8

Precast Concrete Design

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Contents

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. Over the past several years there has been a concerted effort to coordinate the requirements in the *Provisions* with those in ACI 318, so that now there are very few differences between the two. Very briefly, the *Provisions* set forth the following requirements for precast concrete structural systems.

- Precast seismic systems used in structures assigned to Seismic Design Category C must be intermediate or special moment frames, or intermediate precast or special structural walls.

- Precast seismic systems used in structures assigned to Seismic Design Category D must be special moment frames, or intermediate precast (up to 40 feet) or special structural walls.

- Precast seismic systems used in structures assigned to Seismic Design Category E or F must be special moment frames or special structural walls.

- Prestress provided by prestressing steel resisting earthquake-induced flexural and axial loads in frame members must be limited to 700 psi or $f'_c/6$ in plastic hinge regions. These values are different from the ACI 318 limitations, which are 500 psi or $f'_c/10$.

- An ordinary precast structural wall is defined as one that satisfies ACI 318 Chapters 1-18.

- An intermediate precast structural wall must meet additional requirements for its connections beyond those defined in ACI 318 Section 21.4. These include requirements for the design of wall piers that amplify the design shear forces and prescribe wall pier detailing and requirements for explicit consideration of the ductility capacity of yielding connections.

- A special structural wall constructed using precast concrete must satisfy the acceptance criteria defined in *Provisions* Section 9.6 if it doesn't meet the requirements for special structural walls constructed using precast concrete contained in ACI 318 Section 21.10.2.

Examples are provided for the following concepts:

- The example in Section 8.1 illustrates the design of untopped and topped precast concrete floor and roof diaphragms of the five-story masonry buildings described in Section 10.2 of this volume of design examples. The two untopped precast concrete diaphragms of Section 8.1.1 show the requirements for Seismic Design Categories B and C using 8-inch-thick hollow core precast, prestressed concrete planks. Section 8.1.2 shows the same precast plank with a 2-1/2-inch-thick composite lightweight concrete topping for the five-story masonry building in Seismic Design Category D described in Section 10.2. Although untopped diaphragms are commonly used in regions of low seismic hazard, their design is not specifically addressed in the *Provisions*, the *Standard*, or ACI 318.

- The example in Section 8.2 illustrates the design of an intermediate precast concrete shear wall building in a region of low or moderate seismicity, which is where many 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). The *Provisions* have a few requirements beyond those in ACI 318 and these requirements are identified in this example. Specifically, ACI 318 requires that in connections that are expected to yield, the yielding be restricted to steel elements or reinforcement. The *Provisions* also require that the deformation capacity of the connection be compared to the deformation demand on the connection unless Type 2 mechanical splices are used. There are

additional requirements for intermediate precast structural walls relating to wall piers; however, due to the geometry of the walls used in this design example, this concept is not described in the example.

- The example in Section 8.3 illustrates the design of a special precast concrete shear wall for a single-story industrial warehouse building in Los Angeles. For buildings assigned to Seismic Design Category D, the *Provisions* require that the precast seismic force-resisting system be designed and detailed to meet the requirements for either an intermediate or special precast concrete structural wall. The detailed requirements in the *Provisions* regarding explicit calculation of the deformation capacity of the yielding element are shown here.

- The example in Section 8.4 shows a partial example for the design of a special moment frame constructed using precast concrete per ACI 318 Section 21.8. Concepts for ductile and strong connections are presented and a detailed description of the calculations for a strong connection located at the beam-column interface is presented.

Tilt-up concrete wall buildings in all seismic zones have long been designed using the precast wall panels as concrete shear walls for the seismic force-resisting system. Such designs usually have 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. Tilt-up buildings assigned to Seismic Design Category C or higher should be designed and detailed as intermediate or special precast structural wall systems as defined in ACI 318.

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

ACI 318	American Concrete Institute. 2008. *Building Code Requirements for Structural Concrete*.
AISC 360	American Institute of Steel Construction. 2005. *Specification for Structural Steel Buildings*.
AISC Manual	American Institute of Steel Construction. 2005. *Manual of Steel Construction,* Thirteen Edition.
Moustafa	Moustafa, Saad E. 1981 and 1982. "Effectiveness of Shear-Friction Reinforcement in Shear Diaphragm Capacity of Hollow-Core Slabs." *PCI Journal*, Vol. 26, No. 1 (Jan.-Feb. 1981) and the discussion contained in *PCI Journal*, Vol. 27, No. 3 (May-June 1982).
PCI Handbook	Precast/Prestressed Concrete Institute. 2004. *PCI Design Handbook,* Sixth Edition.
PCI Details	Precast/Prestressed Concrete Institute. 1988. *Design and Typical Details of Connections for Precast and Prestressed Concrete,* Second Edition.
SEAA Hollow Core	Structural Engineers Association of Arizona, Central Chapter. *Design and Detailing of Untopped Hollow-Core Slab Systems for Diaphragm Shear*.

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

8.1 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. Reinforced concrete diaphragms constructed using untopped precast concrete elements are not addressed specifically in the *Standard*, in the *Provisions*, or in ACI 318. Topped precast concrete elements, which act compositely or noncompositely for gravity loads, are designed using the requirements of ACI 318 Section 21.11.

8.1.1 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 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 Section 10.2 provides design parameters used in this example. The floors and roofs of these buildings are to be untopped 8-inch-thick hollow core precast, prestressed concrete plank. Figure 10.2-1 shows the typical floor plan of the diaphragms.

8.1.1.1 General Design Requirements. In accordance with ACI 318, untopped precast diaphragms are permitted only in Seismic Design Categories A through C. 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. Per ACI 318 Section 21.1.1.6, diaphragms in Seismic Design Categories D through F are required to meet ACI 318 Section 21.11, which does not allow untopped diaphragms. In previous versions of the *Provisions*, an appendix was presented that provided a framework for the design of untopped diaphragms in higher Seismic Design Categories in which diaphragms with untopped precast elements were designed to remain elastic and connections designed for limited ductility. However, in the 2009 *Provisions*, that appendix has been removed. Instead, a white paper describing emerging procedures for the design of such diaphragms has been included in Part 3 of the *Provisions*.

The design method used here is that proposed by Moustafa. This method makes use of the shear friction provisions of ACI 318 with the friction coefficient, μ, being equal to 1.0. To use $\mu = 1.0$, ACI 318 requires grout or concrete placed against hardened concrete to have clean, laitance free and intentionally roughened surfaces with a total amplitude of approximately 1/4 inch (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. Note that grouted joints with edges not intentionally roughened can be used with $\mu = 0.6$.

The terminology used is defined in ACI 318 Section 2.2.

8.1.1.2 General In-Plane Seismic Design Forces for Untopped Diaphragms. For Seismic Design Categories B through F, *Standard* Section 12.10.1.1 defines a minimum diaphragm seismic design force.

For Seismic Design Categories C through F, *Standard* Section 12.10.2.1 requires that collector elements, collector splices and collector connections to the vertical seismic force-resisting members be designed in accordance with *Standard* Section 14.4.3.2, which amplifies design forces by means of the overstrength factor, Ω_o.

For Seismic Design Categories D, E and F, *Standard* Section 12.10.1.1 requires that the redundancy factor, ρ, be used on transfer forces only where the vertical seismic force-resisting system is offset and the diaphragm is required to transfer forces between the elements above and below, but need not be applied to inertial forces defined in *Standard* Equation 12.10-1.

Parameters from the example in Section 10.2 used to calculate in-plane seismic design forces for the diaphragms are provided in Table 8.1-1.

Table 8.1-1 Design Parameters from Example 10.2

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

1.0 kip = 4.45 kN.

8.1.1.3 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 *Standard* Equation 12.10-1:

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

Calculations for F_{px} are provided in Table 8.1-2.

Table 8.1-2 Birmingham 1 F_{px} Calculations

Level	w_i (kips)	$\sum_{i=x}^{n} w_i$ (kips)	F_i (kips)	$\sum_{i=x}^{n} F_i = V_i$ (kips)	w_{px} (kips)	F_{px} (kips)
Roof	861	861	175	175	807	164
4	963	1,824	156	331	855	155
3	963	2,787	117	448	855	137
2	963	3,750	78	526	855	120
1	963	4,713	39	565	855	103

1.0 kip = 4.45 kN.

The values for F_i and V_i used in Table 8.1-2 are listed in Table 10.2-2.

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

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

Note that the calculated F_{px} in Table 8.1-2 is substantially larger than the specified maximum limit value of F_{px}. This is generally true at upper levels if the *R* factor is less than 5.

To simplify the design, the diaphragm design force used for all levels will be the maximum force at any level, 82 kips.

8.1.1.4 Diaphragm Forces for New York Building. The weight tributary to the roof and floor diaphragms (w_{px}) is the total story weight (w_i) at Level *i* minus the weight of the walls parallel to the force.

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

- Roof:

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

- Floors:

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

Calculations for F_{px} are provided in Table 8.1-3.

Table 8.1-3 New York F_{px} Calculations

Level	w_i (kips)	$\sum_{i=x}^{n} w_i$ (kips)	F_i (kips)	$\sum_{i=x}^{n} F_i = V_i$ (kips)	w_{px} (kips)	F_{px} (kips)
Roof	870	870	229	229	812	214
4	978	1,848	207	436	863	204
3	978	2,826	155	591	863	180
2	978	3,804	103	694	863	157
1	978	4,782	52	746	863	135

1.0 kip = 4.45 kN.

The values for F_i and V_i used in Table 8.1-3 are listed in Table 10.2-7.

The minimum value of $F_{px} = 0.2S_{DS}Iw_{px}$	= 0.2(0.39)1.0(870 kips)	= 67.9 kips (roof)
	= 0.2(0.39)1.0(978 kips)	= 76.3 kips (floors)
The maximum value of $F_{px} = 0.4S_{DS}Iw_{px}$	= 2(67.9 kips)	= 135.8 kips (roof)
	= 2(76.3 kips)	= 152.6 kips (floors)

As for the Birmingham example, note that the calculated F_{px} given in Table 8.1-3 is substantially larger than the specified maximum limit value of F_{px}.

To simplify the design, the diaphragm design force used for all levels will be the maximum force at any level, 153 kips.

8.1.1.5 Static Analysis of Diaphragms. The balance of this example will use the controlling diaphragm seismic design force of 153 kips for the New York building. In the transverse direction, the loads will be distributed as shown in Figure 8.1-1.

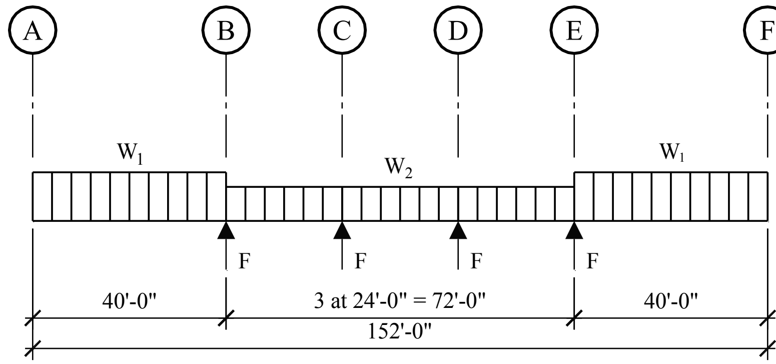

Figure 8.1-1 Diaphragm force distribution and analytical model
(1.0 ft = 0.3048 m)

The *Standard* requires that structural analysis consider the relative stiffness of the diaphragms and the vertical elements of the seismic force-resisting system. Since a pretopped precast diaphragm doesn't satisfy the conditions of either the flexible or rigid diaphragm conditions identified in the *Standard*, maximum in-plane deflections of the diaphragm must be evaluated. However, that analysis is beyond the scope of this document. Therefore, with a rigid diaphragm assumption, 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 8.1-1) are computed as follows:

F = 153 kips/4 = 38.3 kips

The uniform diaphragm demands are proportional to the distributed weights of the diaphragm in different areas (see Figure 8.1-1).

W_1 = [67 psf (72 ft) + 48 psf (8.67 ft)4](153 kips / 863 kips) = 1,150 lb/ft

W_2 = [67 psf (72 ft)](153 kips / 863 kips) = 855 lb/ft

Figure 8.1-2 identifies critical regions of the diaphragm to be considered in this design. These regions are:

- Joint 1: Maximum transverse shear parallel to the panels at panel-to-panel joints
- Joint 2: Maximum transverse shear parallel to the panels at the panel-to-wall joint
- Joint 3: Maximum transverse moment and chord force
- Joint 4: Maximum longitudinal shear perpendicular to the panels at the panel-to-wall connection (exterior longitudinal walls) and anchorage of exterior masonry wall to the diaphragm for out-of-plane forces
- Joint 5: Collector element and shear for the interior longitudinal walls

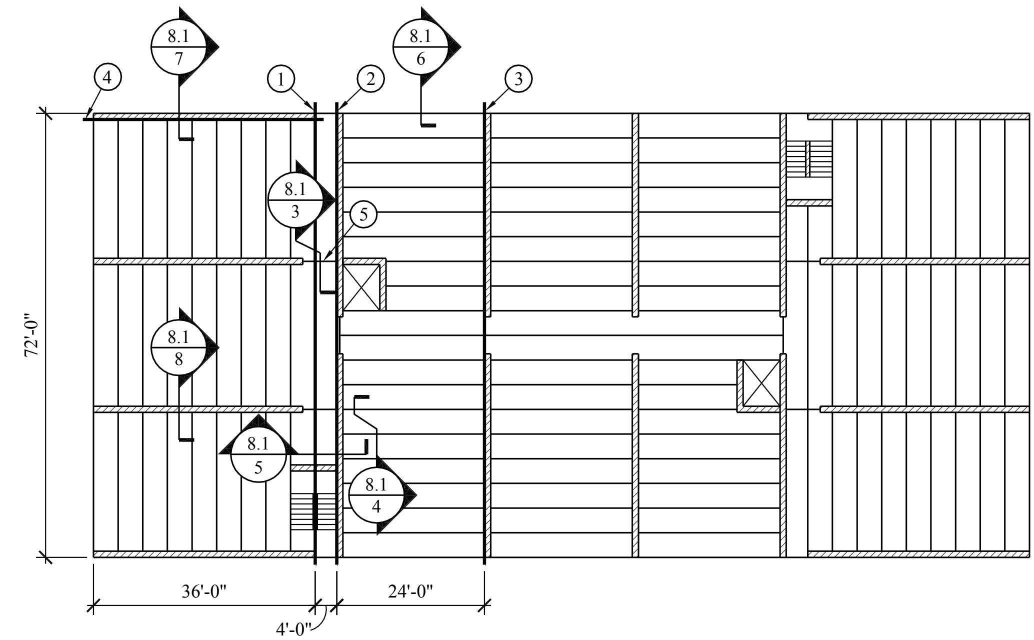

Figure 8.1-2 Diaphragm plan and critical design regions
(1.0 ft = 0.3048 m)

Joint forces are as follows:

- Joint 1 – Transverse forces:

  Shear, V_{u1} = 1.15 kips/ft (36 ft) = 41.4 kips
  Moment, M_{u1} = 41.4 kips (36 ft / 2) = 745 ft-kips
  Chord tension force, $T_{u1} = M/d$ = 745 ft-kips / 71 ft = 10.5 kips

- Joint 2 – Transverse forces:

  Shear, V_{u2} = 1.15 kips/ft (40 ft) = 46 kips
  Moment, M_{u2} = 46 kips (40 ft / 2) = 920 ft-kips
  Chord tension force, $T_{u2} = M/d$ = 920 ft-kips / 71 ft = 13.0 kips

- Joint 3 – Transverse forces:

  Shear, V_{u3} = 46 kips + 0.86 kips/ft (24 ft) – 38.3 kips = 28.3 kips
  Moment, M_{u3} = 46 kips (44 ft) + 20.6 kips (12 ft) - 38.3 kips (24 ft) = 1,352 ft-kips
  Chord tension force, $T_{u3} = M/d$ = 1,352 ft-kips / 71 ft = 19.0 kips

- Joint 4 – Longitudinal forces:

  Wall force, F = 153 kips / 8 = 19.1 kips
  Wall shear along wall length, V_{u4} = 19.1 kips (36 ft)/(152 ft / 2) = 9.0 kips
  Collector force at wall end, $T_{u4} = C_{u4}$ = 19.1 kips - 9.0 kips = 10.1 kips

- Joint 4 – Out-of-plane forces:

The *Standard* has 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 *Standard* Section 12.14 (Seismic Design Category B and greater), the required out-of-plane horizontal force is:

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

According to *Standard* Section 12.11.2 (Seismic Design Category B and greater), the minimum anchorage for masonry walls is:

$400(S_{DS})(I) = 400(0.39)1.0 = 156 \text{ plf}$

According to *Standard* Section 12.11.1 (Seismic Design Category B and greater), bearing wall anchorage must be designed for a force computed as:

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

Standard Section 12.11.2.1 (Seismic Design Category C and greater) requires masonry wall anchorage to flexible diaphragms to be designed for a larger force. Due to its geometry, this diaphragm is likely to be classified as rigid. However, the relative deformations of the wall and diaphragm must be checked in accordance with *Standard* Section 12.3.1.3 to validate this assumption.

$F_p = 0.85(S_{DS})(I)(W_{wall}) = 0.85(0.39)1.0[(48 \text{ psf})(8.67 \text{ ft})] = 138 \text{ plf}$

(Note that since this diaphragm is not flexible, this load is not used in the following calculations.)

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

- Joint 5 – Longitudinal forces:

Wall force, F = 153 kips / 8 = 19.1 kips
Wall shear along each side of wall, V_{u5} = 19.1 kips [2(36 ft) / 152 ft]/2 = 4.5 kips
Collector force at wall end, $T_{u5} = C_{u5}$ = 19.1 kips - 2(4.5 kips) = 10.1 kips

- Joint 5 – Shear flow due to transverse forces:

Shear at Joint 2, V_{u2} = 46 kips
$Q = A\ d$
$A = (0.67 \text{ ft})(24 \text{ ft}) = 16 \text{ ft}^2$
$d = 24 \text{ ft}$
$Q = (16 \text{ ft}^2)(24 \text{ ft}) = 384 \text{ ft}^3$
$I = (0.67 \text{ ft})(72 \text{ ft})^3 / 12 = 20{,}840 \text{ ft}^4$
$V_{u2}Q/I = (46 \text{ kip})(384 \text{ ft}^3) / 20{,}840 \text{ ft}^4 = 0.847$ kip/ft maximum shear flow
Joint 5 length = 40 ft
Total transverse shear in joint 5, V_{u5} = 0.847 kip/ft) (40 ft)/2 = 17 kips

ACI 318 Section 16.5 also has minimum connection force requirements for structural integrity of precast concrete bearing wall building construction. For buildings over two stories tall, there are force

requirements for horizontal and vertical members. This building has no vertical precast members. However, ACI 318 Section 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 for a precast bearing wall structure three or more stories in height are:

- 1,500 pounds per foot parallel and perpendicular to the span of the floor members. The maximum spacing of ties parallel to the span is 10 feet. The maximum spacing of ties perpendicular to the span is the distance between supporting walls or beams.

- 16,000 pounds parallel to the perimeter of a floor or roof located within 4 feet of the edge at all edges.

ACI's tie forces are far greater than the minimum tie forces given in the *Standard* for beam supports and anchorage 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.

8.1.1.6 Diaphragm Design and Details. The phi factors used for this example are as follows:

- 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 Section 16.5 are specified as nominal values, meaning that $\phi = 1.00$ for those forces.

Note that although buildings assigned to Seismic Design Category C are not required to meet ACI 318 Section 21.11, some of the requirements contained therein are applied below as good practice but shown as optional.

