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NEHRP Seismic Design Technical Brief No. 13

Seismic Design of Precast Concrete Diaphragms

A Guide for Practicing Engineers

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NEHRP Seismic Design Technical Briefs

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By
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September 2017
With references to ASCE/SEI 7-16 and ACI 318-14

U.S. Department of Commerce
Wilbur L. Ross, Jr., Secretary

National Institute of Standards and Technology
Kent Rochford, Acting NIST Director and Under Secretary of Commerce for Standards and Technology

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Cover photo—The top photograph shows a precast concrete parking deck structure that uses cast-in-place pour strips to create diaphragm chords and provide load transfer into the shear walls. The bottom photograph shows a precast concrete parking deck structure that uses a dry-system diaphragm.

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1. Introduction

The seismic force-resisting system (SFRS) of a building consists of a three-dimensional ensemble of elements that transmit loads and forces from the point of origin down to the soils underneath the foundation. This system typically consists of horizontal and vertical structural elements. When the building is subject to seismic excitation, the horizontal elements—roof and floor slabs, bending in their own plane, act as diaphragms to transmit the forces horizontally from the point of origin to the vertical elements of the SFRS (**Figure 1-1**). The vertical elements (i.e., walls or frames) transmit the forces down to the foundation. Together, these elements function as a system to provide a complete load path for the seismic forces to flow through the building to the foundation and the soils underneath. Diaphragms not only act to distribute the forces horizontally to the vertical elements of the SFRS, they also tie the vertical elements together to act as a system so that they respond to the seismic forces together, rather than individually. The diaphragms are an integral part of the SFRS of a building and deserve significant attention during the design process.

Seismic design of diaphragms is required for buildings in Seismic Design Categories (SDC) B through F, as defined in the 2018 edition of the *International Building Code* (IBC) (ICC 2018) and ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 2016). Since SDC A is exempt from seismic design, it is not specifically addressed, although many of the diaphragm analysis and design methods described herein are applicable to the design of diaphragms to resist wind forces and provide structural integrity in SDC A buildings as well.

Although horizontal structural elements acting as diaphragms can be truss elements or horizontal diagonal bracing, in most cases diaphragms utilize the floor system and are constructed as essentially solid, planar elements made of wood, steel, concrete, or combinations of these. Concrete diaphragms can be conventionally reinforced or prestressed, and can be cast-in-place concrete slabs (often post-tensioned), topping slabs on metal deck or precast concrete, or interconnected precast concrete slabs without topping, though the last system has not been used much in structures assigned to SDC D, E, or F.

The design forces and analysis requirements for diaphragms are contained in ASCE/SEI 7-16, which is the latest published version of this standard. However, a local jurisdiction may reference an earlier (2010 or 2005) edition in its code regulations. The forward-looking approach in this Guide will facilitate its use over the next several years, because ASCE/SEI 7-16 has been adopted into the 2018 edition of the IBC, which establishes general regulations for buildings. The 2018 IBC adoption of ASCE/SEI 7-16 has no modifications relevant to the design of precast concrete diaphragms.

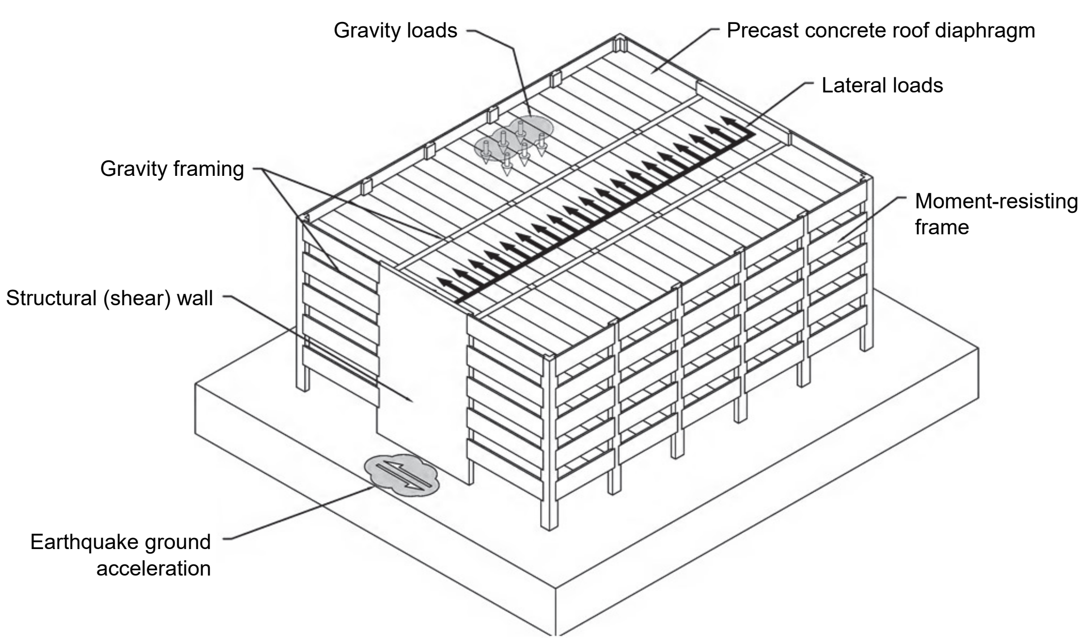

Figure 1-1. Building schematic showing SFRS elements including diaphragms.

Diaphragm Design in General

The National Earthquake Hazards Reduction Program (NEHRP) Seismic Design Technical Brief No. 3, *Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors, Second Edition* (NIST 2016), includes useful information on diaphragm design in general. To preserve space and avoid duplication, such material is not repeated here, unless needed for treatment of conditions beyond the scope of Technical Brief No. 3. This Guide covers seismic design issues pertaining to precast concrete diaphragms with or without cast-in-place topping slabs. Readers not intimately familiar with diaphragm design may benefit from reviewing Technical Brief No. 3, prior to reading this Guide.

This Guide is written primarily for practicing structural engineers and should also be useful to architects, building regulators (building officials and plan checkers), and contractors. Students, educators, and others interested in understanding the basis for the common design methods used for precast concrete diaphragms will find this document a useful beginning step in expanding that understanding.

Section 2 provides an overview of construction and application of precast concrete diaphragms, and Section 3 provides a summary of research on seismic behavior of precast concrete diaphragms. Section 4 presents examples of diaphragm design force level calculations and Section 5 discusses diaphragm analysis. Section 6 presents an overview of the diaphragm design procedure, Section 7 discusses design and detailing considerations for flexure and shear, and Section 8 summarizes qualification and classification of connectors. The Guide concludes with references, notations, abbreviations, and credits.

2. Precast Concrete Diaphragm Applications

2.1 Precast Concrete Construction

Precast concrete is defined as concrete cast in a location that is different from its final position within a structure. Precast concrete is produced by casting concrete in a reusable mold or form, which is then cured in a controlled environment, transported to the construction site, and lifted into place. It is important to note that all precast concrete is jointed, which in many ways dictates how it is designed and constructed.

A precast concrete diaphragm is a diaphragm consisting of precast components with optional cast-in-place pour strips along some or all boundaries and with or without cast-in-place topping slabs.

Precast/prestressed concrete components are used in many building types and for many functions. A review of common building types provides a context for the consideration of diaphragm analysis, design, and behavior.

2.2 Typical Applications in the United States

Parking garages are one of the most common building types constructed with precast/prestressed concrete. These structures can range from small, one-level decks to very large multi-story facilities. They are most commonly constructed with simple-span precast concrete double tees on gravity load framing consisting of simple-span beams and vertically continuous precast concrete columns. The typical bays range from 55 to 65 feet in span, with narrower bays sometimes used for ramps. To accommodate the vertical movement of vehicles, these structures include ramps, unless the building site has sufficient slope to permit separate access at each framed level. Ramp layouts may be single helix or double helix when the slope is sufficiently gradual to permit parking on the ramped bays (**Figure 2-1**). Steeper ramps can be used when they contain only traffic lanes. Parking structures often present formidable design challenges due to long floor spans, limited (often perimeter) seismic force-resistant system locations, and openings due to ramps.

Precast/prestressed concrete framing is also used for industrial facilities and food processing buildings. These buildings are usually constructed with single- or two-story framing, except for process towers. Roof framing typically consists of double tees with 2- or 3-inch thick flanges spanning 60 to 80 feet to provide more column-free interior space.

In recent years, the special requirements for hardened facilities with high resistance to extreme environmental loads have been met with precast/prestressed concrete framing. These buildings are typically single-story, except two-story framing may be used to support emergency generators. Roof framing uses precast concrete double tees with 2-inch thick flanges and 2- to 3-inch thick cast-in-place concrete topping slabs for additional diaphragm integrity.

When fire resistance and durability are important to the safe storage of materials, precast/prestressed concrete is used for warehouse framing. These are usually buildings with single-story framing, similar to industrial buildings.

Multi-family residential, high-rise condominiums and hotels are common uses that are well adapted to precast/prestressed concrete. These buildings are framed with hollow core slabs on loadbearing precast concrete walls or loadbearing reinforced concrete masonry walls.

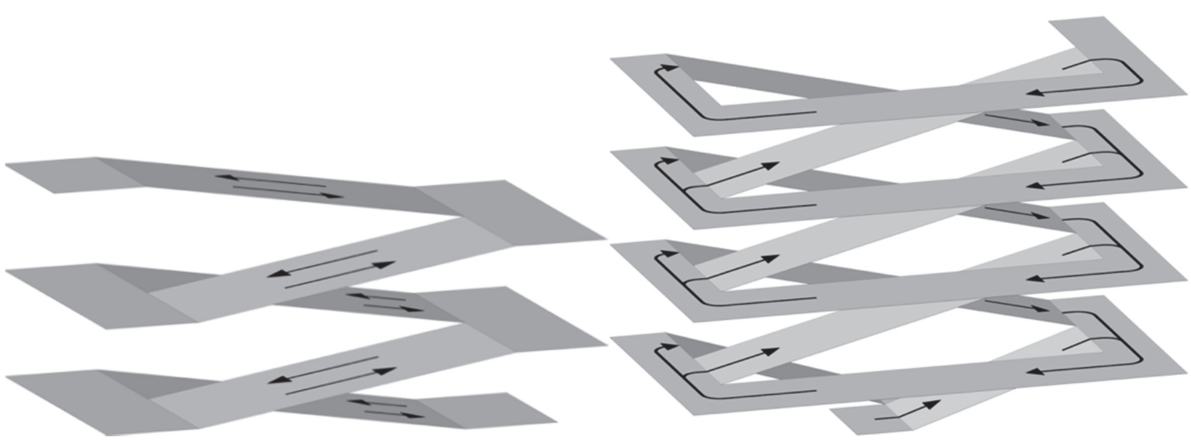

Figure 2-1. Single helix and double helix ramp configurations.

Hollow core slabs typically have no structural topping slabs in low and moderate seismic design categories, but topping slabs are used in high seismic design categories to develop diaphragm behavior when the in-plane lateral forces are larger. The seismic force-resisting system is most commonly loadbearing and non-loadbearing shear walls, with additional specific detailing requirements for structural integrity and resistance to disproportionate collapse. Structural integrity details include interior and perimeter ties in the floors that may contribute to diaphragm behavior.

Although not as commonly as for parking garages, there are several regions in the United States where precast/prestressed concrete is used for office buildings. Office building framing is similar to parking garage framing, but has no requirement for ramping; interior bay spans may be shorter, and cast-in-place concrete topping slabs over double tees are used to provide level floors, due to the natural camber in these prestressed floor members. The perimeter gravity support may be provided by spandrel beams on columns or loadbearing and non-loadbearing walls. Interior framing may include simple-span beam and column framing and walls at stair and elevator cores.

2.3 Precast Floor Units

Diaphragms in precast concrete structures are formed by floor units. Precast concrete floor units are primarily double tees, hollow core planks, and flat slabs. There are other types of components, that are less often used, such as quad tees (shallow 4-stemmed members), but the first three types are used for most floors and roofs. Double tees are most common, with the highest square footage placed. Double tees get their name from the cross-sectional shape shown in **Figure 2-2**.

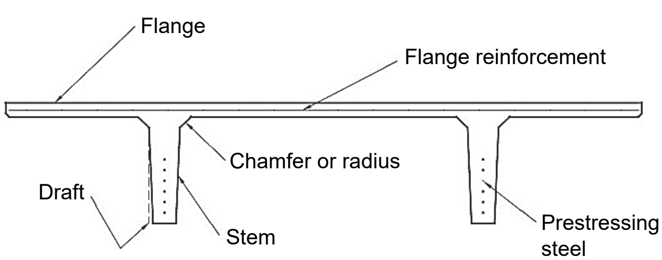

Figure 2-2. Double tee cross section with common terms for parts and features.

A hollow core plank unit (see **Figure 2-3** for cross section) is a precast/prestressed concrete component with continuous voids that reduce the concrete volume and weight compared to a solid slab. These members are manufactured with various methods which include dry-casting (extrusion), fixed forming, and slip-forming. Hollow core casting methods generally preclude the installation of embedded steel plates with anchorage and therefore make assembly with mechanical or welded connections impractical.

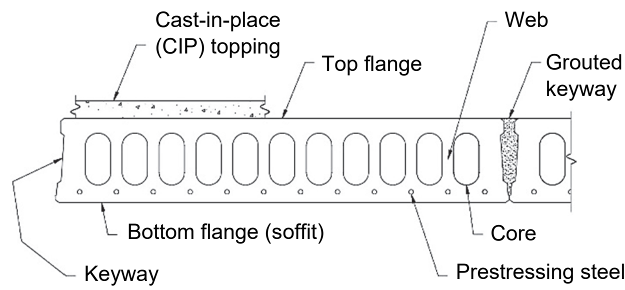

Figure 2-3. Hollow core plank cross section with common terms for parts and features.

Pretensioned precast components are usually cast in long-line beds, normally 300 to 600 feet in length, sharing the set-up of prestressing strand with several members and reducing the effect of strand seating and anchorage losses in each, because it experiences only a small part of the overall elongation of the strands.

2.4 Topped and Untopped Precast Diaphragms

Precast concrete diaphragms can be designed as topped or untopped (also known as pretopped). Topped diaphragms use cast-in-place concrete placed over precast units as the primary load-transfer medium to collect and transmit the in-plane loads. Diaphragms with cast-in-place topping slabs are further distinguished as composite (the topping slab and the precast units acting together as the diaphragm) or non-composite (the topping slab acting alone as the diaphragm), following the provisions of ACI 318-14, Building Code *Requirements for Structural Concrete* (ACI 2014).

A composite topping slab is permitted by ACI 318-14 provided that it "is reinforced and the surface of the previously hardened concrete on which the topping is placed is clean, free of laitance, and intentionally roughened." Composite diaphragms must be at least 2 inches in thickness, but the actual thickness may be greater so that cover requirements for the reinforcing steel are met. For parking garages that are located in regions where deicing salts are used and are exposed to the weather, the minimum required cover is 1½ inches thick. At the joints, this cover is required also below the bottom bar. For these conditions, the topping slab needs to be 3 inches thick plus the combined diameters of orthogonal reinforcement in the topping slab.

ACI 318-14 also permits a noncomposite cast-in-place concrete topping slab, acting alone, to be designed and detailed to resist in-plane earthquake forces. The minimum thickness of noncomposite diaphragms is 2½ inches, but the actual thickness in this case may also be controlled by cover requirements for the reinforcement. According to Fleischman (2014), discussed in detail in Section 3.2, non-composite diaphragms not reliant upon the effects of the underlying precast concrete are largely exempt from consideration as precast concrete diaphragms. However, the presence of underlying precast concrete joints may cause reflection cracking in the topping. It is common practice to tool or cut control joints in the topping slab over the precast concrete joints to prevent these cracks from being irregular. This can result in joint opening at the precast joints similar to that in composite topping slabs, which would then require topping slab thickness considerably greater than the minimum thickness or the thickness required by cover. It should be noted that noncomposite topping slab diaphragms must be designed by ACI 318-14 Section 18.12.5.1 and all other applicable provisions of Section 18.12, four of which are particularly important: (1) Section 18.12.6.1 for minimum thickness of $2^1/_2$ in.; (2) Section 18.12.7.1 for minimum wire reinforcement spacing of 10 in.; (3) Section 18.12.9.1 for shear strength of topping slab diaphragms; and (4) Section 18.12.9.3 for shear strength of topping slab diaphragms. Thus, non-composite topping slabs do have prescriptive requirements on them that differentiate their design from that of cast-in-place slabs.

Because Section 1921.6.11 of the 1997 *Uniform Building Code* (UBC) (ICBO 1997) and corresponding sections in earlier versions of the UBC recognized only a cast-in-place concrete topping slab acting alone as the diaphragm to resist the diaphragm design forces, the practice for states adopting the UBC was to design only non-composite concrete topping slab diaphragms. At least in California, this practice continues. Moreover, in California, double tee flange connections are used just for alignment of the flanges and are not used to transfer in-plane shear. This is not necessarily the practice in other states, such as Washington, also located in former UBC territory.

Untopped precast concrete diaphragms include diaphragms that rely in whole or in part on mechanical connectors between components to transfer in-plane diaphragm forces. Diaphragms may be classified as untopped if they include narrow strips of cast-in-place concrete topping at the ends of the double tees and across the supporting interior beams (inverted tee beams) as well as on the diaphragm ends along the lengths of the double tees to form a cast-in-place perimeter.

Composite Topping Slab Diaphragm

The use of composite action for resisting in-plane seismic forces has questionable meaning for double tee floor and roof diaphragms because at the joints between adjacent double tee flanges, there are only mechanical connections and cast-in-place topping. ACI 318-14 does not include consideration of the effects of these mechanical connections in buildings assigned to high seismic design categories (D, E, or F). It is also not clear if a topping that is considered composite for out-of-plane bending of the member must be considered composite for diaphragm behavior. In parts of the country where the *Uniform Building Code* was used to form the basis of legal codes, it is common practice to design the topping slab alone as the diaphragm.

