the bars that are effective and then compute resisting moments about the X and Y axes. This approximate method is demonstrated here. The #4 bars are effective in resisting $M_x$, and one each of the #3 and #4 bars are effective in resisting $M_y$. For $M_y$ assume that the effective depth extends 1 in. beyond the edge of the angle (equal to twice the thickness of the plate). Begin by assigning one-half of the “corner” #4 to each component.

With $A_{xs} = 0.20 + 0.20/2 = 0.30 \text{ in.}^2$,

$$\phi M_{xs} = \phi A_{xs} f_y jd = (0.9)(0.3 \text{ in.}^2)(60 \text{ ksi})(0.95)(5 \text{ in.}) = 77 \text{ in.-kips} (>76.7).$$

With $A_{ys} = 0.11 + 0.20/2 = 0.21 \text{ in.}^2$,

$$\phi M_{ys} = \phi A_{ys} f_y jd = (0.9)(0.21 \text{ in.}^2)(60 \text{ ksi})(0.95)(5 \text{ in.}) = 54 \text{ in.-kips} (>50.8).$$

Each component is strong enough, so the proposed bars are satisfactory.
Figure 7.3-9 Details of the embedded plate in the DT at the base (1.0 in = 25.4 mm).

Figure 7.3-11 Sketch of connection of load-bearing DT wall panel at the roof (1.0 in = 25.4 mm).
7.3.5.8 Other Connections

This design assumes that there is no in-plane shear transmitted from panel to panel. Therefore, if connections are installed along the vertical joints between DT panels to control the out-of-plane alignment, they should not constrain relative movement in-plane. In a practical sense, this means the chord for the roof diaphragm should not be a part of the panels. Figures 7.3-10 and 7.3-11 show the connections at the roof and DT wall panels. These connections are not designed here. Note that the continuous steel angle would be expected to undergo vertical deformations as the panels deform laterally.

Because the diaphragm supports concrete walls out of their plane, Provisions Sec. 5.2.6.3.2 [4.6.2.1] requires specific force minimums for the connection and requires continuous ties across the diaphragm. Also, it specifically prohibits use of the metal deck as the ties in the direction perpendicular to the deck span. In that direction, the designer may wish to use the top chord of the bar joists, with an appropriate connection at the joist girder, as the continuous cross ties. In the direction parallel to the deck span, the deck may be used but the laps should be detailed appropriately.

In precast double tee shear wall panels with flanges thicker than 2-1/2 in., consideration may be given to using vertical connections between the wall panels to transfer vertical forces resulting from overturning moments and thereby reduce the overturning moment demand. These types of connections are not considered here, since the uplift force is small relative to the shear force and cyclic loading of bars in thin concrete flanges is not always reliable in earthquakes.

![Figure 7.3-10](image_url) Sketch of connection of non-load-bearing DT wall panel at the roof \((1.0\,\text{in} = 25.4\,\text{mm}, 1.0\,\text{ft} = 0.3048\,\text{m})\).