8.1.1.6.1 Joint 1 Design and Detailing. The design must provide sufficient reinforcement for chord forces as well as shear friction connection forces, as follows:

- Chord reinforcement, $A_{s1} = T_{u1}/\phi f_y$ = (10.5 kips)/[0.9(60 ksi)] = 0.19 in^2 (The collector force from the Joint 4 calculations at 10.1 kips is not directly additive.)

- Shear friction reinforcement, $A_{vf1} = V_{u1}/\phi\mu f_y$ = (41.4 kips)/[(0.75)(1.0)(60 ksi)] = 0.92 in^2

- Total reinforcement required = 2(0.19 in^2) + 0.92 in^2 = 1.30 in^2

- ACI tie force = (1.5 kips/ft)(72 ft) = 108 kips; reinforcement = (108 kips)/(60 ksi) = 1.80 in^2

Provide four #5 bars (two at each of the outside edges) plus four #4 bars (two each at the interior joint at the ends of the plank) for a total area of reinforcement of 4(0.31 in^2) + 4(0.2 in^2) = 2.04 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 #4 bars in the interior joints will be provided to meet the requirements of ACI 318 Section 21.11.7.6 (optional):

- Minimum cover = 2.5(4/8) = 1.25 in., but not less than 2.00 in.
- Minimum spacing = 3(4/8) = 1.50 in., but not less than 1.50 in.

Figure 8.1-3 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 #4 bars extend along the length of the interior longitudinal walls as shown in Figure 8.1-3.

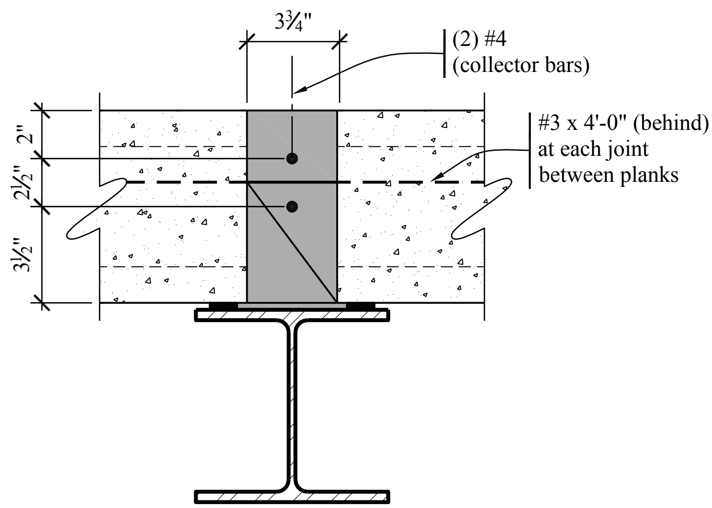

Figure 8.1-3 Interior joint reinforcement at the ends of plank and collector reinforcement at the end of the interior longitudinal walls - Joints 1 and 5
(1.0 in. = 25.4 mm)

Figure 8.1-4 shows the extension of the two #4 bars of Figure 8.1-3 into the region where the plank is parallel to the bars (see section cut on Figure 8.1-2). 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 #4 bars or an equal area of strand) and shows the bars anchored at each end of the plank. The anchorage length of the #4 bars is calculated using ACI 318 Chapter 12:

$$l_d = \left(\frac{f_y \psi_t \psi_e}{25\lambda\sqrt{f_c'}} \right) d_b = \left(\frac{60{,}000\,\text{psi}(1.0)(1.0)}{25(1.0)\sqrt{4{,}000}\,\text{psi}} \right) d_b = 37.9 d_b$$

Using #4 bars, the required l_d = 37.9(0.5 in.) = 18.9 in. Therefore, use l_d = 4 ft, which is the width of the plank.

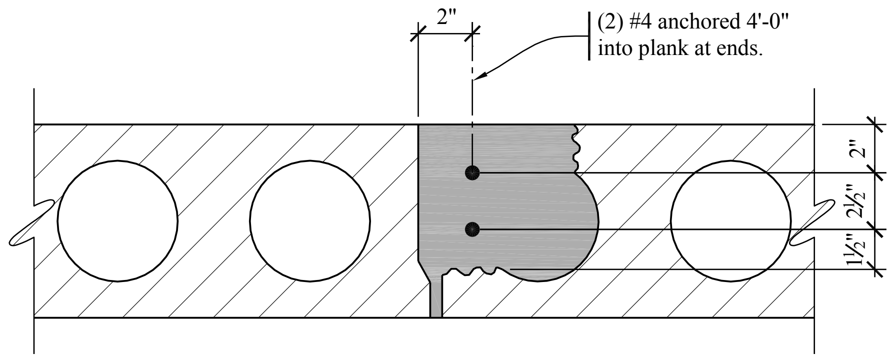

Figure 8.1-4 Anchorage region of shear reinforcement for Joint 1 and collector reinforcement for Joint 5
(1.0 in. = 25.4 mm)

8.1.1.6.2 Joint 2 Design and Detailing. The chord design is similar to the previous calculations:

- Chord reinforcement, $A_{s2} = T_{u2}/\phi f_y$ = (13.0 kips)/[0.9(60 ksi)] = 0.24 in^2

The shear force may be reduced along Joint 2 by the shear friction resistance provided by the supplemental chord reinforcement ($2A_{chord}$ - A_{s2}) and by the four #4 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 follows:

$$V_{u2}^{Mod} = V_{u2} - \left[\phi f_y \mu \left(A_{4\#4}\right)\right] = 46 - \left[0.75\left(60\,\text{ksi}\right)\left(1.0\right)\left(4\times 0.2\,\text{in}^2\right)\right] = 36.0 \text{ kips}$$

This force must be transferred from the planks to the wall. Using the arrangement shown in Figure 8.1-5, the required shear friction reinforcement (A_{vf2}) is computed as:

$$A_{vf2} = \frac{V_{u2}^{Mod}}{\phi f_y\left(\mu\sin\alpha_f + \cos\alpha_f\right)} = \frac{36.0\,\text{kips}}{0.75\left(1.0\sin 26.6° + \cos 26.6°\right)} = 0.60\,\text{in}^2$$

Use two #3 bars placed at 26.6 degrees (2-to-1 slope) across the joint at 6 feet from the ends of the plank (two sets per plank). The angle (α_f) used above provides development of the #3 bars while limiting the grouting to the outside core of the plank. The total shear reinforcement provided is 6(0.11 in^2) = 0.66 in^2. Note that the spacing of these connectors will have to be adjusted at the stair location.

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

V_{u2}-F = 46-38.3 = 7.7 kips

The shear friction resistance provided by #3 bars in the grout key between each plank (provided for the 1.5 klf requirement of ACI 318) is computed as:

$\phi A_{vf} f_y \mu$ = (0.75)(10 bars)(0.11 in^2)(60 ksi)(1.0) = 49.5 kips

The development length of the #3 bars will now be checked. For the 180 degree standard hook, use ACI 318 Section 12.5, l_{dh} times the factors of ACI 318 Section 12.5.3, but not less than $8d_b$ or 6 inches. Side cover exceeds 2-1/2 inches and cover on the bar extension beyond the hook is provided by the grout and the planks, which is close enough to 2 inches to apply the 0.7 factor of ACI 318 Section 12.5.3. For the #3 hook:

$$l_{dh} = 0.7\left(\frac{0.02\psi_e f_y}{\sqrt{f_c'}}\right)d_b = 0.7\left(\frac{0.02(1.0)(60{,}000\,\text{psi})}{\sqrt{4{,}000}\,\text{psi}}\right)0.375 = 4.98 \text{ in. } (\leq 6 \text{ in. minimum})$$

The available distance for the perpendicular hook is approximately 5-1/2 inches. The bar will not be fully developed at the end of the plank because of the 6-inch 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 bar will now be checked. The standard development length of ACI 318 Section 12.2 is used for l_d.

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

Figure 8.1-5 shows the reinforcement along each side of the wall on Joint 2.

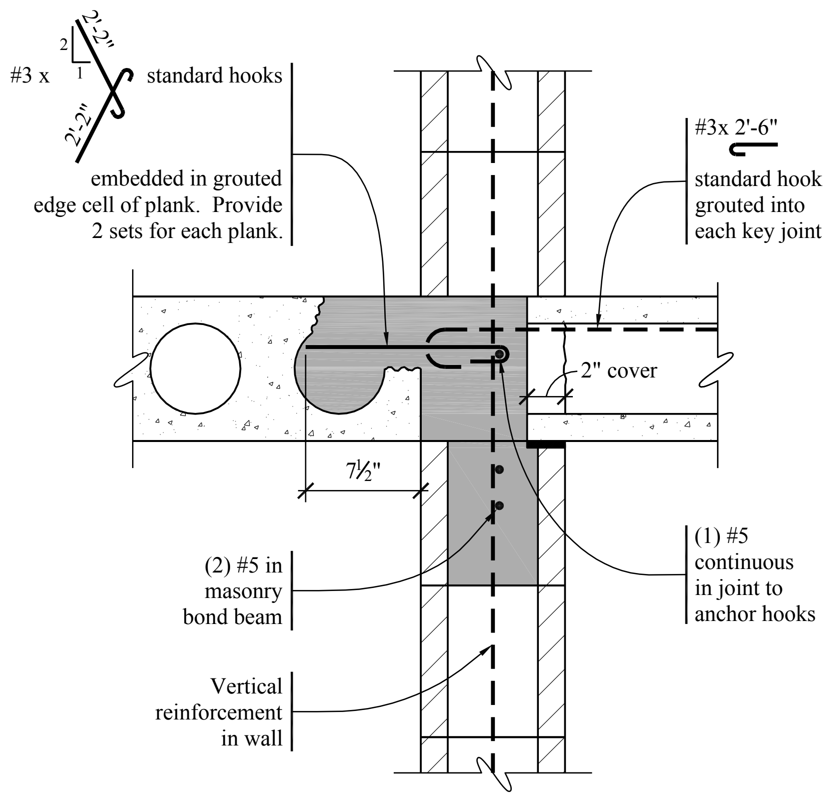

Figure 8.1-5 Joint 2 transverse wall joint reinforcement
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

8.1.1.6.3 Design and Detailing at Joint 3. Compute the required amount of chord reinforcement at Joint 3 as:

$A_{s3} = T_{u3}/\phi f_y$ = (19.0 kips)/[0.9(60 ksi)] = 0.35 in^2

Use two #4 bars, A_s = 2(0.20) = 0.40 in^2 along the exterior edges (top and bottom of the plan in Figure 8.1-2). Require cover for chord bars and spacing between bars at splices and anchorage zones per ACI 318 Section 21.11.7.6 (optional).

- Minimum cover = 2.5(4/8) = 1.25 in., but not less than 2.00 in.

- Minimum spacing = 3(4/8) = 1.50 in., but not less than 1.50 in.

Figure 8.1-6 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 the Joint 4 collector reinforcement calculations and Figure 8.1-7).

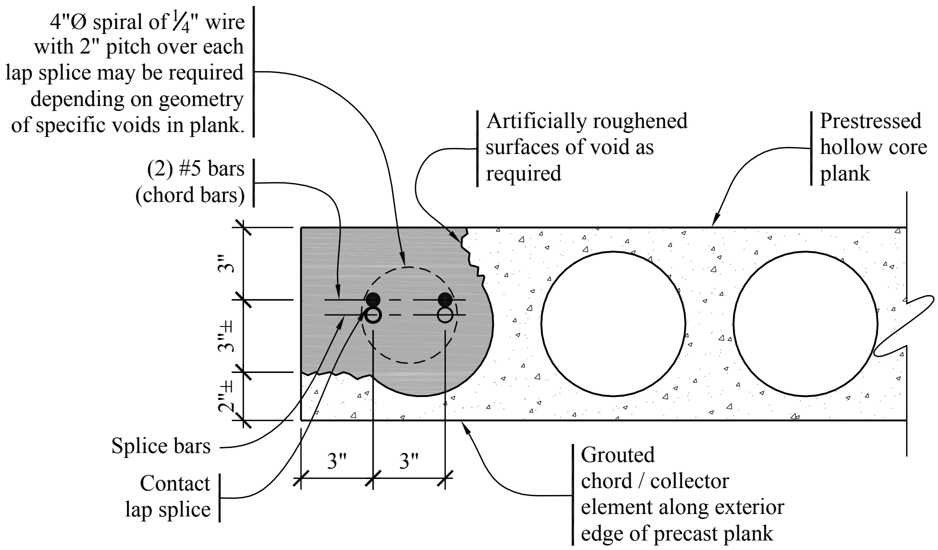

Figure 8.1-6 Joint 3 chord reinforcement at the exterior edge
(1.0 in. = 25.4 mm)

Joint 3 must also be checked for the minimum ACI tie forces. The chord reinforcement obviously exceeds the 16 kip perimeter force requirement. To satisfy the 1.5 kips per foot requirement, a 6 kip tie is needed 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.

8.1.1.6.4 Joint 4 Design and Detailing. The required shear friction reinforcement along the wall length is computed as:

$$A_{vf4} = V_{u4}/\phi\mu f_y = (9.0 \text{ kips})/[(0.75)(1.0)(60 \text{ ksi})] = 0.20 \text{ in}^2$$

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

The required collector reinforcement is computed as:

$$A_{s4} = T_{u4}/\phi f_y = (10.1 \text{ kips})/[0.9(60 \text{ ksi})] = 0.19 \text{ in}^2$$

The two #4 bars, which are an extension of the transverse chord reinforcement, provide an area of reinforcement of 0.40 in^2.

The reinforcement required by the *Standard* for out-of-plane force (156 plf) is far less than the ACI 318 requirement.

Figure 8.1-7 shows this joint along the wall.

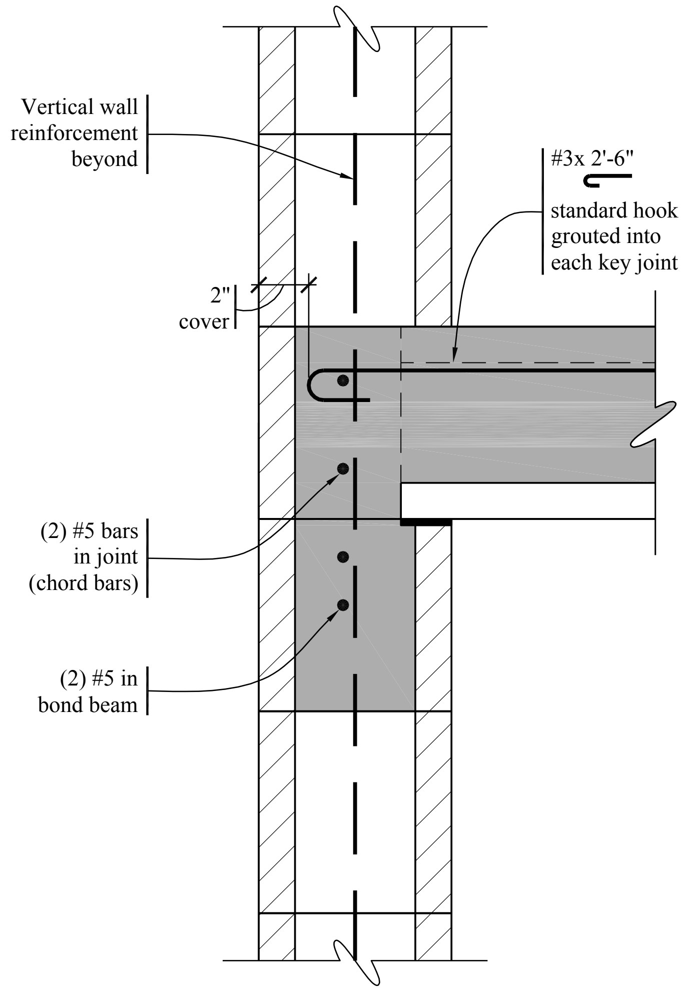

Figure 8.1-7 Joint 4 exterior longitudinal walls to diaphragm reinforcement and out-of-plane anchorage
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

8.1.1.6.5 Joint 5 Design and Detailing. The required shear friction reinforcement along the wall length is computed as:

$$A_{vf5} = V_{u5}/\phi\mu f_y = (16.9 \text{ kips})/[(0.75)(1.0)(0.85)(60 \text{ ksi})] = 0.44 \text{ in}^2$$

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

The required collector reinforcement is computed as:

$$A_{s5} = T_{u5}/\phi f_y = (10.1 \text{ kips})/[0.9(60 \text{ ksi})] = 0.19 \text{ in}^2$$

Two #4 bars specified for the design of Joint 1 above provide an area of reinforcement of 0.40 in^2. Figure 8.1-8 shows this joint along the wall.

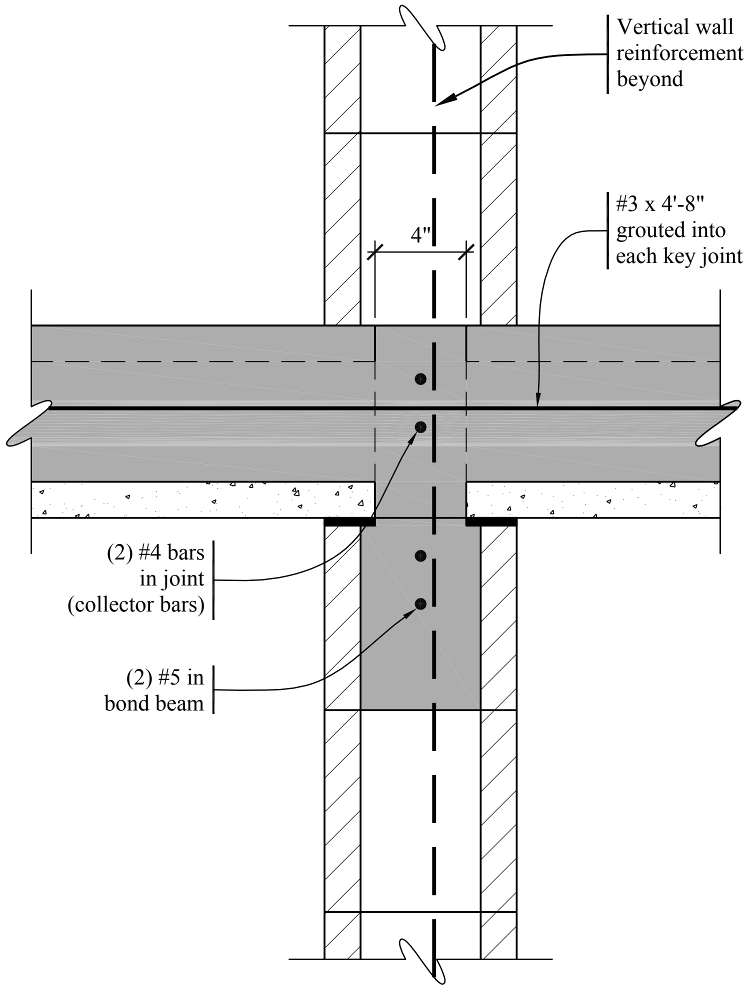

Figure 8.1-8 Wall-to-diaphragm reinforcement along interior longitudinal walls - Joint 5 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

8.1.2 Topped Precast Concrete Units for Five-Story Masonry Building Located in Los Angeles, California (see Sec. 10.2)

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 inches as required for composite topping slabs by ACI 318 Section 21.11.6. The topping is 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-inch-thick hollow-core precast, prestressed concrete plank. Design parameters are provided in Section 10.2. Figure 10.2-1 shows the typical floor and roof plan.

8.1.2.1 General Design Requirements. Topped diaphragms may be used in any Seismic Design Category. ACI 318 Section 21.11 provides design provisions for topped precast concrete diaphragms. *Standard* Section 12.10 specifies the forces to be used in designing the diaphragms.