In some applications, the cast-in-place concrete topping may be limited to only the ends of tees and across the interior inverted tee beams that support them, with no pour strips at ends, exterior spandrel beams, or interior walls. An untopped precast concrete diaphragm may also be a “dry” system which includes no cast-in-place concrete topping at all. The flange-to-flange connections between the double tees in the longitudinal joints are welded mechanical connections. The connections are generally considered only to carry shear across the joints. These connections usually have a closer spacing near midspan of the double tees because they also provide vertical continuity across the joints for flange alignment and for moving loads in parking structures (see **Figure 2-6** in Section 2.5). The various configurations of topped and untopped, composite and non-composite double tee and hollow core diaphragms are illustrated in **Figure 2-4**.

Hollow core diaphragms can be topped or untopped. Topping slabs for hollow core diaphragms are typically designed as non-composite, although the surface roughness produced by manufacturing of the plank by extrusion has been shown to produce a surface with sufficient roughness for composite action; embedment of shear friction reinforcement for horizontal shear transfer, however, is not practical. Untopped hollow

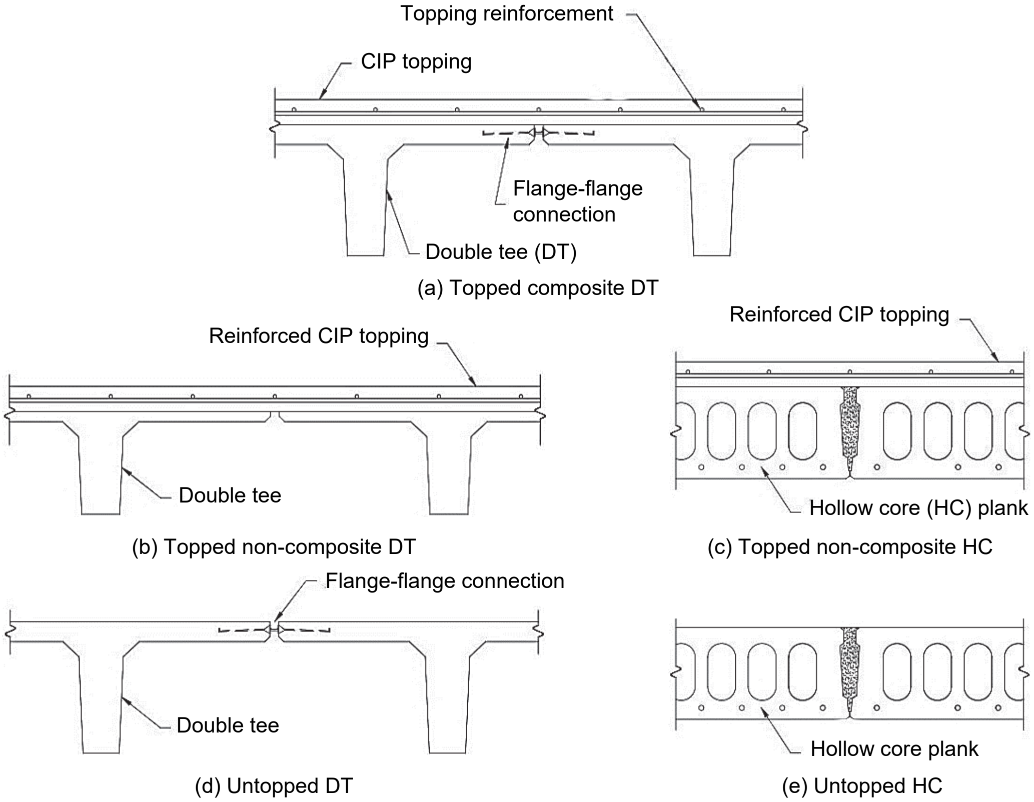

Figure 2-4. Double tee (DT) and hollow core (HC) configurations.

core diaphragms rely on perimeter reinforcement and reinforcement grouted into the joints or cores. In some cases, the flanges of hollow core planks are broken to provide for grouted reinforcement at critical locations. Because of the depth of grouted keyways in hollow core unit diaphragms, reflection cracking in the topping is less likely.

2.5 Chord Connections and Shear Connections

Precast concrete diaphragms are connected and reinforced in a manner that reflects treatment of the diaphragm as a deep horizontal beam with tension and compression chords near the edges to resist in-plane moments through a tension-compression couple and reinforcement in the field as shear reinforcement in the web of the beam (see **Figure 2-5**). From this beam analogy, the end or chord reinforcement has a greater, concentrated area of steel. The use of the cast-in-place pour strips at the edges may make it easier to place the larger steel area using conventional reinforcing details, rather than mechanical connections. In the lower seismic design categories, or in diaphragms with low aspect ratios where the chord reinforcing area is not excessive, qualified mechanical connection may be feasible for double tee diaphragms, allowing a "dry" system to be used. **Figures 2-6** and **2-7** illustrate layouts for different diaphragm types.

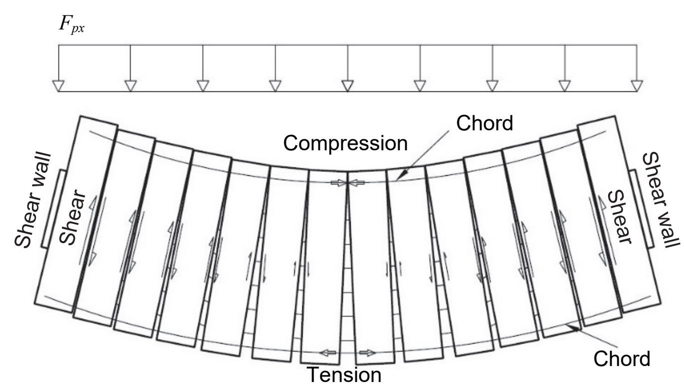

Figure 2-5. Beam analogy for precast diaphragm.

Seismic forces must be transferred from the diaphragm to the vertical elements of the seismic force-resisting system. "A diaphragm or shear wall boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical force-resisting elements" is called a collector or a drag strut. Under previous editions of ASCE/SEI 7, the design of collectors and their connections was required to be for forces amplified by the system overstrength factor. The requirement continues under ASCE/SEI 7-16 Section 12.10.2.1. However, ASCE/SEI 7-16, Section 12.10.3 forces used for diaphragm design are calculated considering system overstrength and higher mode effects. ASCE/SEI 7-16 Section 12.10.3.4 requires collector forces to be further amplified by a factor of 1.5 for all diaphragm design options (the options are discussed in Section 3.3.2). Shear forces are amplified by 1.4R_s, where R_s is the diaphragm force reduction factor given in ASCE/SEI 7-16 Table 12.10-1 for shear design purposes.

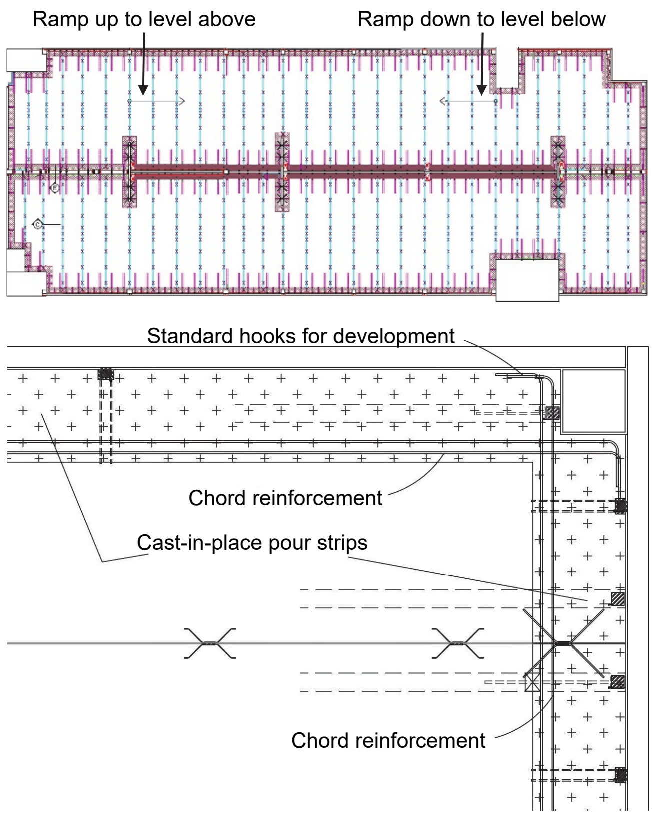

Figure 2-6. Untopped diaphragm with pour strips at ends of double tees and along perimeter.

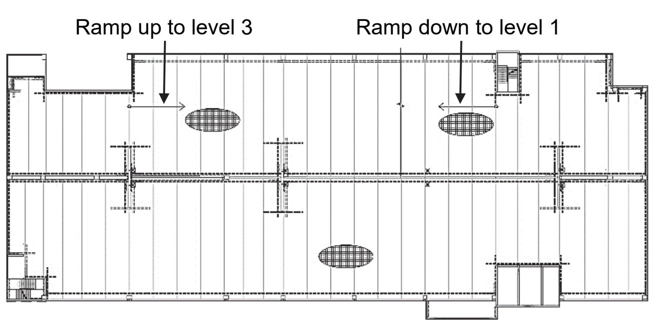

Figure 2-7. Topped diaphragm with chord reinforcement and topping reinforcement.

2.6 *Typical Precast Diaphragm Details*

The details for reinforcement in cast-in-place concrete topping slabs follow the requirements for cover, development length, lap length, bars bends, and hooks that apply to all reinforced concrete designed in accordance with ACI 318-14. The unique details are those for mechanical and other (such as grouted) connections across the joints between precast floor units that are used to tie the components together to form the diaphragm. These connections are chord connections on the perimeter and web or flange (shear) connections along the joints. There are several plant fabricated details that have been developed to provide continuous chords connected across joints in untopped systems. **Figure 2-8** shows a detail for a double tee chord connection that is intended to join embedded reinforcement to form a continuous tie.

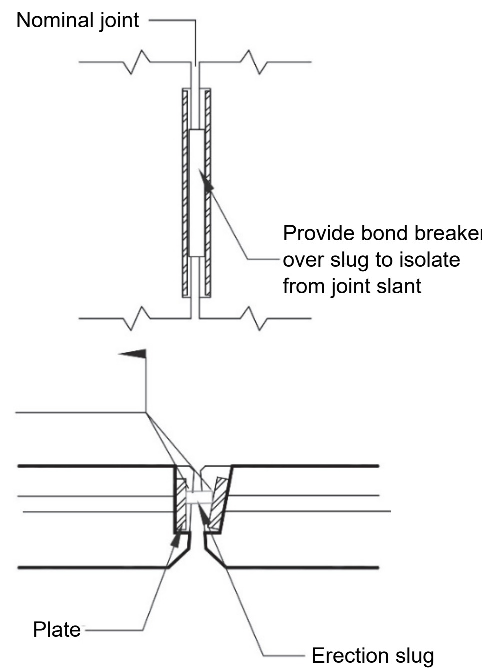

Figure 2-8. Welded chord connection detail, plan (top) and elevation (bottom).

The longitudinal reinforcement is butt-welded to plates located at the edge of the double tee flange. This detail uses a welded slug with intentional eccentricity to the longitudinal bars in an effort to allow for some flexibility to relieve volume change restraint. This feature reduces the resiliency of the connection under cyclic loads and is recommended only for use as a low-deformability detail (see Section 8.1). Special caution is required in the design of this connection to evaluate the eccentricities in the load path and the effects on the welds (Cao and Naito 2007).

To overcome the limitations of this detail, to remove the intentional eccentricity in the path of the forces, and to provide a "fuse" through a ductile link with yield strength less than that of the embedded reinforcement or the welds and the erection jumper plate, commercial assemblies have been developed and tested to provide high-deformability chord connections for untopped double tee diaphragms.

Several configurations have been used for flange-to-flange web connections used for shear transfer parallel to the joints between double tee units. These connections also serve to vertically align double tees with differential camber and force adjacent flanges to deflect together as they undergo vertical loads from vehicles passing from one double tee to another. The details for these connections have developed from simple plant-fabricated plates with reinforcing bar anchorage to custom-shaped commercial inserts with form-setting devices that accommodate the flexibility needed to avoid local concrete damage and reduce fatigue in the welds. These characteristics also provide strength with high deformability.

Flange-to-flange web and chord connections are commercially available and widely used in precast double tee concrete diaphragm construction. These connections provide a range of deformability and strength. The selection of the appropriate connection for the diaphragm should be based on appropriate experimental evaluation as discussed in Section 8.

In the case of hollow core diaphragms, the chord always consists of reinforcing bars placed in the joint where the hollow core units are supported on interior walls or an end wall. The same or similar bars are also utilized as shear friction reinforcement to resist shear parallel to the joint. Typical details that are used for these connections are illustrated in **Figure 2-9**.

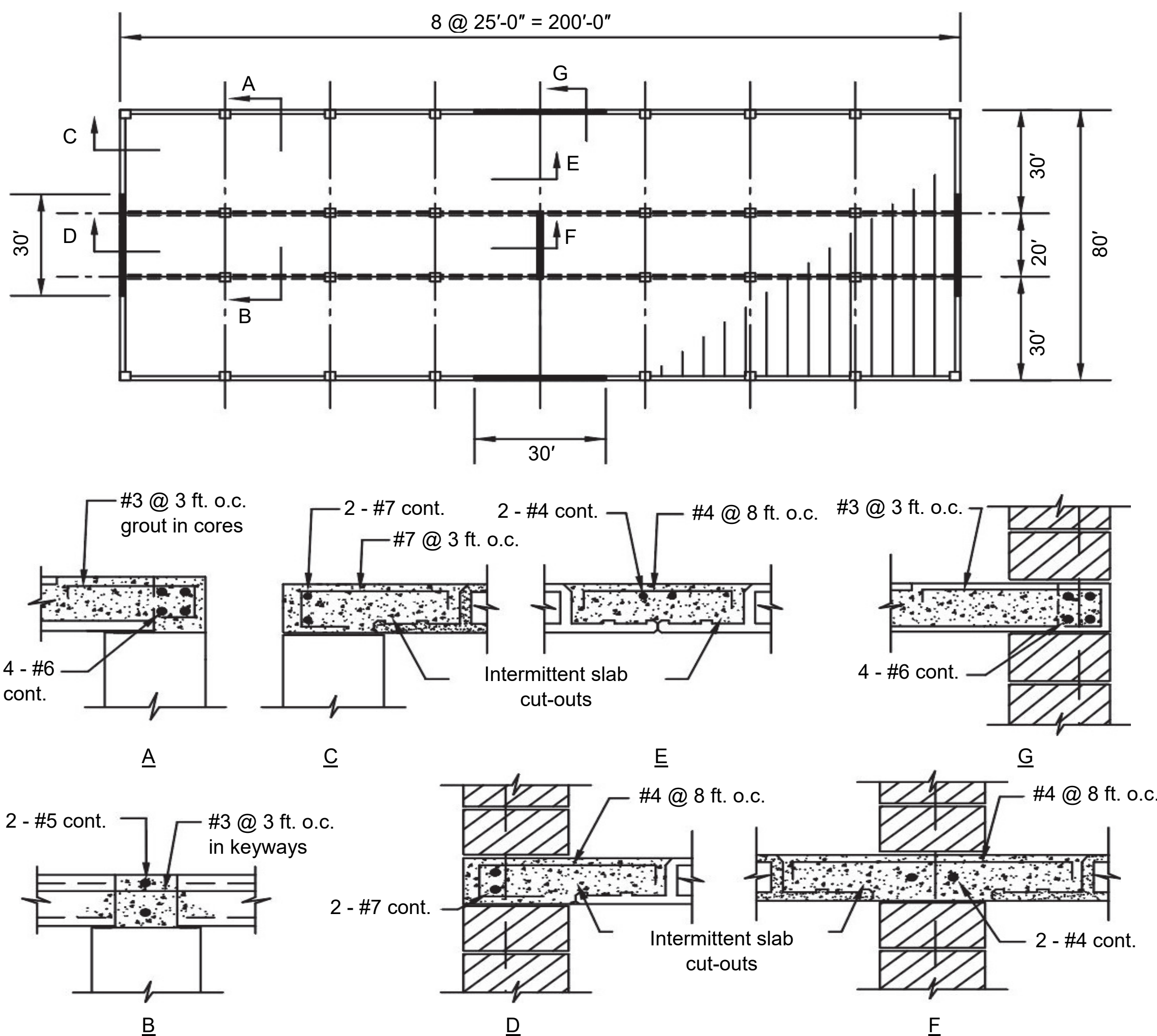

Figure 2-9. Typical chord details and shear friction reinforcement used for hollow core elements (PCI 2015).

2.7 Connections to SFRS

When the diaphragm has a cast-in-place topping slab, the connection to the seismic force-resisting system (SFRS) is often made by using reinforcing dowel bars that are attached with threaded bar inserts qualified as Type 2 mechanical splices in accordance with ACI 318-14. That reinforcement is proportioned to meet the tension and shear friction design requirements for the load transfer. When the topping slab is designed as a composite topping slab, additional mechanical connections between the precast units and the SFRS can also be designed to contribute to the load transfer.

Mechanical connections have also been developed to transfer forces from double tee diaphragm to the frames or walls that form the SFRS. **Figure 2-10** through **Figure 2-12** show details of this type of connection.

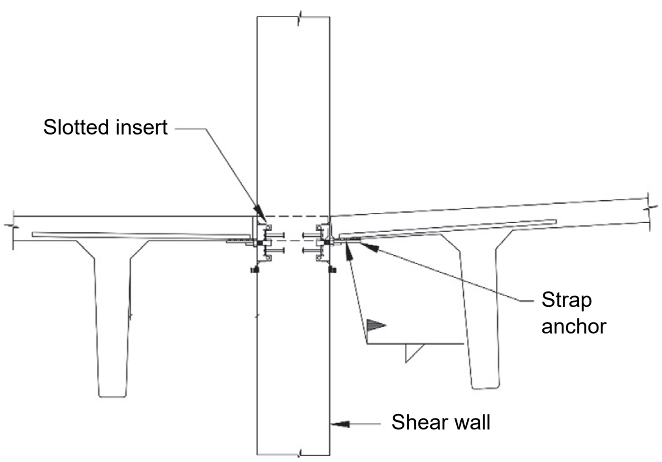

Figure 2-10. Flange-to-wall connection using strap anchors and slotted inserts.

