# Chapter 8 - Building Types S1/S1A: Steel Moment Frames

## 8.1 Description of the Model Building Type

Building Type **S1** consists of an essentially complete frame assembly of steel beams and columns. Lateral forces are resisted by moment frames that develop stiffness through rigid connections of the beam and column created by angles, plates, and bolts, and/or by welding. Moment frames may be developed on all framing lines or only in selected bays. It is significant that no structural walls or steel braces are provided. Floors are cast-in-place concrete slabs or concrete fill over metal deck. These buildings are used for a wide variety of occupancies such as offices, hospitals, laboratories, and academic and government buildings. Figure 8.1-1 shows an example of this building type.

Building Type **S1A** is similar but has floors and roofs that act as flexible diaphragms such as wood or untopped metal deck. One family of these buildings is older warehouse or industrial buildings, while another more recent use is for small office or commercial buildings in which the fire rating of concrete floors is not needed.

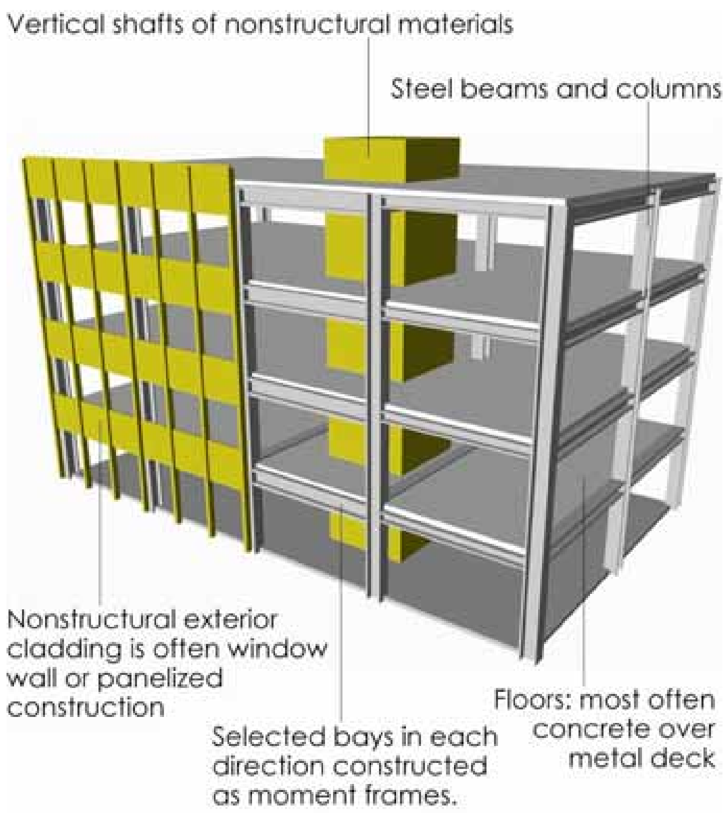



**Figure 8.1-1: Building Type S1: Steel Moment Frames**

### *Variations Within the Building Type*

The use of structural steel in building construction started in the last decades of the 19th century. Lateral load resistance was initially provided by masonry infill. Additional resistance may have been provided by the encasement of the steel members in concrete, though these encased members were not designed with composite considerations. In the 1920s, the use of riveted connections introduced steel moment frames. Beam flanges and webs were joined to the columns by structural shapes, most commonly T-sections. Rivets had low strength and ductility, which limited the overall capacity of the frames. In the 1960s, high strength bolts replaced the rivets. Bolts were faster to install and permitted larger clamping forces, increasing the rigidity of the frames. Connections became smaller when cover plates bolted to the beam flanges and welded to the columns replaced T-sections in the 1950s. By the 1960s, beam flanges and webs were welded directly to the columns to create fully restrained connections, initiating the welded steel moment frame (WSMF) construction era. Shear tabs bolted to the beam webs and welded to the columns later replaced welded beam webs. These welded-flange and bolted-web connections were used extensively in the 1970s and 1980s and are now known as pre-Northridge connections. After the 1994 Northridge earthquake, it was concluded that these connections did not provide enough plastic rotational capacity for most seismic applications (FEMA, 2000c). Several types of connections have been developed and tested to address the flaws in the pre-Northridge connections. The newer connections are designed to develop the full moment capacities of the beams and provide large inelastic rotation capacity.

### *Floor and Roof Diaphragms*

Diaphragms associated with this building type could be either rigid or flexible. The typical rigid diaphragm found in modern buildings consists of structural concrete fill on metal deck. Diaphragm forces transfer to the frames through shear studs welded to the beams. Older steel buildings that were constructed before metal decks were commonly used may have concrete slabs or masonry arches that span between the beams. Flexible diaphragms include bare metal deck or metal deck with nonstructural fill. These are frequently used on roofs that support light gravity loads. Decks could be connected to the steel members with shear studs, puddle welds, screws, or shot pins. The steel members also act as chords and collectors for the diaphragm.

### *Foundations*

There is no typical foundation for this building type. Foundations can be of any type including spread footings, mat footings, and piles, depending on the characteristics of the building, the lateral forces, and the site soil. Spread footings are used when lateral forces are not very high and a firm soil exists. For larger forces and/or poor soil conditions, a mat footing below the entire structure is commonly used. Pile foundations are used when lateral forces are extremely large or poor soil is encountered. The piles can be either driven or cast-in-place. Vertical forces are distributed to the underlying soil through a combination of skin friction between the pile and soil and/or direct bearing at the end of the pile; lateral forces are resisted primarily through passive pressure on the vertical surfaces of the pile cap and piles.

## 8.2 Seismic Response Characteristics

Steel moment frame buildings are generally flexible, but subject to large interstory drifts. The ductility of these buildings is achieved through yielding and plastic hinging of beams and/or shear yielding of column panel zones at beam-column connections. This inelastic behavior allows moment frames to sustain many cycles of loading and load reversals. Historically, it was believed that large plastic rotations could be developed without significant strength degradation. Up until the Northridge earthquake, the performance of steel moment frame buildings was also believed to far exceed that of masonry and concrete buildings based on observations from previous earthquakes.

The Northridge earthquake exposed severe deficiencies in WSMF connections. A significant number of the frames inspected after the earthquake exhibited visible cracking in the beam flange-to-column flange welds. In a few rare cases, the flanges completely fractured and the damage extended into either the shear tab or the column panel zone. Newer buildings, which relied on deeper beams with thick flanges and less redundancy, were discovered to be even more susceptible to this type of damage. In retrospect, a review of data from past earthquakes indicates that WSMF buildings rarely received close inspection following the event, and often these buildings were overlooked due to the more obvious damage in other types of structures (FEMA, 2000d). This oversight also applies to older steel buildings with riveted or bolted connections. These older buildings, though designed as moment frames, may have suffered limited damage due to the interaction of the steel frames with infill walls and concrete cores. Lastly, if steel damage was discovered after an earthquake, engineers often attributed this to poor construction quality (FEMA, 2000d).

## 8.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 8.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Selected deficiencies are further discussed below by category.

### *Global Strength*

The lack of global strength is caused by insufficient frame strength, resulting in excessive demands on the existing frames. Yielding or fracturing of the beams, columns, and/or connections could lead to excessive drifts. As a result, the building could be deemed irreparable after an eathquake.

### *Global Stiffness*

Moment frames are much more flexible than other types of lateral force-resisting systems. Their flexibility could lead to excessive building drifts and interstory drifts. This is likely to cause structural damage to the connections and nonstructural damage to the partitions and cladding. Additional concerns include P- effects and pounding with adjacent buildings. Common rehabilitation measures include strengthening the existing frames or providing new vertical lateral force-resisting elements.

### *Configuration*

Soft story conditions occur when stiffness from one floor to the next changes abruptly. This is common at ground floors of commercial and office buildings with tall first stories. It could also occur at mid-heights of five-story to fifteen-story story tall buildings that have not been designed for higher mode effects and near field motions.

**Table 8.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S1/S1A Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient frame strength | Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | Strengthen beams [8.4.3], columns [8.4.3], and/or connections [8.4.6], [8.4.9] | | Seismic isolation [24.3]<br>Supplemental damping [24.4] | |
| Global Stiffness | Excessive drift | Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | Strengthen beams [8.4.3], columns [8.4.3], and/or connections [8.4.6], [8.4.9] | | Supplemental damping [24.4] | |
| Configuration | Soft story | Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | | | | |
| | Re-entrant corner | Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Collector [8.4.4] | Enhance detailing [8.4.3], [8.4.4] | | | |
| Load Path | Missing collector | Add collector [8.4.4] | | | | |
| | Inadequate shear, flexural, and uplift anchorage to foundation | | Embed column into a pedestal bonded to other existing foundation elements [8.4.5] | Provide steel shear lugs or anchor bolts from base plate to foundation [8.4.5] | | |

| Table 8.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S1/S1A Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Load Path (continued) | Inadequate out-of-plane anchorage at walls connected to diaphragm | | | Tension anchors [16.4.1] | | |
| Component Detailing | Inadequate capacity of beams, columns, and/or connections | | Enhance beam-column connection [8.4.6]<br>Add cover plates or box members [8.4.3]<br>Provide gusset plates or knee braces [9.4.1]<br>Encase columns in concrete | | | |
| | Inadequate capacity of panel zone | | Provide welded continuity plates [8.4.6]<br>Provide welded stiffener or doubler plates [8.4.6] | | | |
| | Inadequate capacity of horizontal steel bracing | Provide additional secondary bracing [9.4.2] | Strengthen bracing elements<br>Reduce unbraced lengths | Strengthen connections | | |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Collectors to distribute forces [8.4.3], [8.4.4]<br>Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | Concrete topping slab overlay<br>Wood structural panel overlay at flexible diaphragms [22.2.1]<br>Strengthen chords [8.4.3], [8.4.4], and [22.2.2] | Add nails at flexible diaphragms [22.2.1] | | |
| | Inadequate shear transfer to frames | | | Provide additional shear studs, anchors, or welds [22.2.7] | | |

**Table 8.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S1/S1A Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Diaphragms (continued) | Inadequate chord capacity | Add steel members or reinforcement [8.4.3], [8.4.4] | | | | |
| | Excessive stresses at openings and irregularities | Add reinforcement [8.4.3]<br>Provide drags into surrounding diaphragm [8.4.4] | | | | Infill opening [22.2.4], [22.2.6] |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

### Load Path

Load path deficiencies in steel moment frame buildings include inadequate collectors and frame anchorage to foundations. In Type **S1** buildings, seismic forces transfer from the diaphragm to the frame through shear studs welded to collectors or directly to the frame beams. The collectors or the connections to the frame may be too weak and insufficient to transfer these forces. Connections from columns to a base plate or pile cap have to resist shear, flexural and potentially uplift forces. Connections that cannot develop these frame forces do not allow the frame to develop its full capacity.

### Component Detailing

The most common detailing deficiencies in steel moment frames are related to beam-column connections. Pre-Northridge welded moment connections consist of complete penetration flange welds and bolted or welded shear tabs. These connections were previously thought to be ductile but were found to fracture at small plastic rotations or even under elastic loads. Rehabilitation techniques primarily focus on forcing the yielding and plastic hinging to occur away from the joint to reduce stresses on the welds. This can be achieved with the use of reduced beam sections (RBS) or strengthening the section of a beam adjacent to the beam-column joint. These techniques are presented in Section 8.4.6 and discussed in detail in *AISC Design Guide 12* (Gross et al., 1999) and FEMA 351 (FEMA, 2000b). Before welded moment connections became common, connections were either riveted or bolted. These types of connections are prone to net section fracture.

Panel zones were previously thought to provide excellent ductility and strain hardening and were given increased predicted shear strength in the 1988 Uniform Building Code (FEMA, 2000c). The increased strength also meant larger inelastic demands in the panel zones. As a result, panel zones are subjected to yielding or buckling before adjacent members fully develop their capacities. Studies performed after the Northridge earthquake found that large panel zone deformations could increase the potential for connection failures (FEMA, 2000c).

Welded column splices are vulnerable to fracture when subjected to large tensile loads (FEMA, 2000b), since they generally use partial penetration groove welds and thus, are not designed for the full capacity of the smaller column.

### Diaphragm Deficiencies

Common diaphragm deficiencies include insufficient in-plane shear strengths, inadequate chords, and excessive stresses at openings. Causes for these deficiencies could be due to lack of slab or fill thickness, lack of reinforcing steel in the slabs, insufficient connections to chord elements, and poor detailing at openings. See Chapter 22 for common rehabilitation techniques.

### Foundation Deficiencies

Foundations that are inadequate do not develop the full capacity of the lateral force-resisting system. Their deficiencies result from insufficient strengths and sizes of footings, grade beams, pile caps, and piles. See Chapter 23 for common rehabilitation techniques.

## 8.4 Detailed Description of Techniques Primarily Associated with This Building Type

### 8.4.1 Add Steel Braced Frame (Connected to an Existing Steel Frame)

***Deficiency Addressed by Rehabilitation Technique***

Moment frame buildings that are insufficient to resist lateral forces or too flexible to control building drifts can be converted into braced frame buildings.

***Description of the Rehabilitation Technique***

The seismic performance of a building may be improved by adding braces to existing welded or riveted steel moment frames. Braces can be added without substantially increasing the mass of the building. Various concentrically braced frame (CBF) configurations should be considered, though some tend to perform much better than others in earthquakes. In addition, systems that meet the provisions for special concentrically braced frames (SCBF) are expected to exhibit stable and ductile behavior in large earthquakes. See Chapter 9 for a discussion of different CBF configurations and their behaviors. Moment frames are not commonly converted to eccentrically braced frames (EBF) due to complicated design and detailing issues that would be encountered. Different brace types can be used, including W-shapes, hollow structural sections (HSS), steel pipes, double angles, double channels, double HSS, and buckling-restrained braces.

***Design Considerations***

Adding braced frames to a moment frame building increases its stiffness considerably. The upgraded structure should be evaluated for higher lateral and overturning forces accordingly. The primary system performance issues are those associated with CBF systems, in which the most vulnerable elements are the braces and their end connections. Thorough knowledge of the existing material behaviors and strengths are necessary to ensure that new and existing elements to interact in the desired manner. Other design issues include the following:

*Research basis:* No references directly addressing the addition of steel braced frames to moment frame buildings have been identified.

*Brace locations:* Preference of brace locations should be given to existing moment frame bays to utilize the strengths of existing members, connections, and foundations. If this is not possible and braces are added at other locations, the existing moment frames should be considered when forces are distributed to the lateral force-resisting system. If the building drifts are large enough, deficiencies in the moment frames may still require mitigation whether the frames are included in the new lateral force-resisting system or not.

*Brace selection:* Use compact and non-slender sections whenever possible to avoid premature fracturing or buckling of the braces during post-yield behavior. Issues related to conventional structural shapes are discussed in Chapter 9. Two particular brace types not common in older braced frame buildings are double HSS section and buckling-restrained braces. Double HSS sections can be used in configurations similar to double angles or channels (Lee and Goel, 1990). They provide reduced fit-up issues and smaller width-to-thickness ratios compared to a single HSS, resulting in increased energy dissipation capacity. The other type of brace, used in a buckling-restrained braced frame (BRBF), is typically used in new buildings but has also been used successfully in new BRBF systems in existing buildings. One example of a brace used in a BRBF consists of a steel core inside a casing, which consists of a hollow structural section (HSS) infilled with concrete grout. Proprietary materials separate the steel core and concrete to prohibit bonding between the two materials. There are other buckling-restrained braces that do not use grout or additional separating agents between the steel and grout. The main advantage of these braces is the ability of the casing to restrain the buckling of the steel core without providing any additional axial force resistance beyond the capacity of the steel core. Provisions for new building BRBF design are included in the *NEHRP Recommended Provisions* (FEMA, 2003) and the *AISC Seismic Provisions for Structural Steel Buildings* (AISC, 2005b).

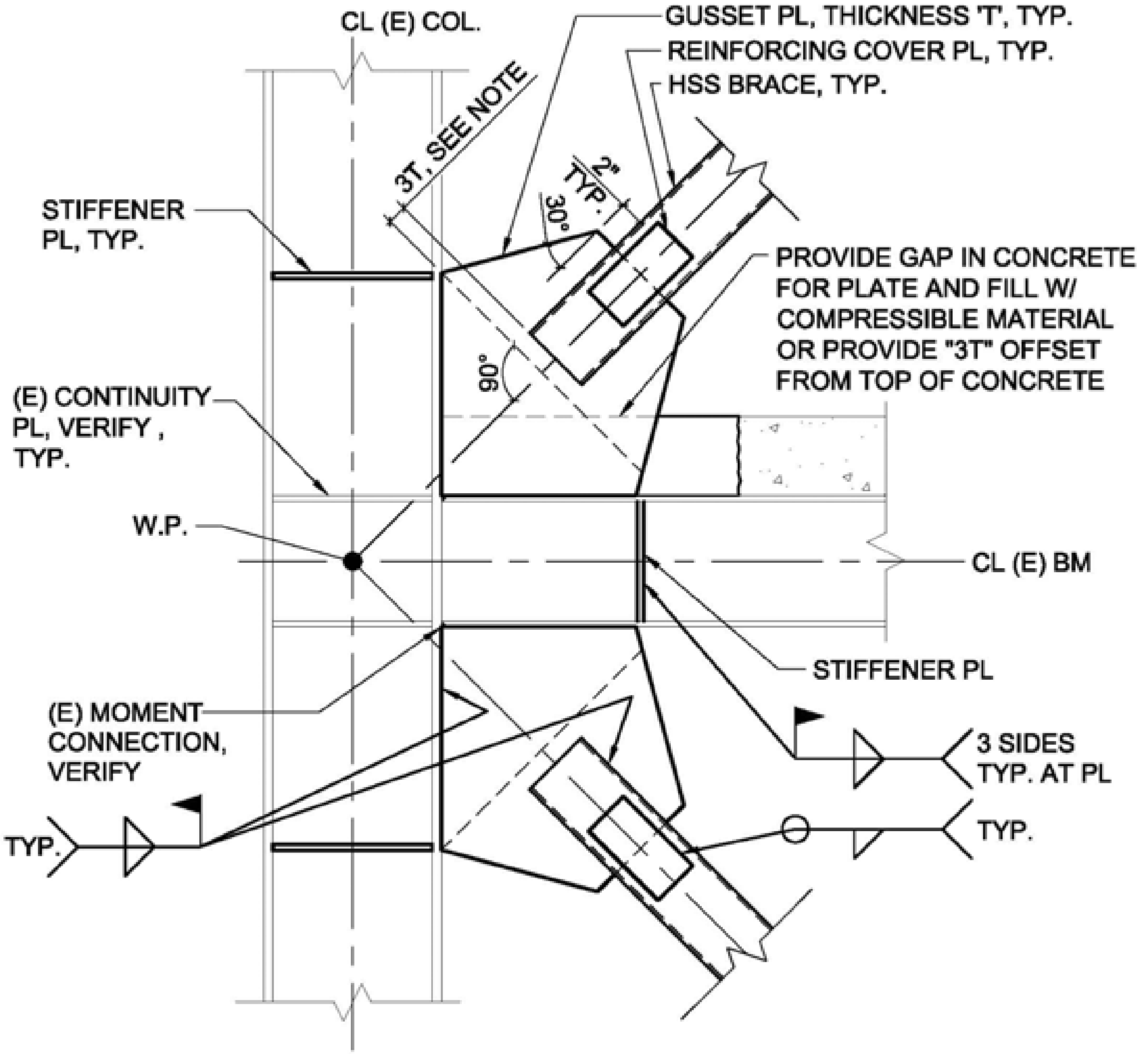
*Nonstructural issues:* The addition of braces to an existing structure changes the architectural character of the building. Braces in exterior frames will be visible in buildings with clear glazing. At interior bays, braces have to be configured to avoid obstruction of existing corridors, doorways, and other building systems. Braces are also commonly exposed and incorporated into the interior architecture. For this particular option, note that gusset plates designed in accordance with the *AISC Seismic Provisions* for SCBF can be fairly large and should be discussed with the architect and tenants. If the braced frames are hidden in partition walls, the architect should be aware that these walls will be thicker than typical walls. Beams that are increased in size and new collectors may affect nonstructural components by reducing clear floor heights. These components typically include suspended ceilings, pipes, conduits, and ducts. Coordination with the architect and other trades should not be overlooked or underestimated.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Connections:* Gusset plates provide greater tolerances for the installation of braces than directly attaching the braces to the frame members. It may be impossible to completely eliminate welding at a braced frame connection in a seismic rehabilitation. Continuity and doubler plates on columns, stiffener plates on beams, brace-to-gusset plate connections, and most gusset plate-to-frame member connections all require welding. A typical fully welded connection appropriate for use in SCBF is shown in Figure 8.4.1-1. If the brace capacity is governed by out-of-plane buckling, stable post-buckling behavior can be achieved by allowing the gusset plate to develop restraint-free plastic rotations. AISC recommends an offset of two times the plate thickness along the brace centerline, measured from the end of the brace to a line perpendicular to the nearest point on the gusset plate constrained from out-of-plane rotation. If gaps are provided between the gusset plate and concrete fill, then only the beam or column can act as constraints, as is the case in Figure 8.4.1-1. On the other hand, if concrete is placed directly against the gusset plate, then the slab can also act as a constraint.

For low to moderate seismic applications, Figure 8.4.1-2 shows a more compact connection. An example of a W-shape used as a brace and welded directly to the beam and column is shown in Figure 8.4.1-3. The two latter connections do not allow for restraint-free plastic rotations out-of-plane and should be used primarily in situations where in-plane buckling of the braces govern or




*Note:*
*AISC recommends 2T to allow for restraint-free plastic rotations.*
*3T is shown here to accomodate overcutting of HSS slots.*

**Figure 8.4.1-1: HSS Brace at Existing Beam-Column Connection in SCBF**

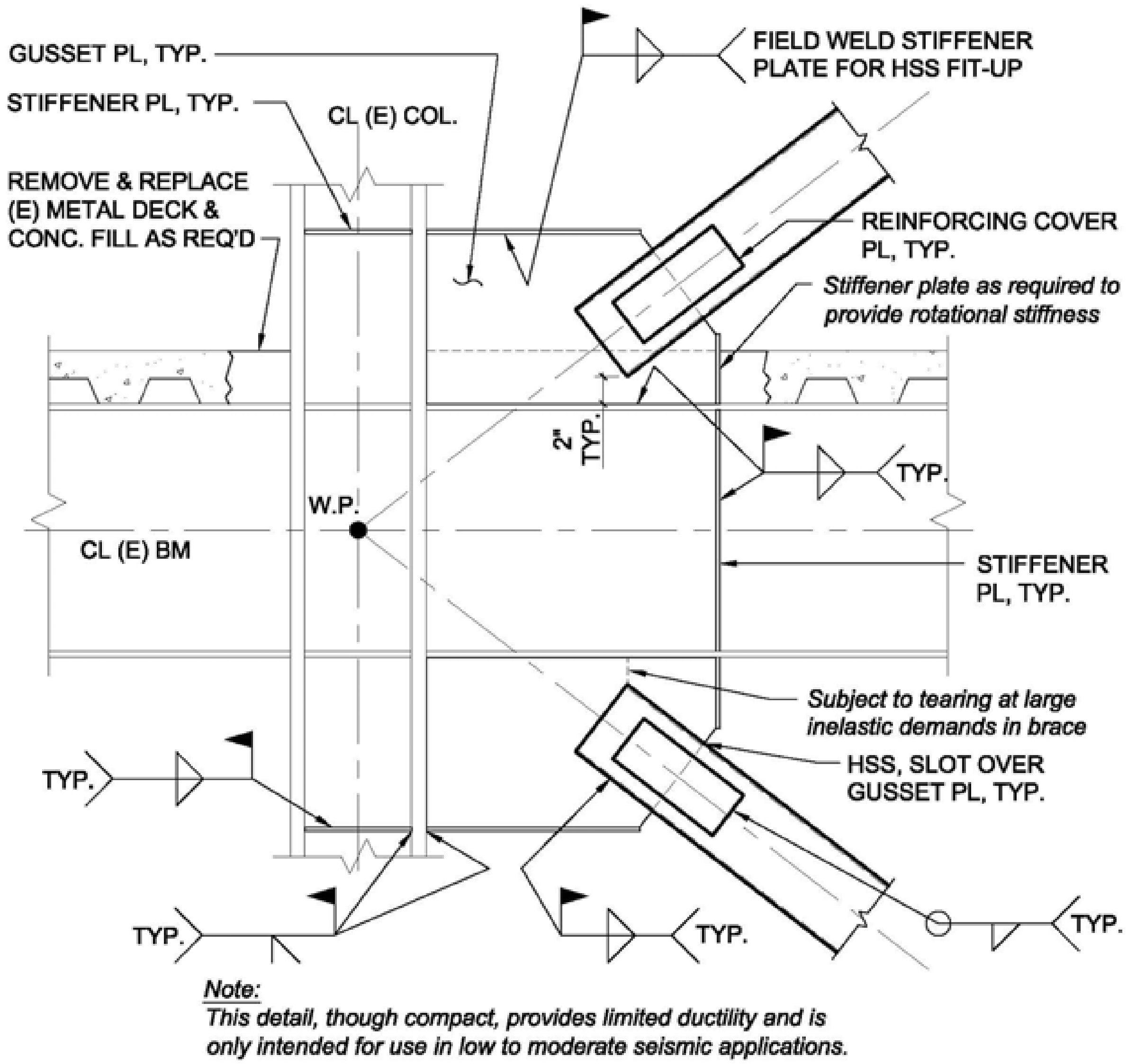



**Figure 8.4.1-2: HSS Brace at Existing Beam-Column Connection in Ordinary CBF**

ductility demands are low. In addition, the connection shown in Figure 8.4.1-3 requires extensive field welding and could present fit-up issues if field dimensions are not verified on a case-by-case basis.

Buckling-restrained braces are typically bolted to gusset plates. Their bolt configurations at these connections allow for very small tolerances. Double angle and double channel braces can be easily bolted to a gusset plate but space restrictions may limit the number of bolts and subsequently, the strength of the connection.

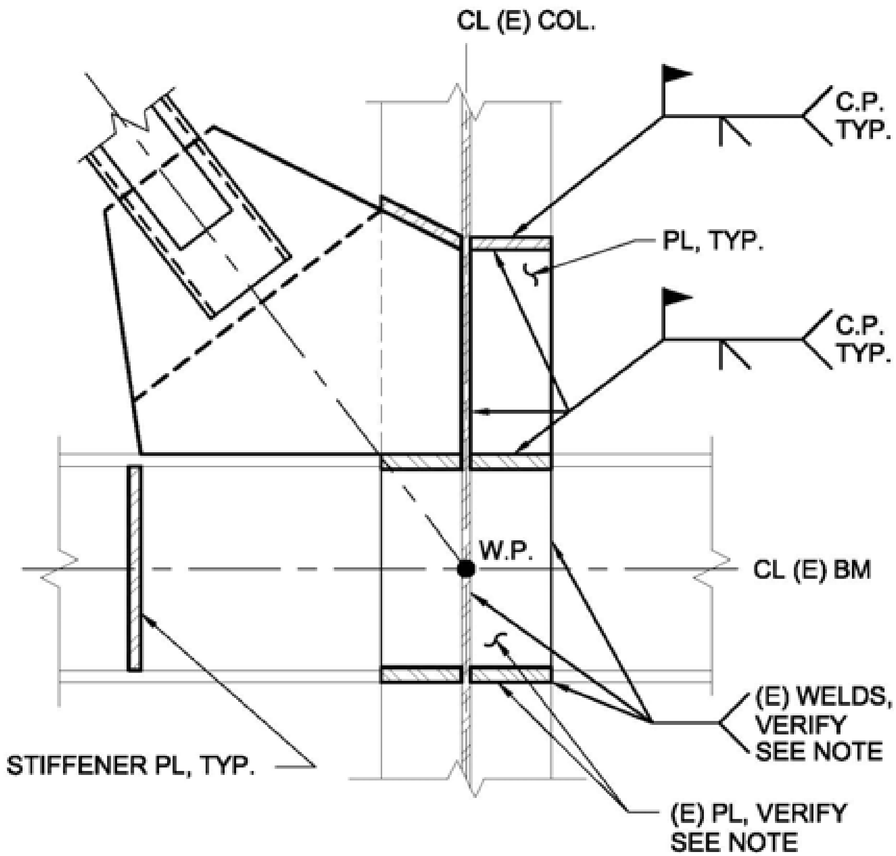



**Figure 8.4.1-3: HSS Braced at Existing Beam-Column Connection**

It may not always be practical or possible to locate a work point at the intersection of the centerlines of the beam and column, e.g. deep beams. This may deemed to be acceptable if the eccentricity is included in the connection design.

*Built-up brace members:* While double angles, double channels, and double HSS offer advantages for installation, more stringent criteria apply to these members when used in SCBF. These include stitch spacing, member compactness, and strength of the stitches (AISC, 2005b).

*Reinforcing cover plates:* HSS and pipe braces are subject to net section fracture at the gusset plate slots (Uriz and Mahin, 2004, and Yang and Mahin, 2005). This brittle failure mode can be eliminated by adding reinforcing cover plates to the sides of the HSS without the slots, as shown in Figure 8.4.1-1 and Figure 8.4.1-2. For pipes, the reinforcing plates can be oriented at right angles to the pipe and appear like stiffeners.

### *Cost/Disruption*

This system could be cost-effective when compared to other alternatives for upgrading steel moment frame buildings. Costs will be less when existing moment frames are converted into braced frames to take advantage of the existing strength and stiffness of the frame members, connections, and foundations. Designs that are simple and details that are not overly complicated will also minimize costs.

Costs can also be reduced if disruption is minimal during construction. Installing braces at the perimeter frames reduces logistical issues associated with working in confined spaces and temporary removal of the nonstructural elements. Noise associated with this type of work is loud and disturbing to the tenants if the building is occupied while the work is being performed.

### *Construction Considerations*

The engineer's involvement during the construction phase is critical during a seismic rehabilitation. The design of the retrofit scheme must not neglect the construction phase and should consider these issues at a minimum:

*Welding issues:* A work environment in which the welder can perform quality welds is critical. This includes an environment with adequate space to properly operate welding equipment, adequate lighting, and a stable work platform for overhead welds, all while ensuring worker health and safety. Additional considerations include venting of welding fumes and fire protection.

*Removal of existing nonstructural elements:* Exterior cladding and interior partitions must often be removed to deliver and install the braces to their final locations. Connection modifications at the roof level may warrant the removal of roofing and waterproofing. Installation of the connection to the underside of beams, column continuity and doubler plates, and beam stiffeners, will affect ceilings, lights, and other mechanical/electrical/puming components. Almost all steel buildings will have some form of fireproofing around the members and connections. With older buildings, asbestos may be present in the fireproofing, which could require costly abatements to expose the members. Older buildings with items of historical significance may require additional coordination and effort to ensure that these items are removed and restored properly. Buildings constructed in the early 20$^{th}$ century commonly had steel members encased in concrete.

*Removal of existing structural elements:* Slabs and metal decks must be chipped and cut away to install connections. Slabs oriented perpendicular to the beams require temporary shoring. Temporary openings in the slab not only require shoring but also consideration for how the openings will be closed. Beams are rarely removed but if unavoidable, the slabs supported by the beams need shoring and the columns supporting the beams may require bracing.

*Construction loads:* Loads from construction activities vary and may be either temporary or permanent. Temporary loads could include the weight of construction equipment and patterned loading on perimeter framing when heavy cladding is temporarily removed. Permanent loads could be induced if connection stiffness is modified, such as converting a simple shear tab support into a fully rigid moment connection, and when members are temporarily removed, causing forces to redistribute.

***Proprietary Concerns***

Braces used in BRBF are proprietary. There are a limited number of manufacturers of the braces used in BRBF.

### 8.4.2 Add Concrete or Masonry Shear Wall (Connected to An Existing Steel Frame)

***Deficiency Addressed by Rehabilitation Technique***

Moment frames buildings that are insufficient to resist lateral forces or too flexible to control building drifts can be strengthened and stiffened by adding shear walls. The shear walls may be used alone as the new lateral force-resisting system or in conjunction with the moment frames.

***Description of the Rehabilitation Technique***

Shear walls can add considerable strength and stiffness to a structure. These walls could be considered as an alternative to adding braced frames if the existing beams and columns are incapable of resisting forces in the braced frame system. Concrete shear walls can be placed using conventional formwork or shotcrete can be used instead if skilled operators for placing shotcrete walls are available. Masonry shear walls, though typically weaker than their concrete counterparts, have the advantage of not requiring formwork when filling the cell cavities with grout.

***Design Considerations***

The addition of a shear wall to an existing steel frame forms a composite shear wall system. The horizontal shear forces are resisted by the wall elements and the vertical overturning forces are primarily resisted by the steel columns that become boundary elements. This system is almost always controlled by shear due to the substantial flexural strength provided by the steel column boundary elements. Though shear cracking and yielding of a wall is not as ductile as flexural hinging at the base of the wall, limiting the wall to small drifts prevents early loss of strength and stiffness degradation. Other design issues are as follows:

*Research basis:* No references directly addressing the addition of shear walls to moment frame buildings have been identified.

*Design forces:* The forces on the structure could increase significantly when this mitigation technique is employed due to the increased stiffness and mass of the walls. The entire structure should be reanalyzed, including all components that were previously determined to have sufficient capacity to resist the forces on the more flexible structure. In-plane forces due to the self-weight of the walls remain in the walls and do not increase demands on other elements. The out-of-plane forces, on the other hand, must be transferred through the diaphragms and collectors to other walls or frames. The design of the wall foundations depends on the magnitude of the forces and the soil properties. If overturning forces are greater than the available counteracting building mass or the soil is poor, some type of pile foundation will be necessary; otherwise, strip footings may suffice.

*Wall locations*: Preference of shear wall locations should be given to existing bays of moment frames to utilize the layout of the existing collectors and the strengths of the existing members and connections. If this is not possible and shear walls are added at other locations, the existing moment frames should be considered when forces are distributed to the lateral force-resisting system. Deficiencies in the moment frames may still require mitigation whether the frames are included in the lateral system or not.

*Continuous walls vs. frames with infill walls:* A continuous wall that encases the existing frame beams and columns, shown in Figures 8.4.2-1 and 8.4.2-2, respectively, provides the most robust and simple design. It may be desired to use a system that utilizes independent wall panels that appear similar to older style steel frame buildings with infill walls. The detailing and behavior would be much different from the older buildings though. The wall panels in each bay do not encase the beams and columns, as shown in Figures 8.4.2-3 and 8.4.2-4, respectively. One advantage of this system is that it may allow the elimination of shoring by not having to remove the metal deck though some slab concrete removal may still be required. Another advantage is that beams that have existing penetrations and the elements penetrating the beams would not be affected. The main drawback is that this system may be limited to use in buildings with lower seismic forces. The wall strengths would be limited by the number of studs that can be installed and the capacity of the beam webs. Another option is to encase the beams but not the columns.

*Coupled walls:* Beams in existing bays of moment frames that are located between new walls will behave like coupling beams, particularly if the bay is relatively short. These beams, subject to high shears and moments, require ductile detailing. The level of coupling between the walls is a function of the stiffness of the beams, which can be determined by a computer model of the system.

*Nonstructural issues:* The addition of walls to an existing structure changes the architectural character of the building. Walls at the exterior will be visible in buildings with clear glazing. At interior bays, walls have to be configured to avoid obstruction of existing corridors, doorways, and other building systems. Walls can be exposed and incorporated into the interior architecture or hidden in partition walls. The architect should be aware that these walls will be thicker than typical partition walls. Encased beams and new collectors affect nonstructural components by reducing clear floor heights. These components typically include suspended ceilings, pipes, conduits, and ducts. Encased columns reduce usable floor space. Coordination with the architect and other trades should not be overlooked or underestimated.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Connection to existing frame:* Positive connection should be provided to the existing frame to transfer seismic loads to the walls and to provide overturning resistance. Welded shear studs or reinforcing bars are typically used. In much older buildings where cast iron members still exist, it is difficult to weld to cast iron due to its high carbon content. Instead, holes for the reinforcing steel bars may have to be drilled in the members to rely on direct bearing for the load paths.

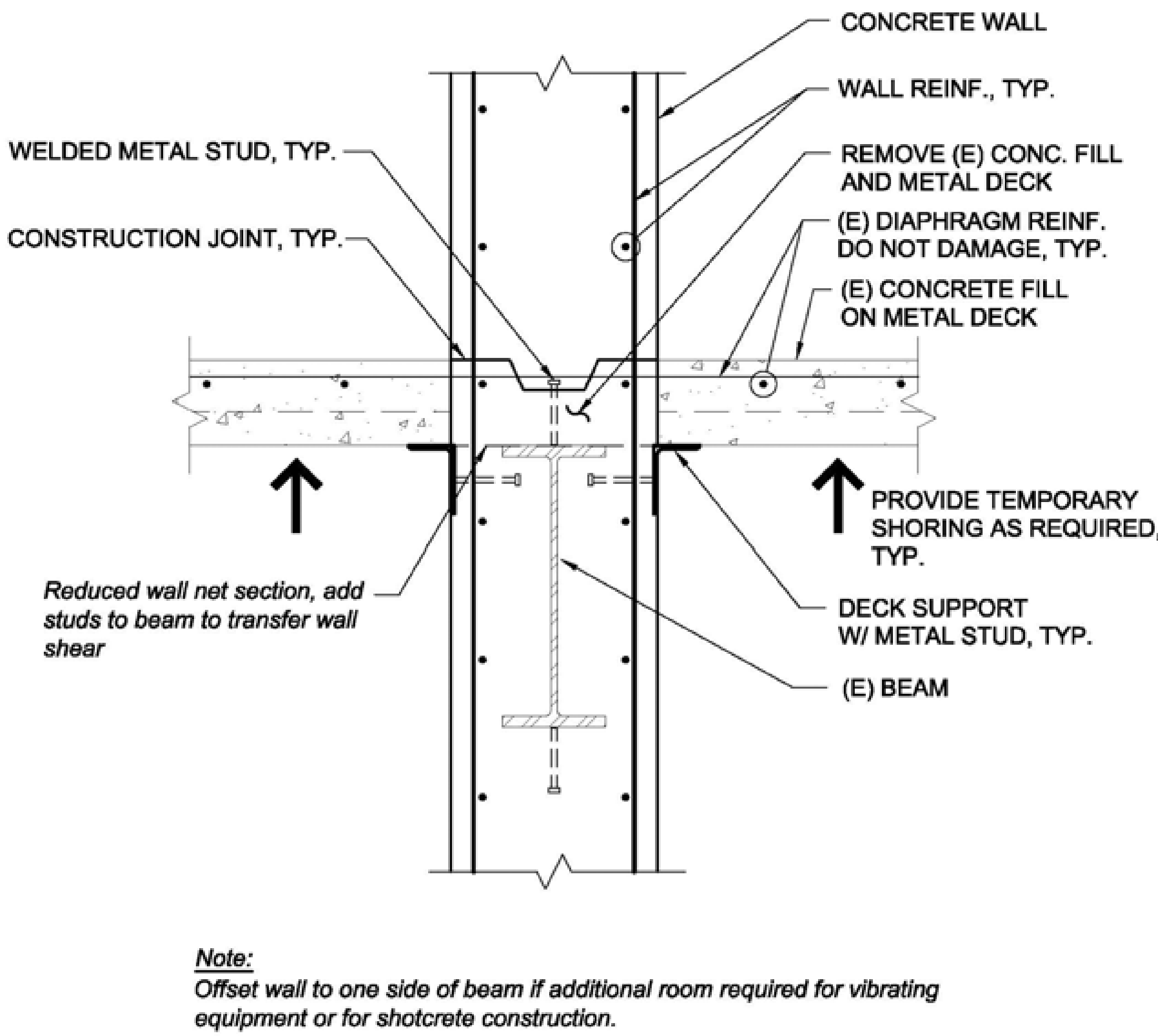



**Figure 8.4.2-1: Cast-in-Place Concrete Wall at Existing Beam**

*Wall reinforcing:* Reinforcing steel in the walls should meet all ACI 318 (ACI, 2005) requirements, including the seismic provisions. Columns with positive connections to the walls could be counted as boundary reinforcing steel limited by the strength of the shear connections. Congestion issues may be encountered at the ends of the walls due to the presence of the columns, confinement steel, and anchorage for horizontal wall reinforcing steel. It may be necessary to drill holes through the columns for the horizontal steel.

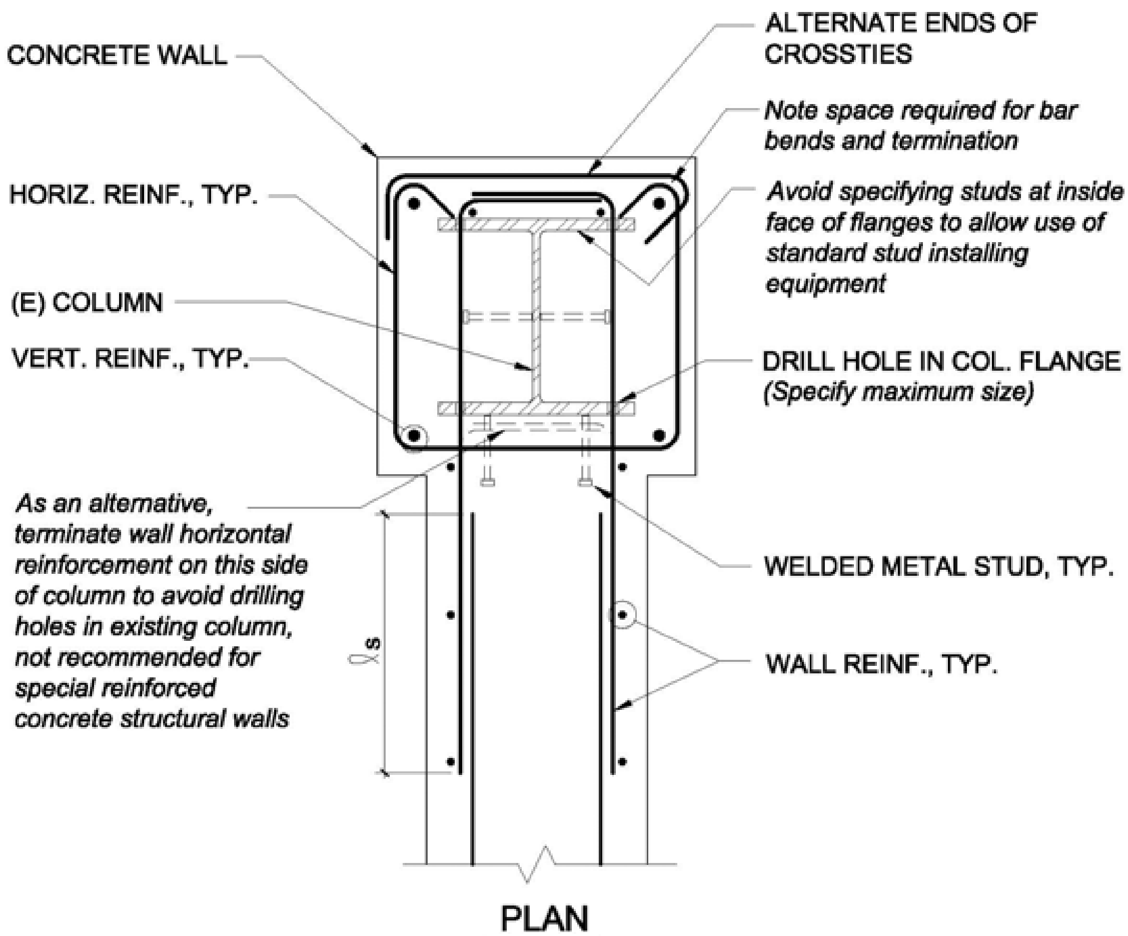



**Figure 8.4.2-2: Cast-in-Place Concrete Wall Encasing Existing Column**

*Mechanical couplers:* Consider using reinforcing steel couplers at areas of congestion. Note that diameters of the couplers could be as much as twice that of the bars and must meet ACI cover requirements for reinforcing steel.

*Effective wall thickness:* At beam flanges, the wall thickness could be reduced considerably, which compromises the shear strength of the wall. There are several ways to add the shear strength of the wall at these locations. Shear studs can be added to the beams to transfer some forces through the beam web. Thickness of the wall can be increased at the beam. Extra shear reinforcing steel can be placed in the wall provided that ACI limits for reinforcing steel are not exceeded.

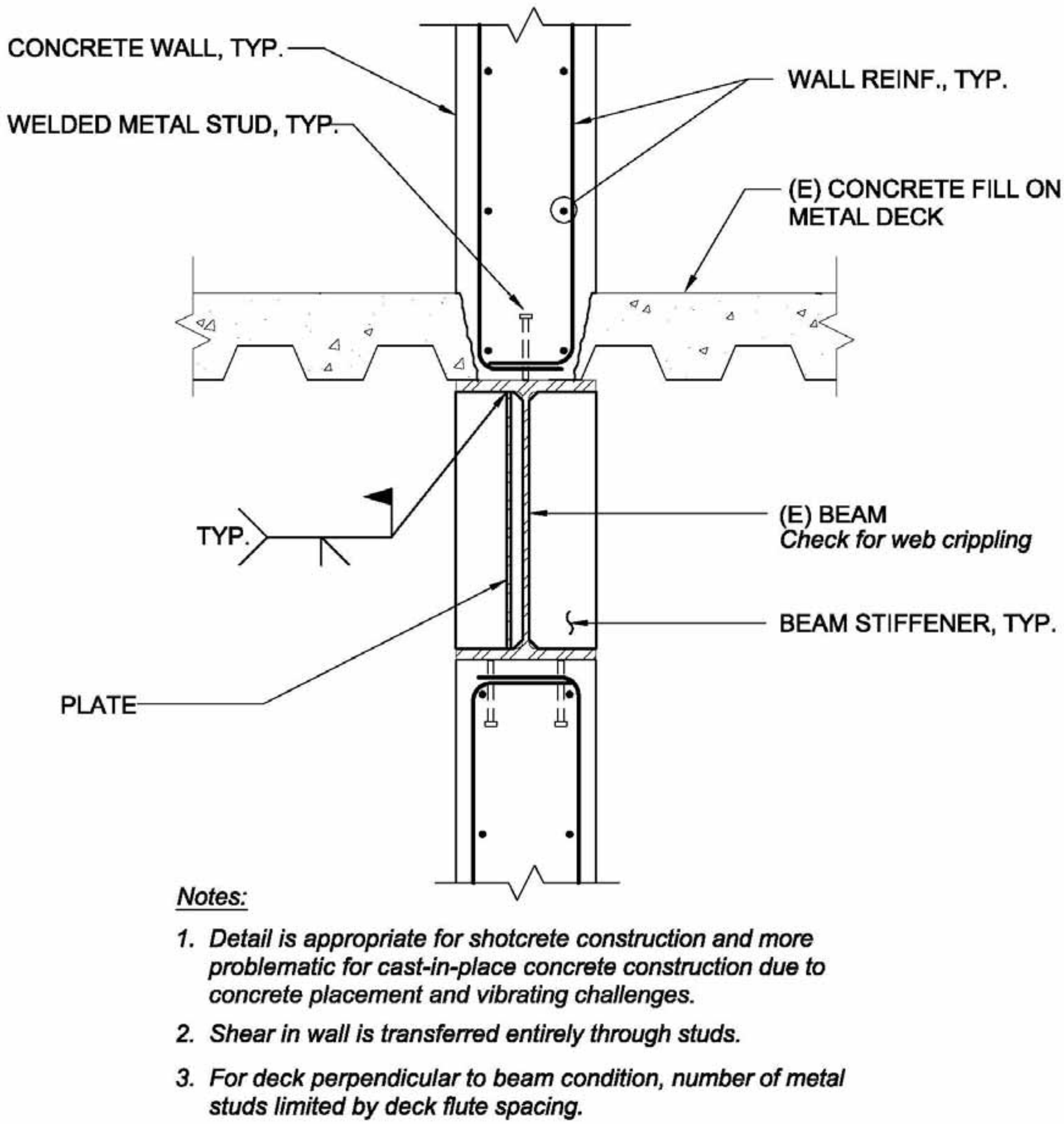



**Figure 8.4.2-3: Discontinuous Wall at Existing Beam**

*Wall connections at slabs:* To avoid damaging the slab reinforcing steel critical for transferring diaphragm forces to the shear walls, the concrete slab could be chipped away without damaging the slab steel. This still allows for a monolithic wall construction with a construction joint most likely located at the top of slab. If the shear forces in the walls are sufficiently low, the entire slab may be preserved and merely roughened at the construction joint. Holes can be drilled in the slab to allow vertical reinforcing steel to pass through.

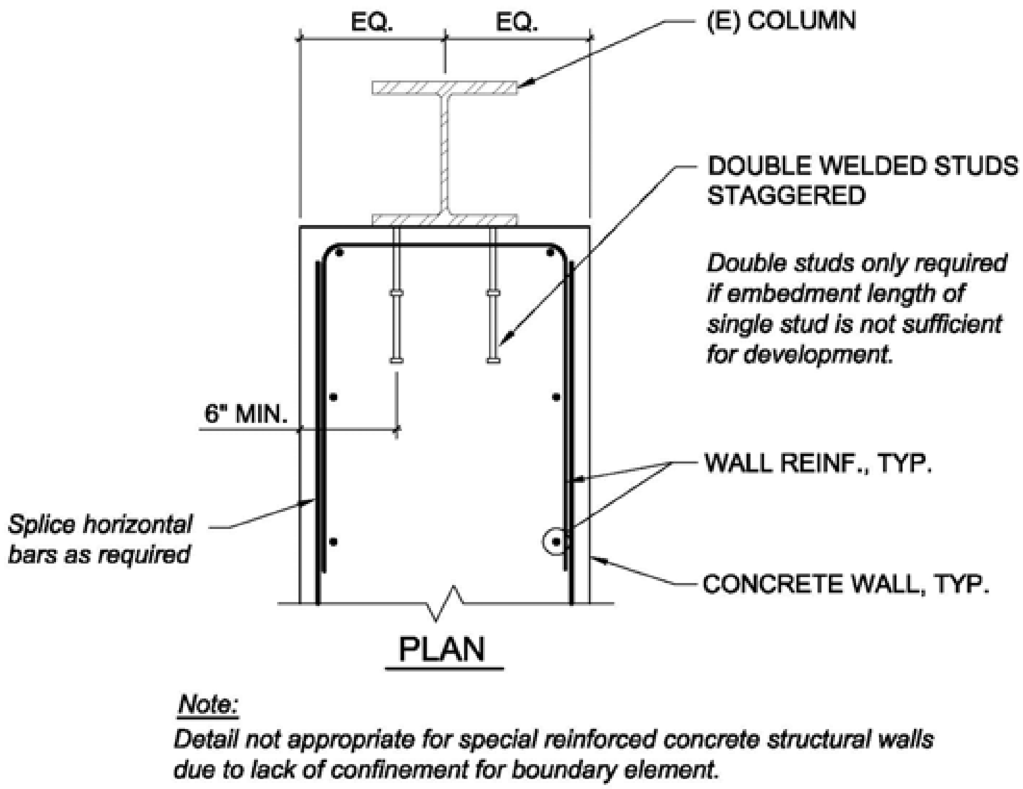



**Figure 8.4.2-4: Wall at Existing Column**

*Frame with infill walls:* Since the purpose of this technique is to create a shear wall system, the detailing should meet this goal by providing continuous load paths for both the laterally induced shear and overturning forces. Welded shear studs or reinforcing bars should be provided along all wall panel edges to achieve this. The horizontal shear force transfers from one wall to another through the studs on the top and bottom flanges and the webs of the beams at each floor. Additional plates could be welded between flanges to reduce stresses in beam webs. The columns become the boundary elements of the shear walls. Their effectiveness in providing overturning resistance may be limited by the number of shear studs or reinforcing bars that can be installed. Consider welding one stud on top of another to increase the embedment depth to avoid pullout failure.

*Offset walls:* Walls do not necessarily have to be centered on the existing beams and columns. By offsetting a wall and using the beam and column webs as the edge of the wall, shoring and metal deck removal only has to be performed on one side of the frame. An offset wall also lends itself to shotcrete construction, while walls with centered steel members have to be cast-in-place since it is not possible to place shotcrete on both sides of a steel member.

### *Cost/Disruption*

Adding concrete walls to an existing building is costly, though some savings can be found in the wall construction. Shotcrete walls are typically cheaper and faster to construct than conventional concrete walls due to the savings in materials and labor associated with formwork. Cost savings can be even greater if shotcrete is applied against an existing wall at a stair or elevator and mechanical shafts. CMU walls are generally less expensive than either shotcrete or cast-in-place concrete. New foundations are almost always required for new walls and could be extremely costly if deep foundations, such as drilled piers, are added.

Installing new walls is disruptive to the occupants because of the noise and vibrations associated with construction. Even if tenants are relocated to parts of the building where the work is not being performed, vibrations associated with cutting, chipping, and drilling of concrete can transmit through the structure. The disruption can be reduced somewhat if the walls are installed at the perimeter.

### *Construction Considerations*

The engineer's involvement during the construction phase is critical during a seismic rehabilitation. The design of the retrofit scheme must not neglect the construction phase and should consider the issues below at a minimum.

*Shotcrete walls:* The quality of a shotcrete wall is highly dependent on the skill of the nozzle operator. Building codes typically require preconstruction test panels constructed and reinforced similarly to the actual walls. This allows for inspection of the finished product, sawcutting to verify the quality of the shotcrete, and coring to determine strength. Additional test panels for overhead joints should also be requested to allow for cores to be taken to inspect the surface preparation and the joint bond. A series of ACI 506 publications provide general information, specifications, certification of nozzle operators, and evaluation of shotcrete (ACI, 1991, 1994, 1995a, 1995b, 1998).

*Congestion:* Congestion issues may be encountered at the ends of the walls due to the presence of the columns, confinement steel, and anchorage for horizontal wall reinforcing steel. Consider using mechanical couplers in heavily reinforced boundary elements and walls where lap splices are impractical.

*Concrete/shotcrete placement:* The type of construction should be determined during the design phase because details for cast-in-place concrete and shotcrete construction are different. It may not always be possible to confidently use either type of construction; and, thus, the details creating such situations should be avoided. During construction, some locations require particular attention when shotcrete and concrete are being placed to ensure that all gaps are filled – tops of wall panels, k-region of steel sections, and construction joints. Figure 8.4.2-5 shows a pilaster at a steel column where shotcrete shadowing restrictions require the use of cast-in-place construction for the pilaster but still allow shotcrete construction away from the pilaster. See Section 21.4.5 for additional discussion concerning concrete and shotcrete construction.

*Removal of existing nonstructural elements:* Exterior cladding and interior partitions may have to be removed to deliver concrete formwork and other equipment. Walls that encase beams at the roof require temporary removal of the roofing and waterproofing. Installation of the walls will affect ceilings, lights, and other mechanical/electrical/pluming components. Nonstructural elements located within the frame bays being strengthened have to be moved for construction of the walls. Permanent relocation may be desired for some of these elements to minimize openings in the new walls. Older buildings with items of historical significance require additional coordination and effort so that these items are not damaged when temporarily removed and are restored properly.

*Removal of existing structural elements:* See general discussion in Section 8.4.1.

*Construction loads:* See general discussion in Section 8.4.1.

***Proprietary Concerns***

Mechanical couplers for reinforcing steel are proprietary.

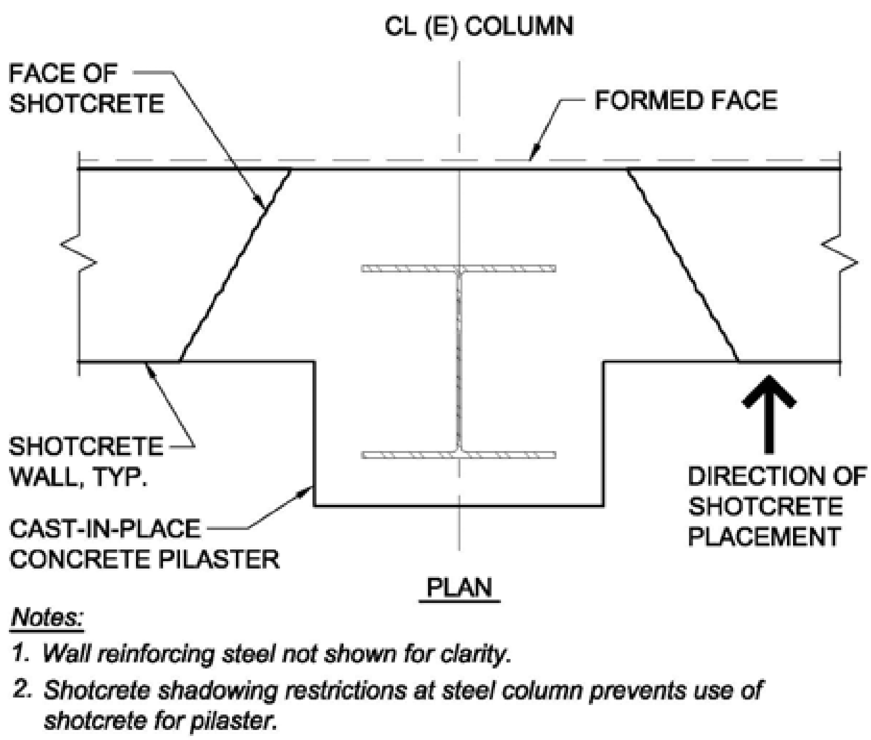



**Figure 8.4.2-5: Combined Shotcrete and Cast-In-Place Construction**

### 8.4.3 Add Steel Cover Plates or Box Existing Steel Member

#### *Deficiency Addressed by Rehabilitation Technique*

Frame members that are inadequate to resist the seismic demands are strengthened with cover plates or by adding side plates to W-shapes to create box sections. This reduces axial and flexural stresses in beams and columns and could also be used to increase the shear strengths of these members.

#### *Description of the Rehabilitation Technique*

Cover plates are welded to the outside of existing flanges, which in effect, increase the flange areas. When strengthening a beam, a cover plate is more commonly attached only to the underside of the beam since the presence of an existing diaphragm makes it difficult and costly to weld to the top of the beam or attach side plates to box the beam. However, for strengthening non-composite beams, top cover plates are virtually indispensable. Conversely for columns, plates are typically welded to the sides of W-shapes because it is much more effective to increase the axial and flexural capacities by converting a column to a box section and the shear capacity by essentially providing two additional webs.

#### *Design Considerations*

Thorough knowledge of the existing material behaviors and strengths are necessary for the new and existing elements to interact in the desired manner. If welding to the existing components, the carbon equivalent of these components need to be verified through as-built records or new testing to determine their weldability; see FEMA 356 (FEMA, 2000e) for further discussion of this issue. Other design issues include the following:

*Research basis:* No references directly addressing the addition of steel cover plates to existing frame members have been identified.

*Beams:* The flexural strength of a beam can be improved by welding cover plates to the bottom if there is a composite slab present. If there is only a bare metal deck, a cover plate on only one side of the beam may not be very effective. However, it could be useful for strengthening beams with large axial forces, primarily collector members.

*Columns:* The overall capacity of a column is determined by axial-flexural interaction. Boxing a column decreases its slenderness and, therefore, increases its axial and flexural capacities. Whether the areas of the new plates can be directly included in computing these capacities depends on their continuity and detailing at a beam-column joint. Except for one side of the exterior columns, beams framing into the columns at each floor will disrupt the continuity of the new plates.

*Foundations:* Where cover plates are added to the columns at their base, a reevaluation of the foundation system is warranted. It is not uncommon for frame columns to develop plastic hinges at their bases and thus, the increase in demand on the foundation may be greater than intended. The strength and stiffness of the base plate and the anchor rods should be evaluated and upgraded accordingly. A more in-depth discussion of column connections to the foundation is provided in Section 8.4.5. Foundation upgrades are not common with this technique, but if required, refer to the chapter on foundations.

*Nonstructural issues:* The beam and column upgrades hardly take up any additional space but minor modifications and temporary relocations may be required for some architectural and M/E/P elements.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Beams:* When a section of beam is strengthened, the cover plate typically attaches to the bottom flange with fillet welds, as shown in Figure 8.4.3-1. The length and size of the welds determine the shear flow between the plate and the existing beam, similar to interactions between other composite elements.

*Columns:* Side plates used to create a box section can be attached to the column flanges with CJP, PP, or fillet welds, as shown in Figure 8.4.3-2. The fillet weld is the simplest to implement since it does not require additional preparations like the other welds, particularly the use of backing plates and beveling of the welded edges. The type of connection used may limit the shear flow between the plates and the existing column.

*Existing floors:* Whether the columns are upgraded with cover plates or box sections, the new plates have to terminate at the existing beams and slabs. Though it is possible to achieve continuity by using additional plates or sections if necessary, this would create detailing and construction complexities and should be avoided. Furthermore, excessive welding around a beam-column joint could create undesirable residual stresses in the joint.

### *Cost/Disruption*

Schemes that involve slab removal, work around a connection, and foundation work are costly. As typical with seismic upgrades, cost and disruption is minimized when schemes are kept simple.

### *Construction Considerations*

See Section 8.4.1 for general discussion of welding issues, removal of existing nonstructural and structural elements, and construction loads.

### *Proprietary Concerns*

There are no known proprietary concerns with this technique.

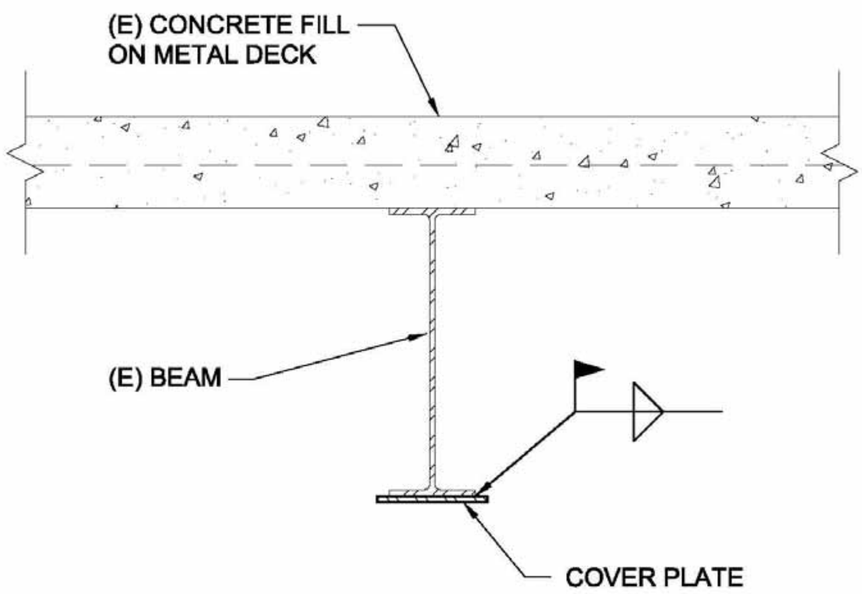



**Figure 8.4.3-1: Cover Plate at Existing Beam**

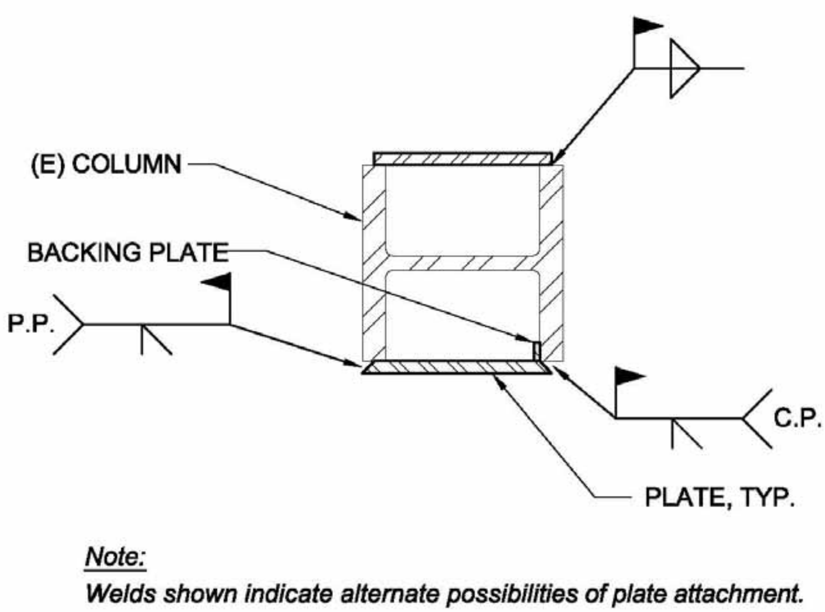



**Figure 8.4.3-2: Box Section at Existing Column**

### 8.4.4 Provide Collector in a Concrete Fill on Metal Deck Diaphragm

***Deficiency Addressed by Rehabilitation Technique***

Beams that are inadequate to transfer collector loads are strengthened.