8.1.2.2 General In-Plane Seismic Design Forces for Topped Diaphragms. The in-plane diaphragm seismic design force (F_{px}) is calculated using *Standard* Equation 12.10-1 but must not be less than $0.2S_{DS}Iw_{px}$ and need not be more than $0.4S_{DS}Iw_{px}$. V_x must be added to F_{px} calculated using Equation 12.10-1 where:

S_{DS} = the spectral response acceleration parameter at short periods

I = occupancy importance factor

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

V_x = the portion of the seismic shear force required to be transferred to the components of the vertical seismic force-resisting system due to offsets or changes in stiffness of the vertical resisting member at the diaphragm being designed

For Seismic Design Category C and higher, *Standard* Section 12.10.2.1 requires that collector elements, collector splices and collector connections to the vertical seismic force-resisting members be designed in accordance with *Standard* Section 12.4.3.2, which combines the diaphragm forces times the overstrength factor (Ω_0) and the effects of gravity forces. The parameters from the example in Section 10.2 used to calculate in-plane seismic design forces for the diaphragms are provided in Table 8.1-4.

Table 8.1-4 Design Parameters from Section 10.2

Design Parameter	Value
Ω_o	2.5
w_i (roof)	1,166 kips
w_i (floor)	1,302 kips
S_{DS}	1.0
I	1.0
Seismic Design Category	D

1.0 kip = 4.45 kN.

8.1.2.3 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 follows:

- Roof:

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

- Floors:

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

Compute diaphragm demands in accordance with *Standard* Equation 12.10-1:

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

Calculations for F_{px} are provided in Table 8.1-5. The values for F_i and V_i are listed in Table 10.2-17.

Table 8.1-5 F_{px} Calculations from Section 10.2

Level	w_i (kips)	$\sum_{i=x}^{n} w_i$ (kips)	F_i (kips)	$\sum_{i=x}^{n} F_i = V_i$ (kips)	w_{px} (kips)	F_{px} (kips)
Roof	1,166	1,166	564	564	1,094	529
4	1,302	2,468	504	1,068	1,158	501
3	1,302	3,770	378	1,446	1,158	444
2	1,302	5,072	252	1,698	1,158	387
1	1,302	6,374	126	1,824	1,158	331

1.0 kip = 4.45 kN.

The minimum value of $F_{px} = 0.2S_{DS}Iw_{px}$	= 0.2(1.0)1.0(1,094 kips)	= 219 kips (roof)
	= 0.2(1.0)1.0(1,158 kips)	= 232 kips (floors)
The maximum value of $F_{px} = 0.4S_{DS}Iw_{px}$	= 2(219 kips)	= 438 kips (roof)
	= 2(232 kips)	= 463 kips (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.

8.1.2.4 Static Analysis of Diaphragms. The seismic design force of 463 kips is distributed as in Section 8.1.1.6 (Figure 8.1-1 shows the distribution). The force is three times higher than that used to design the untopped diaphragm for the New York design due to the higher seismic demand. Figure 8.1-2 shows critical regions of the diaphragm to be considered in this design. Collector elements will be designed for 2.5 times the diaphragm force based on the overstrength factor (Ω_0).

Joint forces taken from Section 8.1.1.5 times 3.0 are as follows:

- Joint 1 – Transverse forces:

  Shear, V_{u1} = 41.4 kips × 3.0 = 124 kips
  Moment, M_{u1} = 745 ft-kips × 3.0 = 2,235 ft-kips
  Chord tension force, $T_{u1} = M/d$ = 2,235 ft-kips / 71 ft = 31.5 kips

- Joint 2 – Transverse forces:

  Shear, V_{u2} = 46 kips × 3.0 = 138 kips
  Moment, M_{u2} = 920 ft-kips × 3.0 = 2,760 ft-kips
  Chord tension force, $T_{u2} = M/d$ = 2,760 ft-kips / 71 ft = 38.9 kips

- Joint 3 – Transverse forces:

  Shear, V_{u3} = 28.3 kips × 3.0 = 84.9 kips
  Moment, M_{u2} = 1,352 ft-kips × 3.0 = 4,056 ft-kips
  Chord tension force, $T_{u3} = M/d$ = 4,056 ft-kips / 71 ft = 57.1 kips

- Joint 4 – Longitudinal forces:

  Wall force, F = 19.1 kips × 3.0 = 57.3 kips
  Wall shear along wall length, V_{u4} = 9 kips × 3.0 = 27.0 kips
  Collector force at wall end, $\Omega_0 T_{u4}$ = 2.5(10.1 kips)(3.0) = 75.8 kips

- Joint 4 – Out-of-plane forces:

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

- Joint 5 – Longitudinal forces:

  Wall force, F = 463 kips / 8 walls = 57.9 kips
  Wall shear along each side of wall, V_{u4} = 4.5 kips × 3.0 = 13.5 kips
  Collector force at wall end, $\Omega_0 T_{u4}$ = 2.5(10.1 kips)(3.0) = 75.8 kips

- Joint 5 – Shear flow due to transverse forces:

  Shear at Joint 2, V_{u2} = 138 kips
  $Q = A\ d$
  A = (0.67 ft) (24 ft) = 16 ft^2
  d = 24 ft
  Q = (16 ft^2) (24 ft) = 384 ft^3
  I = (0.67 ft) (72 ft)^3 / 12 = 20,840 ft^4
  $V_{u2}Q/I$ = (138 kip) (384 ft^3) / 20,840 ft^4 = 2.54 kips/ft maximum shear flow
  Joint 5 length = 40 ft
  Total transverse shear in joint 5, V_{u5} = 2.54 kips/ft) (40 ft)/2 = 50.8 kips

8.1.2.5 Diaphragm Design and Details

8.1.2.5.1 Minimum Reinforcement for 2.5-inch Topping. ACI 318 Section 21.11.7.1 references ACI 318 Section 7.12, which requires a minimum $A_s = 0.0018bd$ for grade 60 welded wire reinforcement. For a 2.5-inch topping, the required A_s = 0.054 in^2/ft. WWR 10×10 - W4.5×W4.5 provides 0.054 in^2/ft. The minimum spacing of wires is 10 inches and the maximum spacing is 18 inches. Note that the ACI 318 Section 7.12 limit on spacing of five times thickness is interpreted such that the topping thickness is not the pertinent thickness.

8.1.2.5.2 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 Section 21.11.7.5. 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 this is not necessary since the joints between precast units are grouted. As developed previously, the chord compressive stress is:

$$6M_{u3}/td^2 = 6(4{,}056\times12)/(2.5)(72\times12)^2 = 157 \text{ psi}$$

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

The required chord reinforcement is:

$$A_{s3} = T_{u3}/\phi f_y = (57.1 \text{ kips})/[0.9(60 \text{ ksi})] = 1.06 \text{ in}^2$$

8.1.2.5.3 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.

$$A_{s4} = A_{s5} = \Omega_0 T_{u4}/\phi f_y = (75.8 \text{ kips})/[0.9(60 \text{ ksi})] = 1.40 \text{ in}^2$$

Use two #8 bars ($A_s = 2 \times 0.79 = 1.58$ in^2) along the exterior edges, along the length of the exterior longitudinal walls and along the length of the interior longitudinal walls. Provide cover for chord and collector bars and spacing between bars per ACI 318 Section 21.11.7.6.

- 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 1.5 in.

Figure 8.1-9 shows the diaphragm plan and section cuts of the details and Figure 8.1-10 shows 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.

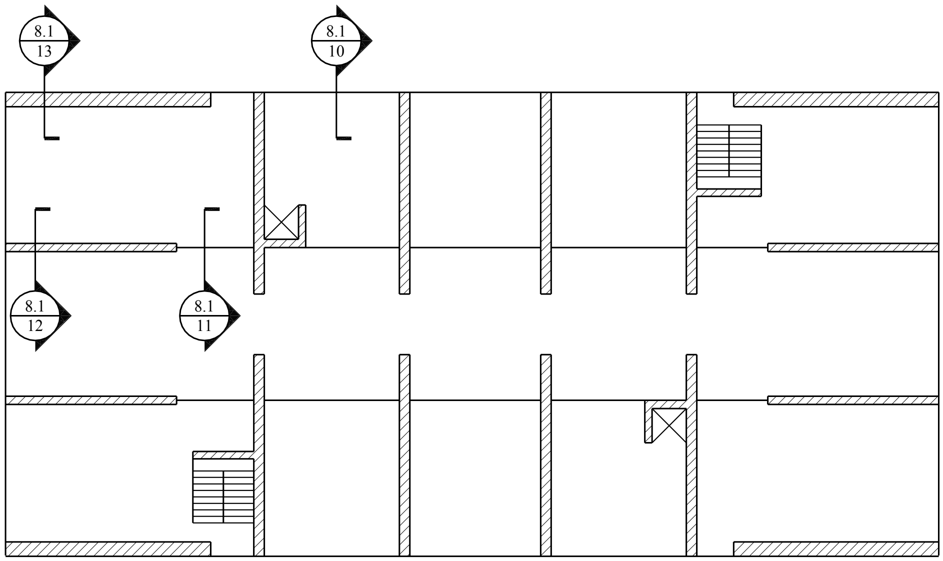

Figure 8.1-9 Diaphragm plan and section cuts

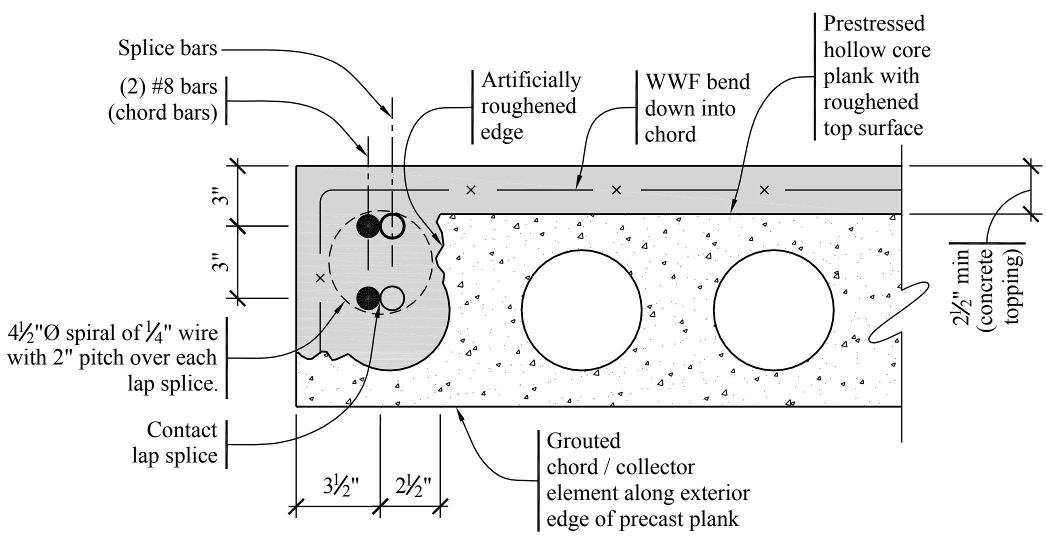

Figure 8.1-10 Boundary member and chord and collector reinforcement
(1.0 in. = 25.4 mm)

Figure 8.1-11 shows the collector reinforcement for the interior longitudinal walls. The side cover of 2-1/2 inches is provided by casting the topping into the cores and by the stems of the plank. A minimum space of 1 inch is provided between the plank stems and the sides of the bars.

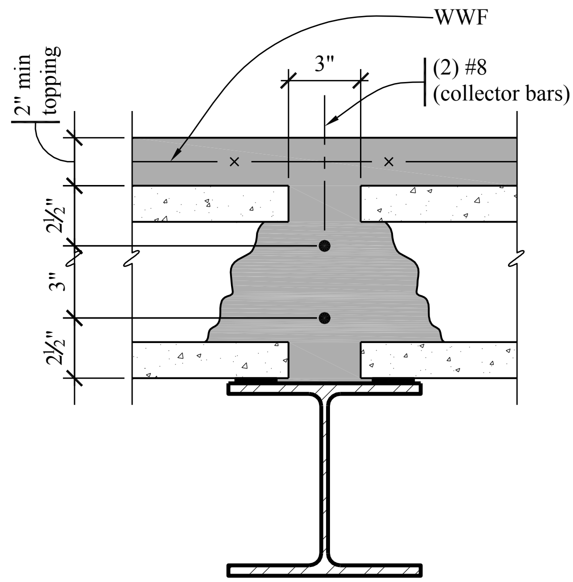

Figure 8.1-11 Collector reinforcement at the end of the interior longitudinal walls - Joint 5
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

8.1.2.5.4 Shear Resistance. In thin composite and noncomposite topping slabs on precast floor and roof members, joints typically are tooled during construction, resulting in cracks forming at the joint between precast members. Therefore, the shear resistance of the topping slab is limited to the shear friction strength of the reinforcing crossing the joint.

ACI 318 Section 21.11.9.1 provides an equation for the shear strength of the diaphragm, which includes both concrete and reinforcing components. However, for noncomposite topping slabs on precast floors and roofs where the only reinforcing crossing the joints is the field reinforcing in the topping slab, the shear friction capacity at the joint will always control the design. ACI 318 Section 21.11.9.3 defines the shear strength at the joint as follows:

$$\phi V_n = \phi A_{vf} f_y \mu = 0.75(0.054 \text{ in}^2/\text{ft})(60 \text{ ksi})(1.0)(0.85) = 2.07 \text{ kips/ft}$$

Note that $\mu = 1.0\lambda$ is used since the joint is assumed to be pre-cracked.

The shear resistance in the transverse direction is:

2.07 kips/ft (72 ft) = 149 kips

which is greater than the Joint 1 shear (maximum transverse shear) of 124 kips.

At the plank adjacent to Joint 2, the shear strength of the diaphragm in accordance with ACI 318 Section 21.11.9.1 is:

$$\phi V_n = \phi A_{cv}\left(2\lambda\sqrt{f_c'} + \rho_t f_y\right) = 0.75\left(2.5\times72\times12\right)\left(2\left(1.0\right)\sqrt{4{,}000} + 0.0018\times60{,}000\right) = 348 \text{ kips}$$

Number 3 dowels are used to provide continuity of the topping slab welded wire reinforcement across the masonry walls. The topping is to be cast into the masonry walls as shown in Figure 8.1-12 and the spacing of the #3 bars is set to be modular with the CMU.

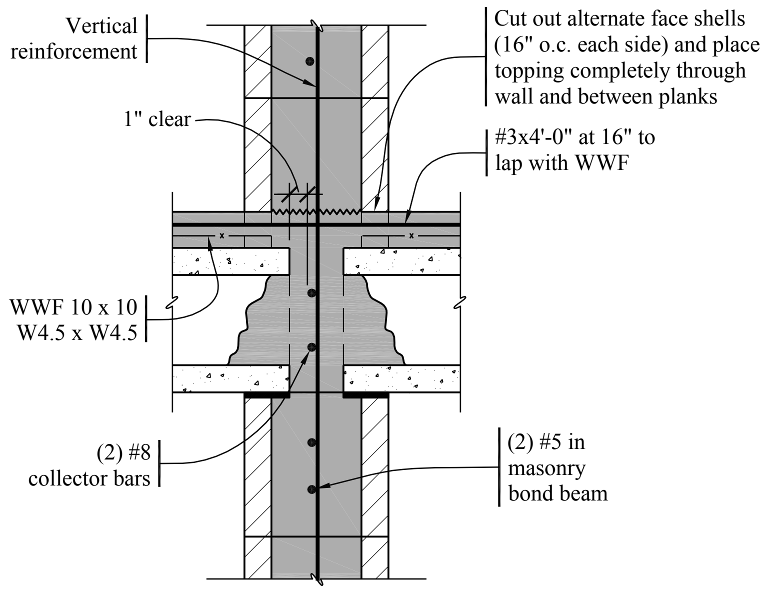

Figure 8.1-12 Wall-to-diaphragm reinforcement along interior longitudinal walls - Joint 5
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

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

$A_{vf4} = V_{u4}/\phi\mu f_y$ = (27.0 kips)/[(0.75)(1.0)(0.85)(60 ksi)] = 0.71 in^2

The required shear reinforcement along the interior longitudinal wall (Joint 5) is:

$A_{vf5} = V_{u5}/\phi\mu f_y$ = (50.8 kips)/[(0.75)(1.0)(0.85)(60 ksi)] = 1.32 in^2

Number 3 dowels spaced at 16" o.c. provide

A_v = (0.11 in^2) (40 ft x 12 in/ft) / 16 in = 3.30 in^2

8.1.2.5.5 Check of Out-of-Plane Forces. At Joint 4, the out-of-plane forces are checked as follows:

$F_p = 0.85\ S_{DS}\ I\ W_{wall}$ = 0.85(1.0)(1.0)(60 psf)(8.67 ft) = 442 plf

With bars at 4 feet on center, F_p = 4 ft (442 plf) = 1.77 kips.

The required reinforcement, A_s = 1.77 kips/(0.9)(60ksi) = 0.032 in^2. Provide #3 bars at 4 feet on center, which provides a nominal strength of 0.11×60/4 = 1.7 klf. This detail satisfies the ACI 318 Section 16.5

required tie force of 1.5 klf. The development length was checked in the prior example. Using #3 bars at 4 feet on center will be adequate and the detail is shown in Figure 8.1-13. The detail at Joint 2 is similar.

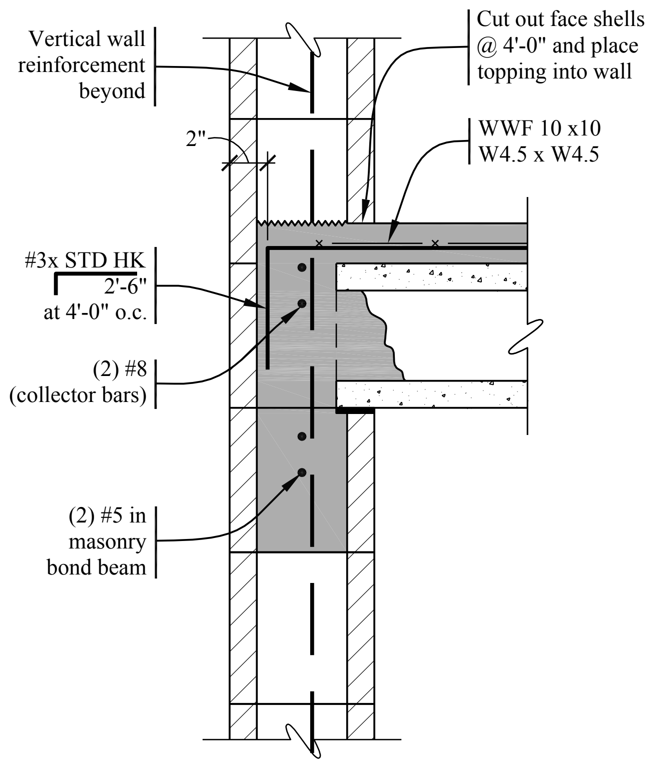

Figure 8.1-13 Exterior longitudinal wall-to-diaphragm reinforcement and out-of-plane anchorage - Joint 4 (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

8.2 THREE-STORY OFFICE BUILDING WITH INTERMEDIATE PRECAST CONCRETE SHEAR WALLS

This example illustrates the seismic design of intermediate precast concrete shear walls. These walls can be used up to any height in Seismic Design Categories B and C but are limited to 40 feet for Seismic Design Categories D, E and F.

ACI 318 Section 21.4.2 requires that yielding between wall panels or between wall panels and the foundation be restricted to steel elements. However, the *Provisions* are more specific in their means to accomplish the objective of providing reliable post-elastic performance. *Provisions* Section 21.4.3 (ACI 318 Sec. 21.4.4) requires that connections that are designed to yield be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement. Alternatively, they can use Type 2 mechanical splices.