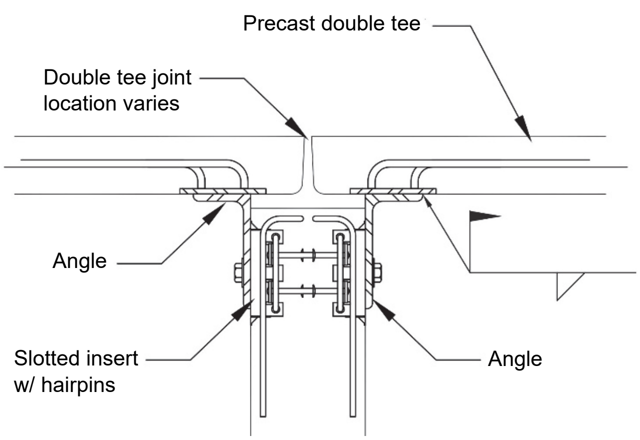

Figure 2-11. Sectional view of flange-to-wall connection under double tee flange using angle and slotted inserts.

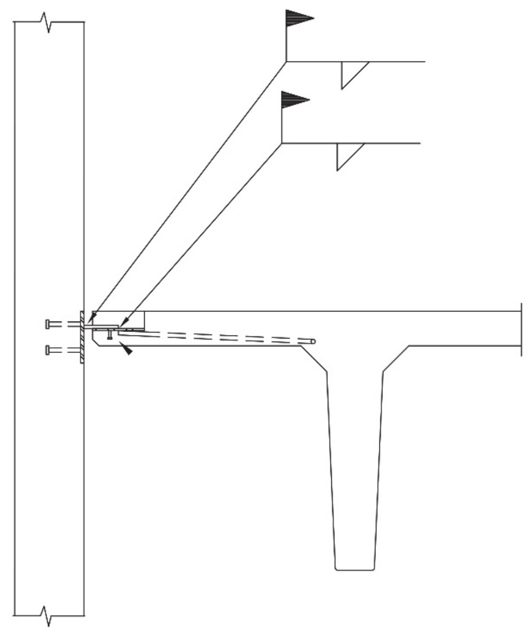

Figure 2-12. Flange-to-wall connection using welded plates and embedded plates with anchors.

Typical connections between hollow core diaphragms and seismic force-resisting systems are illustrated in **Figure 2-9** in Section 2.6.

When mechanical splices are used in the connection between the diaphragm and the vertical elements, ACI 318-14 Section 18.12.7.4 requires Type 2 splices for SDC D, E, or F.

2.8 Distinct Aspects of Precast Structures

Precast concrete structures are different from cast-in-place concrete structures because of the joints between the concrete components that are assembled to form the structure. The load paths in these structures are defined by the connections that are made across these joints. Some of these connections are simply bearing pads or grouted joints that carry gravity loads from spanning members or upper level walls and columns to their supports, but these connections are not intended or suitable to transmit lateral forces. Lateral forces are transferred through connections that are designed specifically for that purpose. Connections that are simple, reduce field labor, and develop strength with least delay to permit assembly (erection) of the precast concrete structure are favored. Better designs are distinctly directional, transferring forces only for the intended purpose without creating unintended restraint. In low or moderate seismic design categories, mechanical connections with sufficient strength and ductility may be used in double tee diaphragms, which do not require the addition of cast-in-place concrete or field-placed reinforcement. Embedded mechanical connections are preferred as they require less material, less time to place, and less field labor to complete. In low or moderate seismic design categories untopped hollow core diaphragms can be designed using chord and shear friction reinforcement in the joints at the ends of the units.

2.9 Practical Connection Considerations

Successful precast construction requires the recognition that all diaphragms are subject to volume changes due to creep, shrinkage, and temperature variations. The restraint of volume change deformation can result in forces that are far greater than the forces caused by the functional use or environmental loads due to wind or earthquakes. Deformations due to volume changes must be accommodated by the precast connections. This encourages the development of connections that are directional in their action and ductile beyond the requirements for cyclic loading from the response to earthquake ground motion.

It needs to be recognized that some connections need to provide for more than just the diaphragm action. An example is chord reinforcement above an inverted tee beam in a pour strip which extends beyond the space above the tee stem to clear the interference from the supporting columns, see **Figure 2-13**. The cast-in-place concrete over the double tee flange is commonly included as part of the composite section resisting bending in the beam, following the effective slab width guidelines for tee beams from ACI 318-14. Some of the compression force in the cast-in-place concrete is in that portion across the joint between double tee and inverted tee beam. That joint is subjected to reflection cracking, and it is recommended practice to tool a control joint into the topping slab above the joint between the inverted tee and the double tee or hollow core unit. The control joint permits proper sealing to prevent leaks. Since this control joint will crack, the transverse reinforcement across the joint not only carries the diaphragm forces but also must transmit the composite slab force using shear friction. Care should be taken to ensure that the transverse reinforcement has adequate strength and stiffness to perform the attributed design actions.

As mentioned above, the flange-to-flange connections between double tees also vertically align double tees with differential camber and force adjacent flanges to deflect together as they undergo vertical loads from vehicles passing from one double tee to another. Although ASCE/SEI 7-16 prescribes a concentrated load of 3,000 lbs for parking structure design, this is an allowance for an occasional tire jack and is not intended for the design of the connections. The forces from wheel loads from regular traffic in a garage are less, but also occur in pairs as vehicles cross the joints. The deflection in the spanning double tee and the local deflection of the flange under the loads from traffic cause forces in the connections that must be accommodated without deterioration, so that the shear connectors remain available to transfer in-plane forces due to earthquakes. It may be noted that hollow core is seldom used in parking structures in the United States.

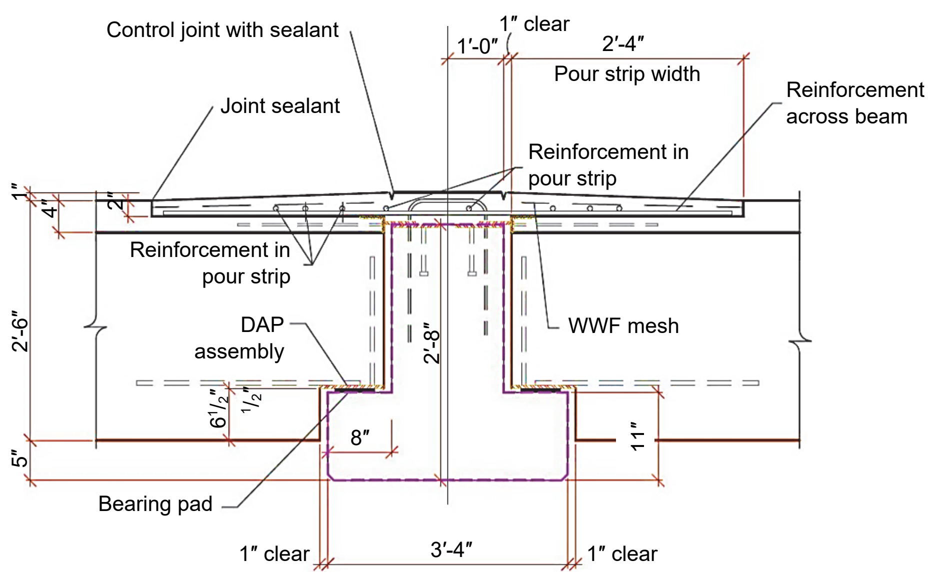

Figure 2-13. Section at inverted tee beam with composite concrete pour strip. DAP indicates dapped end.

2.10 Compatibility Between Horizontal and Vertical Elements

In the design of the connections to the seismic force-resisting system, it is important to recognize that the deformation or elongation in the walls and frames may not be compatible with connections within the undisturbed plane of the diaphragm. These in-plane and out-of-plane movements must be considered in devising proper connection details.

Reinforced concrete shear walls and unbonded post-tensioned concrete walls are systems that dissipate energy through yielding of mild steel reinforcement at the base of the wall. This action is accompanied with "rocking" of the walls or plastic hinge elongation of reinforced concrete shear walls. As a wall rocks, it lifts on the tension end while the compression end stays firmly in contact with the base. This lifting puts a vertical deformation demand on the diaphragm-to-wall connections at the floor levels. Connections must have the capacity to permit this movement, through the use of slotted inserts with vertical slots, or to deform with sufficient ductility to prevent fracture in the connections.

Moment-resisting frames dissipate energy through plastic hinging in the beams at the faces of the columns. As it yields, the reinforcement in a beam elongates and causes the length of the beam to increase. Connections from the diaphragm to the frames must have sufficient ductility to accommodate this deformation and sufficient strength in the floor to maintain its integrity under the forces imparted. It may be noted that the effects of plastic hinge elongation on diaphragms are proportional to the beam depth and the number of joints. Thus it is more of an issue in "tube" precast systems, consisting of closely spaced columns and deep beams, commonly used in New Zealand in the 80's and 90's than it is for typical U.S. systems.

3. Background and Research on Seismic Behavior of Precast Concrete Diaphragms

3.1 Performance in Past Earthquakes

Earthquake damage to precast concrete diaphragms was observed as early as the 1964 Alaskan earthquake (Wood 1967). A large proportion of the 26 structures in Anchorage with precast concrete roof or floor diaphragms exhibited damage including the collapse of the diaphragm. The reconnaissance effort revealed that diaphragm connections to the supporting system were often damaged and that the interconnections between precast elements were either inadequately embedded in the flange or did not exhibit adequate ductility. In the 1971 San Fernando earthquake, damage to precast concrete systems was attributed to a lack of continuity between roof-wall and adjacent panel interfaces (Murphy 1973). Although the damage did not occur on precast floor systems, the failures further emphasized the need to have proper connections between the diaphragm and the seismic force-resisting system (Gates 1981). Based on these observations, changes were integrated into the design codes. This included a minimum seismic design force for connection of precast structural elements integrated into the 1979 Uniform Building Code and a requirement that connections shall have sufficient ductility to preclude concrete fracture or brittle fracture at or near the welds (ICBO 1979).

The 1988 Armenia earthquake resulted in the loss of over 25,000 lives. Most of those losses were due to the failure of several 9-story precast frame structures in Leninakan (Wyllie 1992). The collapsed structures were built using a Soviet-era building system consisting of precast columns, beams, and hollow core diaphragms. The system came in a number of configurations and were designed in accordance with Soviet codes. Wyllie (1992) indicated that the design forces in the Soviet code were lower than those of the 1964 *Uniform Building Code* (ICBO 1964). In addition, detailing used in the systems that facilitated constructability resulted in low ductility under the seismic event. The hollow core diaphragm elements were grouted between elements but the only reinforcement was in the beams at the column lines. Although collapse of the systems was attributed to failure of welded column splices and shear failure of the beams and poor materials, the lack of diaphragm integrity is thought to have prevented redistribution of forces and contributed to the overall lack of performance. Evaluation by Hadjian (1993) also pointed out that the Soviet design approach assumed that the jointed untopped hollow core floor acted as a rigid diaphragm, which was in fact not the case. According to Hadjian, approaches used in design calculations "had nothing to do with the actual response of the structure."

In the decades following the 1971 San Fernando earthquake, many precast diaphragm systems were built in high seismic regions of the United States. Following the 1989 Loma Prieta earthquake, Iverson (1989) inspected garages in the region around the collapsed I-880 bridge. Construction consisted largely of double tee floor diaphragms with topping slabs supported by shear walls and frames. All structures remained in use following the event, with damage limited to minor cracking of the topping slab adjacent to a shear wall in one structure. The subsequent 1994 Northridge earthquake in California, however, illustrated a number of shortcomings in the prevailing design approach of precast concrete diaphragms for regions of high seismicity. Iverson and Hawkins (1994) examined 30 parking garages and observed nine structures with varying degrees of collapse. As noted in their review, collapse appeared to be a result of failure of the gravity system rather than the seismic force-resisting system (SFRS). This failure was associated with the "lack of adequate ties connecting the precast floor elements to one another and to their lateral load [seismic force] resisting systems." These local failures in turn allowed the precast floor elements to fall and impact the floors below, thus producing the observed damage.

Changes to the design standards were developed as a result of the observations from the Northridge earthquake. Evaluation by Wood et al. (2000) of topped diaphragms indicates that welded wire reinforcement crossing the double tee joints did not provide adequate ductility to support joint opening. As a result, changes were implemented in ACI 318-99, Building Code *Requirements for Structural Concrete* (ACI 1999) that required a minimum spacing of 10 inches between longitudinal wires of welded wire reinforcement over joints. Experimental testing of joints with this spacing of longitudinal wires was conducted by Naito et al. (2006), and resulted in tensile fracture of welded wire reinforcement (WWR) at less than 0.20 inch of elongation. Fracture at this level of elongation is expected in accordance with the work of Mirza and

MacGregor (1981), which correlated fracture strain with WWR wire size. This limited deformation capacity would result in a low deformability element categorization for WWR with a 10-inch spacing (see Section 8.1 for connector classification based on deformability). Further experimental work conducted by at Lehigh University showed that high deformability could be achieved when the WWR was fabricated from non-cold drawn steel.

An additional change in ACI 318-99 addressed the use of non-tensioned prestressing strand in chords. Research by Fleischman et al. (1998) indicated that the use of ASTM A416 Grade 270 reinforcement would require large joint openings to achieve the full tensile strength of the strands. Consequently, ACI 318-99 limits the allowable tensile strength to 60 ksi.

International earthquake events further emphasized the importance of precast diaphragm integrity to the ability of the system to redistribute seismic demands. The 1999 Kocaeli earthquake in Turkey resulted in failure of single-story industrial buildings. Evaluation by Saatcioglu et al. (2001) indicated that the collapse was associated with inadequate diaphragms and beam-to-column connections, which resulted in large lateral displacements of the frames. The 2010 El Mayor Cucapah, Baja California earthquake exhibited collapse of a precast parking structure under construction. The core, which was completed, had no damage but the outer areas which were not completed failed due to a lack of completed connections between the precast tees (Meneses 2010). Similar to the damage observed in Turkey, evaluation following the 2012 Emilia (Italy) seismic events indicated that the highly flexible roof systems used were unable to transfer and redistribute seismic forces and prevent excessive frame displacements leading to precast element drops (Belleri et al. 2015).

Evaluation of the 2010 and 2011 earthquakes in Christchurch, New Zealand by Corney et al. (2014) indicated issues related to the performance of precast floor diaphragms. Damage was noted in diaphragms consisting of hollow core panels with a cast-in-place topping. Observed failure included brittle failure of welded wire reinforcement in the topping slab over the precast member joints, failure of the support of the hollow core elements due to torsional rotation of the supporting beams and loss of the load path, and loss of seating due to beam growth under plastic hinge formation. Many of these issues were identified by past research in New Zealand and integrated into their design code. Differences in construction approaches in the United States and New Zealand largely preclude some of these failure modes in U.S. construction.

3.2 *Research on the Performance and Proper Seismic Design of Diaphragms*

Precast concrete diaphragm research has generally focused on the double tee because of its broad use in precast concrete structures.

The Northridge earthquake served as the impetus to a significant research effort on the performance of precast concrete diaphragm systems under seismic demands. Initial research by Fleischman et al. (1998) indicated that significant deformations can occur in the joints of the diaphragm under seismic excitation. This imparts drift demands to the gravity system that can be several times greater than the drift demands in the lateral system. This issue was shown to be a potential cause of the collapses observed in the 1994 Northridge earthquake. Recommendations were made to limit the aspect ratio of the diaphragm or to develop a new design approach that accounts for the jointed nature of precast concrete diaphragms. Wood et al. (2000) pointed out that the prevailing design approach assumed diagonal diaphragm cracking and that it could not occur due the jointed nature of precast construction. Instead, a shear friction approach was recommended. This resulted in new chord provisions in the 1997 UBC and the introduction of web reinforcement requirements in ACI 318-99. Further work progressed both on the response of precast systems (Fleischman and Farrow 2001) and on the design of these systems for high seismic applications (Cleland and Ghosh 2012). Additional research (Farrow and Fleischman 2003; Fleischman and Farrow 2003) illustrated the impact of joint connection strength and stiffness at the key joints between precast members on the performance of the diaphragm. Their research further emphasized that while the performance of diaphragms in low and moderate seismic regions is likely adequate, a new design methodology for long-span precast diaphragms in high seismic regions is needed to address the current design approaches and actual performance.

In 2003, a research effort called Development of Seismic Design Methodology (DSDM) was initiated by the Precast/Prestressed Concrete Institute (PCI) and conducted by Fleischman et. al (2005b) to develop a seismic design method for precast concrete diaphragms that would be suitable for all seismic design categories.

The work was funded by PCI, the National Science Foundation, and the Charles Pankow Foundation. An industry task group helped guide the practicality of the research and helped facilitate eventual codification of the research results. The multi-university research effort was led by researchers at the University of Arizona to provide numerical analysis and eventual parametric studies that would result in a detailed design approach. Large scale physical testing of individual diaphragm connections was performed at Lehigh University. The connection performance properties observed at Lehigh and initial system models developed at Arizona were used to develop a half-scale three-story experimental shake table test structure at the University of California San Diego (UCSD).

The initial program is summarized in Fleischman et al. (2005a, 2005b, and 2013). Responses of conventional diaphragm joint connections in tension (Cao and Naito 2009) and in shear (Naito et al. 2009) were determined. Additional details on chord connections were developed (Cao and Naito 2007) and commercially available connections were tested to characterize their strength, stiffness, and deformation capacity. Details on connection tests can be found in the sidebar. These connection tests and others available in the literature were summarized by Ren and Naito (2013). Utilizing the connection constitutive properties observed, approximate numerical models were developed by Wan et. al (2015) and Zhang et al. (2016) and implemented for parametric evaluation of diaphragms. Analytical research on the effect of spandrel beam to double tee connection characteristics on flexure-controlled diaphragms has also been presented (Wan et al. 2012). An additional aspect of the extensive connection testing was the development of a connection testing approach (Naito and Ren 2013) which has been integrated into ASCE/SEI 7-16 Section 14.2.4.

Experimental results related to the diaphragm aspect of the DSDM project are summarized in Schoettler et al. (2008) and Zhang et al. (2011). The design approach was developed through numerical models which were first calibrated against the UCSD and Lehigh experiments and used to conduct extensive parametric studies. The design approach development can be found in Fleischman and Wan (2007) and is detailed in Zhang and Fleischman (2016). The design approach developed as part of the DSDM study provided the basis for the approach integrated into ASCE/SEI 7-16.