***Description of the Rehabilitation Technique***

Plates can be welded to various portions of the existing beam. Cover plates, discussed in Section 8.4.3, are commonly attached to the underside of beams. To attach plates to top flanges, it may be simplest to orient the plates vertically and weld to the underside of the flanges, as shown in Figure 8.4.4-1, or similarly on top of the bottom flanges. Given all the possible options, the plate locations may be dictated by the eccentricities, continuity options at a beam-column joint, or the presence of nonstructural elements.

***Design Considerations***

Though the purpose of this technique is to strengthen a beam axially, it also changes the flexural properties of the beam by increasing its stiffness. It should be verified that this does not have unintended consequences, such as converting the beam into a frame member. Eccentricities in collectors are typically neglected. However, deep collectors and/or collectors with large forces could be subject to significant moments from these eccentricities. The eccentricities could be reduced if plates are attached to the top flanges but requires other detailing considerations at a beam-column joint.

***Detailing Considerations***

The new plates should attach to the columns with complete joint penetration (CJP) welds. Consider replacing the welds in the existing collector with high notch toughness welds in a high seismic region. Continuity plates directly aligned with the collector elements at a beam-column joint should be used as much as possible but may be offset if all eccentricities and their effects on the joint are considered.

***Cost/Disruption***

This technique is relatively inexpensive compared to other types of steel frame upgrades. The most expensive part of the structural upgrade is associated with the work at the beam-column connections. The nonstructural disruptions to the architectural and M/E/P elements are typical.

***Construction Considerations***

See Section 8.4.1 for general discussions of welding issues, removal of existing nonstructural and structural elements, and construction loads.

***Proprietary Concerns***

There are no known proprietary concerns with this technique.

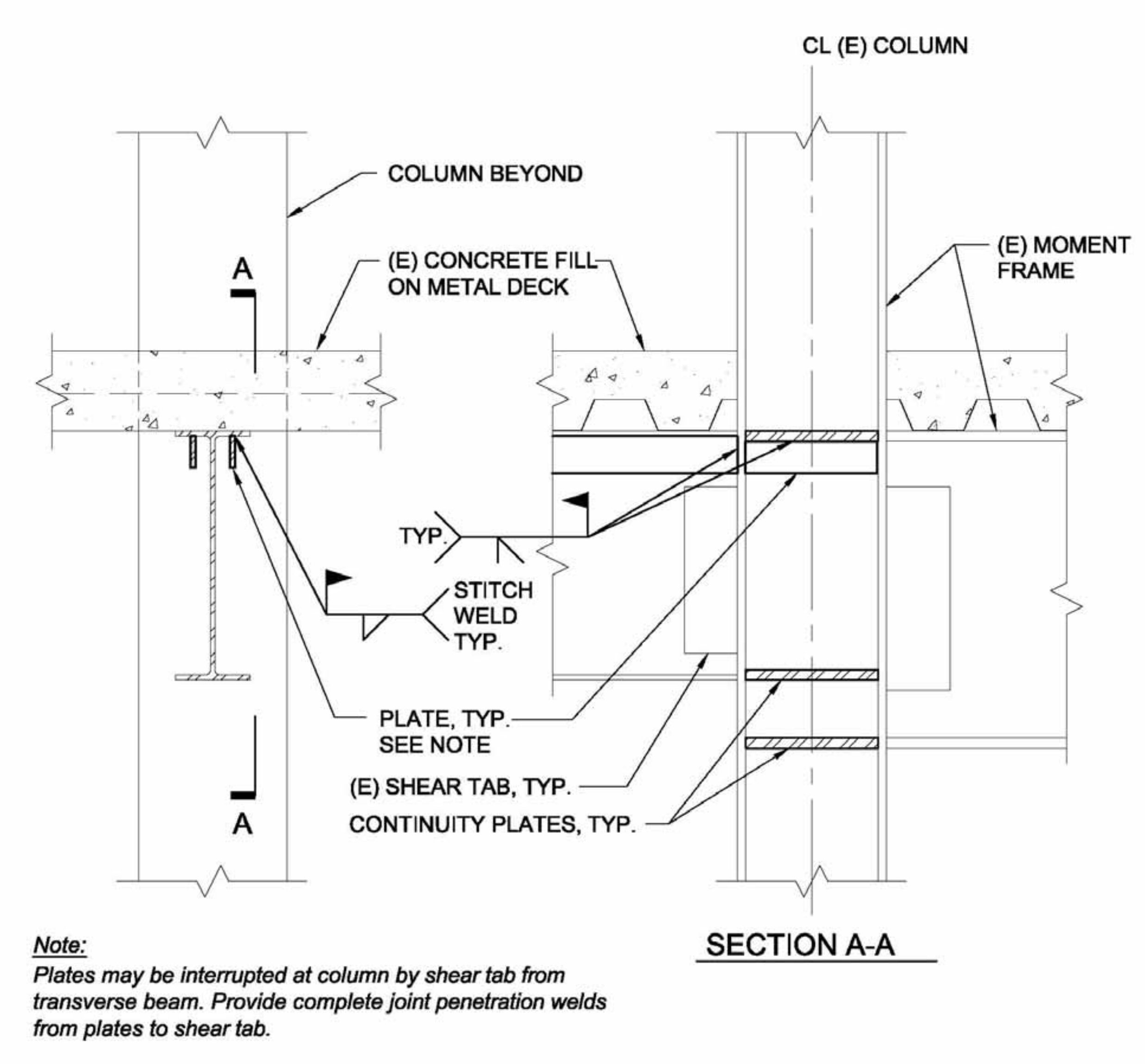



**Figure 8.4.4-1: Plate Collectors at Existing Beam**

### 8.4.5 Enhance Connection of Steel Column to Foundation

***Deficiency Addressed by Rehabilitation Technique***

Frame columns are subject to axial (including possible tension), flexural, and shear forces. To this end, columns with inadequate anchorage to the foundation limit the capacity of a frame. The columns could be part of an existing lateral force-resisting system that do not meet current standards or part of an upgraded system with larger forces resulting from increased stiffness.

### *Description of the Rehabilitation Technique*

Two methods are common for enhancing the column connection to the foundation. First, modifications at the base plate could include the addition of anchor rods, welding shear lugs to the base plate, and/or enlargement of the base plate. The other method is to encase the column in a concrete pedestal. It is also possible to use both of these methods together. The foundation system itself is not addressed here but is discussed in Chapter 23.

Anchor rods can be used to resist tensile forces due to uplift and flexure. Holes drilled in the existing footing for these rods can be filled with a nonshrink high strength grout or chemical adhesive. Shear lugs may be used to transfer shear forces into the foundation. If the existing base plate is not large enough to accommodate the new rods or lugs, new plates can be welded to the existing plate to enlarge its area. This is also necessary if the allowable bearing stress is exceeded due to increased column compression. The increased base plate size leads to greater shear and flexural forces in the plate. It is likely that the plate will not be thick enough to resist these forces. Thus, stiffeners that act as supports can be welded to the base plate to reduce the plate forces. The stiffeners also provide additional load paths from the column to the base plate. Figure 8.4.5-1 shows some of these modifications.

Base plate modifications may not always be practical or possible. Instead, the column can be encased in a concrete pedestal above the footing, as shown in Figure 8.4.5-2. Shear forces would transfer through direct bearing of the column against the pedestal. The pedestal could also be used to transfer uplift and flexural forces by relying on the existing base plate and other mechanisms, such as welded shear studs along the column. The pedestal itself should be detailed as a reinforced concrete column that meets all ACI requirements and seismic provisions.

### *Design Considerations*

Thorough knowledge of the existing material behaviors and strengths are necessary for the new and existing elements to interact in the desired manner. A complete presentation of column base plate design can be found in AISC *Design Guide 1* (Fisher and Kloiber, 2005). Key design issues include the following:

*Research basis:* No references directly addressing the upgrade of connections of steel columns to foundations have been identified.

*Shear lugs:* Columns that are in compression offer shear resistance through friction below the base plate. AISC recommends different friction coefficients depending on the location of the base plate with respect to the top of concrete. Additional shear resistance is required when the compressive force is not great enough or there is tension in the column. Though not common in retrofit applications, shear lugs, or simply plates welded to the bottom of a base plate, have the advantage of shallow embedment compared to anchor rods. The lugs also receive confinement from the base plate above and simplify the design by allowing anchor rods to only resist tension. The design of a shear lug is a function of the allowable bearing stress in the surrounding grout and flexural forces in the lug.

*Anchor rods:* Tension in an anchor rod is developed through bond along its length and/or direct bearing through a hook, bolt head, or nut at the end of the rod. Though it is possible to drill a hole large enough in an existing footing to accommodate a bolt head or nut, manufacturers of grouts and adhesives sometimes limit the size of the a hole for a given rod diameter in which its product can be used as an infill. In this case, the tensile force has to be developed entirely through bond. For bond development, the rod has to be threaded or some other means of mechanical anchorage has to be provided along the rod. Anchor rods are not typically used to transfer shear to the foundation because the mechanism for shear transfer is difficult to define and still subject to debate. If unavoidable, several factors should be considered, including bending of the rod through the oversized hole in the base plate and shear and tension interaction on the rod. If the column shear force is being distributed between both new and existing rods, note that the holes for the existing rods are likely to be oversized and welded plate washers should be verified or provided.

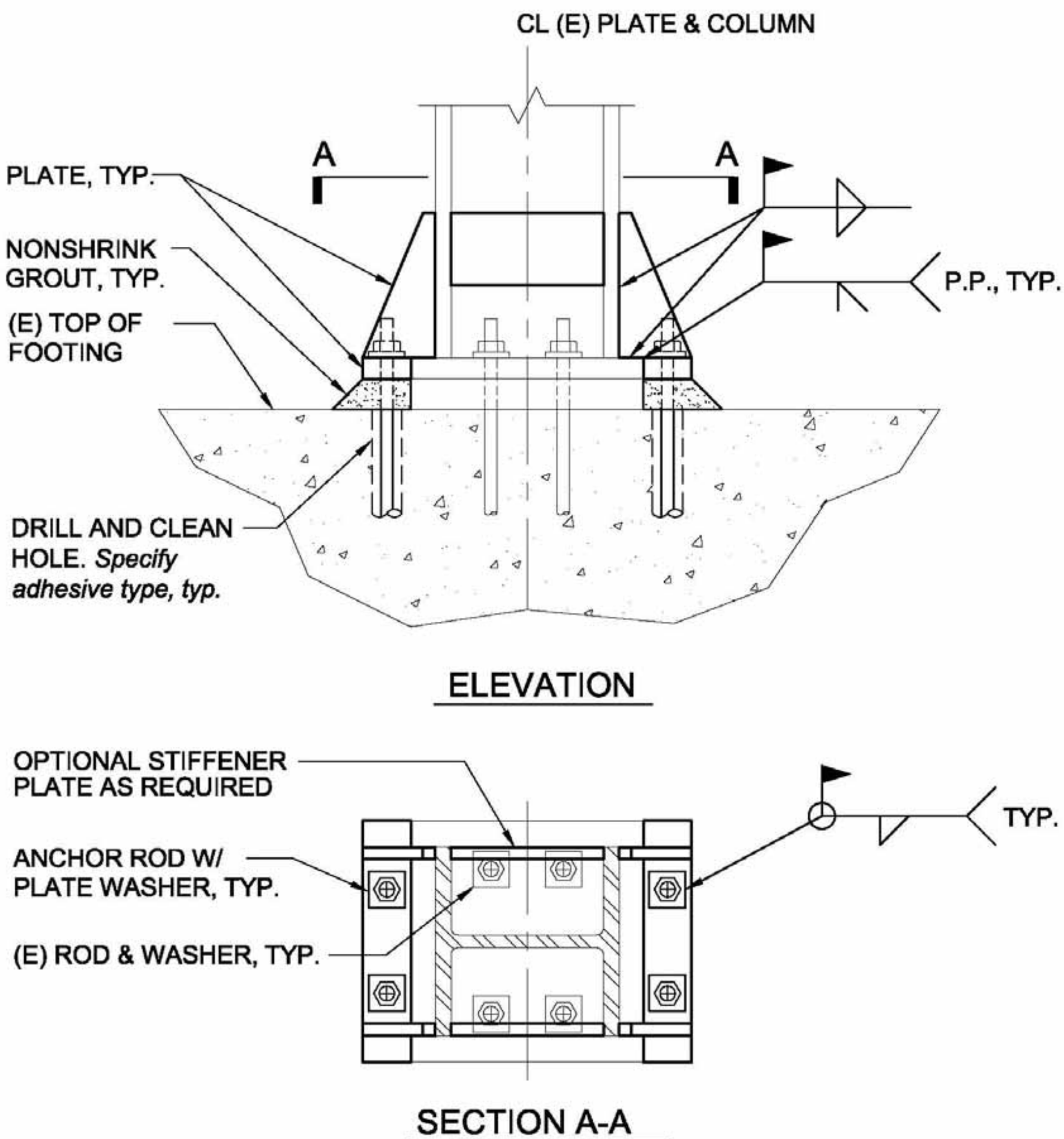



**Figure 8.4.5-1: Modified Base Plate to Increase Uplift Capacity**

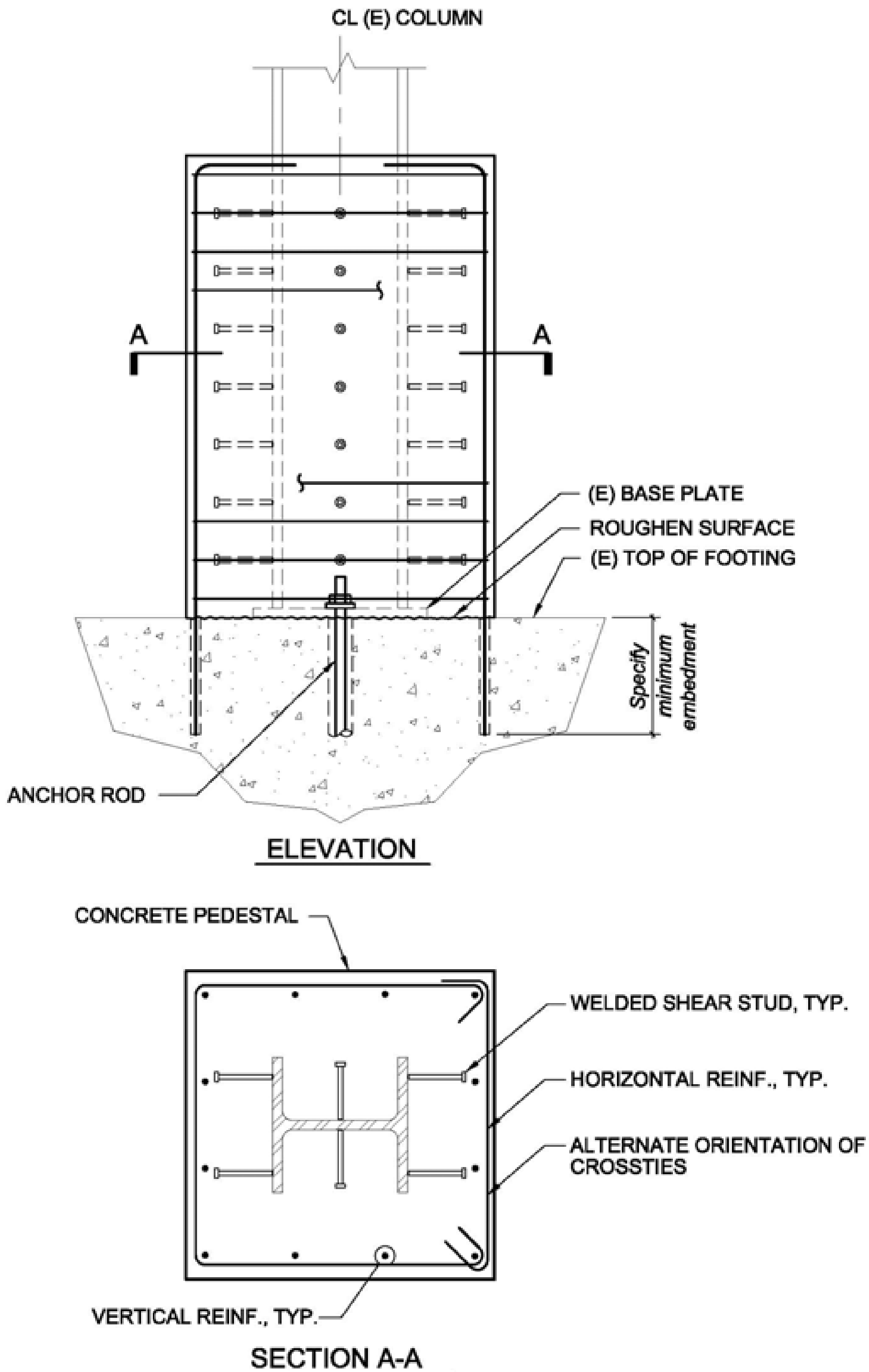



**Figure 8.4.5-2: Concrete Pedestal at Existing Column**

*Stiffeners:* The simplest way to reduce flexural stresses in a base plate is to add stiffeners that essentially provide multiple supports along the base plate. These stiffeners also assist in transferring uplift and flexural forces to the base plate and reduce the weld stresses at the base of a column.

*Nonstructural issues:* Base plates are typically hidden in the slab and thus, the upgrade could remain hidden if it only involves adding anchor rods. Pedestals, on the other hand, require additional space and could present aesthetic issues. However, basements are often used for parking, storage, or as equipment locations, where aesthetic considerations may be negligible.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Base plates:* If the existing base plate can accommodate new anchor rods or lugs, burning holes in the plate is usually acceptable. Otherwise, enlargement of an existing base plate is accomplished by welding new plates to the existing plate. Only partial penetration (PP) welds can be used since the lack of backing precludes using complete joint penetration (CJP) welds unless concrete is removed to allow placement of backing plates. The depth of the PP weld limits the effective thickness of a plate. Also of note is that AISC relaxes its typical edge distance provisions if there is no lateral load on a base plate. The only recommendation is that enough edge distance remains such that the drill or punch does not drift when a hole is made. *AISC Design Guide 1* suggests that one quarter of an inch is enough to meet this condition.

*Anchor rods:* Detailing differs depending on if an anchor rod is designed to resist small shear forces and tension or tension only. For the latter, an oversized hole in the base plate and a thick plate washer over the hole is considered adequate. Since oversized holes are intended to allow for inaccuracies in anchor rod placement when concrete footings are placed, it should be possible to use smaller holes if the holes are being drilled. In addition to the design considerations mentioned above, anchor rods with shear forces require welding heavy plate washers to the base plate. The washers should have close-fit holes to minimize the movement required to engage all of the rods together. Nuts are sometimes welded to the washers but the mechanical properties of the nuts could lead to welding complications.

*Pedestals:* Reinforcement of the pedestal is similar to a reinforced concrete column. Vertical bars provide tensile and flexural strength while hoops or ties provide confinement. At the top of footing, the shear is transferred through shear friction in the anchored reinforcing steel dowels. The concrete surface should be roughened, which also increases the shear friction values. If the pedestal is also used to transfer uplift and flexural forces, additional dowels are required for tension. A single dowel should not be considered to provide both shear and tensile resistance. One mechanism to transfer uplift and flexural forces from the column to the pedestal is the use of welded shear studs along the column. The size and spacing of the shear studs may determine the minimum pedestal diameter and height.

*Concrete anchorage:* Installation of the anchor rods and dowels requires drilling into the existing footing. Consider using a scanning device to avoid damaging the reinforcing steel in the top layer of the footing. Cores may have to be taken to confirm the concrete strength. Many commercial grouts and adhesives are available for bonding the anchor to the concrete. Verify that the selected product is appropriate for the specific anchor in a seismic application. On site inspection and testing of the anchors are mandatory since their performance rely heavily on the installation process. The grout and adhesive product vendors' ICC Evaluation Service reports provide standardized installation procedures and anchorage capacities.

***Cost/Disruption***

The cost of a column connection to foundation upgrade is not very expensive. However, this technique is not usually performed only by itself. Costs would be small relative to an overall lateral force-resisting system upgrade and a foundation upgrade. Disruption could be minimal since typically, there are no tenants in the basement. Even if the basement were used for tenant access, such as parking, only a few columns would be affected at a time.

***Construction Considerations***

The engineer's involvement during the construction phase is critical during a seismic rehabilitation. The design of the retrofit scheme must not neglect the construction phase and should consider these issues at a minimum:

*Welding issues:* See general discussion in Section 8.4.1.

*Removal of existing nonstructural elements:* This technique tends to be less disruptive of nonstructural functions compared to other techniques. However, there may still be some architectural and M/E/P elements, such as partitions and pipes adjacent to the columns, that require temporary removal. See Section 8.4.1 for discussions of fireproofing, asbestos, and concrete encasement.

*Removal of existing structural elements:* To access the base plate, an existing slab and slab reinforcement will probably have to be removed and replaced. Care should be taken to not damage any of the existing structural elements. If the existing base plate is being replaced entirely, a column shoring scheme has to be devised. The existing anchor rods should be cut at the top of footing and the new rods have to be relocated.

*Construction loads:* See general discussion in Section 8.4.1. Construction loads at the basement level are not typically a concern if a slab-on-grade is present.

***Proprietary Concerns***

Many grout and adhesive products are available.

### 8.4.6 Enhance Beam-Column Moment Connection

***Deficiency Addressed by Rehabilitation Technique***

Riveted, bolted, and WSMF connections are upgraded to improve their ability to withstand inelastic rotational demands and develop the plastic moment capacity of the beams.

### *Description of the Rehabilitation Technique*

The techniques discussed in this section were developed specifically to address pre-Northridge WSMF connections. These techniques can be adapted with prudence to existing riveted and bolted connections that are found to be inadequate or to upgrade partially restrained connections. These techniques are covered thoroughly in *AISC Design Guide 12* and FEMA 351 and only briefly presented here. The reduced beam section (RBS) is the only technique that weakens the beam in flexure, which in turn, moves the plastic hinge away from the column and reduces the demand on the complete joint penetration (CJP) welds. Two other methods - welded haunch and bolted bracket – also move the hinge away from the column but strengthen the existing connection and seek to maintain the original flexural capacity of the beam. A similar method employs cover plates over the beam flanges, requiring little additional space. This method is not discussed in detail here but more information can be found in FEMA 351. Additional modifications that should be performed for each technique include adding or verifying the capacity of beam flange continuity plates across the column web and strengthening the panel zone; see the references mentioned above and *AISC Design Guide 13* (Carter, 1999) for reference.

The selection of a particular connection modification depends on specific project factors. There are advantages and disadvantages to each of the methods. While each of the three connections discussed below consistently developed a minimum plastic rotation of 0.02 radian in cyclic loading experiments, the welded haunch and bolted bracket demonstrated higher levels of performance and reliability (Gross et al., 1999). On the other hand, the RBS modification requires no additional space and reduces the beam capacity to enforce a strong-column weak-beam condition. The welded haunch is the only modification that exhibited desirable behavior in tests where the beam top flanges were left in a pre-Northridge condition (Gross et al., 1999). The bolted bracket scheme requires top flange reinforcement but eliminates field welding.

### *Design Considerations*

Connection modifications merely ensure that the connections behave as originally intended in a moment frame. These modifications by themselves do not reduce the frame forces in an earthquake. Other structural upgrades are necessary if the frame does not have sufficient global strength or stiffness to resist the demands.

*Research basis:* Numerous experimental programs have been performed under the auspices of the SAC Joint Venture, the National Science Foundation, the National Institutes of Standards and Technology, and AISC. Most of the results of these tests have been summarized in *AISC Design Guide 12* and the FEMA documents listed in the references. The user is cautioned to extrapolate beyond the conditions that have been tested only with proper considerations of the differences between the tested connections and the building conditions.

*Design forces:* AISC enforces the strong-column weak-beam concept by specifying a minimum column-beam moment ratio in the *AISC Seismic Provisions*. The general intent is to prevent plastic hinges from developing in the columns, which could lead to a soft story. Some yielding may still occur in the columns without causing loss in frame strengths. The required flexural strength of a column is determined from several variables associated with the beam. The beam plastic moment at the critical plastic section – centerline of RBS or tip of haunch and bracket –

includes a strain hardening factor and the expected yield stress of the flanges. An additional moment is a function of the shear in the beam and the distance from the critical plastic section to the column centerline. This shear is a combination of the shear associated with the plastic moment and the gravity loads.

*RBS:* The design of moment frames is often governed by global and interstory drift limits instead of strength requirements. Thus, the reduction in beam strength and stiffness from an RBS could be acceptable within practical limits. If the presence of an existing slab poses construction and design issues, then the top flange may remain intact and only the bottom flange has to be cut; though minimal removal of the existing slab is still required to replace the existing top flange weld. Note that a bottom flange modification only achieves a minimal stress reduction and that both the top and bottom flange CJP welds still require replacement with high toughness weld metal. Various RBS cut shapes have been tested successfully, though the radius cut RBS minimizes stress concentrations. The critical dimension is the depth of the cut, which can also be expressed in terms of flange width reduction as a percentage. AISC recommends a maximum flange reduction of 50% as a practical limit for field modifications. To determine if a flange reduction is adequate, the maximum moment at the face of the column can be computed for the modified beam. If the ratio of this moment to the plastic moment of the beam exceeds 1.05, a bottom flange RBS modification is not recommended. Either a top flange RBS should also be provided or another method should be used.

*Welded haunch:* A tapered haunch welded to a beam bottom flange changes the force transfer mechanism at the connection. Analytical and experimental studies have found that the beam shear transfers through the haunch flange (FEMA, 2000b). This, in turn, reduces the shear stresses in the flange welds and increases the depth of the column panel zone. The welded haunch is the only connection out of the three presented for this technique to reach plastic rotations on the order of 0.03 radian while allowing the top flange weld to remain in a pre-Northridge condition in tests. Additional rotational capacity can be obtained by upgrading the top flange groove weld to a higher toughness, adding a cover plate, or adding another haunch, in order of increasing performance. The two latter modifications can be performed without replacing the top flange weld.

*Bolted bracket:* Considered an alternative to the welded haunch, this option does not require any field welding. A portion of the flange force is transferred to the bracket and reduces the weld stresses. Several bolted bracket configurations are possible and can be applied to either beam flange or both flanges for large beams. Common types of brackets include the haunch bracket, pipe bracket, angle bracket, and double angle bracket. Pipe and angle brackets are relatively compact and can be covered up by the slab for a top flange application. For bottom flange haunch only schemes, a stiff angle is still recommended at the top flange (Gross et al., 1999). The pre-Northridge weld in the top flange can remain in this case even if it is found to have defects.

*Material strength:* Yield strengths of existing materials are best determined from tests of samples taken from the actual members. Samples taken from flanges are preferred over the web. When samples are not available, AISC *Design Guide 12* prescribes some overstrength values for different grades of steel.

*Minor axis connections:* Moment connections to the minor axis of a column are not as common, but sometimes present in older moment frame construction. Their performance in the Northridge earthquake did not result in significant damage. Thus, little work has been done to study the rehabilitation of these connections. Similar concepts presented in this section may be used to rehabilitate these connections with the recognition that the columns may be weaker than the beams, which would also limit the inelastic demands on the connections. Note the governing jurisdiction may require qualification through the testing program described in FEMA 351.

*Nonstructural issues:* Welded haunches and bolted brackets are considerable in size. The recommended welded haunch and bolted bracket depths are approximately one third and one half times the beam depths, respectively. For a W36 beam used in a moment frame, this adds up to 18 inches to the connection depth. This makes the option of adding components to only the bottom flange more attractive, though it could still interfere with the ceiling. At the top flange, the modification would certainly have to be hidden or incorporated into an architectural feature.

***Detailing Considerations***

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Weld filler metal matching and overmatching:* Weld filler metals with greater tensile strength than the connected steel should be used. Flux cored arc welding and shielded metal arc welding electrodes that conform to E70 specifications exhibit overmatching properties compared to common steel specifications, including ASTM A36, A572 (Grade 42 and 50), A913 (Grade 50), and A992 (FEMA, 2000b).

*Weld metal toughness:* Tests on existing connections modified with RBS but no modifications to the existing beam flange welds showed poor performance (Gross et al., 1999). Improved connection performance was achieved for connections where the top and bottom flange welds were replaced with higher toughness weld metal. *AISC Seismic Provisions* now require weld metals with minimum Charpy V-Notch (CVN) toughness of 20 ft-lbs at -20ºF and 40 ft-lbs at 70ºF for demand critical welds.

*Weld backing and access holes:* Backing plates provided for flange welds create a notch effect and also hinder detection of weld flaws at the weld root. Backing should be removed at flanges receiving new CJP groove welds and reinforced with fillet welds. The removal of existing backing without weld replacement is an ineffective upgrade since the existing weld is still likely to have low toughness. The size and shape of the weld access hole should be configured to provide welder access and minimize stress concentrations. FEMA 351 provides a specific weld access hole detail that meets these criteria.

*RBS (Figure 8.4.6-1):* The distance from the face of column to the start of the RBS cut and the length of the cut, more concisely known as the offset and the chord length, respectively, should be kept small as to not allow the moment to increase significantly from the plastic hinge to the column. Yet, the distance to the start of the cut should also be large enough to allow for the flange force from the RBS to be uniformly distributed across the flange at the CJP weld. The length of the cut should also be long enough to control the inelastic strains along the RBS. The radius of the cut is a function of its depth and the chord length. See AISC *Design Guide 12* for specific recommendations and guidelines for selecting these dimensions.

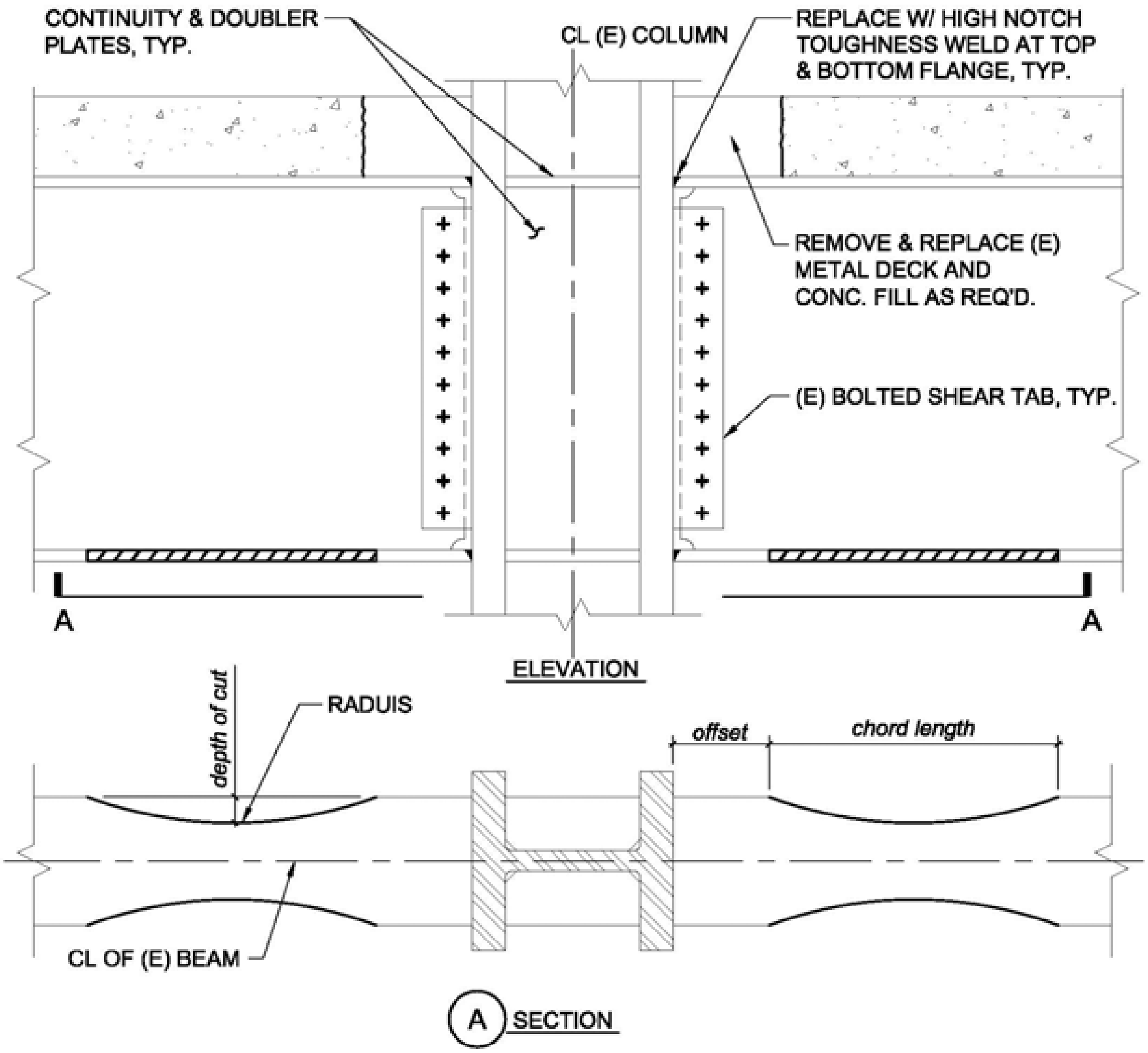



**Figure 8.4.6-1: Reduced Beam Section at Bottom Flange of Existing Beam**
*(adapted from AISC Design Guide 12)*

*Welded haunch (Figure 8.4.6-2):* Haunches can be cut from WT- or W-shapes. Typically, the haunch flange is groove welded to the beam and column flanges while the haunch web is fillet welded to these elements. Experimental programs have consistently used the same haunch geometry—length and depth approximately one half and one third that of the beam depth,

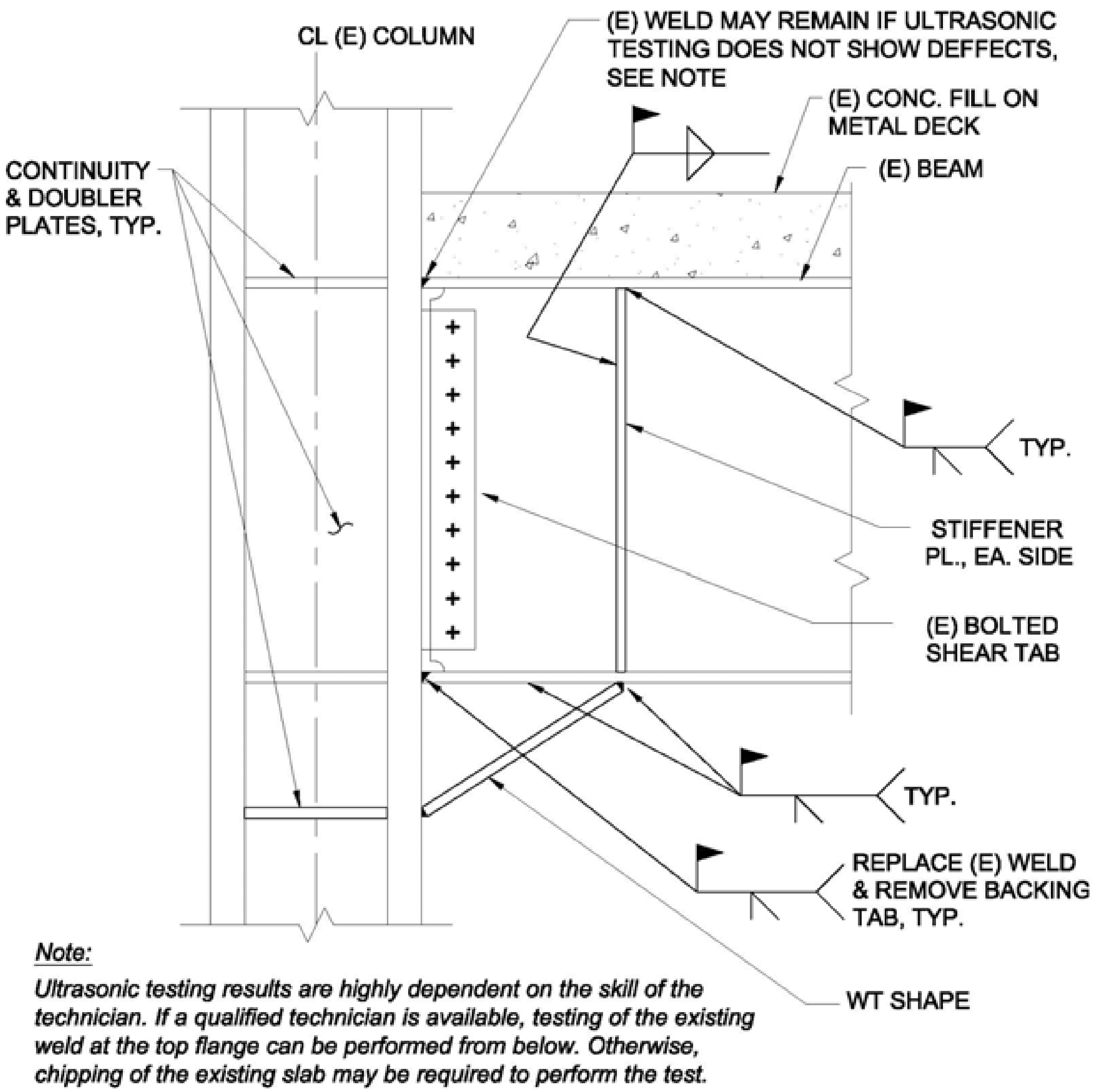



**Figure 8.4.6-2: Welded Haunch at Bottom Flange of Existing Beam**
*(adapted from AISC Design Guide 12)*

respectively. The haunch flange is sized to resist most of the shear force from the beam. The haunch web size can be established to achieve equilibrium. At the intersection of the beam and haunch flanges, web stiffeners should be provided for the beam. Similarly at the column, add continuity plates to align with the haunch flange.

*Bolted bracket:* The preferred configuration for this modification is a haunch bracket at the bottom flange and an angle bracket at the top flange that can be hidden within the slab. If additional reinforcement is necessary at the top flange, angles can be added below the top flange on each side of the web, as shown in Figure 8.4.6-3. Close-fit holes minimize slip in this connection. Tests have demonstrated that these connections are essentially fully rigid and can reach large plastic rotations while allowing the pre-Northridge welds at the flanges to remain (Gross et al., 1999). AISC recommends neglecting the existing welds and designing each bracket for the entire flange tension force. The bottom flange haunch bracket will be slightly larger than the welded haunch for the same beam size. The top flange angle bracket may require several rows of bolts along the beam flange and has to be designed for the prying force on the column flange. Due to limitations in the available L-shapes, the angle bracket may have to be cut from a W-shape.

### *Cost/Disruption*

Connection modifications are locally very disruptive. Modification of a connection typically requires access from two floors to perform the work on each flange. Noise associated with this type of work will spread and disrupt tenants on other floors unless the work is done during off-hours. The RBS is probably cheaper than the other two modifications since it requires the least amount of material and labor. With older buildings, there may be asbestos present in the fireproofing around the steel members, which could require costly abatements to expose the connections.

### *Construction Considerations*

The engineer's involvement during the construction phase is critical during a seismic rehabilitation. The design of the retrofit scheme must not neglect the construction phase and should consider these issues at a minimum:

*Welding/bolting issues:* See general discussion in Section 8.4.1. The bolted bracket modification can be performed without any field welding. Primary issues associated with the bolted bracket consist of typical field bolting issues such as set up, fit-up, and alignment.

*Removal of existing nonstructural elements:* The connection work will affect ceilings, lights, and other mechanical/electrical/pluming components. Connection modifications at the roof level may warrant removal of the roofing and waterproofing. See Section 8.4.1 for discussions of fireproofing, asbestos, and concrete encasement.

*Removal of existing structural elements:* See general discussion in Section 8.4.1.

*Construction loads:* See general discussion in Section 8.4.1.

### *Proprietary Concerns*

There are several proprietary connections that have been developed to upgrade pre-Northridge connections, generally sharing similar design intents. Some of these connections briefly presented in FEMA 351 include the Side Plate connection system, the Slotted Web connection, and one particular type of the bolted bracket connection. The reader should contact the licensors of these technologies for more information.

### 8.4.7 Enhance Column Splice

***Deficiency Addressed by Rehabilitation Technique***

Strengthen welded or bolted column splices that do not meet the detailing and minimum design strength requirements in *AISC Seismic Provisions*.

***Description of the Rehabilitation Technique***

Numerous options exist for upgrading a column splice. The approach and the level of strengthening depend on the type of lateral system since special moment frames have different requirements from other systems in the *AISC Seismic Provisions*. It also depends on if the existing splice is welded or bolted and if field welding is permitted. Field welding may be a necessity in some cases since the *AISC Seismic Provisions* do not allow bolts and welds to share loads on the same faying surface.

Most existing welded splices are likely to be complete joint penetration (CJP) welded or partial joint penetration (PP) welded. CJP welds require beveled transitions to avoid stress concentrations. A beveled transition can be constructed in the field by building up the weld over the thicker column flange or grinding away a portion of the flange. PP welds inherently possess stress concentrations at the unwelded portion of the joint and thus, have higher strength requirements, but do not require beveled transitions. PP welds can be strengthened by welding plates or stiffeners across the splice. When field welding is impractical or undesirable, it may be possible to use bolted plates at an existing welded splice if the bolts are designed to resist the entire load.

The rehabilitation method for a bolted splice depends on the controlling design mode for the splice – whether the splice is governed by net section fracture of the column, yielding of the bolts, or gross yield or net section fracture of the splice plate. Bolts that govern the existing splice capacity could be replaced with stronger bolts, such as replacing A325 with A490. The upgrade may also be as simple as replacing bolts that have threads included in the shear plane with longer bolts. Alternatively, more bolts or larger bolts could be provided, but both would require more extensive field work. Splice plates with insufficient strength can be replaced or new plates could be provided on the opposite side of the existing plate, which would require longer bolts. The existing plates may also be extended by welding new plates to the ends.

***Design Considerations***

Considerations for a column splice include the justification of a load path, preservation of symmetry to avoid eccentric loads, and deformation compatibility if welds and bolts are both used. In addition to the requirements in the *AISC Seismic Provisions*, the *AISC Steel Construction Manual* (AISC, 2005c) has a general discussion of column splices and contains typical details of W-shape splices. Information can be found regarding filler plate sizes for different column depths and erection tolerances. Important design issues include the following:

*Research basis:* No references directly addressing the upgrade of existing column splices have been identified.

*Design strength:* The *AISC Seismic Provisions* specify a minimum strength requirement for all splices as a function of the expected yield strength and the flange area of the column. In addition, PP welded splices have higher strength requirements due to the stress concentration manifested in the crack-like notch at the unwelded side of a flange. Lastly, column splices in special moment frames have to be designed for the full flexural strength of the smaller column and their web splices require a shear strength of twice the plastic flexural strength of the column divided by the story height.

*PP welded splices:* For flange thickness differences between the columns up to 1/8", steel shims can be fillet welded to the inside of the flanges. Greater than 1/8", it may be more practical to attach a plate to the flanges using CJP welds.

*Plate locations:* Plates can be extremely versatile whether they are welded or bolted to the columns. Bolted plates are commonly placed at the outside of flanges, where extra filler plates can be used to make up differences in flange thicknesses. Filler plates are not necessary at the inside of flanges, where a flush surface is already provided if the same nominal depth columns are present. Plates can also be welded from one flange tip to another, which provides increased shear strength and stability across the joint. See Figure 8.4.7-1.

*Nonstructural issues:* The column splices hardly take up any additional space but minor modifications and temporary relocations may be required for some architectural and M/E/P elements.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Welded splices:* Large shrinkage strains could develop when welding heavy sections. Bolted or fillet welded plate slices should be considered as an alternative if there is a possibility of a brittle weld fracture.

*Bolted splices:* If new holes are drilled in the existing column or splice plate, tolerances of the existing holes should be verified to ensure that bolts will be loaded evenly. The capacity of the net section should also be checked. See Figure 8.4.7-2 for a sample detail.

*Web splices:* Column webs in moment frames that use bolted web splices require plates or channels on both sides to reduce eccentricities. This should be considered for column webs in other lateral force-resisting systems though it is not a specific requirement in the *AISC Seismic Provisions*.

### *Cost/Disruption*

The cost of implementing this technique is highly dependent on the level of modification performed to the existing splice. The level of disruption is typical for that of steel frame upgrades, involving vacancy of space, removal of all nonstructural elements around the column, and significant noise.

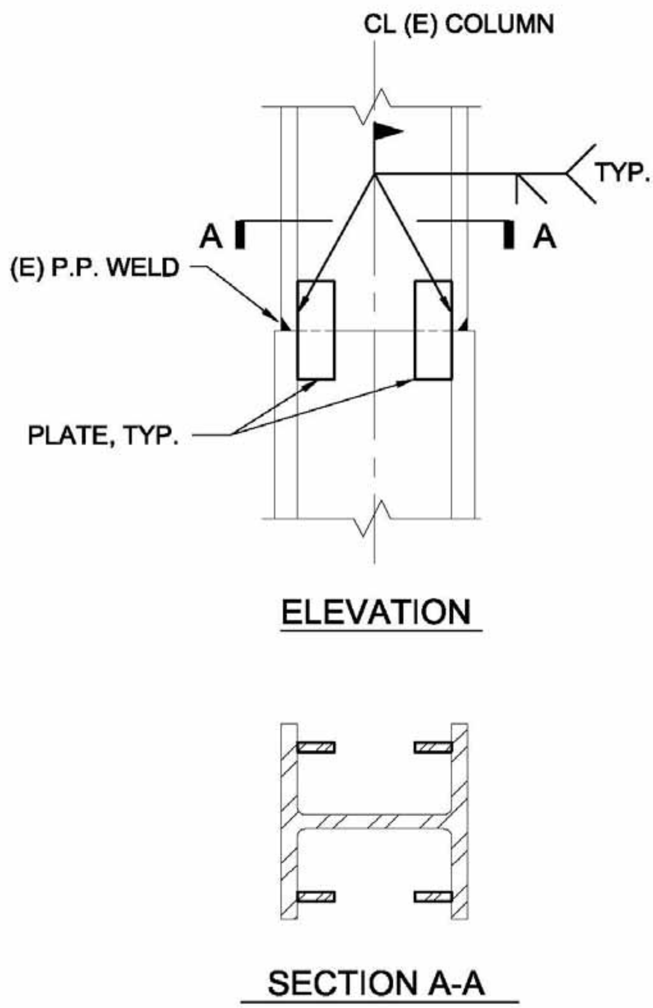



**Figure 8.4.7-1: Welded Splice Upgrade at Existing Column**

### *Construction Considerations*

The engineer's involvement during the construction phase is critical during a seismic rehabilitation. The design of the retrofit scheme must not neglect the construction phase and should consider these issues at a minimum:

*Welding issues:* See general discussion in Section 8.4.1. Though weld fractures were not found at column splices after the Northridge earthquake, an environment in which the welder can perform quality welds is critical.

*Removal of existing nonstructural elements:* See general discussion in Section 8.4.1. Depending on the location of the splice, the upgrade could affect ceilings, lights, and other mechanical/electrical/pluming components.

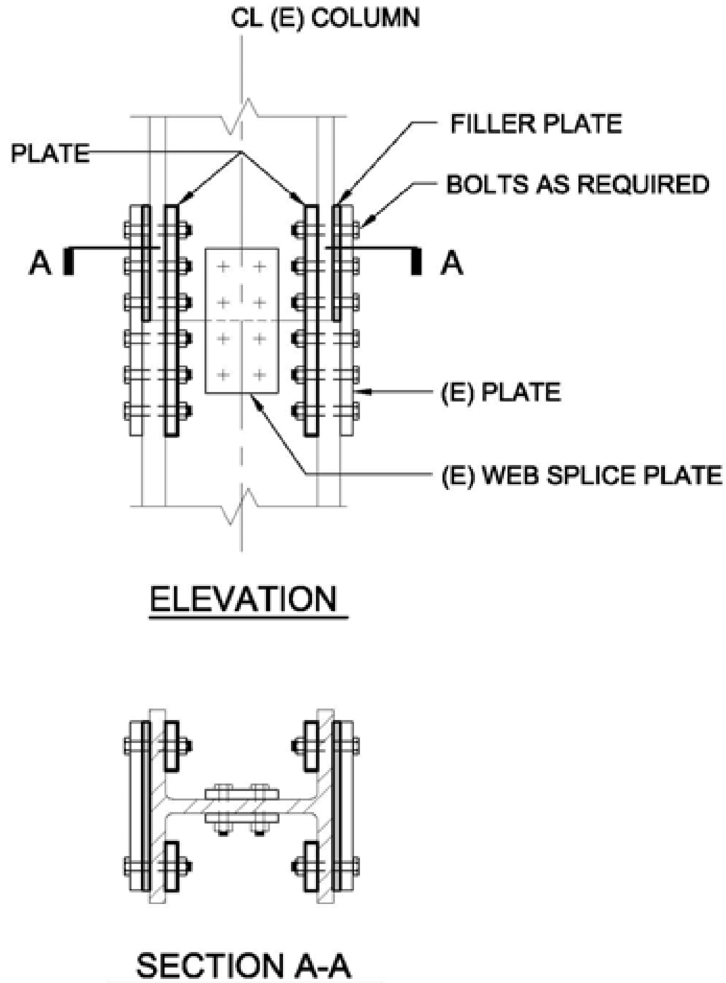



**Figure 8.4.7-2: Bolted Splice Upgrade at Existing Column**

*Removal of existing structural elements:* Due to the critical nature of columns, the removal of existing welds or bolts at a column should be minimized. Column alignment and stability should be maintained at all times.

*Construction loads:* See general discussion in Section 8.4.1. Typically, welding on a loaded column should not create a safety issue, although stability during construction should always be considered. At a minimum, see section in *AISC Steel Construction Manual* (2005c) on column splices and the *AISC Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005a) for other construction considerations.

***Proprietary Concerns***

There are no known proprietary concerns with this technique.

### 8.4.8 Add Steel Plate Shear Wall (Connected to An Existing Steel Frame)

***Deficiency Addressed by Rehabilitation Technique***

Moment frames buildings that are insufficient to resist lateral forces or too flexible to control building drifts can be strengthened and stiffened by adding steel plate shear walls (SPSW), as shown in Figure 8.4.8-1A. The shear walls may be used alone as the new lateral force-resisting system or in conjunction with the moment frames.

***Description of the Rehabilitation Technique***

Shear walls can add considerable strength and stiffness to a structure. It should be considered as an alternative to adding braced frames if additional stiffness is required. Compared to concrete shear walls, steel plate shear walls are lighter and add less seismic mass to a structure. They also take up less space and may be more economical to construct, particularly in taller buildings where the costs of delivering formwork and pumping concrete are significant. The behavior of this system is analogous to braced frames that rely on tension-only braces, as well as plate girders whereby tension fields develop along the diagonals. However, neither of these examples completely characterizes the behavior of an SPSW. The beams and columns in an SPSW behave as boundary elements that are subject to a complex array of forces and require a considerable amount of stiffness to develop the capacity of the steel panels. Large strut forces are imposed on the beams while columns are subject to axial, flexural, and shear forces. Thus, preference of shear wall locations should be given to existing bays of moment frames, in which the members and connections are less likely to require modifications for use in the SPSW system. The energy dissipating mechanisms in an earthquake include tension yielding and eventual tearing of the diagonal tension fields, shear yielding of the plates, compressive buckling of the plates along the diagonal compression fields, and slipping of the bolted connections to the fin plates when used.

In an existing building, fin plates at the wall boundaries would be field welded to the beams and columns first. Large steel panels would then be welded or bolted to these fin plates, as illustrated in Figure 8.4.8-1B. Panel splices could also be welded or bolted in the field. If welded splices are used, it is recommended to provide full penetration welds with the backing bars removed after welding. Openings are acceptable if stiffeners are provided around the edges. The panels themselves may be unstiffened or stiffened. Provisions for the design of this system have been included in the 2005 *AISC Seismic Provisions*. AISC is also in the process of publishing a design guide for unstiffened SPSW.

***Design Considerations***

Adding shear walls to a moment frame building increases its stiffness considerably. The upgraded structure should be evaluated for higher lateral and overturning forces accordingly. This system is almost always controlled by shear due to the substantial flexural strength provided by the steel column boundary elements.

*Research basis:* No references directly addressing the addition of SPSW to moment frame buildings have been identified. However, if the inelastic deformations can be limited to the steel plates, the seismic performance of the strengthened structure is not expected to differ from new

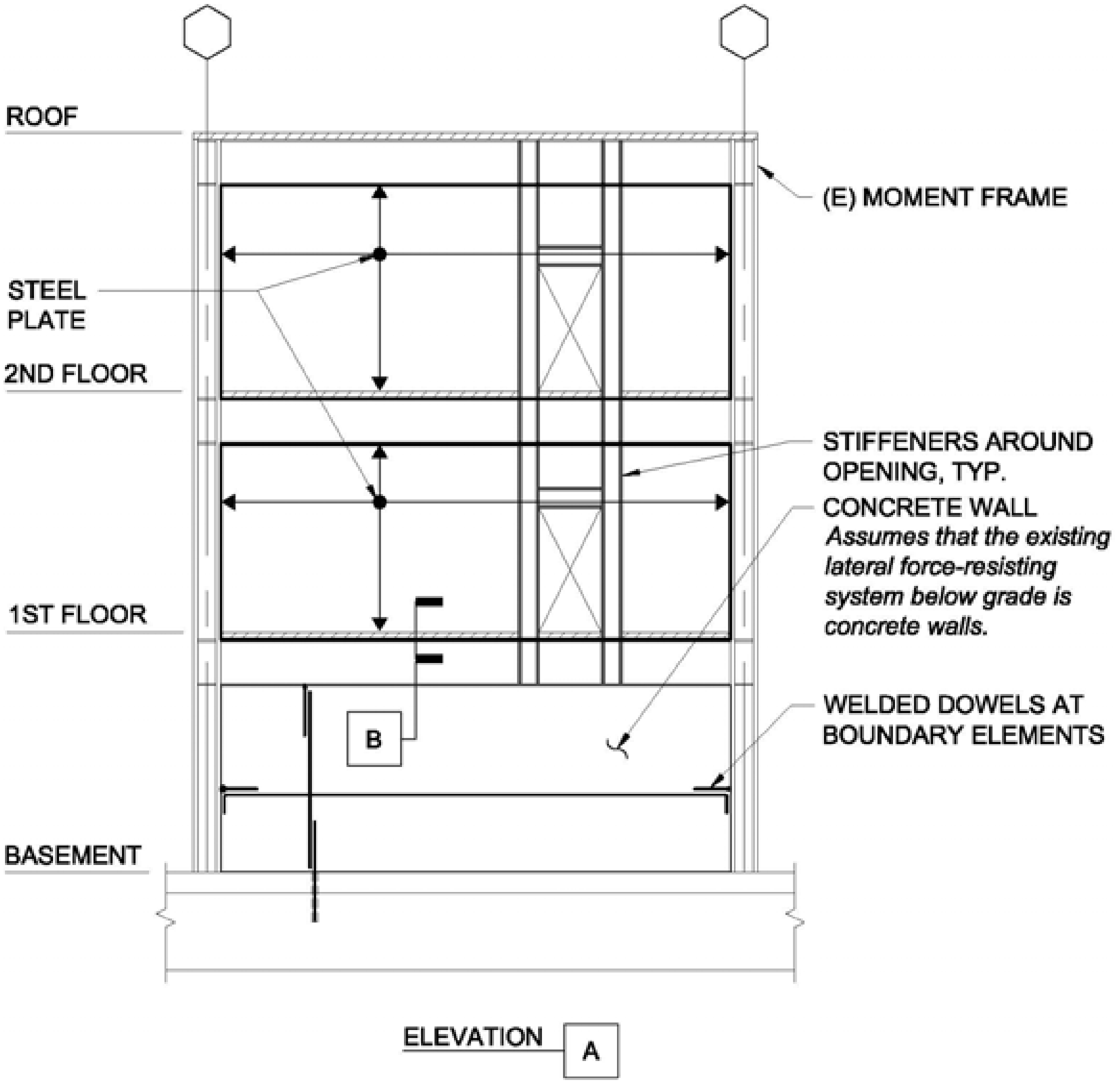



**Figure 8.4.8-1A: Unstiffened Steel Plate Shear Wall**

buildings utilizing SPSW. A summary of the major research on this system as well as its seismic performance in past earthquakes can be found in Astaneh-Asl (2001).

*Unstiffened vs. stiffened walls:* Stiffened walls tend to exhibit higher shear strength though both types of walls can be expected to exhibit ductile behavior. Buckling of the steel plates in unstiffened walls allow tension fields to develop and resist the lateral forces. Stiffeners, such as plates or channels, can be welded to the steel panels to prevent buckling of steel plates. These walls are more likely to yield in shear instead of developing tension fields. The use of stiffeners also permits the panels themselves to be thinner than panels in unstiffened walls. The panels in stiffened wall participate in resisting the overturning forces because buckling does not occur in the panels. Consequently, overturning forces in unstiffened walls are primarily resisted by the columns.

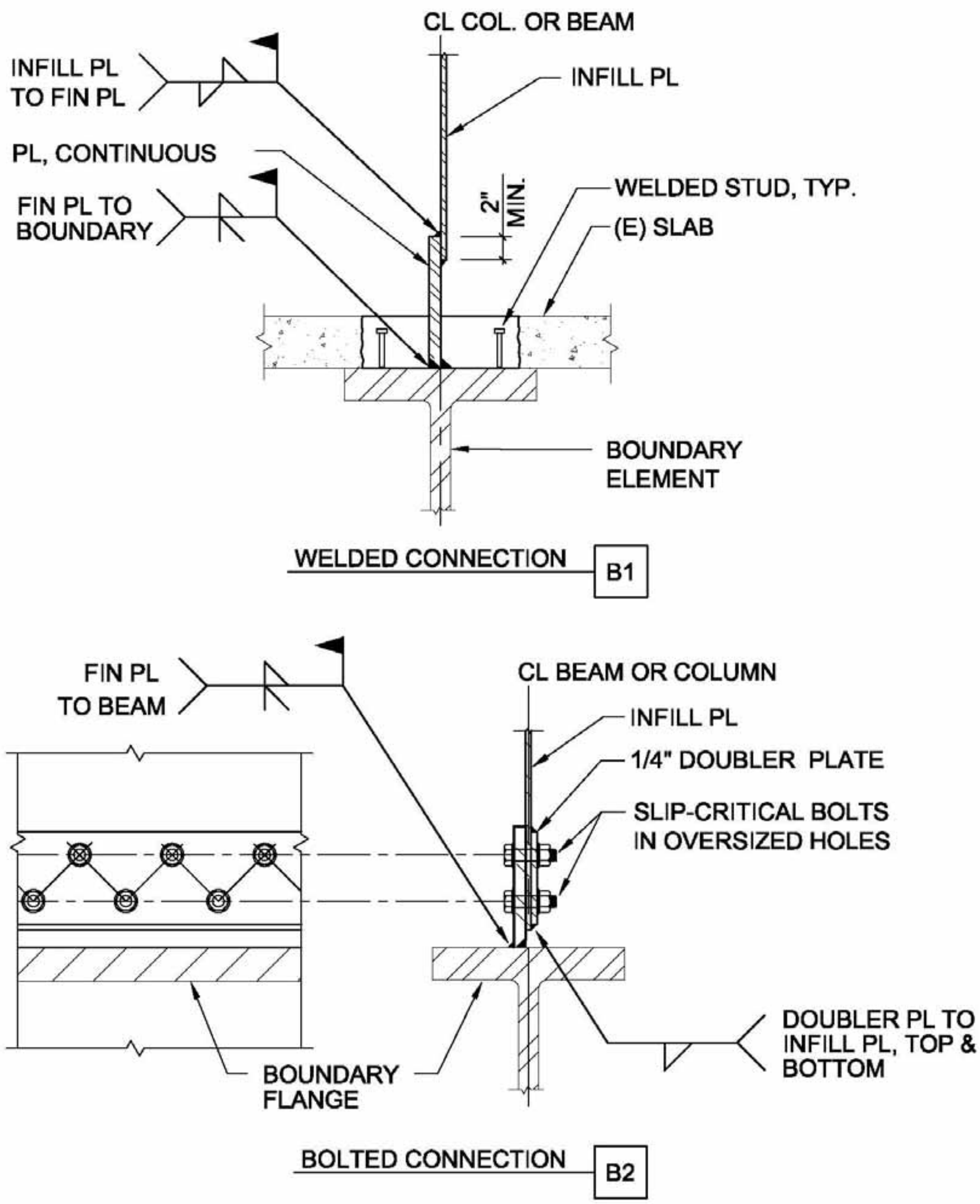



**Figure 8.4.8-1B: Fin Plate Connection Options**

*Nonstructural issues:* The addition of walls to an existing structure changes the architectural character of the building. Walls at the exterior will be visible in buildings with clear glazing. At interior bays, walls have to be configured to avoid obstruction of existing corridors, doorways, and other building systems. Walls can be exposed and incorporated into the interior architecture or hidden in partition walls.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Steel connections:* Welds from the fin plates to beams and columns should develop the capacity of the steel panels through full penetration welds or fillet welds on both sides of the plates. To allow for construction and field tolerances, the steel panels can be lapped with the fin plates and connected using fillet welds along both edges of the lap.

*Connections at slabs:* To avoid damaging the slab reinforcing steel critical for transferring diaphragm forces to the shear walls, the concrete slab could be chipped away without damaging the slab steel and individual fin plates could be placed between the reinforcing steel. The diaphragm forces are transferred to the wall through shear studs on the beams. Alternatively, if continuous fin plates are required and/or the shear studs are inadequate, the reinforcing steel in could be cut and welded directly to the fin plates. In this case, forces transfer from the slab into the wall through shear friction.

### *Cost/Disruption*

As always, the cost of a strengthening scheme depends on the project and its unique requirements. There are no issues with the SPSW system that is known to cost significantly more than adding braced frames or concrete shear walls to a moment frame building. Unstiffened walls are cheaper and less labor intensive than stiffened walls. New foundations are almost always required for new walls and could be extremely costly if deep foundations, such as drilled piers, are added.

Installing new walls is disruptive to the occupants because of the noise and vibrations associated with construction. Even if tenants are relocated to parts of the building where the work is not being performed, vibrations associated with cutting, chipping, and drilling of concrete as well as the installation of steel panels can transmit through the structure. The disruption can be reduced somewhat if the walls are installed at the perimeter.

### *Construction Considerations*

See Section 8.4.1 for general discussions of welding issues, removal of existing nonstructural and structural elements, and construction loads.

### *Proprietary Concerns*

There are no known proprietary concerns with this technique.