Additional requirements are contained in the *Provisions* for intermediate precast walls with wall piers (*Provisions* Sec. 14.2.2.4 [ACI 318 Sec. 21.4.5]); however, these requirements do not apply to the solid wall panels used for this example.

8.2.1 Building Description

This precast concrete building is a three-story office building (Occupancy Category II) in southern New England on Site Class D soils. The structure utilizes 10-foot-wide by 18-inch-deep prestressed double tees (DTs) spanning 40 feet 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-inch-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 8.2-1, the building has a regular plan. The precast shear walls are continuous from the ground level to 12 feet above the roof. The walls of the elevator/mechanical pits are cast-in-place below grade. The building has no vertical irregularities. The story height is 12 feet.

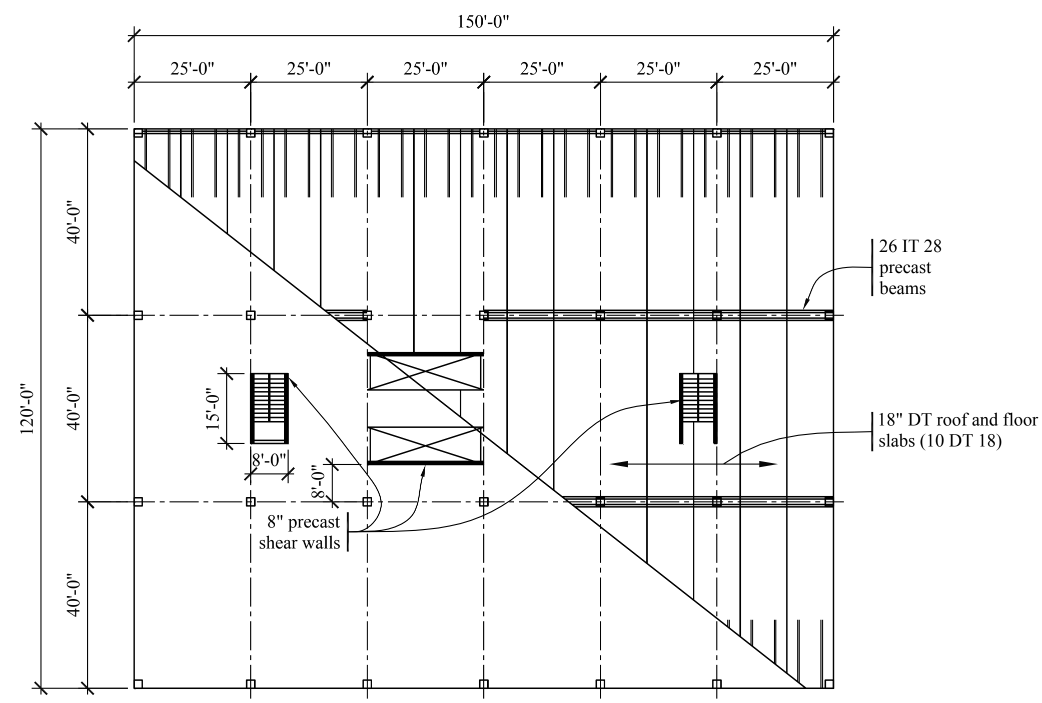

Figure 8.2-1 Three-story building plan
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

The precast walls are estimated to be 8 inches thick for building mass calculations. These walls are normal-weight concrete with a 28-day compressive strength, f'_c, of 5,000 psi. Reinforcing bars used at the ends of the walls and in welded connectors are ASTM A706 (60 ksi yield strength). The concrete for the foundations and below-grade walls has a 28-day compressive strength, f'_c, of 4,000 psi.

8.2.2 Design Requirements

8.2.2.1 Seismic Parameters. The basic parameters affecting the design and detailing of the building are shown in Table 8.2-1.

Table 8.2-1 Design Parameters

Design Parameter	Value
Occupancy Category II	$I = 1.0$
S_S	0.266
S_1	0.08
Site Class	D
F_a	1.59
F_v	2.4
$S_{MS} = F_a S_S$	0.425
$S_{M1} = F_v S_1$	0.192
$S_{DS} = 2/3\ S_{MS}$	0.283
$S_{D1} = 2/3\ S_{M1}$	0.128
Seismic Design Category	B
Basic Seismic Force-Resisting System	Bearing Wall System
Wall Type	Intermediate Precast Shear Walls
R	4
Ω_0	2.5
C_d	4

A Bearing Wall System is defined in the *Standard* as "a structural system with bearing walls providing support for all or major portions of the vertical loads." In the 2006 International Building Code, this requirement is clarified by defining a concrete Load Bearing Wall as one which "supports more than 200 pounds per linear foot of vertical load in addition to its own weight." While the IBC definition is much more stringent, this interpretation is used in this example. Note that if a Building Frame Intermediate Precast Shear Wall system were used, the design would be based on R=5, Ω_o=2 ½ and C_d=4½.

Note that in Seismic Design Category B an ordinary precast shear wall could be used to resist seismic forces. However, the design forces would be 33 percent higher since they would be based on $R = 3$, $\Omega_o = 2.5$ and $C_d = 3$. Ordinary precast structural walls need not satisfy any provisions in ACI 318 Chapter 21.

8.2.2.2 Structural Design Considerations

8.2.2.2.1 Precast Shear Wall System. This system is designed to yield in bending at the base of the precast shear walls without shear slip at any of the joints. The remaining connections (shear connectors

and flexural connectors away from the base) are then made strong enough to ensure that the inelastic action is forced to the intended location.

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 to preclude premature yielding of other connections. For this particular example, the vertical bars at the ends of the shear walls (see Figure 8.2-6) 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 (see Figure 8.2-7) at the same joint. The connections are designed to provide the necessary shear resistance and avoid slip without providing increased flexural strength at the connection since 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.

8.2.2.2.2 Building System. No height limits are imposed (*Standard* Table 12.2-1).

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 *Standard* Table 12.6-1, use of the Equivalent Lateral Force (ELF) procedure (*Standard* Sec. 12.8) is permitted.

Orthogonal load combinations are not required for this building (*Standard* Sec. 12.5.2).

Ties, continuity and anchorage must be considered explicitly 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 they are not designed in this example.

Diaphragms need to be designed for the required forces (*Standard* Sec. 12.10), but that design is not illustrated here.

The bearing walls must be designed for a force perpendicular to their plane (*Standard* Sec. 12.11), but design for that requirement is not shown for this building.

The drift limit is $0.025h_{sx}$ (*Standard* Table 12.12-1), but drift is not computed here.

ACI 318 Section 16.5 requires minimum strengths for connections between elements of precast building structures. The horizontal forces were described in Section 8.1; the vertical forces will be described in this example.

8.2.3 Load Combinations

The basic load combinations require that seismic forces and gravity loads be combined in accordance with the factored load combinations presented in *Standard* Section 12.4.2.3. Vertical seismic load effects are described in *Standard* Section 12.4.2.2.

According to *Standard* Section 12.3.4.1, ρ = 1.0 for structures in Seismic Design Categories A, B and C, even though this seismic force-resisting system is not particularly redundant.

The relevant load combinations from ASCE 7 are as follows:

$$(1.2 + 0.2S_{DS})D \pm \rho Q_E + 0.5L$$

$$(0.9 - 0.2S_{DS})D \pm \rho Q_E$$

Into each of these load combinations, substitute S_{DS} as determined above:

$$1.26D + Q_E + 0.5L$$

$$0.843D - Q_E$$

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

8.2.4 Seismic Force Analysis

8.2.4.1 Weight Calculations. For the roof and two floors:

18-inch double tees (32 psf) + 2-inch topping (24 psf)	= 56.0 psf
Precast beams at 40 feet	= 12.5 psf
16-inch 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

Note that since the design snow load is 30 psf, it can be ignored in calculating the seismic weight (*Standard* Sec. 12.7.2). The weight of each floor including the precast shear walls is:

$$(120 \text{ ft})(150 \text{ ft})(92 \text{ psf} / 1{,}000) + [(15 \text{ ft})4 + (25 \text{ ft})2](12 \text{ ft})(0.10 \text{ ksf}) = 1{,}788 \text{ kips}$$

Considering the roof to be the same weight as a floor, the total building weight is W = 3(1,788 kips) = 5,364 kips.

8.2.4.2 Base Shear. The seismic response coefficient, C_s, is computed using *Standard* Equation 12.8-2:

$$C_S = \frac{S_{DS}}{R/I} = \frac{0.283}{4/1} = 0.0708$$

except that it need not exceed the value from *Standard* Equation 12.8-3 computed as:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.128}{0.29(4/1)} = 0.110$$

where T is the fundamental period of the building computed using the approximate method of *Standard* Equation 12.8-7:

$$T_a = C_r h_n^x = (0.02)(36)^{0.75} = 0.29 \text{ sec}$$

Therefore, use C_s = 0.0708, which is larger than the minimum specified in *Standard* Equation 12.8-5:

$$C_s = 0.044(S_{DS})(I) \geq 0.01 = 0.044(0.283)(1.0) = 0.012$$

The total seismic base shear is then calculated using *Standard* Equation 12.8-1 as:

$$V = C_s W = (0.0708)(5{,}364) = 380 \text{ kips}$$

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.

8.2.4.3 Vertical Distribution of Seismic Forces. The seismic lateral force ,F_x, at any level is determined in accordance with *Standard* Section 12.8.3:

$$F_x = C_{vx} V$$

where:

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

Since the period, T, is less than 0.5 seconds, k = l in both building directions. With equal weights at each floor level, the resulting values of C_{vx} and F_x are as follows:

- Roof: C_{vr} = 0.50; F_r = 190 kips
- Third Floor: C_{v3} = 0.333; F_3 = 127 kips
- Second Floor: C_{v2} = 0.167; F_2 = 63 kips

8.2.4.4 Horizontal Shear Distribution and Torsion

8.2.4.4.1 Longitudinal Direction. Design each of the 25-foot-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-foot-long stairwell walls in the transverse direction. The forces for each of the longitudinal walls are shown in Figure 8.2-2.

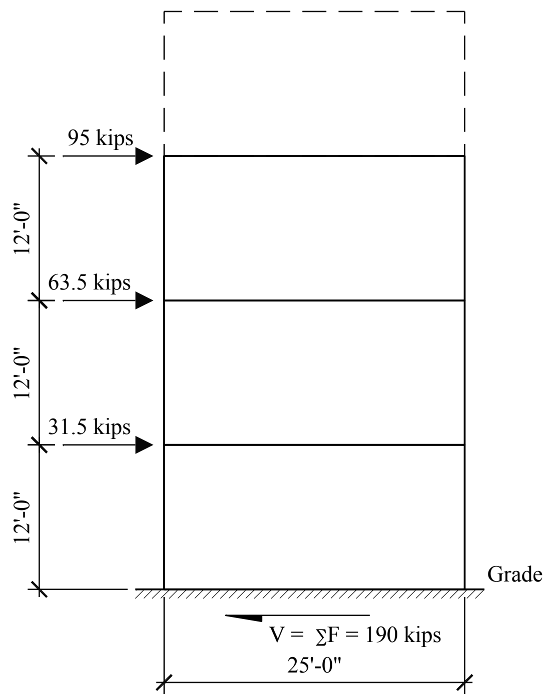

Figure 8.2-2 Forces on the longitudinal walls
(1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m)

8.2.4.4.2 Transverse Direction. Design the four 15-foot-long stairwell walls for the total shear including 5 percent accidental torsion (*Standard* Sec. 12.8.4.2). 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 follows:

V = 380/4 + 380(0.05)(150)/[(100 ft moment arm) × (2 walls in each set)] = 109 kips

Thus:

F_r = 109(0.50) = 54.5 kips

F_3 = 109(0.333) = 36.3 kips

F_2 = 109(0.167) = 18.2 kips

Seismic forces on the transverse walls of the stairwells are shown in Figure 8.2-3.

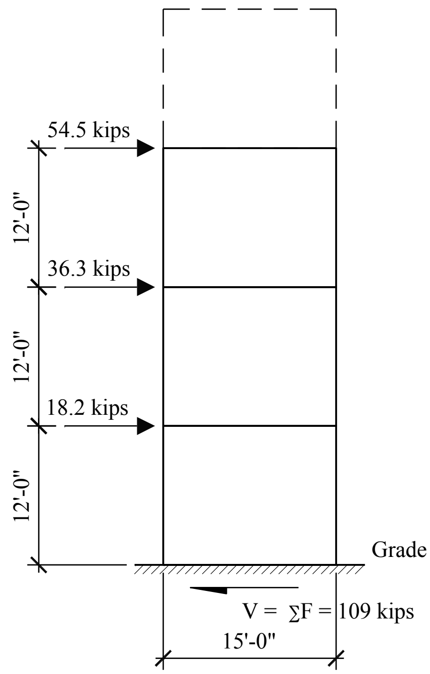

Figure 8.2-3 Forces on the transverse walls
(1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m)

8.2.5 Proportioning and Detailing

The strength of members and components is determined using the strengths permitted and required in ACI 318 Chapters 1 through 19, plus Sections 21.1.2 and 21.4.

8.2.5.1 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 (PT) bar inserted through the stack of panels, but the behavior is different than assumed by ACI 318 Section 21.4 since the PT bars don't yield. If PT bars are used, the system should be designed as an Ordinary Precast Shear Wall (allowed in SDC B.) For a building in a higher seismic design category, a post tensioned wall would need to be qualified as a Special Precast Structural Wall Based on Validation Testing per 14.2.4.

8.2.5.1.1 Longitudinal Direction. The free-body diagram for the longitudinal walls is shown in Figure 8.2-4. 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 load at 180 psf between the shafts and a roof load at 20 psf, are included. The weight for the floors includes double tees, ceiling and partitions (total load of 70 psf) but not beams and columns. Floor live load is 50 psf, except 100 psf is used in the elevator lobby. Roof snow load is 30 psf. (The elevator penthouse is so small that it was ignored in computing the gross seismic forces on the building, but it is not ignored in the following calculations.)

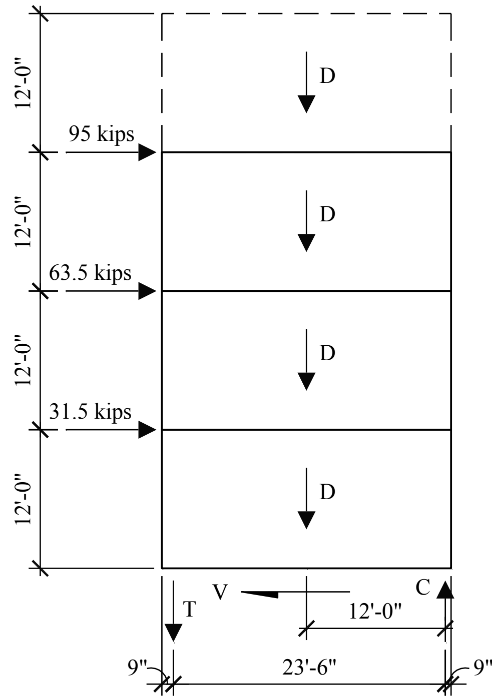

Figure 8.2-4 Free-body diagram for longitudinal walls
(1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m)

At the base:

M_E = (95 kips)(36 ft) + (63.5 kips)(24 ft) + (31.5 kips)(12 ft) = 5,320 ft-kips

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

$\sum L$ = (25 ft)(48 ft / 2)(0.05 ksf)(2) + (25 ft)(8 ft / 2)(0.1 ksf) = 60 + 10 = 70 kips

$\sum S$ = (25ft)(48 ft + 24 ft)(0.03 ksf)/2 = 27 kips

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

$P_{max} = 1.26D + 0.5L + 0.2S$ = 397 kips

$P_{min} = 0.843D$ = 239 kips

The axial load is quite small for the wall panel. The average compression P_{max}/A_g = 0.165 ksi (3.3 percent of f'_c). Therefore, the tension reinforcement can easily be found from the simple couple shown in Figure 8.2-4.

The effective moment arm is:

$jd = 25 - 1.5 = 23.5$ ft

and the net tension on the uplift side is:

$$T_u = \frac{M}{jd} - \frac{P_{\min}}{2} = \frac{5{,}320}{23.5} - \frac{239}{2} = 107 \text{ kips}$$

The required reinforcement is:

$A_s = T_u/\phi f_y$ = (107 kips)/[0.9(60 ksi)] = 1.98 in^2

Use two #9 bars (A_s = 2.0 in^2) at each end with Type 2 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. Note that if it is desired to reduce the bar size up the wall, the design check of ACI 318 Section 21.4.3 must be applied to the flexural strength calculation at the upper wall panel joints.

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

T_u = (3 kip/ft)(25 ft)/2 = 37.5 kips

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

Although no check for confinement of the compression boundary is required for intermediate precast shear walls, it is shown here for interest. Using the check from ACI 318 Section 21.9.6, the depth to the neutral axis is:

- Total compression force, $A_s f_y + P_{max}$ = (2.0)(60) + 397 = 517 kips
- Compression block, a = (517 kips)/[(0.85)(5 ksi)(8 in. width)] = 15.2 in.
- Neutral axis depth, $c = a/(0.80)$ = 19.0 in.

The maximum depth (c) with no boundary member per ACI 318 Equation 21-8 is:

$$c \le \frac{l}{600\left(\delta_u / h_w\right)}$$

where the term (δ_u/h_w) shall not be taken as 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 percent of the gross moment of inertia, ignoring the effect of axial loads and applying the C_d factor of 4 to the results gives a ratio (δ_u/h_w) far less than 0.007. Therefore, applying the 0.007 in the equation results in a distance, *c,* of 71 inches, far in excess of the 19 inches required. Thus, ACI 318 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:

$$\sigma = \frac{P}{A} + \frac{M}{S} = \frac{389}{8(25)12} + \frac{6(5{,}320)}{8(25)^2(12)} = 694 \text{ psi}$$

The limiting stress is $0.2f'_c$, which is 1,000 psi, so no transverse reinforcement is required at the ends of the longitudinal walls.

8.2.5.1.2 Transverse Direction. The free-body diagram of the transverse walls is shown in Figure 8.2-5. The weight of the precast concrete stairs is 100 psf and of the roof over the stairs is 70 psf.

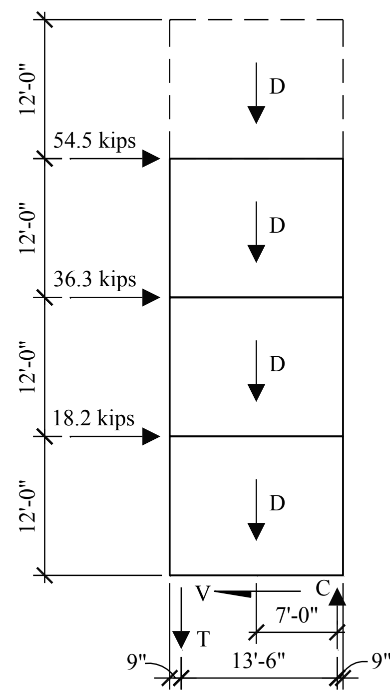

Figure 8.2-5 Free-body diagram of the transverse walls
(1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m)

The transverse wall is similar to the longitudinal wall.