Flange-to-Flange Connection Tests

Individual tests of flange-to-flange connections have been conducted by researchers over the past 50 years. Data have been generated on both proprietary and non-proprietary connections subject to shear, axial, and combined shear and axial demands in the plane of the diaphragm. Experiments were conducted using a variety of subassemblies that do not necessarily conform to the qualification requirements of Section 8 of this Guide. Experimental data have been generated on bent reinforcing bar hairpin connections (Venuti 1970; Kallros 1987; Naito et al. 2009), welded plate connections (Pincheira et al. 1998; Naito et al. 2006; Cao and Naito 2007), angle plate connections (Spencer and Neille 1976), and a number of proprietary connections. Many of these connection test data sets are summarized in (Oliva 2000; Naito and Ren 2008; Cao and Naito 2009; Ren and Naito 2013).

3.3 Codification of Research Findings

ACI 318-14 Section 18.12, applicable to buildings assigned to SDC D, E, or F, contains design requirements for two types of diaphragms. The cast-in-place composite topping slab diaphragm transmits seismic forces to vertical elements of the seismic force-resisting system through composite action of precast double tees or hollow-core units and a cast-in-place topping slab. In a cast-in-place non-composite topping slab diaphragm, the topping slab acting alone as the diaphragm transmits seismic forces to vertical elements of the seismic force-resisting system. Both types of diaphragms require a cast-in-place topping slab. An untopped precast concrete diaphragm does not comply with the requirements of ACI 318-14 Section 18.12. However, the section does not specifically prohibit the use of untopped precast concrete diaphragms in buildings assigned to SDC D, E, or F.

Cleland and Ghosh (2002) suggested that it is possible to design untopped precast concrete diaphragms for buildings assigned to high Seismic Design Categories (D and above) under the equivalency provision that is now in ACI 318-14 Section 18.2.1.7. The paper outlined how such equivalency could be achieved. However, designs based on the equivalency provision require project by project approval from authorities having jurisdiction.

Much broader applications of precast concrete diaphragms in high-seismic applications, not requiring project by project approval, is now possible for the first time because of codification of the results of research discussed above.

3.3.1 Alternative Diaphragm Design Force Level of ASCE/SEI 7-16

ASCE/SEI 7-16 includes a new Section 12.10.3, Alternative Design Provisions for Diaphragms including Chords and Collectors. The section provides for an alternative determination of diaphragm design force level, which is mandatory for precast concrete diaphragms in buildings assigned to SDC C, D, E, or F. The alternative is permitted to be used for other precast concrete diaphragms, cast-in-place concrete diaphragms, and wood diaphragms supported on wood light-framed construction. Section 12.10.3 in ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010) does not apply to steel deck diaphragms. All diaphragms other than precast concrete diaphragms in SDC C, D, E, or F buildings may continue to be designed using the design force level in ASCE/SEI 7-16 Section 12.10.1, which is carried over unchanged from ASCE/SEI 7-10.

ASCE/SEI 7-16 Section 12.10.3 presents an elastic diaphragm design force as the statistical sum of first mode effect and higher mode effects. The first mode effect is reduced by the *R*-factor of the seismic force-resisting system, but then amplified by the overstrength factor, Ω_0, because vertical element overstrength will generate higher first mode forces in the diaphragm. The effect caused by higher mode response is not reduced. In recognition of the deformation capacity and overstrength of the diaphragm, the elastic diaphragm design force from the first and higher modes of response is then reduced by a diaphragm force reduction factor, R_s. Detailed explanation of ASCE/SEI 7-10 Section 12.10.3 appears in the Commentary to ASCE/SEI 7-16.

3.3.2 Precast Concrete Diaphragm Design of ASCE/SEI 7-16

To go hand-in-hand with the alternative diaphragm design force level in Section 12.10.3, ASCE/SEI 7-16 includes a precast diaphragm design procedure in Section 14.2.4, which is based on the DSDM research program mentioned previously and includes a connector qualification protocol. The requirements of Section 14.2.4 are in addition to the requirements set forth in ACI 318-14 Section 18.12, Diaphragms and Trusses.

Table 3-1 shows the applicability of ASCE/SEI 7-16 Section 14.2.4 to various types of precast concrete diaphragms. Note that ASCE/SEI 7-16 Section 14.2.4 requires use of the alternative design forces of Section 12.10.3 in diaphragm design. Connector classification based on deformability is discussed in Section 8.1 below.

The design procedure in ASCE/SEI 7-16 Section 14.2.4 presents the designer with three diaphragm design options: Elastic, Basic, and Reduced. The options concern the target performance of a diaphragm when subject to earthquake excitation. The Elastic Design Option (EDO) seeks to keep the diaphragm elastic in the Maximum Considered Earthquake (MCE). The Basic Design Option (BDO) seeks to keep the diaphragm elastic in the design earthquake, while permitting controlled inelastic behavior in the MCE. The Reduced Design Option (RDO) permits controlled inelastic behavior even in the design earthquake.

The choice of options is not unrestricted, but depends on the Diaphragm Seismic Design Level (DSDL), which is a function of the seismic design category, the number of stories, the diaphragm span, and the diaphragm aspect ratio. The EDO is permitted for: (1) Low Seismic Demand Level; and (2) Moderate Seismic Demand Level, provided the diaphragm design force is increased by 15 percent. The BDO is permitted for: (1) Low Seismic Demand Level; (2) Moderate Seismic Demand Level; and (3) High Seismic Demand Level, provided the diaphragm design force is increased by 15 percent. The RDO is permitted to be used for all Seismic Demand Levels. For locations in California and Nevada, the RDO will typically be the choice.

The EDO permits any type of diaphragm connector to be used, including those classified as Low Deformability Elements (LDE). If the BDO is selected, connectors qualifying as Moderate Deformability Elements (MDE) need to be used as a minimum. Connectors qualifying as High Deformability Elements (HDE) need to be used exclusively if the RDO is chosen.

A precast concrete diaphragm connector is assigned a deformability classification based on its measured deformation capacity in tension. The measurement requires testing, which is more generally required to establish the performance characteristics of strength, stiffness, and deformation capacity of the precast concrete diaphragm connectors under in-plane shear and in-plane tension. The testing must follow a protocol that is part of ASCE/SEI 7-16 Section 14.2.4.

For precast concrete diaphragms deigned using ASCE/SEI 7-16 Section 14.2.4 and ACI 318-14, R_s is 0.7, 1.0, or 1.4 for EDO, BDO, and RDO, respectively. The required shear strength for a diaphragm must be amplified by the diaphragm shear overstrength factor, Ω_v, which is taken equal to $1.4R_s$.

Table 3-1. Applicability of ASCE/SEI 7-16 Section 14.2.4 to Various Types of Precast Concrete Diaphragms

Type of Precast Diaphragm	Type of Connector / Joint Reinforcement	Applicability of ASCE/SEI 7-16 Section 14.2.4
Cast-in-place noncomposite topping slab diaphragm	Designed by ACI Section 18.12.5.1 and all other applicable provisions of ACI 318-14 Section 18.12	The applicability of ASCE/SEI 7-16 Sections 12.10.3 and 14.2.4 depends on a number of factors. A definite determination in this regard cannot be made because under certain practical combinations of factors, it may be proper to design these systems as cast-in-place diaphragms. The design professional has to evaluate the particular case to make a determination.
Hollow core with cast-in-place composite topping slab diaphragm	Only connectors are reinforcement in topping slab and reinforcement in joints or reinforcement in cores that are broken open so the bars can be grouted in.	The topping slab reinforcement typically qualifies as high-deformability elements per ASCE/SEI 7-16 Section 14.2.4.3.5. For other reinforcement, see note below.
Double tee with cast-in-place composite topping slab diaphragm	Reinforcement in topping slab. Also, mechanical connections using plate anchorage and welded plates.	The reinforcement typically qualifies as high-deformability elements per ASCE/SEI 7-16 Section 14.2.4.3.5. The connectors need to be qualified per ASCE/SEI 7-16 Section 14.2.4.4.
Hollow core without topping slab diaphragm	Reinforcement in joints or reinforcement in cores that are broken open so the bars can be grouted in.	See note below.
Double tee without topping slab diaphragm	Mechanical connections using plate anchorage and welded plates.	The connectors need to be qualified per ASCE/SEI 7-16 Section 14.2.4.4.

Note: With hollow core slabs, mechanical connections using plate anchorage and welded plates are not practical because the components are most commonly made by an extrusion process. Diaphragm connections in untopped floors are limited to reinforcement in joints, or in cores that are broken open so the bars can be grouted in. When planks intersect with orthogonal orientation, it may be necessary to break the top of a plank so that reinforcing with hooks can be placed and grouted in to match joints in the intersecting planks.

Data may not be available to support high-deformability classification for the reinforcing bars in joints and cores. The biggest concern is about the bars in the joints. When the diaphragm chord cracks, it is probable that the joints between slabs will open and bond will be lost. The *PCI Hollow Core Manual* (Buettner and Becker 1998) thus suggests using bars in keyways only for SDC A and B and that bars should be anchored in cores for SDC C and higher. Also, the bars described only transfer shear perpendicular to the longitudinal joints into the boundary elements. For shear parallel to the longitudinal joints, shear friction reinforcement in an edge element is used. This is based on Moustafa (1981). This reinforcement also works as chord reinforcement and should be considered high deformability steel in that capacity.

4. Diaphragm Design Force Level

ASCE/SEI 7-16 provides two different diaphragm design force levels in Section 12.10.1 and 12.10.3. The force level in Section 12.10.1 has been in use since before the first edition of the *International Building Code* and has been applicable to diaphragms of all materials designed by ASCE/SEI 7-10 and earlier editions of ASCE/SEI 7. ASCE/SEI 7-16 Section 12.10.1 can be used for design of precast concrete diaphragms in buildings assigned to SDC B, but cannot be used for design of precast concrete diaphragms in buildings assigned to SDC C, D, E, or F. The latter must, under ASCE/SEI 7-16, be designed using the force level in Section 12.10.3. This section presents computation of design forces by ASCE/SEI 7-16 Sections 12.10.1 and 12.10.3 in a step by step manner. Comparisons of the resulting diaphragm seismic design force levels along the heights of a number of precast concrete buildings assigned to various SDCs are also presented in this section.

4.1 ASCE/SEI 7-16 Diaphragm Design Force Level

The following describes in a step-by-step fashion the determination of diaphragm seismic design force by following ASCE/SEI 7-16 Section 12.10.1.

Step 1: ASCE/SEI 7-16 Section 12.7.2 defines effective seismic weight, W, and w_x is the portion of W that is tributary to level x (**Figure 4-1**). The assignment of w_{px} is different from w_x only in that the weights of the walls parallel to the earthquake forces may be excluded from w_{px} and thus are likely smaller in magnitude.

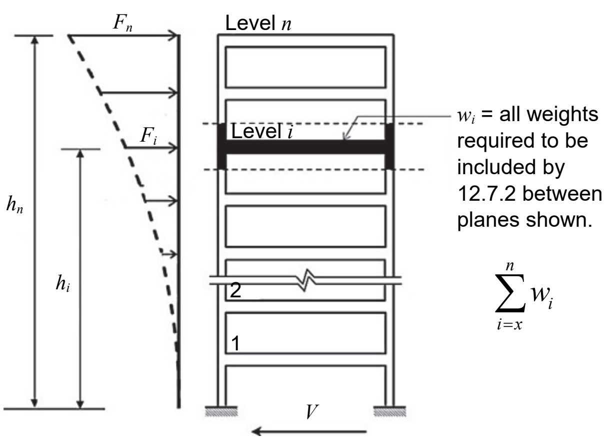

Figure 4-1. Seismic weights and lateral forces obtained from vertical distribution of design base shear at various floor levels (FEMA 2015).

Step 2: Determine w_i for all levels from x to n with n being the roof level. The total seismic weight of level x and above is equal to:

$$\sum_{i=x}^{n} w_i$$

Step 3: Determine the seismic design base shear, V, from ASCE/SEI 7-16 Section 12.8.1.

Step 4: Determine the story force, F_i (portion of V induced at level i) for all levels from x to n using ASCE/SEI 7-16 Section 12.8.3. The shear acting at level x is equal to:

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

Step 5: Determine the diaphragm design force at level x, F_{px}, using ASCE/SEI 7-16 Section 12.10.1.1 and the provided equation:

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

Note that the F_{px} forces are higher than the F_x forces except at the roof where they are equal. This is because the diaphragm forces, F_{px}, are influenced to a larger extent by higher mode effects than the design forces, F_x, for the seismic force-resisting system. See **Figure 4-2** showing both the F_x and F_{px} forces for an eight-story moment-resisting frame office building with precast concrete diaphragms for all floors and the roof. Note that the minimum and maximum design values for F_{px} forces are also shown.

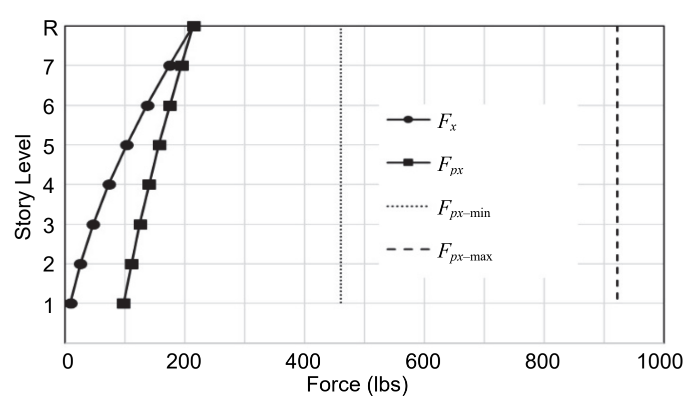

Figure 4-2. Distribution of F_x and F_{px} forces along the height of an example eight-story office building.

Step 6: Check F_{px} against maximum and minimum values using the following:

F_{px} shall not be less than F_x or $0.2S_{DS}I_e w_{px}$ and

F_{px} need not be greater than $0.4S_{DS}I_e w_{px}$

where S_{DS} is the 5 percent damped spectral response acceleration parameter at short periods, and I_e is the importance factor.

Note that the F_{px} forces are controlled by the minimum allowed force of $0.2S_{DS}I_e w_{px}$.

4.2 Alternative Diaphragm Design Force

The following describes in a step-by-step fashion the determination of diaphragm seismic design force by following ASCE/SEI 7-16 Section 12.10.3.

4.2.1 Determine the Effective Diaphragm Seismic Weight, w_{px}

ASCE/SEI 7-16 Section 12.7.2 defines effective seismic weight, W. The portion of W that is tributary to level x is w_x. The magnitude of w_{px} is different from w_x only in that the weights of the walls parallel to the earthquake forces may be excluded from w_{px}. The seismic weight of a diaphragm level is the same regardless of whether ASCE/SEI 7-16 Section 12.10.1 or 12.10.3 is used for the diaphragm design.

4.2.2 Determine the Diaphragm Design Force Reduction Factor, R_s

The diaphragm design force reduction factor, R_s, found in **Table 4-1**, accounts for the diaphragm overstrength and the inelastic deformation capacity of a diaphragm. For diaphragm systems with inelastic deformation capacity sufficient to permit controlled inelastic response under the design earthquake, R_s is typically greater than 1.0, so that F_{px} is reduced relative to the design force demand for a diaphragm that remains linear elastic under the design earthquake. For diaphragm systems that do not have sufficient inelastic deformation capacity, R_s could be 1.0, or even as low as 0.7 (and, effectively, no longer a reduction factor), so that linear elastic force-deformation response can still be expected under the MCE.

A flexure-controlled diaphragm is a diaphragm with a flexural yielding mechanism, which limits the maximum forces that develop in the diaphragm, and having a design shear strength greater than the shear corresponding to the nominal flexural strength. A shear-controlled diaphragm is a diaphragm that does not meet the requirements of a flexure-controlled diaphragm.

Table 4-1 Diaphragm Design Force Reduction Factor, R_s
(values taken from ASCE/SEI 7-16 Table 12.10-1)

Precast Concrete Diaphragm Designed in Accordance with ASCE/SEI 7-16 Section 14.2.4 and ACI 318-14	Diaphragm Design Force Reduction Factor, R_s	
	Shear-Controlled[1]	Flexure-Controlled[1]
EDO[2]	0.7	0.7
BDO[3]	1.0	1.0
RDO[4]	1.4	1.4

[1] Diaphragm behavior defined in ASCE/SEI 7-16 Section 11.2
[2] Elastic Design Option defined in ASCE/SEI 7-16 Section 11.2.
[3] Basic Design Option defined in ASCE/SEI 7-16 Section 11.2.
[4] Reduced Design Option defined in ASCE/SEI 7-16 Section 11.2.

4.2.3 Determine the Diaphragm Design Acceleration Coefficient, C_{px}

In order to determine the diaphragm design acceleration coefficient C_{px} at level x, the values of C_{p0} (at base), C_{pi} (at an intermediate level at 80% of the structural height above the base), and C_{pn} (at the highest level of the seismic force-resisting system, SFRS) need to be determined first.

Step 1: Determine C_{p0}, diaphragm design acceleration (force) coefficient at the base of the structure using ASCE/SEI 7-16 Section 12.10.3.2.1.

$$C_{p0} = 0.4S_{DS}I_e$$

Step 2: Determine C_{pi}, diaphragm design acceleration (force) coefficient at a height equal to 80 percent of h_n above the base, using ASCE/SEI 7-16 Section 12.10.3.2.1.

C_{pi} is the greater of the two values given by:

$$C_{pi} = 0.8C_{p0}$$

$$C_{pi} = 0.9\Gamma_{m1}\Omega_0 C_s$$

where:

Γ_{m1} is the first mode contribution factor:

$$\Gamma_{m1} = 1 + 0.5z_s\left(1 - \frac{1}{N}\right)$$

z_s is the modal contribution coefficient modifier dependent on seismic force-resisting system (see **Table 4-2**).