## 8.4.9 Convert an Existing Steel Gravity Frame to a Moment Frame

### *Deficiency Addressed by Rehabilitation Technique*

This technique addresses moment frames buildings that only require slight gains in strength and/or stiffness. The extent of connection modification depends on the seismic hazard and the type of moment frame.

### *Description of the Rehabilitation Technique*

Converting existing gravity frame connections to moment frame connections does not increase the strength or stiffness of a structure significantly unless a large majority of the gravity frame column connections are made moment resisting. The strength and stiffness gain are also limited because the existing beams and columns used in the gravity frame are typically much lighter than the moment frame members. However, these members could also be strengthened as part of the rehabilitation scheme. This technique has less impact on the architectural character of the building than adding braced frames or shear walls.

The simplest method for implementing this technique is through the addition of welded flange cover plates from the beam to the column without any modification of the bolted shear tab. The beam flanges remain unattached from the column since the gap typically exceeds the maximum permitted root opening size for full penetration welds. This method should only be used with Ordinary Moment Frame applications.

In high seismic regions where moment frames have to meet Special Moment Frame requirements, a more sophisticated connection upgrade that forces the beam yielding to occur away from the connection should be provided. This could range from adding welds to the bolted shear tab for the method described above to welding top and bottom haunches from the beam to the column. Several methods are presented in Section 8.4.6 as well as FEMA 350 (FEMA, 2000a) and FEMA 351 (FEMA, 2000b). The level of upgrade ultimately depends on the expected ductility demand and the performance objective.

### *Design and Detailing Considerations*

The strong-column weak-beam concept should still be a primary design consideration. All detailing issues related to the design of moment frame connections need to be considered, including weld filler metal matching, weld metal toughness, removal of weld backing, and column flange and web reinforcing. For Ordinary Moment Frames, joint reinforcing should be limited in order to permit some yielding, as to not place excessive demands on the flange to column connections. For Special Moment Resisting Frames, other requirements such as width-thickness limitations, lateral bracing requirements, etc., should be checked to be in accordance with the *AISC Seismic Provisions*. In most cases, it is expected that the existing gravity beam and column configurations will be such that it will be difficult to meet the requirements for Special Moment Frames without other major modifications. As a result, it is expected that in most cases, the Ordinary Moment Frame requirements would apply.

### *Cost/Disruption*

These issues are discussed in Section 8.4.6 for moment connections. Connection upgrades are typically less disruptive than adding braced frames or shear walls. The costs could vary depending on if existing moment frame connections are also being upgraded and the total number of connections being modified.

### *Construction Considerations*

See Section 8.4.6 for general discussions of issues related to moment connections.

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# Chapter 9 - Building Types S2/S2A: Steel Braced Frames

## 9.1 Description of the Model Building Type

Building Type **S2** consists of a frame assembly of steel beams and columns. Lateral forces are resisted by diagonal steel members placed in selected bays. Floors are cast-in-place concrete slabs or concrete fill over metal deck. These buildings are typically used for buildings similar to steel moment frames, although more often for low-rise applications. Figure 9.1-1 shows an example of this building type.

Building Type **S2A** is similar but has floors and roof that act as flexible diaphragms such as wood or untopped metal deck. This is a relatively uncommon building type and is used primarily for small office or commercial buildings in which the fire rating of concrete floor is not needed.

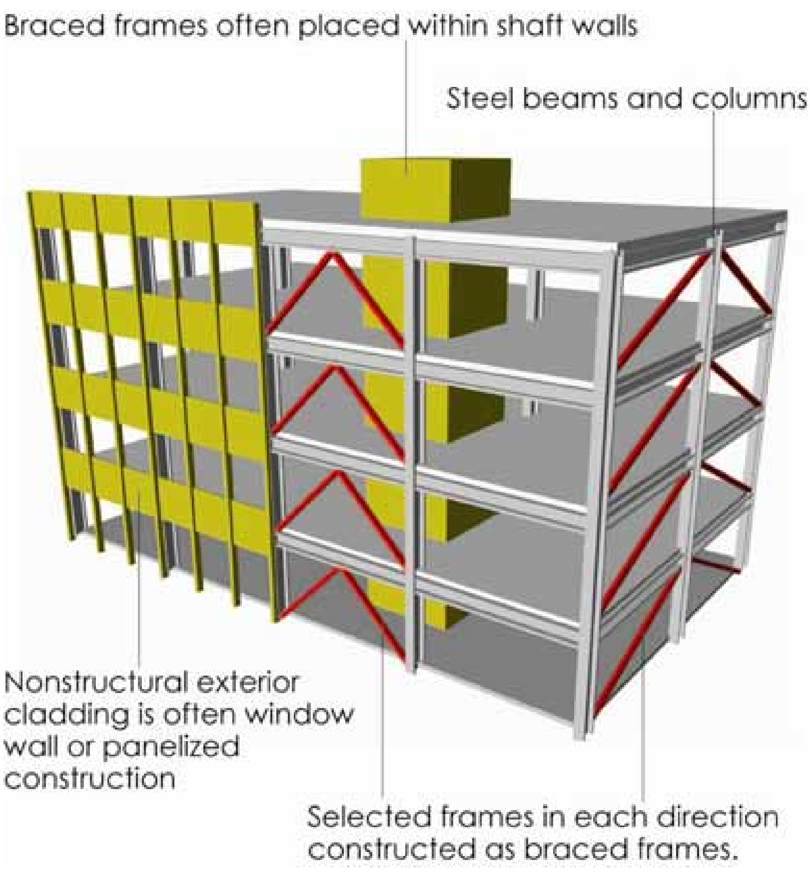


**Figure 9.1-1: Building Type S2: Steel Braced Frames**

### *Variations Within the Building Type*

The two principal types of braced frame configurations are concentrically braced frames (CBFs) and eccentrically braced frames (EBFs). In a CBF, the centerlines of members that meet at a joint all intersect at a single point. These frames behave as vertical truss systems by transferring lateral loads primarily through axial loading of beams, columns, and braces. During earthquakes, inelastic behavior is typically limited to the braces and connections. Common CBF configurations include diagonal bracing, X-bracing (or 2-story X-bracing), V-bracing (or inverted-V-bracing), and K-bracing. The diagonal braces in an EBF are offset at joints such that link beams separate the ends of braces from columns or other braces. Inelastic behavior is concentrated in the links while all members outside of the links remain elastic or near elastic. Link beams can be located adjacent to a column or at the center of a beam. Common types of braces include W-shapes, hollow structural sections (HSS), steel pipes, double angles, and double channels.

Braces are either welded directly to the beams and columns or welded to gusset plates. It is standard practice on the West Coast to weld gusset plates to the beams and columns. Away from the West Coast, the plate is more commonly welded to the beam and bolted to the column with a pair of angles or a WT-shape. Beam-column connections vary depending on whether there is a brace at the joint or not. Connections range from simple shear tabs to fully welded moment connections.

### *Floor and Roof Diaphragms*

Diaphragms associated with this building type may be either rigid or flexible. The typical rigid diaphragm found in modern buildings consists of structural concrete on metal deck. Diaphragm forces transfer to the frames through shear studs welded to the beams. Older steel buildings that were constructed before metal decks were commonly used may have concrete slabs or masonry arches that span between the beams. Flexible diaphragms include bare metal deck or metal deck with nonstructural fill. These are frequently used on roofs that support light gravity loads. Decks could be connected to the steel members with shear studs, puddle welds, screws, or shot pins. The steel members also act as chords and collectors for the diaphragm.

### *Foundations*

There is no typical foundation for this building type. Foundations can be of any type, including spread footings, mat footings, and piles, depending on the characteristics of the building, the lateral forces, and the site soil. Spread footings are used when lateral forces are not very high and a firm soil exists. For larger forces and/or poor soil conditions, a mat footing below the entire structure is commonly used. Pile foundations are used when lateral forces are extremely large or poor soil is encountered. The piles can be either driven or cast-in-place. Vertical forces are distributed to the underlying soil through a combination of skin friction between the pile and soil and/or direct bearing at the end of the pile; lateral forces are resisted primarily through passive pressure on the vertical surfaces of the pile cap and piles.

## 9.2 Seismic Performance Characteristics

Braced frames are generally considered to be stiff systems in the elastic range. Their nonlinear response depends on their ability to redistribute forces between bays and drifts between stories.

Braces and connections in CBF undergo large inelastic deformations in tension and compression into the post-buckling range. Ductility of CBF systems in past earthquakes have been limited by local failures of braces and connections. It is thought that CBF systems that are properly designed and detailed can possess ductility in excess of that previously assigned to these systems (AISC, 2005). Yet, recent experimental testing at UC Berkeley found that special concentrically braced frames (SCBF), designed with an inverted-V configuration using the 1997 *AISC Seismic Provisions*, fractured the HSS braces after only a few cycles of loading in the inelastic range (Uriz and Mahin, 2004). Also, the damage concentrated at the level that first experienced brace buckling, resulting in weak story response. As of this writing, these results are still under investigation.

EBF systems approach the higher performance levels of structural systems such as buckling-restrained braced frames and fluid or friction damped frames (Horne et al., 2001). By limiting nonlinear action to the link beams, the post-yield behavior of the system and the maximum demand on the frame can be better predicted. The ductility of an EBF system is dependent on the length and detailing of the link beams and whether shear or flexural yielding governs their inelastic response.

## 9.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

Undesirable behaviors of CBF systems that have been observed in past earthquakes include fracture of connection elements, fracture of braces, and local buckling of braces. These failures, in turn, cause excessive demands on other elements in the system and lead to overall frame failure. See Table 9.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Selected deficiencies are further discussed below by category.

### *Global Strength*

The lack of global strength to resist the seismic demands is a direct result of weak frames. In some cases, the existing beams and columns in a frame may possess enough capacity to accommodate upgrades to the braces and connections. If the beams and columns cannot make this accommodation, braces can be added to other bays to create new frames and thereby, reduce the demand on the existing frames. This would be more easily achieved for CBF systems than for EBF systems, which have special link beam requirements (AISC, 2005).

### *Global Stiffness*

Braced frames are extremely stiff in the elastic range. Concerns with global stiffness occur in the inelastic range when braces are prone to buckling and cause a loss of stiffness. Limited post-elastic stiffness is potentially provided by the frame and non-frame columns in the system (Tang and Goel, 1987; Hassan and Goel, 1991).

### *Configuration*

*Concentrically braced frames:* K-bracing and inverted-V-bracing configurations exhibit undesirable behavior when a brace buckles. The remaining tension brace at a joint imparts an unbalanced force onto the column or beam. This is particularly hazardous for frames with K-bracing since it may cause a column to fail entirely. In frames that use inverted-V-bracing, the unbalanced force is additive to gravity loads on beams.

| Table 9.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S2/S2A Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient frame strength | Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | Strengthen braces [9.4.2], beams [8.4.3], columns [8.4.3], and/or connections [9.4.1] | | Seismic isolation [24.3]<br>Supplemental damping [24.4] | |
| Global Stiffness | Excessive drift | Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | Strengthen braces [9.4.2], beams [8.4.3], columns [8.4.3], and/or connections [9.4.1] | | Supplemental damping [24.4] | |
| Configuration | Soft story | Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Steel plate shear wall [8.4.8] | | | | |
| | Re-entrant corner | Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Collector [8.4.4] | Enhance detailing [8.4.3], [8.4.4] | | | |
| Load Path | Missing collector | Collector [8.4.4] | | | | |
| | Inadequate shear, flexural, and uplift anchorage to foundation | | Embed column into a pedestal bonded to other existing foundation elements [8.4.5] | Provide steel shear lugs or anchor bolts from base plate to foundation [8.4.5] | | |
| | Inadequate out-of-plane anchorage at walls connected to diaphragm | | | Tension anchors [16.4.1] | | |

| **Table 9.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S2/S2A Buildings** | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing | Inadequate capacity of braces and/or connection | Replace braces [9.4.1] | Increase area of braces [9.4.2]<br>Make braces composite elements [9.4.2]<br>Improve b/t ratios [9.4.2] | Add bolts and welds [9.4.1]<br>Increase size of gusset plates [9.4.1] | | |
| | Inadequate capacity of beams, columns, and/or connections | | Add cover plates or box members [8.4.3]<br>Provide gusset plates or knee braces [9.4.1] | Provide gusset plates [9.4.1] | | |
| | EBFs not conforming to current standards | | Check current EBF design standards [9.4.2] | | | |
| | Inadequate capacity of horizontal steel bracing | Provide additional secondary bracing [9.4.2] | Strengthen bracing elements [9.4.2]<br>Reduce unbraced lengths [9.4.2] | Strengthen connections [9.4.1] | | |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Collectors to distribute forces [8.4.4]<br>Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2] | Concrete topping slab overlay<br>Wood structural panel overlay at flexible diaphragms [22.2.1]<br>Strengthen chords [8.4.3], [8.4.4], and [22.2.2] | Add nails at flexible diaphragms [22.2.1] | | |
| | Inadequate shear transfer to frames | | | Provide additional shear studs, anchors, or welds [22.2.7] | | |
| | Inadequate chord capacity | Add steel members or reinforcement [8.4.3], [8.4.4] | | | | |

| Table 9.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S2/S2A Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Diaphragms (continued) | Excessive stresses at openings and irregularities | Add reinforcement [8.4.3]<br>Provide drags into surrounding diaphragm [8.4.4] | | | | Infill opening [22.2.4], [22.2.6] |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

*Eccentrically braced frames:* The inelastic response of a link beam is influenced by its length relative to the ratio $M_P/V_p$ of the link section (AISC, 2005). Link beams that exhibit shear yielding have greater inelastic deformation capacity than ones that exhibit flexural yielding. *AISC Seismic Provisions* (AISC, 2005) permit shear yielding links to have four times the plastic rotation angles of flexural yielding links. Configurations with link beams adjacent to columns are susceptible to weld fractures found in pre-Northridge connections unless special detailing measures are taken to reduce the demands on these welds.

### Load Path

Load path deficiencies in steel braced frame buildings include inadequate connections, collectors, and frame anchorage to foundations. Brace connections may have less strength than the braces. In Type **S2** buildings, seismic forces transfer from the diaphragm to the frame through shear studs welded to collectors or directly to the frame beams. The collectors or the connections to the frame may be too weak to transfer these forces. Connections from columns to base plates or pile caps must resist shear, flexural, and potential uplift forces. Connections that cannot develop these frame forces prevent the frame from developing its full capacity.

### Component Detailing

*CBF:* HSS and pipe braces with high b/t ratios and other steel shapes that lack compactness are subject to local buckling or fracture after a limited number of inelastic cycles. Ductility of these braces can be improved by infilling with concrete or adding longitudinal stiffeners. Alternatively, the braces can be replaced with double HSS sections, which can be used in configurations similar to double angles or channels (Lee and Goel, 1990). HSS and pipe braces are also subject to net section fracture at the gusset plate slots (Uriz and Mahin, 2004). This deficiency can be mitigated by adding reinforcing plates to the sides of the HSS without the slots, as shown in Figure 8.4.1-1 and Figure 8.4.1-2. For pipes, the reinforcing plates can be oriented at right angles to the pipe and appear like stiffeners. Brace connections that are only designed for the axial capacity of the braces may not be adequate to generate the full strength of the braces. To achieve good post-inelastic response, all eccentricities in the connection must be considered. A brace that buckles in the plane of the gusset plates should have its end connections designed for the full axial load and flexural strength of the brace (AISC, 2005). A brace that buckles out-of-plane should ensure that the gusset plates can develop restraint-free plastic rotations without buckling.

*EBF:* Frames that rely on link-to-column connections have traditionally utilized similar detailing as pre-Northridge connections at beam-column joints. These connections should be reevaluated in the wake of the findings following the Northridge earthquake. Also, experimental research has found that link beams at the first floor undergo the largest inelastic deformation and have the potential to create a soft story (AISC, 2005).

### Diaphragm Deficiencies

Common diaphragm deficiencies include insufficient in-plane shear strengths, inadequate chords, and excessive stresses at openings. Causes for these deficiencies could be due to lack of slab or fill thickness, lack of reinforcing steel in the slabs, insufficient connections to chord elements, and poor detailing at openings. See Chapter 22 for common rehabilitation techniques.

### *Foundation Deficiencies*

Foundations that are inadequate do not develop the full capacity of the lateral force-resisting system. Their deficiencies result from insufficient strengths and sizes of footings, grade beams, pile caps, and piles. See Chapter 23 for common rehabilitation techniques.

## 9.4 Detailed Description of Techniques Primarily Associated with This Building Type

### 9.4.1 Enhance Braced Frame Connection

#### *Deficiency Addressed by Rehabilitation Technique*

Adequate capacities of connections are essential to the proper performance of a braced frame. Connections with insufficient strength and/or ductility to develop stable inelastic frame behavior are strengthened or replaced.

#### *Description of the Rehabilitation Technique*

Brace end connections commonly rely on additional connection elements (e.g., gusset plates), but the braces may also be directly welded to the beam and column through a moment connection. Moment connections are also used for link-to-column connections in an EBF and sometimes used for beam-to-column connections in both EBF and CBF systems. The mitigation approach is different depending on whether the existing connection relies on a connection member or a moment connection.

If the existing connection members have sufficient capacity, the most economical alternative may be to increase the connection capacity by providing additional welds or bolts. This typically only allows for a limited increase in capacity since existing brace connection configurations can rarely accommodate significant modifications. If the existing connection members have inadequate capacity, the existing configuration and accessibility need to be assessed to determine whether adding supplemental connection members or replacing the existing connection members with members of greater capacity is more economical. Supplementing the existing connection eliminates the challenges associated with removal of existing connection welds and temporary support of the braces.

The primary concern with moment connections used in braced frames is the use of low notch toughness weld metals. For braces that are expected to develop plastic hinges at their ends, consider replacing the existing welds with a high notch toughness weld metal. Beam-to-column connections in CBF do not typically experience large flexural forces and likely do not need to be upgraded. An exception occurs when the braces or their end connections fail and frame action becomes the primary mechanism for lateral force resistance; however, this is not a recommended design approach, as it does not ensure stable post-elastic behavior. In EBF configurations where the link beam is adjacent to the column, an upgrade should be considered given the large demands on the link beam and its critical nature.

### *Design Considerations*

*AISC Seismic Provisions* does not permit the sharing of loads by both welds and bolts on the same faying surface. Thus, a bolted brace to gusset plate connection should only be enhanced with bolts or replaced entirely with welds. Note it is not uncommon in some regions for a gusset plate to be bolted to the column through a shear plate and welded to the beam since these are separate faying surfaces. In addition to having thorough knowledge of the existing material behaviors to ensure that the new and existing elements to interact in the desired manner, other design issues include the following:

*Research basis:* No references directly addressing upgrades of braced frame connections have been identified.

*Design forces:* Brace connections that are only designed for the axial capacity of the braces may not be adequate to generate the full strength of the braces. A brace that buckles in the plane of the gusset plates should have its end connections designed for the full axial load and flexural strength of the brace (Astaneh-Asl et al., 1986). This recommendation may be more appropriate for high seismic applications.

*Bolted connections:* Bolts that govern the existing connection capacity could be replaced with stronger bolts, such as replacing A325 with A490. The upgrade may also be as simple as replacing bolts that have threads included in the shear plane with longer bolts. More bolts could be added if the existing configuration allows for it. Larger bolts could be provided but this would reduce the net section capacities due to enlargement of the holes.

*Welded connections:* Existing fillet welds can be thickened provided the welds are not of low notch toughness weld metals or found to be inadequate through material testing. Otherwise, the existing welds should be removed and replaced with high notch toughness weld metals. A typical fully welded connection appropriate for use in SCBF is shown in Figure 8.4.1-1. For low to moderate seismic applications, Figure 8.4.1-2 shows a more compact connection. Welded brace connections to the weak axis of columns are complex and expensive; an example of this type of connection is shown in Figure 8.4.1-3.

*Moment connections:* As mentioned above, moment connections that are subject to large flexural forces should be upgraded with high notch toughness weld metals. This would primarily apply to braces that directly connect to beams and columns in SCBF and some OCBF in high seismic applications. EBF link beams that are adjacent to columns also fall into this category. Link beams develop large flexural forces whether shear or flexural yielding governs.

*Nonstructural issues:* Gusset plates designed in accordance with the *AISC Seismic Provisions* for SCBF can be fairly large and should be discussed with the architect and tenants.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Gusset plates:* A brace in a SCBF that buckles out-of-plane could form plastic hinges at midspan and in the gusset plates at each end. The gusset plates should provide restraint-free plastic rotations without buckling. *AISC Seismic Provisions* suggests a minimum distance of two times the plate thickness between the end of the brace and the assumed line of restraint to achieve this. Consider increasing this distance to three times the plate thickness to accommodate over cutting of the slots in HSS and other erection tolerances. Connections that do not allow for restraint-free plastic rotations out-of-plane should be used primarily in situations where in-plane buckling of the braces govern or ductility demands are low.

*Bolted connections:* If new holes are drilled in the existing brace and connection member, tolerances of the existing holes should be verified to ensure that bolts will be loaded evenly.

*Weld filler metal matching and overmatching:* Weld filler metals with slightly greater tensile strength than the connected steel should be used. Flux cored arc welding and shielded metal arc welding electrodes that conform to E70 specifications exhibit overmatching properties compared to common steel specifications, including ASTM A36, A572 (Grade 42 and 50), A913 (Grade 50), and A992 (FEMA, 2000b).

*Weld metal toughness: AISC Seismic Provisions* now require weld metals with minimum Charpy V-Notch (CVN) toughness of 20 ft-lbf at -20ºF for all welds in the lateral force-resisting system. There is also an additional requirement of 40 ft-lbs at 70ºF for demand critical welds.

#### ***Cost/Disruption***

Connection modifications are locally very disruptive. Noise associated with this type of work will spread and disrupt tenants on other floors unless the work is done during off-hours. These modifications could be particularly costly if existing gusset plates have to be replaced.

#### ***Construction Considerations***

See Section 8.4.1 for general discussions of welding issues, removal of existing nonstructural and structural elements, and construction loads.

#### ***Proprietary Concerns***

There are no known proprietary concerns with this technique.

### 9.4.2 Enhance Strength and Ductility of Braced Frame Member

#### ***Deficiency Addressed by Rehabilitation Technique***

Inadequate beams, columns, and braces are strengthened or replaced to achieve ductile frame behavior.

#### ***Description of the Rehabilitation Technique***

The ductility of an existing brace can be enhanced by reducing its slenderness, which can be accomplished by decreasing its unbraced length, infilling hollow sections with concrete, or adding longitudinal stiffeners. The unbraced length of a brace can be reduced by adding secondary bracing members that are not part of the primary lateral force-resisting system. Infilling existing hollow sections with concrete can reduce the severity of local buckling (Liu and

Goel, 1988; Lee and Goel 1987). An effective width-thickness ratio for the infilled member is determined by multiplying the width-thickness ratio of the section by the factor (0.0082 x KL/r + 0.264), applicable to braces with KL/r values between 35 and 90 (Goel and Lee, 1992). Adding longitudinal stiffeners presents the least field complications; the stiffeners could consist of plates or small angle sections.

If both strength and ductility are required, new braces have to be added. Some configurations may lend themselves to schemes that allow the existing braces to remain. These include single angle, double angle, and channel braces that can be doubled; rolled sections can also be cover plated. In other cases, it is more practical to replace the existing brace with a new brace, of which numerous options exist. The increase in brace strength may require upgrades to other components of the braced frame, such as the brace connections, beams, and columns. Connection upgrades are discussed in the Section 9.4.1. In many cases, the most cost-effective alternative for increasing the capacity of the existing beams and columns in is to add cover plates or side plates to create box sections. This technique is discussed in Section 8.4.3.

### *Design Considerations*

It would be preferable to limit the strengthening of the existing braces to the capacity of the other members of the lateral force-resisting system, including the foundations, to avoid triggering too many upgrades. Thorough knowledge of the existing material behaviors and strengths are necessary for the new and existing elements to interact in the desired manner. Other design issues include the following:

*Research basis:* No references directly addressing upgrades of braced frame members have been identified.

*Existing brace strengthening:* Significant modifications to an existing brace could trigger strengthening or redesign of its end connections. Strengthening of existing K- or inverted-V-bracing should be undertaken only after careful evaluation of the additional bending forces following the buckling of the compression bracing. Where the existing bracing in these systems is found to have inadequate capacity, the preferred solution is to replace it with a diagonal or X-bracing configuration.

*Secondary bracing:* A brace member is designed to resist both tension and compression forces, but its capacity for compression stresses is limited by potential buckling and is therefore less than the capacity for tensile stresses. Since the design of the system generally is based on the compression capacity of the brace, some additional capacity may be obtained by simply reducing the unsupported length of the brace by means of secondary bracing provided the connections have adequate reserve capacity or can be strengthened for the additional loads.

*New brace selection:* If existing braces are replaced, use compact and non-slender sections whenever possible to avoid premature fracturing or buckling of the braces during post-yield behavior. Two particular brace types not common in older braced frame buildings are double HSS sections and buckling-restrained braces. Double HSS sections can be used in configurations similar to double angles or channels (Lee and Goel, 1990). They provide reduced fit-up issues and smaller width-to-thickness ratios compared to a single HSS, resulting in increased energy dissipation capacity. The other type of brace, used in a buckling-restrained braced frame (BRBF), is typically used in new buildings but has also been used successfully in new BRBF systems in existing buildings. One example of a brace used in a BRBF consists of a steel core inside a casing, which consists of a hollow structural section (HSS) infilled with concrete grout. Proprietary materials separate the steel core and concrete to prohibit bonding between the two materials. There are other buckling-restrained braces that do not use grout or additional separating agents between the steel and grout. The main advantage of these braces is the ability of the casing to restrain the buckling of the steel core without providing any additional axial force resistance beyond the capacity of the steel core. Provisions for new building BRBF design are included in the *NEHRP Recommended Provisions* (FEMA, 2003) and the *AISC Seismic Provisions for Structural Steel Buildings*. Note that significant connection modifications may be required when braces are replaced.

*Nonstructural issues:* Brace modifications, when exposed, will affect the interior architecture or if hidden in partition walls, these walls may be thicker than typical walls. Beams that are increased in size affect nonstructural components by reducing clear floor heights. These components typically include suspended ceilings, pipes, conduits, and ducts. Coordination with the architect and other trades should not be overlooked or underestimated.

### *Detailing Considerations*

In addition to obtaining the latest drawings for the building including as-built drawings, if available, and conducting comprehensive field surveys, the following issues should be noted:

*Built-up brace members:* While double angles, double channels, and double HSS offer advantages for installation, special criteria apply to these members when used in a SCBF. Buckling of these types of braces imposes large shear forces on the stitches. Therefore, closer stitch spacing and higher stitch strengths are required. More stringent member compactness is also necessary for ductility and energy dissipation.

*Reinforcing cover plates:* HSS and pipe braces are subject to net section fracture at the gusset plate slots (Uriz and Mahin, 2004). This brittle failure mode can be eliminated by adding reinforcing cover plates to the sides of the HSS without the slots, such as the ones shown in Figures 8.4.1-1 and 8.4.1-2. For pipes, the reinforcing plates can be oriented at right angles to the pipe and appear like stiffeners. Additional information regarding the design of these plates can be found in *Limiting Net Section Fracture in Slotted Tube Braces* (Yang and Mahin, 2005).

### *Cost/Disruption*

Designs that are simple and details that are not overly complicated will minimize costs associated with this technique. This could include maximizing the use of existing members, minimizing connection upgrades, and reducing the amount of field welding.

Costs can also be reduced if disruption is minimal during construction. Installing braces at the perimeter frames reduces logistical issues associated with working in confined spaces and temporary removal of the nonstructural elements. Noise associated with this type of work is loud and disturbing to the tenants if the building is occupied while the work is being performed.

***Construction Considerations***

See Section 8.4.1 for general discussions of welding issues, removal of existing nonstructural and structural elements, and construction loads. Connecting members, such as gusset plates, that are not being replaced should be protected when braces are removed.

***Proprietary Concerns***

Braces used in buckling-restrained braced frames (BRBF) are proprietary. There are a limited number of manufacturers of the braces used in BRBF.

## 9.5 References

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# Chapter 10 - Building Type S4: Steel Frames with Concrete Shear Walls

## 10.1 Description of the Model Building Type

Building Type **S4** consists of an essentially complete frame assembly of steel beams and columns. The floors are concrete slabs or concrete fill over metal deck. These buildings feature a significant number of concrete walls effectively acting as shear walls, either as vertical transportation cores, isolated in selected bays, and/or as a perimeter wall system. The steel column and beam system may act only to carry gravity loads or may have rigid connections to act as a moment frame to form a dual system. This building type is generally used as an alternate for steel moment or braced frames in similar circumstances. These buildings will usually be mid- or low-rise. Figure 10.1-1 shows an example of this building type.

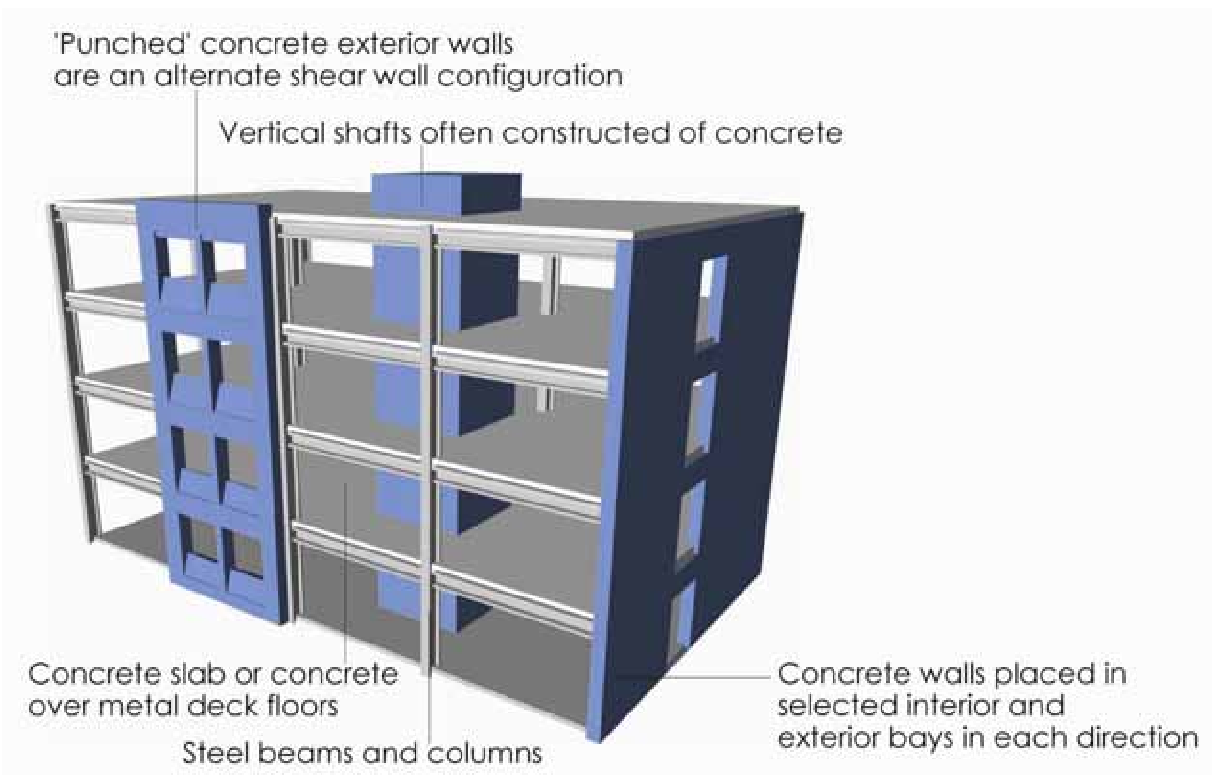



**Figure 10.1-1: Building Type S4: Steel Frames with Concrete Shear Walls**

## 10.2 Seismic Performance Characteristics

In older buildings, the steel frame carries only gravity loads while all lateral loads are resisted by the concrete shear walls. In modern buildings, both lateral systems work together in proportion to relative rigidity. Generally, except in tall buildings, these systems tend to behave more like shear wall structures due to the much greater stiffness of the walls. The contribution of the steel moment frame to the lateral capacity of the building is a function of the number of frames and the detailing of the beam-column joints. See performance characteristics described in Section 14.2 for concrete shear wall buildings and Section 8.2 for steel moment frame buildings.

## 10.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See deficiencies and techniques described in Section 13.3 and 14.3 for concrete shear wall buildings and Section 8.3 for steel moment frame buildings.

## 10.4 Detailed Description of Techniques Primarily Associated with This Building Type

See recommended techniques in Section 13.4 for concrete shear wall buildings and Section 8.4 for steel moment frame buildings.

## 10.5 References

See references in Section 13.5 and 14.5 for concrete shear wall buildings and Section 8.5 for steel moment frame buildings.

**Table 10.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S4 Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane wall shear strength | Concrete/masonry shear wall [8.4.2]<br>Braced frame [8.4.1] | Concrete wall overlay [21.4.8]<br>Fiber composite wall overlay [13.4.1]<br>Steel overlay | | Seismic isolation [24.3]<br>Reduce flexural capacity [13.4.4] | |
| | Insufficient flexural capacity | Concrete/masonry shear wall [8.4.2]<br>Braced frame [8.4.1] | Add or enhance chords | | | |
| | Insufficient frame strength | Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2] | Strengthen beams [8.4.3], columns [8.4.3], and/or connections [8.4.6] | | Seismic isolation [24.3] | |
| Global Stiffness | Excessive drift | Concrete/masonry shear wall [8.4.2]<br>Braced frame [8.4.1]<br>Moment frame | Strengthen beams [8.4.3], columns [8.4.3], and/or connections [8.4.6]<br>Concrete wall overlay [21.4.5] | | | |
| | Inadequate capacity of coupling beams | Concrete/masonry shear wall [8.4.2]<br>Braced frame [8.4.1] | Strengthen beams [13.4.2]<br>Improve ductility of beams [13.4.2] | | | Remove beams |
| Configuration | Discontinuous walls | Concrete/masonry shear wall [8.4.2] | Enhance existing column for overturning loads | Improve connection to diaphragm [13.4.3] | | Remove wall |
| | Soft story | Concrete/masonry shear wall [8.4.2]<br>Braced frame [8.4.1] | | | | |
| | Re-entrant corner | Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2]<br>Collector [8.4.4] | Enhance detailing [8.4.3], [8.4.4] | | | |

**Table 10.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S4 Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Torsional layout | Add balancing walls [8.4.2], braced frames [8.4.1], or moment frames | | | | |
| Load Path | Missing collector | Add collector [8.4.4] | Strengthen existing beam [8.4.3] or slab<br>Enhance splices or connections of existing beams [8.4.4] | | | |
| | Discontinuous Walls | Provide new wall support components to resist the maximum expected overturning moment | Strengthen the existing support columns for the maximum expected overturning moment [8.4.3]<br>Provide elements to distribute the shear into the diaphragm at the level of discontinuity [13.4.3] | | | |
| | Inadequate shear, flexural, and uplift anchorage to foundation | | Embed column into a pedestal bonded to other existing foundation elements [8.4.5] | Provide steel shear lugs or anchor bolts from base plate to foundation [8.4.5] | | |
| | Inadequate out-of-plane anchorage at walls connected to diaphragm | | | Tension anchors [16.4.4] | | |

**Table 10.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S4 Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing | Wall inadequate for out-of-plane bending | Add strongbacks [21.4.3] | Concrete wall overlay [21.4.5] | | | |
| | Wall shear critical | | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [13.4.1] | | Reduce flexural capacity of wall [13.4.4] | |
| | Inadequate capacity of beams, columns, and/or connections | | Enhance beam-column connections [8.4.6]<br>Add cover plates or box members [8.4.3]<br>Provide gusset plates or knee braces [9.4.1]<br>Encase columns in concrete [8.4.2] | | | |
| | Inadequate capacity of panel zone | | Provide welded continuity plates [8.4.6]<br>Provide welded stiffener or doubler plates [8.4.6] | | | |
| | Inadequate capacity of horizontal steel bracing | Provide additional secondary bracing [9.4.2] | Strengthen bracing elements [9.4.2]<br>Reduce unbraced lengths [9.4.2] | Strengthen connections [9.4.1] | | |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Collectors to distribute forces [8.4.4]<br>Moment frame<br>Braced frame [8.4.1]<br>Concrete/masonry shear wall [8.4.2] | Concrete topping slab overlay<br>Strengthen chords [8.4.3], [8.4.4] | | | |

| Table 10.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S4 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| | Inadequate shear transfer to frames | | | Provide additional shear studs, anchors, or welds [22.2.7] | | |
| Diaphragms (continued) | Inadequate chord capacity | Add steel members or reinforcement [8.4.3], [8.4.4] | | | | |
| | Excessive stresses at openings and irregularities | Add reinforcement [8.4.3] Provide drags into surrounding diaphragm [8.4.4] | | | | Infill opening [22.2.4], [22.2.6] |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

# Chapter 11 - Building Types S5/S5A: Steel Frames with Infill Masonry Shear Walls

## 11.1 Description of the Model Building Type

Building Type **S5** is normally an older building that consists of an essentially complete gravity frame assembly of steel floor beams or trusses and steel columns. The floor consists of masonry flat arches, concrete slabs or metal deck and concrete fill. Exterior walls, and possibly some interior walls, are constructed of unreinforced masonry, tightly infilling the space between columns and between beams and the floor such that the infill interacts with the frame to resist lateral movement. Windows and doors may be present in the infill walls, but to effectively act as a shear resisting element, the infill masonry must be constructed tightly against the columns and beams. The steel gravity framing in these buildings may include truss spandrels or knee braces on the exterior walls, or partially restrained beam-column connections in a more extensive pattern. The steel frame also is often cast in concrete for fireproofing purposes. The buildings intended to fall into this category normally feature exposed clay brick masonry on the exterior and are common in commercial areas of cities with occupancies of retail stores, small offices, and hotels. Figure 11.1-1 shows an example of this building type.

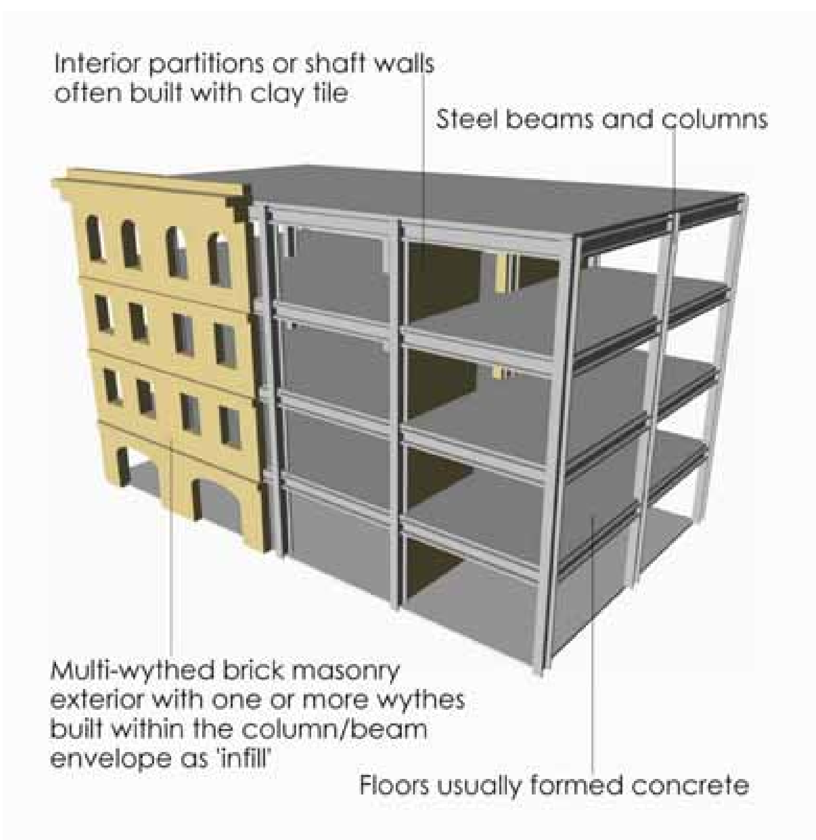



**Figure 11.1-1: Building Type S5: Steel Frames with Infill Masonry Shear Walls**

The **S5A** building type is similar but has floors and roof that act as flexible diaphragms such as wood, or untopped metal deck. This type of building will almost always date to the 1930 or earlier.

### *Variations Within the Building Type*

The building type was identified primarily to capture the issues of interaction between unreinforced masonry and steel gravity framing. The archetypical building has solid clay brick at the exterior with one wythe of brick running continuously past the plane of the column and beam and two or more wythes infilled within the plane of the column and beam. The exterior wythe of clay brick forms the finish of the building although patterns of terra cotta, stone, or precast concrete may be attached to the brick or laid up within the brick. However, there can be many variations to this pattern depending on the number and arrangement of finished planes on the exterior of the building. For example, the full width of the infill wall may be located with the plane of the column and beam with a pilaster built out and around the column and a horizontal band of brick or other material covering the beam. The beam is often placed off center of the column, usually on the out-board side. In extreme conditions, the primary plane of the masonry wall may not directly engage the column at all. In these cases, strut compression must be transferred eccentrically through the masonry surrounding the column, reducing effectiveness.

In some buildings the steel frame is encased in concrete, primarily for fireproofing. This encasement is normally reinforced with mesh and may contribute to overall frame stiffness and to connection strength and stiffness of partially restrained steel connections. Importantly, at the perimeter frames, the concrete encasement forms a smooth surface at the masonry interface and probably encouraged a neater fit during construction. Concrete encasement of columns also will assist in transferring eccentric strut loading into the column-frame system

Hollow clay tile masonry may also be used as an exterior infill material. Although this material often has a very high compression strength, the net section of material available to form the compression strut within the frame will normally contribute a lateral strength of only a small percentage of the building weight. The material being brittle and the wall being highly voided, these walls may also lose complete compressive strength quite suddenly. Therefore, walls of hollow clay tile infill will probably not contribute a significant portion of required lateral resistance except in areas of low seismicity and/or when walls are arranged as infill on both the exterior and interior of the building.

More recent buildings may have unreinforced concrete block masonry configured as an exterior infill wall, with a variety of finish materials attached to the outside face of the concrete block. Similar to hollow clay tile walls, these walls may exhibit moderate to low compressive strength and brittle behavior that marginalizes their usefulness as lateral elements. In addition, hollow concrete block exterior walls often will not be installed tight to the surrounding framing, eliminating infill compression strut behavior.

### *Floor and Roof Diaphragms*

The earliest version of this building type may include floors constructed of very shallow masonry arches spanning between steel beams. A relatively flat top surface is created with masonry rubble or light-weight cementitious fill and the floor is finished with wood sheathing. In some cases, the thrust from the arches is resisted by tie rods running perpendicular and through the steel beams. The only diaphragm action provided by such floors is the finish wood sheathing and the lateral flexibility of this system is incompatible with the stiff but brittle masonry arches.

Building Type **S5A** will have heavy timber floors with one or more layers of sheathing forming a diaphragm. The flexibility of such diaphragms will often form a seismic deficiency because, assuming no interior shear elements, the large drift at the diaphragm mid-span will damage perpendicular walls and gravity framing. Specific strengthening techniques for this building type are not covered here. For generalized strengthening of diaphragms, see Chapter 22.

Most typically, the floor and roof are cast-in-place concrete slabs spanning between beams. The concrete slab is often integral with lightly reinforced concrete surrounding each beam. This building type can also be found with metal deck and concrete floor slabs.

### *Foundations*

There is no typical foundation for this building type. Foundations can be found of every type depending on the height of the building, the span of the gravity system and the site soil. The exterior walls are exceptionally heavy and typically will be supported by a continuous concrete footing or often a continuous concrete wall forming a basement space below.

## 11.2 Seismic Response Characteristics

Most steel frame infill buildings will incorporate some beam column connections with moment resistance, either from top and bottom chord truss connections, knee bracing, or partially restrained tee or angle connections. The restraint is often enhanced by cast-in-place concrete cover. The lateral strength and stiffness of these systems is difficult to assess, although some testing has been done (Roeder et al., 1996). See also Abrams (1994). Unless the perimeter infill is penetrated with large openings, the frame will be far more flexible than the infill. Therefore, both in terms of stiffness and strength, the exterior infill walls typically will form the effective lateral system for this building type. The effectiveness of the system depends on the size and extent of openings and articulation of the plane of the wall. With solid or nearly solid infill panels, strut action will be stiff and strong. As openings in panels increase in size, struts or combinations of struts cannot effectively form around the opening and the steel columns and beams will begin to work as a moment frame, with “fixity” at the beam-column joint provided by the masonry. For low and moderate intensity shaking, the exterior walls may provide adequate strength to satisfy the specified performance objective. As the shaking demand increases, the masonry will tend to crack and spall, losing stiffness and potentially creating a falling hazard. The complete steel gravity system, characteristic of this building type, is generally expected to provide sufficient stability to prevent collapse, particularly if designed for lateral resistance. However, in configurations with large height-to-width ratios, end or corner columns could fail in compression or at tension splices, potentially leading to partial collapse.

This building type is often characterized by a commercial store-front first floor with little or no infill at that level on one or more faces of the building. This condition can cause a soft story condition or a severe torsional response if open on one or two sides only. Such conditions can lead to concentration of seismic deformation at the open level, potentially leading to local P-delta failure. This open commercial story was a common feature in many buildings of this type that were shaken in the 1906 San Francisco earthquake, but there were no story-mechanisms reported. It is speculated that the soft story provided isolation for the upper stories and that the displacement demand, for reasons unknown, did not exceed the story capacity. In fact, there have been no reports of collapses or damage that suggested imminent collapse in typical U.S. multistory office-like steel infill buildings in strong ground motion. Earthquakes providing such tests include the 1906 San Francisco, the 1933 Long Beach, and to a lesser extent, the 1994 Northridge events. In general, current seismic evaluation technology does not reach the same conclusion.

## 11.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 11.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Deficiencies related to steel moment frames and masonry shear walls are shown in Table 5.3-1 and Table 18.3-1, respectively. Selected deficiencies are further discussed below by category.

### *Global Strength*

The overall strength provided by the exterior walls may be insufficient to prevent serious degradation and resulting amplified displacements in the building that can lead to irreparable damage or even instability. The strength may be limited by inadequate number of panels of infill, excessive openings, or masonry weak in compressive strength. The standard approach to such deficiencies will be to add new, relatively stiff lateral force-resisting elements such as concrete shear walls or steel braced frames often located on the interior between existing columns. Concrete walls can also be added at the perimeter on the inside face of the masonry. This procedure is usually conceptualized and analyzed as a concrete shear wall rather than an infill to the frame.

Fiber composite layers also can be added to the face of masonry to enhance infill strut action. Although this technique has been tested for increasing shear strength of URM walls, little research is available directly on the effects of adding these layers to infill panels.

Unless the masonry is completely doweled or connected to supporting backing, the damage state of the masonry wall must be estimated for the expected drifts of the combined system to determine if the desired performance has been achieved.

### *Global Stiffness*

For this building type, the methods for adding stiffness are similar to those adding strength.

### *Configuration*

Two global configurational deficiencies are common in this building type. The first is a soft and weak story at the street level created by commercial occupancies with exterior bays with little or

**Table 11.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S5/S5A Buildings**

| **Deficiency** | | **Rehabilitation Technique** | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Inadequate length of exterior wall | Interior concrete walls [8.4.2]<br>Interior steel braced frames [8.4.1] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6] | | | |
| | Excessive sized openings in infill panels | Interior concrete walls [8.4.2]<br>Interior steel braced frames [8.4.1] | Infill selected openings [21.4.7]<br>Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6] | | | |
| | Inadequate columns for overturning forces | | Add cover plates or box members [8.4.3]<br>Encase columns in concrete | | | |
| | Weak or deteriorated masonry | Interior concrete walls [8.4.2]<br>Interior steel braced frames [8.4.1] | Point outside and/or inside wythes of masonry<br>Inject wall with cementitious grout<br>Add concrete or fiber composite overlay on exterior walls pier and/or spandrel [21.4.5], [21.4.6] | | | |
| Global Stiffness | See inadequate strength | | | | | |
| Configuration | Soft or weak story | Interior concrete walls [8.4.2]<br>Interior steel braced frames [8.4.1] | | | | |

| Table 11.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S5/S5A Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Torsion from one or more solid walls | Balance with Interior concrete walls<br>Balance Interior steel braced frames | | | | Remove selected infill panels on solid walls |
| | Irregular Plan Shape | Balance with interior concrete walls<br>Balance with interior steel braced frames | | | | |
| Load Path | Out-of-plane failure of infill due to loss of anchorage or slenderness of infill | Provide vertical strongback wall supports [21.4.3] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6] | | | Remove infill |
| | Inadequate connection of finish wythe to backing | | Add interwythe tie [21.4.12] | | | |
| | Inadequate collectors | Add steel collector on surface of concrete [12.4.3]<br>Embed or add collector in concrete floor slab [12.4.3] | Strengthen beam to column or beam to beam splices | | | |
| Component Detailing | Inadequate columns splice for tension due to uplift force induced by infill | | | Add splice plates<br>Provide splice through added reinforced concrete encasement | | |
| | Inadequate beam column connection to resist compression thrust | | | Strengthen connection in shear | | |

| Table 11.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for S5/S5A Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing (continued) | Weak or incompletely filled joint between masonry and surrounding steel components | | | Repair or fill voids to provide essentially continuous bearing. | | |
| Diaphragms | Flat masonry arch diaphragm | Add diagonal steel braced diaphragm under floor [22.2.8]<br>Remove top layers of floor construction and add concrete slab diaphragm | Add tension ties to prevent loss of arch action [22.2.8] | | | |
| | | | | | | |
| Foundation | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

no infill. This deficiency can be corrected by adding selected bays of infill or by adding shear walls or braced frames at this level. The second common issue is a plan torsional irregularity created by solid masonry walls on property lines coupled with walls with many openings on street fronts. If shown by analysis to be necessary, torsional response can be minimized by stiffening the more flexible side of the building with more infill or by the addition of lateral elements. In rare cases, the solid walls can be balanced with the open side by selected removal of panels or disengagement of the infill strut action.

### *Load Path*

The primary load path issue with this building type is to assure that the mass of the exterior walls will not become disengaged from the frame which will both prevent infill strut action as well as to create a significant falling hazard on the street below.

In-plane, the articulation of the exterior walls may result in offsets of the wall plane between floors. The presence of a complete load path and maintenance of confinement for strut formation must be reviewed in such instances.

If new lateral load-resisting elements are added, existing slab and steel beam construction may need to be strengthened to provide adequate collectors.

### *Component Detailing*

In order to qualify as an infill lateral force-resisting element, the infill must be installed tight to the surrounding steel frame. Loose or incomplete infill can be mitigated with local patching of the masonry or by injection of cemetitious or epoxy grout. However, unless the building is gutted for remodeling purposes, this procedure will be extremely disruptive.

The detailing of the steel frame forming the confinement for the masonry is important to achieve infill strut behavior. The connection of beam to column must be capable of resisting the strut compression forces from the masonry. Many different configurations are possible, each with a different potential weakness, but the shear capacity of the beam-to-column connection is often critical. In addition, column splices may be inadequate to transfer the overturning forces created by strut action. Critical connections normally can be strengthened with steel plates.

### *Diaphragm Deficiencies*

A wide variety of concrete diaphragms can be found in this building type. Solid slab-type floors will often provide an adequate diaphragm while joisted floors may include only a thin, poorly reinforced continuous slab with low shear capacity. The connection of slabs to exterior wall should be reviewed because dowels or other positive connections may not have been provided.

See Chapter 21 on URM construction for discussion of wood diaphragms in this type of building.

Flat masonry arch floors are problematic. The diaphragm capacity of such built up construction has not been established. Damage causing loss of arch action can create falling hazards or vertical load failures. Removal and replacement may not be feasible, either from a pure economic standpoint or due to historical preservation issues. The added weight of a new concrete slab is often difficult to accommodate, even if top layers of the existing floor are removed.

If space is available, a new steel diagonal frame diaphragm can be added underneath such floors. FRP can be layered on the masonry arches to better secure them in place. New lateral force-resisting elements can be added to minimize the need for diaphragm action.

#### *Foundation Deficiencies*

No systematic deficiency in foundations should be expected solely due to the characteristics of this building type.

#### *Other Deficiencies*

Although deterioration of material, in general, is not covered in this document, it is known that most buildings of this type have no reliable waterproofing system for the exterior steel framing, particularly the columns. Significant damage to columns from water infiltration has been noted in several cases, and this condition should be investigated before assuming that the perimeter frame is a significant lateral force-resisting element.

## 11.4 Detailed Description of Techniques Primarily Associated with This Building Type

Most significant recommendations listed in Table 11.3-1 are similar to techniques more commonly associated with other building types such as steel framed buildings (**S1**, **S2**, or **S4**), unreinforced masonry bearing wall buildings (**URM**), or general techniques applied to concrete diaphragms. Details concerning these techniques can be found in other chapters.

## 11.5 References

Abrams, D.P. (Editor), 1994, *Proceedings from the NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, National Center for Earthquake Engineering Research, Buffalo, NY.

Roeder, C.W., Leon, R.T., and Preece, F.R., 1996, “Expected Seismic Behavior of Older Steel Structures,” *Earthquake Spectra*, EERI, Vol. 12, No. 4, Oakland, CA, pgs 805-824.

# Chapter 12 - Building Type C1: Concrete Moment Frames

## 12.1 Description of the Model Building Type

These buildings consist of concrete framing, either a complete system of beams and columns or columns supporting slabs without gravity beams. Lateral forces are resisted by cast-in-place moment frames that develop stiffness through rigid connections of the column and beams. The lateral force-resisting frames could consist of the entire column and beam system in both directions, or the frames could be placed in selected bays in one or both directions. An important characteristic is that no significant concrete or masonry walls are present, or that they are adequately separated from the main structure to prevent interaction. Some buildings of this type have frames specifically designed for lateral loads, but also have interacting walls apparently unaccounted for in the design. These buildings could be classified as moment frames and the wall interaction would immediately be considered a seismic deficiency. Alternately, these buildings could be classified as Building Type **C2f** (Shear Wall with Gravity Frames). Older concrete buildings may include frame configurations that were not designed for lateral load, but if no walls or braces are present, the frames become the effective lateral force system and should be included in this building category. Buildings of this type that include integral concrete or masonry walls on the perimeter should be considered as Building Type **C2f** or **C3**. Floors may be a variety of cast-in-place or precast concrete. Buildings with concrete moment frames are generally used for most occupancies listed for steel moment frames, but are also used for multistory residential buildings.

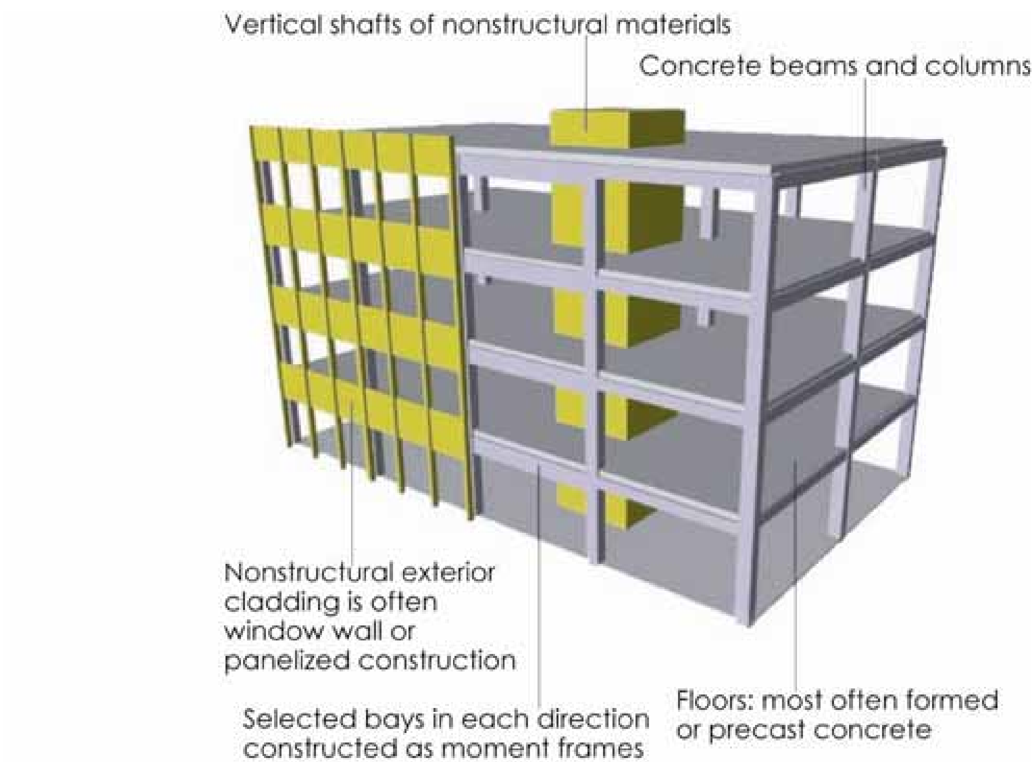



**Figure 12.1-1: Building Type C1: Concrete Moment Frames**

### *Variations Within the Building Type*

The primary variation within this type is the type of frame and the number of frames included. Frames can range from column-girder systems of one bay on each face of the building to systems that employ every column coupled with two-way slabs. Frames classified by code as ductile or semi-ductile by code beginning in the late 1960s and early 1970s are far more constrained in configuration due to prescriptive rules governing girder configuration, strong column-weak beam, and limitations on joint shear.

### *Floor and Roof Diaphragms*

The floor and roof diaphragms in this building type are essentially the same as the bearing wall system, and are almost always cast-in-place concrete. The diaphragms are stiff and strong in shear because the horizontal slab portion of the gravity system is either thick or frequently braced with joists. However, one way joist systems could be inadequate in shear in the direction parallel to the joists. Collectors are seldom in place and transfer of load from diaphragm to shear wall must be carefully considered.

### *Foundations*

There is no typical foundation for this building category. Foundations could be found of every type depending on the height of the building, the span of the gravity system and the site soil.

## 12.2 Seismic Response Characteristics

This building type must be separated into older frame systems, often not even designed for lateral loads and including few, if any, features that would assure ductile behavior, and frames specifically designed to exhibit ductility under seismic loading. Rules for design of ductile concrete frames were developed during the 1960s.

Older, non-ductile frame buildings, assuming an insignificant amount of concrete or masonry walls are present, will be far more flexible than other concrete buildings, and will probably be relatively weak. Most importantly, columns are often not stronger than beam or slab system, forcing initial yielding in these key elements. In addition, unless spiral ties were used, the column will typically fail in shear before a flexural hinge can form. Buildings with these characteristics are among the most hazardous in the U.S. inventory and are in danger of collapse in ground motion strong enough to initiate shear failures in the columns. Buildings of this type that are configured such that initial hinging occurs in the floor system will exhibit stiffness and strength degradation and large drifts, but unless exceptionally weak, are far less likely to collapse. The ratio of the inherent strength of the frame—designed for lateral loads or not—compared to the seismic demand has a large influence on the performance, and frames in low and moderate seismic zones may be at less risk for this reason.

Semi-ductile frames, with some but not all of current design features for concrete frames, likely will perform better, particularly if the columns are protected by basic strength and are designed to be flexurally controlled. However, many of these early concrete frames may be excessively weak and suffer from high ductility demands which could have serious consequences if a soft or weak story is present due to architectural configuration or column layout.

Buildings with “fully ductile” frames are expected to perform well, unless vertical or horizontal configuration irregularities concentrate inelastic deformation on certain structural components.

## 12.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 12.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Selected deficiencies are further discussed below by category.

### *Global Strength*

Although lack of ductility is the overwhelming deficiency for this building category, low strength may contribute to poor performance. It is difficult to add significant strength within the confines of the existing frames and most often new elements of braced frames or shear walls are added in these buildings.

### *Global Stiffness*

See *Global Strength*.

### *Configuration*

The most common configuration issue in this building type is a soft or weak story created by a non-typical story height. If the building is not to receive new walls or frames as part of a global retrofit, such configuration deficiencies can be minimized or eliminated with local strengthening of columns.

### *Load Path*

There are no load path issues particular to this building type.

### *Component Detailing*

The major deficiencies of this building type are due to inadequate component detailing, namely the structural components of the frame. Current requirements for “ductile frames” include capacity design techniques to assure flexural yielding in both girders and columns, as well as, for the most part, to limit yielding to the floor system. Retrofit procedures to obtain this ductile behavior of the frames are difficult, disruptive, and expensive, and are therefore seldom done. In high seismic zones, retrofit of these buildings is normally accomplished by adding new, stiffer lateral force-resisting elements that prevent significant ductility demand on the frames.

Some research has been completed to investigate methods of retrofit for concrete moment frames (see Section 12.4.6), and in lower seismic zones where demands over and above gravity designs are not great, local strengthening and confinement of frame elements may be practical.

### *Diaphragm Deficiencies*

The most common diaphragm deficiency in this building type is a lack of adequate collectors. The addition of effective collectors in an existing diaphragm is difficult and disruptive. Existing strength to deliver loads to the shear walls should be studied carefully before adding new collectors.

**Table 12.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C1 Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient number of frames or weak frames | Concrete/masonry shear wall [12.4.2]<br>Steel braced frame [12.4.1]<br>Concrete or steel moment frame<br>Steel moment frame | Increase size of columns and/or beams [12.4.5] | | Remove upper story or stories [24.2]<br>Seismically isolate [24.3]<br>Supplemental damping [24.4] | |
| Global Stiffness | Insufficient number of frames or frames with inadequate stiffness | Concrete/masonry shear wall [12.4.2]<br>Steel braced frame [12.4.1]<br>Concrete or steel moment frame | Increase size of columns and/or beams [12.4.5]<br>Fiber composite wrap of gravity columns [12.4.4]<br>Concrete/steel jacket of gravity columns [12.4.5]<br>Provide detailing of all other elements to accept drifts | | Supplemental damping [24.4] | Remove components creating short columns |
| Configuration | Soft story or weak story | Add strength or stiffness in story to match balance of floors | | | | |
| | Re-entrant corner<br><br>Torsional layout | Add floor area to minimize effect of corner<br>Add balancing walls, braced frames, or moment frames | | Provide chords in diaphragm | | |

| Table 12.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C1 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Incidental walls failing or causing torsion | Add balancing walls, braced frames, or moment frames | Uncouple incidental walls<br>Convert incidental walls to lateral elements walls | | | Remove incidental walls |
| Load Path | Inadequate collector | Add or strengthen collector [12.4.3] | | | | |
| Component Detailing | Lack of Ductile detailing--general | | Perform selected improvements to joints [12.4.6] | | Seismic isolation [24.3] | |
| | Lack of ductile detailing: Strong column-weak beam | | Jacket columns [12.4.4] | | | |
| | Lack of ductile detailing: Inadequate shear strength in column or beam | | Fiber composite wrap [12.4.4]<br>Concrete/steel jacket [12.4.5] | | | |
| | Lack of ductile detailing: Confinement for ductility or splices | | Fiber composite wrap [12.4.4]<br>Concrete/steel jacket [12.4.5] | | | |
| Diaphragms | Inadequate in-plane shear capacity | Concrete or masonry shear wall [12.4.2]<br>Braced frame [12.4.1]<br>Moment frame | R/C topping slab overlay<br>FRP overlays [22.2.5] | | | |

| Table 12.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C1 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Diaphragms (continued) | Inadequate chord capacity | New concrete or steel chord member [12.4.3] | | | | |
| | Excessive stresses at openings and irregularities | Add chords [12.4.3] | | | | Infill openings [22.2.4] |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

# 12.4 Detailed Description of Techniques Primarily Associated with This Building Type

## 12.4.1 Add Steel Braced Frame (Connected to a Concrete Diaphragm)

***Deficiencies Addressed by the Rehabilitation Technique***

Inadequate global shear capacity
Inadequate lateral displacement (global stiffness) capacity

***Description of the Rehabilitation Technique***

Addition of steel diagonal braced frames to an existing concrete moment frame building is a method of adding strength and/or stiffness to the structural system. The steel braces can be added without a significant increase in the building weight. The new braces will commonly be some configuration of concentric braced frame (CBF); it is very uncommon to use an eccentrically braced frame (EBF) due to costs and difficult detailing issues associated with the link mechanism. Any of a variety of diagonal brace configurations may be used, as well as a variety of brace member section types. Figure 12.4.1-1 shows several common configurations. Common connections of the new brace to the existing concrete structure are shown in Figures 12.4.1-2A, 12.4.1-2B, and 12.4.1-2C.