At the base:

M_E = (54.5 kips)(36 ft) + (36.3 kips)(24 ft) + (18.2 kips)(12 ft) = 3,052 ft-kips

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

$\sum L$ = 2(12.5 ft / 2)(10 ft / 2)(0.05 ksf)(2) + (15 ft)(8 ft / 2)(0.1 ksf)(3)
= 6 + 18 = 24 kips

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

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

P_{min} = 0.843(107) = 90.5 kips

jd = 15 - 1.5 = 13.5 ft

$T_u = (M_{net}/jd) - P_{min}/2$ = (3,052/13.5) - 90.5/2 = 181 kips

$A_s = T_u/\phi f_y$ = (181 kips)/[0.9(60 ksi)] = 3.35 in^2

Use two #10 and one #9 bars (A_s = 3.54 in^2) at each end of each wall with a Type 2 coupler at each bar for each panel joint. All three bars at each end of the panel will also extend up through all three levels for simplicity. Following the same method for boundary member check as on the longitudinal walls:

- Total compression force, $A_s f_y + P_{max}$ = (3.54)(60) + 148 = 360 kips
- Compression block, a = (360 kips)/[(0.85)(5 ksi)(8 in. width)] = 10.6 in.
- Neutral axis depth, $c = a/(0.80)$ = 13.3 in.

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

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

Since σ < 1,000 psi, no transverse reinforcement is required at the ends of the transverse walls. Note how much closer to the criterion this transverse wall is by the compression stress check.

The overturning reinforcement and connection are shown in Figure 8.2-6.

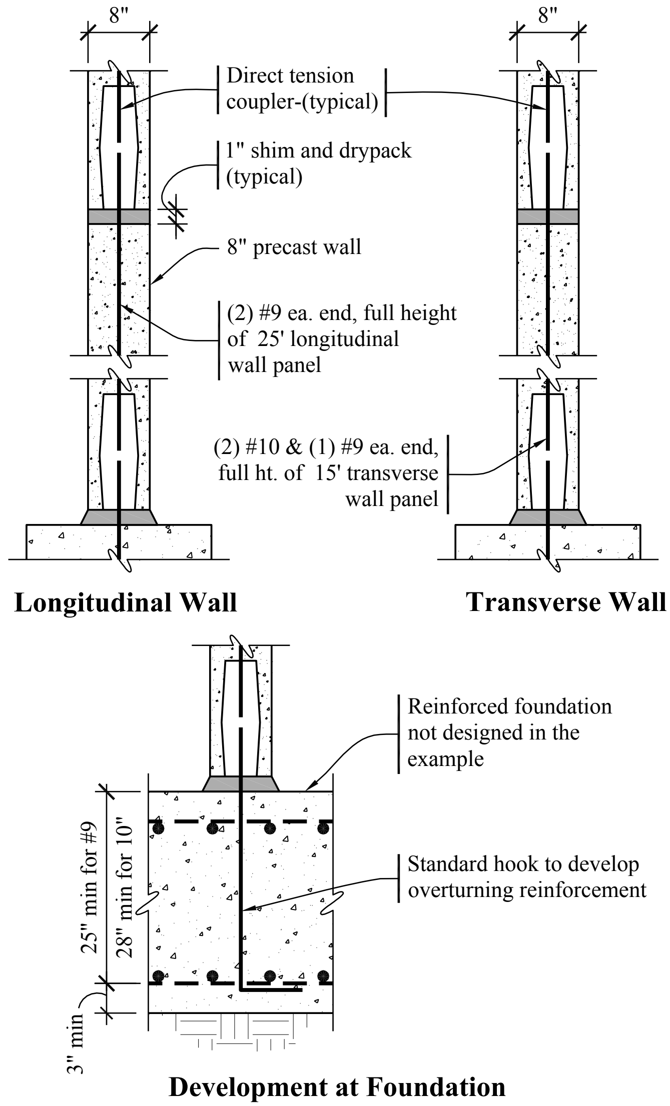

Figure 8.2-6 Overturning connection detail at the base of the walls
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

ACI 318 Section 21.4.3 requires that elements of the connection that are not designed to yield develop at least $1.5S_y$. This requirement applies to the anchorage of the coupled bars.

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:

$$1.5l_{dh} = \frac{1.5(0.7)(1{,}200)d_b}{\sqrt{f_c'}} = \frac{1{,}260(1.128)}{\sqrt{4{,}000}} = 22.5 \text{ in.}$$

Similarly, for the #10 bar, the length is 25.3 inches.

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 feet by 28 feet by 3 feet 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. Note that the *Standard* permits the overturning effects at the soil-foundation interface to be reduced under certain conditions.

8.2.5.2 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. With care taken to detail the grouted joint, shear friction can provide a reliable mechanism to resist this shear. Alternatively, the joint can be designed with direct shear connectors that will prevent slip along the joint. That concept is developed here.

8.2.5.2.1 Longitudinal Direction. The design shear force is based on the yield strength of the flexural connection. The flexural strength of the connection can be approximated as follows:

$$\frac{M_y}{M_u} = \frac{A_s f_y jd + P_{max}(jd/2)}{M_E} = \frac{\left(2.0\text{in}^2\right)\left(60\text{ ksi}\right)\left(23.5\text{ ft}\right)+\left(397\text{ kip}\right)(23.5\text{ ft}/2)}{5{,}320\text{ ft-kips}} = 1.41$$

Therefore, the design shear, V_u, at the base is 1.5(1.41)(190 kips) = 402 kips.

The base shear connection is shown in Figure 8.2-7 and is to be flexible vertically but stiff horizontally in the plane of the panel. The vertical flexibility is intended to minimize the contribution of these connections to overturning resistance, which would simply increase the shear demand.

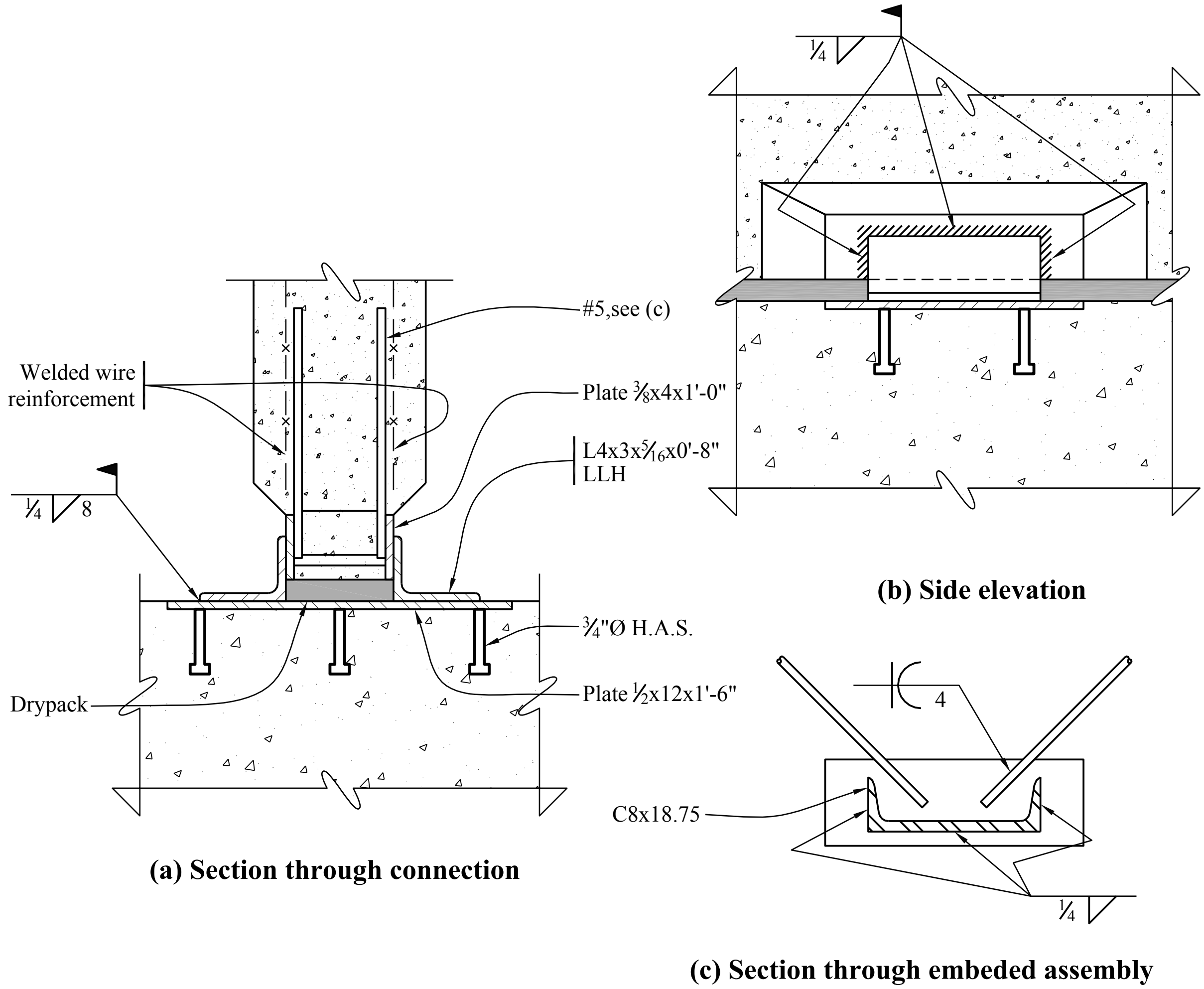

Figure 8.2-7 Shear connection at base
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

In the panel, provide an assembly with two face plates measuring 3/8" × 4" × 12" 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-inch-diameter headed anchor studs as shown. In the field, weld an L4×3×5/16 × 0'-8", long leg horizontal, on each face. The shear capacity of this connection is checked as follows:

- Shear in the two loose angles:

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

- Weld at toe of loose angles:

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

- Weld at face plates, using Table 8-8 in AISC Manual (13th edition):

  $\phi V_n = \phi C C_1 D l$ (2 sides)
  ϕ =0.75
  C_1 = 1.0 for E70 electrodes
  L = 8 in.
  D = 4 (sixteenths of an inch)
  K = 2 in. / 8 in. = 0.25
  a = eccentricity, summed vectorally: horizontal component is 4 in.; vertical component is 2.67 in.; thus, al = 4.80 in. and a = 4.8 in. / 8 in. = 0.6 from the table. By interpolation, C = 1.73
  ϕV_n = 0.75(1.73)(1.0)(4)(8)(2) = 83.0 kip

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

- Bearing of concrete at steel channel:

  $f_c = \phi(0.85f'_c) = 0.65(0.85)(5 \text{ ksi}) = 2.76 \text{ ksi}$

  The C8 has the following properties:

  t_w = 0.487 in.
  b_f = 2.53 in.
  t_f = 0.39 in. (average)

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

  $M_p = \phi F_y t_w^2/4 = (0.9)(50 \text{ ksi})(0.487^2 \text{ in}^2)/4 = 2.67$ in-kip/in.

  $M_u = f_c[(b\text{-}t_w)^2/2 - (t_w/2)^2/2] = 2.76[(b - 0.243 \text{ in.})^2 - (0.243 \text{ in.})^2]/2$

  setting the two equal results in b = 1.65 inches.

  Therefore, bearing on the channel is:

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

To the bearing capacity on the channel is added the four #5 diagonal bars, which are effective in tension and compression; ϕ = 0.75 for shear is used here:

$\phi V_s = \phi f_y A_s \cos\alpha = (0.75)(60 \text{ ksi})(4)(0.31 \text{ in}^2)(\cos 45°) = 39.5$ kip

Thus, the total capacity for transfer to concrete is:

$\phi V_n = \phi V_c + \phi V_s = 46.6 + 39.6 = 86.1$ kip

The capacity of the plate in the foundation is governed by the headed anchor studs. ACI 318 Appendix D has detailed information on calculating the strength of headed anchor studs. ACI 318 Section D3.3 has additional requirements for anchors resisting seismic forces in Seismic Design Categories C through F. 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, which is required by ACI 318 Section D3.3.4:

$\phi V_{sa} = \phi\, n\, A_{se} f_{uta}$ = (0.65)(6 studs)(0.44 in^2 per stud)(60 ksi) = 103 kip

In summary, the various shear capacities of the connection are as follows:

- Shear in the two loose angles: 130.5 kip
- Weld at toe of loose angles: 89.1 kip
- Weld at face plates: 83.0 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 =402/83.0 = 4.8

Use five connection assemblies, equally spaced along each side (4'-0" on center works well to avoid the end reinforcement). The plates are recessed to position the #5 bars within the thickness of the panel and within the reinforcement of the panel.

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

$T = M_p/x = F_y l t^2/4\,/\,(l\text{-}k)$ = (36 ksi)(8 in.)(0.3125^2 in^2)/4]/(4 in. - 0.69 in.) = 2.12 kips

There are five assemblies with two loose angles each, giving a total vertical force of 21 kips. The moment resistance is this force times half the length of the panel, which yields 265 ft-kips. The total demand moment, for which the entire system is proportioned, is 5,320 ft-kips. Thus, these connections will add approximately 5 percent to the resistance and ignoring this contribution is reasonable. If a straight plate measuring 1/4 inch by 8 inches (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 would require a much higher shear force to develop a plastic hinge at the wall base.

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

$$\phi V_c = \phi 2 A_{cv}\sqrt{f_c'}\,hd = 0.75(2)\sqrt{5{,}000}\,(8)(23.5)(12) = 239 \text{ kips}$$

Because $\phi V_c \geq V_u$ = 190 kips, the wall is adequate for shear without even considering the reinforcement. Note that the shear strength of the wall itself is not governed by the overstrength required for the connection. However, since $V_u \geq 0.5\phi V_c$ = 120 kips, ACI 318 Section 11.9.8 requires minimum wall reinforcement in accordance with ACI 318 Section 11.9.9 rather than Chapters 14 or 16. For the minimum required ρ_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 reinforcement, WWF 4×4 - W4.0×W4.0, one on each face. The shear reinforcement provided is:

$$A_v = 0.12(2) = 0.24 \text{ in}^2/\text{ft}$$

Next, compute the required connection capacity at Level 2. Even though the end reinforcing at the base extends to the top of the shear wall, the connection still needs to be checked for flexure in accordance with *Provisions* Section 21.4.3 (ACI 318 Sec. 21.4.4). At Level 2:

$$M_E = (95 \text{ kips})(24 \text{ ft}) + (63.5 \text{ kips})(12 \text{ ft}) = 3{,}042 \text{ ft-kips}$$

There are two possible approaches to the design of the joint at Level 2.

First, if Type 2 couplers are used at the Level 2 flexural connection, then the connection can be considered to have been "designed to yield," and no overstrength is required for the design of the flexural connection. In this case, the bars are designed for the moment demand at the Level 2 joint.

Alternately, if a non-yielding connection is used at the Level 2 connection, then to meet the requirements of *Provisions* Section 21.4.4 (ACI 318 Sec. 21.4.3), the flexural strength of the connection at Level 2 must be $1.5S_y$ or:

$$M_u = 1.5(1.41)M_E = 1.5(1.41)(3{,}042 \text{ ft-kips}) = 6{,}433 \text{ ft-kips}$$

At Level 2, the gravity loads on the wall are:

$$\begin{aligned}\sum D &= \text{wall} + \text{exterior floors/roof} + \text{lobby floors} + \text{penthouse floor} + \text{penthouse roof}\\ &= (25 \text{ ft})(36 \text{ ft})(0.1 \text{ ksf}) + (25 \text{ ft})(48 \text{ ft}/2)(0.070 \text{ ksf})(2) + (25 \text{ ft})(8 \text{ ft}/2)(0.070 \text{ ksf})(1) +\\ &\quad (25 \text{ ft})(8 \text{ ft}/2)(0.18 \text{ ksf}) + (25 \text{ ft})(24 \text{ ft}/2)(0.02 \text{ ksf})\\ &= 90 + 84 + 7 + 18 + 6 = 205 \text{ kips}\end{aligned}$$

$$\sum L = (25 \text{ ft})(48 \text{ ft} / 2)(0.05 \text{ ksf})(1) + (25 \text{ ft})(8 \text{ ft} / 2)(0.1 \text{ ksf}) = 30 + 10 = 40 \text{ kips}$$

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

$$P_{max} = 1.26(205) + 0.5(40) + 0.2(27) = 285 \text{ kips}$$

$$P_{min} = 0.843(205) = 173 \text{ kips}$$

Note that since the maximum axial load was used to determine the maximum yield strength of the base moment connection, the maximum axial load is used here to determine the nominal strength of the Level 2 connection. For completeness, the base moment overstrength provided should be checked using the minimum axial load as well and compared to the moment strength at Level 2 using the minimum axial load.

$$\phi M_n = 0.9\left[A_s f_y jd + P_{max}(jd/2)\right] = 0.9\left[\left(2.0\text{ in}^2\right)\left(60\text{ ksi}\right)\left(23.5\text{ ft}\right)+\left(285\text{ kips}\right)\left(23.5\text{ ft/2}\right)\right]$$
$$= 5{,}552\text{ ft-kips}$$

Therefore, the non-yielding flexural connection at Level 2 must be strengthened.

Provide:

$$T_u = \frac{M_u}{jd} - \frac{P_{min}}{2} = \frac{6{,}433}{23.5} - \frac{285}{2} = 131\text{ kips}$$

The required reinforcement is:

$A_s = T_u/\phi f_y$ = (131 kips)/[0.9(60 ksi)] = 2.43 in^2

In addition to the two #9 bars that extend to the roof, provide one #6 bar developed into the wall panel above and below the joint. Note that no increase on the development length for the #6 bar is required for this connection since the connection itself has been designed for the loads to promote base yielding per *Provisions* Section 21.4.4 (ACI 318 Sec. 21.4.3).

Since the Level 2 connection is prevented from yielding, shear friction can reasonably be used to resist shear sliding at this location. Also, because of the lack of flexural yield at the joint, it is not necessary to make the shear connection flexible with respect to vertical movement should an embedded plate detail be desired.

The design shear for this location is:

$V_{u,Level\ 2}$ = 1.5(1.41)(95+63.5) = 335 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, is* 335/83.0 = 4.04. Use four plates, equally spaced along each side.

Figure 8.2-8 shows the shear connection at the second and third floors of the longitudinal precast concrete shear wall panels.

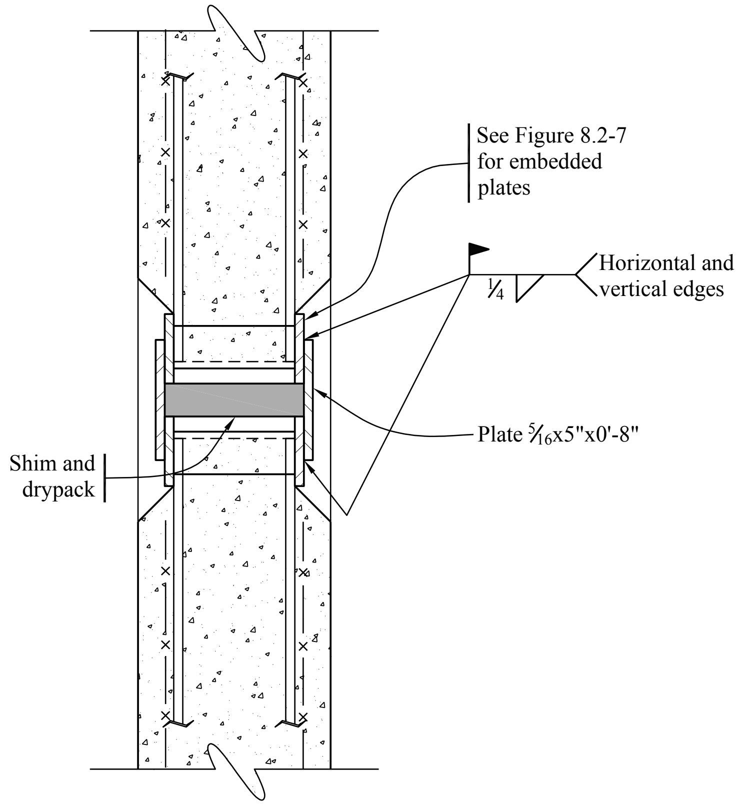

Figure 8.2-8 Shear connections on each side of the wall at the second and third floors (1.0 in = 25.4 mm)

8.3 ONE-STORY PRECAST SHEAR WALL BUILDING

This example illustrates the design of a precast concrete shear wall for a single-story building in a region of high seismicity. For buildings assigned to Seismic Design Category D, ACI 318 Section 21.10 requires that special structural walls constructed of precast concrete meet the requirements of ACI 318 Section 21.9, in addition to the requirements for intermediate precast structural walls. Alternately, special structural walls constructed using precast concrete are allowed if they satisfy the requirements of ACI ITG-5.1, *Acceptance Criteria for Special Unbonded Post-Tensioned Precast Structural Walls Based on Validation Testing (ACI ITG 5.1-07)*. Design requirements for one such type of wall have been developed by ACI ITG 5 and have been published by ACI as *Requirements for Design of a Special Unbonded Post-Tensioned Precast Shear Wall Satisfying ACI ITG-5.1 (ACI ITG 5.2-09)*. ITG 5.1 and ITG 5.2 describe requirements for precast walls for which a self-centering mechanism is provided by post-tensioning located concentrically within the wall. More general requirements for special precast walls are contained in *Provisions* Section 14.2.4. Section 14.2.4 is an updated version of Section 9.6 of the 2003 *Provisions*, which formed the basis for ITG 5.1 and ITG 5.2.