Table 4-2. Modal Contribution Coefficient Modifier, z_s

Description	z_s
Buildings designed with buckling restrained braced frame systems defined in ASCE/SEI 7-16 Table 12.2-1	0.30
Buildings designed with moment-resisting frame systems defined in ASCE/SEI 7-16 Table 12.2-1	0.70
Buildings designed with dual systems defined in ASCE/SEI 7-16 Table 12.2-1 with special or intermediate moment resisting frames capable of resisting at least 25% of the prescribed seismic forces	0.85
Buildings designed with all other seismic force-resisting systems	1.00

Step 3: Determine C_{pn}, diaphragm design acceleration (force) coefficient at h_n, using ASCE/SEI 7-16 Section 12.10.3.2.1.

$$C_{pn} = \sqrt{\left(\Gamma_{m1}\Omega_0 C_s\right)^2 + \left(\Gamma_{m2}C_{s2}\right)^2} \geq C_{pi}$$

where:

Γ_{m2} is the higher mode contribution factor:

$$\Gamma_{m2} = 0.9 z_s\left(1-\frac{1}{N}\right)^2$$

C_{s2} is the higher mode seismic response coefficient, and is the smallest of four values given by:

$$C_{s2} = (0.15N + 0.25)I_e S_{DS}$$

$$C_{s2} = I_e S_{DS}$$

$$C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} \qquad \text{For } N \geq 2$$

$$C_{s2} = 0 \qquad \text{For } N = 1$$

Step 4: Use Figure 12.10.3-1 to determine C_{px} (see ASCE/SEI 7-16 Section 12.10.3.2).

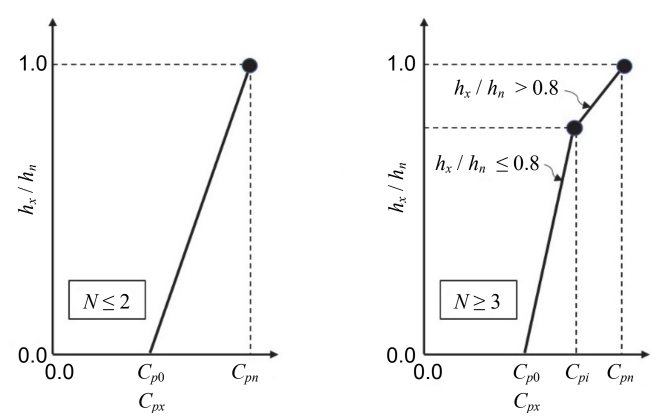

Figure 4-3. Calculating the design acceleration coefficient C_{px} in buildings two stories or less and in buildings three stories or more in height (values taken from ASCE/SEI 7-16 Figure 12.10-2).

4.2.4 Determine Diaphragm Design Force at Level x, F_{px}

$$F_{px} = \frac{C_{px}}{R_s} w_{px}$$

$$\geq 0.2 S_{DS} I_e w_{px}$$

4.3 Comparison of Design Force Levels

Comparisons of diaphragm seismic design force levels of ASCE/SEI 7-16 Sections 12.10.1 and 12.10.3 along the heights of a number of precast concrete buildings assigned to various SDCs are presented in this section. Note that ASCE/SEI 7-16 Section 12.10.1 is only used for the purpose of comparison of diaphragm design force levels, since this section is not applicable for precast concrete diaphragms in structures assigned to SDC C, D, E, or F.

4.3.1 Example 1: Four-Story Perimeter Shear Wall Precast Concrete Parking Structure (SDC C, Knoxville, TN)

The structure for Example 1, shown in **Figure 4-4**, is a four-story perimeter shear wall precast concrete parking structure assigned to SDC C.

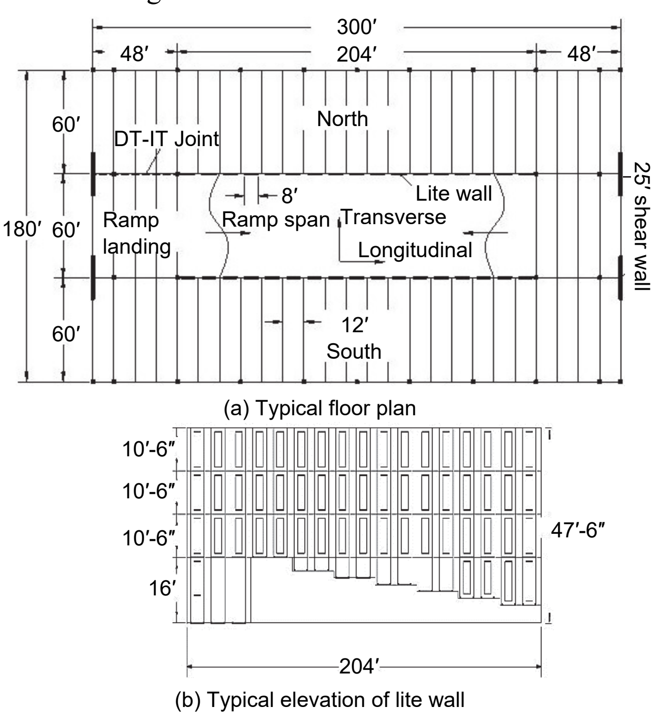

Figure 4-4. Example 1: Four-story perimeter shear wall precast concrete parking structure.

As seen in the plan view in **Figure 4-4a**, the parking structure has three bays with a central ramp. The structural plan has a footprint of 300 feet by 180 feet, resulting in 300 feet by 60 feet dimensions for each sub-diaphragm. The floor-to-floor height is 10.5 feet for a typical story and 16 feet for the first story. The SFRS in the transverse direction is composed of four 25-foot long perimeter precast walls, two at each end of the structure. The SFRS in the longitudinal direction consists of 34 interior lite walls flanking the central ramp (see elevation in **Figure 4-4b**).

The comparison of diaphragm design force levels for the structure in the transverse direction by ASCE/SEI 7-16 Section 12.10.1 (marked by 12.10.1, SDC C) and by ASCE/SEI 7-16 Section 12.10.3 for each design option (marked EDO – SDC C, BDO – SDC C, and RDO – SDC C) is illustrated in **Figure 4-5**.

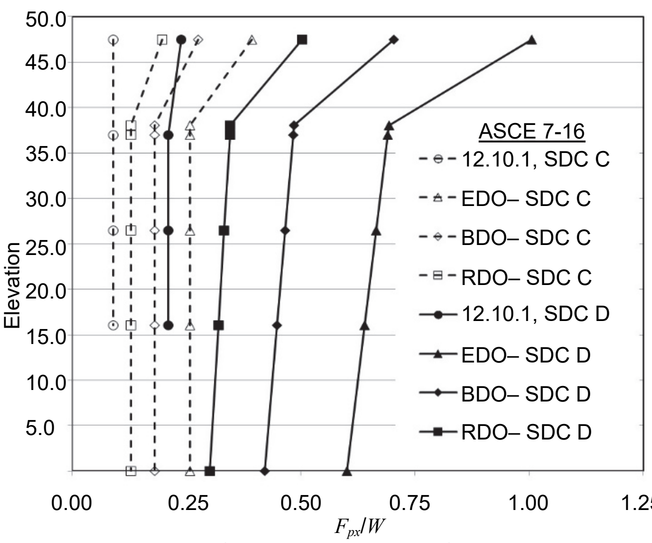

Figure 4-5. Design force level comparisons for Example 1 and 2 structures.

4.3.2 Example 2: Four-Story Interior Shear Wall Precast Concrete Parking Structure (SDC D, Seattle,WA)

The structure for Example 2, shown in **Figure 4-6**, is a four-story interior shear wall precast concrete parking structure assigned to SDC D. As seen in the plan view in **Figure 4-6a**, the parking structure has three bays with a central ramp. The structural plan has a footprint of 300 feet by 180 feet, resulting in 300-feet by 60-feet dimensions for each sub-diaphragm. The floor-to-floor height is 10.5 feet for a typical story and 16 feet at the first story. The SFRS in the transverse direction is composed of four 25-feet long interior reinforced concrete walls. The SFRS in the longitudinal direction consists of 34 interior lite walls flanking the central ramp (see elevation in **Figure 4-6b**). The comparison of diaphragm design force levels for the structure in the transverse direction by ASCE/SEI 7-16 Section 12.10.1 (marked 12.10.1, SDC D) and by ASCE/SEI 7-16 Section 12.10.3 for each design option (marked EDO – SDC D, BDO – SDC D, and RDO – SDC D) is also illustrated in **Figure 4-5**.

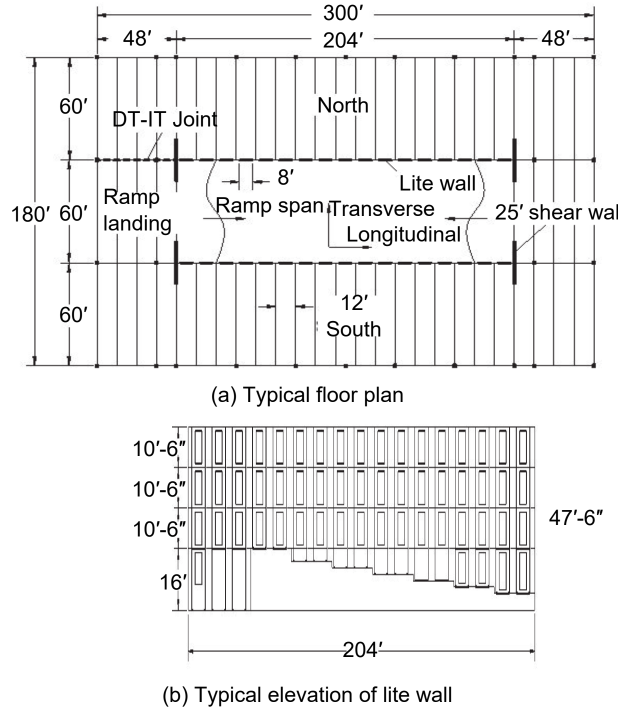

Figure 4-6. Example 2: Four-story interior shear wall precast concrete parking structure.

4.3.3 Example 3: Eight-Story Precast Concrete Moment-Resisting Frame Office Building (SDC C, Knoxville, TN and SDC D, Seattle, WA)

The structure for Example 3, shown in **Figure 4-7**, is an eight-story precast concrete moment resisting frame office building. The structure has three bays with a footprint of 230 feet by 147 feet. The story height is 13 feet for a typical story and 15 feet for the first story. The SFRS in the transverse as well as in the longitudinal direction is composed of inter+mediate moment resisting frames for SDC C and special moment resisting frames for SDC D. The precast floor system consists of double tees with a 3-inch topping slab. The comparison of diaphragm design force levels for the SDC C structure in either direction by ASCE/SEI 7-16 Section 12.10.1 (marked 12.10.1, SDC C) and by ASCE/SEI 7-16 Section 12.10.3 for each design option (marked EDO – SDC C, BDO – SDC C, and RDO – SDC C) is illustrated in **Figure 4-8**. The comparison of diaphragm design force levels for the SDC D structure in either direction by ASCE/SEI 7-16 Section 12.10.1 (marked 12.10.1, SDC D) and by ASCE/SEI 7-16 Section 12.10.3 for each design option (marked EDO – SDC D, BDO – SDC D, and RDO – SDC D) is also illustrated in **Figure 4-8**.

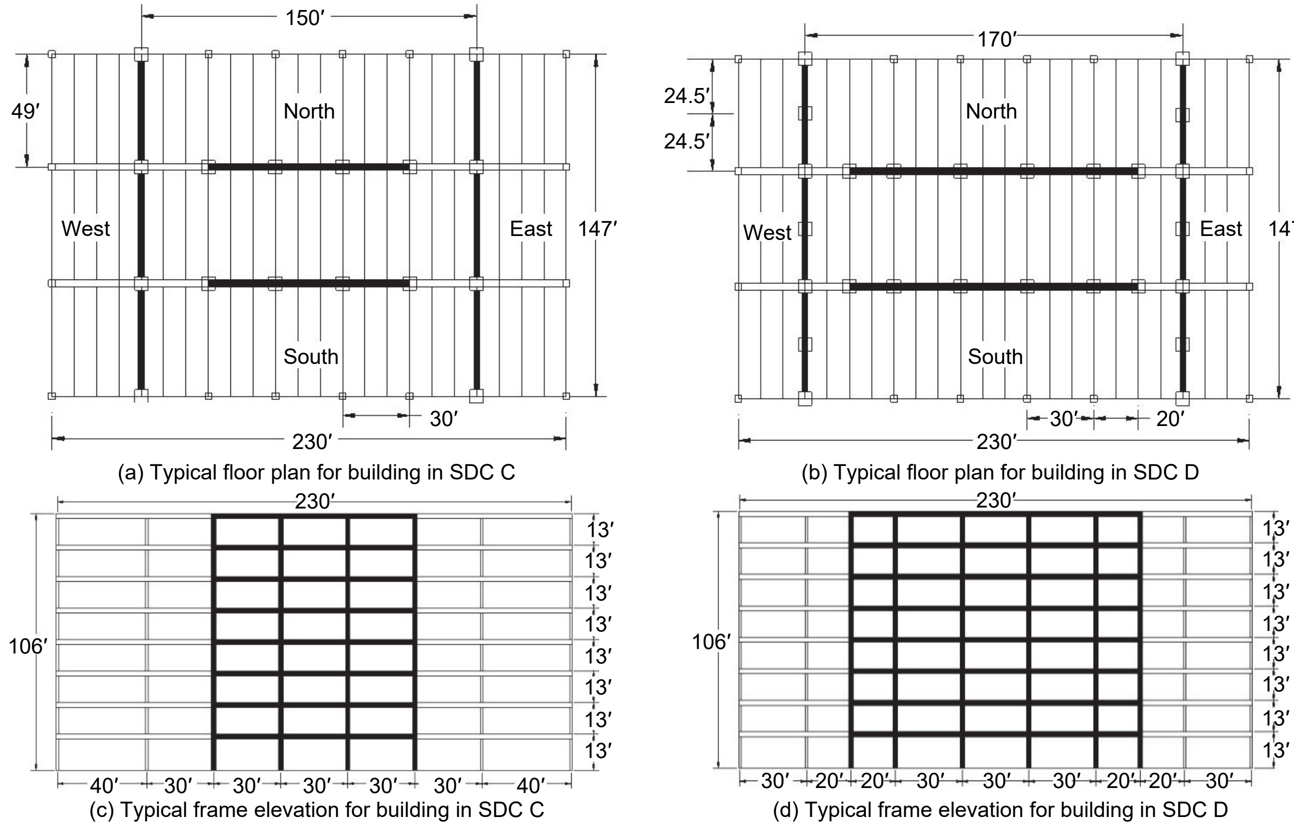

Figure 4-7. Example 3: 8-story moment resisting frame office building of precast concrete designed for SDC C and D.

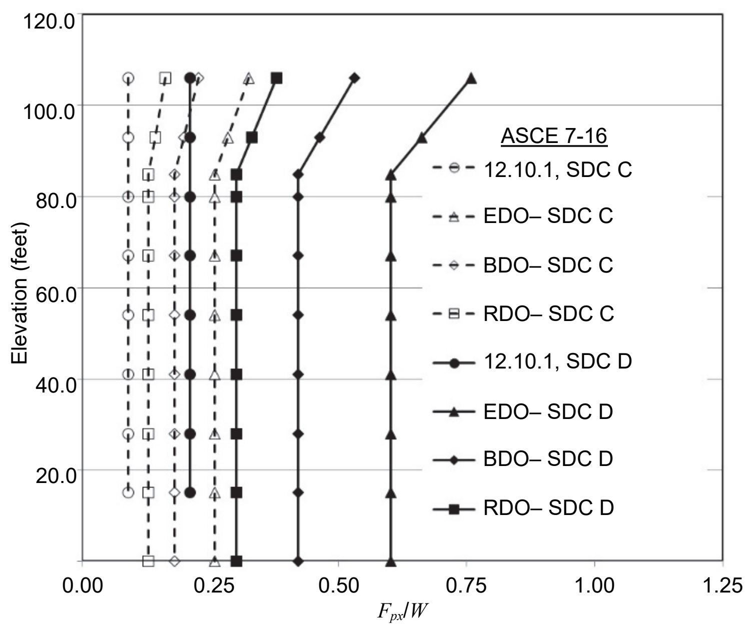

Figure 4-8. Design force level comparisons for Example 3 buildings designed for SDC C and D.

4.3.4 Example 4: Eight-Story Precast Concrete Shear Wall Office Building (SDC C, Knoxville, TN and SDC D, Seattle, WA)

The structure for Example 4, shown in **Figure 4-9**, is an eight-story precast concrete perimeter shear wall office building. The structure has three bays with a footprint of 230 feet by 147 feet. The story height is 13 feet for a typical story and 15 feet for the first story. The SFRS in the transverse direction is composed of two perimeter ordinary reinforced concrete shear walls for SDC C and four perimeter special reinforced concrete shear walls for SDC D. The SFRS in the longitudinal direction is composed of four perimeter ordinary reinforced concrete shear walls for SDC C and four perimeter special reinforced concrete shear walls for SDC D. The precast floor system consists of double tees with a 3-inch toping.

The comparison of diaphragm design force levels for the SDC C structure in either direction by ASCE/SEI 7-16 Section 12.10.1 (marked 12.10.1, SDC C) and by ASCE/SEI 7-16 Section 12.10.3 for each design option (marked EDO – SDC C, BDO – SDC C, and RDO – SDC C) is illustrated in **Figure 4-10**. The comparison of diaphragm design force levels for the SDC D structure in either direction by ASCE/SEI 7-16 Section 12.10.1 (marked 12.10.1, SDC D) and by ASCE/SEI 7-16 Section 12.10.3 for each design option (marked EDO – SDC D, BDO – SDC D, and RDO – SDC D) is also illustrated in **Figure 4-10**.