***Design Considerations***

*Research basis:* Design of the lateral force-resisting system for the building should account for the stiffness of both the braced frame system and the existing concrete moment frames. While basic research regarding adding braced frames at the interior of a concrete moment frame building has not been identified, research in the 1980s at the University of Texas at Austin on frames at the exterior façade demonstrated the ability of the new steel braced frames to increase the deformation capacity of the non-ductile concrete frames (Jones and Jirsa, 1986). A schematic detail of the connection used in this testing is shown in Figure 12.4.1-3.

*Braced frame – concrete frame interaction:* Most designs of braced frame retrofits will be governed by maintaining drifts within the range of acceptability for the existing concrete elements. This can be accomplished by setting up a model that includes both the stiffness of the braced frame and of the concrete frame and meeting acceptability requirements for the displacements (or psuedo forces) in the concrete elements. Some engineers prefer to consider only the braced frames as a new lateral system, determine real drift demand for that system, and then check that drift for acceptability superimposed on the existing frame

In taller buildings, the possible incompatibility between vertical cantilever behavior of discrete braced frames and the existing moment frames must be assessed. Existing beams or slabs, if unusually thick, that frame directly into the ends of new braces may restrain the global flexural deformation of the brace and require special consideration. Finally, due to the wide variety of nonlinear behavior of braced frames that is dependent on configuration and detailing, it may be difficult to obtain an adequate understanding of overall deformation compatibility using linear methods.

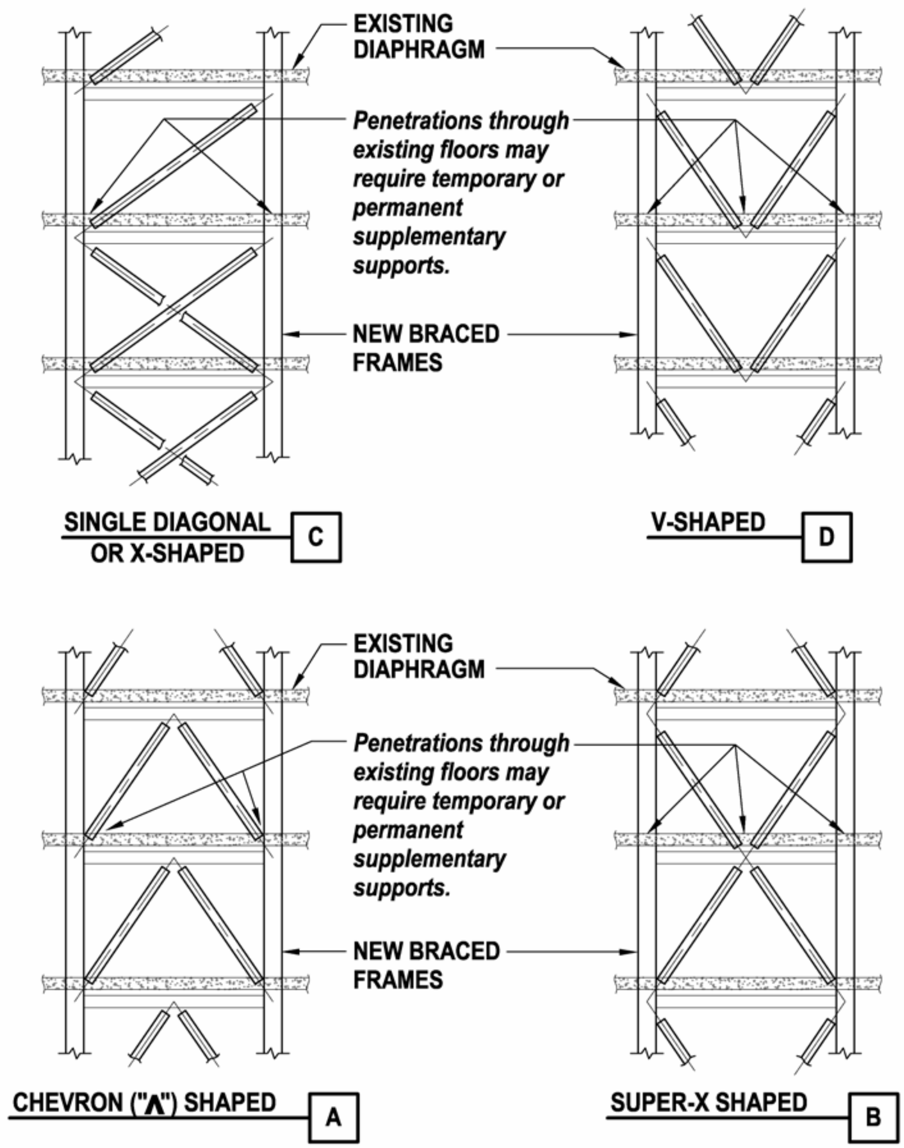



**Figure 12.4.1-1: Typical Braced Frame Configurations**

*Braced frame location:* The new braces may be located on the exterior or interior of the building. An exterior location generally allows for easier construction access and perhaps less cost, but is visible, exposed to the environment and probably will impact exterior building finishes. Braces placed parallel to the façade can be connected to the exterior faces of perimeter spandrel beams, perimeter moment frames or edges of floor and roof diaphragms relatively easily, but will most likely cross in front of some windows. Alternatively, exterior bracing may be placed as buttresses, perpendicular to the existing façade. This configuration will probably require more extensive new collectors to deliver lateral forces from the diaphragms but may allow creation of new stair or elevator shafts, or perhaps additional floor area. For projects that include expansion of or additions to the existing building, the new braces could be located in the adjacent new construction, tied to the existing building.

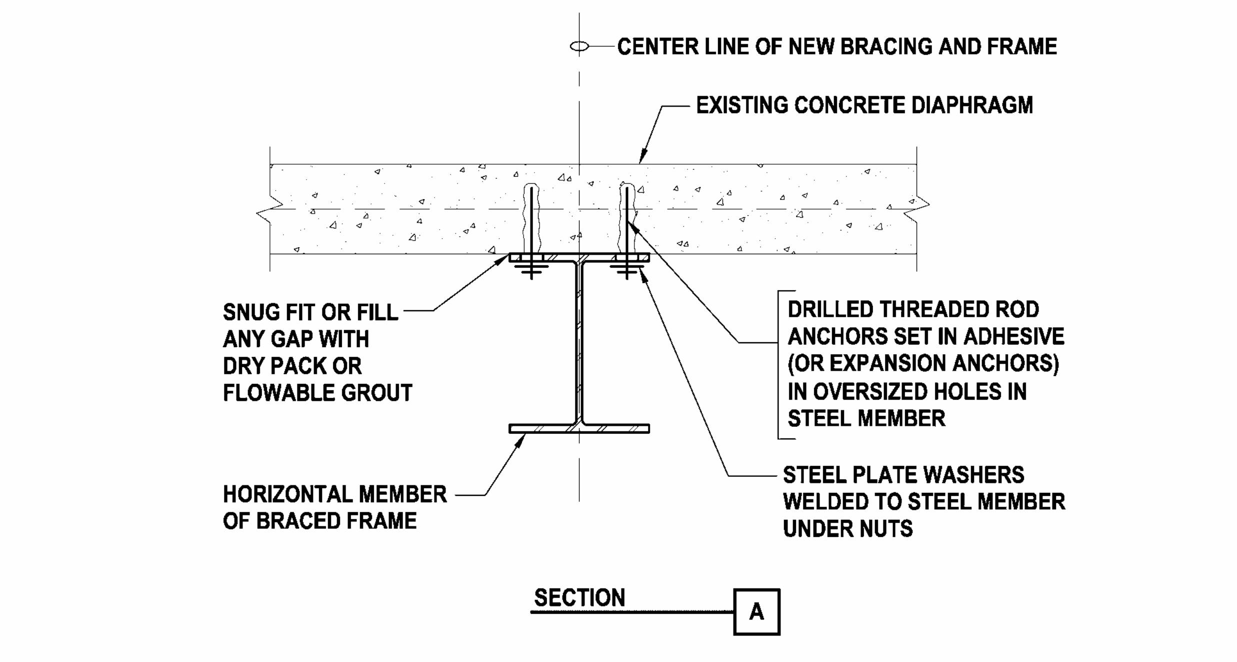



**Figure 12.4.1-2A: Typical Connection to Concrete Diaphragm**

Interior braces will most commonly be located along existing frame lines, particularly at moment frame bays. This will allow for best use of any existing diaphragm chords and collectors and for best moment frame – braced frame interaction. In some cases however, interior braces will be located offset from existing column-frame lines to minimize direct impact on existing structural or architectural components or to simplify the frame-diaphragm connections.

The addition of new braced frames to a building will always impact the architectural character and functional uses of the building to some degree. Selection of preferred brace locations must be made considering these issues, such as space layout, corridor locations, doorways, windows, main M/E/P distribution runs, as well as the structural or construction considerations.

*Braced frame configuration and member section type:* In most cases where diagonal steel braces are used to strengthen or stiffen a concrete frame building, a complete braced frame including horizontal beam and column members, as well as the diagonal braces themselves, is employed. Installation of diagonal bracing members between existing concrete columns is difficult because transfer of a large concentrated axial force from the concrete members through a localized connection with a limited number of anchors is rarely feasible. The steel columns are often continuous, passing through the floors, from foundations up to the roof or highest level required to avoid transfer of load from the steel system in and out of the concrete at each floor. In some cases, columns can be connected to adjacent concrete columns, but if the concrete column becomes part of the primary chord, reinforcing splice locations must be carefully considered.

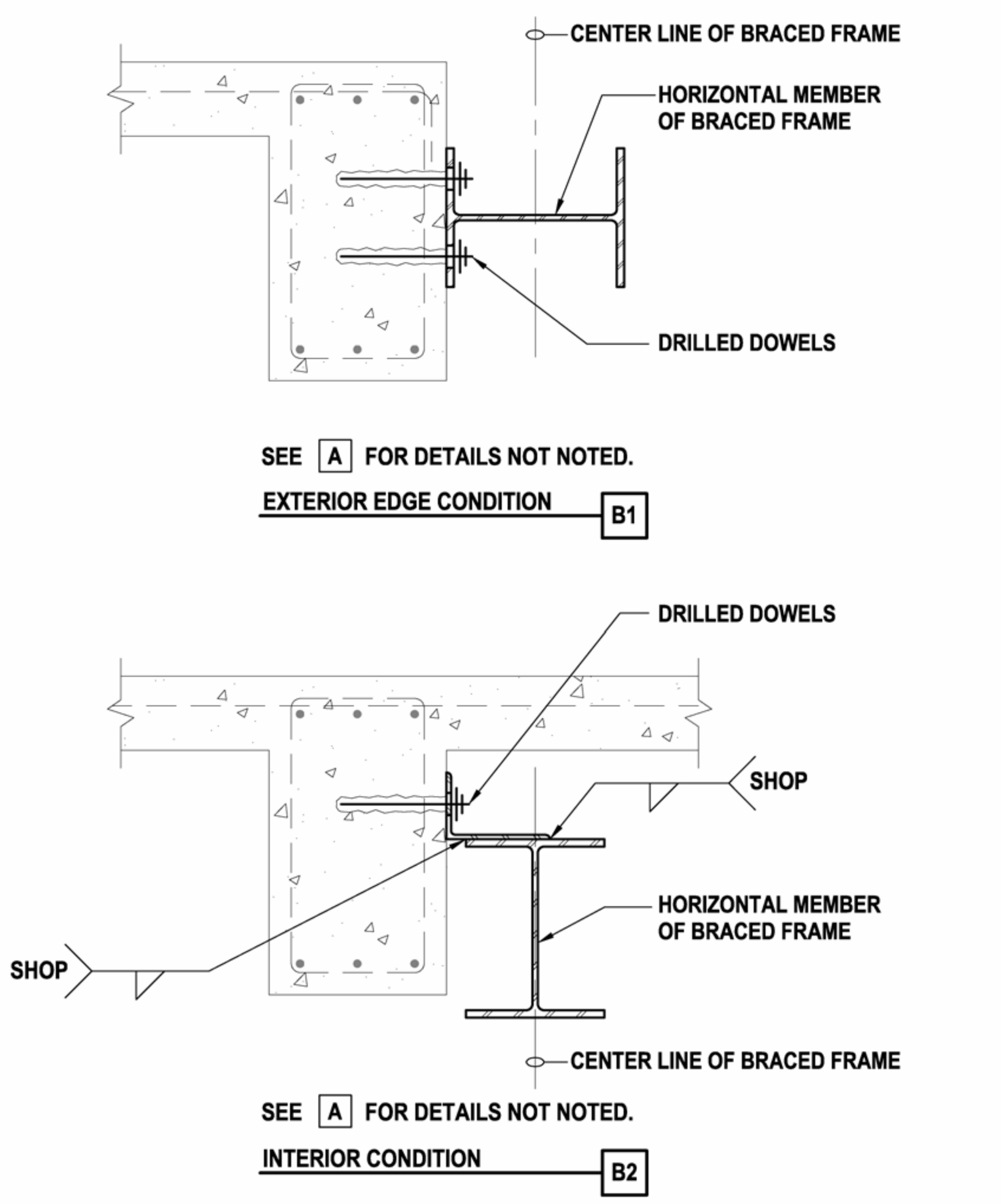



**Figure 12.4.1-2B: Typical Connection to Existing Concrete Beam**

New steel horizontal elements are similarly needed to facilitate the connections of the diagonal and to transfer forces from each floor into the frame. These steel elements are generally placed below the floor and roof diaphragms or adjacent to beams or spandrels. The diagonal steel braces may be placed in any of the commonly used configurations indicated in Figure 12.4.1-1;

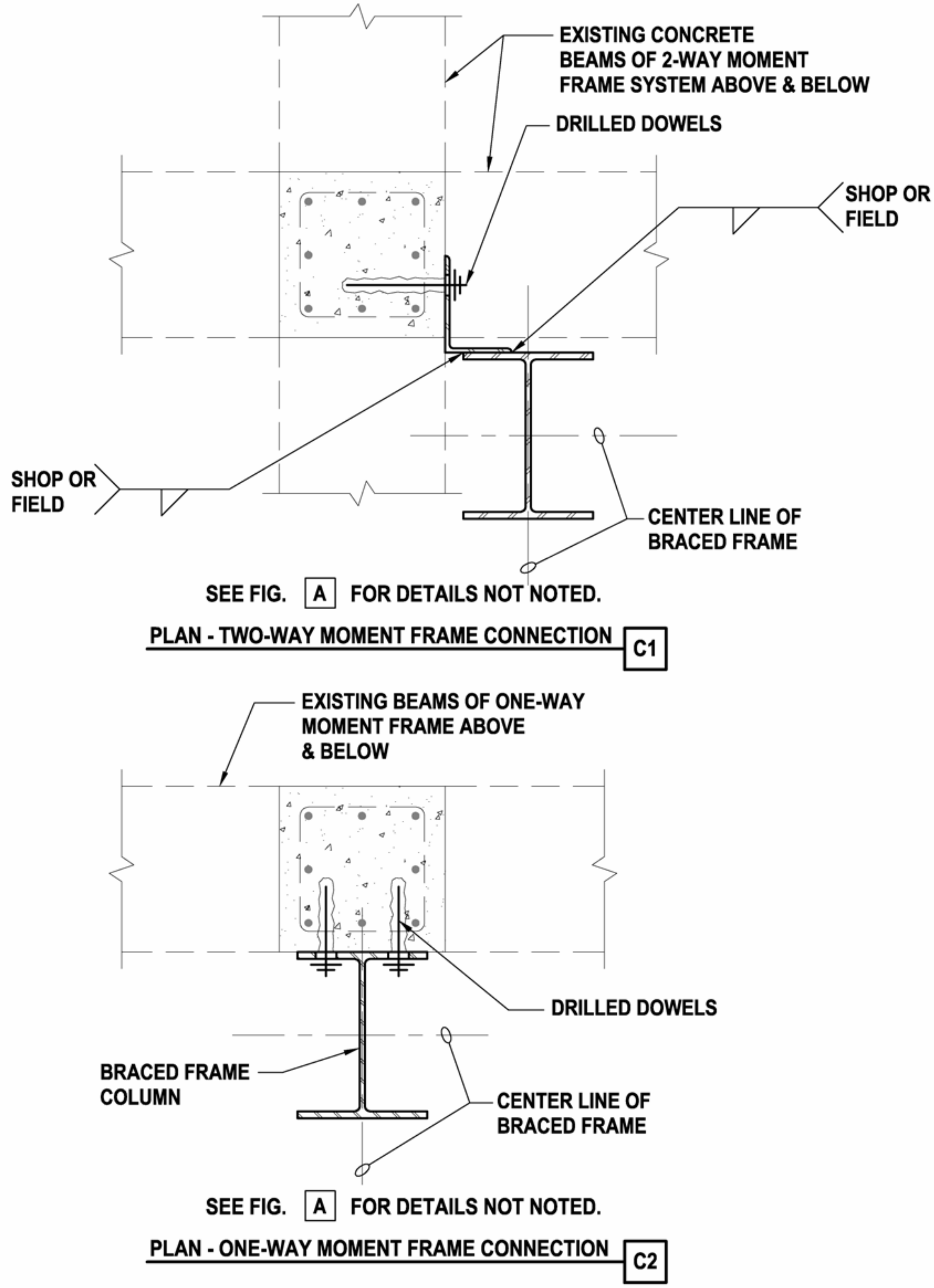



**Figure 12.4.1-2C: Typical Connection to Existing Concrete Column**

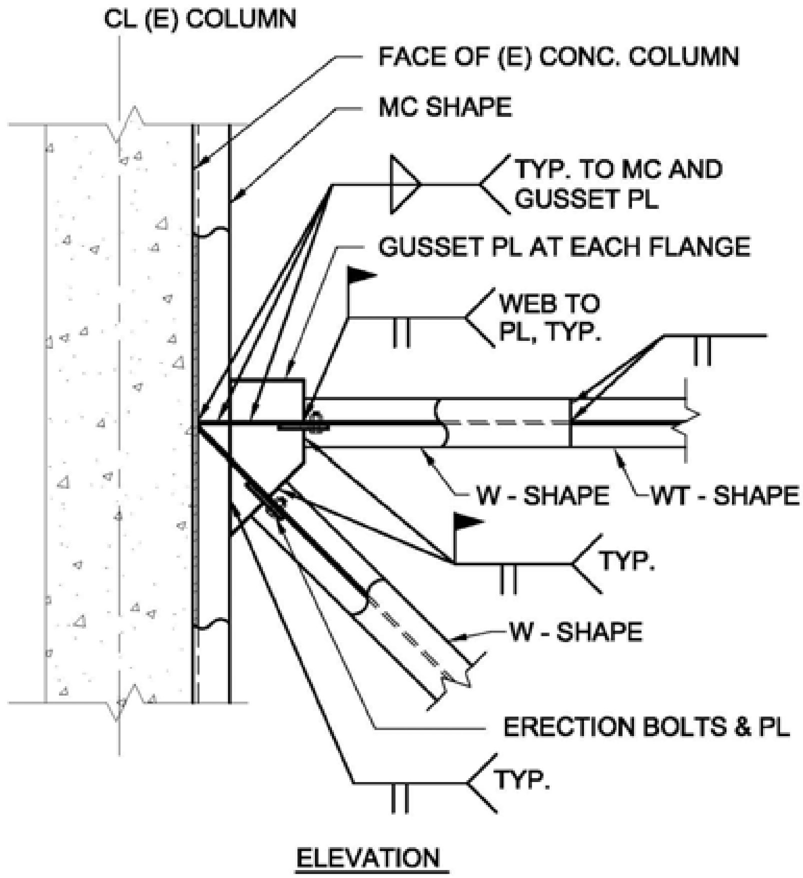



**Figure 12.4.1-3: Test Specimen Connection Detail for Braced Frame**

single diagonal or X-shaped, V-shaped, chevron (inverted-V) shaped, or "super-X" shaped (a combination of chevron and V braces in alternate stories forming a two-story X shape). Two-story X-bracing has the advantage over V- or inverted-V-bracing should a compression brace buckle. In the latter configurations, the remaining tension brace has an unbalanced vertical component that has to be resisted by the beam. For an X-bracing configuration, even if a compression brace buckles, the force in the remaining tension brace is transmitted directly to the tension brace on the opposite side of the beam.

Configuration will be selected based on consideration of structural issues, relative strength, stiffness and performance, as well as of several other issues including aesthetics, conflicts with doorways, corridors or windows, M/E/P systems, or the number of connections and penetrations. Column and beam members are often W-shapes, but may be other shapes such as channels or hollow structural section (HSS) tubes to improve aesthetics or to ease detailing. Diagonal members may be of any typically used sections including W-shapes, hollow (HSS) pipes or tubes, or double channels, angles or HSS tubes.

Buckling-Restrained Braced Frames (BRBFs), in which steel plates or cruciform shaped braces are surrounded by unbonded concrete in such a way as to prevent bucking of the brace, act essentially the same in tension and compression. The yield strength of a bay braced with one or more of these braces can be relatively accurately predicted. In situations where many, lightly loaded braces will be employed, sufficient global strength can be obtained by designing the braces to yield prior to yielding or other failure of the existing columns, preventing the need to retrofit the columns.

***Detailing Considerations***

*Connection to existing concrete floor and roof diaphragms:* A significant concern associated with installing a new steel braced frame in a concrete building is the connection of the beam at the top of the frame in each story to the underside of the existing concrete diaphragm overhead. The primary concern is that a relatively large shear force must be transferred from the overhead diaphragm into the new steel bracing below through a relatively localized connection using discreet anchors/bolts. The connection is generally made by one or more rows of concrete anchors as shown in Figure 12.4.1-2A. Typically, the anchors are threaded rods set in epoxy, but drilled expansion anchors may be used if they provide sufficient force transfer capacity and adequate testing to show they can resist cyclic loading. An alternate connection method, installed from the top down, consists of providing large holes in the concrete slab to expose the steel beam sufficiently to installed welded dowels to the top flange. The hole is then backfilled with cementitious or epoxy grout. In many cases however, the shear capacity of the existing concrete diaphragm is inadequate to deliver the relatively large shear force within the length of the braced frame. In those cases, a collector will be required (refer to Section 12.4.3).

*Connection to existing moment frames:* New braced frames are often located on or alongside of the existing moment frame lines. This generally allows for better use of the existing collectors (beams) to deliver diaphragm forces to the bracing and, perhaps, use of the existing frame columns and footings to help resist overturning and uplift forces. It is generally preferable to locate the new braces alongside of the existing moment frames instead of as an “infill” within the width of the existing concrete frame beams and columns.

For diagonal braces installed in an “infill” configuration, it is often extremely difficult to transfer large seismic forces from the surrounding concrete members through very localized connections with a limited number of discreet anchors. Also, if steel columns or vertical members are used in the “infill” frame, it is virtually impossible to provide vertical continuity from floor to floor through the existing concrete beams. Furthermore, if connections of sufficient strength can be made, the anchors must be threaded into or through the relatively densely reinforced beams and columns and, where collector strengthening is required, the added collector components will not connect directly to the new braces. In addition, physical installation and fit-up of the new braces and their connecting gusset plates often becomes significantly more difficult.

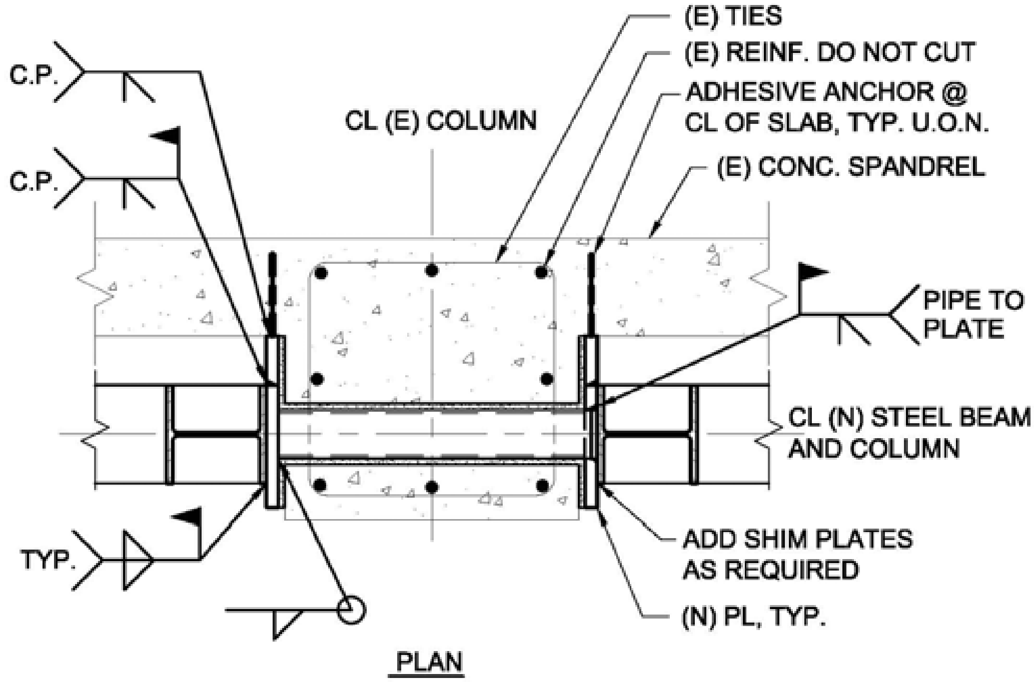
These detailing difficulties can be reduced or avoided by placing the new braced frame alongside of the concrete moment frame. In most cases, placing the new bracing alongside an exterior frame will allow the greatest ease in detailing. Bracing in this location will almost always require installation of a complete new steel braced frame instead of only the new diagonal braces themselves. In this configuration, the connection of the concrete diaphragm to steel braced frame can be made as discussed above or by installing anchors into the side of the adjacent concrete frame beam as shown in Figure 12.4.1-2B. Braced frame overturning forces are carried directly by the new steel column members. However, if concrete beam framing occurs in two directions, the new steel columns will generally need to be offset from the existing concrete column on a 45-degree diagonal to provide continuity of the new column through the floors without interference with the existing concrete beams. Overturning resistance can be obtained by connecting the new steel column to the existing concrete column (see Figure 12.4.1-2C1) and footing. For cases where the new braced frame can be placed on the exterior of the building, the new steel columns can be continuous and the connections to the adjacent concrete columns or pilasters can be made with relative ease. If diaphragm collector strengthening is required, the additional collector can be installed alongside of the existing beam line and can be connected directly to the new braced frame.

*Exterior bracing at offset columns:* There are some buildings where the exterior columns protrude farther out than the exterior beams. The Jones and Jirsa (1986) research can be applied in these situations, where the new steel framing is placed adjacent to the beams and in the plane of the outer portion of the protruding concrete column as shown in Figures 12.4.1-3 through 12.4.1-5. The primary challenges lie in connecting the two types of frames and delivering loads into the braced frames. As an alternative to drilling numerous holes for bolts or dowels into the concrete columns, steel lugs can be provided at each floor. In this approach, steel pipes are inserted through cores drilled through the concrete columns and filled with grout. Next, the pipes are welded to steel plates on the sides of the concrete columns, which then provide surfaces for welding to the columns of the braced frames. An example of this connection is shown in Figure 12.4.1-4. If required, horizontal forces can also be transferred directly to the braced frames through the braced frame beams. The beams are welded to steel plates, which are connected to the concrete slab or beams at the building perimeter with dowels, bolts, or lugs, as shown in Figure 12.4.1-5.

Exposed exterior braced frames require simple and clean connections that fit the architectural character of the building. Use of W-shapes for the braced frame members can eliminate gusset plates and allow direct connection of the members through complete joint penetration welds. Shop welding of the connections and on-site prefabrication of the braced frames will minimize field welding on the structure. W-shapes also simplify other architectural issues by not allowing rainwater or debris to accumulate.

*Installation of additional collectors:* Installation of new braced frames in a concrete frame structure, especially in one with a distributed frame system, will often result in increased diaphragm demands at the individual braces. An advantage of locating the new brace at an existing frame line is that the existing beams can then be used as a collector. However, insufficient continuity and/or laps of reinforcing steel combined with highly concentrated diaphragm demand may still require strengthening of the existing collector (refer to Section 12.4.3).

*Footings:* Addition of steel braced frames to an existing building will almost always require construction of new footings, or augmentation of existing ones, to resist the concentrated overturning demand. In many cases, the overturning uplift demand will require installation of tie




*Installation Procedure:*

1. *Install adhesive anchor and plate on each side of column.*
2. *Core hole for pipe through column.*
3. *Install pipe with shop welded plate on one side.*
4. *Install plate with hole and weld to pipe.*
5. *Weld cap plate to pipe.*
6. *Weld plate attached to pipe to plate attached to adhesive anchor.*
7. *Fill pipe and annular space solid with nonshrink grout.*

**Figure 12.4.1-4: Braced Frame to Concrete Column Connection**

downs. Alternatively, the new frame can be located between two existing column frame lines, instead of directly on or along one frame line, and new foundations or grade beams can be used to engage more than one existing column to resist the uplift demand.

### *Cost/Disruption Considerations*

The cost and level of disruption associated with installation of steel bracing is generally less than that of shear walls. The number of penetrations that need to be cut through the existing concrete structure and of drilled dowels and anchors may be less than for the shear wall alternative, and the work is generally not as wet or messy. Also, it will not be necessary to prepare any existing concrete surfaces that will be in contact with new steel members. The new members are discrete and welded or bolted connections are localized. However, there will be noise and vibrations resulting from the required cutting and drilling that will make continued occupancy difficult.

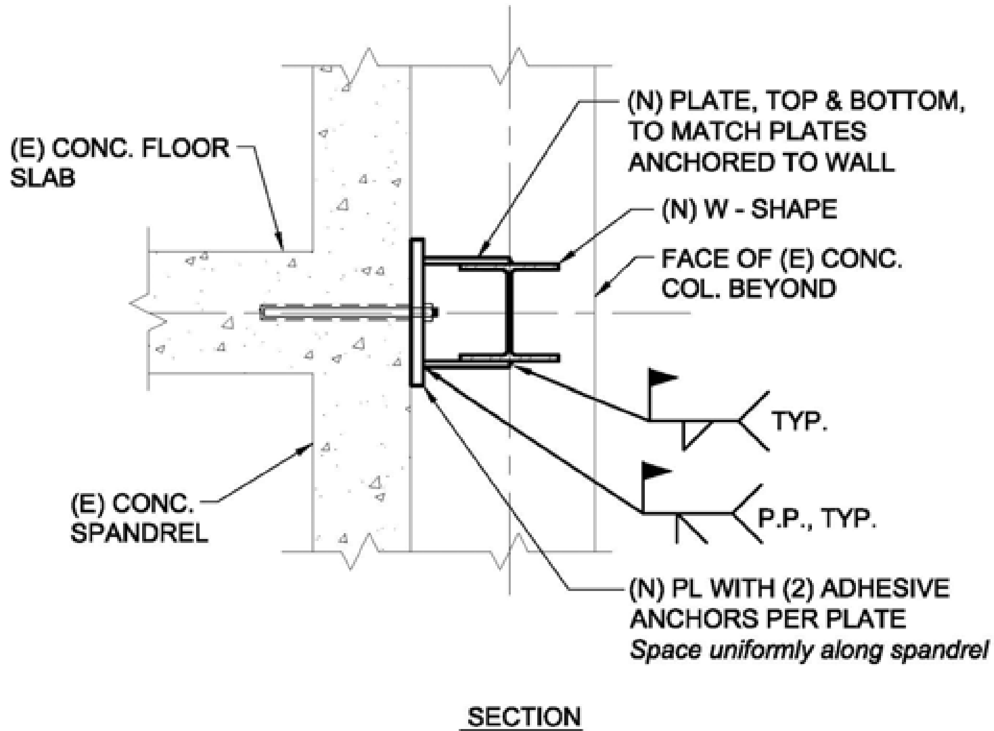



**Figure 12.4.1-5: Braced Frame to Slab Connection**

Invariably, some architectural and M/E/P system components will require relocation or replacement.

### *Construction Considerations*

The primary construction consideration will be fit up and installation of the steel braces and their connections. The desire to limit the number of splice connections must be balanced against the difficulties of installing longer members such as multistory columns. Installation of the diagonal braces will require careful planning and will often require member splices in the field. Installation of drilled threaded rod or expansion anchors will require some precision and extensive use of templates and oversized holes, to assure proper fit with the steel members. In some cases, the steel members themselves could be used as the template for the anchors.

### *Proprietary Concerns*

In general, there are no proprietary concerns related to installation of steel diagonal braced frames in a building. The one exception, however, occurs if buckling-restrained braces are used.

### 12.4.2 Add Concrete or Masonry Shear Wall (Connected to a Concrete Diaphragm)

#### *Deficiencies Addressed by the Rehabilitation Technique*

Inadequate global shear capacity
Inadequate lateral displacement (global stiffness) capacity

#### *Description of the Rehabilitation Technique*

Addition of shear walls to an existing concrete frame building is a common method of adding significant strength and/or stiffness to the structure. The new walls may be of cast-in-place concrete, shotcrete or fully grouted concrete masonry unit (CMU) construction.

#### *Design Considerations*

*Research basis:* No research focused on the overall effects of adding shear walls to existing concrete frames has been identified. The effects of surface preparation, concrete strength, and interface reinforcement on interface shear capacity between new and existing concrete were examined by Bass, Carrasquillo, and Jirsa (1985). These tests indicate that surfaces prepared with heavy sandblasting exhibit shear capacities greater than or equal to those exhibited by chipped surfaces or surfaces prepared with shear keys. Increased concrete strength resulted in increased interface shear capacity for chipped surfaces and those prepared with shear keys, but it had little effect on the shear capacity of interface surfaces prepared by sandblasting. Specimens in which drypack mortar was used exhibited a significantly smaller shear capacity than those where new concrete was cast directly against the interface. Increasing the amount or embedment depth of reinforcement across the interface resulted in greater interface shear capacity.

*Frame-wall interaction:* Most designs of shear wall retrofits will be governed by maintaining drifts within the range of acceptability for the existing concrete frame elements. This can be accomplished by setting up a model that includes both the stiffness of the shear walls and of the concrete frame and meeting acceptability requirements for the displacements (or psuedo forces) in the concrete frame elements. Some engineers prefer to consider only the shear walls as a new lateral system, determine the expected drift demand for that system and then check that drift superimposed on the existing frame for acceptability.

In taller buildings, the possible incompatibility between vertical cantilever behavior of discrete shear walls and the existing concrete frames must be assessed. Existing beams or slabs, if unusually thick, that frame directly into the ends of new walls may restrain the global flexural deformation of the wall and require special consideration. Two shear walls are often purposely placed in line and connected by a short beam to form a coupled shear wall system. In this system, the coupling beams are specially designed to accept significant inelastic deformations. Seldom can two such walls utilize existing beams as coupling beams due to inadequate detailing. Thus coupling beams, when employed, are installed new, as part of the system.

*Frame-wall configuration:* A primary design consideration is determination of whether or not the existing concrete frames may be used as an effective part of a combined system. Are the existing frame columns strong enough and/or well detailed enough to serve as the chord/boundary member of a shear wall without improvement? Are the frame beams detailed well enough to serve as coupling beams or to be incorporated into the wall itself? These considerations may limit the choices of wall-frame physical relationship: that is, should the walls be placed 1) within the plane of the existing concrete frames, 2) as vertically continuous walls alongside of, and joined to, the existing frames, or 3) as separate vertical elements independent of the frames? The first alternative is often best avoided as noted in the *Detailing Considerations* discussion below. Considering alternates 2 and 3, it must be determined if the frames are capable of becoming part of the shear wall (primarily as chord elements) or if it is beneficial to prevent direct interaction by placing the shear walls free of the existing concrete frame elements. In some cases, it is not feasible to stiffen the building into the range of acceptable deformation of the existing frames, and improvement in deformation capacity may be required in addition to the addition of new walls.

*Wall location:* The new walls may be placed on the exterior or interior of the building. An exterior location generally allows for easier construction access and perhaps less cost, but is visible, exposed to the environment and may impact exterior building finishes. Walls placed parallel to the façade can be connected to the exterior edges of floor and roof diaphragms or perimeter concrete frames relatively easily, but will most likely require closure or reduction in size of some windows. Alternatively, exterior walls may be placed as buttresses perpendicular to the existing façade. This configuration will probably require more extensive new collectors to deliver lateral forces from the diaphragms but may allow creation of new stair or elevator shafts, or even of additional floor area. For projects that include expansion of or additions to the existing building, the new walls could be located in the adjacent new construction.

Interior walls located along frame lines, particularly at moment frame bays will often allow for best use of any existing diaphragm chords and collectors. Beams that frame directly into the ends of new walls may behave like coupling beams as described above. In some cases, interior walls are better located offset from existing column-frame lines to minimize direct impact on existing structural or architectural components or to simplify the wall-diaphragm connections.

The addition of shear walls to a building will always impact the architectural character and functional uses of the building. Selection of preferred wall locations must be made considering these issues, such as space layout, corridor locations, doorways, windows, main M/E/P distribution runs, as well as the structural or construction considerations.

### *Detailing Considerations*

*Connection to existing concrete floor and roof diaphragms:* Arguably, the most significant detail associated with installing a new shear wall in a concrete building is the connection at the top of the new wall to the underside of the existing concrete diaphragm overhead. The construction joint must be made tight, without any gapping, to facilitate transfer of shear forces from the overhead diaphragm into the new wall below and to minimize the possibility of joint slip. See the discussion under the *Research basis* section.

Typical details of this connection for a new cast-in-place concrete wall below an existing concrete flat slab are shown in Figures 12.4.2-1A and 12.4.2-1B. The vertical dowels must be sufficient to transfer forces from the existing diaphragm and from the new wall above (if it exists), to the lower wall. Shears can also be transferred across this joint with large diameter

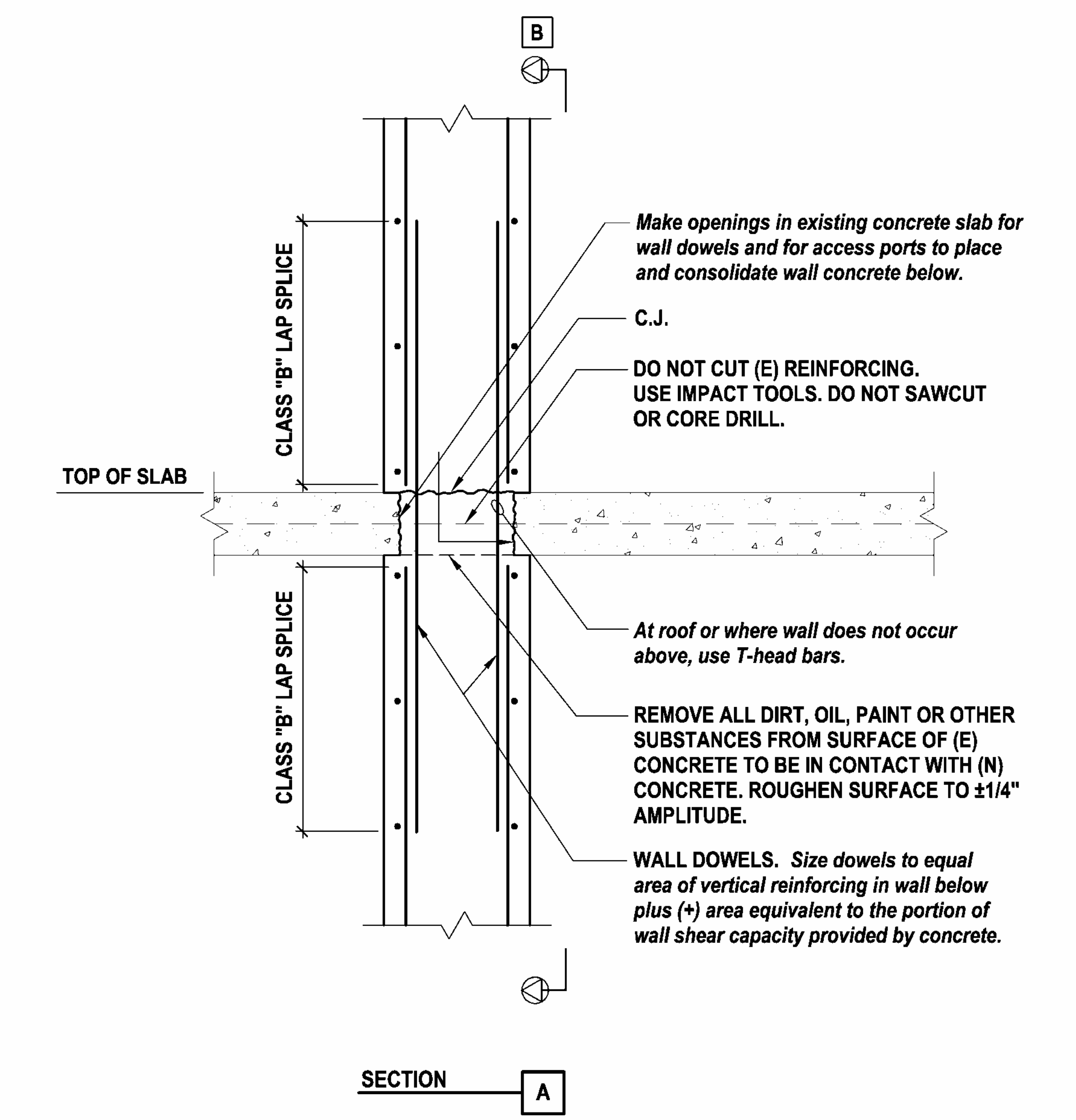



**Figure 12.4.2-1A: Concrete Wall Connection to Concrete Slab**

pipes or structural shapes. The holes made through the existing slab must serve not only to install the dowels, but also to allow for placement and consolidation of the wall concrete. The concrete head created by placement up to the top of slab coupled with cleaning and roughening the existing concrete contact surface by either sandblasting or chipping will provide the best joint available. The larger holes through the slab will also be more like intermittent shear keys. The holes should be drilled or made with impact tools instead of saws or core drills to avoid cutting or damaging existing slab reinforcement. Prior to cutting the holes, temporary shores may be required below the slab along each side of the row of holes. The concrete should be placed through the slab openings into the forms below, up to top of slab, to provide some head on the joint at the underside of the diaphragm.

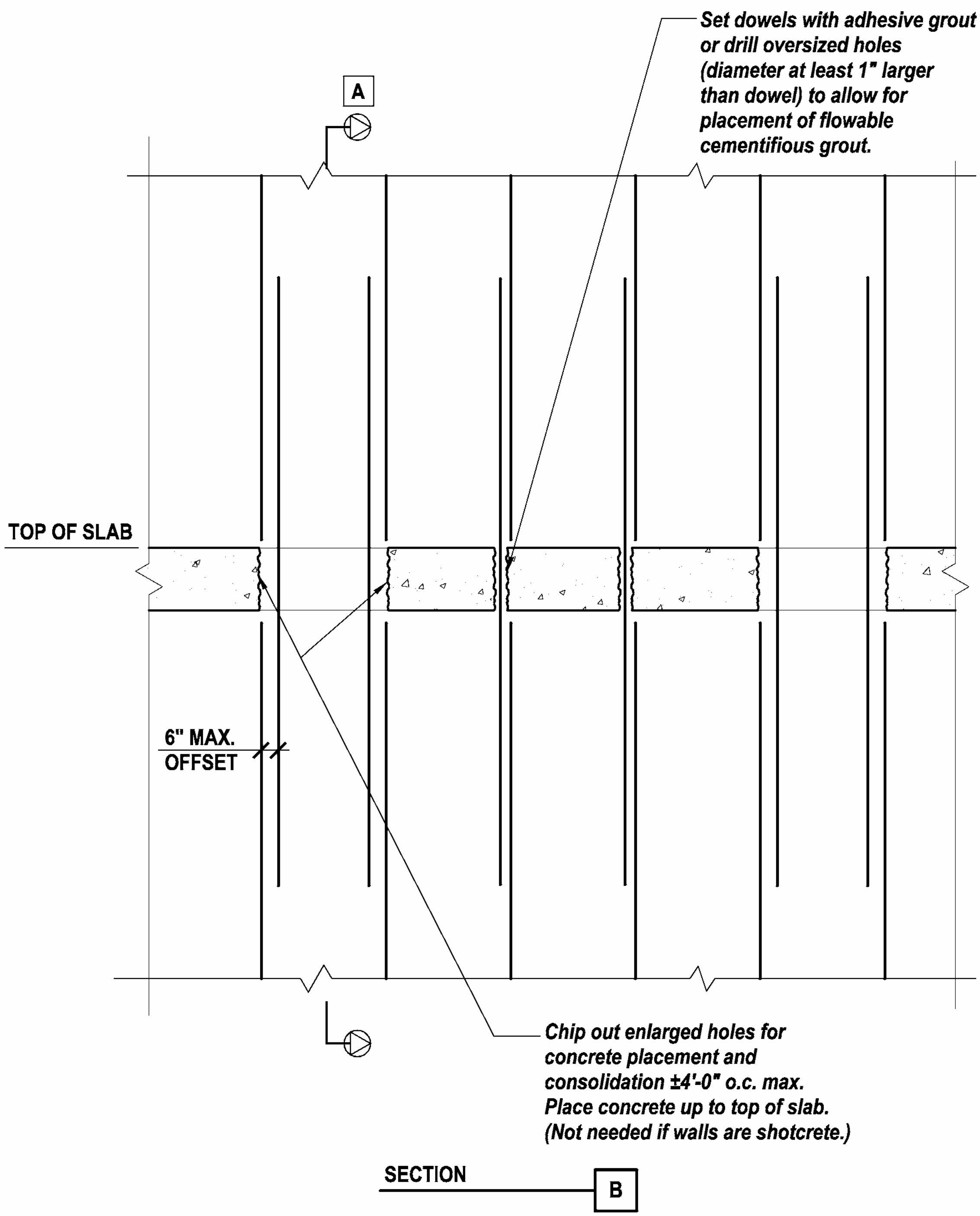



**Figure 12.4.2-1B: Concrete Wall Connection to Concrete Slab – Partial Elevation View**

If the new wall is shotcrete, special care is required by the nozzle operator when placing the shotcrete directly at the underside of the slab to provide a tight, well-bonded joint free of rebound or gaps. To minimize the possibility of creating a gap due to sagging, the last lift of shotcrete should be a short one. In the end, however, such a well-bonded joint is often not achieved at the slab soffit and remedial work, similar to crack-injection repairs, is likely to be needed. As an alternative, the holes through the slab needed for placement of the vertical rebar dowels could be made oversized, sufficient to allow placement of pourable, cementitious grout at the top of the shotcrete wall below, similar to the cast-in-place concrete alternative.

For CMU wall construction, the masonry units will typically be constructed up to within one or two courses of the overhead slab soffit, leaving enough of a gap to allow placement of the upper lift of grout. Preferably, the gap should be formed and grouted from above through holes in the slab similar to cast-in-place concrete alternative described above. Consolidation of the upper lift of grout should be performed through holes in the slab above. Although the gap can be dry packed from below, this is a considerably less effective alternative as confirmed by the research results noted above.

Regardless of whether the new wall is cast-in-place concrete, shotcrete or CMU, some shrinkage or sagging will probably occur creating a crack at the joint. To account for the resulting reduction in effective aggregate interlock along this joint, it may be prudent to use a lower coefficient of friction, and increase the size the vertical dowels.

Figure 12.4.2-1C shows the conditions where the existing concrete diaphragm is in a pan joist or waffle slab system instead of a flat slab. For these types of floor or roof systems, the joists or waffle ribs must be preserved to avoid shoring. However, there is likely to be more flexibility in the extent of the openings that can be made through the slab between the joists/ribs, and temporary shoring will generally not be required. Where the new wall is parallel to the joists, it is preferable to locate the wall offset from the joist as shown in the detail. The holes in the slab may be made as intermittent keys, similar to the flat slab condition discussed above, or they can be made as relatively long slots or as a continuous opening the length of the wall. Additional diaphragm to wall shear transfer capacity can be obtained by doweling into the side of the adjacent rib.

Where the new wall is perpendicular to the joists, or at a waffle slab condition, the slab can be removed between the ribs as indicated in Figure 12.4.2-1D. Since installation of continuous horizontal wall bars through the perpendicular ribs is generally not possible, installation of one or two horizontal hoop ties may be required at the upper portions of the wall between the ribs. For CMU wall construction, the masonry will stop below the joists or ribs, and the large vertical gap up to the slab, between the ribs, will be completed with poured concrete.

*Connection to existing frames:* New walls are often located on or alongside existing frame lines. This generally allows for better use of the existing diaphragm collectors (beams) and of the existing frame columns as wall chords or boundary elements. It is almost always preferable to locate the wall alongside the frame beams instead of as an “infill,” within the width of the frame

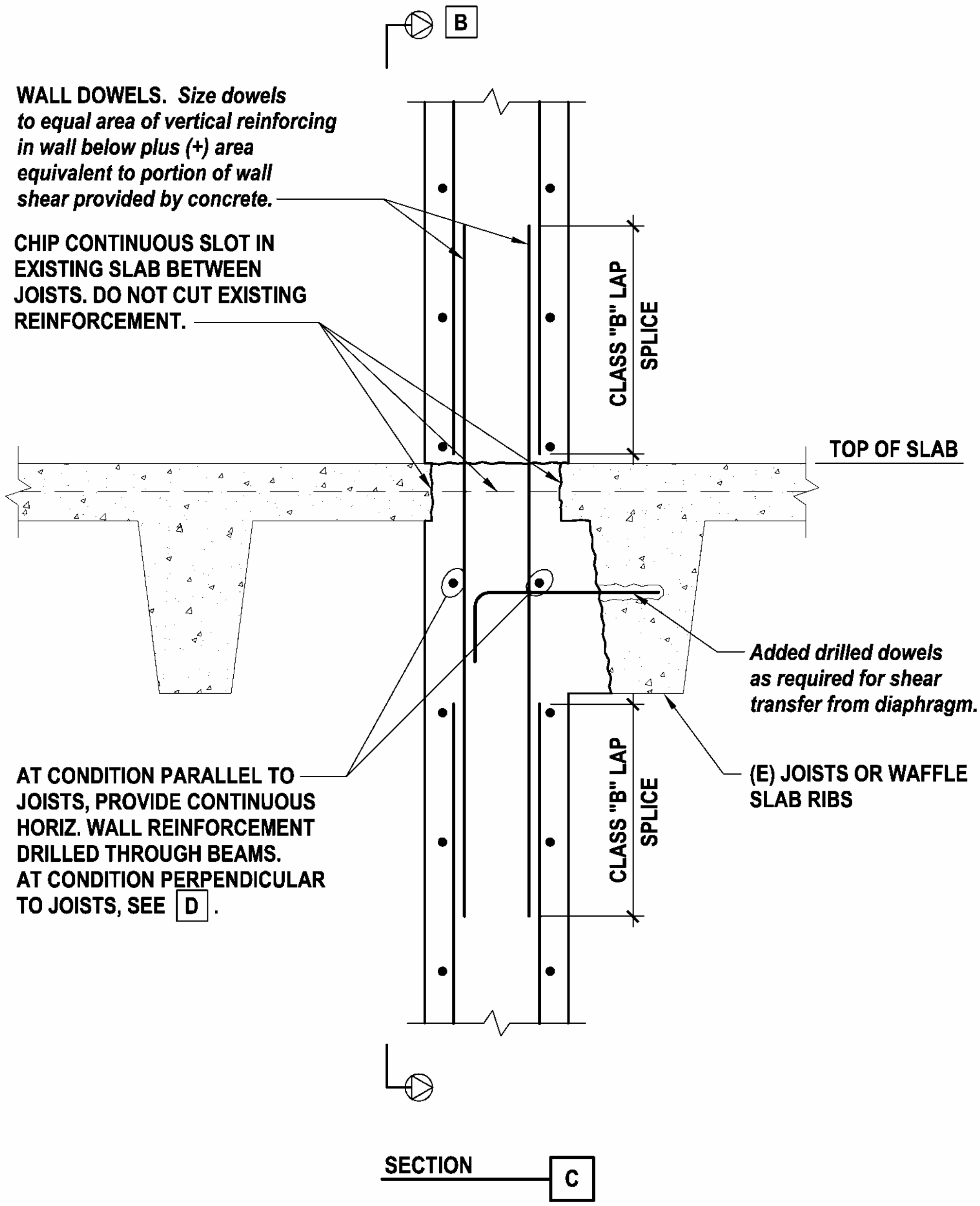



**Figure 12.4.2-1C: Connection of Concrete Wall to Concrete Joists or Waffle Slab**

beams and columns. When placed alongside the frame, the wall-diaphragm connections are as discussed above, and additional shear transfer and wall chord capacity can be obtained by doweling into the side of the beam and the column, respectively. Also, if diaphragm collector strengthening is required, the additional collector can be installed alongside the existing beam line and will be lead directly to the new wall. In the “infill” configuration, the vertical wall dowels must be threaded through the relatively densely reinforced beams, concrete placement and consolidation becomes significantly more difficult, and additional collectors do not connect directly to the new wall.

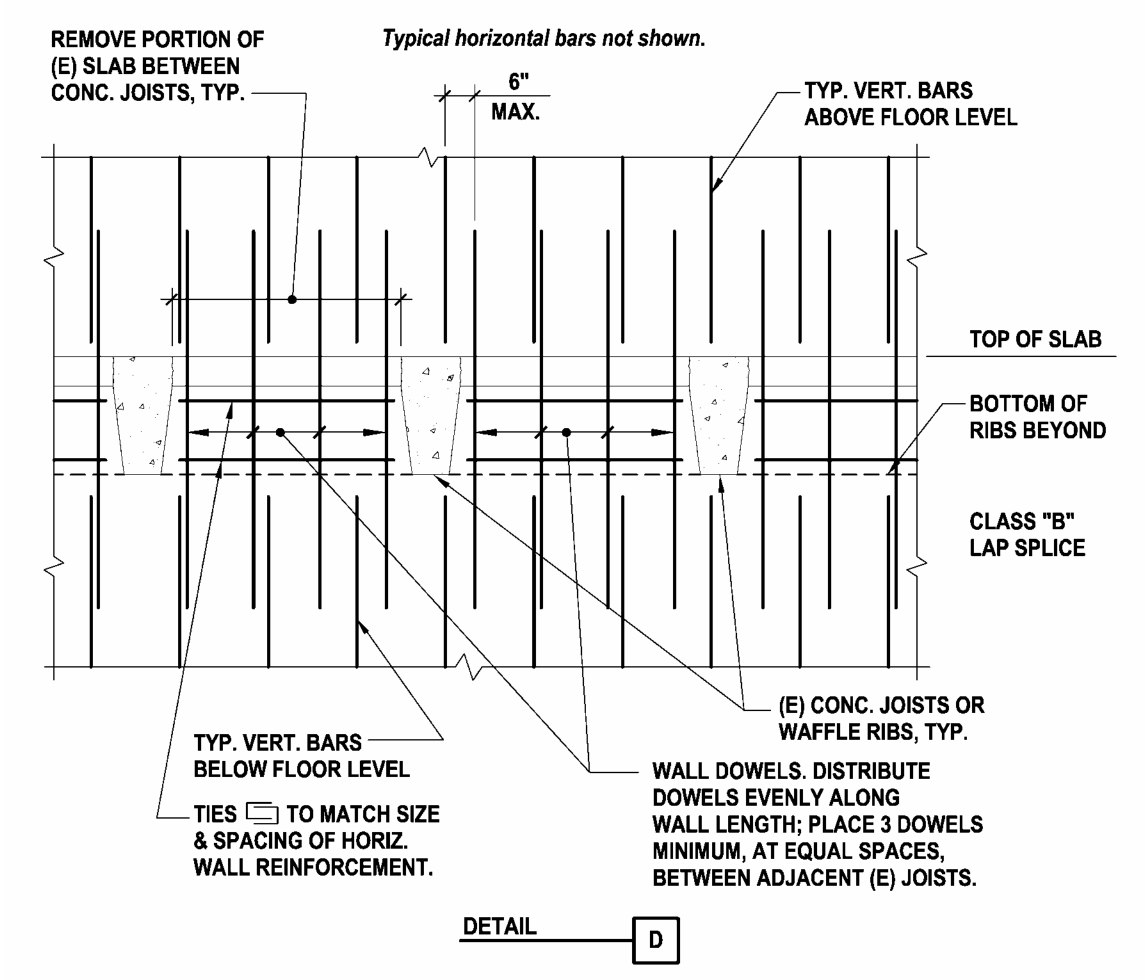



**Figure 12.4.2-1D: Concrete Wall Connection to Waffle Slab – Partial Elevation View**

If the existing columns have sufficient strength and appropriate reinforcement detailing, they may be used as the wall chord or boundary element, by doweling into the column. The effectiveness of this is limited by the amount of doweling that can be installed. In many cases, however, the existing column will require strengthening or jacketing, or new wall chords will be needed.

*Installation of additional collectors:* Installation of shear walls in a frame structure, especially in one with a complete frame system, will result in increased diaphragm demands at the individual walls. An advantage of locating the new wall at an existing frame line is that the existing beams can then be used as a collector. However, insufficient rebar continuity and/or laps combined with highly concentrated diaphragm demand may still require strengthening. Refer to Section 12.4.3.

*Footings:* Addition of concrete or masonry shear walls will almost always require construction of new footings, or augmentation of existing ones, to support the added weight as well as to resist the increased and/or concentrated overturning demand. In many cases, the overturning uplift demand will require installation of tie downs. Where the new wall is located between column frame lines, instead of directly on or along one frame line, new foundations can be used to engage more than one column to resist the uplift demand.

### *Cost/Disruption Considerations*

In general, shotcrete walls are less expensive than cast-in-place concrete because at least one side of the wall forming is eliminated. If shotcrete can be applied against an existing wall at stair, elevator or mechanical shafts, the cost savings of shotcrete is even greater. CMU walls are generally less costly, per square foot, than either shotcrete or cast-in-place concrete walls. However, CMU walls may not provide comparable strength or stiffness, requiring the addition of more linear feet of CMU walls than either cast-in-place concrete or shotcrete walls.

Construction of new shear walls in an existing building can be very disruptive to any building occupants. Noise, vibration, and dust associated with many operations, especially cutting holes through and drilling dowels into concrete, can be transmitted throughout a concrete structure. Placing cast-in-place concrete, shotcrete or even grouted masonry is a wet process and very messy. Shotcreting in an enclosed area creates differential pressures that can spread debris beyond nominal construction barriers. Also, excavation and drilling operations and the use of mechanized and/or truck mounted equipment associated with installation of new foundations can be very disruptive.

### *Construction Considerations*

The existing concrete surfaces to be in contact with the new concrete walls should be cleaned of all finishes, paint, dirt, or other substances and then be roughened to at least attempt to provide 1/4” minimum amplitude aggregate interlock at joints and bonded surfaces. At overhead joints where such preparations may be less effective, as discussed in the *Detailing Considerations* section above, additional dowels can be used with less roughening.

For shotcrete applications, separate trial test “panels” at the overhead joints should be included with the normal preconstruction test panels. These test joints should be cored to inspect the adequacy of the surface preparation and the joint bond. Nozzle operators should have several years experience with similar structural seismic improvement applications.

In addition to the usual concrete/shotcrete core sampling and testing, the overhead joints should be cored to allow inspection of the joint quality and determine whether or not repairs are needed.

For CMU shear walls, practical limitations on placement of wall reinforcing steel must be considered. In particular, use of “seismic comb” type of joint reinforcement (a prefabricated mesh of welded wire reinforcement used as transverse reinforcement at boundary elements of CMU walls) has often proven to be very difficult to install and the resulting rebar congestion interferes with grouting operations.

### 12.4.3 Provide Collector in a Concrete Diaphragm

***Deficiencies Addressed by the Rehabilitation Technique***

Inadequate or missing collector
Inadequate diaphragm chord capacity

***Description of the Rehabilitation Technique***

Addition of a new collector or strengthening of an existing collector is often needed when new steel braced frames or concrete shear walls are added to an existing building. The new collector must extend as far as necessary, often one or more bays from one or both ends of the new brace or wall, to draw the required shear demand from the existing diaphragm. The new collector will be constructed of reinforced concrete or steel, generally depending on whether the general building upgrade involves installation of new concrete shear walls or steel braced frames. The new collector will most often be installed at the underside of floor. At roofs, the collector may be placed either from below or above the roof.

In cases where the existing diaphragm chord is absent or inadequate, the mitigation approach will be similar to that used for collectors.

***Design Considerations***

*Research basis:* For new reinforced concrete collectors, see the discussion of tests by Bass, Carrasquillo, and Jirsa (1985) in Section 12.4.2.

For new steel collectors, Jiménez-Pacheco and Kreger (1993) tested single anchor connections between existing concrete and new steel members in order to examine shear transfer along the interface between these two elements. Results indicate that sandblasting the steel surface and applying a layer of epoxy at the interface between steel and concrete can substantially increase the force level at which the interface begins to slip. Also, the use of spring washers may reduce long-term anchor bolt relaxation, maintaining the first-slip force capacity over time. For applications where significant inelastic deformations are expected, a thick layer of nonshrink grout between the steel and concrete was found to increase deformation capacity, though it decreased ultimate strength slightly. Filling the annulus between the bolt and washer with epoxy resulted in greater connection stiffness than that exhibited by specimens with unfilled annuli or those filled with nonshrink grout.

*Material selection - reinforced concrete or steel:* In reinforced concrete buildings with some sort of concrete slab floor system, especially one with joists, waffle ribs or beams crossing the path of the collector, the most common material choice for the new collector is reinforced concrete. Often, this choice is made because concrete is aesthetically compatible with the surrounding structure, especially in a condition exposed to view. However, concrete is selected principally because it is compatible with the deformation characteristics of the diaphragm it is connected to. A concrete collector is bonded to, and is integral with, the concrete slab diaphragm and the strain deformations of the collector are the same as the deformations of the diaphragm system. At a steel plate collector, the elongation of the plate is not compatible with the diaphragm slab. As the collector load accumulates towards the connection to the new wall or brace, the elongation of the plate accumulates as well. The threaded rod anchors connecting the plate collector to the diaphragm in the zone of greatest elongation can become overloaded to failure by the plate bearing on the bolts. This can lead to a “zipper-like” failure mode as the adjacent anchors assume the load of the failed anchors and become overloaded in turn. This behavior can occur even at relatively short collectors if the elongation exceeds the available annulus gap around the anchor. To avoid this, special detailing is required as discussed in the *Detailing Considerations* section below.

*Impact on architectural and M/E/P systems and components:* A new collector often must extend one or more entire bays away from the new wall or brace in order to draw the necessary load from the existing concrete diaphragm. Installation of the new collector at the underside of the existing floor slab impacts any existing ceilings, partitions, ductwork, plumbing, lighting, etc., located along its entire length. As a result, the new collectors will often have a greater impact on the building’s other systems than the new walls or braces themselves. Furthermore, consideration of these impacts will often affect placement of the new walls or braces. In many cases, the new walls and their associated collectors are located along the exterior edge of the building specifically to avoid or minimize these impacts on other building systems, especially in a case where building occupancy is maintained during the construction.