8.3.1 Building Description

The precast concrete building is a single-story industrial warehouse building (Occupancy Category II) located in the Los Angeles area on Site Class C soils. The structure has 8-foot-wide by 12.5-inch-deep

prestressed double tee (DT) wall panels. The roof is light gage metal decking spanning to bar joists that are spaced at 4 feet on center to match the location of the DT legs. The center supports for the joists are joist girders spanning 40 feet 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 feet and there is a 3-foot parapet. Figure 8.3-1 shows the plan of the building, which is regular.

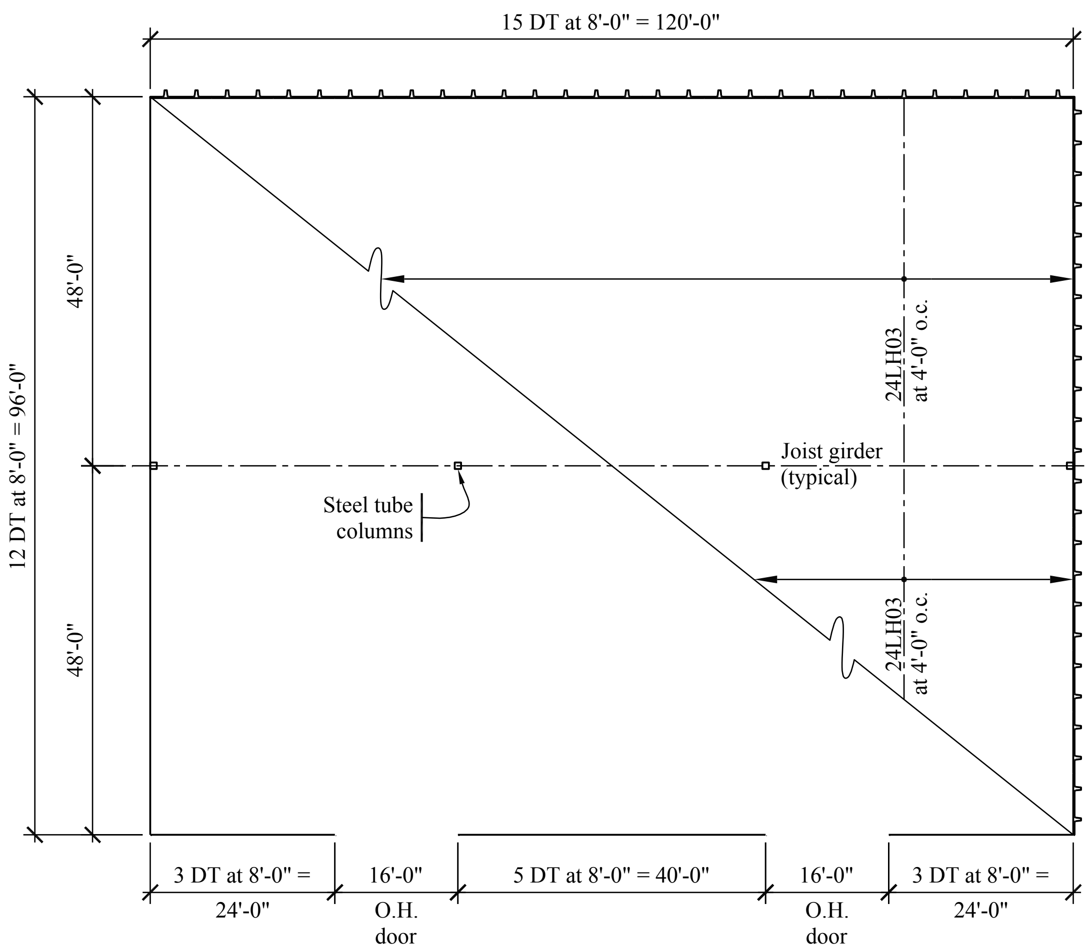

Figure 8.3-1 Single-story industrial warehouse building plan
(1.0 ft = 0.3048 m)

The precast wall panels used in this building are typical DT wall panels commonly found in many locations but not normally used in southern California. For these wall panels, an extra 1/2 inch 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.

The wall panels are normal-weight concrete with a 28-day compressive strength of $f'c$ = 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 of $f'c$ = 4,000 psi.

In *Standard* Table 12.2-1 the values for special reinforced concrete shear walls are for both cast-in-place and precast walls. In Section 2.2, ACI 318 defines a special structural wall as "a cast-in-place or precast wall complying with the requirements of 21.1.3 through 21.1.7, 21.9 and 21.10, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls." ACI 318 Section 21.10 defines requirements for special structural walls constructed using precast concrete, including that the wall must satisfy all of the requirements of ACI 318 Section 21.9.

Unfortunately, several of the requirements of ACI 318 Section 21.9 are problematic for a shear wall system constructed using DT wall panels. These include the following:

1. ACI 318 Section 21.9.2.1 requires reinforcement to be spaced no more than 18 inches on center and be continuous. This would require splices to the foundation along the DT flange.

2. ACI 318 Section 21.9.2.2 requires two curtains of reinforcement for walls with shear stress greater than $2\lambda\sqrt{f'c}$. For low loads, this might not be a problem, but for high shear stresses, placing two layers of reinforcing in a DT flange would be a challenge.

3. While ACI 318 Section 21.1.5.3 allows prestressing steel to be used in precast walls, ACI 318 Commentary R21.1.5 states that the "capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member. In interpreting experimental results, the length of the yield region has been related to the relative magnitudes of nominal and yield moments." Since prestressing steel does not have a defined yield plateau, the ratio of nominal to yield moment is undefined. This limits the ability of the structural member to develop inelastic rotation capacity—a key assumption in the definition of the *R* value for a special reinforced concrete wall system.

Therefore, these walls will be designed using the ACI category of intermediate precast structural walls.

8.3.2 Design Requirements

8.3.2.1 Seismic Parameters of the *Provisions.* The basic parameters affecting the design and detailing of the building are shown in Table 8.3-1.

Table 8.3-1 Design Parameters

Design Parameter	Value
Occupancy Category II	$I = 1.0$
S_S	1.5
S_1	0.60
Site Class	C
F_a	1.0
F_v	1.3
$S_{MS} = F_a S_S$	1.5
$S_{M1} = F_v S_1$	0.78
$S_{DS} = 2/3\ S_{MS}$	1.0
$S_{D1} = 2/3\ S_{M1}$	0.52
Seismic Design Category	D
Basic Seismic Force-Resisting System	Bearing Wall System
Wall Type	Intermediate Precast Structural Wall
R	4
Ω_0	2.5
C_d	4

8.3.2.2 Structural Design Considerations

8.3.2.2.1 Intermediate Precast Structural Walls Constructed Using Precast Concrete. The intent of the intermediate precast structural wall requirements 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 ACI 318 Section 21.4, these connections must yield only in steel elements or reinforcement and all other elements of the connection (including shear resistance) must be designed for 1.5 times the force associated with the flexural yield strength of the connection.

Yielding will develop in the dry connection at the base by bending in 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 $1.5S_y$ requirements of ACI 318 Section 21.4.3. 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.

8.3.2.2.2 Building System. The height limit in Seismic Design Category D (*Standard* Table 12.2-1) is 40 feet.

The metal deck roof acts as a flexible horizontal diaphragm to distribute seismic inertia forces to the walls parallel to the earthquake motion *(Standard* Sec. 12.3.1.1).

The building is regular both in plan and elevation.

The redundancy factor, ρ, is determined in accordance with *Standard* Section 12.3.4.2. For this structure, which is regular and has more than two perimeter wall panels (bays) on each side in each direction, $\rho = 1.0$.

The structural analysis to be used is the ELF procedure (*Standard* Sec. 12.8) as permitted by *Standard* Table 12.6-1.

Orthogonal load combinations are not required for flexible diaphragms in Seismic Design Category D (*Standard* Sec. 12.5.4).

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

Ties, continuity and anchorage (*Standard* 12.11) must be considered explicitly when detailing connections between the roof and the wall panels. This example does not include the design of those connections, but sketches of details are provided to guide the design engineer.

There are no drift limits for single-story buildings as long as they are designed to accommodate predicted lateral displacements (*Standard* Table 12.12-1, Footnote c).

8.3.3 Load Combinations

The basic load combinations (*Standard* Sec. 12.4.2.3) require that seismic forces and gravity loads be combined in accordance with the following factored load combinations:

$$(1.2 + 0.2S_{DS})D \pm \rho Q_E + 0.5L + 0.2S$$

$$(0.9 - 0.2S_{DS})D \pm \rho Q_E + 1.6H$$

At this flat site, both S and H equal 0. Note that roof live load need not be combined with seismic loads, so the live load term, L, can be omitted from the equation. Therefore:

$$1.4D + \rho Q_E$$

$$0.7D - \rho Q_E$$

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

8.3.4 Seismic Force Analysis

8.3.4.1 Weight Calculations. Compute the weight tributary to the roof diaphragm:

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

The total weight of the roof is computed as:

(120 ft × 96 ft)(12 psf / 1,000) = 138 kips

The exterior DT wall weight tributary to the roof is:

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

Total building weight for seismic lateral load, W = 138+236 = 374 kips

8.3.4.2 Base Shear. The seismic response coefficient (C_s) is computed using *Standard* Equation 12.8-2 as:

$$C_S = \frac{S_{DS}}{R/I} = \frac{1.0}{4/I} = 0.25$$

except that it need not exceed the value from *Standard* Equation 12.8-3, as follows:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.52}{0.189(4/1)} = 0.69$$

where T is the fundamental period of the building computed using the approximate method of *Standard* Equation 12.8-7:

$$T_a = C_r h_n^x = (0.02)\left(20.0\right)^{0.75} = 0.189 \text{ sec}$$

Therefore, use C_s = 0.25, which is larger than the minimum specified in *Standard* Equation 12.8-5:

$C_s = 0.044(S_{DS})(I) \geq 0.01 = 0.044(1.0)(1.0) = 0.044$

The total seismic base shear is then calculated using *Standard* Equation 12.8-1, as:

$V = C_sW = (0.25)(374) = 93.5$ kips

8.3.4.3 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.

8.3.4.3.1 Longitudinal Direction. The total shear along each side of the building is $V/2$ = 46.75 kips. The maximum shear on longitudinal panels (at the side with the openings) is:

V_{lu} = 46.75/11 = 4.25 kips

On each side, each longitudinal wall panel resists the same shear force as shown in the free-body diagram of Figure 8.3-2, where D_1 represents roof joist reactions and D_2 is the panel weight.

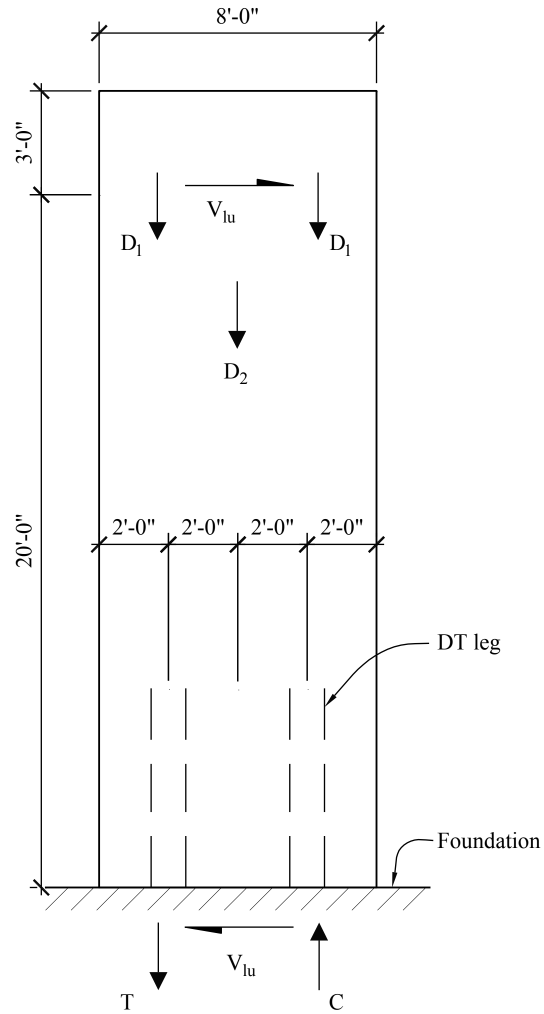

Figure 8.3-2 Free-body diagram of a panel in the longitudinal direction
(1.0 ft = 0.3048 m)

8.3.4.3.2 Transverse Direction. Seismic forces on the transverse wall panels are all equal and are:

V_{tu} = 46.75/12 = 3.90 kips

Figure 8.3-3 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.

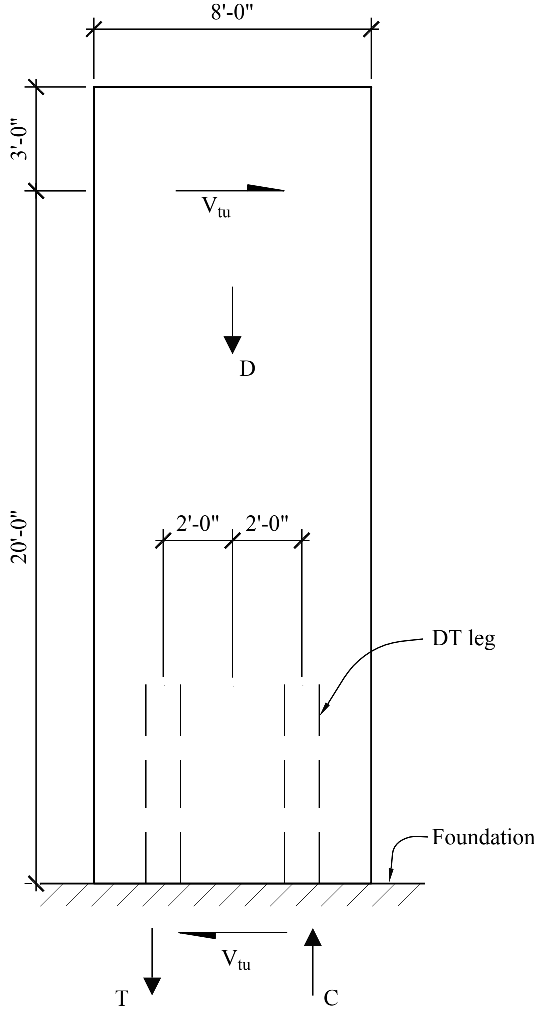

Figure 8.3-3 Free-body diagram of a panel in the transverse direction
(1.0 ft = 0.3048 m)

8.3.5 Proportioning and Detailing

The strength of members and components is determined using the strengths permitted and required in ACI 318 including Chapter 21.

8.3.5.1 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. ACI 318

Section 21.4 requires that dry connections at locations of nonlinear action comply with applicable requirements of monolithic concrete construction and satisfy both of the following:

1. Where the moment action on the connection is assumed equal to $1.5M_y$, the co-existing forces on all other components of the connection other than the yielding element shall not exceed their design strength.

2. The nominal shear strength for the connection shall not be less than the shear associated with the development of $1.5M_y$ at the connection.

8.3.5.1.1 Longitudinal Direction. Use the free-body diagram shown in Figure 8.3-2. 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 = (4.25 kips)(20 ft) = 85.0 ft-kips

- Dead loads:

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

  D_2 = 0.042(23)(8) = 7.73 kips

  ΣD = 2(1.08) + 7.73 = 9.89 kips

  $1.4D$ = 13.8 kips

  $0.7D$ = 6.92 kips

Compute the tension force due to net overturning based on an effective moment arm, *d,* of 4.0 feet (the distance between the DT legs). The maximum is found when combined with $0.7D$:

$T_u = M_E/d - 0.7D/2$ = 85.0/4 - 6.92/2 = 17.8 kips

8.3.5.1.2 Transverse Direction. For the transverse direction, use the free-body diagram of Figure 8.3-3. 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 = (3.90 kips)(20 ft) = 78.0 ft-kips

The dead load of the panel (as computed above) is D_2 = 7.73 kips and $0.7D$ = 5.41.

The tension force is computed as above for d = 4.0 feet (the distance between the DT legs):

T_u = 78.0/4 - 5.41/2 = 16.8 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.

8.3.5.2 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 an L5×3-1/2×3/4 × 0'-6-1/2" 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 8.3-4.

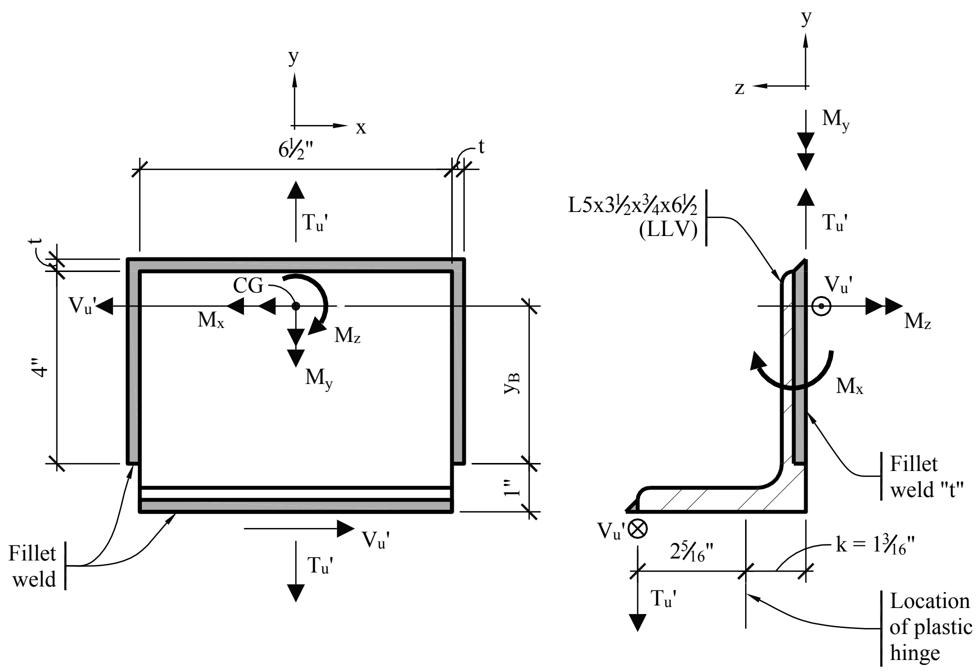

Figure 8.3-4 Free-body of the angle and the fillet weld connecting the embedded plates in the DT and the footing (elevation and section)
(1.0 in = 25.4 mm)

The location of the plastic hinge in the angle is at the toe of the fillet (at a distance, *k,* from the heel of the angle.) The bending moment at this location is:

$M_u = T_u(3.5 - k) = 17.8(3.5 - 1.1875) = 41.2$ in.-kips

$$\phi_b M_n = 0.9F_y Z = 0.9(50)\left[\frac{6.5(0.75)^2}{4}\right] = 41.1 \text{ in-kips}$$

Providing a stronger angle (e.g., a shorter horizontal leg) will simply increase the demands on the remainder of the assembly. Using ACI 318 Section 21.4.3, the tension force for the remainder of this

connection and the balance of the wall design are based upon a probable strength equal to 150 percent of the yield strength. Thus:

$$T_{pr} = \frac{M_n(1.5)}{3.5 - k} = \frac{(50)(6.5)(0.75)^2 / 4}{0.9(3.5 - 1.1875)} \times 1.5 = 27.0 \text{ kips}$$

The amplifier, required for the design of the balance of the connection, is:

$$\frac{T_{pr}}{T_u} = \frac{27.0}{17.8} = 1.52$$

The shear on the connection associated with this force in the angle is:

$$V_{pr} = V_E \frac{T_{pr}}{T_u} = 4.25 \times 1.52 = 6.46 \text{ kips}$$

Check the welds for the tension force of 27.0 kips and a shear force 6.46 kips.