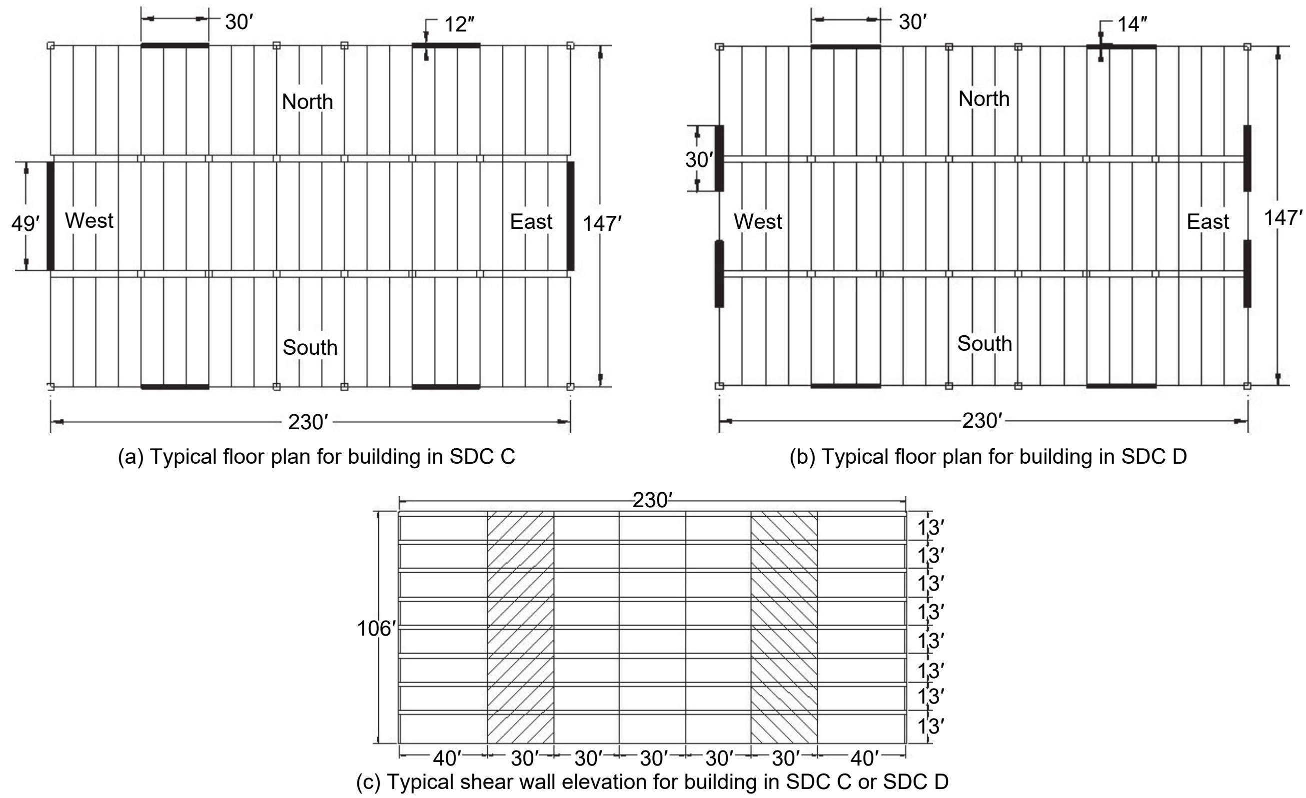

Figure 4-9. Example 4: Eight-story precast concrete shear wall office building designed for SDC C and D.

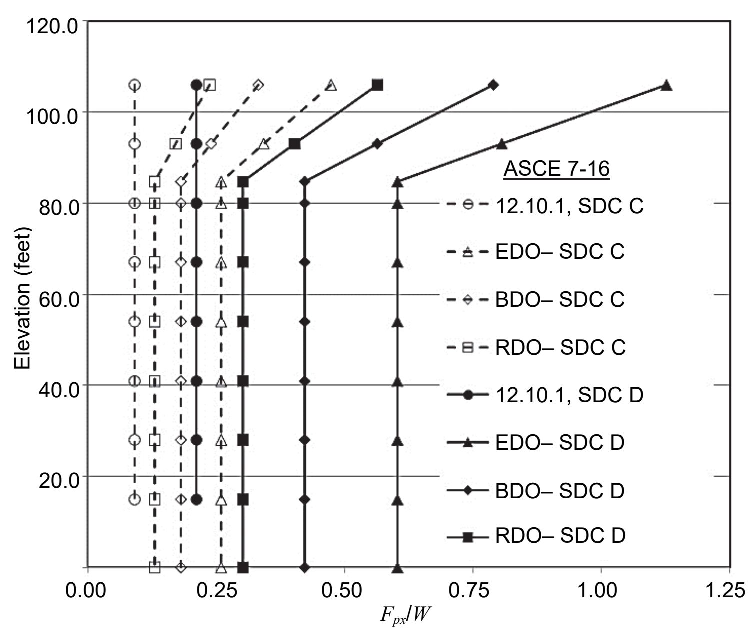

Figure 4-10. Design force level comparisons for Example 4 buildings designed for SDC C and D.

5. Diaphragm Analysis

5.1 Flexible versus Rigid Diaphragms

The prescriptive determination of a diaphragm as flexible or rigid is made in ASCE/SEI 7-16 based on conditions that do not apply directly to precast concrete construction. A general provision permits diaphragms to be considered flexible if the in-plane deflection under lateral load is more than two times the average story drift of the adjoining vertical elements. What is important for the overall analysis of the seismic force-resisting system is that a diaphragm is classified as rigid if it can distribute the horizontal forces to the vertical elements of the seismic force-resisting system in proportion to their relative stiffnesses. This section will review the behavior of common types of precast concrete diaphragms for guidance as to where they should be defined as rigid, semi-rigid, or flexible.

The behavior of diaphragms as rigid or flexible depends on many factors, including spans, aspect ratios, jointing, and connections. It also depends on the relative stiffnesses of the vertical elements of the seismic force-resisting system. In precast concrete structural systems, walls are a preferred system of lateral support (Cleland and Ghosh 2012). Walls are located based on the functional and architectural requirements of the building. It is common to proportion and detail walls to mobilize dead loads to resist overturning. These walls may be loadbearing or connected to adjacent loadbearing members. This approach may reduce the uplift demands on the supporting foundations, but impose high demands on the diaphragm to distribute the in-plane loads over long spans and around interior walls with less stiffness to other walls that are capable of offering more resistance. Long-span diaphragms may not be sufficiently stiff or strong with practical connection and reinforcement design; so they may behave as flexible and not rigid diaphragms.

Unless a precast diaphragm clearly qualifies as rigid or flexible, ASCE/SEI 7-16 Section 12.3.1 requires that "the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption)." One way to address this requirement is an envelope approach of considering the most severe effects of both rigid and flexible diaphragm assumptions. The system design for a precast concrete structure should consider the diaphragm and the seismic force-resisting system together, so that lateral force distributions resulting from assumptions of rigid and flexible diaphragms are not significantly different (Cleland and Ghosh 2012).

5.2 Beam Analogy

The most common method of analysis for precast concrete diaphragms models the floor plate as a deep horizontal beam in the plane of the floor. This beam analogy implies a combination of flexural and shear behavior. The components of the diaphragm according to beam analogy are illustrated in **Figure 5-1**. For this approach, the chord reinforcement or connections are designed to carry the entire calculated in-plane bending moment and the web reinforcement or connections are designed to carry the entire in-plane shear across panel joints parallel to the load. Chord forces are calculated from a flexural analysis using a tension/compression couple between the centroids of the chord reinforcement at the floor edges. For simple-span diaphragms, the highest chord forces are developed midway between supports. For diaphragms in buildings with multiple vertical systems that create multiple spans, the maximum moments may occur at these supports or at other locations. The shear generally accumulates to its largest magnitude at the supports provided by the vertical systems.

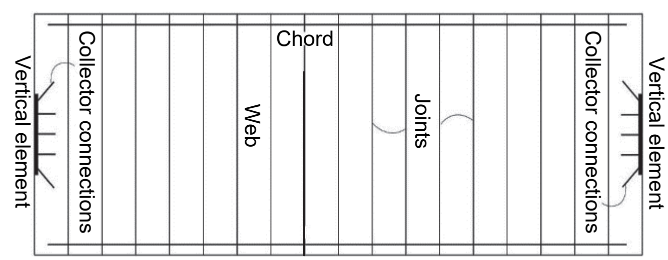

Figure 5-1. Diaphragm components according to beam analogy.

In precast floors and roofs without composite topping, the individual components comprising a floor or roof diaphragm rely on mechanical connections at the joints to transmit the shear and flexure. Joints between the precast components that are perpendicular to the seismic force-resisting system include connections that transfer forces across the joint, analogous to horizontal shear flow (VQ/I) in a composite beam. This is illustrated in **Figure 5-2**.

The beam analogy method is a simplified version of a free-body method for analysis. This method does not consider deep beam effects and will miss tension forces that develop independently of the tension-compression couple that results from beam flexure. The penalties for using this method may produce more conservative results and heavier designs than may be obtained using other free-body methods or finite element analysis.

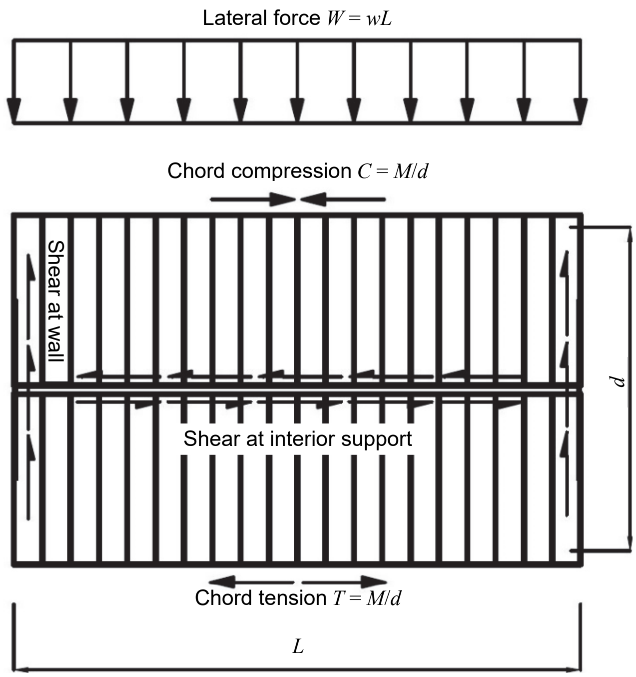

Figure 5-2. Diaphragm forces in a precast floor with multiple bays.

5.3 Strut and Tie Analysis

Precast concrete diaphragms may be modeled using tension ties and compression struts in the plane of the diaphragm following load paths developed by components and their connections.

ACI 318-14 Chapter 23 provides an alternate method of analysis for concrete structures that have regions where the plane sections assumption for linear distribution of strain is not valid. This method has been applied to deep beams and other members with regions of large discontinuities. Since diaphragms may, by beam analogy, be considered deep beams with large discontinuities, strut-and-tie modeling can be adapted to diaphragms. The commentary to ACI 318-14 Section 18.12.8.1, states: "Strut-and-Tie models are potentially useful for designing diaphragms with openings."

Strut-and-tie modeling requires some idea of the lateral distribution to the vertical elements of the SFRS so that the model can be developed with struts parallel to the orientation of initial cracking. The development of strut-and-tie models for precast concrete diaphragms must include consideration of the jointing and the load paths created by connections. In diaphragms formed with cast-in-place concrete topping, the placement of reinforcement is guided by the locations of tension ties in the model. Since ties cannot be placed in untopped diaphragms in accordance with strut and tie modeling, the method may not be applicable to them.

5.4 Free-Body Analysis

"Internal forces at all potential critical sections in the diaphragm can be determined by taking the applied diaphragm forces (F_{px}) and reactions on the diaphragm and evaluating appropriate free-bodies around each critical section using the principles of statics." (Part 1, p. 6, Fleischman 2014). This method uses free body cuts through diaphragms or sub-diaphragms with boundary conditions determined to satisfy global equilibrium.

As outlined in Part 3 of Fleischman (2014), the free body method to calculate the diaphragm internal forces can be performed as follows:

- Cut a free body for the diaphragm or sub-diaphragm (typically at the interfaces between diaphragm and lateral force-resisting system and internal beam)
- Apply the appropriate amplified diaphragm design forces
- Calculate the reaction forces at boundaries of the diaphragm free body to satisfy the global equilibrium for a static determinate free body. For an indeterminate free body, assumptions have to be made based on mechanics
- Determine the diaphragm internal forces using local equilibrium with sections cut at diaphragm joints
- Consider the loading in each orthogonal direction individually; use the maximum case of combined (M, N, and V) forces

Free body diagrams have been developed for common precast concrete configurations for parking garages and for office buildings and included in Fleischman (2014). Adjustments for the effects of alternate load paths are considered based on research. These configurations reveal conditions that might be generalized to other configurations. The analysis made for parking garage configurations with ramp walls on the interior and inverted tee beams at end bays for traffic cross-over shows that diaphragms that span end to end to exterior shear walls develop tension along these end beams, which reduces the diaphragm moment. When the shear walls are moved to the interior and to the ends of the ramp, this tension goes to zero. The interior ramp walls act in their plane to resist diaphragm forces, reducing the chord tension forces by approximately 15 percent.

For configurations with irregularities or those that vary from the prototypes that were presented in Fleischman (2014), it may be more appropriate to provide analysis by the finite element method, discussed below.

5.5 Finite Element Analysis

The use of finite element analysis software has become common practice in the structural engineering design office. Diaphragm analysis may be made utilizing a 3D representation of a structure developed for overall design of that structure. The interaction of structural modeling with Building Information Modeling (BIM) is advancing. The 3D information can be valuable for defining the structural dimensions to be included in the analysis, but an abundance of caution is needed when applying these exchanges between graphics and analysis to precast concrete design. The exchange may create a monolithic representation of the double tee to beam or beam to column connection that can develop negative moments from out-of-plane loading and in-plane moments for lateral forces where none occur in the actual structure. It is important to recognize that practically all horizontal components in precast concrete are simply supported. Beam and double tee components have bearing connections at the bottom and horizontal force connections at the top with open joints over the depth of the component. Line elements may be difficult to utilize to model these situations. For analysis of diaphragms that use double tees, only the double tee flanges, with or without topping, participate in the transfer of horizontal diaphragm forces, and the analysis model should reflect this.

Whether the analytical model is a three-dimensional representation of the whole structure or separate two-dimensional representation of individual floors, the level of discretization in the diaphragm modeling can be considered based on the complexity of the diaphragm and the level of accuracy needed for design. An assumption of a simple rigid diaphragm may be sufficient, satisfying statics but not considering in-plane deformations. More information on load paths may be derived by using a membrane model with in-plane deformation. If a better approximation of absolute diaphragm contribution to drift is needed, the stiffness of the membrane can be modified to reflect general cracking or joint opening using reduced moduli of elasticity and shear. For some complex configurations, using discrete jointing and connections (Fleischman and Wan 2007) can provide more accurate estimates of connection forces and diaphragm deformations.

When using separate floor-level 2D plate element analysis, forces that are consistent with the interaction with the vertical elements of the seismic force-resisting system can be applied. A 2D analysis with a rigid diaphragm or plate or membrane elements may be sufficient. In some cases, a hybrid approach with uniform plate elements for the double tee flanges, with joints and connections between tees and beams represented to reveal the demands on these seams, can be used. Complex diaphragms may need to have the double tee to double tee as well as beam to double tee connections included in the analysis model (Cleland and Ghosh 2012). These models are generally set up for a linear elastic analysis subjected to equivalent lateral forces. It is only very rarely needed to perform nonlinear analysis of precast concrete diaphragms. For this, guidance is provided in Wan et al. (2015) and Zhang et al. (2016).

Two-dimensional models do not directly include the vertical elements of the seismic force-resisting system. Therefore, the SFRS reaction forces are applied to the diaphragm model at the locations of the vertical elements of the SFRS. The in-plane applied forces are body forces developing in the floor, and can be approximated as a distributed load. The total reaction forces will be equal and opposite to the total in-plane distributed diaphragm force and made proportional to the distributed forces from the lateral load analysis. Adjustments in the body forces may be necessary for reaction forces determined from analysis with accidental torsion. The orthogonal direction should be considered separately, but orthogonal forces in the vertical system from in-plane torsion should be included. Some nodal restraint will be required for the stability of the analysis model, but these should be limited to avoid introducing unintended load paths. The model should not be over-constrained. If the applied forces are balanced, the incidental forces at these restraints will be small.

Figure 5-3 shows a partial framing plan of a precast floor level with beams, double tees, and surrounding walls. A concern in this diaphragm is the transfer of diaphragm forces across the joints of multiple beam lines and the effects of re-entrant corners from the irregular layout.

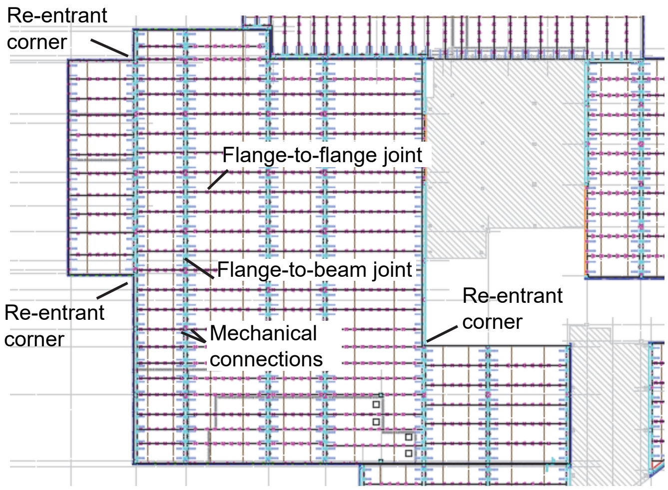

Figure 5-3. Partial framing plan of diaphragm.

A finite element model was developed with continuous plate elements across the double tee flanges but with joints at the ends of the tees to the beams. The joint is bridged with elements that represent the mechanical connections. The finite element mesh was made finer along the beam joints to capture local effects, but the meshing has a uniform spacing that does not necessarily follow the flange-to-flange joints, as shown in **Figure 5-4**. The finite element analysis used elements that made it possible to improve the interface between beam elements and plate elements. The connections also include a representation of the development of the tail bars into the finite element mesh rather than simply connecting nodes across the joints. This provides a truer representation of the connection interface with the beam and double tee flanges.

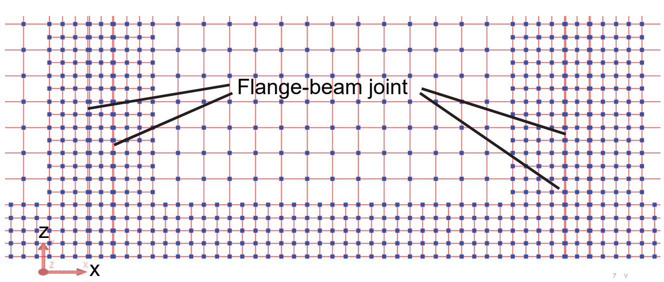

Figure 5-4. Beam and flange mesh detail showing joints.