In some cases, it may be possible to locate the new collector at the top surface of the existing diaphragm. At roofs, new collectors can be placed on top of the roof diaphragm, provided that any conflicts with roof mounted equipment, pads or penthouses can be accommodated or avoided. More importantly, placement of collectors on top of the roof slab requires careful consideration of the impact on roof drainage and waterproofing systems. At floors, the opportunity exists if a new concrete topping or structural overlay is proposed. In this case, the reinforcement for the collector can be embedded in the topping. Also, if a new raised floor system is being installed, it may be possible to locate a new collector in the space beneath the new floor.

*Weight of new collector:* The gravity load capacity of the existing slab, waffle ribs, joists or adjacent beams must be adequate to support the additional weight of the new collector, especially for a new concrete collector that may represent a considerable load. In some cases, the new collector may need to be designed to support itself as it spans between existing girders or columns. In others, it may be required to adjust the location of the collector, and the new shear walls, if the existing floor or roof slab cannot support the new loads.

### ***Detailing Considerations***

*Connection of a reinforced concrete collector to existing concrete diaphragms or collectors:* A typical detail of the installation of a new reinforced concrete collector to the underside of an existing concrete slab diaphragm is shown in Figure 12.4.3-1. The primary considerations are to provide a good bond between the new and existing concrete and to provide adequate access ports for concrete placement and consolidation. The contact surface must be thoroughly cleaned and roughened for good shear transfer performance. It is best to place concrete from above through pour ports made in the diaphragm. The ports will need to be at least 4 inches in diameter. Care must be taken to locate existing diaphragm reinforcement before cutting the ports to avoid cutting any bars in what is likely to be a lightly reinforced slab.

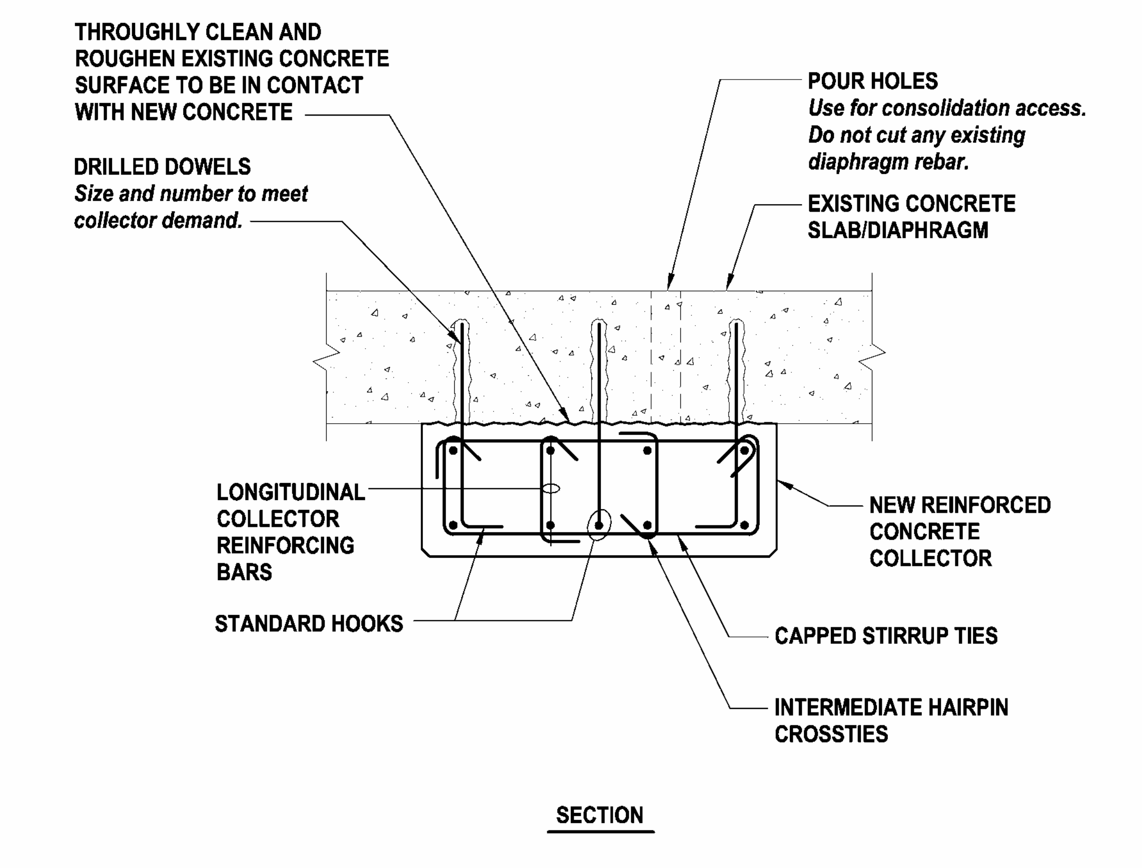



**Figure 12.4.3-1: Concrete Collector at Concrete Slab**

The required length of collector will be determined primarily by the existing diaphragm shear capacity. Lightly reinforced diaphragms can deliver only a limited load per foot, requiring long collectors. Also, for thin diaphragm slabs, the shear capacity of each drilled dowel will be limited, requiring more dowels. If the collector crosses any existing beam or girder, a splice must be made through the existing member. Horizontal holes can be drilled through the member and dowels installed to lap with the main collector reinforcing bars on each side. Care must be taken to avoid cutting any reinforcement, either main longitudinal bars or stirrups, in the existing beam.

If the existing floor or roof diaphragm is a waffle or pan joist system, the continuous collector will almost always be placed below the ribs, as shown in Figure 12.4.3-2, to avoid excessive drilling and rebar splicing. In this condition, the voids between the ribs, above the dropped collector, will be filled with reinforced concrete. Advantages of this condition are that the drilled dowels can be installed into the sides of the ribs instead of the relatively thin cover slab, and making pour ports through the slab is likely to be less problematic. Also, although the new collector may weigh more in this condition, the waffle or joist ribs are much more likely to have adequate strength to support the added weight.

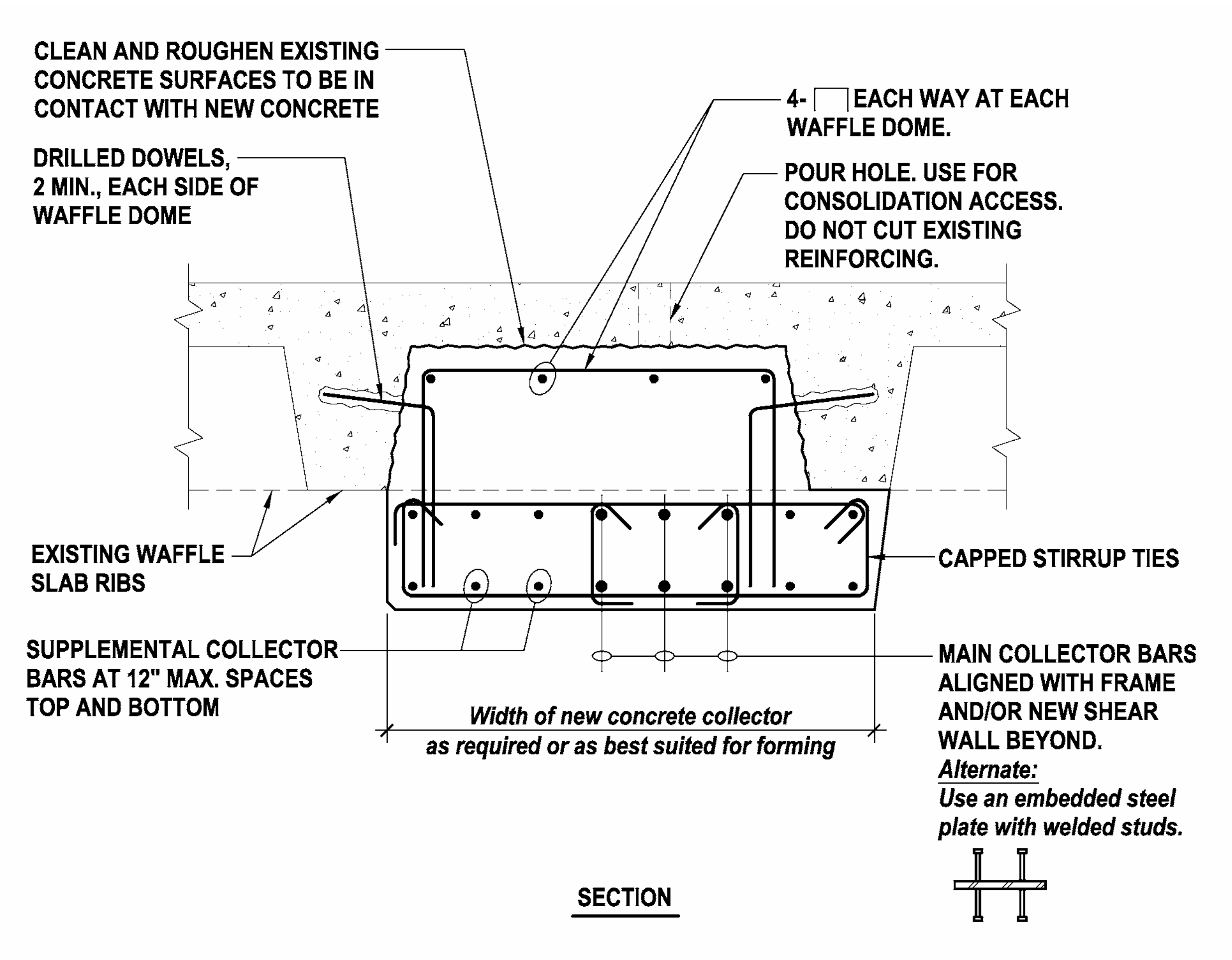



**Figure 12.4.3-2: Concrete Collector at Waffle Slab**

In many cases, the new collector will occur along an existing beam or girder line. Often, this will occur if the task is to strengthen a diaphragm edge chord or if the new walls occur at the building's exterior. Figure 12.4.3-3 shows two generic conditions that can be used in this case. In this condition, the dowels will always be placed into the beam, and the combined beam-collector member will easily be designed support the added weight. However, special care should be taken to avoid cutting any beam stirrups or slab diaphragm reinforcement, especially at an exterior edge condition, with the pour holes.

In any of these collector configurations, a significant portion of the main reinforcement can be provided by a steel plate instead of by bar reinforcement (refer to Figure 12.4.3-2). This option may be best for conditions of very high loads, where installation of a high strength steel plate may be preferred over placement of many large bars.

*Connection of a steel collector to existing concrete diaphragms:* Steel plate also may be used as the collector in lieu of a reinforced concrete member. A steel collector will have to be installed in

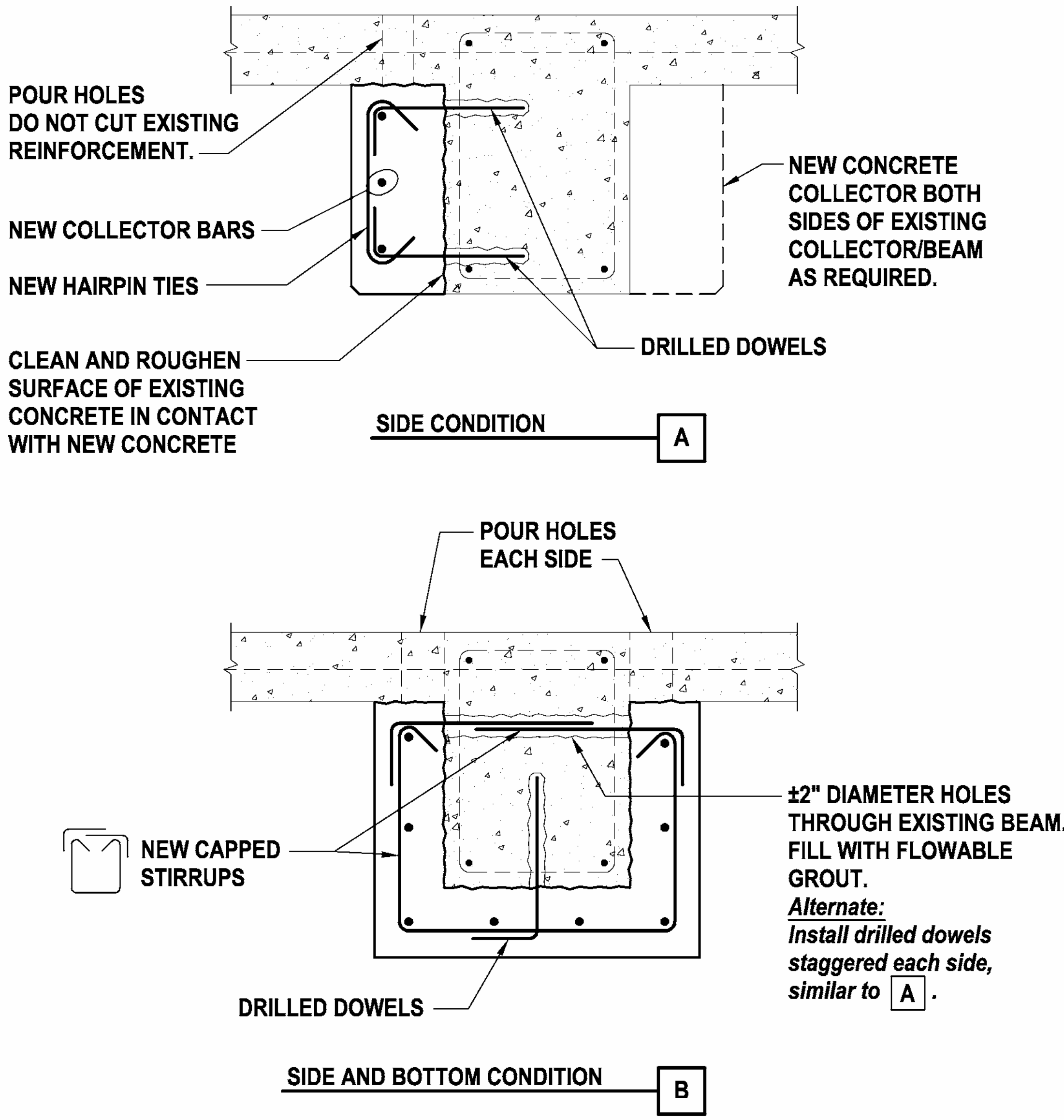



**Figure 12.4.3-3: Concrete Collector at Existing Beam**

manageable sections, generally about 10 to 20 feet in length, and will be connected to the concrete diaphragm with drilled threaded rod anchors set in adhesive or epoxy. In almost all cases, the steel plates will be installed at the top of the diaphragm as shown in Figure 12.4.3-4. Although possible, it is extremely difficult to install heavy plate sections, connect the bolts and make the necessary welded splices from below.

As discussed in the *Design Considerations* section above, the primary concern with a steel plate collector is its lack of strain compatibility with the concrete diaphragm, unless the collector is

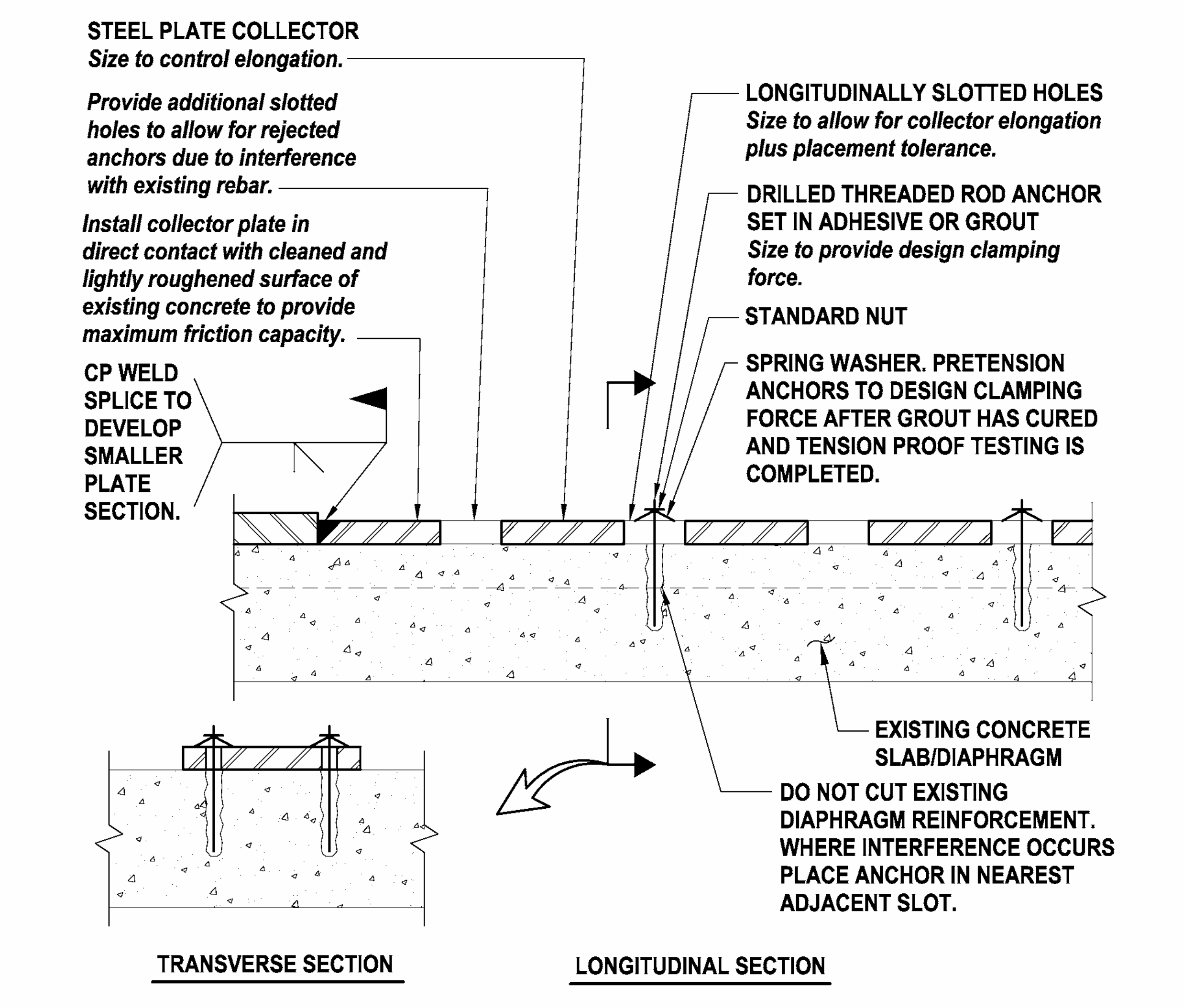



**Figure 12.4.3-4: Steel Plate Collector**

very short. The strain deformation of a steel collector will vary from zero at its free end to a maximum at the connection to the wall or brace while the concrete diaphragm will not experience similar deformations. In effect, the steel collector will stretch like a very stiff rubber band relative to the concrete diaphragm. This relative deformation is difficult to accommodate, especially in relatively long collectors. To do that, several conditions must be considered. First, the various plate sections of the collector must be stepped in size so the strain is distributed relatively equally along the length of the collector. Second the plates must be sized to limit the maximum elongation to a reasonable amount of about one or two inches. Third, the threaded rod anchors must be installed in slotted holes to allow the design elongation to occur without bearing on and overloading the anchors. Fourth, to allow the slip to occur between the collector and diaphragm, load transfer must be accomplished by friction using specially calibrated spring washers to generate the appropriate clamping force in the anchors.

### *Cost/Disruption Considerations*

Collectors have significant cost/disruption impact in a retrofit project primarily due to their length. They impact many building systems over a relatively large area compared to the impact associated with the walls themselves. This is especially true if general renovation of architectural and M/E/P systems is not included in the project. Thus, any available means of reducing collector length will probably be cost effective. A collector installed at the exterior edge of a diaphragm will generally be less costly than one installed in the interior and one installed above the diaphragm will be easier to install and, generally, less costly than one installed from below. However, installation of any collector can be very disruptive to any building occupants, due to the noise and vibration caused by drilling and coring through concrete, as well as the likely need to relocate various utilities and service distribution systems.

A comparison of the cost between reinforced concrete and steel plate collectors is very difficult to make. In general, the cost of either type of collector installed from below the diaphragm will likely be similar, because so much of the cost will be related to the impact on other systems. The cost of a steel plate collector may be less than one of reinforced concrete, but only for collectors installed from above the diaphragm, and particularly on a roof.

### *Construction Considerations*

Existing concrete surfaces to be in contact with new concrete or steel plate collector should be cleaned of all finishes, paint, or other substances that could impair bond and shear transfer capability. Surfaces to receive new concrete should be roughened to provide ¼” amplitude aggregate interlock to prevent slip. However, since slip is expected to occur as a steel collector elongates, only light sandblasting may be required to assure development of the appropriate friction.

Installation of grouted anchors and/or dowels for steel plate collectors will require relative precision. They must be installed at the middle of the long slotted holes, with only a small tolerance, to allow the plate to elongate without bearing on the anchor. If existing rebar is encountered at a location, that location should be abandoned and the anchor installed in an available adjacent or nearby slot. Since it is reasonable to expect that this will occur with some frequency, a substantial amount of extra slotted holes must be available. For instance, if anchors are required at 12” on center, slotted holes should be provided at 6” on center.

The complete penetration welded splices of the relatively thick steel plate collectors are likely to be problematic. Making one-sided complete penetration welds in relatively thick plates will cause the plates to curl. To control this, the plate sections may need to be anchored down, with anchors placed in addition to the required shear anchors, and welded in place. Also, removal of backup bars will be difficult or impossible. Notches may be made into the concrete slab, and any remaining gap between the bottom of the steel plate collectors and the concrete diaphragm slab must be filled with grout to assure adequate friction at the concrete-steel interface.

Overhead construction of reinforced concrete collectors will require careful consideration and planning of how the reinforcing steel is placed and secured, prior to closing up the forms from below. Making the pour access ports and any sleeve holes for continuous rebar will require careful scanning of the slabs, waffle ribs, and beams to locate existing reinforcement to avoid cutting any existing rebar.

While the inspection, sampling, and testing required for reinforced concrete collectors is not particularly different from what is required for other seismic force-resisting reinforced concrete work, some special considerations do occur for steel plate collectors. The welded splices will require careful, non-destructive testing and thorough inspection. The shear anchors must be located at the middle of the slotted holes with some precision, and they must be extensively proof tested in tension. The holes with anchors must be free of any grout that could reduce the range of slip. Any gap between the bottom of the plate and the concrete slab must be grouted. The installation of the spring washers must be carefully inspected and tested to assure development of the design clamping force.

***Proprietary Concerns***

The basic materials are generic.

### 12.4.4 Enhance Column with Fiber-Reinforced Polymer Composite Overlay

***Deficiencies Addressed by the Rehabilitation Technique***

Inadequate shear capacity
Inadequate concrete compression strain and stress capacity due to lack of concrete confinement
Inadequate lap splice

***Description of the Rehabilitation Technique***

The use of a fiber-reinforced polymer (FRP) overlay with columns has proven to be an efficient rehabilitation technique in both the building and bridge construction industries. Columns are overlaid with unidirectional fibers in a horizontal orientation, thus providing shear strengthening and confinement similar to that provided by hoops and spirals used with circular columns, and stirrups and ties used with rectangular columns. The confinement enhances the concrete compression characteristics, provides a clamping action to improve lap splice connections, and provides lateral support for column longitudinal bars.

The preferred strength hierarchy for a building type structure is strong-column, weak-beam. Where the strength hierarchy results in weak-column, strong-beam (and is not considered acceptable by the designer due to, perhaps, concern for a soft story mechanism), the use of FRP overlay as flexural strengthening should not be used, unless there are extenuating circumstances and a very detailed analysis and design are performed. The uncertainty of strain compatibility between the FRP and column longitudinal bars and between the FRP and substrate, the lack of vertical strain capacity as a result of using FRP as longitudinal reinforcement, and the anchorage of the FRP at column ends and at points of contra-flexure deem this approach as undesirable. Other techniques presented in this document should be used in this situation.

See Section 13.4.1, "Enhance Shear Wall with Fiber-Reinforced Polymer Composite Overlay, Fiber-Reinforced Polymer Composite Overview," for background information.

***Design Considerations***

*Research basis:* Seible and Innamorato (1995) is one of the original ground-breaking research efforts of this topic and serves as an excellent source for understanding and design equations.

The primary deficiencies of a column are typically the lack of shear strength capacity and post-yield shear deformation capacity, as observed during many earthquake events and in laboratory testing. For shear assessment, two column locations need to be evaluated for shear: the end region within the plastic hinge zone (where the concrete shear strength degrades), and the region away from the flexural hinges, where there is no concrete shear strength degradation.

The FRP overlay provides confinement to enhance the concrete strain and stress capacity. Confinement is more effective for circular sections than rectangular sections. For circular sections, the passive radial pressure exerted by the FRP overlay on the gross concrete section, which is a result of the concrete lateral dilation, provides confinement. Dilation (similar to the concrete splitting action when performing a pure axial compression test on a concrete cylinder) occurs as result of the compression force, which is influenced by the level of axial load and flexural forces. For a rectangular section, dilation is only arrested at the corners of the section, thus relying on the concrete to arch between the corners, resulting in a reduced concrete core size. Due to the lack of effective confinement by the FRP, it is recommended to limit the rectangular section to a 1.5 depth-to-width aspect ratio and a width or depth dimension less than 36 inches, unless a special study is performed.

The confinement afforded by this technique does marginally increase the flexural strength and stiffness of the column, but not to the degree of concrete jacketing. The marginal increase is due to the higher concrete stress capacity of the cover and core concrete, hence reduced neutral axis depth, and is located within the what is called the *primary* hinge zone. This increase is over about half the depth of the column at each column end (where double-curvature occurs). Consequently, there is a greater moment demand just beyond this region, within the *secondary* hinge zone. Confinement enhancement, therefore, extends from the end of the column through the primary and secondary hinge zones, as shown in Figure 12.4.4-1A. Note that the categorization of *primary* and *secondary* hinge zones comes from Seible and Innamorato(1995).

The confinement pressure also serves to clamp the lap splice connection of the column longitudinal bars. The thickness (effective clamping pressure) needed for lap splices is derived differently from the confinement requirements, however, as test results indicate that, at a dilation strain of about 0.001, lap splice slippage is initiated. These results, combined with the empirically derived radial pressure requirement to prevent slippage, determine the FRP overlay thickness.

The FRP overlay thickness is determined for each of the three deficiencies; zones for these are shown in Figure 12.4.4-1A. As noted by Priestley, Seible, and Calvi (1996), the maximum of the three at any section should be provided; it should not be the sum, as reported in some other documents. This is because the lap-splice clamping action and compression concrete confinement occurs on opposite sides of the column, so the maximum requirement of the two will serve both well. Shear resistance of the FRP occurs along the column face parallel to shear load direction. The FRP anchors the concrete compression strut and is designed to maintain

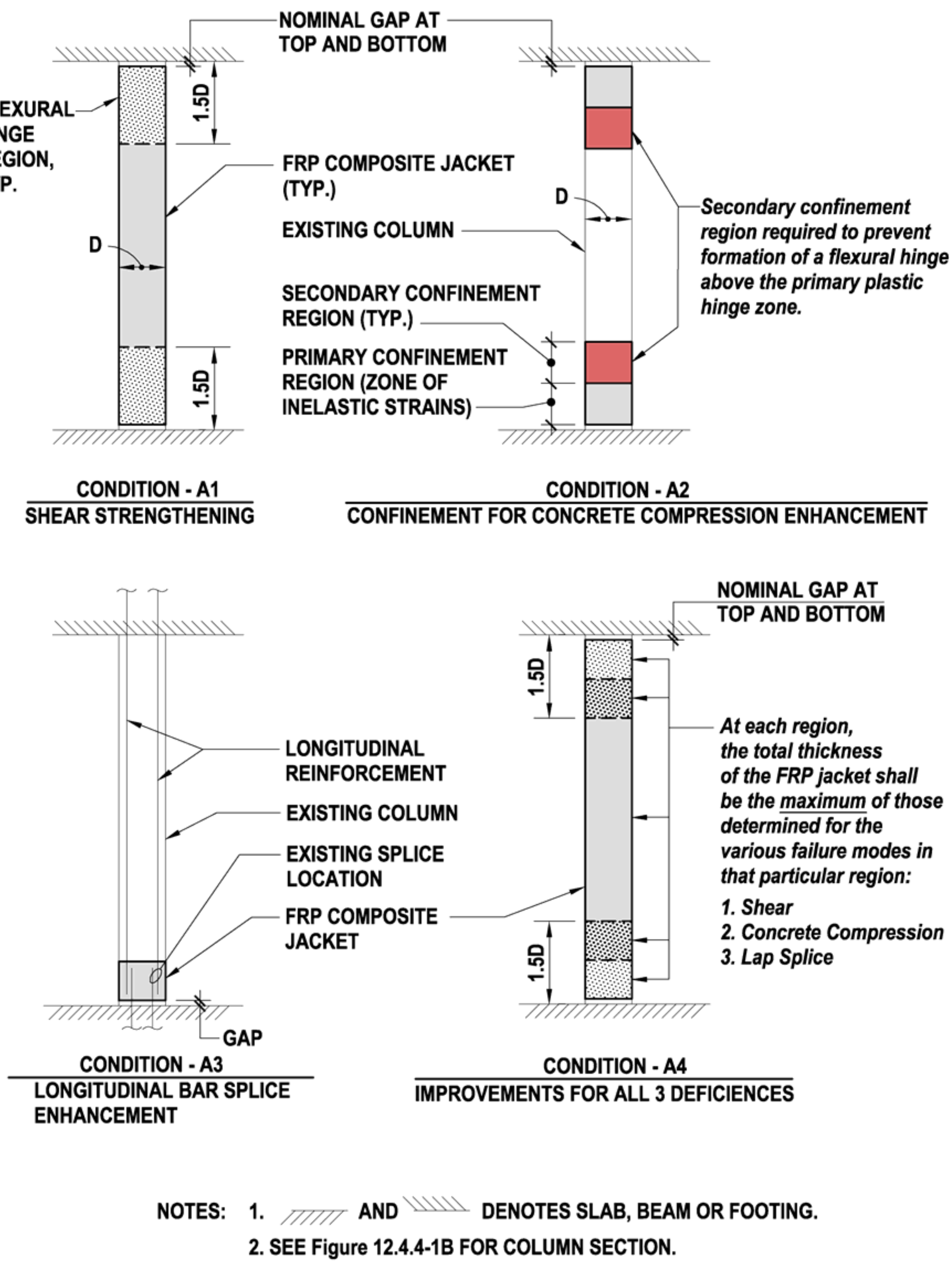



**Figure 12.4.4-1A: Seismic Retrofit of Columns Using FRP Composites**

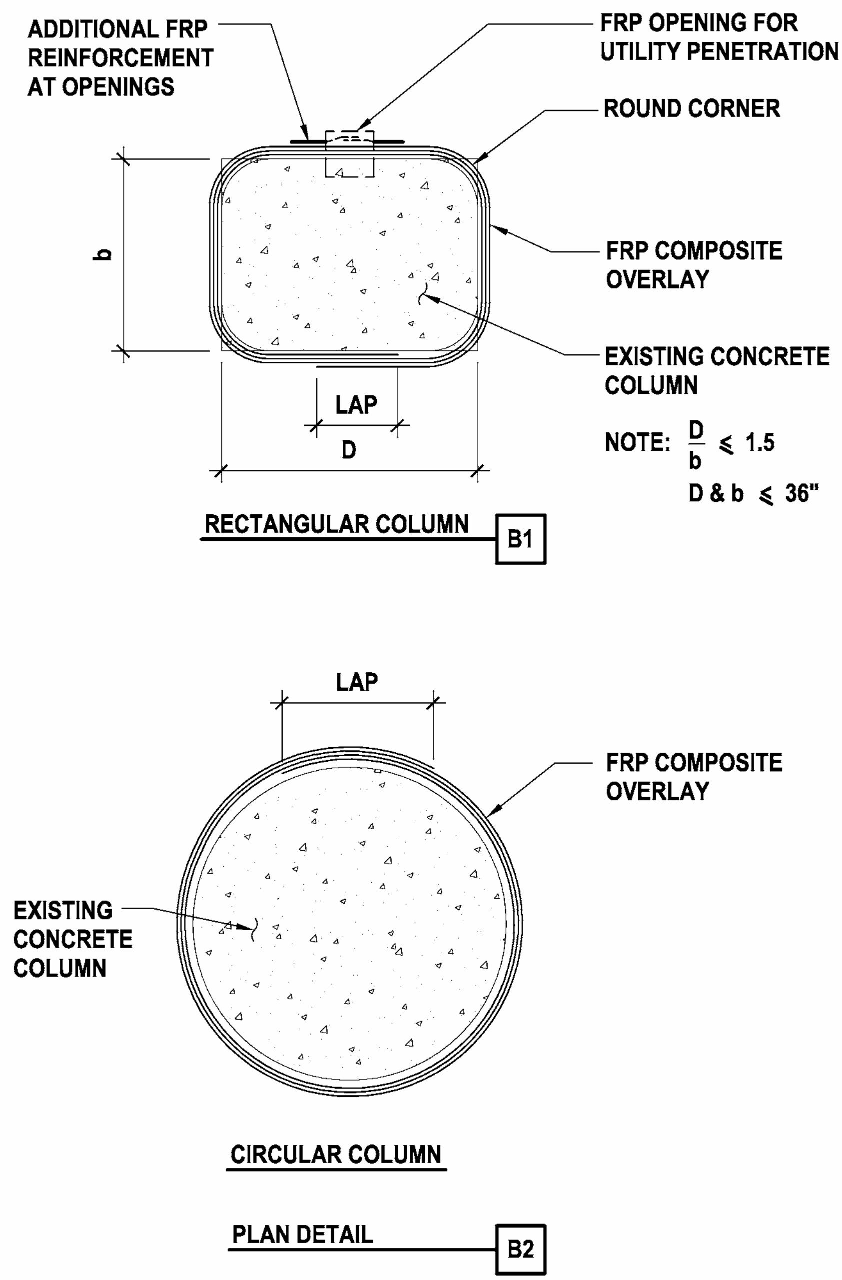



**Figure 12.4.4-1B: Seismic Retrofit of Columns Using FRP Composites**

aggregate interlock over the crack length. This contribution is a by-product of the hoop tension required for the confinement, so FRP composite thickness for shear need not be added to that required for confinement.

With successful mitigation of the three deficiencies, column flexural hinges can be developed and deformation capacity will be increased.

### *Detailing Considerations*

The lap splices should be staggered, like that done with steel reinforcement, to mitigate a weak plane. As shown in Figure 12.4.4-1A, a nominal gap of about ½" between the FRP composite and the boundary elements (slab, beam or footing) is provided to prevent the overlay from bearing and, consequently, increasing the flexural capacity or stiffness of the column.

### *Cost/Disruption*

To appropriately evaluate the cost of a retrofit scheme using and FRP overlay in comparison to traditional retrofit concepts (such as concrete or steel jacketing), one needs to consider the cost of the raw material, the level of specialization required by the contractor to install the system, the cost of labor and equipment, the cost of quality control and quality assurance, the temporary impact of disruption during construction, and the permanent impact to the building functions. Although FRP overlays are relatively expensive compared to steel and concrete, they can offer advantages when only limited access is available or minimal disruption of existing conditions is desired.

### *Construction Considerations*

Access all around the column is often limited due to partition walls, ceilings and other architectural components, as well as structural elements of a building. Conditions at the base of the column in the lowest story must be carefully considered. The extent of FRP application normally will extend to the top of footing which may require local slab removal. The floor slab in the area may also interact with the column and affect strengthening requirements. This effect should be considered or a gap placed between the slab and column to prevent interaction.

### *Proprietary Concerns*

See Section 13.4.1 for brief discussion of proprietary concerns.

## 12.4.5 Enhance Concrete Column with Concrete or Steel Overlay

### *Deficiencies Addressed by the Rehabilitation Technique*

Inadequate shear capacity
Inadequate axial compression capacity
Inadequate flexural plastic hinge confinement
Inadequate lap splice

### *Description of the Rehabilitation Technique*

Adding a fiber reinforced polymer (FRP) composite overlay to a concrete column is a recent approach to addressing seismic deficiencies, and is discussed in Section 12.4.4. Adding a concrete or steel jacket is a more traditional method of enhancing a deficient concrete column,

Figure 12.4.5-1 provides examples of concrete and steel jacketing for a rectangular column. Because FRP overlays have become more common, this section will focus on FRP overlays.

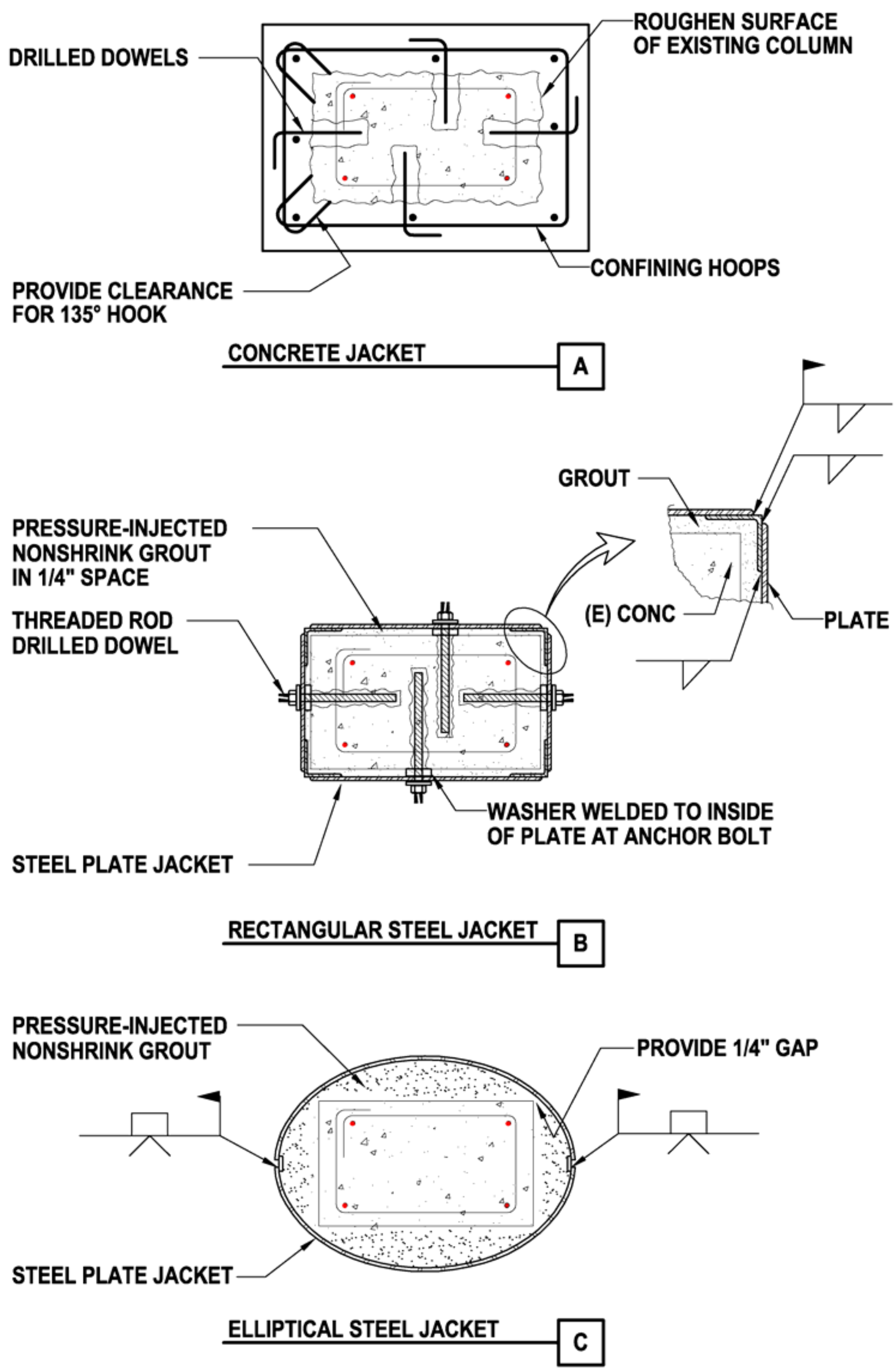



**Figure 12.4.5-1: Concrete and Steel Overlays for Concrete Columns**

### *Design Considerations*

*Research basis:* Some research on concrete overlays is contained in FIB (2003); other related work is discussed in Section 12.4.5. Design of the overlay or jacket uses typical ACI 318 concrete design principles. Sufficient drilled dowels between the overlay and existing concrete should be provided to achieve composite action. Research on steel overlays includes Engelhardt, et al., (1994).

### *Detailing Considerations*

*Concrete overlays:* Concrete jacketing will take up a larger cross section than either FRP overlays or steel overlays. The surface of the existing concrete must be roughened appropriately. Reinforcing steel will need to be in at least two pieces to get it around the existing column. 135 degree hooks are required for confining ties and may dictate the size of the overlay in order to provide enough room for the hook extension.

*Steel jackets:* Steel jackets require at least two pieces to get around the existing column and involve field welding. Like FRP overlays, when the aspect ratio of a rectangular column gets too large, the jacket becomes less effective. The California Department of Transportation (Caltrans) uses elliptical jackets in these situations. The corners of the existing column will need to be trimmed so the steel can pass by. There is a gap between the steel and the concrete of at least ¼" that is filled with grout. A gap is also provided at the ends of the column to permit rotation without engagement in bearing of the steel jacket.

### *Cost/Disruption*

To appropriately evaluate the cost of a retrofit scheme using and FRP overlay in comparison to traditional retrofit concepts (such as concrete or steel jacketing), one needs to consider the cost of the raw material, the level of specialization required by the contractor to install the system, the cost of labor and equipment, the cost of quality control and quality assurance, the temporary impact of disruption during construction, and the permanent impact to the building functions. Although FRP overlays are relatively expensive compared to steel and concrete, they can offer advantages when only limited access is available or minimal disruption of existing conditions is desired.

### *Construction Considerations*

*Concrete overlays:* Because of the difficulty of placing an overlay on all sides of an existing column, concrete overlays are typically done with cast-in-place concrete, rather than shotcrete. The need for formwork is a significant disadvantage for concrete overlays, compared to FRP and steel overlays. Placing the concrete and vibrating are also challenging due to access limitations at the top of the column where it runs into beams or slabs. Pour ports or holes in the diaphragm are needed.

*Steel overlays:* Steel overlays are typically 3/16" or ¼" thick and become quite heavy. Access and lifting issues in existing buildings can force the overlay to be broken down into pieces, increasing the amount of field welding necessary to join the pieces back together.

***Proprietary Concerns***

Unlike FRP overlays, no proprietary issues have been identified with using concrete of steel jackets.

### 12.4.6 Enhance Concrete Moment Frame

***Deficiencies Addressed by the Rehabilitation Technique***

Inadequate global shear capacity
Inadequate lateral displacement (global stiffness) capacity
Inadequate ductile detailing for shear strength, confinement, or strong column/weak beam

***Description of the Rehabilitation Technique***

An alternative method to either add strength and stiffness to an existing concrete moment frame or correct non-ductile detailing deficiencies of the frame members is by direct enhancement: increasing the size of the columns and beams of the frame with new reinforced concrete. This method entails adding a jacket of reinforced concrete around the existing columns and beams, an approach similar to jacketing by steel or fiber wrap. The new concrete may be either cast-in-place or shotcrete. This approach is relatively rarely employed in the U.S. because it is labor intensive, but when existing openings must be maintained and walls or braced frames cannot be installed, it may be rehabilitation technique worth considering.

***Design Considerations***

*Research basis:* Alcocer and Jirsa (1991) tested reinforced concrete beam-column connections subjected to unidirectional and bidirectional cyclic loading up to displacements equivalent to 4% interstory drift. Jacketing of columns resulted in strong column – weak beam behavior, with increases in peak strength of over four times the existing assembly. Jacketing of beams as well as columns improved joint confinement, decreased stresses on existing beam reinforcement, and provided some additional strength increases. The use of jacketing to repair damaged frames was also tested. When comparing the performance of a specimen in which a damaged column and joint were repaired by jacketing to that of a similar specimen in which the original column and joint were undamaged, the damaged and repaired assembly exhibited 65% of the strength at 2% drift and 50% of the stiffness at 0.5% drift relative to the undamaged retrofitted specimen.

Reinhorn, Bracci, and Mander (1993) performed shake table tests on a one-third scale model of a three-story, one-bay by three-bays, concrete moment frame. The tests compared the performance of an unretrofitted building, subjected to peak ground acceleration 0.30 g, to that of a similar structure retrofitted by jacketing interior columns, post-tensioning added longitudinal column reinforcement, and providing a reinforced concrete “fillet” infill around beam-column joints. These retrofit measures were intended to ensure strong column - weak beam behavior, enhance joint shear capacity, and improve anchorage for discontinuous beam reinforcement. Tests demonstrated that the retrofitted structure exhibited significantly reduced column damage, especially in the first story, and improved ductility in yielding beams; however, lateral displacements remained quite large, reaching a maximum inter-story drift of 2.1% at the first floor.

*Impact on architectural and M/E/P systems and components:* Enlarging columns, beams, or joints may impact existing ceilings, partitions, ductwork, plumbing, lighting, or usable floor space. The designer should keep in mind that the space required for access, formwork, and finishes will be greater than the final dimensions of the jacketed member.

*System design:* New transverse and longitudinal reinforcement should be designed such that members are flexurally-governed and beams yield before columns do. The jacketing of the frame will result in increases in system stiffness that may result in increased attracted load. Both new and existing concrete should be considered in developing composite properties for modeling and design. Joint shear can be the weak link in existing moment frames. A significant focus of the Alcocer and Jirsa (1991) research was to provide special angles at the joint region to improve joint confinement. Stiffness at 0.5% drift was well predicted when cracked properties of 0.5EI were used for the beams and 0.5EI were used for the columns. Stiffness decreased about 40% to 50% as drifts increased from 0.5% to 2.0%.

***Detailing Considerations***

*Surface preparation:* In order for a new reinforced concrete jacket to act compositely with an existing member, sufficient bond must exist between the new and existing concrete. The Alcocer and Jirsa (1991) tests made use of findings from Bass, Carrasquillo, and Jirsa (1985). In the Alcocer and Jirsa (1991) tests, they used a handheld electric chipping hammer to reveal some aggregate. The amplitude is not documented in the report, but the implication is that it was less than the traditional ¼" amplitude value. Dust was cleaned with a thick brush and vacuum cleaner. A bonding agent was not used, and the existing concrete surfaces were not saturated in all cases. Bonding agents in current practice are not common, but some engineers do recommend prewetting the existing concrete.

*Column jacketing:* Beams are typically the same plan dimension or narrower that the supporting column. New column bars can then pass by the existing beam bars through holes cut in the floor slab. Alcocer and Jirsa (1991) explored distributed longitudinal bars vs. bundled bars in the corners of the new jacket, but did not find significant differences. It is not possible to put a one-piece closed hoop around an existing column. One approach is to place a U-bar on three sides with a single leg closure piece, such as shown in Figure 12.4.4-2A. Alcocer and Jirsa (1991) used two overlapping L-shaped bars, each with 135-degree hooks at the ends. A minimum column jacket thickness of 4" was recommended.

*Beam jacketing:* When beams are jacketed, the bottom and sides can be increased in dimension, but typically the top of beam must remain at the existing top of slab level. In the Alcocer and Jirsa (1991) tests, slots were cut in the top of the slab just above the beam with holes at the ends of the slot through slab. Inverted U-bars were placed in the slot and through the holes. The bottom of the inverted U-bars overlapped around beam longitudinal steel with U bars surrounding the beam. A minimum beam jacket thickness of 3" is recommended. Care must be taken to investigate existing beam reinforcing, so that appropriate locations for new beam longitudinal bars can be located to miss existing reinforcing in orthogonal beams and in columns.

*Joint enhancement:* In the Alcocer and Jirsa tests (1991), vertical steel angles were placed in holes cut through the slab at the four corners of the column and then welded to horizontal steel bars above the slab and below the beam, to form a cubical cage. In the Reinhorn, Bracci, and Mander (1993) tests, joint strength was enhanced by infilling between orthogonal beams with concrete to create a diamond-shaped “fillet” in plan. Reinforcing at the outer ends of the fillet was placed through holes cut into the beam just under the slab and at the base of the beam.

#### *Cost/Disruption*

The cost and disruption associated with jacketing a concrete frame is significant because it potentially involves complicated formwork, reinforcement, and concrete pouring over much of the building, as opposed to more common shear wall approaches where the walls are only placed in localized areas. Beam jacketing is much more invasive and time consuming and is typically less important than the column jacketing. Alcocer and Jirsa (1991) reported that the construction time required for jacketing beams and columns was nine times that required for jacketing columns alone.

#### *Construction Considerations*

*Noise and disruption:* Removing existing column, beam, ceiling and floor finishes is disruptive. Chipping the cover concrete off of existing frame members is noisy and disruptive to occupants, as is cutting holes in the floor slab. As such this particular technique can be less desirable than many others.

*Mix design:* Due to the narrow thickness of the jacket and difficult working conditions, the mix design should emphasize ease of placement, by using small size aggregate and water-reducing admixtures. In the Alcocer and Jirsa (1991) tests, they used 3/8” maximum size aggregate and superplasticizer.

*Concrete placement:* Concrete should be placed from above through holes in the slab for both columns and slabs. Holes have to be big enough for both the concrete hose and the vibrator. Consideration can be given to leaving gaps at the top of the forms on the sides of the beams where they meet the slabs for air relief vents.

#### *Proprietary Concerns*

The basic materials are generic.

## 12.5 References

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# Chapter 13 - Building Type C2b: Concrete Shear Walls (Bearing Wall Systems)

## 13.1 Description of the Model Building Type

Reinforced concrete walls in a building will act as shear walls whether designed for that purpose or not. Therefore, cast-in-place concrete buildings that contain any significant amount of concrete wall will fall into this category. However, there are two distinctly different types of concrete wall buildings, those that contain an essentially complete beam/slab and column gravity system, and those that use bearing walls to support gravity load and have only incidental beam and column framing. In this document, these building types have been separated and are designated **C2f** for the gravity frame system and **C2b** for the bearing wall. This section covers the bearing wall type. In this type of building, all walls usually act as both bearing and shear walls. The building type is similar and often used in the same occupancies as Building Type **RM2**, namely in mid- and low-rise hotels and motels. This system is also used in residential apartment/condominium type buildings.

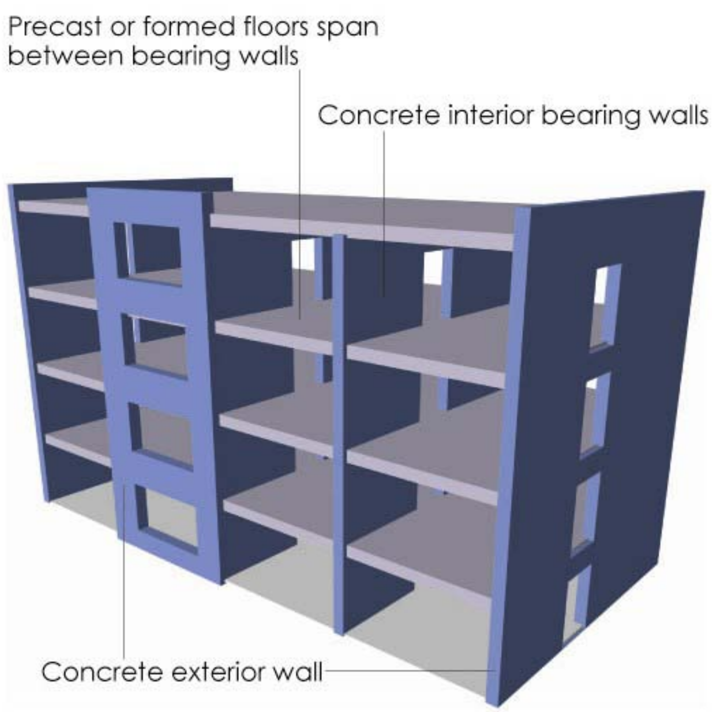


**Figure 13.1-1: Building Type C2b: Concrete Shear Walls (Bearing Wall Systems)**

### *Variations Within the Building Type*

In order for this framing system to be efficient, a regular and repeating pattern of concrete walls are required to provide support points for the floor framing.  In addition, since it is difficult and expensive to make significant changes in the plan during the life of the building, planning flexibility is not normally an important characteristic when this structural system is employed. The occupancy type that most often fit these characteristics are residential buildings, including dormitories, apartments, motels, and hotels. These buildings will often be configured with reinforced concrete bearing walls between rooms—also acting as shear walls in the transverse direction, and reinforced concrete walls on the interior corridor acting primarily as shear walls in the longitudinal direction.  Sometimes the longitudinal lateral system includes the exterior wall system, although this wall is normally made as open as possible.  In any case, the wide variation in structural layouts and occupancies that is included in Building Type **C2f** is not seen in Type **C2b**.

It is seldom possible to plan a building layout that provides complete gravity support with walls. Often, local areas are supported with isolated columns, and sometimes beams and girders are also necessary, but story heights in these buildings are usually small and added depth in the floor framing system is difficult to obtain.  The extent of such beam and column framing often causes confusion between Building Types **C2b** and **C2f**, but buildings should have an essentially complete gravity frame system to be placed in **C2f**.  If significant plan area is supported solely by walls, the structures are normally classified as **C2b**.

There are important variations in floor framing systems employed in this building type, and their adequacy to act as a diaphragm is an important characteristic of this building type as discussed below.

### *Floor and Roof Diaphragms*

The parallel layouts of supporting walls and the need to minimize story heights normally leads to the use of one-way uniform-depth concrete floor systems.  Cast-in-place and precast systems, both conventionally reinforced and prestressed, have been employed.  The precast systems are often built up of narrow planks, which may not provide an adequate diaphragm unless a cast-in-place topping is provided.  In addition, the precast systems may be placed with only a very narrow bearing area on the supporting walls, which may be inadequate to provide vertical support during seismic movements.  The adequacy of the shear connection between slab and walls is also often an issue for both cast-in-place and precast systems.

### *Foundations*

The bearing walls obviously require some kind of starter beam at grade for construction purposes and this often leads to a simple continuous grade beam system.  In poor soils, piles or drilled piers may be added below the grade beam.  A continuous mat foundation may also be employed due to the short spans and total length of bearing points in this building type.

## 13.2 Seismic Response Characteristics

Due to the extent of wall, bearing wall buildings will be quite stiff. Elastic and early post-elastic response will therefore be characterized with lower-than average drifts and higher-than-average floor accelerations. Damage in this range of response should be minimal.

Overall post-elastic response may often include rocking at the foundation level. If rocking does not occur, the height-to-length ratio of shear walls in these buildings may force shear yielding near the base, which may lead to strength and stiffness degradation.

Global stability may also be compromised by poor connections between floor slab construction and bearing walls.

### *Shear Wall Behavior*

When subjected to ever increasing lateral load, individual shear walls or piers will first often force yielding in spandrels, slabs, or other horizontal components restricting their drift, and eventually either rock on their foundations, suffer shear cracking and yielding, or form a flexural hinge near the base. Shear and flexural behavior is quite different, and estimates of the controlling action are affected by the distribution of lateral loads over the height of the structure.

Yielding of spandrels, slabs, or other coupling beams can cause a significant loss of stiffness in the structure. Flexural yielding will tend to maintain the strength of the system, but shear yielding, unless well detailed, will degrade the strength of the coupling component and the individual shear wall or pier will begin to act as a cantilever from its base. In this building type, the coupling elements are often slabs, and their lack of bending stiffness may reduce or eliminate significant coupling action.

Rocking is often beneficial, limiting the response of the superstructure. However, the amplified drift in the superstructure from rocking must be considered. In addition, if varying wall lengths or different foundation conditions lead to isolated or sequencing rocking, the transfer of load from rocking walls must be investigated. In buildings with basements, the couple created from horizontal restraint at the ground floor diaphragm and the basement floor/foundation (sometimes called the “backstay” effect) may be stiffer and stronger than the rocking restraint at the foundation and should be considered in those configurations.

Shear cracking and yielding of the wall itself is generally considered undesirable, because the strength and stiffness will quickly degrade, increasing drifts in general, as well as potentially creating a soft story or torsional response. However, in accordance with FEMA 356, shear yielding walls or systems can be shown to be adequate for small target displacements. Type **C2b** buildings will often fall into this category.

Flexural hinging is considered ductile in FEMA 356 and will degrade the strength of the wall only for larger drifts. Similar to rocking, the global effect of the loss of stiffness of a hinging wall must be investigated.

## 13.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 13.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Selected deficiencies are further discussed below by category.

### *Global Strength*

Due to the extensive use of walls, buildings of this type seldom have deficiencies in this category, unless significant degradation of strength occurs due to shear failures.

### *Global Stiffness*

Similar to strength, global stiffness is seldom a problem in this building type. However, the effect of coupling slabs on initial stiffness and the potential change in stiffness due to yielding of these coupling slabs or wall-beams over doors should be investigated.

### *Configuration*

The most common configuration deficiencies in this building type are weak or soft stories created by walls that change configuration or are eliminated at the lower floors. It is difficult to provide the needed ductility at the weak story, and often strength must be added. Completely discontinuous walls also create a load transfer deficiency for both overturning and shear. In such cases, collectors are often needed in the floor diaphragm, and supporting columns need axial strengthening.

### *Load Path*

A common deficiency in this building is weakness in the load path from floor to walls, either collector weaknesses or shear transfer weakness immediately at the floor wall interface. Local transfer can be strengthened by adding concrete or steel corbel elements, dowels, or combinations of these components. As indicated above discontinuous walls also often create load path deficiencies.

### *Component Detailing*

The most common detailing problem in this building type is an imbalance of shear and flexural strength in the walls, leading to pre-emptive shear failure. This deficiency may be shown to be not critical with small displacement demands, walls can be strengthened in shear with overlays of concrete, steel, or FRP.

The layout of walls often forces coupling between walls through the slab system or across headers of vertically aligned doors. These coupling components are seldom designed for the coupling distortions that they will undergo, particularly in older buildings. Short lengths of slabs between adjacent walls receive damage by coupling action that could compromise the gravity capacity. It is difficult to add strength or ductility to these slab areas, but vertical support at support points can be supplemented by corbels of steel or concrete. Damage to headers over doors often does not contribute to deterioration of overall response and can sometimes be acceptable. Local areas of wall can also be strengthened by overlays of concrete, steel, or FRP.

**Table 13.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C2b Buildings**

| **Deficiency** | | **Rehabilitation Technique** | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane wall shear strength | Concrete/masonry shear wall [12.4.2] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [13.4.1]<br>Steel overlay | | Seismic isolation [24.3]<br>Reduce flexural capacity [13.4.4] | |
| | Insufficient flexural capacity | Concrete/masonry shear wall [12.4.2] | Add chords [12.4.3] | | | |
| | Inadequate capacity of coupling beams | Concrete/masonry shear wall [12.4.2] | Strengthen beams [12.4.2]<br>Improve ductility of beams [12.4.2] | | | Remove beams |
| Global Stiffness | Excess drift (normally near the top of the building) | Concrete/masonry shear wall [12.4.2] | Fiber composite wrap of columns to improve lateral displacement capability [12.4.4]<br>Provide detailing of all other elements to accept drifts<br>Concrete wall overlay [21.4.5] | | Supplemental damping [24.4] | |
| Configuration | Discontinuous walls | Add wall or adequate columns beneath [12.4.2] | Fiber composite wrap of supporting columns [12.4.4]<br>Concrete/steel jacket of supporting columns [12.4.5] | Improve connection to diaphragm [13.4.3] | | Remove wall |
| | Soft story or weak story | Add strength or stiffness in story to match balance of floors | | | | |

**Table 13.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C2b Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Re-entrant corner | Add floor area to minimize effect of corner | | Provide chords in diaphragm [12.4.3] | | |
| | Torsional layout | Add balancing walls [12.4.2] | | | | |
| Load Path | Inadequate collector | Add steel or concrete collector [12.4.3] | | | | |
| | Inadequate slab bearing on walls | | | Add diagonal dowels [13.4.3]<br>Add steel ledger [13.4.3] | | |
| Component Detailing | Wall inadequate for out-of-plane bending | Add strongbacks [21.4.3] | Concrete wall overlay [21.4.5] | | | |
| | Wall shear critical | | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [13.4.1] | | Reduce flexural capacity [13.4.4] | |
| Diaphragms | Precast components without topping | | Improve interconnection [22.2.11]<br>Add topping | | | |
| | Inadequate in-plane shear capacity | | R/C topping slab overlay<br>Fiber composite overlay [22.2.5] | | | |
| | Inadequate shear transfer to walls | | Add diagonal drilled dowels [13.4.3]<br>Add steel angle ledger [13.4.3] | | | |

**Table 13.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C2b Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Diaphragms (continued) | Inadequate chord capacity | New concrete or steel chord member [12.4.3] | | | | |
| | Excessive stresses at openings and irregularities | Add chords [12.4.3] | | | | Infill openings [22.2.4] |
| Foundations | See Chapter 23 | | | | | |

[ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques.

### *Diaphragm Deficiencies*

Precast floor systems used in this building type often provide inadequate diaphragm behavior that could lead to bearing failures at the floor wall interface, particularly when no topping slab is present. Some topping slabs used primarily for leveling and smoothing the floor are inadequately tied to the precast elements or the walls, and are too thin or poorly reinforced to act as diaphragms on their own. See Chapter 22.

### *Foundation Deficiencies*

This building type often places large demands on the foundation system. If rocking is shown to be a controlling displacement fuse for the building, the foundations must be investigated to assure that these displacements can safely occur. See Chapter 23.

## 13.4 Detailed Description of Techniques Primarily Associated with This Building Type

### 13.4.1 Enhance Shear Wall with Fiber-Reinforced Polymer Composite Overlay

#### *Fiber-Reinforced Polymer Composite Overview*

In addition to this section, seismic rehabilitation techniques using fiber-reinforced polymers are described in several other sections in this document, including Section 12.4.4, “Enhance Column with Fiber-reinforced Polymer Composite Overlay,” and Section 22.2.5, “Enhance Slab with Fiber Reinforced Polymer Composite Overlay.” This section provides a general overview of FRP characteristics.

*Composite makeup and application:* The construction industry’s term *Fiber-Reinforced Polymer* (FRP) refers to a composite material made up of carbon, fiberglass or aramid (Kevlar) fibers that are bound together by either a resin or ester polymer. These are commonly referred to as glass fiber-reinforced polymer (GFRP) composite, carbon fiber-reinforced polymer (CFRP) composite, or aramid fiber-reinforced polymer (AFRP) composite, respectively. The raw fibers (synonymous to filaments) can be woven to form mesh (uni- or bi-directional with the orientation of the warp to the weft at 45 or 90 degrees), collected together to form a carbon tow winding (an untwisted bundle of continuous filaments) or sheet, or pultruded to form a prefabricated plate or other shape.

Usually, at least two layers of FRP are applied to the exterior concrete (substrate) surface. For beam and column applications, one layer should be considered sacrificial, due to the possibility of abrasion and the fact that lap splices are used and potentially compromise a layer’s effectiveness.

*Mechanical properties:* The mechanical property in the direction of the fibers of GFRP, CFRP and AFRP is an essentially linear-elastic response followed by sudden rupture. The three material systems have different rupture strains and moduli. This variation becomes an important consideration when selecting the fiber type. While the rupture strain of each material is different, they are all significantly less (by approximately an order of magnitude) than that of conventional concrete reinforcement, which leads to compatibility issues.