The *Provisions* Section 21.4.4 (ACI 318 21.4.3) requires that connections that are designed to yield be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement. For yielding of a flat bar (angle leg), this can be checked by calculating the ductility capacity of the bar and comparing it to C_d. Note that the element ductility demand (to be calculated below for the yielding angle) and the system ductility, C_d, are only equal if the balance of the system is rigid. This is a reasonable assumption for the intermediate precast structural wall system described in this example.

The idealized yield deformation of the angle can be calculated as follows:

$$P_y = \frac{M_n}{L} = \frac{50(6.5)(0.75^2)/4}{2.25} = 19.8 \text{ kips}$$

$$\Delta_{y,idealized} = \frac{P_y L^3}{3EI} = \frac{19.83(2.31^3)}{3(29{,}000)(6.5 \times 0.75^3/12)} = 0.012 \text{ in.}$$

It is conservative to limit the maximum strain in the bar to $\varepsilon_{sh} = 15\varepsilon_y$. At this strain, a flat bar would be expected to retain all its strength and thus meet the requirement of maintaining 80 percent of its strength.

Assuming a plastic hinge length equal to the section thickness:

$$\phi_p = \frac{15\varepsilon_y}{d/2} = \frac{15(50/29{,}000)}{0.75/2} = 0.06897$$

$$\Delta_{sh} = \phi_p L_p \left(L - \frac{L_p}{2}\right) + \Delta_y = 0.06897(0.75)\left(2.31 - \frac{0.75}{2}\right) + 0.012 = 0.112 \text{ in.}$$

Since the ductility capacity at strain hardening is 0.112/0.012 = 9.3 is larger than $C_d = 4$ for this system, the requirement of *Provision* Section 21.4.4 (ACI 318 Sec. 21.4.3) is met.

8.3.5.3 Welds to Connection Angle. Welds will be fillet welds using E70 electrodes.

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

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

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

  For which the limiting stress is 22.3 ksi.

Size a fillet weld, 6.5 inches long at the angle to the embedded plate in the footing. Using an elastic approach:

$$\text{Resultant force} = \sqrt{V_{pr}^2 + T_{pr}^2} = \sqrt{6.46^2 + 27.0^2} = 27.8 \text{ kips}$$

$$A_w = 27.8/22.3 = 1.24 \text{ in}^2$$

$$t = A_w/l = 1.24 \text{ in}^2 / 6.5 \text{ in.} = 0.19 \text{ in.}$$

For a 3/4 inch angle leg, use a 5/16 inch fillet weld. Given the importance of this weld, increasing the size to 3/8 inch 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 6.5 inches long across the top and 4 inches long on each vertical leg of the angle. Using the free-body diagram of Figure 8.3-4 for tension and Figure 8.3-5 for shear, the weld moments and stresses are:

$$M_x = T_{pr}(3.5) = 27.0(3.5) = 94.5 \text{ in-kips}$$
$$M_y = V_{pr}(3.5) = (6.46)(3.5) = 22.6 \text{ in-kips}$$
$$M_z = V_{pr}(y_b + 1.0)$$
$$= 6.46(2.77 + 1.0) = 24.4 \text{ in-kips}$$

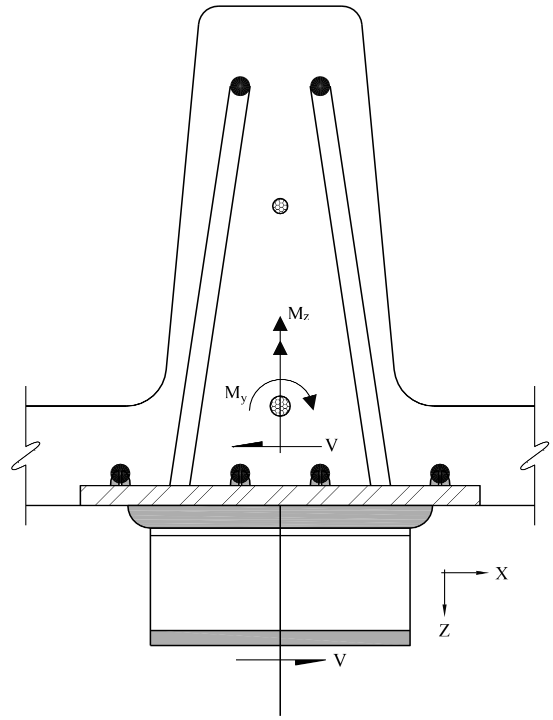

Figure 8.3-5 Free-body of angle with welds, top view, showing only shear forces and resisting moments

For the weld between the angle and the embedded plate in the DT as shown in Figure 8.3-5, the section properties for a weld leg (t) are:

A = 14.5t in^2

I_x = 25.0t in^4

I_y = 107.4t in^4

$I_p = I_x + I_y$ = 132.4t in^4

y_b = 2.90 in.

x_L = 3.25 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 8.3-4.

$$\sigma_x = \frac{V_{pr}}{A} + \frac{M_z y_b}{I_p} = \frac{6.46}{14.5t} + \frac{(24.4)(2.90)}{132.4t} = \frac{0.98}{t} \text{ ksi}$$

$$\sigma_y = -\frac{T_{pr}}{A} + \frac{M_z x_L}{I_p} = -\frac{27.0}{14.5t} + \frac{(24.4)(3.25)}{132.4t} = \frac{-1.26}{t} \text{ ksi}$$

$$\sigma_z = -\frac{M_y x_L}{A} - \frac{M_z y_b}{I_p} = -\frac{(22.6)(3.25)}{107.4t} - \frac{(94.5)(2.90)}{25.0t} = \frac{-11.8}{t} \text{ ksi}$$

$$\sigma_R = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2} = \frac{1}{t}\sqrt{0.98^2 + 1.26^2 + 11.8^2} = \frac{11.9}{t} \text{ ksi}$$

Thus, t = 11.9/22.3 = 0.53 inch, which can be taken as 9/16 inch. Field welds are conservatively sized with the elastic method for simplicity and to minimize construction issues.

8.3.5.4 Panel Reinforcement. Check the maximum compressive stress in the DT leg. Note that for an intermediate precast structural wall, ACI 318 Section 21.9.6 does not apply and transverse boundary element reinforcing is not required. However, the cross section must be designed for the loads associated with 1.5 times the moment that yields the base connectors.

Figure 8.3-6 shows the cross section used. The section is limited by the area of dry-pack under the DT at the footing.

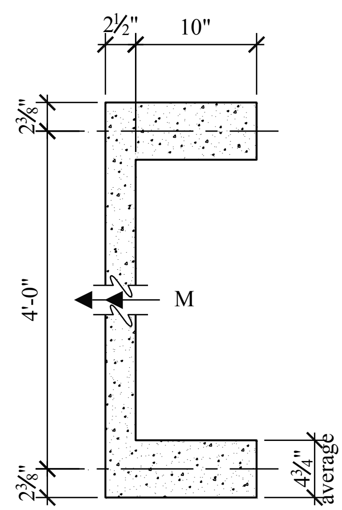

Figure 8.3-6 Cross section of the DT dry-packed at the footing
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

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:

A = 227 in^2

S = 3240 in^3

$$\sigma_z = \frac{P}{A} + \frac{M_E}{S} = \frac{13,800}{227} + \frac{1.52(85,000 \times 12)}{3,240} = 539 \text{ ksi}$$

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-inch-diameter strand and one 3/8-inch-diameter strand. Figure 8.3-7 shows the location of these prestressed strands.

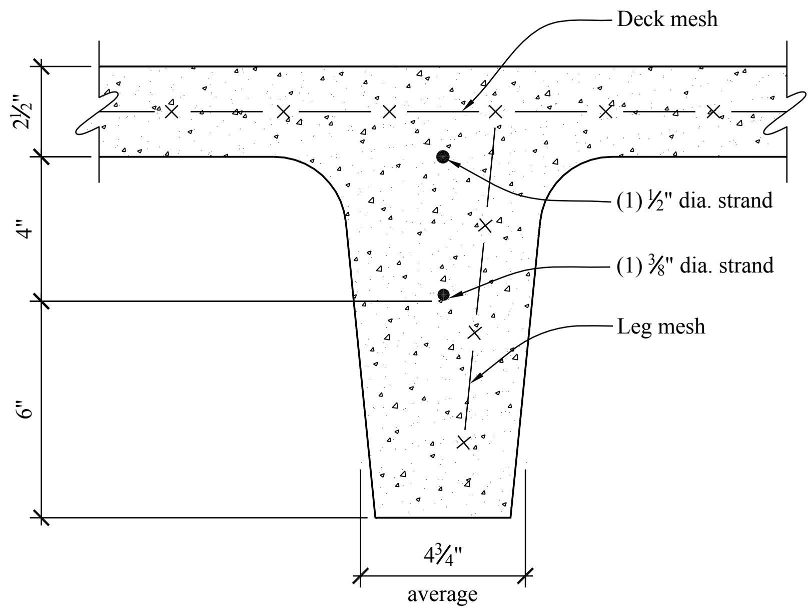

Figure 8.3-7 Cross section of one DT leg showing the location of the bonded prestressing tendons or strand (1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

Next, compute the compressive stress resulting from these strands. Note that the moment at the height of strand development above the footing, about 26 inches for the effective stress (f_{se}), is less than at the top of footing. This reduces the compressive stress by:

$$\frac{(4.25)(26)}{3,240} \times 1,000 = 34 \text{ psi}$$

In each leg, use:

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

$A = 168$ in^2

$e = y_b$ - $CG_{Strand} = 9.48 - 8.57 = 0.91$ in.

S_b = 189 in^3

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

Therefore, the total compressive stress is approximately 539 + 402 - 34 = 907 psi.

Since yielding is restricted to the steel angle and the DT is designed to be 1.5 times stronger than the yield force in the steel angle, the full strength of the strand can be used to resist axial forces in the DT stem, without concern for yielding in the strand.

D_2 = (0.042)(20.83)(8) = 7.0 kips

P_{min} = 0.7(7.0 + 2(1.08)) = 6.41 kips

M_E = (1.52)(4.25)(17.83) = 115.2 ft-kips

$T_{u,stem} = M_E/d - P_{min}/2$= 25.5 kips

The area of tension reinforcement required is:

$A_{ps} = T_{u,stem}/\phi f_{py}$ = (25.5 kips)/[0.9(270 ksi)] = 0.10 in^2

The area of one 1/2-inch-diameter strand and one 3/8-inch-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 is 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 318 Section 11.9. The demand on each panel is:

$V_u = V_{pr}$ = 6.46 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 318 Section 11.9.

$$\phi V_c = \phi 2\lambda\sqrt{f_c'}hd = 0.75(2)(1.0)\sqrt{5{,}000}(2.5)(48) = 12.7 \text{ kips}$$

Since V_u of 6.46 kips exceeds $\phi V_u/2$ of 6.36 kips, provide minimum reinforcement per ACI 318 Section 11.9.9.2. Using welded wire reinforcement, the required areas of reinforcement are:

$A_v = A_{vh}$ = (0.0025)(2.5)(12) = 0.075 in^2/ft

Provide 6×6 – W4.0×W4.0 welded wire reinforcement.

$A_{sv} = A_{sh}$ = 0.08 in^2/ft

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.

8.3.5.5 Tension and Shear at the Footing Embedment. Reinforcement to anchor the embedded plates is sized for the same tension and shear. Reinforcement in the DT leg and in the footing will be welded to embedded plates as shown in Figure 8.3-8.

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

$$A_{s,Sloped} = \frac{T_{u,stem}}{\phi f_y \cos\theta} = \frac{27.0}{(0.9)(60)\cos 26.5°} = 0.56 \text{ in}^2$$

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

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

ϕV_n = 0.75(0.2)(2)(60)(cos26.5°) = 16.1 kips

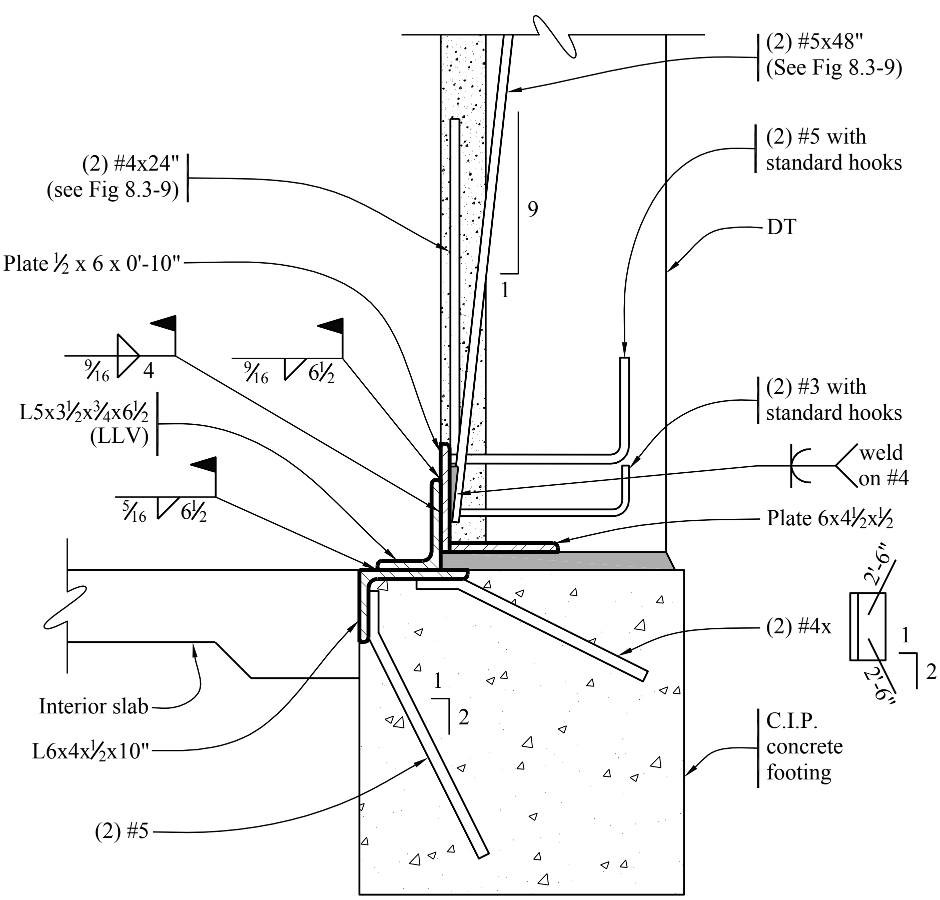

Figure 8.3-8 Section at the connection of the precast/prestressed shear wall panel and the footing
(1.0 in = 25.4 mm)

8.3.5.6 Tension and Shear at the DT Embedment. The area of reinforcement for the welded bars of the embedded plate in the DT, which develops tension as the angle bends through cycles, is:

$$A_s = \frac{T_{u,stem}}{\phi f_y \cos\theta} = \frac{27.0}{(0.9)(60)\cos 6.3°} = 0.503 \text{ in}^2$$

Two #5 bars are adequate. Note that the bars in the DT leg are required to extend upward the development length of the bar, which would be 22 inches. In this case, they will be extended 22 inches past the point of development of the effective stress in the strand, which totals approximately 48 inches.

The same embedded plate used for tension will also be used to resist one-half the nominal shear. This shear force is 6.46 kips. The transfer of direct shear to the concrete is easily accomplished with bearing on the sides of the reinforcing bars welded to the plate. Two #5 and two #4 bars (explained later) are welded to the plate. The available bearing area is approximately A_{br} = 4(0.5 in.)(5 in.[available]) = 10 in^2 and the bearing capacity of the concrete is ϕV_n = (0.65)(0.85)(5 ksi)(10 in^2) = 27.6 kips, which is greater than the 6.46 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.) Using the weld capacity for the #5 bar:

$$\phi V_n = (0.75)(0.6)(70 \text{ ksi})(0.2)(0.625 \text{ in.})(2) = 7.9 \text{ kips/in}$$

The shear demand is prorated among the four bars as (6.46 kip)/4 = 1.6 kips. The tension demand is $T_{u,stem}$/2(13.5 kips). The vectorial sum of shear and tension demand is 13.6 kips. Thus, the minimum length of weld is 13.6/7.9 = 1.7 inches.

8.3.5.7 Resolution of Eccentricities at the DT Embedment. Check the twisting of the embedded plate in the DT for M_z. Use M_z = 24.4 in-kips.

$$A_s = \frac{M_z}{\phi f_y (jd)} = \frac{24.4}{0.9(60)(9.0)} = 0.05 \text{ in}^2$$

Use one #4 bar on each side of the vertical embedded plate in the DT as shown in Figure 8.3-9. This is the same bar used to transfer direct shear in bearing.

Check the DT embedded plate for M_y (equal to 22.6 in-kips) and M_x (equal to 94.5 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 8.3-9). It is relatively straightforward to compute the resultant moment magnitude and direction, assume a triangular 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. That approximate method is demonstrated here. The #5 bars are effective in resisting M_x and one each of the #3 and #5 bars are effective in resisting M_y. For M_y assume that the effective depth extends 1 inch beyond the edge of the angle (equal to twice the thickness of the plate). Begin by assigning one-half of the "corner" #5 bar to each component.

With $A_{sx} = 0.31 + 0.31/2 = 0.47$ in^2:

$$\phi M_{nx} = \phi A_s f_y jd = (0.9)(0.47 \text{ in}^2)(60 \text{ ksi})(0.95)(5 \text{ in.}) = 120 \text{ in-kips } (> 94.5 \text{ in-kips})$$

With $A_{sy} = 0.11 + 0.31/2 = 0.27$ in^2:

$$\phi M_{ny} = \phi A_s f_y jd = (0.9)(0.27 \text{ in}^2)(60 \text{ ksi})(0.95)(5 \text{ in.}) = 69 \text{ in-kips } (> 22.6 \text{ in-kips})$$

Each component is strong enough, so the proposed bars are satisfactory.