The model can provide a prediction of the deformed shapes of the diaphragm under the lateral forces in the two orthogonal directions. One direction is shown in **Figure 5-5**.

The finite element model provides plate forces in the areas of discontinuity from the joints. Principal stresses can also be determined at discontinuities and at boundaries, shown in **Figure 5-6**. The figure shows that the stress distribution is not simple, and would not be obtained using a simpler analysis method. The forces along joints can be determined by cutting sections along these lines and collecting the sum of forces to get tension and shear forces at these lines for proportioning flange reinforcement and flange-to-flange connections.

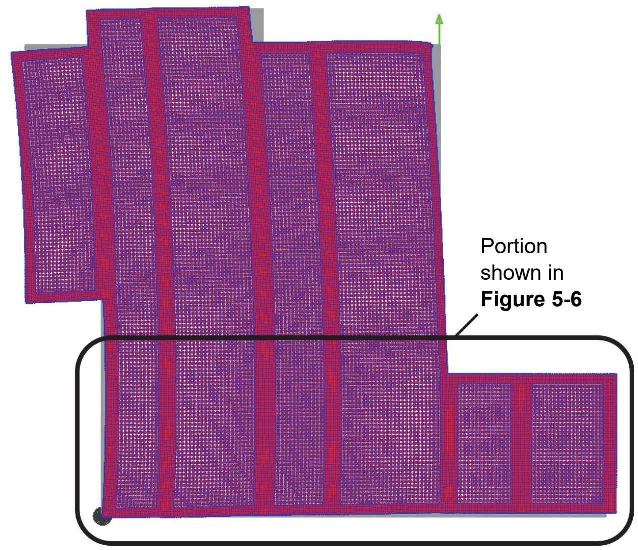

Figure 5-5. Deformed shape of finite element model representing the framing plan in Figure 5-3 for one direction of lateral loading.

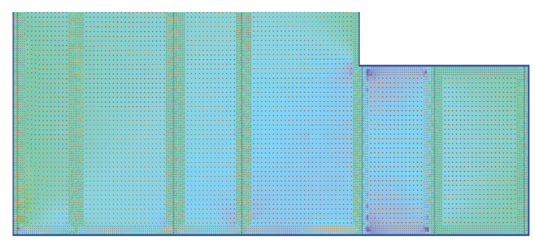

Figure 5-6. Detail of model output showing maximum principal stresses with darker color indicating increase at inside corner and extended sub-diaphragm.

The finite element model for this diaphragm provides an understanding of the distribution of forces along load paths that cross beam to tee joints. These results are likely to be more reliable and less conservative than might be determined by trying to force this irregular shape to fit an equivalent beam model. A monolithic diaphragm model may be sufficient to return forces without the interior joints and connections. Cutting sections at joints or boundaries provides the numerical results needed to proportion the connections. The option of using discrete modeling, while not essential, may make the determination of connection forces simpler.

6. Precast Concrete Diaphragm Design Procedure

The following describes the seismic design of topped or untopped precast concrete diaphragms according to ASCE/SEI 7-16 Section 14.2.4, Additional Design and Detailing Requirements for Precast Concrete Diaphragms. Seismic design by ASCE/SEI 7-16 Section 14.2.4 is required when the design force level of ASCE/SEI 7-16 Section 12.10.3 is used. For precast concrete diaphragms in buildings assigned to SDC C, D, E, or F, the use of that design force level is mandatory. For precast concrete diaphragms in buildings assigned to SDC B, use of the design force level of ASCE/SEI 7-16 Section 12.10.3 is optional. These requirements set forth in ACI 318-14 Chapter 12, Diaphragms, and Section 18.12, Diaphragms and Trusses, remain applicable to a precast concrete diaphragm designed by ASCE/SEI 7-16 Section 14.2.4.

6.1 Determine Diaphragm Seismic Demand Level

The diaphragm seismic demand level is a function of the seismic design category a building is assigned to, the number of stories in the building, the diaphragm span as defined in ASCE/SEI 7-16 Section 14.2.4.1.1, and the diaphragm aspect ratio as defined in ASCE/SEI 7-16 Section 14.2.4.1.2. There are three levels: low, moderate, and high. The significance of the diaphragm seismic demand level is explained in **Table 6-1**. The diaphragm seismic demand level leads to the selection of the diaphragm design option. In fact, the diaphragm design option cannot be chosen without the diaphragm seismic demand level. For structures assigned to SDCs B and C, the diaphragm seismic demand level is automatically designated as low. For structures assigned to SDC D, E, or F, the diaphragm seismic demand level is determined from **Figure 6-1** (ASCE/SEI 7-16 Figure 14.2.4-1).

Table 6-1. Significance of Diaphragm Seismic Design Level

Diaphragm Seismic Demand Level	What does it mean?
Low	Low seismic vulnerability; automatically assigned to SDC B and SDC C diaphragms
Moderate	Moderate seismic vulnerability
High	High seismic vulnerability

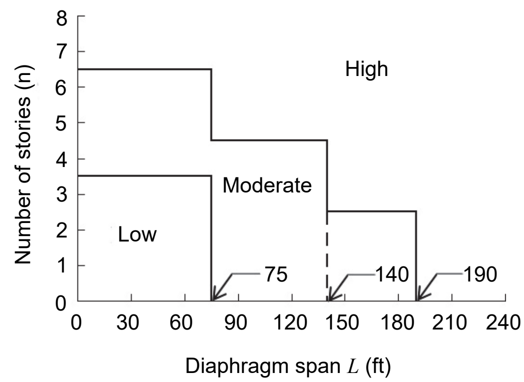

Figure 6-1. Diaphragm seismic demand level (ASCE/SEI 7-16 Figure 14.2-1, printed with permission from ASCE).

6.2 Determine Diaphragm Design Option and Corresponding Connector or Joint Reinforcement Deformability Requirement

The diaphragm design option addressed in ASCE/SEI 7-16 Section 14.2.4.2 provides a mechanism for selecting the target performance of a diaphragm when subject to earthquake excitation, in terms of the diaphragm design option.

The flow chart in **Figure 6-2** illustrates (1) which diaphragm design option is permitted to be used when, and (2) the corresponding minimum precast concrete diaphragm connector or joint reinforcement classification that would need to be used per ASCE/SEI 7-16 Section 14.2.4.3.

6.3 Comply with Qualification Procedure

This step is to ensure that the selected connector or joint reinforcement meets connector or joint reinforcement qualification requirements per ASCE/SEI 7-16 Section 14.2.4.4.

See discussion of qualification procedure in Section 8.

6.4 Amplify Required Shear Strength

Determine the diaphragm force reduction factor, R_s, from **Table 4-1** (ASCE/SEI 7-16 Table 12.10-1).

Amplify the required shear strength for the diaphragm by the diaphragm shear overstrength factor, Ω_v, which is to be taken equal to $1.4R_s$.

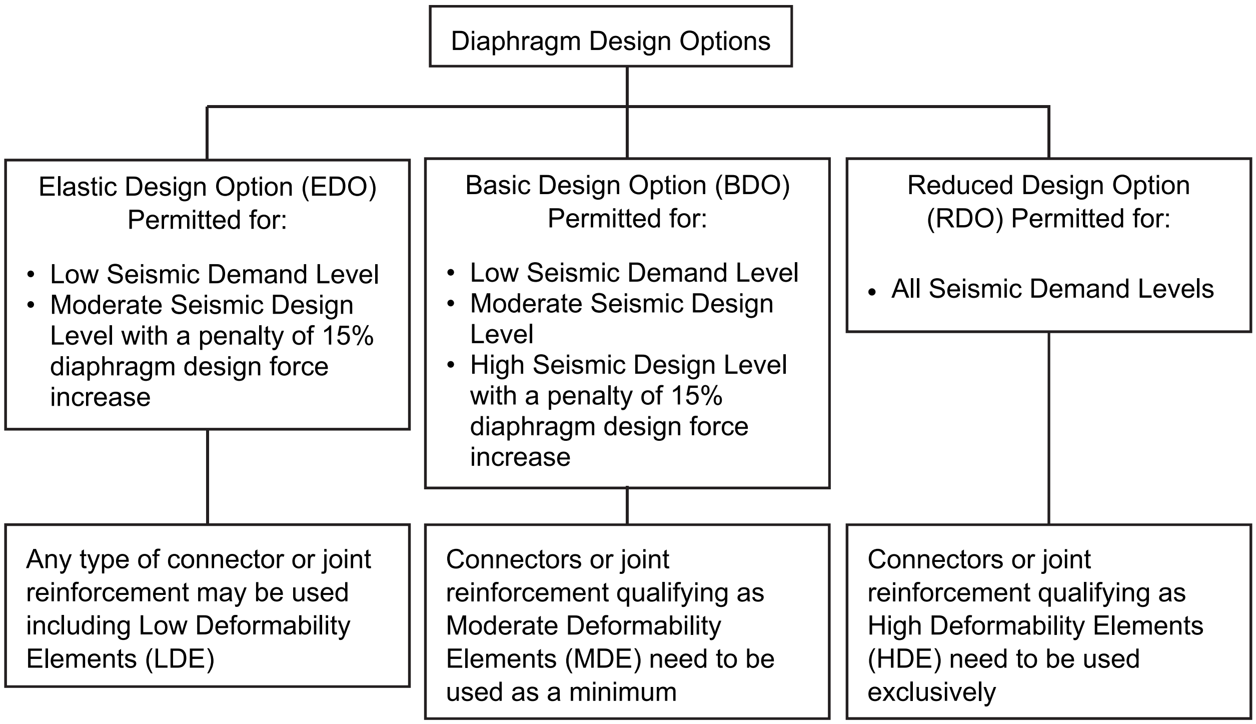

Figure 6-2. Diaphragm seismic design level – diaphragm design option – connector classification.

7. Diaphragm Design and Detailing for Flexure and Shear

7.1 Design for Flexure

Although precast concrete diaphragms may not be analyzed using beam analogy, most diaphragms have regions near the boundaries or at openings, which are subjected primarily to axial loads. When the reinforcement required for these areas forms a couple with corresponding reinforcement on the opposite side of the diaphragm in resistance to flexure, it is commonly designated as chord steel. Flexural strength is determined using the same assumptions applied to beams. When the axial forces develop a mechanism for transfer of the in-plane shear from a sub-diaphragm into a main diaphragm, or to the vertical elements of the seismic force-resisting system, the reinforcement is considered to act as a collector. A chord for lateral forces in one direction may act as a collector for forces in the orthogonal direction (see **Figure 7-1**). ASCE/SEI 7-16 Section 12.10.3.4 requires: "In structures assigned to Seismic Design Category C, D, E, or F, collectors and their connections, including connections to vertical elements, shall be designed to resist 1.5 times the diaphragm inertial forces from Section 12.10.3.2 plus 1.5 times the design transfer forces."

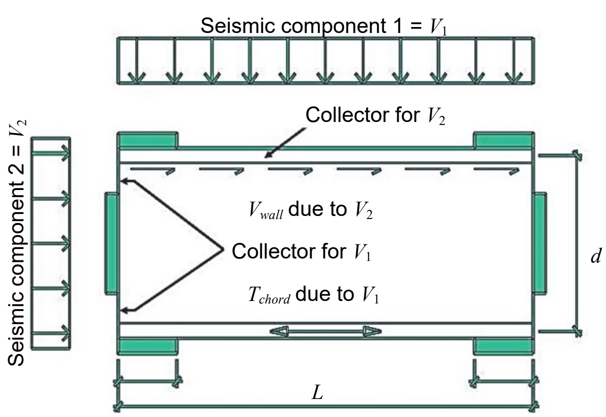

Figure 7-1. Diaphragm chords and collectors.

For higher Seismic Design Categories (D, E and F), ACI 318-14 prescribes additional requirements for the design of collector elements. For designs which use the design forces required by ASCE/SEI 7-16 Section 12.10.3.4, the higher limits for compressive stresses should be applied. Collector elements with compressive stress exceeding f_c' must have transverse reinforcement in accordance with the requirements for special moment frame columns with area prescribed in Table 18.12.7.5. For precast concrete systems with cast-in-place concrete topping or thin precast slab sections, the detailing of such reinforcement is generally not feasible. It is important to proportion the load paths in the precast diaphragm to avoid these requirements.

7.2 Amplification of Required Shear Strength

The level of force used for flexural design in the diaphragm is determined based on the performance objective consistent with the type of connections used. The underlying objective of all design options is that shear connections should remain elastic up to the maximum considered earthquake. The diaphragm shear overstrength factor, Ω_v, is thus applied to the diaphragm design forces. The level of diaphragm shear overstrength required relative to the diaphragm flexural strength varies with the design option. The factor is calibrated so that the shear design force is the same for all design options.

7.3 Design for Shear

As indicated above, the intent of the provisions in ASCE/SEI 7-16 Section 14.2.4 is to maintain elastic shear response in the precast diaphragm up to the MCE level earthquake.

The nominal shear strength for a concrete diaphragm is calculated in accordance with ACI 318-14 Equation 18.12.9.1:

$$V_n = A_{cv}(2\lambda\sqrt{f_c'} + \rho f_y)$$

where λ is the modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal-weight concrete of the same compressive strength; ρ is the ratio of A_s to bd with A_s indicating area of nonprestressed longitudinal tension reinforcement, b the width of compression face of member, and d the distance from extreme compression fiber to centroid of longitudinal tension reinforcement.

ACI 318 limits the calculation of A_{cv} to the thickness of the topping slab for non-composite topping slab diaphragms and the combined thickness of the cast-in-place topping and the precast elements for composite topping slab diaphragms. The diaphragm is only as thick as the topping slab at the joints between precast double tee flanges. It is recognized that joints will tend to open up even when covered with a topping slab. ACI 318-14 Section 18.12.9.3 prescribes that "above joints between precast concrete elements in noncomposite and composite cast-in-place topping slab diaphragms" the shear strength is limited to the strength provided by the reinforcement crossing the joint as shear friction reinforcement. For this case, the coefficient of friction, μ, is taken as 1.0. Chord reinforcement crossing the joints may be considered for its contribution to shear friction, but at least one-half of the shear friction steel required must be uniformly distributed along the joint and also meet the area and spacing requirements for temperature and shrinkage reinforcement.

Mechanical connections that are qualified (see Section 8) may be used to provide the strength for the transfer of shear across the joints—alone for untopped diaphragms and in concert with topping slab reinforcement for topped diaphragms. Following the same principles applied to topping reinforcement, the contribution of the chord reinforcement or connections to the transfer of shear can be considered as well. Limits imposed on the amount of contribution of end reinforcement and for the distribution of shear strength along the joint should apply to mechanical connections as well.

7.4 *Design for Connection Interaction along the Joints*

Diaphragm joints include those between the floor units, at the interior beams and walls, and from the diaphragm components to the seismic force-resisting system. Connections must be designed to transmit forces across all joints. Design for the connections at precast concrete diaphragm joints includes the determination of joint stiffness to permit an estimate of the diaphragm contribution to drift. The stiffness and yield strength of a connection is determined in the qualification testing and can be derived from the backbone curve derived from such testing.

Each joint in the precast concrete diaphragm may have many connections and connections of different types and intended actions. These connections must be sufficient to provide the total strength to resist the diaphragm internal forces: a combination of tension, shear and flexure. The DSDM research (Fleischman 2014) suggested a general interaction equation for qualification of the joint reinforcement or connections:

$$\sqrt{\left(\frac{|M_u|}{\phi_f M_n}+\frac{N_u}{\phi_f N_n}\right)^2+\left(\frac{\Omega_v V_u}{\phi_v V_n}\right)^2}\le 1.0$$

where M_n is the nominal flexural strength at section; M_u is the factored bending moment at section; N_n is the nominal strength in tension; N_u is the factored tension normal to cross section; V_n is the nominal shear strength; V_u is the factored shear force at section; ϕ_f is the strength reduction factor for flexure and tension and is taken as 0.9; and ϕ_v is the strength reduction factor for shear and is taken as 0.85.

It is notable that the strength reduction factor for shear for qualified shear connections or reinforcement is 0.85, not the lower value of 0.75 in general use in ACI 318. The connection qualification protocol provides a level of reliability that merits the higher factor, and the design target is elastic response in the maximum considered earthquake.

The use of the general equation deserves some caution. Some of the connections with $\phi_n N_n$ strength may actually be in compression under M_u action at the joint. The equation, however, provides an assessment of the total array of connections in each joint to meet the total force and moment demands at that joint. It is important to include the shear resistance provided by the chord reinforcement or connections.

For some design cases, it is important to include the contribution of flexible diaphragm deformation to the lateral story drift of gravity system columns remote from SFRS elements. These cases include diaphragms with aspect ratios somewhat greater than 3 and buildings with more than 3 to 5 stories, depending on the design option being used. The diaphragm geometry considered in the evaluation of the in-plane stiffness is shown in **Figure 7-2**, from the DSDM report (Fleischman 2014).

When using the joint geometry shown in **Figure 7-3**, the depth of a compression block is determined as for a reinforced concrete section, with tension contribution from chord connections, web connections, and topping web reinforcement, if it is provided. Equilibrium is used to determine the depth of compression. For topped systems, the compression distribution is taken as linear. If there is a pour strip, the compression contact will likely be confined to the depth of the strip, which holds the untopped joint open. For this case, the center of compression can be taken as the center of the chord reinforcement. For a dry system without cast-in-place concrete, the center of compression must be estimated based on the compressive stiffness of the chord connection relative to that of the shear connections. The joint stiffness is a combination of the flexural and shear stiffnesses. Flexural stiffness is derived on the basis of a combination of rotation at the joint and rotation within the components. To use gross section properties of the diaphragm, it is necessary to use these properties in combination with an effective modulus of elasticity and an effective shear modulus. This allows the use of a simple non-discrete finite element model of the diaphragm to predict deformations.

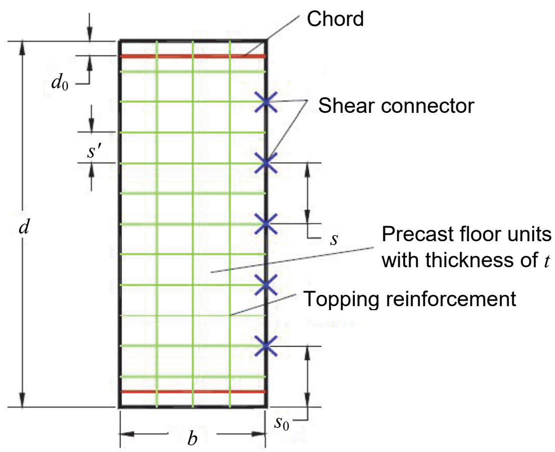

Figure 7-2. Diaphragm geometry (Fleischman 2014).