Creep (increase in strain), stress rupture (reduced tensile capacity) and stress corrosion (corrosion that is dependent on presence of stress to occur) are phenomena that occur when FRP is subjected to sustained loads, such as flexural strengthening of slabs for long-term gravity loads. Each type of composite responds differently to loading regime, environmental conditions, and matrix and fiber make-up: AFRP is prone to creep under sustained load and moderately so to stress corrosion, GFRP is prone to stress rupture (as low as 20% of the ultimate), but CFRP is robust to sustained loads. Page 115 of FIB (2001) reference provides more information. This document addresses seismic loading, which is of short duration; hence, FRP is not affected by these characteristics. Accidental sustained stress loading for seismic applications may occur and should be given special study. An example is the FRP diaphragm overlays where the fiber composite transfer on top of a floor to a wall can act as negative slab reinforcement and resist subsequent live loads. Gravity load enhancement is not addressed in this document, but it should be noted that it is affected by this characteristic.

For the same FRP composite, mechanical properties can vary between manufacturers and sometimes within the same manufacturer over the course of material production for a project. It is recommended that the contract documents clearly specify the performance requirements (such as force per unit width for each application) and the minimum ultimate rupture strain.

*Requirements at the FRP-to-substrate interface:* There are “contact-critical” and “bond-critical” applications between the composite material and the substrate surface.

Contact-critical applications are mostly limited to beam and column shear and confinement enhancement techniques, and it is preferred that the composite be wrapped around all sides of the element (i.e., made continuous). These applications do not require shear flow capacity between the composite to the substrate, so paint and other smooth finish materials may be left in place.

Bond-critical applications, such as discontinuous applications and wall applications, require shear flow capacity between the FRP overlay and the substrate. (Note that flexural strengthening for gravity load enhancement of slabs and beams is a bond-critical application, but it is not included in this document.) The surface preparation is important and a concrete surface profile of 3 (CSP 3) is required (as described in ICRI, 2003), which calls for all loose laitance, the weaker outer cement paste layer, and any unsound concrete to be removed. Light sand or water blasting readily achieves the desired result. The surface should be dust free at the time of applying the first resin layer. Resin putty is used to fill the voids, provide a smooth surface, and create a chamfered corner. Paint and other surface finishes must be checked for hazardous materials before removal.

All of the fiber composites have the inherent tendency to rupture prematurely at stress concentrations. Such concentrations are formed by burrs, sharp edges, protrusions, etc., in the substrate. Any sharp edge, or protrusion of any kind, must be removed during the preparation of the substrate to ensure a smooth surface free of dirt, grease, oil and finishes. Existing elements damaged by cracks or corrosion should be repaired prior to applying this rehabilitation technique.

The substrate surface should be essentially flat, so that the fibers are straight when positioned. An uneven surface prevents the fibers from lying straight, and upon loading, will tend to straighten, which will compromise the bond capacity. An out-of-plane angle of 1-2% will compromise the bond strength by initiating peeling of the FRP overlay from the substrate.

Typically, the weak link in bond-critical applications is the tensile rupture of the substrate (although the resin should also be checked). This weak link obtains peak strength at about 1/32-inch displacement, followed by complete loss of bond strength at about 1/3-inch. Further, test results and observed behavior show a finite development length of about a few inches and beyond that no additional load is developed in the composite material, meaning that the fiber material does not necessarily develop its full strength (see Teng et al. (2002) Figures 2.3 and 2.7). This behavior is unlike rebar reinforcement, where the bar development length is sized, and the bond area is provided to attain full rebar strength. The design, therefore, for FRP composite requires focus on bond strength and an awareness of crack patterns and the locations of inelastic deformations.

*Fiber and mechanical anchors:* Significant research on new means of anchoring FRP composite to a substrate is occurring at this time. The most common anchor consists of inserting carbon fiber or a mixture of carbon and glass fibers into a drilled hole in the substrate. The hole is then filled with epoxy, and the protruding strands are splayed to a cone or fan shape that is used to lap with the composite overlay. The splay is located between the overlay layers to enhance the splice connection. There are many variations of this approach being studied. Anchor spacing, edge distance and other issues that influence the performance are also being investigated. The design of the anchor and FRP composite overlay system, therefore, should be case specific and based on the most recent research.

*Durability:* Composite materials, if manufactured correctly and with the appropriate finish applied, can provide corrosion resistance, ultraviolet light resistance, fire resistance, and tolerance to variations in temperature – making them generally suitable for most environments. Specific consideration should be given to galvanic corrosion of CFRP, particularly where there is an electrolyte present and potential for mixing of metals. Although CFRP is otherwise corrosion resistant, it is a conductor and will change properties when heated. Locations where lightning strikes may occur, such as garage rooftops or exterior of buildings, will warrant the use of other FRP materials or grounding of metal grid to protect the CFRP. See FIB (2001), Section 9.11. For each environment application, the experience of manufacturers and researchers should be considered, and the manufacturer's warranties should be carefully considered. The designer must consider the myriad of environmental factors and develop an FRP solution that is appropriate for those conditions, and, more importantly ensure that the appropriate fiber and polymer material, surface protection, and finish is specified. As an example, where high humidity and/or high alkalinity are present, carbon fiber is the preferred choice. Where ultraviolet light is present, surface protection of the composite can be achieved with an acrylic or polyurethane based paint applied when the polymer is still tacky. For more durability information, refer to FIB (2001), Chapter 9.

*Constructibility:* In addition to the costs of material and installation of FRP composite, concrete surface preparation and final appearance requirements must also be considered. The surface finish requirements, as described above in the *Interface Requirements* section, need to be included and made clear in the contract documents. The woven texture of the mat applications does read through to the finish surface. If additional surface finishing is required, such as paint or a cementitous appearance, this can add significant cost, and should be coordinated with the architect and owner. For seismic rehabilitation applications, the FRP overlay is similar to structural steel braces, in that they are not typically required to support gravity loads and thus do not require fireproofing. However, some local jurisdictions do require that a fire-protecting surface be provided, such as intumescent paint.

The resin's shelf-life, pot-life, ambient temperature, ventilation, substrate moisture state and other environmental factors need to be carefully considered by the design team. For example, due to issues with offgassing during and following installation for some time, not only will ventilation be needed, but occupants may need to be temporarily displaced. Quality control procedures, which should be required by the contract documents, need to address these issues and be verified as acceptable by the engineer of the project and the inspector of record.

*Research basis:* Teng, et al. (2002) provides a summary of recent research projects and design equations suggested by researchers. Design equations and construction quality assurance requirements are also present in model codes, such as ACI 440.2R-02 (2002) and FIB 14 (2001). For a summary of the mechanical properties and design procedures, see Priestley, et al. (1996). For detailed material mechanical properties and chemical background information, see Kaw (1997).

### *Deficiency Addressed by the Rehabilitation Technique*

Inadequate shear capacity in a concrete shear wall

### *Description of the Rehabilitation Technique*

An FRP overlay is a technique that is used to enhance the in-plane shear capacity of a reinforced concrete shear wall. The overlay can be applied to one or both sides of a wall, and where possible, should wrap around the ends of the wall to aid in anchoring the overlay. The rehabilitation technique is bond-critical, regardless of whether or not the material wraps around the end of the walls. Uni-directional (horizontal-oriented) fibers are used to enhance the shear capacity, creating a predominately flexural post-yield response. Vertically-oriented fibers in bi-directional layouts will limit the vertical strains to that of the FRP composite, inhibiting the ductile behavior. Therefore, the rehabilitation technique for walls is limited to horizontally oriented fibers, unless there are extenuating circumstances. The shear enhancement may change the wall's response from a shear-dominated behavior to a flexural, sliding shear, or rocking behavior. At coupling beams, vertically-oriented fibers are typically used. See Figure 13.4.1.1A for examples of wall and coupling beam layouts.

### *Design Considerations*

*Research basis:* For wall-specific criteria, there has been an increasing amount of research on strengthening of unreinforced masonry walls, but less information on reinforced concrete walls. Ghobarah (2004) is one source; and Laursen, Seible, and Hegemier (1995), though based on CMU specimens, can be extended to some degree to reinforced concrete walls.

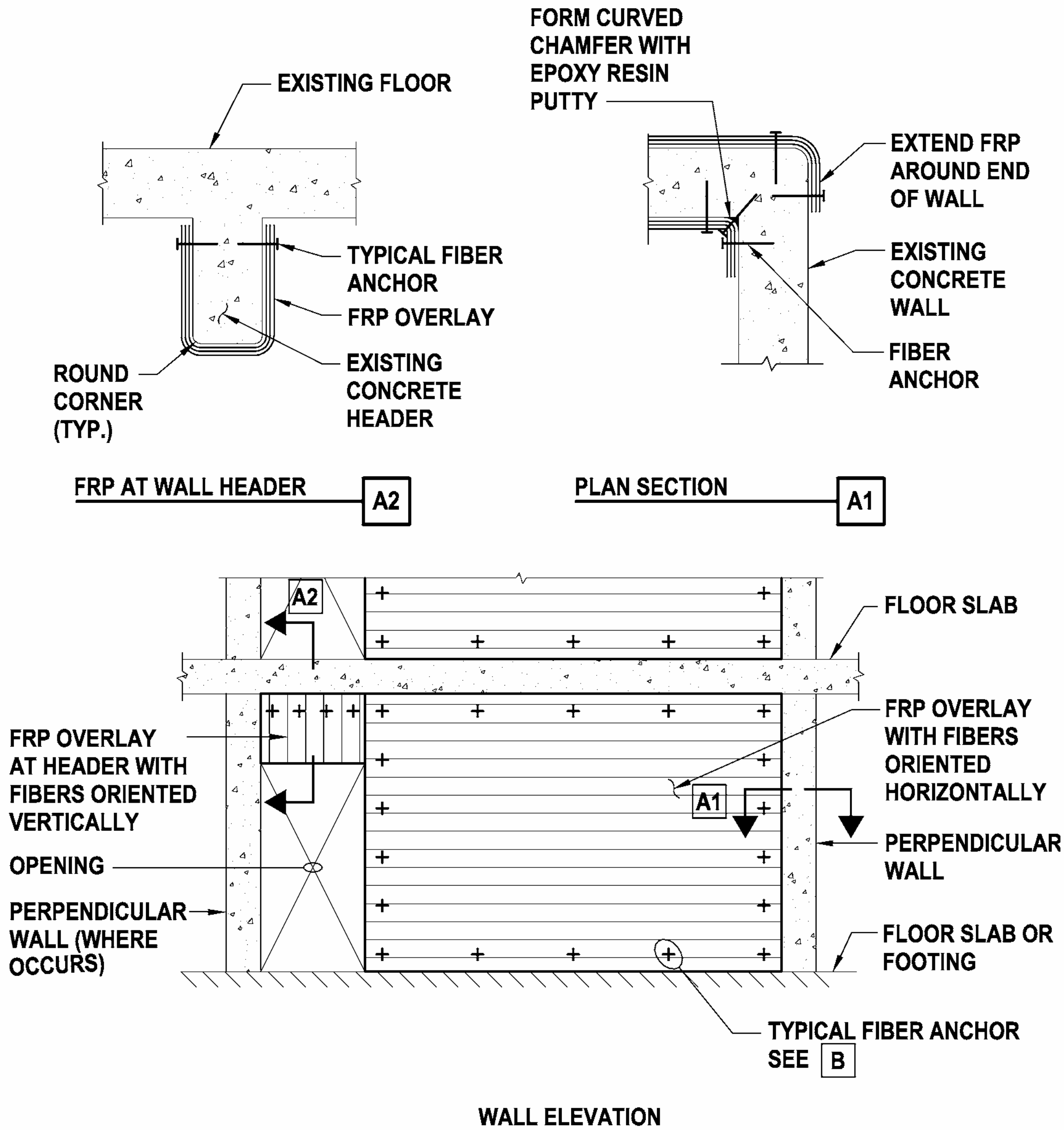


**Figure 13.4.1-1A: Shear Strengthening of Concrete Shear Walls Using FRP Composite**

The shear resistance contribution from the FRP is obtained in a similar manner to that used for wall reinforcement; the horizontal bars and FRP resist the horizontal shear (see Section 5.1 in Laursen, Seible, and Hegemier, 1995). The effective fiber area per unit width, and its contribution to shear resistance, is limited to the bond and anchorage strength capacity; providing additional fiber area will not provide additional shear capacity. Testing has been limited to mostly single and double layers of FRP per side of wall. Wrapping the material around the ends of the wall and/or providing fiber anchors will enhance the effectiveness of the overlay, particularly where cracks form, by increasing the anchorage and bond of the FRP to the substrate

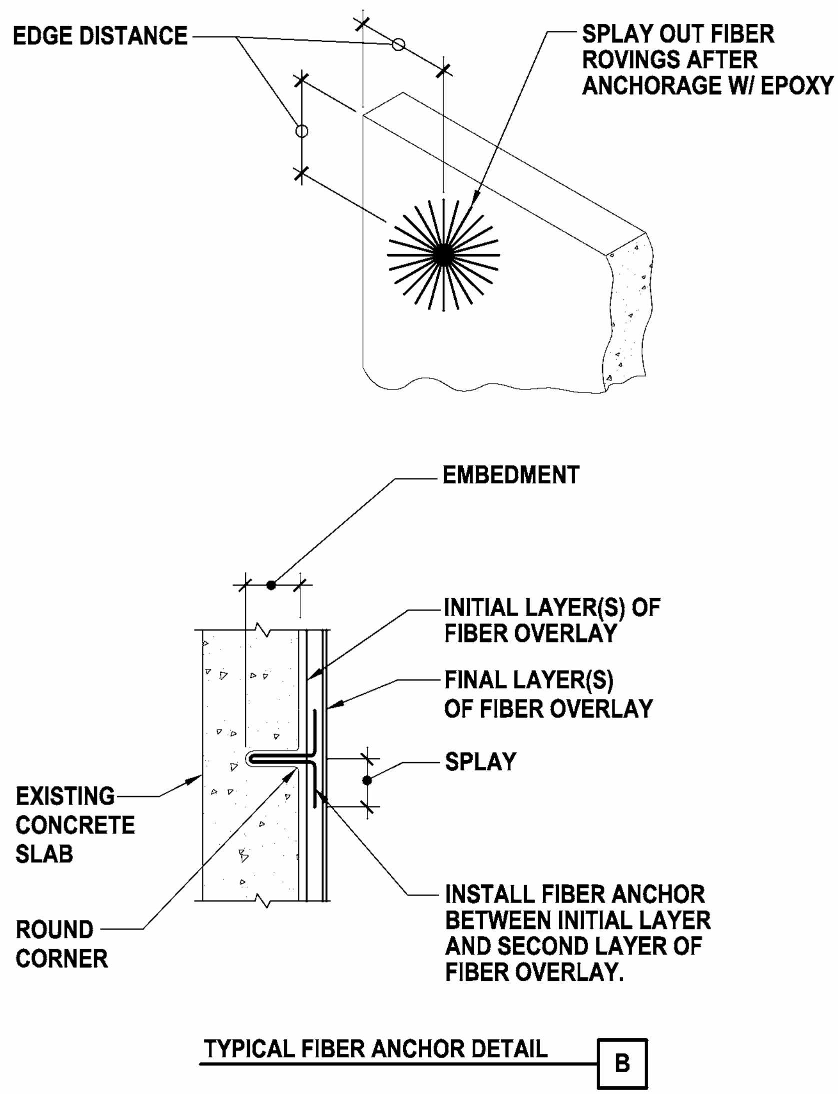


**Figure 13.4.1-1B: Fiber Anchor Details**

In terms of wall shear strength, the FRP overlay can be additive to the wall's concrete and reinforcement contribution. The ductility, however, is dependent on the type of governing mechanism. For shear-dominated walls (where the post-yield deformations require slippage at crack locations), the FRP has limited ability to accommodate such deformation; hence, there is likely very limited ductility. Conversely, a wall dominated by flexural yielding is able to accommodate the plastic deformations by the yielding of the wall vertical reinforcement, provided that the wall vertical reinforcement is developed. As a result, the typical goal in adding an FRP overlay is to make a wall flexurally-critical. For the case where flexural yielding occurs first, and is then followed by shear yielding as a result of the reduced concrete shear contribution (due to the reduced aggregate interlock effectiveness), ductility is significantly less than that with a flexural yielding response. Moreover, there is essentially no confining pressure afforded by the use of FRP overlay, except locally, where a fiber anchor is used. Therefore, lap splice performance is not enhanced with this technique and, depending on the size of bar, cover, and lap length, limited ductility may result.

Although testing with bi-directional (at plus and minus 45 degrees) fibers in wall overlays shows enhanced shear and flexural strengths and moderately enhanced displacement ductility capacity, the vertical force component of the fibers contribution to the flexural strength may have adverse affects. This may reduce the distribution of inelastic straining and/or change the strength hierarchy to that of shear controlled. Until further research is performed, horizontally-oriented FRP strengthening is recommended.

***Detailing Considerations***

Given the high dependence on the bond strength of the FRP overlay to the substrate, in-situ bond testing should be included as a requirement in the contract documents. A testing program will verify the design assumptions and assist in providing quality assurance. If pilasters are present, either within the wall length or at wall ends, installing fiber anchors or removing portions of the pilaster should be considered to enhance the anchoring of the FRP overlay.

***Construction Considerations***

As discussed in the *FRP Composite Overview* section earlier, the engineer should inspect the surface of the wall elements to be rehabilitated and note in the contract documents the surface condition and wall configuration (e.g., wall corner profile and wall-to-slab configuration). To aid in developing a sound bid price, the contract documents need to record the as-built condition, including surface anomalies and configuration, and the surface preparation requirements. If the surface has been board formed with wood planks, for example, calling this out in the construction documents will enable a more accurate bid. Bid documents should also require the contractor to view existing conditions before bidding.

***Proprietary Concerns***

Although the basic materials are generic, the fabric that will be supplied is proprietary. Strand orientation and density, epoxy overlays, preparation requirements, and installation procedures may be different between suppliers. Some suppliers may not have experienced applicators in the area of the project. These variations must be considered to achieve an adequate specification, particularly if competition between suppliers is desirable.

### 13.4.2 Enhance Deficient Coupling Beam or Slab

***Deficiencies Addressed by the Rehabilitation Technique***

Inadequate shear and bending capacity of coupling beams or slabs
Inadequate ductile detailing for shear strength

***Description of the Rehabilitation Technique***

Coupling beam deficiencies can be encountered in any type of building structure that has shear walls for their primary lateral force-resisting system. In a bearing wall structure, however, the coupling beam is often not a beam at all but only the relatively thin concrete slab linking the adjacent walls across a corridor. In some cases, particularly near the top of taller buildings, the restraint of a slab or beam in-line with the flexural deformed shape of a narrow shear wall can create coupling beam type issues at the tip of a single wall. In many cases, the slab consists of precast (and prestressed) hollow core planks, with or without a topping, with limited reinforcing steel at the link beam-wall joint instead of cast-in-place concrete. Furthermore, because bearing wall buildings are often residential buildings with short story heights, there is often no ability to create a dropped beam, and it is very difficult or impossible to increase the strength or ductility of the linking slab-beam itself. In these cases, the mitigation approach is to install a steel or concrete corbel to provide supplemental vertical support.

For those cases where there actually is a beam or deeper header linking the walls, the most common mitigation method to strengthen it or to correct its non-ductile detailing deficiencies is by direct enhancement with new added reinforced concrete. The new concrete is generally added to one face of the coupling beam, in concert with concrete strengthening of the adjacent shear walls as well. In some cases, new reinforced concrete can be added to both faces and extended along the walls for development. In addition, or as an alternative, it may be possible to install a limited number of diagonal dowels to enhance the shear capacity of the coupling beam to wall joint. In a case where there is enough beam or header depth, two groups of tied bars could be placed in an X-configuration, embedded within the new beam reinforcement cage. The new concrete may be either cast-in-place or shotcrete.

### *Design Considerations*

If the restraint of the slabs and beams described above is not needed for overall structural stiffness, the element should be evaluated for gravity support in a damaged state. No mitigation may be required, particularly in cases where cast-in-place concrete slabs exist. For cases where gravity support is inadequate, the approach may be the installation of a supplemental support in the form of new corbels placed under locations of damage. If there is no ceiling, care should be taken to minimize the visual impact of the corbel and connectors. If existing M/E/P system components conflict with installation of these supplemental supports, they should be relocated as required to make this minimum mitigation effort.

For cases where an actual coupling beam or header is being augmented, the contribution of both the existing and new portions of the now composite member should be considered. Also, since it will be very difficult to provide sufficient strength in the augmented beam, the primary goal should be to provide adequate ductility to survive the expected rotations and continue to provide gravity load support and shear transfer capability. If the new concrete work is exposed to view, consideration should be given to the nature of the forming and surface finishes desired in relation to the surrounding existing elements. For cases where a deeper beam or header exists, the new ductile member should be designed to provide all required strength and ductility.

### *Detailing Considerations*

*Connection of new corbel to existing concrete elements:* A typical detail showing the installation of a new steel corbel is shown in Figure 13.4.2-1. The steel angle provides supplemental support for the coupling slab at the wall joint where the most damage is expected to occur. For a precast plank slab condition (as shown in the detail), especially without a cast-in-place topping, consideration should be given to extending the supplemental steel supports along the full length of the link between the coupled walls. Installation of drilled anchors into the bottom of the

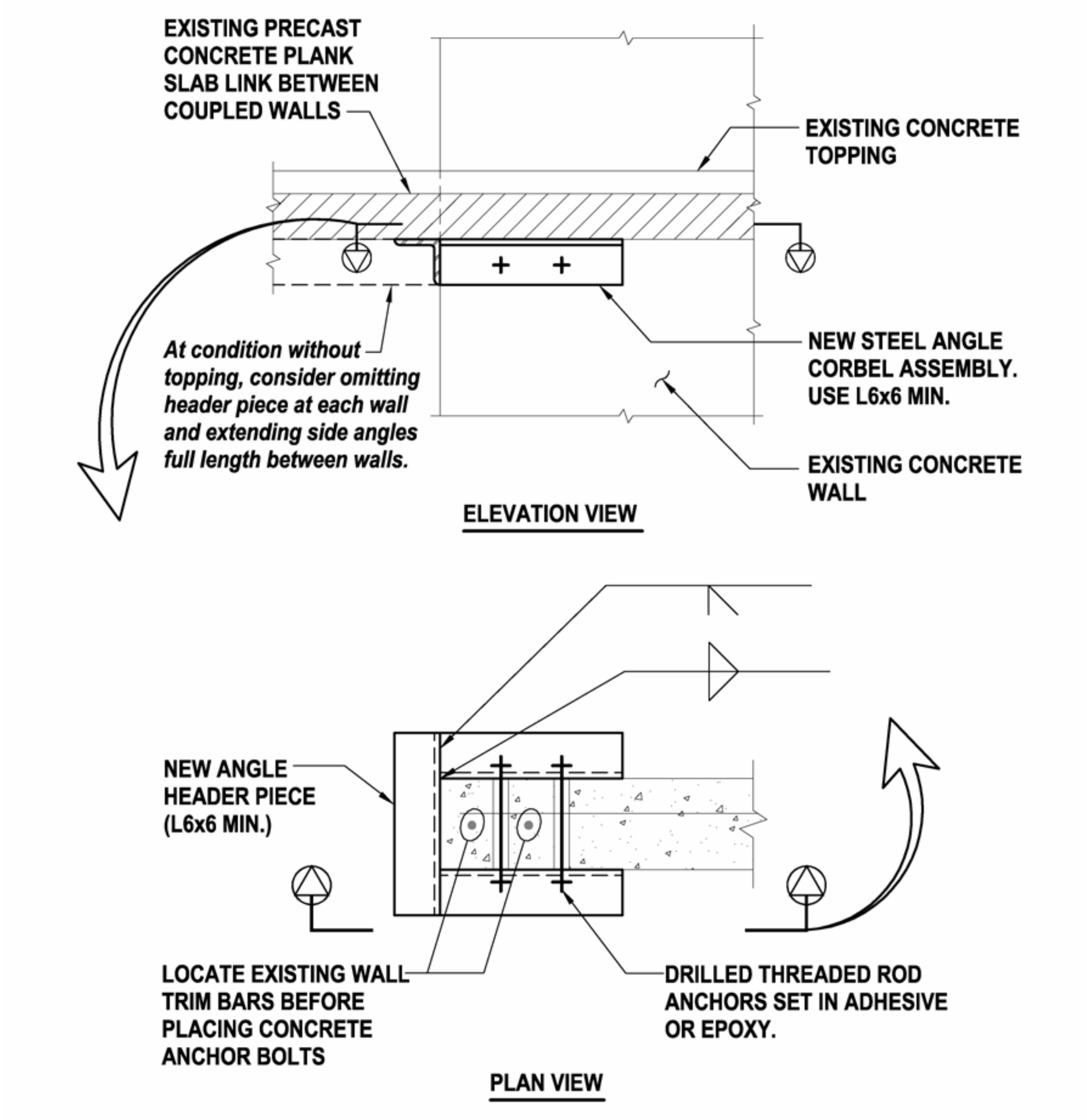


**Figure 13.4.2-1: Typical Corbel at Linking Slab**

planks may be considered. However, it may be difficult to avoid both prestressing tendons and hollow core voids. Consider use of screen-tube anchors (see Chapter 21) if voids cannot be avoided.

*Connection of new concrete to the existing concrete beam and adjacent walls:* Figure 13.4.2-2 shows an elevation and section view of a typical augmentation of a relatively shallow coupling beam or header. The surface of the existing concrete should be thoroughly cleaned and roughened to provide a good bond and interaction between the existing and new portions of the

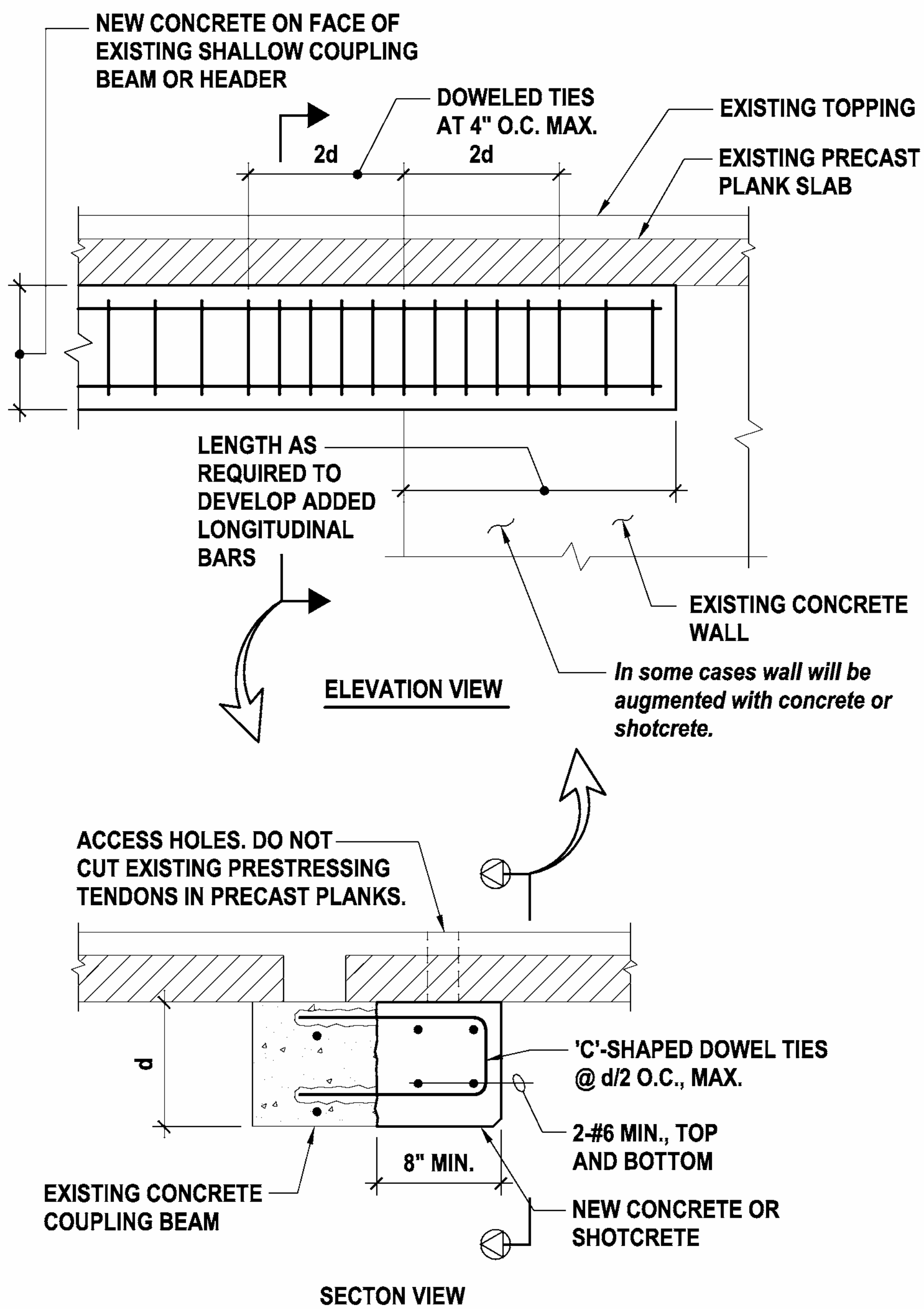


**Figure 13.4.2-2: Typical Strengthening of Shallow Coupling Beam**

composite member. The new concrete should be extended along the walls far enough to develop the longitudinal bars. If cast-in-place concrete is used, access holes will be required through the slab for placement and consolidation. Care must be taken to avoid damage to any prestressing tendons in the precast planks. Figure 13.4.2-3 shows typical details for installation of X-shaped rebar cages at deeper coupling beams or headers. The new concrete should be thick enough to allow placement of the crossing longitudinal reinforcement cages inside of the confining beam ties.

***Cost/Disruption Considerations***

Installation of steel corbel supplemental supports is an inexpensive, minimal mitigation approach. However, costs could increase substantially if relocations of M/E/P systems are required. The disruption associated with a single installation of this work is relatively local, but similar work will likely be required at all or most of the walls throughout the building. At all cases where new concrete or shotcrete is added to existing coupling beams, the cost and level of disruption will increase substantially. Refer to similar discussion in Sections 12.4.2 and 12.4.3.

***Construction Considerations***

For installation of a steel corbel, care should be taken to locate existing reinforcing steel in the wall prior to drilling or coring. Also, the pattern of prestressing tendons should be located before any drilling into precast planks is begun. For augmentation of coupling beams, refer to similar discussions at Sections 12.4.2 and 12.4.3.

***Proprietary Concerns***

The basic materials are generic.

### 13.4.3 Enhance Connection Between Slab and Walls

***Deficiencies Addressed by the Rehabilitation Technique***

Inadequate bearing for precast slab planks at wall
Inadequate shear transfer capacity from precast plank diaphragms to shear wall

***Description of the Rehabilitation Technique***

For most cases involving inadequate bearing support for precast floor planks at the walls, the typical mitigation approach will be to install steel (or concrete) ledgers secured to the wall under the slab-wall joint. This approach can also be used to increase joint shear capacity by including concrete anchors drilled up into the slab. An alternative approach to increasing joint shear capacity is to install diagonal dowels, or shear pins, through the joint. Occasionally, if adequate story height is available and gravity load issues can be addressed adequately, a new reinforced concrete topping slab could be placed over the existing diaphragm.

***Design Considerations***

For cases where the concern is focused only on providing adequate bearing, installation of a steel ledger angle anchored to the wall is a very simple and direct technique. Provision of about six inches of additional bearing will be enough to assure against loss of vertical support, so anchors drilled into the slab or precast concrete planks may be omitted. Use of a concrete ledger is

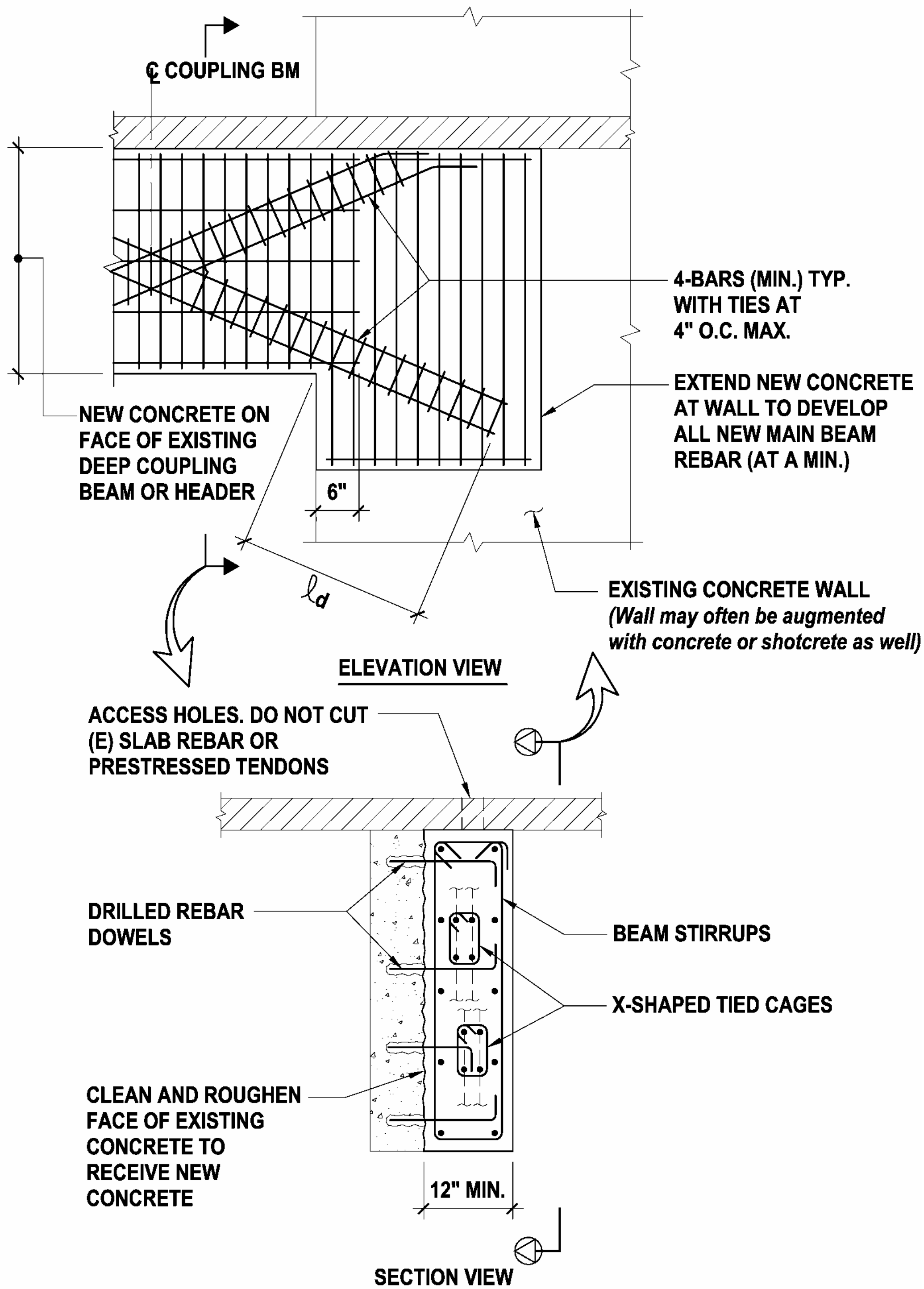


**Figure 13.4.2-3: Strengthening of Deep Coupling Beam**

certainly possible, but would seem appropriate only if concrete collectors were also being installed. However, a steel angle or channel section ledger could still be used, linking the end of a concrete collector to the wall.

For cases where diaphragm-wall shear transfer is to be augmented, either vertical slab anchors or diagonal dowels, drilled and placed from above, may be added. However, the added shear capacity that can be provided by these techniques is limited by the shear capacity of the diaphragm slab immediately adjacent to the wall. If the local diaphragm capacity is inadequate, then a new collector will be required to engage a greater extent of the diaphragm (refer to Section 12.4.3).

If a new concrete topping slab is to be placed, sufficient reinforcement probably can be included to serve as the collector. However, new diagonal dowels or perhaps a new ledger will likely be needed to insure transfer of the collected diaphragm shear demand down through the lightly reinforced slab-wall construction joints into the wall below.

Consideration should be given to the treatment of any architectural finishes and M/E/P system components mounted on the wall in the affected areas.

### *Detailing Considerations*

Typical details for installation of new steel ledgers and drilled diagonal dowels are shown in Figures 13.4.3-1 and 13.4.3-2, respectively. For these techniques, little or no prior cleaning or preparation of the existing concrete surfaces will be required. Alternatively, refer to Section 12.4.3 for discussion related to installation of a concrete ledger. Where drilled concrete anchors or drilled diagonal dowels (acting as shear pins through the slab-wall joint region) are to be installed through precast concrete planks, the prestressed tendons must be avoided. Also, consideration should be given to means of dealing with any hollow core voids in the planks that may be encountered. Screen tubes similar to those used in brick masonry anchorage details could be used, or the voids could be filled, at least locally, with grout.

### *Cost/Disruption*

Installation of steel section ledgers and drilled concrete anchors and dowels are simple and well known basic techniques that should be relatively inexpensive. There will be noise and vibration associated with the drilling, but the work is essentially “dry” and not particularly messy. However, given the nature of this type of structure, there are likely to be many walls distributed throughout the building, thus the work will be pervasive. If concrete ledgers are installed, the level of cost and disruption will be considerably higher (refer to the discussions regarding installation of concrete collectors at Section 12.4.3).

### *Construction Considerations*

Installation of the drilled concrete anchors or diagonal dowels may require some precision to avoid prestressing tendons or hollow core voids. Once the pattern of these items is defined, a steel template, perhaps the steel ledger itself, can be used. If some flexibility in the exact location of the anchors is required, consider using oversized holes and welded plate washers, and provide extra holes.

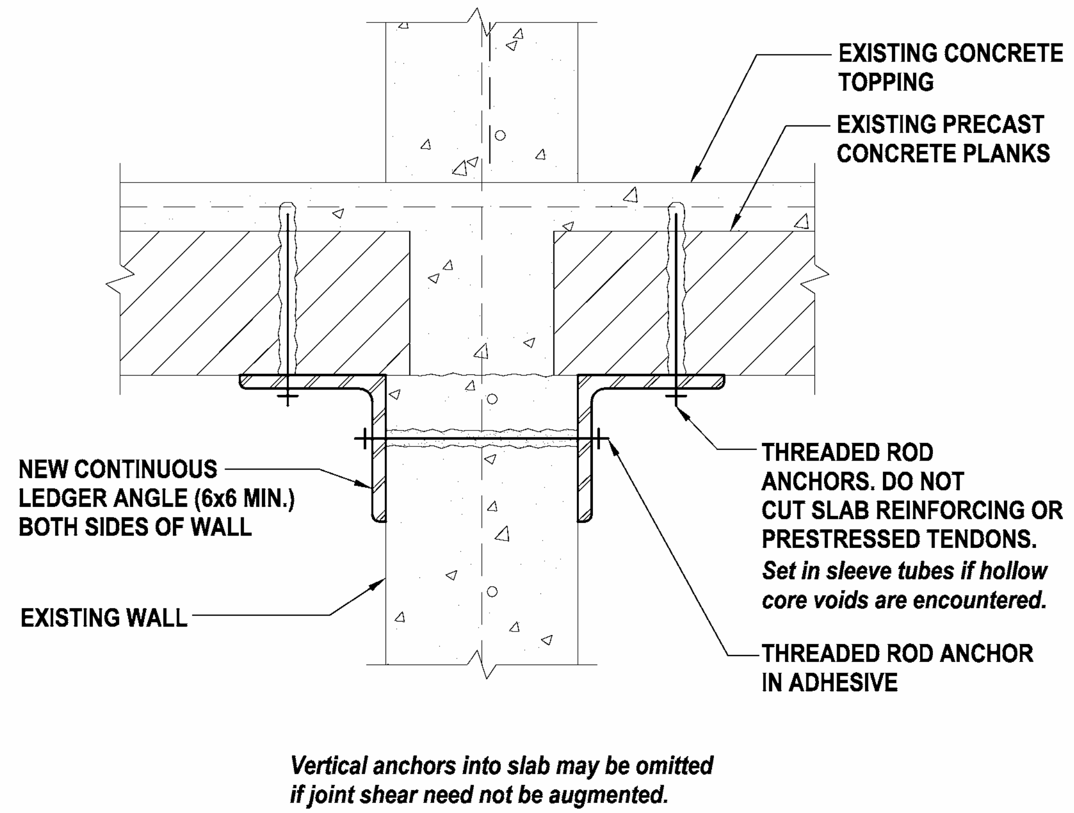


**Figure 13.4.3-1: Added Support and Shear Strength at Slab-Wall Joint**

The drilled anchors and dowels will require testing, which should be performed by persons experienced in torque and/or tension testing of diagonal dowels and overhead installations. For the drilled shear pin dowels that do not project out of the slab, additional similar dowels can be installed for testing purposes only.

***Proprietary Concerns***

The basic materials are generic.

### 13.4.4 Reduce Flexural Capacity of Shear Walls to Reduce Shear Demand

***Deficiency Addressed by the Rehabilitation Technique***

Degradation of shear-critical walls

***Description of the Rehabilitation Technique***

In buildings with many walls and high strength, many of the walls may be shear-critical and prone to strength degradation and/or possible reduction of gravity support capacity. A sudden

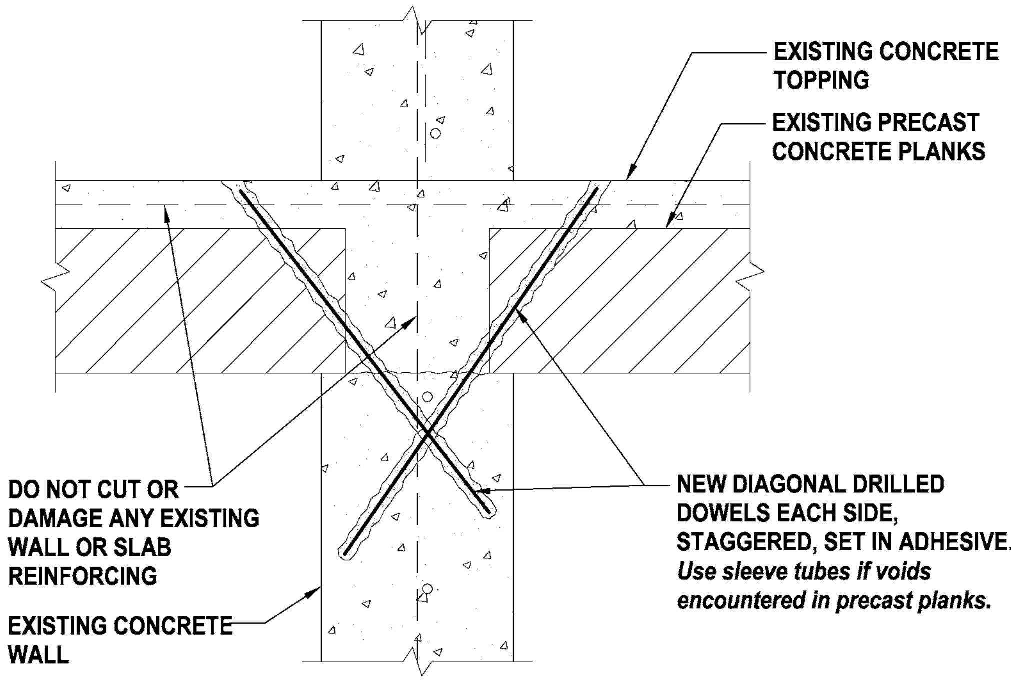


**Figure 13.4.3-2: Added Shear Capacity at Slab-Wall Joint**

loss of lateral strength and stiffness, particularly all at one floor, should be avoided. To avoid this condition, an alternative to shear enhancement is to reduce the level of possible shear demand that the wall can experience by reducing flexural capacity. This approach may be applied to one or more walls, in a system with adequate global strength, to change the expected post-yield behavior from a brittle shear failure to a more ductile flexural yielding. This approach also may be applied in other circumstances to an individual wall that cannot be strengthened for some reason, but must be protected from serious damage, even as other walls are strengthened or new walls are added in other locations within the system. This technique will generally involve cutting a number of the existing wall chord bars. However, an alternative approach that may be applied to a long wall would be to make one or more vertical cuts in the wall, creating a series of shorter panels with reduced flexural capacity.

### *Design Considerations*

Application of this technique to a particular shear wall is intended to replace a brittle shear failure mechanism with a more ductile flexural hinging mechanism. It is not recommended that this technique be applied in conditions that may result in creating a brittle tension failure mechanism. Therefore, the number and location of cut bars must be determined with careful analysis of the detail of the expected mechanism (strain compatibility, flexural hinge length, etc.). Bar cuts should be sufficiently staggered and a major reduction in flexural capacity should not be attempted with this method. The impact of this technique on architectural and M/E/P systems and components is likely to be relatively limited. Only fixtures in the immediate vicinity of the chord bar cuts or vertical wall slices are likely to be affected. However, client perception of this reductive technique may be decidedly negative.

### *Detailing Considerations*

Cutting the selected existing wall chord bars must be done with great care and precision to avoid cutting adjacent chord bars or the confining transverse reinforcement. If the wall reinforcement can be adequately mapped by using metal detector, x-ray or ground penetrating radar techniques, then the selected bars could be cut by coring through the concrete cover. However, congested chord reinforcement and/or closely spaced transverse ties may require that the concrete cover be chipped away to expose the bars before cutting with either a core drill or a torch.

In the case of reducing the overall length of a long wall with vertical cuts, the cuts can be made with a circular concrete saw. In order to extend the cuts as close to the floor slabs as possible the cuts may be made from each side of the wall, to keep the depth of the cut and the radius of the saw blade to about half the wall thickness.

### *Cost/Disruption*

This technique will be less costly than alternative methods to increase the shear capacity of a wall. The extent of the work is very localized and the impact on nonstructural components will be limited. However, there will always be noise and vibration associated with any concrete chipping, coring or sawcutting, and the latter two operations can be very wet and messy.

### *Construction Considerations*

Access must be available to the locations where chord bars are to be cut or the wall is to be sawcut. However, this will certainly be less of an imposition than any other alternative to increase the shear capacity of the wall.

Employment of this technique will most likely require special scheduling and/or sequencing considerations relative to the other strengthening work, to avoid creating a weakened structure during the course of the overall retrofit project.

### *Proprietary Concerns*

The basic materials are generic.

## 13.5 References

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# Chapter 14 - Building Type C2f: Concrete Shear Walls (Gravity Frame Systems)

## 14.1 Description of the Model Building Type

Reinforced concrete walls in a building will act as shear walls whether designed for that purpose or not.  Therefore, concrete buildings that contain any significant amount of concrete wall will fall into this category.  However, there are two distinctly different types of concrete wall buildings: those that contain an essentially complete beam/slab and column gravity system, and those that use bearing walls to support gravity load and have only incidental beam and column framing.  In this document, these building types have been separated and are designated **C2f** for the gravity frame system and **C2b** for the bearing wall. This section covers the building with gravity framing system.  Although it is typically assumed that the gravity framing is not part of the lateral force-resisting system, the framing could add stiffness to the building, particularly near the top of taller buildings.  This building type is very common and has been used in a wide variety of occupancies and in all sizes.

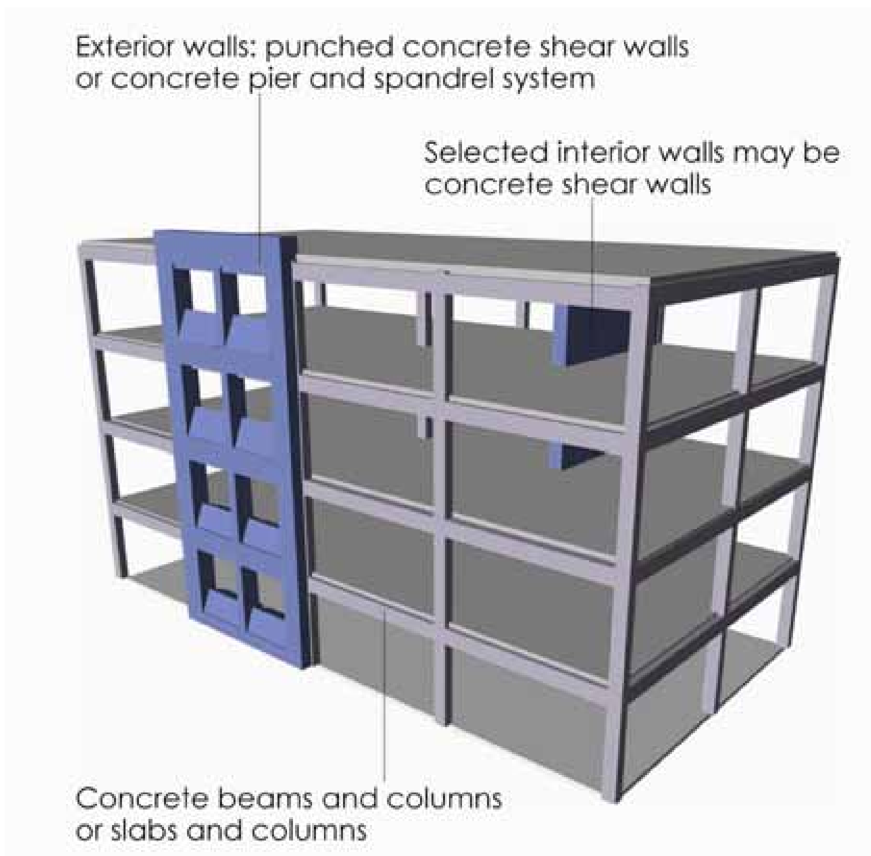



**Figure 14.1-1: Building Type C2f: Concrete Shear Walls (Gravity Frame Systems)**

### *Variations in Framing Systems*

There are wide overall variations within this building type due to the possible configuration and extent of the concrete walls, the many types of vertical framing systems used, and the lateral stiffness interaction between the two. In buildings with incidental concrete walls and a substantial beam-column gravity frame system, this building type merges with Building Type **C1**. If the building type is unclear, reference should be made to both Chapter 12 and this chapter.

Gravity frame systems in this building type include cast-in place concrete beam and slab, one-way joists, two-way waffles, and two-way or flat slabs.

In older buildings that are seismically deficient, the walls were often intended for fire protection of vertical shafts, as exterior closure walls, or as bearing walls. However, buildings built in regions of high seismicity in the 1950s, 1960s or early 1970s often were designed with a shear wall lateral force-resisting system, but they are now found deficient due to low global strength, a highly torsional plan layout or detailing that leads to premature shear failure

In buildings designed with shear walls, the walls are either strategically placed around the plan, or at the perimeter. Shear walls systems placed around the entire perimeter almost always contain windows and other perimeter openings and are often called punched shear walls. Older buildings will have concrete walls somewhat arbitrarily placed in plan.

### *Floor and Roof Diaphragms*

The floor and roof diaphragms in this building type are essentially the same as the bearing wall system and are almost always cast-in-place concrete. The diaphragms are stiff and strong in shear because the horizontal slab portion of the gravity system is either thick or frequently braced with joists. However, one way joist systems could be inadequate in shear in the direction parallel to the joists. Collectors are seldom in place and transfer of load from diaphragm to shear wall must be carefully considered.

### *Foundations*

There is no typical foundation for this building category. Foundations could be found of every type depending on the height of the building, the span of the gravity system and the site soil.

## 14.2 Seismic Response Characteristics

Shear wall buildings, unless configured with only incidental or minimal walls, will typically be quite stiff. Elastic and early post-elastic response will therefore be characterized with lower-than average drifts and higher-than-average floor accelerations. Damage in this range of response should be minimal.

Overall post-elastic response is highly dependent on the specific characteristics of the shear walls and the gravity frame components.

### *Shear Wall Behavior*

The walls must first be evaluated to determine if they contribute sufficient strength or stiffness to be considered significant. Often walls around vertical shafts are thin and lightly reinforced and will have little effect on the overall building response. Although retrofit techniques are similar such a building may be classified as Type **C1.**

When subjected to ever increasing lateral load, individual shear walls or piers will first often force yielding in spandrels or other horizontal components restricting their drift, and eventually either rock on their foundations, suffer shear cracking and yielding, or form a flexural hinge near the base. Shear and flexural behavior is quite different, and estimates of the controlling action are affected by the distribution of lateral loads over the height of the structure.

Yielding of spandrels or other coupling beams can cause a significant loss of stiffness in the structure. Flexural yielding will tend to maintain the strength of the system, but shear yielding, unless well detailed, will degrade the strength of the coupling component and the individual shear wall or pier will begin to act as a cantilever from its base.

Rocking is often beneficial, limiting the response of the superstructure. However, the amplified drift in the superstructure from rocking must be considered. In addition, if varying wall lengths or different foundation conditions lead to isolated or sequencing rocking, the transfer of load from rocking walls must be investigated. In buildings with basements, the couple created from horizontal restraint at the ground floor diaphragm and the basement floor/foundation (often termed the “backstay” effect) may be stiffer and stronger than the rocking restraint at the foundation and should be considered in those configurations.

Shear cracking and yielding of the wall itself is generally considered undesirable, because the strength and stiffness will quickly degrade, increasing drifts in general, as well as potentially creating a soft story or torsional response. However, in accordance with FEMA 356 (FEMA 2000), shear yielding walls or systems can be shown to be adequate for small target displacements.

Flexural hinging is considered ductile in FEMA 356 and will degrade the strength of the wall only for larger drifts. Similar to rocking, the global effect of the loss of stiffness of a hinging wall must be investigated.

### *Gravity Frame Behavior*

The lateral strength and stiffness of gravity frames will vary considerably among buildings in this type. In some configurations of this building type, the gravity frame will not significantly participate in the response. However, it is not uncommon in these buildings for a stiff and brittle gravity system to dominate both response and the extent of damage. For example, if concrete spandrels or sills on the perimeter of the building restrain the gravity columns (the “short column”), the column must take the full story drift over a short height, potentially causing shear failure and loss of gravity load capacity. Other gravity systems, such as flat slab or heavy beam and column systems, can also be sensitive to drifts, particularly to the increased drifts near the top of buildings with walls of a height-to-width ratio over 3. The frame action of the gravity system of these buildings may be beneficial or could form a deficiency, but in any case the interaction with the shear walls should be considered.

## 14.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 14.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Selected deficiencies are further discussed below by category.

### *Global Strength*

Older buildings placed in this category may have minimal shear walls and seismic displacements will likely put excessive demand on the walls regardless of the shear or flexural behavior.  The most common method of mitigating this deficiency is to add more shear walls, although for smaller buildings steel braced frames have been used.  The nonlinear behavior of steel braced frames must be carefully studied for compatibility with the other shear walls and the gravity frame.  Additional bending resistance of existing walls can be obtained by enhancing existing chords, although such walls should not be strengthened to become shear critical.  If the majority of walls are shear-critical and strength degradation is the primary concern, shear strength can be added to the existing walls with concrete or FRP overlays.

### *Global Stiffness*

The most important issue in a building of this type that exhibits large interstory drifts is the ability of the gravity system to accept the drifts while sustaining their loads.  Excess drifts could be caused by inadequate length of wall, by rocking at the foundations, or, at the upper stories, by a deformed shape characterized by bending deformations.  In most buildings, strength and stiffness are closely related, and inadequate stiffness is mitigated by adding new elements or stiffening existing ones, which generally will also increase strength.  Damping can also be added to reduce drifts but care must be taken to achieve the desired damping with small displacement expected in a shear wall building.

### *Configuration*

The two most common configuration deficiencies in this building type are 1) severe torsion caused by eccentrically placed shafts or towers, and 2) shear walls or full stories that are weak or soft from openings in the walls or from discontinuous walls that may not run through the ground floor or basement floor.  Completely discontinuous walls also create a load transfer deficiency for both overturning and shear.

### *Load Path*

Common load path deficiencies include discontinuous shear walls, as discussed above, and collectors for the shear walls.  Collectors can be added with steel members or new reinforcing and concrete.

### *Component Detailing*

Shear walls in most older buildings meet none of the current detailing requirements covering minimum shear reinforcement, for confinement of chords, and the walls are commonly shear critical.  FEMA 356 allows these deficiencies at controlled displacement levels.  A common

| Table 14.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C2f Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane wall shear strength | Concrete/masonry shear wall [12.4.2]<br>Steel braced frame [12.4.1]<br>Steel plate shear wall | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [13.4.1]<br>Steel wall overlay | | Seismic isolation [24.3]<br>Reduce flexural capacity [13.4.4] | |
| | Insufficient flexural capacity | Concrete/masonry shear wall [12.4.2]<br>Steel braced frame [12.4.1] | Add or enhance chords [12.4.3] | | | |
| | Inadequate capacity of coupling beams | Concrete/masonry shear wall [12.4.2]<br>Steel Braced frame [12.4.1] | Strengthen beams<br>Improve ductility of beams [13.4.2] | | | Remove beams |
| Global Stiffness | Excess drift (normally near the top of the building) | Concrete/masonry shear wall [12.4.2]<br>Steel braced frame [12.4.1] | Fiber composite column wrap [12.4.4]<br>Concrete/steel column jacket [12.4.5]<br>Provide detailing of all other elements to accept drifts<br>Thicken walls | | Supplemental damping [24.4] | |
| Configuration | Discontinuous walls | Concrete/masonry shear wall [12.4.2] | Enhance existing column for overturning loads | Improve connection to diaphragm [13.4.3] | | Remove wall |
| | Soft story or weak story | Add strength or stiffness in story to match balance of floors | | | | |

| Table 14.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C2f Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Re-entrant corner | Add floor area to minimize effect of corner | | Provide chords in diaphragm [12.4.3] | | |
| | Torsional layout | Add balancing walls, braced frames, or moment frames | | | | |
| Load Path | Inadequate collector | Add steel collector [12.4.3]<br>Add concrete collector [12.4.3] | Strengthen existing beam or slab<br>Enhance splices or connections of existing beams | | | |
| | Discontinuous Walls | Provide new wall support components to resist the maximum expected overturning moment | Strengthen the existing support columns for the maximum expected overturning moment<br>Provide elements to distribute the shear into the diaphragm at the level of discontinuity | | | |
| Component Detailing | Wall inadequate for out-of-plane bending | Add strongbacks | Concrete wall overlay [21.4.8] | | | |
| | Wall shear critical | | Concrete wall overlay [21.4.8]<br>Fiber composite wall overlay [13.4.1] | | Reduce flexural capacity of wall [13.4.4] | |
| | Inadequate displacement capacity of gravity columns | | Enhance ductility (see also global stiffness) | | | |

**Table 14.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C2f Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Diaphragms | Inadequate in-plane shear capacity | | Concrete topping slab overlay<br>Fiber composite overlays [22.2.5] | | | |
| | Inadequate chord capacity | New concrete or steel chord member [12.4.3] | | | | |
| | Excessive stresses at openings and irregularities | Add chords [12.4.3] | | | | Infill openings [22.2.4] |
| Foundations | See Chapter 23 | | | | | |

[ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques.

improvement to these walls is to enhance shear strength to be equal or greater than the maximum that can be developed in the wall, based on bending strength.

### *Diaphragm Deficiencies*

The most common diaphragm deficiency in this building type is a lack of adequate collectors. The addition of effective collectors in an existing diaphragm is difficult and disruptive. Existing strength to deliver loads to the shear walls should be studied carefully before adding new collectors.

### *Foundation Deficiencies*

See Chapter 23.

## 14.4 Detailed Description of Techniques Primarily Associated with This Building Type

The most relevant recommendations listed in Table 14.3-1 are similar to techniques also associated with other concrete building types such as **C1** and **C2b** or general techniques applied to concrete diaphragms. Details concerning these techniques can be found in other chapters.

## 14.5 References

FEMA, 2000, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356, Federal Emergency Management Agency, Washington, D.C.

# Chapter 15 - Building Types C3/C3A: Concrete Frames with Infill Masonry Shear Walls

## 15.1 Description of the Model Building Type

Building Type **C3** is normally an older building that consists of an essentially complete gravity frame assembly of concrete columns and floor systems. The floors consist of a variety of concrete systems including flat plates, two-way slabs, and beam and slab. Exterior walls, and possibly some interior walls, are constructed of unreinforced masonry, tightly infilling the space between columns horizontally and between floor structural elements vertically, such that the infill interacts with the frame to form a lateral force-resisting element. Windows and doors may be present in the infill walls. The buildings intended to fall into this category often feature exposed clay brick masonry on the exterior. Figure 15.1-1 shows an example of this building type.

It is important to note that similar buildings with exterior masonry infill sills below windows that extend column to column do not behave with strut action and should be classified as Building Type **C1** or **C2f**. In fact, such infill sills often create “short columns” that must absorb the entire story drift over the unrestrained height of the window, which can often be an extreme deficiency in poorly reinforced columns. The **C3A** building type is similar but has floors and roof that act as flexible diaphragms such as wood, or untopped metal deck.

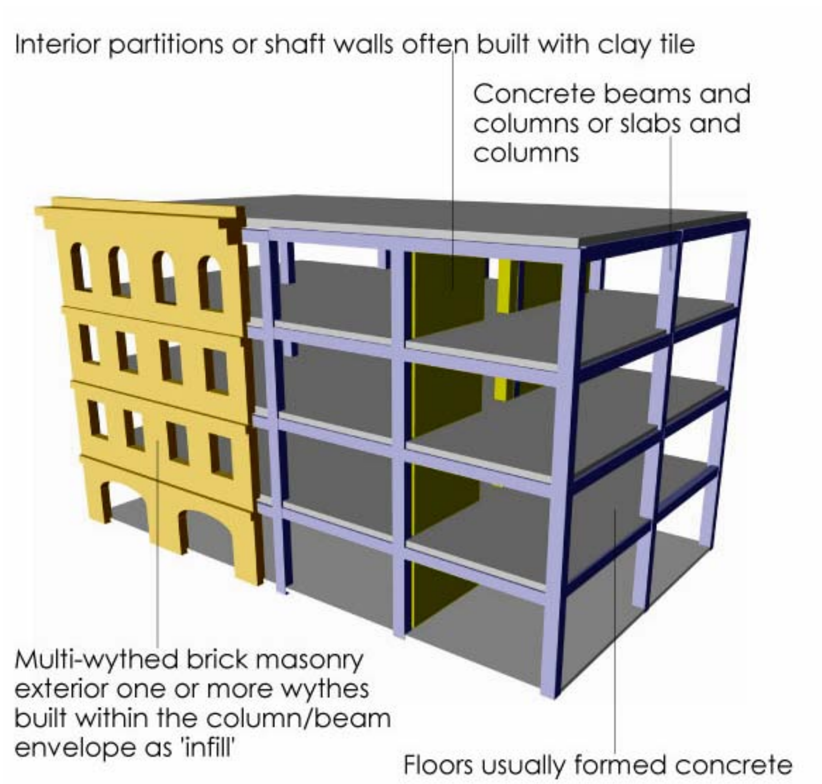



**Figure 15.1-1: Building Type C3: Concrete Frames with Infill Masonry Shear Walls**

### *Variations Within the Building Type*

The building type was identified primarily to capture the issues of interaction between unreinforced masonry and concrete gravity framing. The archetypical building has solid clay brick at the exterior with one wythe of brick running continuously past the plane of the column and beam and two or more wythes infilled within the plane of the column and beam. The exterior wythe of clay brick forms the finish of the building although patterns of terra cotta, stone, or precast concrete may be embedded into the brick. However, there can be many variations to this pattern depending on the number and arrangement of finished planes on the exterior of the building. For example, the full width of the infill wall may be located with the plane of the column and beam with a pilaster built out and around the column and a horizontal band of brick or other material covering the beam; the beam may also be slightly offset from the centerline of the column to accommodate the pattern of exterior finishes.