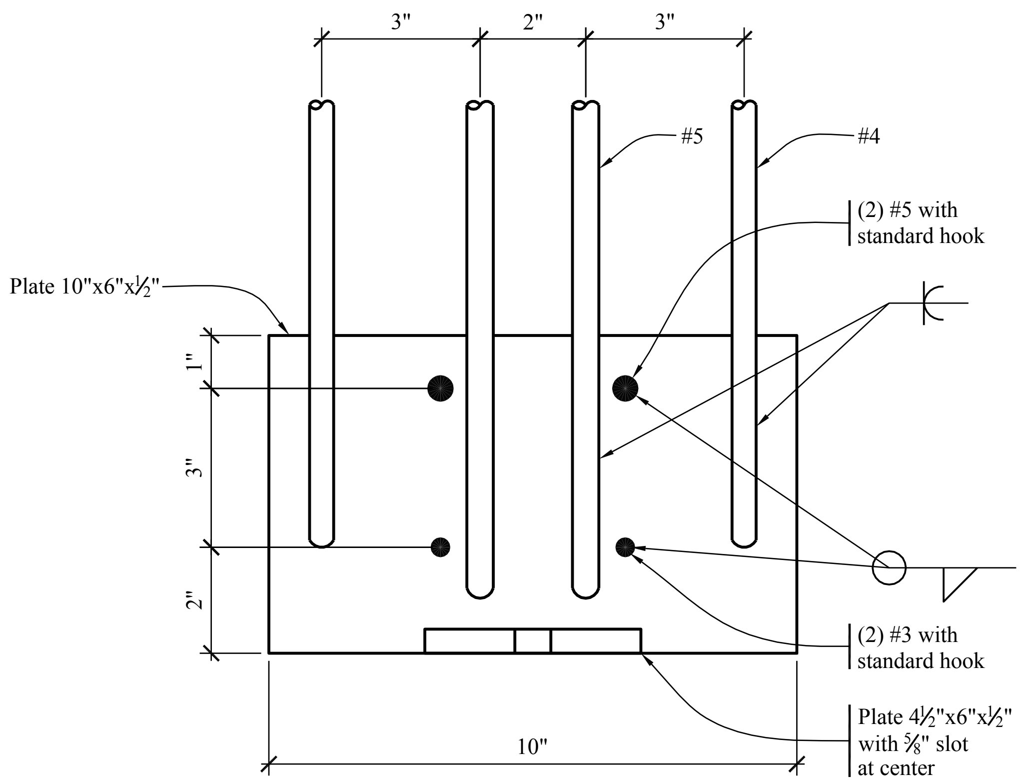

Figure 8.3-9 Details of the embedded plate in the DT at the base
(1.0 in = 25.4 mm)

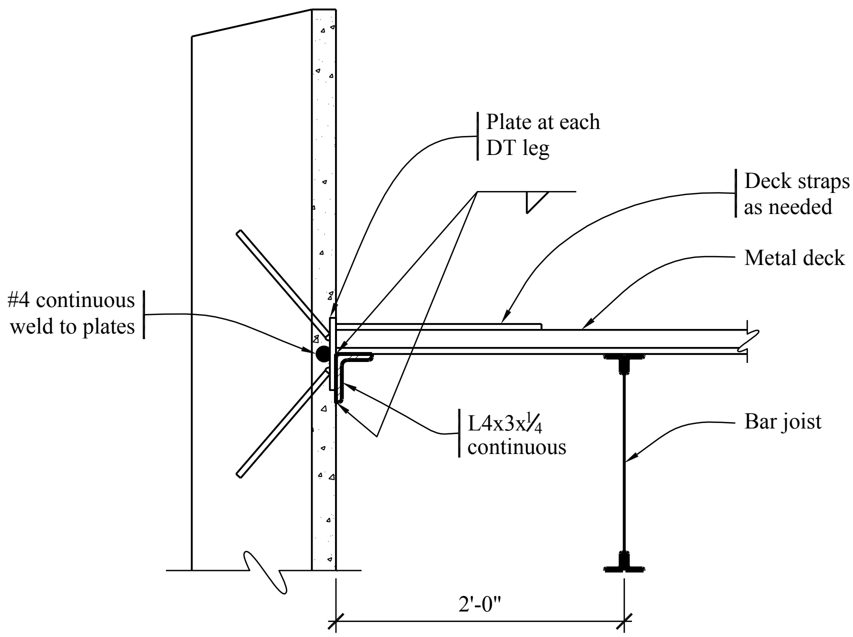

Figure 8.3-10 Sketch of connection of non-load-bearing DT wall panel at the roof
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

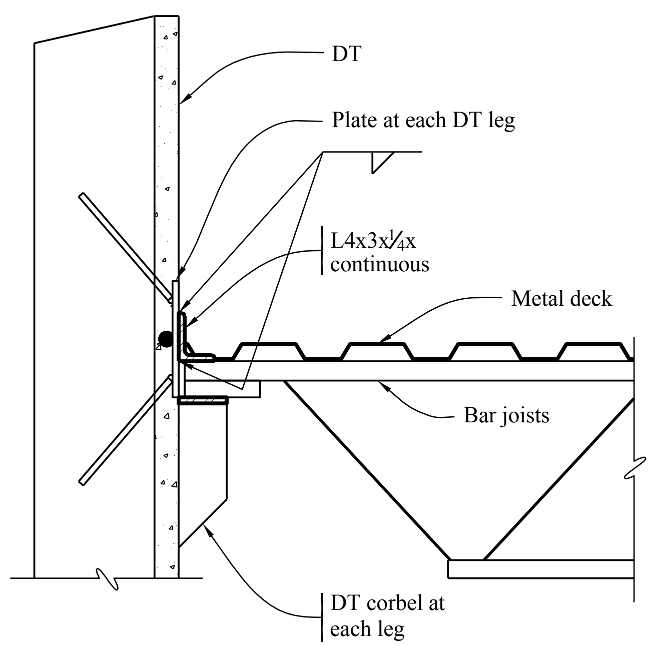

Figure 8.3-11 Sketch of connection of load-bearing DT wall panel at the roof
(1.0 in = 25.4 mm)

8.3.5.8 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 8.3-10 and 8.3-11 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, *Standard* Section 12.11.2.1 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 accordingly.

In precast DT shear wall panels with flanges thicker than 2-1/2 inches, 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.

8.4 SPECIAL MOMENT FRAMES CONSTRUCTED USING PRECAST CONCRETE

As for special concrete walls, the *Standard* does not distinguish between a cast-in-place and a precast concrete special moment frame in Table 12.2-1. However, ACI 318 Section 21.8 provides requirements for special moment frames constructed using precast concrete. That section provides requirements for designing special precast concrete frame systems using either ductile connections (ACI 318 Sec. 21.8.2) or strong connections (ACI 318 Sec. 21.8.3.) ACI 318 Section 21.8.4 also explicitly allows precast moment frame systems that meet the requirements of ACI 374.1, *Acceptance Criteria for Moment Frames based on Structural Testing*.

8.4.1 Ductile Connections

For moment frames constructed using ductile connections, ACI 318 requires that plastic hinges be able to form in the connection region. All of the requirements for special moment frames must still be met, plus there is an increased factor that must be used in developing the shear demand at the joint.

It is interesting to note that while Type 2 connectors can be used anywhere (including in a plastic hinge region) in a cast-in-place frame, these same connectors cannot be used closer than $h/2$ from the joint face in a ductile connection. The rationale behind this requirement is that in a jointed system, a concentrated crack occurs at the joint between precast elements in a ductile connection. Thus the rotation is concentrated at this location. Type 2 connectors are actually strong connections, relative to the bar, as they are designed to develop the tensile strength of the bar. The objective of Type 2 connectors is that they relocate the yielding away from the connector, into the bar itself.

If a Type 2 connector is used at the face of a column as shown in Figure 8.4-1 and the bar size is the same in both the column and the beam, yielding will occur at the joint at the face of the column but not be able to spread into the beam to develop a plastic hinge, due to the strength of the connector. This concentrates the yielding in the bar to the left of the connector and likely will fracture the bar when significant rotation is imposed on the beam.

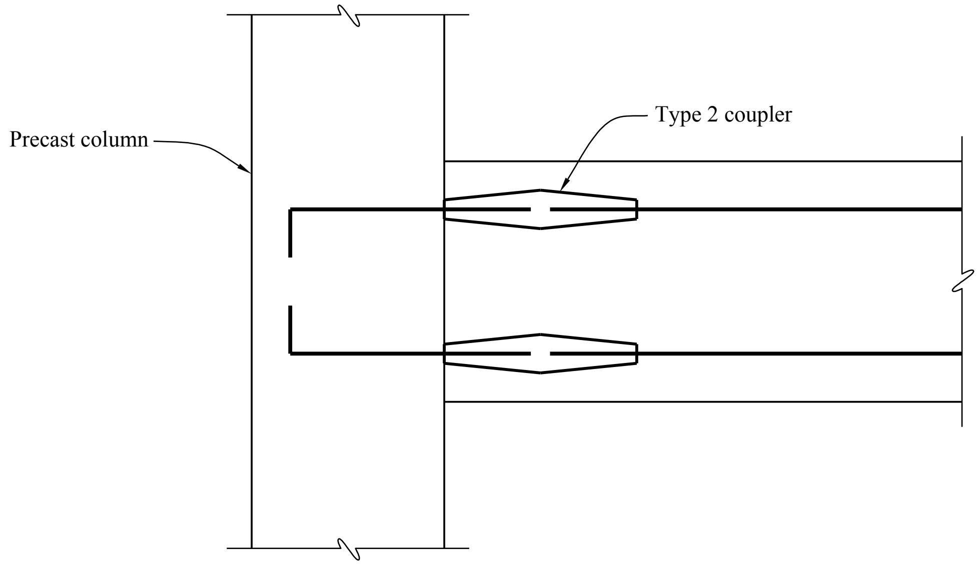

Figure 8.4-1 Type 2 coupler location in a strong connection
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

In a ductile connection, frame yielding takes place at the connection. This is most easily accomplished by extending the reinforcement out of the precast column element and coupling this rebar at the end of the precast beam. Since the couplers have to be located a minimum distance of $h/2$ from the joint face (i.e., column face) the resulting gap between the precast beam and precast column is filled with cast-in-place concrete as shown in Figure 8.4-2.

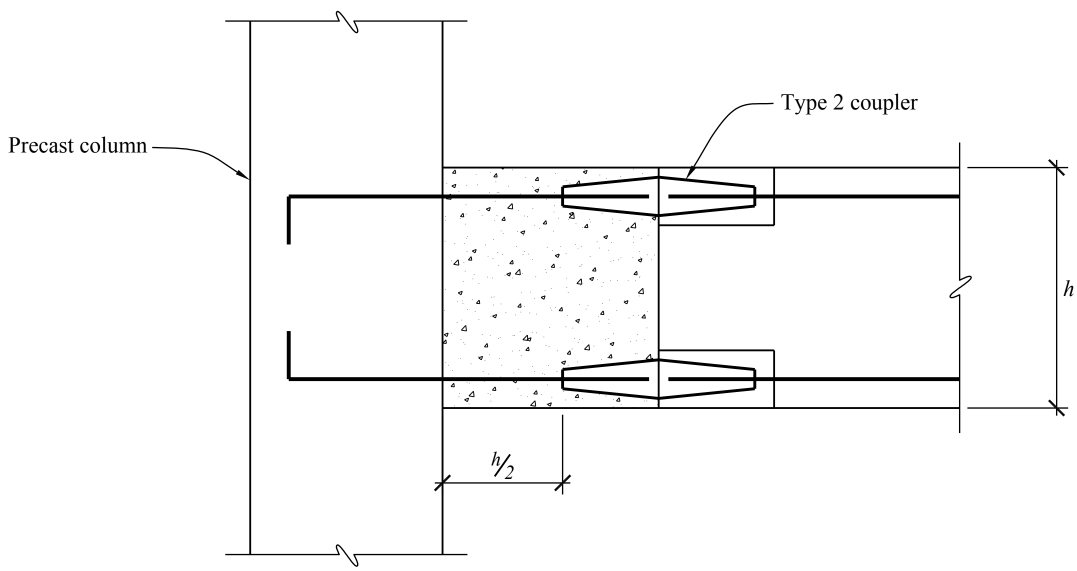

Figure 8.4-2 Type 2 coupler location in a ductile connection
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

8.4.2 Strong Connections

ACI 318 also provides design rules for strong connections used in special moment frames. The concept is to provide connections that are strong enough to remain elastic when a plastic hinge forms in the beam. Thus the frame behavior is the same as would occur if the connection were monolithic.

Using the frame in Figure 8.4-3 (ignoring gravity forces for simplicity), design forces for the plastic hinge region and the associated forces on the precast connection are computed. Assuming inflection points at mid-height of the columns and a seismic shear force of V_{col} on each column:

$$V_b = V_{col} \frac{H_c}{L_b}$$

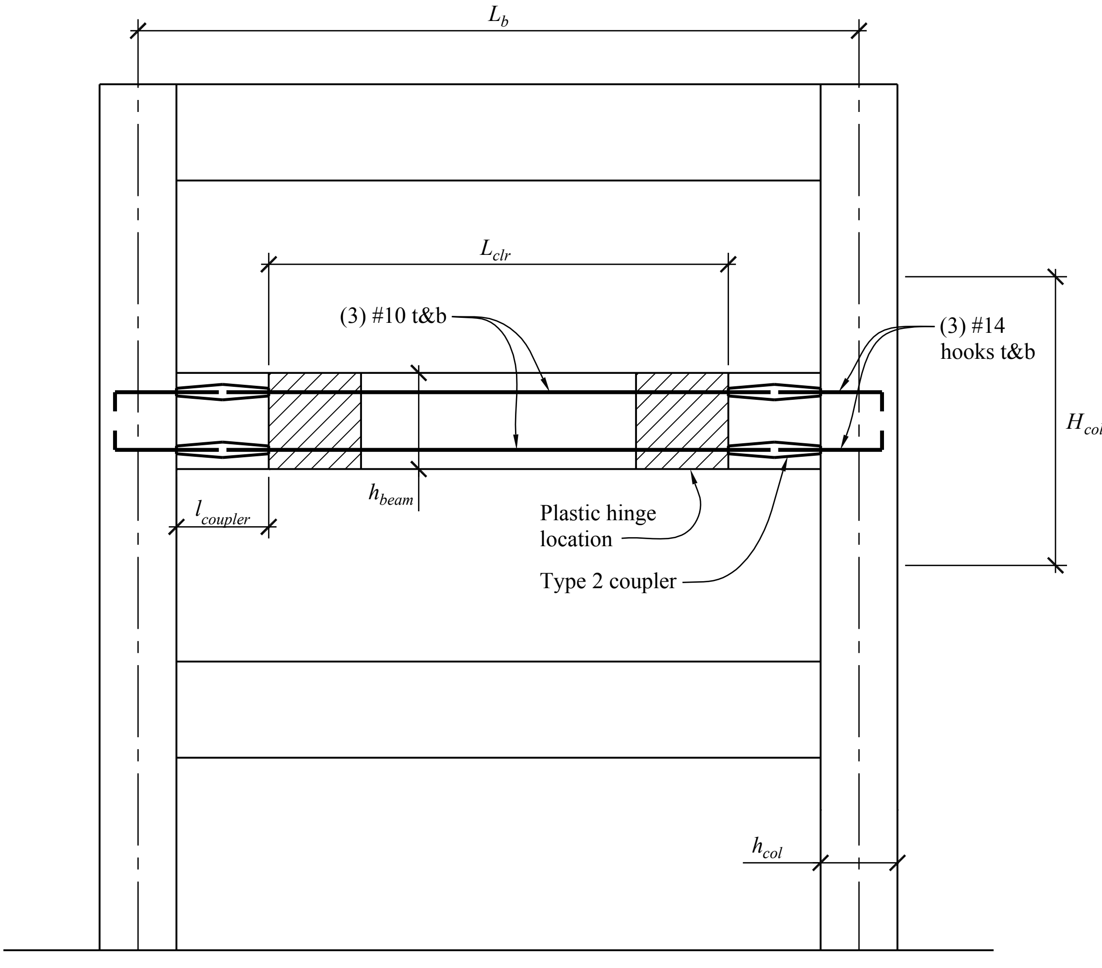

Figure 8.4-3 Moment frame geometry
(1.0 in = 25.4 mm, 1.0 ft = 0.3048 m)

Under seismic loads alone, the shear is constant along the beam length. Therefore, the moment at the joint between the end of the beam and the column is:

$$M_{joint} = V_b \frac{L_b - h_{col}}{2}$$

The plastic hinge, however, will be relocated to the side of the Type 2 coupler away from the column. With a coupler length of $l_{coupler}$, the moment at the end of the coupler is:

$$M_b = M_{joint} - V_b l_{coupler}$$

In order to ensure that the hinge forms at the intended location (away from the precast connection), the connection needs to be designed to be stronger than the moment associated with the development of the plastic hinge. This is done by upsizing the bar that is anchored into the column.

8.4.2.1 Strong Connection Example. In the following numerical example, a single-bay frame is designed to meet the requirements of a precast frame using strong connections at the beam-column interface. Using Figure 8.4-3 and the following geometry:

H_{col} = 12 ft

h_{col} = 36 in.

L_b = 30 ft (column centerline to column centerline)

$l_{coupler}$ = 18 in.

$L_{clr} = L_b - h_{col} - 2l_{coupler}$ = 24 ft (distance between plastic hinge locations)

h_{beam} = 42 in.

Reinforcing the beam with three #10 bars top and bottom, the nominal moment strength of the beam is:

$$a = \frac{A_s F_y}{0.85 f_c' b} = \frac{3(1.27)(60)}{0.85(5)(18)} = 3.0 \text{ in.}$$

$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2} \right) = 0.9(3)(1.27)(60)\left(33 - \frac{3.0}{2} \right) / 12 = 540 \text{ ft-kips}$$

This is the moment strength at the plastic hinge location. The strong precast connection must be designed for the loads that occur at the connection when the beam at the plastic hinge location develops its probable strength.

Therefore, the moment strength at the beam-column interface (which also is the precast joint location) must be at least:

$$M_{u,joint} = M_{pr} \frac{L_b - h_{col}}{L_{clr}}$$

Where:

$$M_{pr} = \phi M_n \frac{1.25}{\phi} = 540\frac{1.25}{0.9} = 750 \text{ ft-kips}$$

Therefore, the design strength of the connection must be at least:

$$M_{u,joint} = 750\frac{30-36/12}{24} = 843 \text{ ft-kips}$$

Using #14 bars in the column side of the Type 2 coupler:

$$a = \frac{A_s F_y}{0.85 f_c' b} = \frac{3(2.25)(60)}{0.85(5)(18)} = 5.3 \text{ in.}$$

$$\phi M_n = \phi A_s F_y\left(d - \frac{a}{2}\right) = 0.9(3)(2.25)(60)\left(33 - \frac{5.3}{2}\right)/12 = 921 \text{ ft-kips}$$

which is greater than the load at the connection (843 ft-kips) when the plastic hinge develops.

If column-to-column connections are required, ACI 318 Section 21.8.3(d) requires a 1.4 amplification factor, in addition to loads associated with the development of the plastic hinge in the beam. Locating the column splice near the point of inflection, while difficult for construction, can help to make these forces manageable.

The beam shear, when the plastic hinge location reaches its nominal strength, is:

$$V_{beam} = \frac{\phi M_n}{L_{clr}/2} = \frac{540}{24/2} = 20 \text{ kips}$$

Assuming inflection points at the mid-span of the beam and mid-height of the column, the column shear is:

$$V_{col} = V_{beam}\frac{L_b}{H_{col}} = 20\frac{30}{12} = 50 \text{ kips}$$

However, the column shear must be amplified to account for the development of the plastic hinge.

$$V_u = V_{col}\frac{M_{pr}}{\phi M_n} = 50\frac{750}{540} = 69 \text{ kips}$$

The column design moment is:

$$M_u = V_u\frac{(H_{col} - h_{beam})}{2} = 69\frac{12-42/12}{2} = 293 \text{ ft-kips}$$

At the connection, this moment is amplified by 1.4 for a strong connection design moment of 410 ft-kips. This moment must be combined with the axial load on the connection from both gravity loads and amplified seismic forces.

The balance of the design is the same as for a cast-in-place special moment frame.