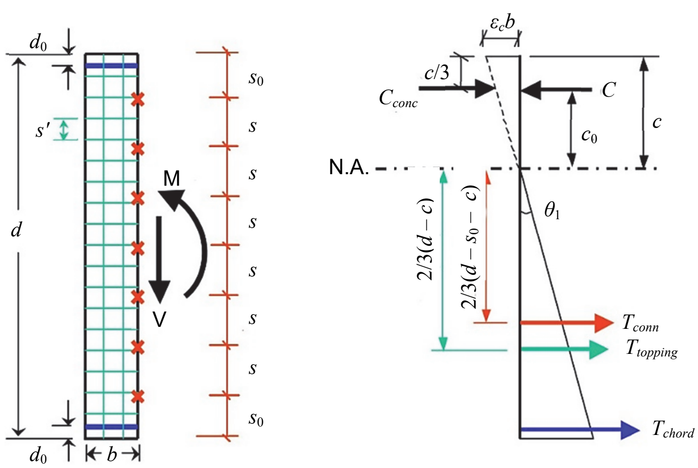

Figure 7-3. Free body diagram of a joint to determine effective section (Fleischman 2014).

8. Qualification and Classification of Connectors

8.1 Connector Deformability Limits

Proper performance of a diaphragm under seismic demands requires that the appropriate connection type be utilized for the design option chosen: EDO, BDO, or RDO. Due to the jointed nature of the precast diaphragm, the occurrence of inelastic behavior is concentrated at the joints and consequently in the connections that bridge the joints. As such, a diaphragm design approach that reduces design force levels below those needed for elastic response is contingent on the ability of the diaphragm connectors or joint reinforcement to accommodate inelastic deformations.

Diaphragm connectors or joint reinforcement is categorized as Low, Medium, or High Deformability Elements (LDE, MDE, or HDE) based on their ability to provide various degrees of tension deformation capacity. Although the diaphragm joint is subject to both in plane tension and shear, the overstrengths inherent in the design approaches preclude shear yielding. Consequently, connector classification is based solely on the tension deformation capacity. In accordance with ASCE/SEI 7-16 Section 14.2.4.3, an LDE has a tension deformation capacity less than 0.3 inch (7.5 mm), an MDE has a tension deformation capacity between 0.3 and 0.6 inch (7.5 and 15 mm), and an HDE has a tension deformation capacity greater than 0.6 inch (15 mm). As such, the diaphragm design options are tied to the appropriate choice of connections and joint reinforcement. For example, an RDO approach, which allows for yielding in the diaphragm in the design earthquake, will require the use of high deformability connectors or HDE to ensure that the precast elements remain connected under the seismic demand. A BDO approach assumes less yielding and therefore requires at least MDE and allows HDE, but precludes the use of LDE. The EDO approach allows for the use of any connection or joint reinforcement type, since the system will remain elastic under the MCE seismic demands. A summary of the connection classifications allowed with each design option is presented in **Table 8-1**.

To determine the deformation limits needed for the three design options, a parametric study was conducted on precast diaphragms (Zhang and Fleischman 2016). Nonlinear three-dimensional finite element analyses were conducted on precast concrete diaphragm systems designed using the EDO, BDO, and RDO. Variables included aspect ratio, span length, number of stories, diaphragm element type, and seismic force-resisting system (SFRS). The variables used are summarized in **Table 8-2**. The tension deformation limits determined for LDE, MDE, and HDE pertain to the maximum deformations reached in the MCE event. The deformation limits with respect to the diaphragm response are illustrated in **Figure 8-1**. The maximum anticipated joint openings were estimated in the study and found to exhibit levels less than the limits prescribed.

Table 8-1. Connector or Joint Reinforcement Classifications

Connector or Joint Reinforcement Classification		Deformability of Element: Low (LDE)	Moderate (MDE)	High (HDE)
Tension Deformation Capacity		< 0.3 inch	0.3 inch to 0.6 inch	≥ 0.6 inch
Seismic Design Option	EDO	Rcmd*	Allowed	Allowed
	BDO	NA	Rcmd*	Allowed
	RDO	NA	NA	Rcmd*

*Rcmd = Recommended

Table 8-2. Parametric Study Variables

Length (feet)	240	180	120	60	128	96
Aspect Ratio	4	3	2	1	4	3
No. of Stories	6	6	4	4	2	2
Diaphragm Type	DT[1]	DT[1]	DT[1]	HC[2]	HC[2]	HC[2]
Vertical SFR Element	SW[3]	SW[3]	SW[3]	MF[4]	MF[4]	MF[4]
SDC	E	E	D	D	C	C

[1]Double Tee Beam [2]Hollow Core Beam [3]Shear Wall [4]Moment Frame

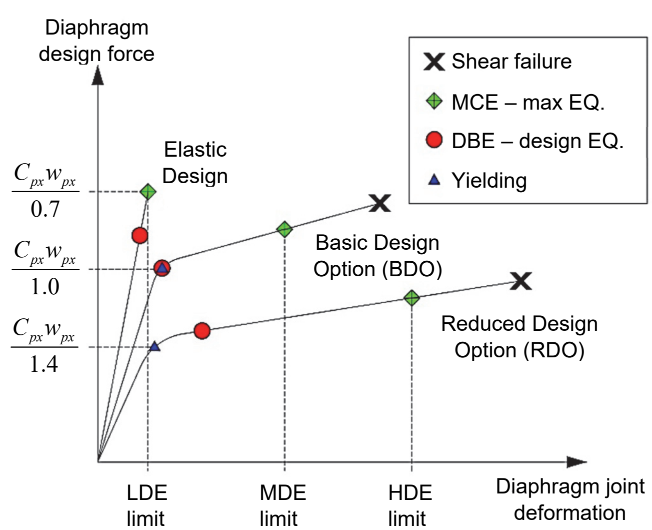

Figure 8-1. Diaphragm deformation relative to design option.

8.2 Loading Protocol

Tension deformation capacity shall be determined by testing of individual elements following the cyclic testing protocols defined in ASCE/SEI 7-16 Section 14.2.4.4, Precast Concrete Diaphragm Connector and Joint Reinforcement Qualification Procedure. The exception to this is deformed bar reinforcement meeting ASTM A615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement* (ASTM 2016a) or ASTM A706, *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement* (ASTM 2016b), placed in cast-in-place concrete topping or cast-in-place concrete pour strips, which is prequalified as HDE. The loading protocol requires cyclic tension testing of the connections using an appropriate testing subassembly that mimics the boundary conditions that exist in the full diaphragm. To ensure reproducibility of the results, 3 to 6 tests are required. The protocol provides detailed recommendations on how to create the test subassembly, execute the test, process the data and determine the resulting tension deformation. Additional requirements are included, which allow for the determination of the connection tension and shear stiffness, strength, and shear deformation capacity. These values are necessary for design of connection spacing and system modeling as discussed in Section 7.

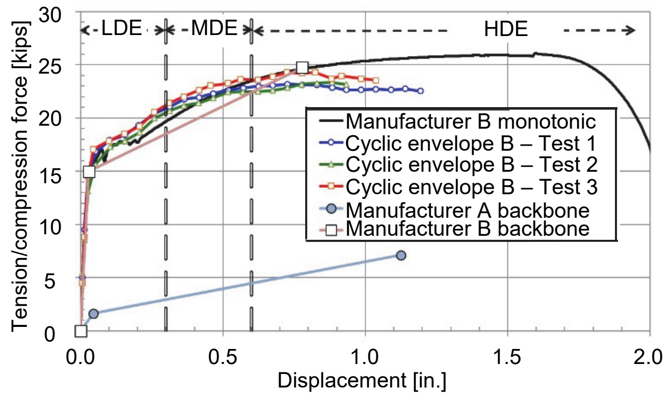

Figure 8-2. HDE connection response.

8.3 Classification of Connectors

A considerable number of connections have been experimentally evaluated through in-plane testing. Experiments on connections started with the work of Venuti (1970) on bent reinforcing bar hairpin flange-to-flange connections and continued to the work that established the testing methodology incorporated into ASCE/SEI 7-16. A number of the connection studies are summarized in Ren and Naito (2013). The initial testing methodology was first reported in Naito and Ren (2013) and was later modified in the course of inclusion in ASCE/SEI 7-16. As discussed in Section 8.2, the methodology requires a minimum number of tests to determine the deformation limits. This requirement was not part of the initial testing recommendation and, as such, few connections have undergone characterization in accordance with the testing protocol included in ASCE/SEI 7-16. The simplified tension backbone curves from a proprietary HDE flange-to-flange web connection (manufacturer A) and a proprietary HDE chord connection (manufacturer B) are illustrated in **Figure 8-2**. Also included in this figure are the monotonic and cyclic envelopes for the HDE chord connection.

9. Detailing and Constructability Issues

9.1 Field Experiences

Actual placement of reinforcement in topping slabs and pour strips requires consideration of the configurations and physical conditions of precast concrete framing. Prestressing in concrete sections causes the members to camber upward as the prestress applies an internal negative moment to offset the positive moment from gravity loads. Most uses anticipate level floors, or floors with uniform slope over the camber, although floors with uniform topping slab thickness may be acceptable in parking garages if the slopes to drain are sufficient to overcome the rise from camber.

There are other functional conditions that require detailing considerations in the layout and placement of reinforcement in cast-in-place topping slabs or pour strips. In some cases, there are steps in floor elevations that need to be made continuous for the transfer of in-plane forces across the steps. **Figure 9-1** shows reinforcement for such a step, in addition to the reinforcement projecting above the supporting inverted tee beam that is used to achieve composite behavior of the beam with the topping slab. The top of the beam is often recessed below the top of the double tee to provide additional space for longitudinal reinforcement and to allow for the development of the projecting ties above the beam. Even when the top of the beam is recessed 1½ inches below the top of double tee, and there is an additional 2-inch wash for a pour strip over the nominal topping slab thickness (2 to 3 inches) in a parking garage, it may not be possible to achieve the full tension development length for standard hooks for the projecting stirrups when the top cover is considered. The use of less than full l_d in this case is not uncommon, but has been successful by the inclusion of longitudinal reinforcement within the stirrup corners. The height of crowns in the pour strips may be limited by local floor slopes or by headroom limits to the beam above.

Cover requirements must also be considered for the topping slab thickness when the reinforcement includes crossing bars. Although welded-wire reinforcement has been a popular material for topping slab reinforcement, ACI 318-14 places limits. Section 18.12.7.1 requires the spacing of wires parallel to the joints between precast members to be greater than or equal to 10 inches for structures assigned to SDC D, E, or F. The intent of this requirement is to provide sufficient length between the anchored points of the wire to accommodate the strain demand of joint opening. A check of the strain demand at joints due only to temperature changes indicates that 10 inches may not be sufficient to avoid wire yielding. This has led to a trend to use deformed bars as topping slab reinforcement, and the dimensions of the crossing bars added to cover requirements may lead to thicker topping slabs.

Figure 9-1. Reinforcement placement in progress for topping slab diaphragm with step in floor elevation.

With pour strips in parking garages, the use of the thickness of the perimeter wash to provide for the cover needed for the chord reinforcement may be limited by the interference of columns that are inboard of the perimeter spandrel beams. **Figure 9-2** shows the plan view of a pour strip passing a column so that the 3-feet wide strip is effectively reduced to one foot at the column, where the thickness of the concrete in the strip can be no more than to the top of the adjacent untopped flange. Since there is also tolerance for alignment between the adjacent flanges, variations in the precast surface under the pour strip may further reduce the cover that is provided.

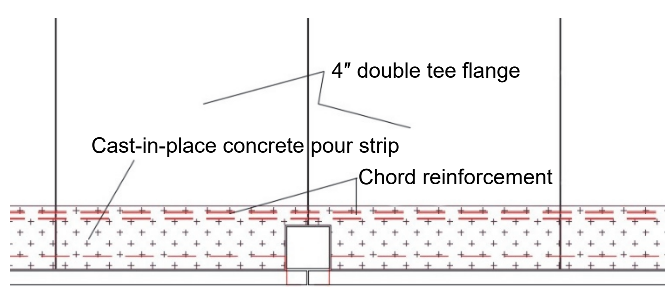

Figure 9-2. Pour strip at inboard exterior column.

Large openings in the diaphragm must accommodate the transfer of forces from the interior of the diaphragm to the seismic force-resisting system. These openings often alter the direct load paths and create re-entrant corners subject to higher concentrated stresses. An advantage of having a finite element analysis solution is that these discontinuities can be evaluated directly.

9.2 Welding Specifications

Welding of reinforcement is governed by AWS D1.4, *Structural Welding Code – Reinforcing Steel* (AWS 2015a). This document governs the fabrication of embedded plates or parts that use reinforcement for anchorage. Procedures are required to be qualified. Most of the field connections made by welding, however, are not made by welding reinforcement but by welding plates between the embedded steel plates; these welded connections are governed by AWS D1.1, Structural *Welding Code – Steel* (AWS, 2015b). Many of these welds are made using prequalified welding procedures. For welds that are not qualified under AWS D1.1, it is required that they be subject to the protocol for qualification that would enable them to be used.

Many of the typical flange-to-flange connections using commercial embedded parts requiring welding of steel bar slugs between vertical or near vertical surfaces are not pre-qualified. Since connections used as diaphragm connections are subject to qualification to establish strength and stiffness under cyclic loading, these connections may also need to have their welding procedures qualified in accordance with the applicable AWS provisions.

9.3 Camber Tolerances

Camber is the effect of the application of pretensioning force below the neutral axis of a precast component, which causes upward bending. This effect varies between components. The variation can occur due to variations in casting or storage. Members cast on Friday and stripped (detensioned) on Monday will likely have less camber than the member cast on Monday and stripped on Tuesday. Pieces stored on top of a stack of products and exposed to the sun and shading the pieces below will likely have higher camber than those lower pieces. Shorter pieces set next to longer pieces will have differences in camber between the flanges. Tolerances are specified in ACI ITG-7-09, *Specification for Tolerances for Precast Concrete* (ACI 2009). For camber of double tees, the tolerance is ± ⅛ inch per 10 feet, ± ¾ inch total. This allows for difference between adjacent flanges as great as 1½ inches, but this variation is not permitted in final alignment. It is not uncommon for actual field conditions to exceed the tolerance limits, requiring an engineering evaluation for the acceptance of the deviation. For final alignment of a floor using untopped flanges, the total difference is limited to ¼ inch. Construction methods must allow for weighting or jacking to bring adjacent flanges to this limit. The design of the connections must be such as to provide the forces needed to hold this final alignment. With time, the force imposed by alignment tends to dissipate; so that after several months, released connections will not see a rebound in the surface separation.

9.4 Special Inspections

Any welding used in the erection of precast concrete structures is typically subject to special inspection. Where the welding joins qualified connections for high-deformability performance, the standard requires such welding to be subject to continuous special inspection. If high-deformability connections are used in lower seismic design categories to warrant the use of higher diaphragm force reduction factors, these connections should also be subject to continuous special inspection.

9.5 Erection

Precast construction does not proceed level by level; it is "up and out" construction. This sequence is controlled by site access and the reach of the crane. Most precast structures are set by a crane that operates within the footprint of the structure, causing the crane to move away from the leading edge of erection as construction proceeds. The effects of this sequence of construction need to be considered in defining the sequence of connections and the requirements for temporary bracing. It is important to consider this sequence for its effects on the stability of the completed structure. Generally, full completion of welded connections must not lag far behind the leading edge. The placement of cast-in-place concrete for pour strips or topping slabs, however, may be delayed until practically all of the precast concrete has been set. The overall stiffness of the mechanical connections must, in conjunction with temporary bracing, be sufficient to hold the structure within dimensional tolerances until the final connections and cast-in-place concrete are placed.

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11. Notations and Abbreviations

Notations

A_{cv} gross area of concrete section bounded by diaphragm slab thickness t and total depth h measured in the direction of shear force considered

A_s area of nonprestressed longitudinal tension reinforcement

b width of compression face of member

C_{px} design acceleration coefficient

d distance from extreme compression fiber to centroid of longitudinal tension reinforcement

f_c' specified compressive strength of concrete

f_y specified yield strength of reinforcement

I_e importance factor

l_d development length in tension

M_n nominal flexural strength at section

M_u factored bending moment at section

N_n nominal strength in tension

N_u factored tension normal to cross section

R_s diaphragm design force reduction factor

S_{DS} design, 5 percent damped, spectral response acceleration parameter at short periods

V shear force on section; also total design lateral force or shear at the base

V_n nominal shear strength

V_u factored shear force at section

V_x shear at level x

W effective seismic weight

w_x portion of W that is tributary to level x

z_s modal contribution coefficient modifier dependent on seismic force-resisting system

Γ_{m1} first mode contribution factor

Γ_{m2} higher mode contribution factor

λ modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength

μ coefficient of friction

ϕ_f strength reduction factor for flexure and tension

ϕ_v strength reduction factor for shear

ρ redundancy factor based on the extent of structural redundancy present in a building

Ω_0 overstrength factor

Ω_v diaphragm shear overstrength factor

Abbreviations

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	ASTM International
ATC	Applied Technology Council
BDO	Basic Design Option
BIM	Building Information Modeling
DT	Double tee
DSDL	Diaphragm Seismic Design Level
DSDM	Development of Seismic Design Methodology
EDO	Elastic Design Option
EEG	Earthquake Engineering Group
HC	Hollow core
HDE	High Deformability Elements
IBC	International Building Code
ICC	International Code Council
LDE	Low Deformability Elements
MCE	Maximum Considered Earthquake
MDE	Moderate Deformability Elements
NEHRP	National Earthquake Hazards Reduction Program
NIST	National Institute of Standards and Technology
PCI	Precast/Prestressed Concrete Institute
RDO	Reduced Design Option
SDC	Seismic Design Category
SEAOC	Structural Engineers Association of California
SEI	Structural Engineering Institute
SRFS	seismic force-resisting system
TAC	Technical Activities Council
UBC	Uniform Building Code
WWR	welded wire reinforcement

12. Credits

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