Hollow clay tile masonry may also be used as an exterior infill material. Although this material often has a high compression strength, the net section of material available to form the compression strut within the frame will normally contribute a lateral strength of only a small percentage of the building weight. The material being brittle and the wall being highly voided, these walls may also lose complete compressive strength quite suddenly. Therefore, walls of hollow clay tile infill will probably not contribute a significant portion of required lateral resistance except in areas of low seismicity and/or when walls are arranged as infill on both the exterior and interior of the building.

More recent buildings may have unreinforced concrete block masonry configured as an exterior infill wall, with a variety of finish materials attached to the outside face of the concrete block. Similar to hollow clay tile walls, these walls may exhibit moderate to low compressive strength and brittle behavior that marginalizes their usefulness as lateral elements. In addition, hollow concrete block exterior walls often will not be installed tight to the surrounding framing, eliminating infill compression strut behavior.

### *Floor and Roof Diaphragms*

Floors are often flat plates or two-way slabs. Beam and slab or beam and joist systems will also be found in this building type. Typically, these slabs provide adequate diaphragms.

Building Type **C3A** will have heavy timber floors with one or more layers of sheathing forming a diaphragm. The flexibility of such diaphragms will often form a seismic deficiency because, assuming no interior shear elements, the large drift at the diaphragm mid-span will damage perpendicular walls and gravity framing. Specific strengthening techniques for this building type are not covered here. For generalized strengthening of diaphragms, see Chapter 22.

### *Foundations*

There is no typical foundation for this building type. Foundations can be found of every type depending on the height of the building, the span of the gravity system and the site soil. The exterior walls are exceptionally heavy and typically will be supported by a continuous concrete footing or often a continuous concrete wall forming a basement space below.

## 15.2 Seismic Response Characteristics

Both in terms of stiffness and strength, the exterior infill walls will form the effective lateral system for this building type. The effectiveness of the system depends on the size and extent of openings and articulation of the plane of the wall. With solid or nearly solid infill panels, strut action will be stiff and strong. As openings in panels increase in size, struts or combinations of struts cannot effectively form around the opening, and the concrete columns and beams may begin to work as a moment frame, with "fixity" at the beam-column joint provided by the masonry. For low and moderate intensity shaking, the exterior walls may provide adequate strength to satisfy the specified performance objective. As the shaking demand increases, the masonry will tend to crack and spall, losing stiffness and potentially creating a falling hazard. The complete concrete gravity system, characteristic of this building type, will provide additional stability, but may quickly degrade in strength and stiffness due to inadequate column reinforcing. However, in wall configurations with large height-to-width ratios, end or corner columns could experience large compression or tension loads from overturning, leading to rapid degradation of column lateral and gravity capacity.

This building type is often characterized by a commercial store-front first floor with little or no infill at that level on one or more faces of the building. This condition can cause a soft story condition or a severe torsional response if open on one or two sides only. Such conditions can lead to concentration of seismic deformation at the open level, degradation of the columns, and possible P-delta failure.

## 15.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 15.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Deficiencies related to specifically to concrete moment frames and to masonry shear walls are shown in Table 12.3-1 and Table 18.3-1, respectively. Selected deficiencies are further discussed below by category.

### *Global Strength*

The overall strength provided by the exterior walls may be insufficient to prevent serious degradation and resulting amplified displacements that can lead to irreparable damage or even instability. The strength may be limited by inadequate number of panels of infill, excessive openings, or masonry weak in compressive strength. The standard approach to such deficiencies will be to add new, relatively stiff lateral force-resisting elements such as concrete shear walls or steel braced frames often located on the interior between existing columns. The infill itself could also be strengthened by adding a layer of reinforced concrete on the interior surface, although whether such a system is acting as infill or as a shear wall must be checked by analysis. Unless all infill has been backed by a support system, the damage state of the infill wall must be estimated for the expected drifts of the combined system to determine if the desired performance has been achieved.

### *Global Stiffness*

The most important issue in a building of this type that exhibits large interstory drifts is the ability of the gravity system to accept the drifts while sustaining their loads. In most buildings,

**Table 15.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C3/C3A Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Inadequate length of exterior wall | Interior concrete walls [12.4.2]<br>Interior steel braced frames [12.4.1] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6] | | | |
| | Excessive sized openings in infill panels | Interior concrete walls [12.4.2]<br>Interior steel braced frames [12.4.1] | Infill selected openings [21.4.7]<br>Fiber composite wall overlay [21.4.6] | | | |
| | Inadequate columns for overturning forces | | Add confinement<br>Add tensile capacity on outside surface of column. | | | |
| | Weak or deteriorated masonry | Interior concrete walls [12.4.2]<br>Interior steel braced frames [12.4.1] | Point outside and/or inside wythes of masonry<br>Inject wall with cementitious grout<br>Fiber composite overlay [21.4.6] | | | |
| Global Stiffness | See Global Strength | | | | | |
| Configuration | Soft or weak story | Interior concrete walls [12.4.2]<br>Interior steel braced frames [12.4.1] | | | | |
| | Torsion from one or more solid walls | Balance with Interior concrete walls<br>Balance interior steel braced frames | | | | Remove selected infill panels on solid walls |

**Table 15.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C3/C3A Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Irregular Plan Shape | Balance with Interior concrete walls<br>Balance with Interior steel braced frames | | | | |
| Load Path | Out-of-plane failure of infill due to loss of anchorage or slenderness of infill | | Provide surface wall supports [13.4.3]<br>Shotcrete overlays [21.4.5]<br>Fiber composite overlay [21.4.6] | | | Remove infill |
| | Inadequate connection of finish wythe to backing | | Add connections | | | |
| | Inadequate collectors | Add steel collector or concrete collector [12.4.3] | | | | |
| Component Detailing | Inadequate columns splice for tension due to uplift force induced by infill | | | Add splice plates<br>Provide splice through added reinforced concrete encasement | | |
| | Inadequate beam column connection to resist compression thrust | | | Strengthen connection in shear with steel or fiber composite | | |

| Table 15.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for C3/C3A Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing (continued) | Weak or incompletely filled joint between masonry and surrounding steel components | | | Create appropriate clean void and repack with masonry and/or mortar.<br>Inject voids with cementitious grout or epoxy | | |
| Diaphragms | See Chapter 22 | | | | | |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

strength and stiffness are closely related, and inadequate stiffness is mitigated by adding new elements or stiffening existing ones, which generally will also increase strength. Damping can also be added to reduce drifts, but care must be taken to achieve the desired damping with the small displacement expected in a shear wall building.

### *Configuration*

Two configurational deficiencies are common in this building type. The first is a soft and weak story at the street level created by commercial occupancies with exterior bays with little or no infill. This deficiency can be corrected by adding selected bays of infill or by adding shear walls or braced frames at this level. The second common issue is a plan torsional irregularity created by solid masonry walls on property lines coupled with walls with many openings on street fronts. If shown by analysis to be necessary, torsional response can be minimized by stiffening the more flexible side of the building with more infill or by the addition of lateral elements. In rare cases, the solid walls can be balanced with the open side by selected removal of panels or disengagement of the infill strut action.

### *Load Path*

The primary load path issue with this building type is to assure that the mass of the exterior walls will not become disengaged from the frame which will both prevent infill strut action as well as to create a significant falling hazard on the street below.

In-plane, the articulation of the exterior walls may result in offsets of the wall plane between floors. The presence of a complete load path and maintenance of confinement for strut formation must be reviewed in such instances.

If new lateral load-resisting elements are added, existing slab and/or beam construction may need to be strengthened to provide adequate collectors.

### *Component Detailing*

In order to qualify as an infill lateral force-resisting element, the infill must be installed tight to the surrounding concrete elements. Loose or incomplete infill can be mitigated with local patching of the masonry or by injection of cemetitious or epoxy grout.

As previously noted, the infill adjacent to columns must be of sufficient stiffness to provide a floor to floor diagonal strut. With narrow piers surrounding columns, the jamb and header masonry may restrain the column such that the entire story drift must be absorbed in the central length of the column, often creating dangerous shear failures.

The detailing of the concrete frame forming the confinement for the masonry is important to achieve infill strut behavior. The connection of beam to column must be capable of resisting the strut compression forces from the masonry. The shear capacity of the beam-to-column connection is often critical. Strengthening of these connections may require removal of considerable masonry to obtain adequate access. Some of the techniques developed to strengthen concrete moment frames may be applicable. In addition, splices in vertical reinforcing of column splices may be inadequate to transfer the tensile overturning forces created by strut action. These areas can be confined to reduce the required splice length or augmented with additional tensile strength from the surface.

### *Diaphragm Deficiencies*

Concrete slab diaphragms often will be adequate. The connection of slab to exterior wall should be reviewed.

See Chapter 21 on URM construction for discussion of wood diaphragms in this type of building.

### *Foundation Deficiencies*

No systematic deficiency in foundations should be expected solely due to the characteristics of this building type.

## 15.4 Detailed Description of Techniques Primarily Associated with This Building Type

Most significant recommendations listed in Table 15.3-1 are similar to techniques more commonly associated with other building types such as various concrete buildings (**C1**, **C2b**, or **C2f**), unreinforced masonry bearing wall buildings (**URM**), or general techniques applied to concrete diaphragms. Details concerning these techniques can be found in other chapters.

# Chapter 16 - Building Type PC1: Tilt-up Concrete Shear Walls

## 16.1 Description of the Model Building Type

Building Type **PC1** is constructed with concrete walls, cast on site and tilted up to form the exterior of the building. **PC1** buildings are used for many occupancy types including warehouse, light industrial, wholesale and retail stores, and office. The majority of these buildings are one story; however, tilt-up buildings of up to three and four stories are common, and a limited number with more stories exist. For many years, tilt-up buildings have been primarily large box-type buildings with the tilt-up walls at the building perimeter; this is by far the largest group of **PC1** buildings in the U.S. inventory. In recent years, tilt-up construction has expanded to more varied uses and building configurations. Figure 16.1-1 illustrates one example of a **PC1** building.

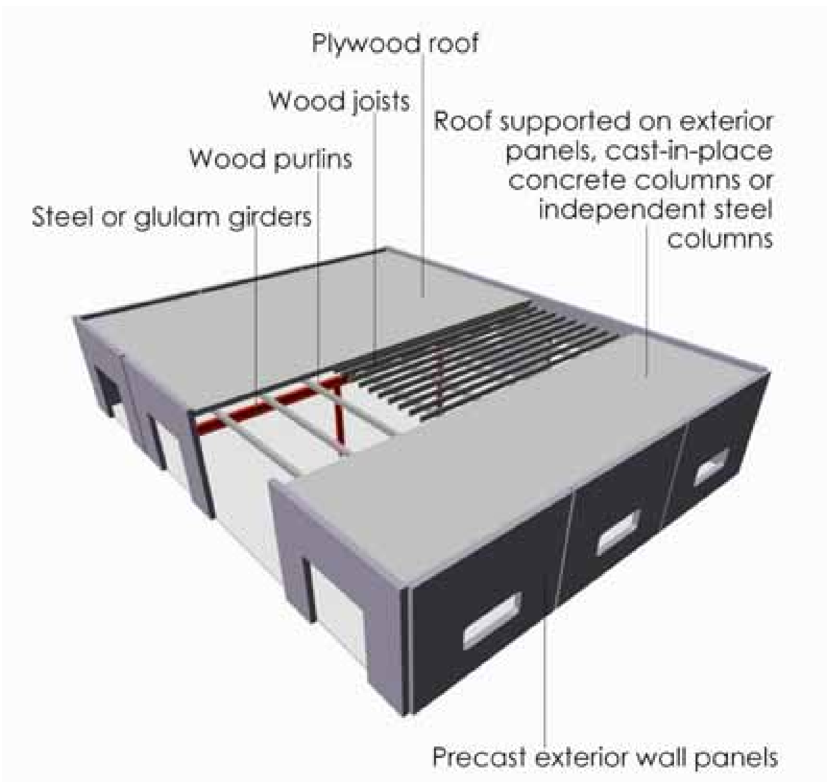



**Figure 16.1-1: Building Type PC1: Tilt-Up Concrete Shear Walls**

Key to the **PC1** building type as addressed by this chapter is the combination of a flexible roof diaphragm and rigid walls. Lateral forces in **PC1** buildings are resisted by flexible wood sheathed or steel deck roof diaphragms, wood, composite steel deck, or precast floor diaphragms, and tilt-up concrete shear walls. In some local areas, walls are concrete panels or T-beams cast off-site, rather than on-site; **PC1** also applies to this variation.

One-story **PC1** buildings with flexible roof diaphragms are the primary focus of this chapter; however, discussion of wall-to-flexible diaphragm anchorage is equally applicable to the roof of multistory **PC1** buildings. For other rehabilitation issues in multistory buildings, refer to the **C1** Building Type. For variations with rigid diaphragms at floors and roof, see Building Type **PC2**.

*Guidelines for Seismic Evaluation and Rehabilitation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures* or *SEAONC Guidelines* (SEAONC, 2001) provides a substantial collection of information on West Coast **PC1** building configurations, experience with earthquake performance, rehabilitation priorities, and techniques for rehabilitation. This chapter highlights the major rehabilitation considerations from the *SEAONC Guidelines* document, and provides suggested adaptations for construction variations. Provisions addressing rehabilitation can also be found in *International Existing Building Code (IEBC) Appendix Chapter A2* (ICC, 2003b), *City of Los Angeles Building Code Chapter 91* (City of Los Angeles, 2002) and *Guidelines for Seismic Retrofit of Existing Buildings* (GSREB) (ICBO, 2001). These provisions focus on wall anchors, diaphragm cross-ties, and collectors, with the goal of hazard reduction. This chapter and the *SEAONC Guidelines* address a broader range of rehabilitation issues and techniques.

The *Tilt-up Construction and Engineering Manual* (Tilt-Up Concrete Association, 2004), now in its sixth edition, serves as a primary resource for design and detailing practice for new tilt-up construction.

### *Walls*

Tilt-up exterior walls are the primary vertical elements in the lateral force-resisting system. Tilt-up buildings with large plan areas may have interior tilt-up walls or braced steel frames providing additional lateral resistance. Large-scale construction of tilt-up buildings began in the 1950s and 1960s with primarily solid wall panels for warehouse and light-industrial use. As tilt-up construction expanded to commercial use in the 1970s and 1980s, the wall panels changed to include large window and door openings and multistory construction. The publications *Recommended Tilt-up Wall Design* (SEAOSC, 1979) and *Test Report on Slender Walls* (ACI-SEASC, 1982) document the change from use of code-prescribed height to thickness (*h/t*) limits of 25 for bearing walls and 36 for other walls to use of much higher *h/t* ratios, in combination with rational analysis of slenderness effects. Other code changes of interest have occurred for wall pier reinforcing requirements and wall panel connection to the foundation or slab-on-grade.

Most tilt-up panels are a single piece from the foundation to the top of the building; however, some tilt-up systems use separate wall panels at upper stories, or lintel (spandrel) panels that are supported on other tilt-up wall panels. Welded connections of these panels can be damaged when they restrict panel movement under earthquake load. Prior to start of rehabilitation design, it is important to identify the tilt-up panel joint and support locations and connection condition. Surface treatments (such as exposed aggregate) and reveal joints are commonly used to visually enhance tilt-up wall panels. These treatments effect the location and dimensions of critical sections for design.

Connections between adjacent tilt-up panels are often relied on to provide continuity for diaphragm chords and collectors. Connection types have varied over the years. Early tilt-up walls were often joined by cast-in-place pilasters. Later, welded connections between cast-in embedments or between horizontal reinforcing steel were commonly used. Both of these connection systems experience some problems with fractures at welds due to panel shrinkage and temperature movement. As a result, the codes placed carbon equivalent requirements on rebar to be welded, and detailing to minimize restraint of panel movement was pursued. In *Recommended Tilt-up Wall Design* (SEAOSC, 1979), minimal interconnection between the panels and special detailing of chord and collector connections are suggested to allow movement. Detailing suggestions included 1) breaking the bond between the chord reinforcing and concrete for one fourth the panel width in from each panel end and 2) use of steel angle chord/collector members connected in the center portion of the panel, but slotted near the panel ends. The panel connection detailing should be considered in evaluating the behavior of the chords and collectors and in the distribution of shear to the tilt-up panels.

### *Gravity Load Support at the Building Perimeter*

It is most common for roof and floor framing members to be supported on the tilt-up panels at the building perimeter; however, some contractors have found it advantageous to provide steel gravity columns at the inside face of the tilt-up walls. This separation of the gravity and lateral systems makes construction tolerances for items embedded in the tilt-up panels and construction sequence less critical. The use of columns requires some modification of wall anchorage details at girders, but has little or no effect on the typical wall to diaphragm attachments, or fundamental building behavior.

### *Floor and Roof Diaphragms*

The **PC1** roof system will generally be of either wood or steel construction. In the western states (primarily California, Oregon, Washington), roof systems are almost exclusively wood structural panel sheathed. Subpurlins (joists), purlins, and girders are most often wood; however, open web trusses with wood chords or nailers are also used. Wood girders are most often supported on steel columns at the building interior. Sheathing fastening and therefore unit shear capacity in wood structural panel diaphragms generally varies by nailing zone or area. In older West Coast **PC1** buildings, roofs can be lumber sheathed, and roof framing can include bow-string trusses.

Outside of the western states, roof systems are almost exclusively sheathed with steel decking with rigid foam or nonstructural concrete insulation. The roof framing system is most commonly of steel open web trusses (bar joists) used in combination with truss girders or hot-rolled steel beams.

### *Interior Additions*

Mezzanines and interior second stories constructed within tilt-up buildings are common. The interior addition may be seismically separated and braced independently of the building shell (exterior walls and roof), or may be attached to and supported by the shell. Attachment to the building shell raises two potential issues for the tilt-up: 1) whether the seismic load to the tilt-up system is significantly increased beyond that considered in initial design, and 2) whether the interior addition restrains seismic deflections of the building and creates an unintended load path. Both of these issues should be addressed in evaluation and rehabilitation.

### *Foundations*

Tilt-up buildings are often constructed on continuous or isolated spread footings; however, drilled pier and grade beam foundations are common in some regions. Often tilt-up wall panels are set on top of shims and grout pads on top of the foundation. Where continuous foundations are provided, grout may then be placed under the full length of the panel to provide continuous bearing. Often, no direct connection is provided between the wall panel and foundation. While not common practice in older tilt-up construction, in many buildings constructed in the 1980s and later, wall panel connection to the slab-on-grade were provided, allowing transfer of horizontal seismic forces to the slab. The most common connection uses rebar dowels, cast into the wall panel and a slab closure strip. Other approaches include welded or bolted connections between cast-in connection plates or threaded inserts. See Section 16.4.5 for further discussion.

Due to the stiffness of the wall system, tilt-up buildings are best located on sites with very stable soils; however, they are often relegated to poor soil sites. Where tilt-up buildings are located on sites with soils subject to expansion, consolidation, or liquefaction, the effects of any damage to the foundation and wall connections due to soil movement should be considered in building evaluation.

## 16.2 Seismic Response Characteristics

The seismic response of the classic large box-type one-story **PC1** building is characterized by rigid wall and flexible diaphragm behavior. In this type of building, the concrete walls will have a very short period, while the diaphragm has long-period behavior. Amplification of seismic forces near the center of wood sheathed diaphragms loaded in the transverse direction can be significant. This creates high demand on the diaphragm and out-of-plane wall anchorage near the center of the diaphragm, and it can also generate very high shear demand at the diaphragm connection to the end walls. This behavior has been replicated in instrumented buildings and in laboratory testing (Fonseca, Wood and Hawkins, 1996). Roof diaphragm amplification in the longitudinal direction can be lower, depending on the diaphragm aspect ratio; however, significant overstrength can result in large anchorage forces in this direction also.

Observation of the earthquake performance of these buildings has identified as important 1) understanding the magnitude of the wall anchorage force and 2) detailing to eliminate weak links in the wall anchorage connection. With each subsequent earthquake since 1971, building code requirements have been revised to reflect changing understanding of the magnitude of forces generated in wall anchors and requirements for proper detailing; simply providing positive connections from the walls to the diaphragm has not resulted in adequate performance. A variety of weaknesses in the connections have kept anchorages from developing adequate capacity. Included are cross-grain tension of wood ledgers, rotation of non-symmetric connectors, low-cycle fatigue of straps that buckle under compression loads, net section fracture of straps with punches bolt holes, etc. The current code wall anchorage force and detailing requirements reflect a history of knowledge gained for wood structural panel diaphragms in large box-type buildings.

Recent construction of **PC1** buildings has moved away from this classic large box-type building with wood structural diaphragms. In most regions of the United States, **PC1** buildings are now constructed with steel deck diaphragms. Many **PC1** box-type buildings are being constructed with very tall walls, in which out-of-plane behavior could potentially modify building seismic response. Tilt-up construction has expanded to include a wide variety of building uses with different building characteristics, including smaller and less regular plans, less regular diaphragm configurations, and some with interior as well as exterior tilt-up walls. The seismic response characteristics of these building configurations are not known. Their behavior is of much interest, however, because current building code provisions may not adequately address these variations and materials. The Tilt-Up Concrete Association has just initiated the TCA Seismic Performance Initiative, with the intent of identifying and developing strategic plans to resolve issues of building performance and code requirements for design and detailing. Initial work will focus on design models developed from instrumented building behavior (Freeman, Searer and Gilmartin, 2002). Research on tilt-up building performance is also in progress at Canterbury University, New Zealand.

## 16.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

**PC1** buildings with wood sheathed roof diaphragms have experienced structural damage and partial building collapse in a number of California earthquakes, as well as the 1964 Alaska earthquake. Partial collapse is almost exclusively associated with inadequate connection of the walls to the flexible roof diaphragm for out-of-plane loading. A variety of other types of damage to the wall panels, connections and roof diaphragms have been observed, including some interior diaphragm failures observed in the Northridge earthquake. These observations have been almost exclusively of buildings with wood diaphragms.

Currently used design provisions for new buildings [IBC (ICC, 2003a), ASCE 7 (ASCE, 2005), and NEHRP (FEMA, 2004)] contain one set of requirements addressing wall anchors and cross-ties for concrete and masonry walls used with flexible diaphragms. These requirements do not do not differentiate between wood and steel deck diaphragms in application these requirements, nor do they differentiate between different possible tilt-up building configurations and behaviors. Opportunities to observe earthquake performance of steel deck diaphragms and newer building configurations have been limited to date. Until performance information is available from research or earthquake experience, the vulnerable behavior seen in wood diaphragm **PC1** buildings needs to be considered a possibility for all **PC1** buildings. The fundamental rehabilitation concept is positive anchorage of tilt-up walls to supporting diaphragms, with anchorage load paths adequate for forces both away from and towards the diaphragm.

The *SEAONC Guidelines* document provides a detailed list of rehabilitation measures and relative priorities based on both potential hazard and the level of design of the existing construction. The major categories of deficiencies and rehabilitation (not prioritized) are:

Out-of-plane wall anchors to walls and pilasters
Diaphragm cross-ties
Collectors
Diaphragm strength, stiffness and openings
Wall in-plane shear connections
Wall in-plane capacity
Wall in-plane base anchorage
Wall out-of-plane bending

These deficiencies and rehabilitation measures are included in Table 16.3-1 and the general discussion below; however, the compilation of information in the *SEAONC Guidelines* is recommended as a useful resource.

### Global Strength and Stiffness

Global strength and stiffness of the tilt-up walls have not been seen as a significant source of damage in **PC1** buildings to date. This is likely due to the length of solid wall provided in older buildings, minimum reinforcing ratios, and the tendency to design at low stress levels so that a single layer of reinforcing without special detailing can be used. Strength and stiffness deficiencies are most likely to occur at wall lines with significant number of large penetrations and other locations where loads are carried by a limited number of panels. Rehabilitation measures include addition of new vertical elements, enhancing existing walls, and infilling openings in existing walls.

### Configuration

Poor distribution of shear walls can result in torsionally irregular behavior of **PC1** buildings. Common occurrences include an entire line of highly perforated tilt-up panels such as at loading dock walls in distribution and storage facilities and street front walls in commercial buildings. Concrete cracking and spalling have been seen in perforated wall panels that act as frames. The most direct approach to rehabilitation of this condition is the addition of strength and stiffness in line with the perforated wall. This can be accomplished through addition of new shear walls, enhancing of existing shear walls, or addition of steel braced frames.

Re-entrant corners are reasonably common in large box-type **PC1** buildings, either due to in-set panels (Figure 16.3-1), or an L-shaped building plan. The in-set walls create a hard spot in the diaphragm, restraining it from the deflection required to transmit load to the end wall. In most cases, the in-set walls will have to be considered shear walls supporting the diaphragm. Where chords and collectors have not been provided at the re-entrant corner, the diaphragm has been seen to pull away from the wall, damaging gravity and lateral load connections. Rehabilitation at re-entrant corners requires the provision of adequate chords and collectors, shear transfer to the in-set wall panels, and possibly the strengthening of the diaphragm, wall panels, and connections to the foundation. In some cases, high earthquake loads in existing elements may make it necessary to add new vertical elements in line with the re-entrant corner. The *SEAONC Guidelines* suggest that there may be diaphragm continuity over this interior diaphragm support, increasing the diaphragm reaction to the in-set wall line. Alternately, it may be possible to allow the wall at the re-entrant corner to rock, or to separate the wall from the diaphragm allowing the diaphragm to span to the exterior wall. These approaches require complex detailing, however.

Some very large tilt-up buildings are constructed in configurations that resemble multiple buildings alongside each other, as seen in Figure 16.3-2. This figure illustrates a single large tilt-up building constructed in three sections, separated by roof expansion joints. Thermal movements in the steel deck diaphragms make the expansion joints necessary. If each of the three building sections were provided a complete lateral force-resisting system and separated by adequate seismic joints, this building configuration would be of little concern. As shown in Figure 16.3-2, however, building sections are often laterally braced off of adjacent building sections, using shear transfer through the expansion joint. This configuration raises concerns of

| Table 16.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for PC1 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane strength of shear walls or frames | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Fiber composite wall overlay [13.4.1]<br>Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| Global Stiffness | Insufficient in-plane stiffness of shear walls or frames | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Fiber composite wall overlay [13.4.1]<br>Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| Configuration | Torsionally irregular plans (highly perforated wall line) | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Enhance existing collector [7.4.2]<br>Fiber composite wall overlay [13.4.1]<br>Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| | Re-entrant corners | Steel braced frame [7.4.1]<br>Collector [7.4.2]<br>Concrete/masonry shear wall [21.4.8] | Enhance existing collector [7.4.2]<br>Fiber composite wall overlay [13.4.1]<br>Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| | Incidental bracing | | | | | Isolate component from lateral force-resisting system |
| Load Path | Inadequate or missing wall-to-diaphragm tie for out-of-plane load – exterior and interior walls | | | Wall-to-diaphragm anchorage [16.4.1] plus diaphragm cross-ties [22.2.3] | | |

**Table 16.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for PC1 Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Load Path (continued) | Inadequate anchorage to diaphragms for in-plane forces – exterior and interior walls | | | Wall-to-diaphragm shear anchors [21.4.8] | | |
| | Beam or girder connection to tilt-up wall inadequate for wall out-of-plane loads | | Enhance beam or girder connection [16.4.2] | | | |
| | Inadequate connection at base of tilt-up panel | | | Wall-to-foundation connections [16.4.3] | | |
| | Inadequate collectors | Add collector [7.4.2] | Enhance existing collector [7.4.2] | | | |
| Component Detailing | Wall inadequate for out-of-plane bending | | Wall strongback or pilaster [21.4.3] | | | |
| | Inadequate detailing of narrow wall piers | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.7] | Supplement component to provide adequate load path [13.4.1], [21.4.5]<br>Add backup vertical supports where bearing might be lost [21.4.11] | | | |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Steel braced frame [7.4.1]<br>Concrete or masonry wall [21.4.8] | Enhance existing diaphragm [22.2.1]<br>Horizontal braced frame | | | |
| | Inadequate chord capacity | Add chord [22.2.2] | Enhance existing chord | | | |
| | Excessive stresses at openings and irregularities | | Enhance diaphragm detailing | | | |
| | Excessive stresses at openings and irregularities | | Enhance diaphragm detailing | | | |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

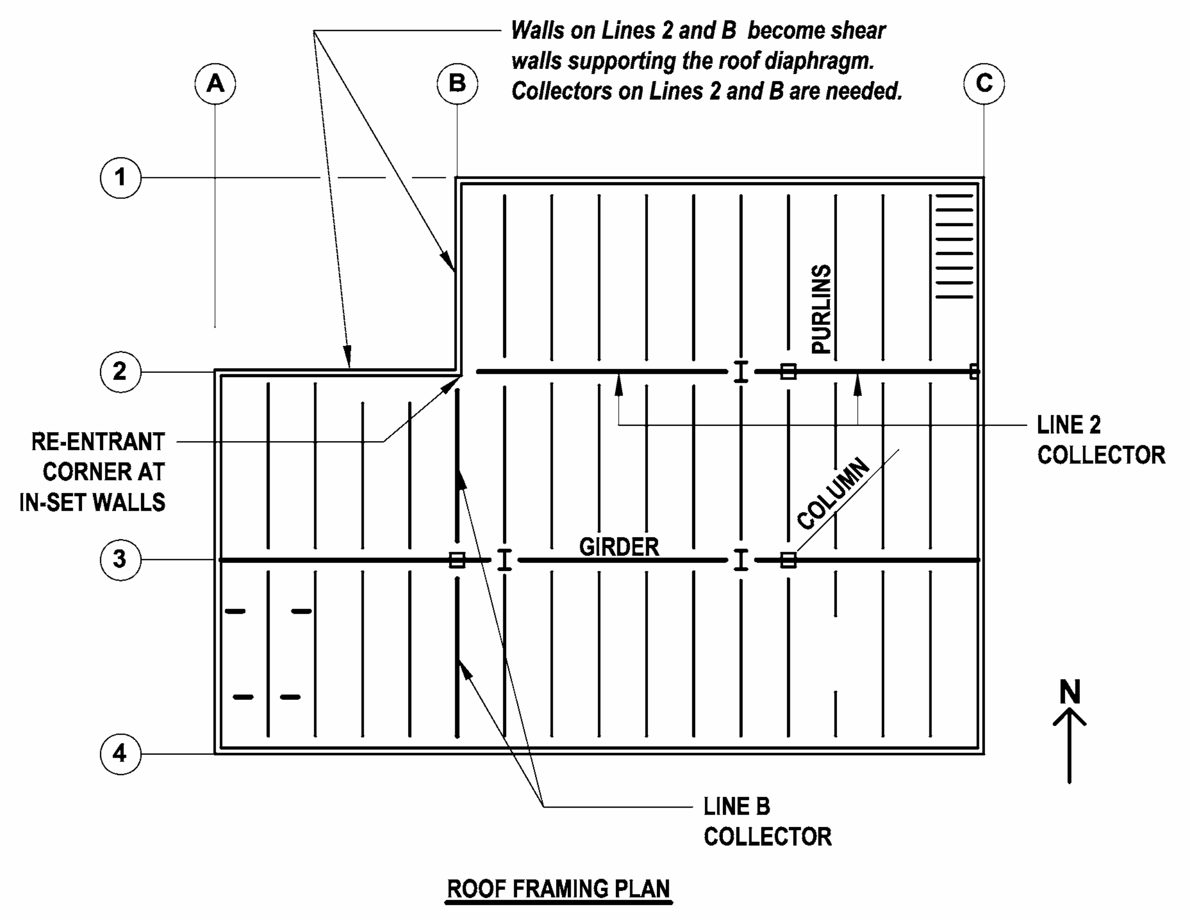
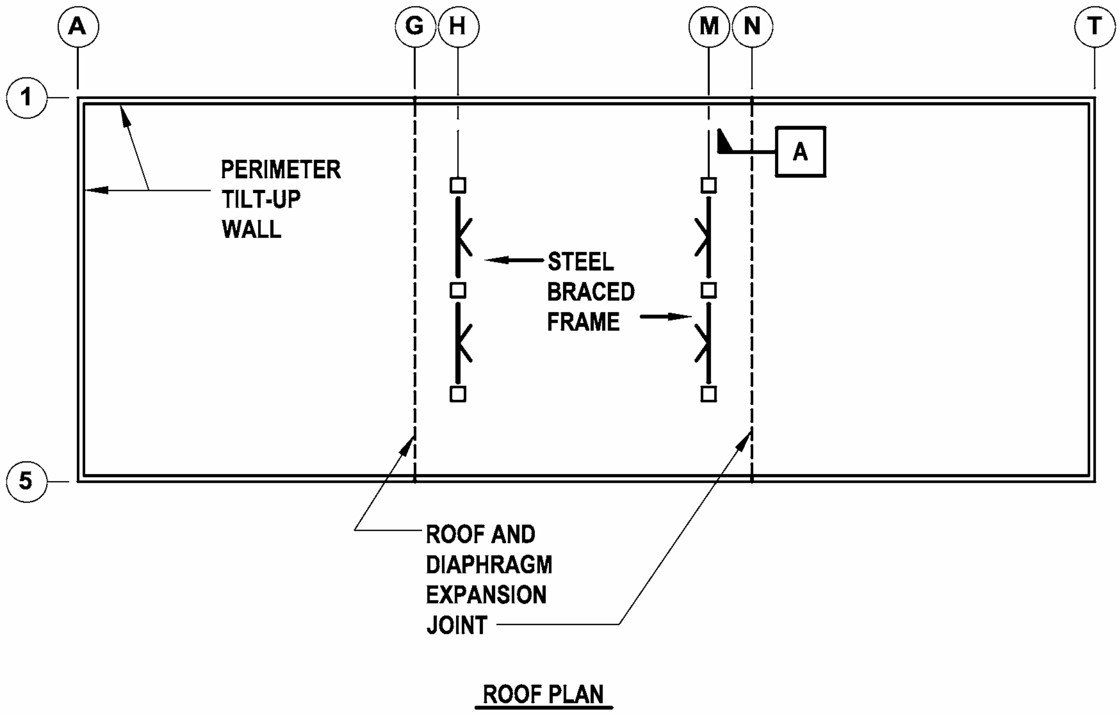



**Figure 16.3-1: Plan of PC1 Building with Re-Entrant Corner at In-Set Panels**

deformation compatibility between the building sections under earthquake loading. Incompatible deformations are likely to significantly compromise shear transfer through the expansion joint. In addition, common details used for the expansion joint may not perform adequately under expected seismic forces. Details often involve significant eccentricities. The eccentricities may create forces in roof framing members that were not envisioned in the framing member design. The most direct approach to rehabilitation of this building type is the addition of vertical elements such that each building section is independently braced. Alternatively, the expansion joint connection can be improved such that it can reliably transfer anticipated earthquake forces while accommodating anticipated building movement.

Another configuration concern is the occurrence of components (such as mezzanines) that act as incidental bracing, creating an unintended load path. For the **PC1** building seismic resisting system to work as intended, the diaphragm must be able to deflect, and the walls must be able to deflect out-of-plane to follow the diaphragm. Ideally rehabilitation of incidental bracing would involve isolation from the building shell. Alternately, the incidental bracing could be analyzed as part of the structural bracing system.



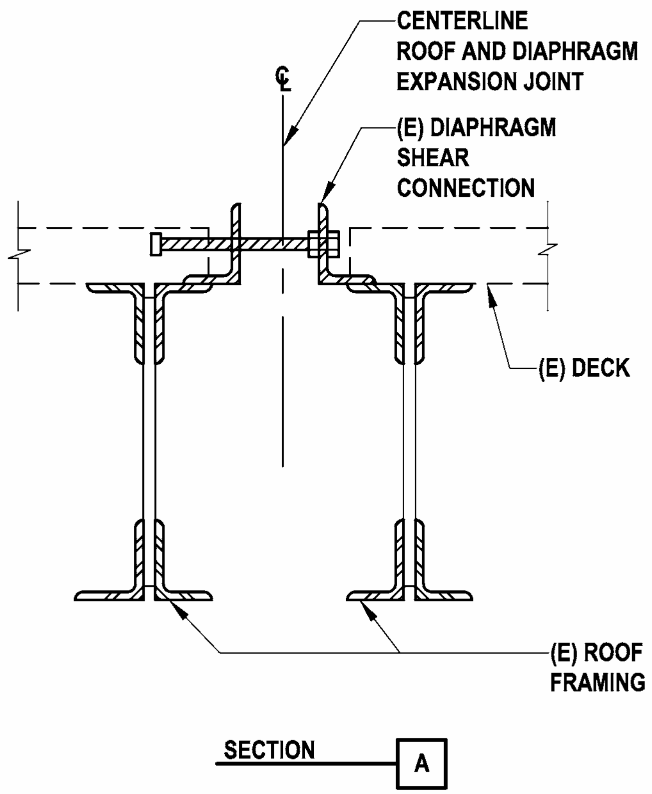




**Figure 16.3-2: Plan and Detail of Large PC1 Building Constructed in Three Sections**

### *Load Path*

As previously mentioned, out-of-plane anchorages between the tilt-up walls and the diaphragm have been the primary source of damage and focus of rehabilitation in wood diaphragm **PC1** buildings. Conceptually, the approach for both new construction and rehabilitation has been to create continuous diaphragm cross-ties between exterior walls on opposite sides of the building. Rehabilitation of wall out-of-plane anchorage is discussed in this chapter. Diaphragm cross-ties are discussed in Chapter 22. Wall anchorage for in-plane shear commonly uses different connectors than for out-of-plane loads. In-plane shear connection is discussed as part of the diaphragm chord discussion in Chapter 22.

Girder gravity load connections to tilt-up walls provide diaphragm-to-wall anchorage and will therefore have to resist wall out-of-plane anchorage forces. These anchorage forces may or may not have been considered in initial connection design. Even if considered, the connection may not be adequate for currently required loads. Rehabilitation of these connections is often required.

The addition of or enhancement of existing collectors may be required in order to transmit diaphragm forces to the resisting shear walls. This is particularly of concern when a limited number of solid panels are intended to carry a significant portion of the building shear. Although not as common, there is also significant concern when vertical offsets in the roof diaphragm result in incomplete chords or collectors. Any breaks or offsets in chords or collectors need to be carefully evaluated. In the 1993 Guam earthquake (EERI, 1995), a high bay portion of a forklift repair shop was braced off of lower bays on each side. Incomplete collectors from the high-bay diaphragm to low bay shear walls resulted in damage.

The anchorage at the tilt-up panel at the wall base may also be deficient. In some older tilt-ups, no positive connections were made from the wall to the foundation or slab; friction was relied on for force transfer both in-plane and out-of plane. Most tilt-up panels in recently constructed buildings will have a base connection, either to the foundation or more likely to the adjacent slab-on-grade; however, it is possible for the connection to be inadequate. Rehabilitation most often involves the addition of new wall to slab connections.

### *Component Detailing*

Component detailing deficiencies include inadequate out-of-plane wall capacity. This deficiency may occur due to increases in design seismic forces or inadequate consideration of panels with openings. It is seldom practical to address wall capacity by adding reinforcing and concrete thickness to individual wall sections, so addition of wall pilasters or strongbacks is common. Where pilasters are added to tilt-up walls, the pilasters stiffen the wall for out-of-plane forces, allowing two-way spanning of the wall and attracting high out-of-plane forces to the pilaster and pilaster-to-diaphragm connection. A pilaster-to-roof diaphragm anchorage must be provided to accommodate the concentrated wall out-of-plane force.

Tilt-up panels with large openings generally have narrow wall panels on each side. These panels and the spandrel wall over the opening act as a concrete frame. In loading dock and storage facilities, entire walls can be made up of frames. Because the narrow wall piers do not meet the ACI 318 (ACI, 2005) definition of a column, many have been constructed with standard wall detailing. The 1994 and later editions on the UBC (ICBO, 1994) defined and created reinforcing requirements for wall piers, which have been brought into the IBC (ICC, 2003a), but are not in ACI 318. Where the wall piers are required for resistance to gravity and lateral loads, rehabilitation may be required. Rehabilitation for lateral loads may involve the additional of new vertical elements, enhancing of existing elements, or filling in some of the openings. In addition, loss of vertical support for the roof adjacent to the panel may be of concern. Rehabilitation for vertical loads may be approached by either enhancing the wall element, or providing back-up vertical supports.

#### *Diaphragm Deficiencies*

Due to changes in building code requirements, it is very common for diaphragms in areas of high seismic hazard to have inadequate in-plane shear capacity. Regardless, the *SEAONC Guidelines* indicates that diaphragm overstresses have rarely been associated with significant earthquake damage. In areas of moderate seismic hazard, this may or may not be a significant deficiency. Diaphragm strength and stiffness deficiencies are most often rehabilitated by enhancing the existing diaphragm. The addition of new vertical elements to reduce diaphragm span and therefore diaphragm shears is also an effective approach, but such an approach is not as commonly used. Diaphragm enhancement is addressed in Chapter 22.

Many California tilt-up buildings have been constructed with solid-sawn roof purlins with sizes ranging from 4x12 to 4x16. These framing members may have calculated overstresses under existing dead and live loads due to reductions in allowable stresses from in-grade values introduced in the 1991 NDS (AF&PA, 1991). When this is the case, removal and replacement of roof sheathing is a preferred alternative to overlays in order to keep additional dead load a minimum. Diaphragm enhancement using staples may also provide required strength. Sometimes, lighter roofing finish materials can be installed where existing roofing is removed, to further reduce overstresses.

Other diaphragm deficiencies include inadequate chord capacity and stress concentrations at large diaphragm openings and re-entrant corners. Rehabilitation at re-entrant corners primarily involves the provision of adequate chords and collectors. The same is true at large diaphragm openings.

## 16.4 Detailed Description of Techniques Primarily Associated with This Building Type

### 16.4.1 Enhance Wall-to-Diaphragm Anchorage

#### *Deficiency Addressed by Rehabilitation Technique*

This rehabilitation technique addresses enhanced anchorage of walls into wood sheathed or steel deck diaphragms for out-of-plane loads. Cross-ties are a required continuation of the wall anchorage system. See Section 22.2.3.

Wall anchors and cross-ties should be the highest priority for rehabilitation of wood diaphragm **PC1** buildings. This section illustrates the basic rehabilitation requirements for wall anchorage of wood and steel diaphragm buildings. Refer to the *SEAONC Guidelines* for more exhaustive treatment of detailing.

The *SEAONC Guidelines* provides details of code requirements and observed damage over many years. Earthquake damage to wall anchorage has been observed not only in older buildings, but also in recently constructed buildings, where the type and installation of wall anchors was critical to their ability to perform. Just providing anchors is not enough; attention to detailing and field installation is critical.

### *Description of the Rehabilitation Technique*

Figure 16.4.1-1A illustrates a roof plan for a **PC1** building with a wood diaphragm. Figures 16.4.4-1B through 16.4.4-1D, located as shown on the roof plan, illustrate common rehabilitation measures for wall-to-diaphragm anchorage. Also illustrated in Figure 16.4.1-1A are subdiaphragms used as part of the diaphragm cross-tie system, described in detail in Chapter 22. While subdiaphragms are always used in wood structural panel diaphragms, they are only occasionally used in steel deck diaphragms.

Figures 16.4.1-1B and 16.4.1-1C illustrate anchorage from the west wall to a subdiaphragm extending between Lines A and B. Similarly, the east wall is anchored to a subdiaphragm between Lines G and H. The primary objective is to create an adequate load path for out-of-plane loads acting both into the diaphragm (compression) and away from the diaphragm (tension). This type of anchorage is generally provided every four to eight feet on center. The load path into the diaphragm includes both wall to framing member anchorage and sheathing nailing to transfer loads from the framing member to the sheathing. Figure 16.4.1-1B relies on existing sheathing nailing, while Figure 16.4.1-1C adds sheathing nailing.

Where framing member ends are tight against the wood ledger and the wood ledger is tight against the exterior wall, the compression load path can be carried by the framing members. Experience has shown that gaps often occur between member ends and the ledger. The movement required to close these gaps has been enough to damage the roof sheathing and damage devices used for tension anchorage.

Tie-down devices can provide both the tension and compression load path. In order to do this, the device must be rated for both tension and compression loading by the manufacturer, and it must be installed in accordance with the manufacturer's instructions for compression load. This generally involves limiting the unsupported length of the tie-down rod and providing additional nuts and washers at the tie-down seat. Very few tie-down devices are currently available that are rated for both tension and compression loads. Building codes such as the now require that forces and stresses induced by eccentricities in the connection be addressed, and the *SEAONC Guidelines* encourages use of tie-downs placed symmetrically on the purlin or sub-purlin in order to minimize eccentric beam loading and encourages use of stiff tie-downs in order to minimize deformation demand and possible diaphragm damage resulting from deformation. For girders, symmetrical tie-downs are recommended but not as critical as with smaller framing members.

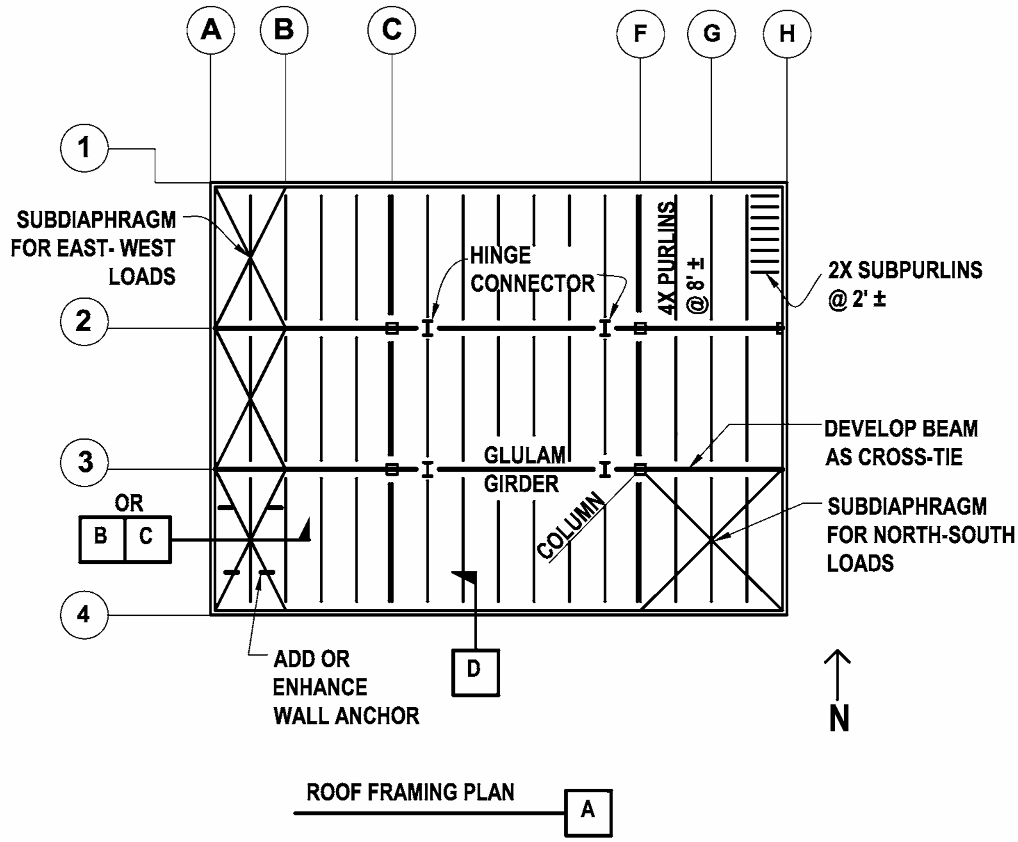



**Figure 16.4.1-1A: Roof Plan with Wall Out-of-Plane Anchorage for Flexible Wood Diaphragm**

Figures 16.4.1-1A and 16.4.1-1C do not show wall anchors attached to single existing 2x subpurlins. It is recommended that anchorage be to a 3x or wider member, or multiple 2x's as shown in Figure 16.4.1-1B. While it may be possible for a single 2x4 or 2x6 to be shown adequate in calculation, adequate installation and performance are extremely difficult to achieve.

The wall anchorage system needs to extend across the width of the subdiaphragm. In Figure 16.4.1-1C, this involves providing extra pairs of tie-down devices between subpurlins across the first purlin. Again, both tension and compression load paths are needed. The required subdiaphragm depth is determined from the number of nails required to transfer subdiaphragm forces into the main diaphragm. Depending on subdiaphragm requirements, additional tie-down pairs could be required across more purlins. Subdiaphragm requirements are discussed in Chapter 22.

Details 16.4.1-1B and 16.4.1-1C show work from top of the diaphragm. See Chapter 22 for alternates for working from below. Location of access needs to be decided early on in the design process and will drive both calculations and detailing of the rehabilitation work.

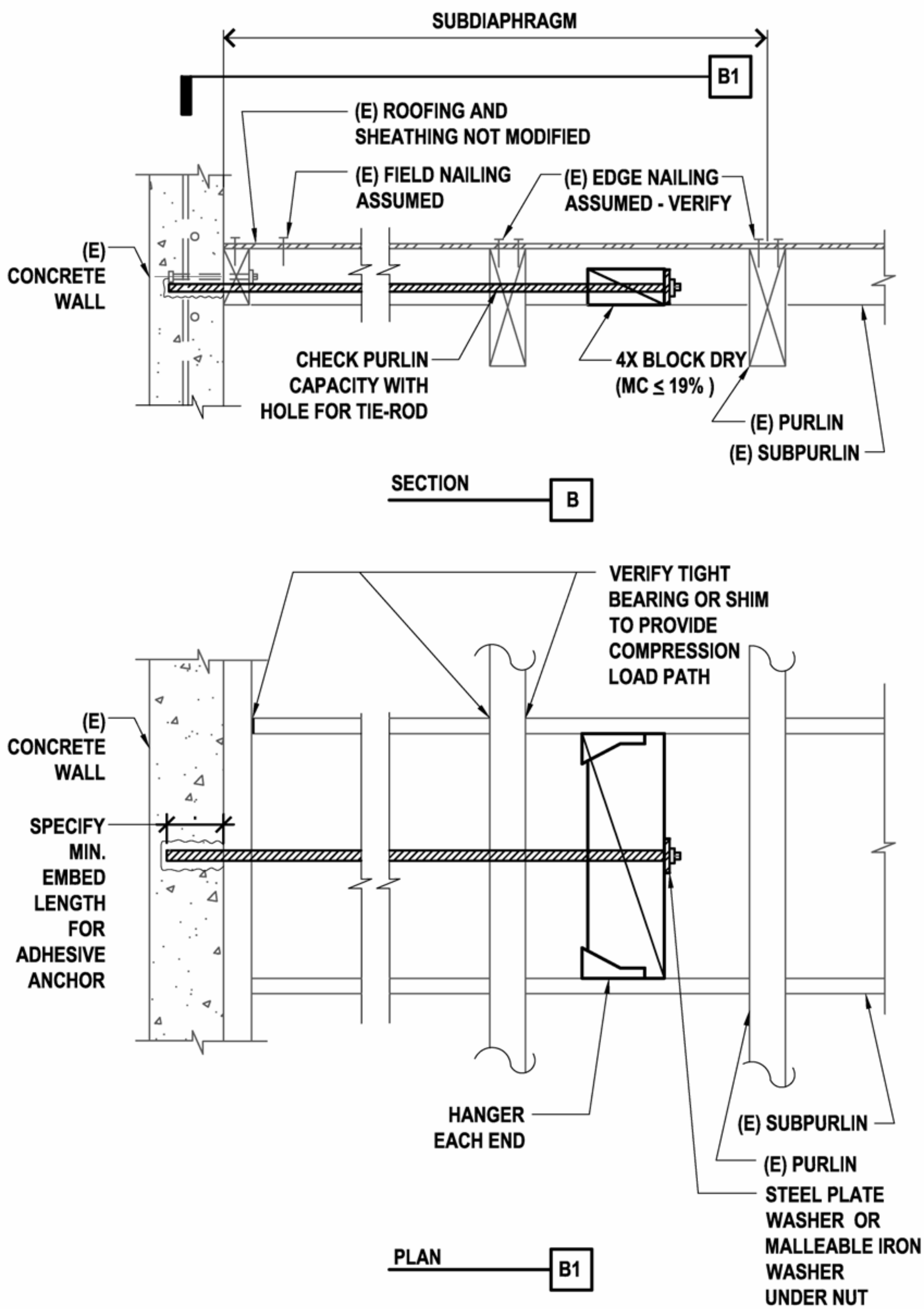



**Figure 16.4.1-1B: Wall Out-of-Plane Anchorage for Flexible Wood Diaphragm at Subpurlins – Roofing Not Removed**

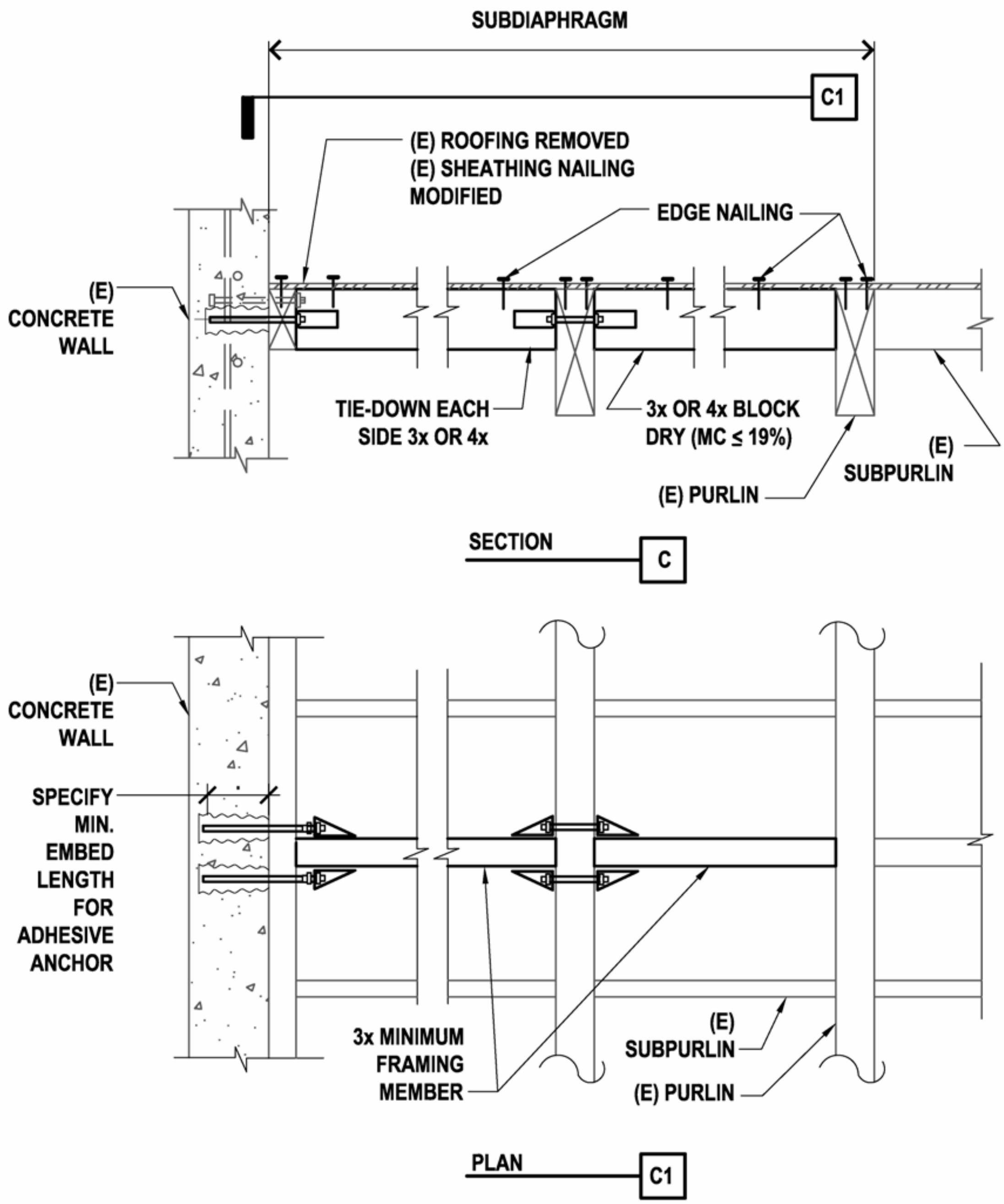



**Figure 16.4.1-1C: Wall Out-of-Plane Anchorage for Flexible Wood Diaphragm at Subpurlins – Roofing Removed**

Figure 16.4.1-1D illustrates anchorage of the north and south walls to a purlin. In this case, the purlin is long enough to extend across the subdiaphragm width (extending between Lines 1-2 and 3-4), so additional pairs of tie-downs are not needed. As is previous details, both tension and compression load paths must be maintained.

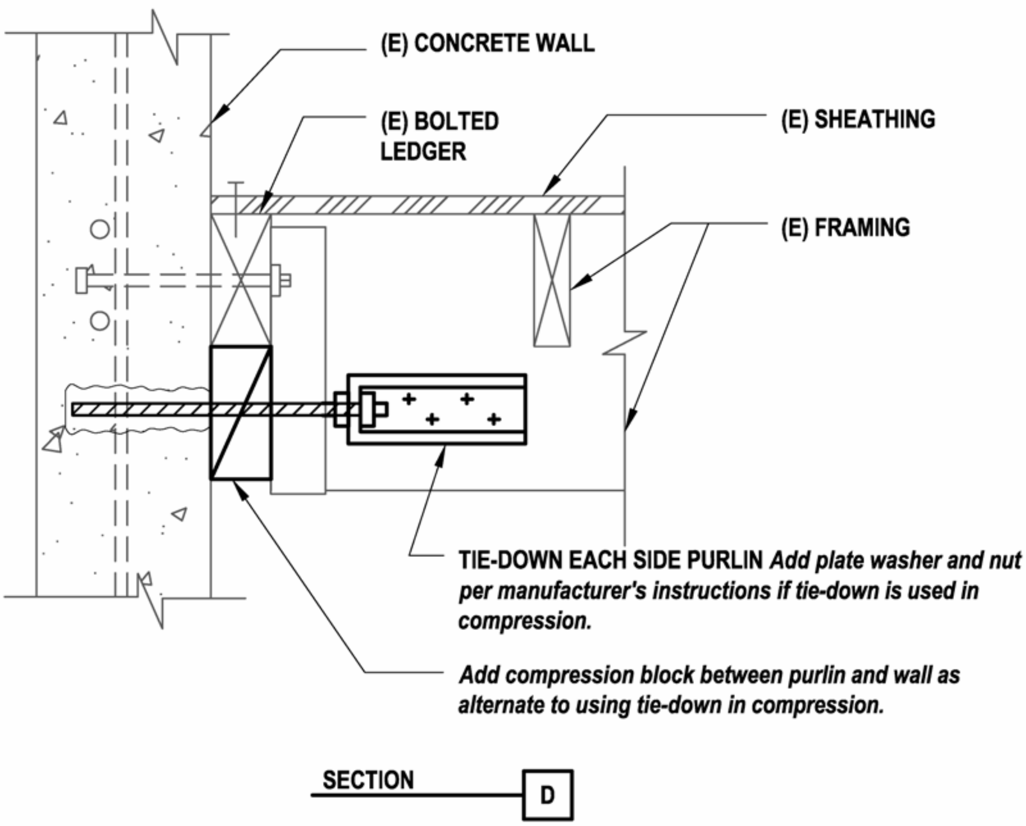



**Figure 16.4.1-1D: Wall Out-of-Plane Anchorage for Flexible Wood Diaphragm at Purlins**

Figure 16.4.1-2A illustrates a similar roof plan for a **PC1** building with a steel deck diaphragm. It is important to note in this figure that subdiaphragms (as shown in Figure 16.4.1-1A) are not used. Instead, the steel deck provides a continuous cross-tie in the east-west direction, while in the north-south direction open web joists provide direct cross-ties across the entire diaphragm width at each wall anchor location. This is the primary approach used in new steel deck diaphragm construction. Subdiaphragm concepts can be applied to steel deck construction, but are not common.

Figures 16.4.1-2B and 16.4.1-2C provide wall to diaphragm anchorage details. In Figure 16.4.1-2B, wall anchorage forces are transmitted to the steel deck. The deck section, deck edge fastening, and deck end splices need to be checked for wall anchorage tension and compression forces. Justification of the capacity may be by calculation or testing. The balance of the load path also needs to be checked and enhanced as required. In the detail shown, supplemental adhesive

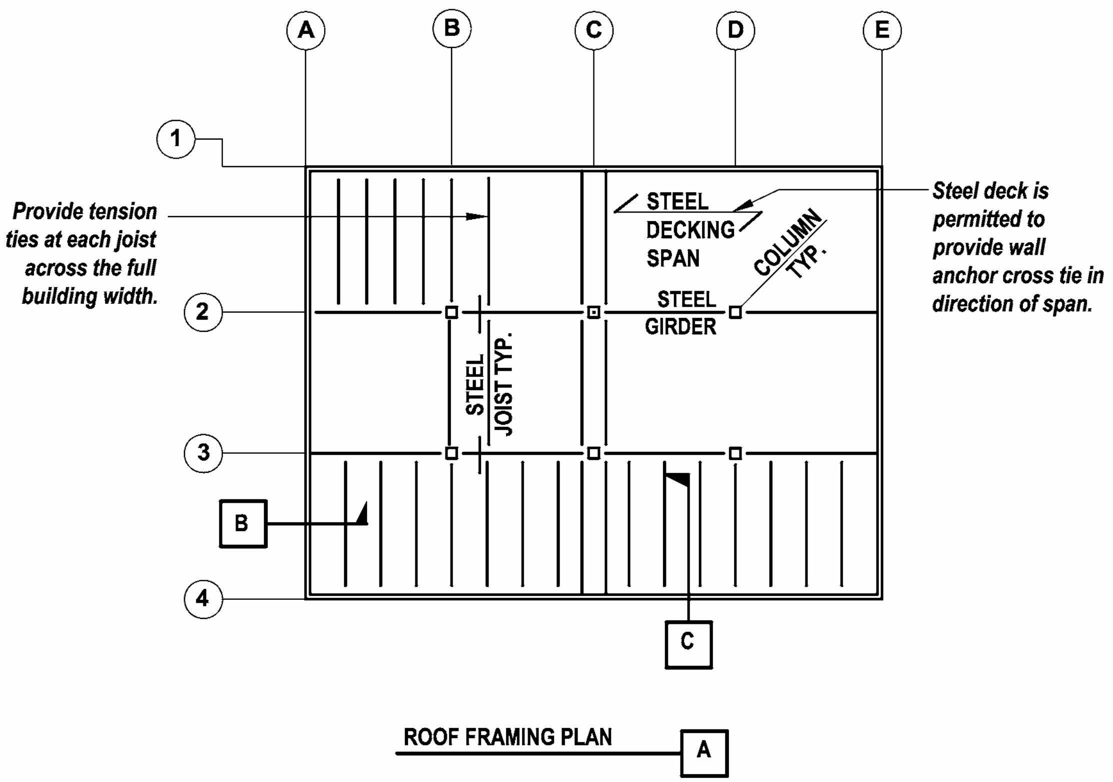



**Figure 16.4.1-2A: Roof Plan with Wall Out-of-Plane Anchorage for Flexible Steel Diaphragm**

anchors to the wall and a second ledger angle are provided. Prying action in the steel ledger angle needs to be considered in determining wall anchor forces.

Figure 16.4.1-2C shows wall anchorage to a steel open web joist. The wall anchorage connection may be through the joist seat in new construction; however, a supplemental anchor to the joist is likely needed in rehabilitation. The joist must be checked for wall anchorage forces and any applicable eccentricities. Details in Chapter 22 address joist to joist connections to complete the cross-tie.

Compression forces can be carried in tie-down devices, if rated for compression by the manufacturer. Unsupported tie-down rod lengths must be kept short, and additional nuts and washers are needed to transfer compression. Again, wall anchorage loads need to be transferred into the decking. In this case existing welding or screwing of the decking is relied upon. If this fastening is not adequate, the roofing will need to be removed to allow additional fastening.

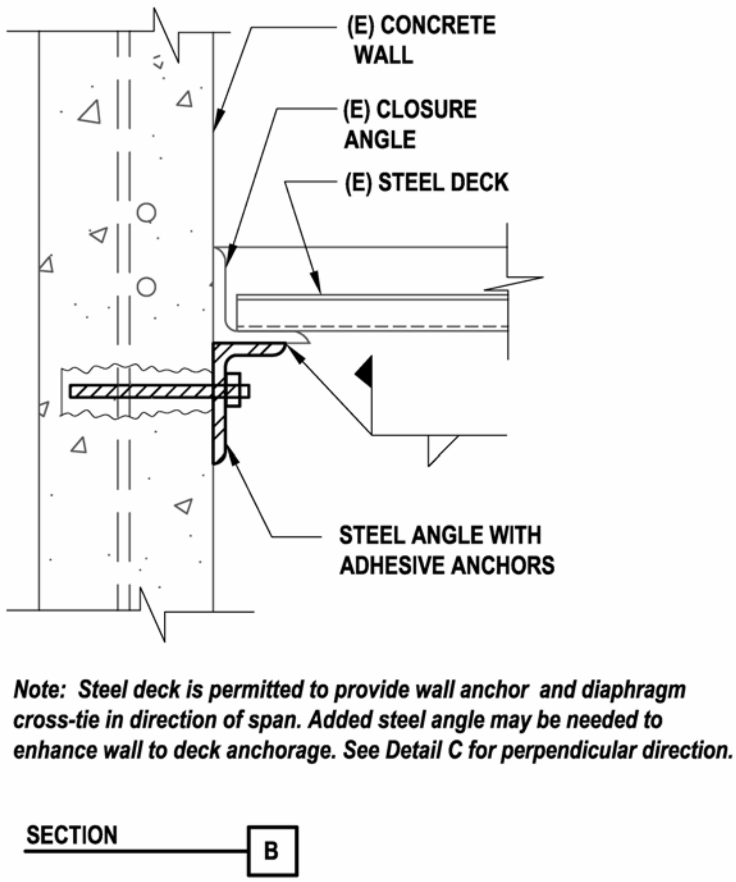



**Figure 16.4.1-2B: Wall Out-of-Plane Anchorage for Flexible Steel Diaphragm – to Decking for Load Parallel to Flutes**

### *Design and Detailing Considerations*

*Research basis:* No research applicable to the performance or adequacy of enhanced anchorage methods has been identified; however, the demands created in flexible diaphragms have been studied by Fonseca, Wood and Hawkins (1996); Hamburger and McCormick (1994); Ghosh and Dowty (2000); and Freeman, Searer, and Gilmartin (2002).

As discussed in Section 16.1, even wall anchorages constructed or rehabilitated in the 1980s and early 1990s were observed to have been damaged in the 1994 Northridge earthquake. The reader is referred to the extensive discussion in the *SEAONC Guidelines* for design and detailing considerations and lessons learned.

*Anchor type and installation:* A variety of proprietary anchors are available for anchorage to existing concrete walls. Both manufacturer literature and ICC Evaluation Service reports should be consulted for information on conditions of use, allowable loads, and installation and inspection requirements. It is important to make sure that the anchor type is appropriate for the material to which it will be connected and is approved for seismic loads. The diameter of drilled holes is specified in installation requirements for each anchor type; variation from this size often

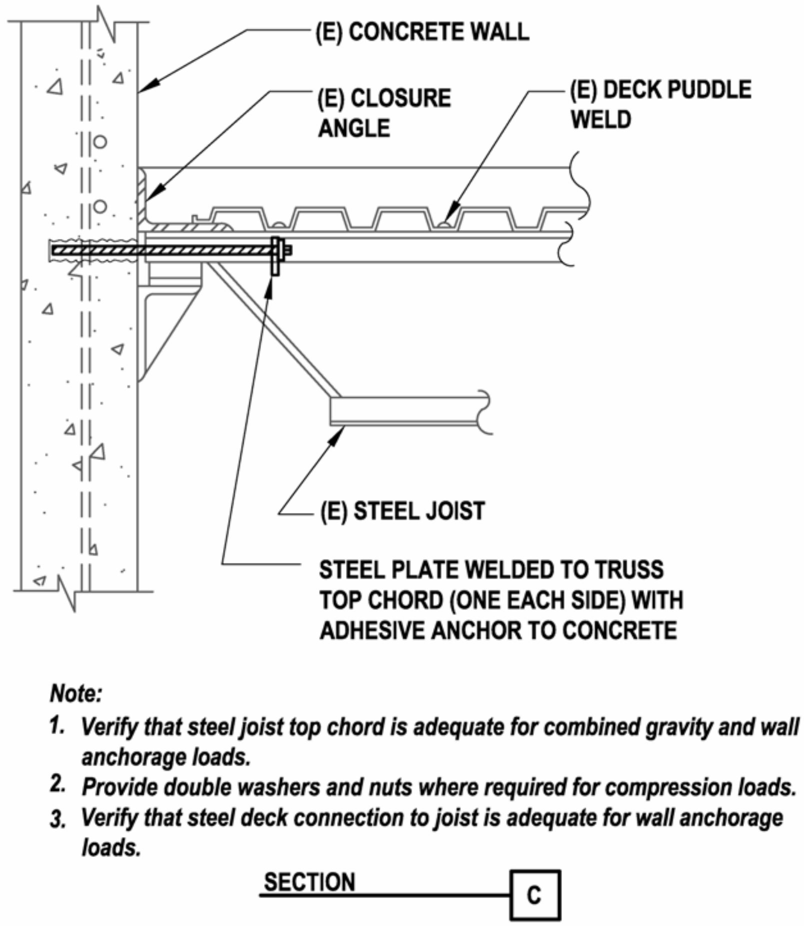



**Figure 16.4.1-2C: Steel Open Web Joist Anchorage to Exterior Wall**

leads to inadequate anchor capacity. Most manufacturers have caulking gun-like devices that make field placement of epoxy fairly simple and automatically mix two-part adhesives. Generally, these types of adhesives provide more than adequate strength, and there is no need to use more complicated high-strength adhesive types. The cleaning of holes prior to placing adhesive anchors is paramount for anchor capacity. When not well cleaned, the anchors can pull out at a small fraction of the design load. It is common to pull-test a portion of the adhesive anchors to verify adequate installation. The pull test load is usually in the range of one to two times the tabulated allowable stress design tension load. The bridge used for testing generally makes a concrete cone pull-out failure unlikely. The test load should not be near yield load for bolts or adhesive pull-out failure loads.

Anchors added in rehabilitation will often have to work in combination with existing cast-in anchors. In order to allow load sharing, anchorage of similar stiffness is desirable. This is often best achieved with adhesive anchors. In addition, anchorage provisions for new buildings have moved towards having the attachment to concrete be capable of developing the yield capacity of the steel anchor in order to promote ductile connection behavior. Again, this is best achieved with adhesive anchors. Anchor types other than adhesives should be carefully evaluated.

***Cost, Disruption and Construction Considerations***

When rehabilitation work is undertaken on the roof diaphragm of a **PC1** building, it is important that the cost and the preferred location for work (from the underside or top of roof) take into account the combination of work, rather than considering one piece at a time. If several diaphragm measures will be undertaken, it will quickly become cost-effective to remove the roof and allow work from the top. This is particularly true if a steel deck requires several rehabilitation measures.

***Proprietary Concerns***

There are no proprietary concerns with this rehabilitation technique, other than the use of proprietary connectors and adhesive anchors as part of the assemblage.

### 16.4.2 Enhance Beam and Girder Connection to Supporting Elements

***Deficiency Addressed by Rehabilitation Technique***

This rehabilitation technique addresses enhancement of girder gravity load connections to tilt-up walls. While primarily intended to carry gravity loads, these connectors should also be adequate to resist wall out-of-plane loads.

***Description of the Rehabilitation Technique***

Tilt-up walls may have pilasters supporting girder loads. In older tilt-up buildings, these are often cast-in-place pilasters, while in newer buildings pilasters are cast as part of the tilt-up panel. The pilaster acts as a wall stiffener, allowing the wall to span both horizontally and vertically under out-of-plane earthquake loading. This attracts more load to the pilaster, and the top of pilaster reaction for out-of-plane loading will be higher than at a typical wall anchor. The gravity load connection generally also serves as the girder-to-pilaster connection for out-of-plane loads. Two deficiencies are common with this connection: 1) inadequate confinement around anchor bolts embedded in the pilaster top and 2) inadequate connection to the girder. Tension loads on the connector have led to splitting of the column top, pulling away the wedge of concrete in front of the anchor bolts. In recent codes, the placement of three closely spaced ties at anchor bolts has been required to reinforce across the anticipated concrete crack. Where added ties have not been provided, a collar around the pilaster top can provide external reinforcement (Figure 16.4.2-1). The collar should be relatively stiff to minimize splitting of the concrete before load is taken up. The second issue is inadequate connection to the wood girder, including bolt capacity and placement. Where the girder seat connection to concrete is adequate or can be enhanced, inadequate bolt capacity can be mitigated with addition of bolt tabs (Figure 16.4.2-2). Alternately, it is possible to use new wall anchors from the girder to the wall, bypassing the girder seat (Figure 16.4.2-3). Again, this connection must be stiff.

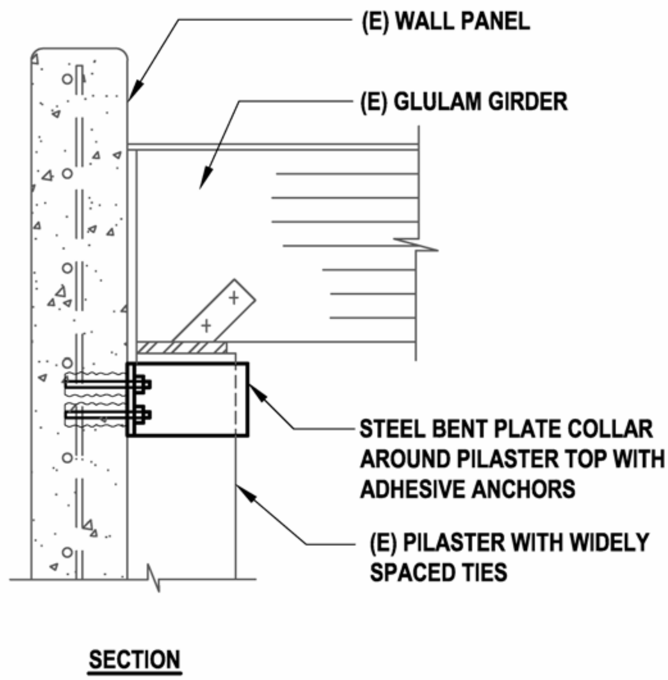



**Figure 16.4.2-1: Enhanced Girder Connection – Collar at Pilaster**
*Adapted From SEAONC (2001)*

Girders that are supported directly on a flat wall panel using a steel U-bracket bolted or welded to the panel (Figure 16.4.2-3) will also attract wall out-of-plane forces. As is true with wall pilasters, a girder and U-bracket are likely to provide a stiffer load path for wall out-of-plane loads than adjacent anchors. For this reason, use of a wall anchorage force greater than used for adjacent anchors is encouraged. The girder connection should have the ability to resist wall anchorage loads in combination with gravity loads. Anchorage of the bracket to the panel will often be adequate for both gravity and lateral loads; however, the bracket attachment to a wood girder will often not have the quantity or placement of bolts required for tension loads. Addition of steel tabs and bolts will add capacity and place bolts where end distances are adequate for tension loads. Where the steel connection to the concrete is not adequate, the out-of-plane anchor might bypass the existing connection and connect the girder directly to the wall. Figure 16.4.2-2 shows two approaches, one with a tie-down on each side of the girder and a second with a tie-down on the girder bottom. The out-of-plane wall anchor should be as stiff as possible to minimize damage to the gravity connection.

***Design and Detailing Considerations***

*Research basis:* No research applicable to this rehabilitation measure has been identified.

See Section 16.4.1 wall anchorage and the *SEAONC Guidelines* for additional detailed discussion.

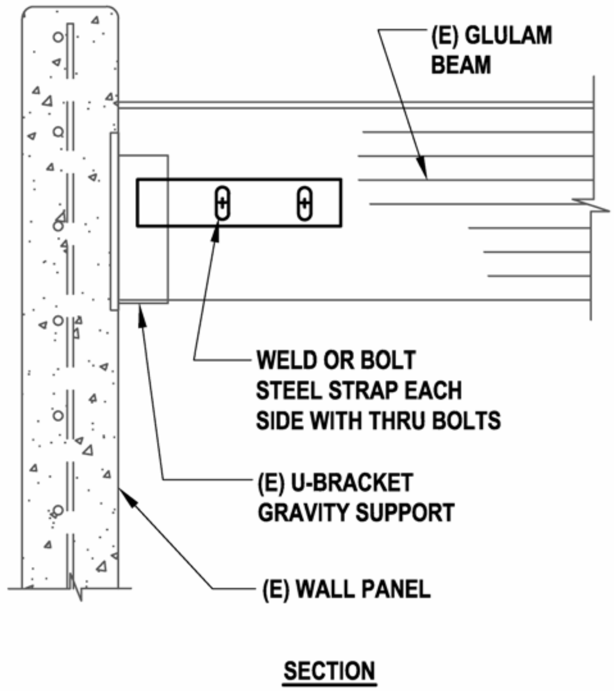



**Figure 16.4.2-2: Enhanced Girder Connection at U-hanger**
*Adapted From SEAONC (2001)*

### *Proprietary Concerns*

There are no proprietary concerns with this rehabilitation technique, other than the use of proprietary connectors as part of the assemblage.

## 16.4.3 Enhance Anchorage at Base of Tilt-Up Panels

### *Deficiency Addressed by Rehabilitation Technique*

This rehabilitation technique addresses the addition or enhancement of connections between the tilt-up wall panel and foundation or adjacent slab-on-grade to resist in-plane shear and overturning forces and out-of-plane wall anchorage forces.

### *Description of the Rehabilitation Technique*

Rehabilitation of base connections in **PC1** buildings for in-plane and out-of-plane shear loads is most commonly accomplished by addition of steel angles and adhesive anchors between the wall panel and adjacent slab-on-grade. This is illustrated in Figure 16.4.3-1. In some instances, the slab-on-grade may not have been thickened adjacent to the tilt-up panel. When this is the case, it may be necessary to remove and recast a thicker pour strip in order to get adequate anchorage. The connection shown would flex if the wall were to uplift. Where uplift connection capacity is required, a direct tension connection of the wall to the footing below is recommended.

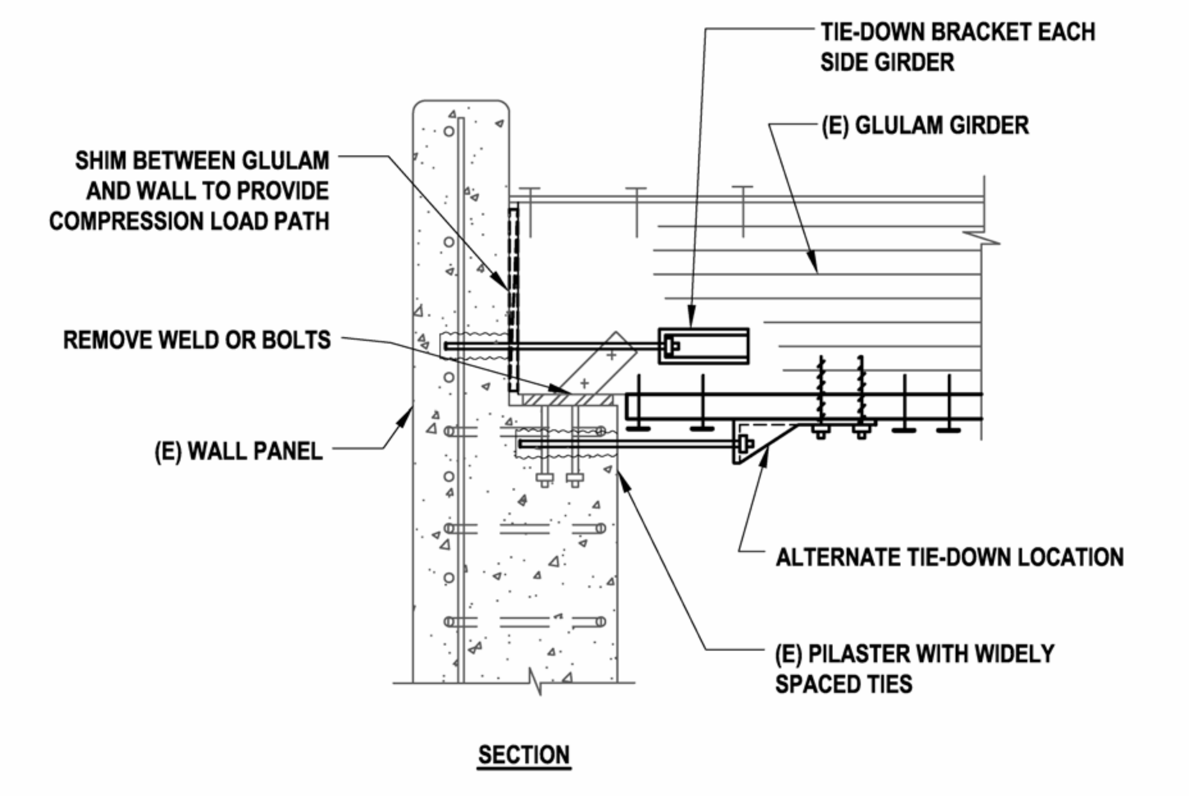



**Figure 16.4.2-3: Enhanced Girder Connection at Pilaster**
*Adapted From SEAONC (2001)*

Variations in base conditions include 1) older **PC1** buildings that may not have any doweling because friction was relied on to resist forces at the base of the panel and 2) welded connections between cast-in embeds in the wall panel and slab, similar to PC2 wall panel connections.

***Design and Detailing Considerations***

*Research basis:* No research applicable to this rehabilitation technique has been identified.

The *SEAONC Guidelines* provide discussion of a variety of possible existing conditions, changes in code requirements, and implications for retrofit.

***Proprietary Concerns***

There are no proprietary concerns with this rehabilitation technique, other than the use of proprietary adhesive anchors as part of the assemblage.

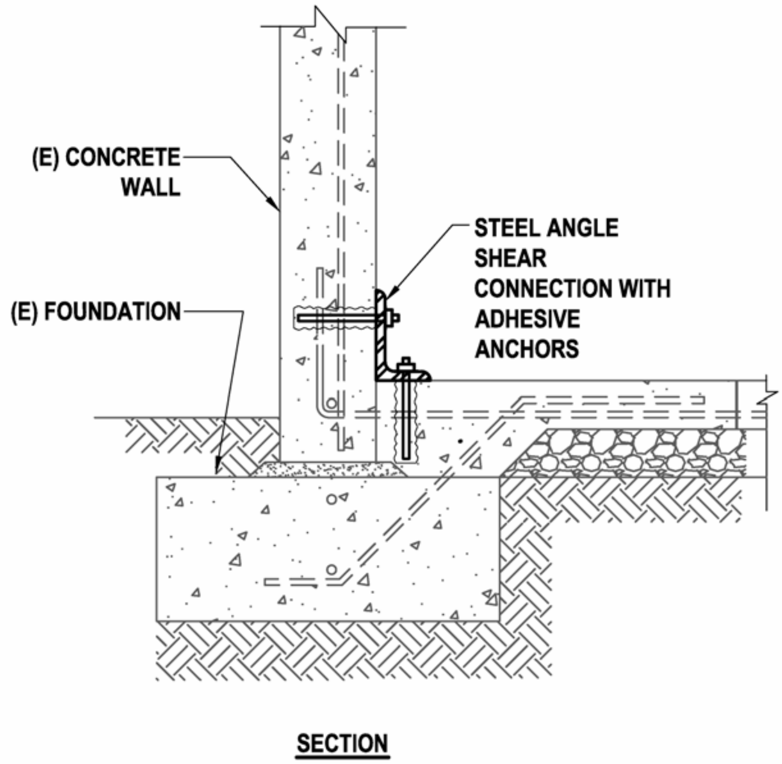



**Figure 16.4.3-1: Enhancement of Tilt-up Panel Base Connection**

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Extensive reference and bibliography listings are included in SEAONC (2001). The following only includes references specifically cited in this chapter.

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# Chapter 17 - Building Type PC2: Precast Concrete Frames with Shear Walls

## 17.1 Description of the Model Building Type

Buildings designated as **PC2** include wide ranging combinations of precast and cast-in-place concrete elements. Precast members may be limited to a floor system of hollow core or T-beam construction, or may include all elements of the gravity and lateral load systems. For this chapter, Building Type **PC2** includes concrete wall or frame buildings in which any of the horizontal or vertical elements of the lateral load system are of precast concrete, except for flexible diaphragm buildings which are addressed as Building Type **PC1** in Chapter 16.

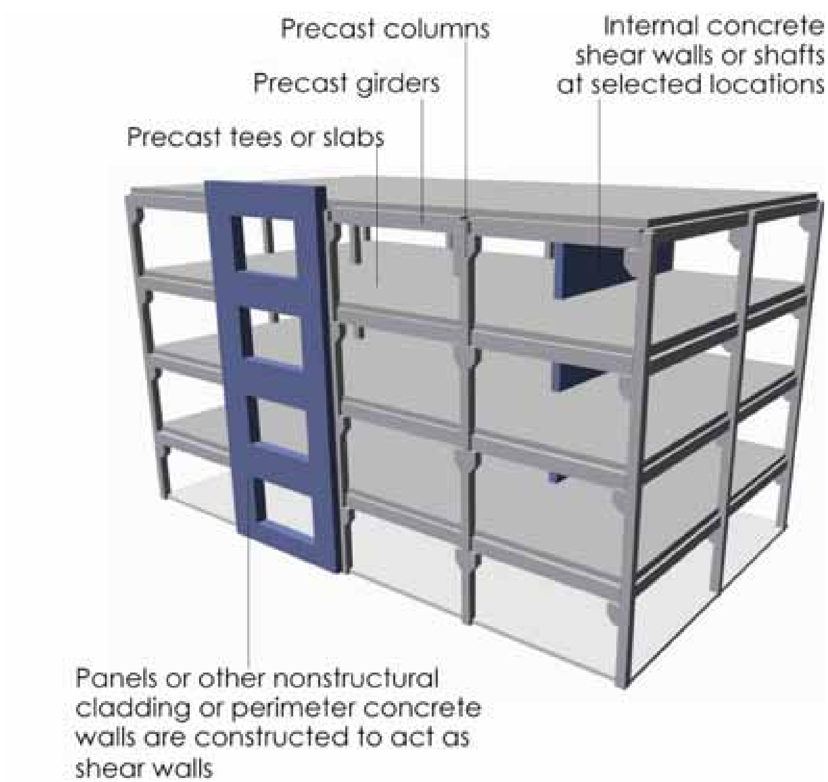


**Figure 17.1-1: Building Type PC2: Precast Concrete Gravity Frames with Shear Walls**

Extensive use of hollow core floor systems in buildings with concrete and masonry walls in southern regions of the United States makes this the single largest group of **PC2** buildings. Parking garages (used exclusively for parking rather than mixed use) represent the next largest group of **PC2** buildings and a substantial portion of the current **PC2** building inventory in the U.S. The **PC2** building type has also been used for a variety of other occupancy types in the U.S. and internationally, including mid-rise office, hotel, and residential buildings, low-rise residential, commercial, and prison buildings.

Over the past decade, significant effort has been devoted to development and testing of precast ductile moment frame systems through the PRESSS (Priestley et al., 1999). These systems are not addressed in this chapter due to their very recent development and state-of-the-art detailing.

### *Gravity-Carrying Load Systems*

Special attention is needed to **PC2** buildings in which concrete frames (beams, girders, and columns or moment frames) resist gravity load, or a combination of gravity and seismic load. Very important to the performance of all concrete buildings with frames, including **PC2** buildings, is the lack of ductile detailing in concrete columns not designated as part of the seismic force-resisting system. These columns in many instances do not have confining steel adequate to accommodate the drift imposed by the seismic force-resisting system and as a result fail through longitudinal bar buckling and concrete crushing. Requirements for estimation of building drift have changed over time, and understanding of potential building deflection has improved with each observation of earthquake performance. As a result, it is important to revisit the ability of non-ductile columns to accommodate estimated drifts, even if they were checked when initially designed. In some precast buildings, the division of initial design responsibility between one engineering firm for the gravity load system and a second firm for the seismic force-resisting system may have contributed to inability to accommodate estimated building deflections. In earthquake performance to date, diaphragm deflections have been a large contributor to deflection of non-ductile gravity systems. Vertical elements, and most particularly moment frames, could also contribute significantly to gravity system deflection.

Following the 1994 Northridge earthquake, column detailing requirements and methods of estimating building deflection for purposes of gravity frame design were modified in codes and standards. Concrete buildings constructed recently in areas of high seismic hazard should perform significantly better than those designed under older provisions. Research continues to develop a better understanding of sources of diaphragm deflection.

When considering the ability of gravity load-carrying systems to accommodate building deflection, a related issue of importance is proper accounting for column stiffness and restraint in analysis. This, again, is a concern for all concrete buildings including **PC2** buildings. Short columns will attract higher forces due to increased stiffness and have been seen to fail as a result. Short columns can be created accidentally due to inadequate separation of the column from nonstructural components such as guardrails. In addition, systematic problems with short columns can occur at parking garage ramps. Analysis models need to pay special attention to these and other sources of shortened columns or columns with increased end fixity.

**PC2** buildings with gravity and lateral loads supported exclusively by structural walls do not have the same issues of deflection of non-ductile columns. Connections tend to be the primary issue of importance to these systems for both gravity and seismic load systems.

### *Shear Walls and Frames*

Building Type **PC2** may have a lateral force-resisting system of concrete shear walls or moment frames, cast-in-place or precast. In **PC2** buildings, critical behavior of shear walls is generally governed by connections including: diaphragm to shear wall, shear wall above to shear wall or foundation below, and interconnection of shear walls within a story. In **PC2** buildings with precast frames, field connections within the frame are critical to performance, as is ductile detailing. Connection practice has varied widely over time and by geographic region.

### Floor and Roof Diaphragms

In California, precast floor T-beams or hollow core planks are covered by a cast-in-place topping slab, reinforced to provide diaphragm action. These toppings need to be clearly differentiated from topping slabs plant-applied to individual precast members, which do not serve the same function of structurally interconnecting adjacent members. Welded connections between embedded inserts or plates may also be used to aid in alignment of members during erection, but are generally not relied on for diaphragm action. Reinforcing bars are often added in the cast-in-place topping slab to act as diaphragm chords and collectors, and welded wire fabric is used to provide shear reinforcement.

In other areas of the United States, common methods of joining floor sections include use of grouted hollow core joints (grout placed in the joint between two adjacent panels, relying on adhesion and/or friction for shear transfer) or welded insert plates. Cast-in-place topping slabs are not commonly used. In some areas outside the U.S., hollow core planks are installed with no connection or grouting between adjacent planks.

As per the discussion of gravity load systems, deflection of the diaphragm system has been seen as a significant contributor to building deflection is past earthquakes. Discussion of diaphragm behavior and rehabilitation can be found in this chapter, Chapter 20 for masonry wall buildings, and Chapter 22 for detailed discussion of diaphragm rehabilitation.

### Parking Structure Issues

Parking structure **PC2** buildings have unique characteristics that deserve specific discussion, some applicable to parking structures regardless of structural system and others specific to precast construction. These issues include the following.

Many parking structures have large plan areas, and considerations of security and restraint against temperature, creep, and shrinkage movement lead to concentration of the shear walls at the building perimeter near the center of each side (Figure 17.2-1). This configuration leads to long diaphragm spans with significant shear, moment, and collector demands. With these high demands, it is possible for the diaphragm, rather than the vertical elements to control building dynamic behavior. This is of concern in all systems, but particular in precast systems due to the lower level of inherent diaphragm continuity.

Compared to other building uses, parking garages have greatly minimized finish and cladding systems, resulting in low levels of nonstructural damping and energy dissipation.

Ramps in parking structures may act as tension and compression struts between floors, resulting in demands not anticipated during design. This behavior can be avoided by inclusion of seismic joints at one end of each ramp; however, seismic joint detailing is difficult to accommodate in precast concrete construction, making use of a fixed connection more likely. Unless the effect of the ramp is specifically considered in analysis, force transfer through the ramp can result in seismic forces bypassing the intended resisting system and significant redistribution of forces in the diaphragms and vertical elements. An analytical study of ramp effects discussed in Lyons, Bligh, Purlinton, and Beaudoin (2003) suggests that while the effect of ramps is significant in moment frame buildings, it is less significant and can often be managed in design of shear wall buildings. The effect of ramps should be considered in evaluation and rehabilitation of parking structure buildings.

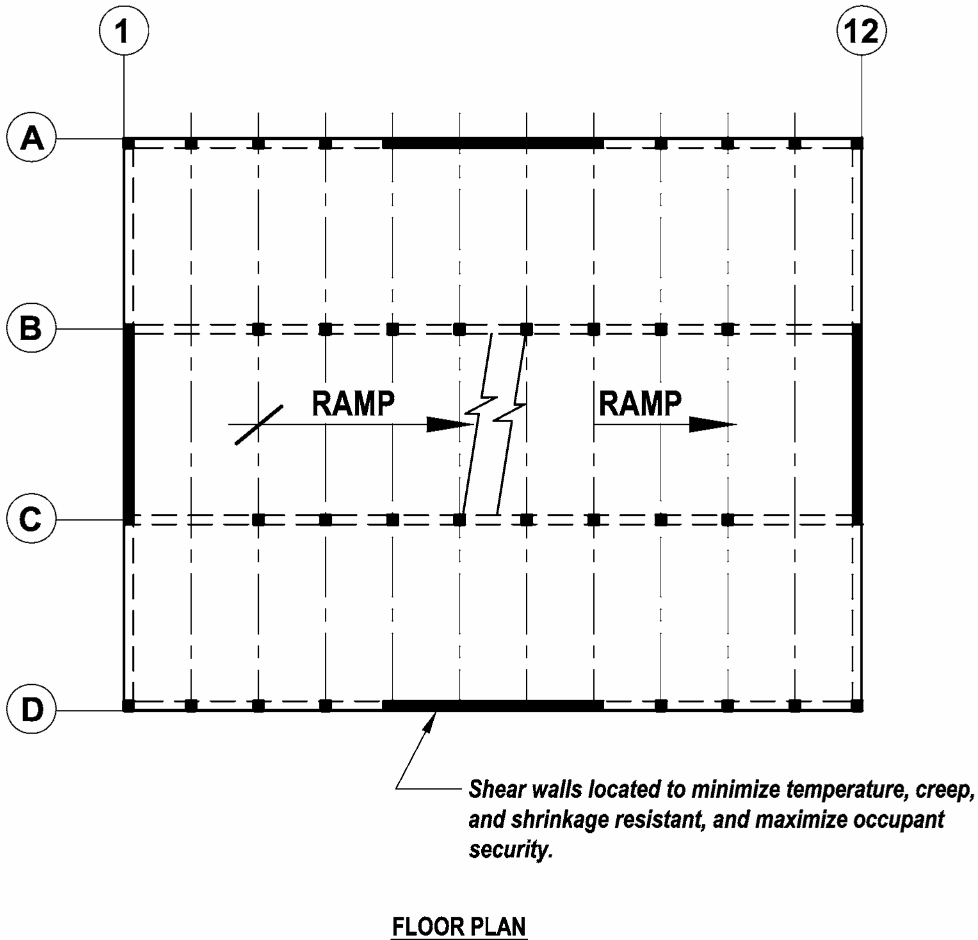


**Figure 17.2-1: Plan of Common Parking Structure Configuration**

## 17.2 Seismic Response Characteristics

**PC2** buildings occur with a wide range of vertical element types. In most cases, the vertical element type will dictate the building seismic response: shear wall buildings will have short period response, while frame buildings will have a longer period. In **PC2** buildings, stiff diaphragm behavior will generally be intended. Parking structure **PC2** buildings with long diaphragm spans, however, have been observed to have inelastic behavior concentrated in the diaphragms rather than vertical shear wall or frame elements. To date, this has been brittle behavior resulting in premature diaphragm failure; however, with development of proper detailing it may be possible to achieve stable long-period diaphragm behavior.

## 17.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

Construction of **PC2** buildings in areas of high seismic hazard in the U.S. has been of limited quantity and recent compared to most other building types, resulting in limited opportunities to observe earthquake performance; the City of Los Angeles and SEAOSC (1994) and EERI (1996) reported on performance of one group of **PC2** buildings following the 1994 Northridge earthquake. Out of an estimated 100 parking garages (precast and cast-in-place) in heavily shaken areas in the Northridge earthquake, eight had partial collapses, and an additional 20 had at least 25% damage (City of Los Angeles and SEAOSC, 1994). The task force looked at approximately 30 structures; of 26 structures with damage, approximately half contained some precast elements (Mooradian, 2005).

Within limited experience to date, life-safety performance of other **PC2** buildings in the U.S. has been good; however, performance in other countries has identified concerns that could be applicable to U.S. construction. See below for general discussion and Table 17.3-1 for a detailed compilation of common seismic deficiencies and rehabilitation techniques for the **PC2** building type.

### *Global Strength and Stiffness*

Insufficient in-plane shear wall strength and stiffness are possible seismic deficiencies in **PC2** buildings and particularly in parking garages where shear wall length is generally limited. Rehabilitation to address inadequate shear wall strength and stiffness can include addition of new vertical elements or strengthening of existing elements, as summarized in Table 17.3-1. Addition of and enhancement to elements in **PC2** buildings is very similar to other concrete building types; however, several additional cautions are in order. First, the configuration of existing precast members, including cast-in voids and prestressing tendons must be carefully studied to allow connection of new elements to existing construction. Second, precast and post-tensioned systems are configured to minimize damaging effects of movement due to temperature variation, shrinkage and creep; these effects should be considered in the addition of new vertical elements.

Insufficient in-plane moment frame strength is a possible seismic deficiency in **PC2** buildings and particularly of concern where the frame might not have been initially designed for seismic loads. Where strength is a concern, it is likely that stiffness, connections, and ductile detailing will also be inadequate and that major addition or enhancement of vertical elements is required.

### *Configuration*

Torsional irregularities can lead to possible seismic deficiencies in **PC2** buildings, as in any other building type, increasing deformation demand in local portions of the structure. One of the parking garages investigated by the City of Los Angeles and SEAOSC Task Force (City of Los Angeles and SEAOSC, 1994) had shear walls on three sides and an open front of the fourth side. The report speculates that excessive deflection at the open-front allowed girders to move sideways off of supporting columns, resulting in total collapse. The torsionally irregular building configuration appears to have contributed to collapse, along with inadequate diaphragm stiffness and component connections. Torsional deficiencies are most directly addressed through the addition of new vertical elements, as indicated in Table 17.3-1. Design to accommodate the concentrations of force and deformation demand may be an alternative.

| Table 17.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for PC2 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane strength of shear walls or frames | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [12.4.2] | Fiber composite wall overlay [13.4.1]<br>Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| Global Stiffness | Inadequate stiffness of shear walls or frames | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [12.4.2] | Fiber composite wall overlay [13.4.1]<br>Concrete wall overlay [21.4.5]<br>Infill wall openings | | | |
| Configuration | Torsional irregularity | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [12.4.2] | | | | |
| | Incompatible deformation of building sections | | | | | Provide seismic separation of portions with different behavior. See general discussion of seismic separation. |
| | Distance between shear walls too large | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [12.4.2] | | | | |
| Load Path | Inadequate force transfer, diaphragm to shear wall, shear wall above to shear wall below, shear wall to foundation | | | Enhance anchorage [17.4.2] | | |

**Table 17.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for PC2 Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Load Path (continued) | Inadequate connection of beam or girders to supporting elements | Supplemental vertical supports [21.4.11] | | Enhance anchorage | | |
| | Inadequate collectors | Add collector [17.4.1] | Enhance existing collector [17.4.1] | | | |
| Component Detailing | Gravity columns inadequate to accommodate drift | Reduce building drift to level acceptable for gravity elements – see global stiffness | Enhance column ductility with jacketing [12.4.4] | | | Remove or reconfigure portions of structure creating short columns [17.4.3] |
| | Inadequate wall strength | See Global Strength | See Global Strength | | | |
| | Inadequate frame connection detailing | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [12.4.2] | Enhance existing frame connections | | | |
| Diaphragms | Inadequate strength and/or stiffness | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [12.4.2] | Enhance shear transfer within diaphragm [22.2.11] | | | |
| | Inadequate shear transfer to walls | | See load path | | | |
| | Inadequate chord capacity | Add chord [17.4.1] | Enhance existing chord [17.4.1] | | | |
| | Excessive stresses at openings and irregularities | | Enhance diaphragm detailing | | | |
| | Re-entrant corners | | Enhance diaphragm detailing | | | |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation measures. | | | | | | |

One of the observations from the Northridge earthquake was that performance of parking garages decreased as the distance between supporting shear walls increased. Long diaphragm spans resulted in high shear and flexure demands on the diaphragms, causing yielding and fracture of diaphragm reinforcing (City of Los Angeles and SEAOSC, 1994; and Wood, Stanton & Hawkins, 2000). This can be considered either a configuration deficiency or a diaphragm deficiency. The addition of vertical elements will contribute significantly to reduction of demand and deflection. See global strength and stiffness discussion. Diaphragm shear capacity, chords, and collectors can also be enhanced, as discussed in the diaphragm deficiency section.

Where **PC2** buildings are comprised of several sections separated by seismic joints, movement can be incompatible and separation inadequate.

### *Load Path*

Deficiencies in the load path connections between diaphragms and vertical elements, story to story and wall to foundation are of significant concern in every type of **PC2** building. Connection and load path detailing in the existing **PC2** building stock is thought to range from systems with no positive connections at all, to potentially brittle welded connections, to recently developed connections and systems that may allow better performance than cast-in-place buildings. Good performance was reported for limited examples of low-rise **PC2** shear wall buildings in the Northridge earthquake (Iverson and Hawkins, 1994) and the Kobe earthquake (Ghosh, 1995). In the Guam earthquake (EERI, 1995) damage to the shear wall to foundation connections in a mid-rise hotel building caused extensive local spalling, apparently related to eccentricities within the connections. In the Armenia earthquake (EERI, 1989) low to mid-rise large panel precast buildings performed well. The good performance was attributed to floor panels that spanned to bearing walls on all four sides, and to the redundancy of these systems. In contrast, the connections between precast column members in frame buildings were very vulnerable, likely due to eccentricities introduced in site modifications to longitudinal bar splices and poor column confinement. Overall load path connections are important to the performance of **PC2** buildings, and attention to detail and eccentricities is important. Rehabilitation of load path connections in shear wall type buildings will generally involve external mechanical connections.

While not as obvious an issue in **PC2** buildings as in **PC1**, as part of load path considerations concrete wall panels must be adequately connected to resist out-of-plane forces. The connections that are transferring diaphragm forces to the shear walls are generally also used to resist out-of-plane forces, so rehabilitation of these connections should consider both demands.

The connection between girders and supporting columns and other similar connections may require rehabilitation in order to provide a continuous load path. The movement of a girder off of the supporting column due to building deflection is the most obvious concern. It has additionally been postulated that, in the Northridge earthquake, some gravity members may have been pulled free of supports due to high vertical accelerations or vibrations. At one time, it was common to have fairly heavy connections between girders and columns at the point of bearing. Connection design would likely have controlled by calculated forces, and welded connection behavior could be brittle. It is now more common to minimize or eliminate connection between the column and girder at the bearing point and resulting restraint of support movement and to rely instead on the column and girder each being doweled into the diaphragm system. This approach minimizes unintended restraint and resulting damage. As in new construction, rehabilitation of this connection would best be accomplished by connection of each member into the diaphragm system. This approach is best suited to the intended behavior of the building system. Alternate approaches could include use of restraint cables, as is common in bridge rehabilitation or use of secondary vertical supports, as is common in rehabilitation of unreinforced masonry buildings (Section 21.4.10). There is no broad consensus on the contribution of vertical accelerations or vibrations to damage of **PC2** buildings in the Northridge earthquake. In locations where vertical thrust is a concern, rehabilitation measures could specifically take vertical demands into consideration.

In some precast systems, design of connections between elements has been based on the concept that the precast system emulates performance of an equivalent cast-in-place system. In frame members, the splicing of reinforcing steel is key to this performance. Over the years, the understanding of demands on splices and adequacy of splice technologies under earthquake loading has changed, as well as understanding of desirable locations of splices and controlling behavior modes within concrete frame systems, both precast and cast-in-place. Rehabilitation of connections between members in precast frames is difficult, and addition of vertical elements to limit frame drift may be a more practical solution.

### *Component Detailing*

The inability of columns in gravity load systems to accommodate building drift has been pointed out as a significant deficiency in earlier discussion. Rehabilitation approaches include either adding additional vertical elements to reduce drift or enhancing columns with fiber-reinforced polymer wraps or similar systems to allow ductile behavior. These rehabilitation measures are discussed further in Chapter 12. The City of Los Angeles/SEAOSC report (1994) identified column wrapping as the only rehabilitation measure for parking garages that could be recommended as both practical and economical. Rehabilitation for the related issue of short columns can sometimes be as simple as creating an adequate joint between the column and the incidental restraint. Where this is not possible, reducing drift or enhancing column ductility are recommended rehabilitation approaches.

### *Diaphragm Deficiencies*

Insufficient in-plane strength and stiffness of diaphragms are of significant concern over a range of precast systems.

In parking garages with long diaphragm spans, insufficient shear strength has been identified as a likely contributor to poor performance of **PC2** parking garages in the 1994 Northridge earthquake (Wood, Stanton & Hawkins, 2000), where pre-earthquake cracking of cast-in-place topping slabs occurred along the joists between T-beams. Analytical studies identified two strength-related issues that had not been considered previously. First, the pre-earthquake cracking of the topping slab along T-beam joints meant that the topping slab concrete was not contributing shear strength, leaving the reinforcing behavior acting in a shear-friction mode. Second, the limited strain capacity of the welded wire fabric reinforcing was being exceeded in commonly observed pre-earthquake crack widths, leaving the reinforcing vulnerable to brittle fracture during earthquake loading. Further, inadequate performance of chord and collector reinforcing in topping slabs has also been identified as a deficiency contributing to damage. In the Northridge earthquake, reinforcing bars serving as chords and collectors in cast-in-place topping slabs appear to have yielded and subsequently buckled (City of Los Angeles/SEAOSC, 1994). This behavior is a combination of strength and stiffness issues. In order to limit diaphragm deflection, maintain the integrity of the chord and collector members, and have the vertical elements control building dynamic behavior, it would be desirable to not have the chord and collector reinforcing yield. Since the Northridge earthquake, ACI 318 (ACI, 2005) and the building codes have taken initial steps towards addressing observed problems. Nakaki (1998) observed that the prescriptive steps taken by ACI 318 did not always result in improved behavior. Nakaki proposed simplified approaches to estimation of force and deformation demand for the purpose of ensuring elastic diaphragm design. This work has been incorporated into an appendix chapter of the *NEHRP Provisions* (FEMA, 2004) for untopped diaphragms.

In diaphragms without cast-in-place topping slabs, connections between adjacent planks or T-beams often use embedded steel plates with field-welded connections, or grout connections. Questions arise as to the pre-earthquake adequacy of these connections. Welded connections are often used to correct differences in camber between adjacent members during initial erection and are often stressed by moving vehicle point loads and shrinkage and creep movement of the building. Observations of connections suggest that reduced capacity prior to earthquake loading may be common. This combines with a changing understanding of earthquake demands on the connection and the interaction of shear demands and deformation due to flexural or tension loading. The complete lack of connection between hollow core floor planks within diaphragms appears to have been a primary contributor to collapse of nine-story residential precast concrete frame buildings in the 1988 Armenia earthquake (EERI, 1989).

A significant integrated analytical and experimental research program is currently underway to develop a comprehensive design methodology for precast concrete diaphragm systems. The project intends to address the discrepancy between current design practice, based on inelastic behavior concentrating in vertical elements and observed performance in which substantial inelastic behavior has occurred in diaphragms (Wan et al., 2004; and Naito and Cao, 2004). The project proposes to determine force and deformation demands required for design, connection details to support the performance, and address deformation relative to the gravity load-carrying system. This information will be invaluable for both new design and rehabilitation. Testing will include individual connections, joints, and half-size components. Analytical modeling of full buildings is being used to identify critical demands. Of particular interest is the simultaneous occurrence of shear and tension or compression on connections normally considered to carry only shear. Published information to date (Naito and Cao, 2004) provides a database of connector properties from existing literature and suggests a simplified analysis model based on initial finite element testing. Additional information should be available over the next several years.

Rehabilitation of diaphragm chord and collector members is reasonably practical due to the focused locations of work. Where possible, it is easiest to add reinforcing steel collectors on the top of the floor system in new cast concrete curbs. Where chords and collectors need to occur at building interior locations where traffic must cross, more complex solutions are required. Unstressed post-tensioning tendons may be a desirable alternative to rebar in some locations; however, it must be kept in mind that stresses must be kept low in order to minimize deformation, so the high strength does not give particular advantage. In undertaking rehabilitation of the chord and collector members, it must be acknowledged that there is lack of consensus on the interaction of shear and flexure demand in these buildings. Care must be taken not to induce brittle shear failure of the floor diaphragm as a result of flexural strengthening.

Rehabilitation of inadequate shear capacity is significantly more difficult. Most precast floor systems will have little capacity to support vertical load from additional topping slab thickness, and removal of existing toppings over large areas is not practical. Research conducted on connections between precast wall panels can be applied to connections between precast diaphragm members. Research by Pantelides, Volnyy, Gergeley, and Reaveley (2003), discussed in relation to load path connection in Section 17.4.2, could be applied to precast diaphragms. In addition a research program is currently being conducted by the Precast/Prestressed Concrete Institute investigating development of ductile panel-to-panel connections in precast diaphragms. See Section 22.2.11 for further discussion.

Rehabilitation for large openings and re-entrant corners in **PC2** buildings involves providing adequate chord and collector members in the vicinity of the opening or corner. Rehabilitation methods discussed in Section 17.4.1 are applicable. Framing bays at ramps in parking garages may need to be treated as openings for purposes of diaphragm design.

## 17.4 Detailed Description of Techniques Primarily Associated with This Building Type

To date, very little rehabilitation of **PC2** buildings has occurred in the U.S. As a result, the following discussion of rehabilitation measures draws from limited available research, suggested details for new **PC2** construction, and application of rehabilitation techniques for concrete buildings to the specific configurations of precast elements.

### 17.4.1 Add or Enhance Collector or Chord in Existing Precast Diaphragm

***Deficiencies Addressed by Rehabilitation Technique***

This rehabilitation technique addresses deficient diaphragm boundary members – chords and collectors at diaphragm boundaries and at interior openings and re-entrant corners.

***Description of the Rehabilitation Technique***

In diaphragms with cast-in-place topping slabs, existing chords and collectors are likely to be reinforcing bars at the edge of the cast-in-place slab and in line with shear walls. The most practical rehabilitation technique is addition of structural steel sections or reinforcing bars at the diaphragm boundary locations. Where boundary members occur at the perimeter of parking structures, it may be possible to encase the steel sections or reinforcing in new concrete curbs on top of the existing deck, as shown conceptually in Figure 17.4.1-1. Where boundary members extend across the floor plate where foot or vehicle traffic will occur, alternate chord and collector locations are required, as shown conceptually in Figures 17.4.1-2 and 17.4.1-3. For both chords and collectors, shear transfer capability between the boundary member and the structural diaphragm needs to be provided. Adhesive anchors or reinforcing dowels are the most likely

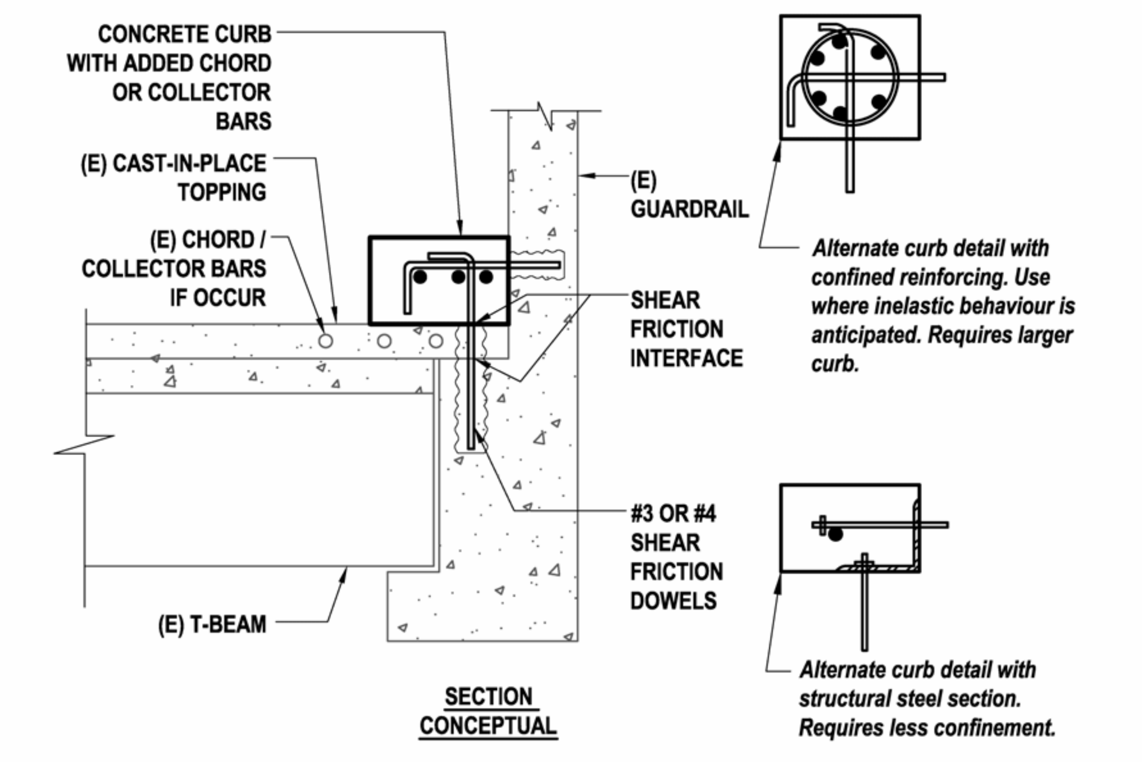


**Figure 17.4.1-1: Added or Enhanced Chord or Collector at Floor Perimeter with Cast-In-Place Topping Slab**

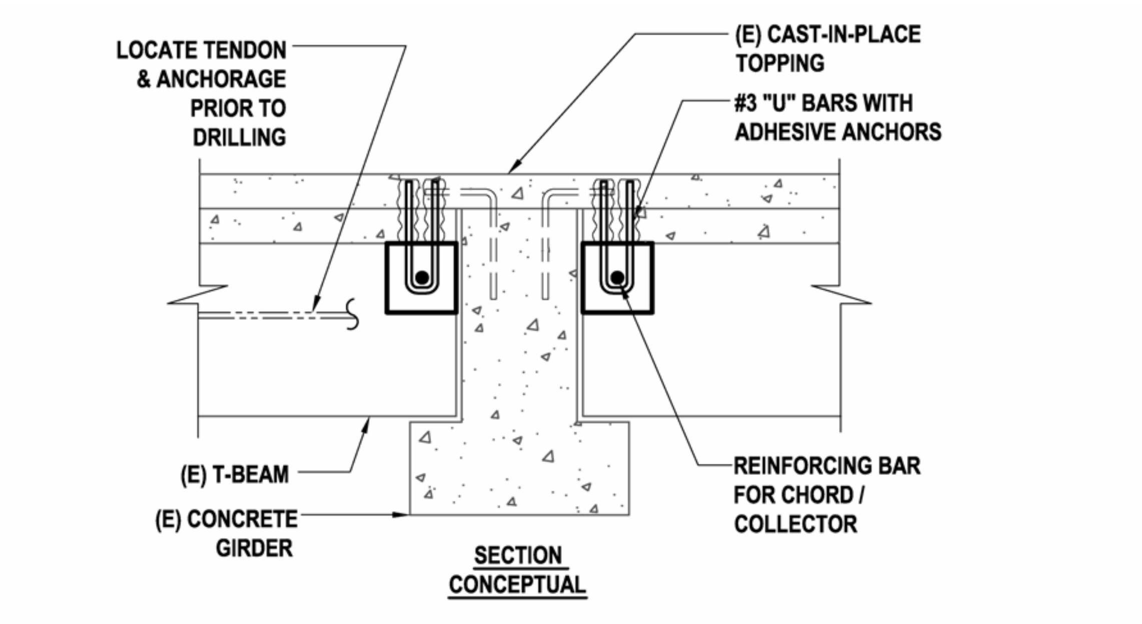


**Figure 17.4.1-2: Added or Enhanced Chord or Collector at Floor Interior with Cast-In-Place Topping Slab**

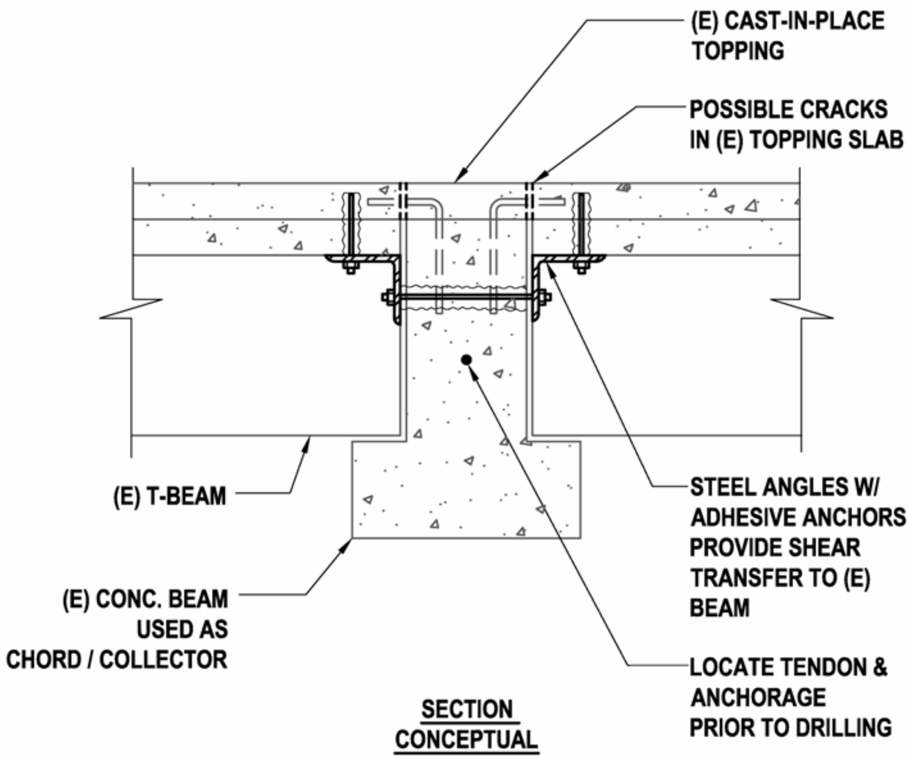


**Figure 17.4.1-3: Added or Enhanced Chord or Collector at Floor Interior with Cast-In-Place Topping Slab**

methods of attachment. Chord and collector splices must also be detailed. For collectors, transfer of the collector force to the vertical elements of the lateral force-resisting systems is required. Figure 17.4.1-4 shows a conceptual approach where the chord/collector curb runs by the face of the shear wall and dowels in over the full shear wall length. Figure 17.4.1-5 illustrates considerations when collector steel is to be doweled into the end of an existing shear wall, which may not be advisable.

In diaphragms without cast-in-place topping slabs, reinforcing in wall panels, floor panels, or beams will likely serve as chord and collector reinforcing. Where these floor panels or beams are precast, the connection between members is likely to be the weak link in chord and collector capacity. Figures 17.4.1-6 and 17.4.1-7 show the concept of added steel angles used as chords and collectors. The steel angle also serves as the connection between the wall and diaphragm for in-plane and out-of-plane forces.

***Design Considerations***

*Research basis:* No research applicable to this rehabilitation measure has been identified.

The objective of this rehabilitation method is to enhance the ability of the diaphragm to perform adequately and to deliver forces to the vertical elements of the lateral force-resisting system. In

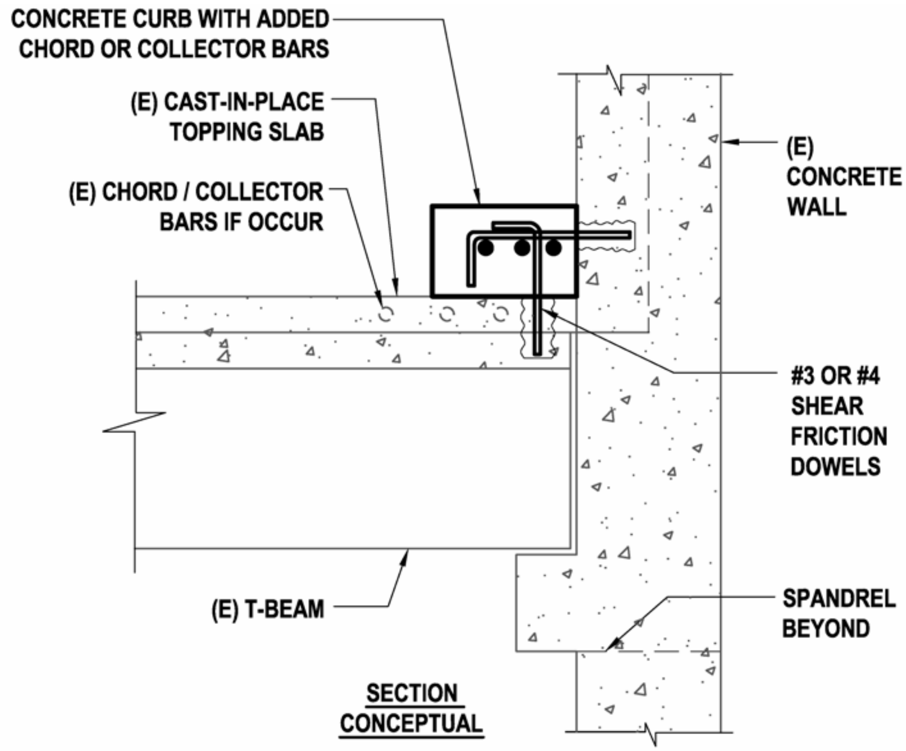


**Figure 17.4.1-4: Added Collector Anchorage to Shear Wall with Cast-In-Place Topping Slab**

order to achieve this, careful consideration of the existing diaphragm configuration and shear strength is needed in order to determine what behavior can be achieved and what detailing approach is best. In long-span diaphragms, yielding of a chord member may be a preferred behavior in order to protect against in-plane shear failure. Where this is true, it should be anticipated that the chord member is likely to elongate in one or more locations under tension loading, opening up gaps between adjacent precast members. When loading reverses, compression will be carried by the chord member until gaps close. In order to prevent local buckling failure of the chord, it is advisable to either use a structural steel section that can be adequately braced against buckling with a reasonable adhesive anchor spacing, or provide confinement of reinforcing bars, as would be provided for a concrete column. These alternatives are shown at the right hand side of Figure 17.4.1-1. Both of these alternatives will be more costly and difficult to install. In diaphragm configurations where yielding of the chord and collector members can be avoided, providing additional reinforcing may be less costly than detailing for buckling restraint. Where this approach is taken, a careful evaluation of anticipated forces is needed. In general, it is the philosophy of ASCE 7 seismic provisions for inelastic behavior to be focused in vertical elements rather than collectors, allowing the vertical elements of the seismic force-resisting system to control building dynamic behavior. This may not be achievable in diaphragms where the collector serves as a chord for loading in the perpendicular direction, and avoiding shear failure is paramount. In this case, use of a detail that restrains buckling is recommended.

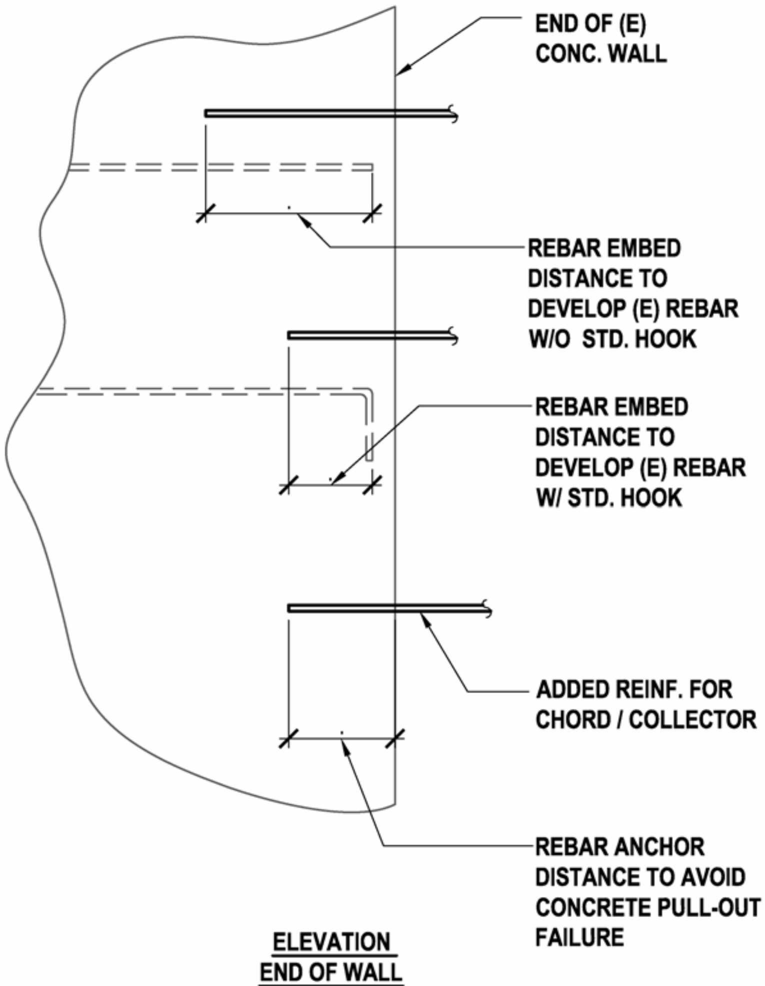


**Figure 17.4.1-5: Rebar Embedment Considerations in Collector Anchorage to Existing Shear Wall**

For the chord configurations shown in Figure 17.4.1-6, the existing original connection (if any) likely consists of intermittent cast-in embed plates and a welded plate connection. The capacity of anchors embedded in the concrete and welds would have been sized to meet load requirements (wind or earthquake) applicable at time of construction. Even if earthquake loading were considered, the need to allow for forces in excess of design levels, ductility, and energy dissipation likely would not have been considered. Because inelastic behavior in the precast walls and diaphragms is very unlikely, it must be anticipated that inelastic behavior will concentrate in the connections between members. With many existing connections, evaluation would likely identify failure of the anchors embedded in concrete as the weak link. This is an undesirable weak link due to lack of ductility. Unless extreme overstrength has been provided, to allow the connection to remain elastic, rehabilitation of the connections is needed in order to avoid this weak link. ACI 318 Appendix D (ACI, 2005) provisions for anchorage to concrete require that that design be governed by tensile or shear strength of a ductile steel element rather the concrete capacity. This requirement is particularly appropriate for rehabilitation of connections in precast wall buildings.

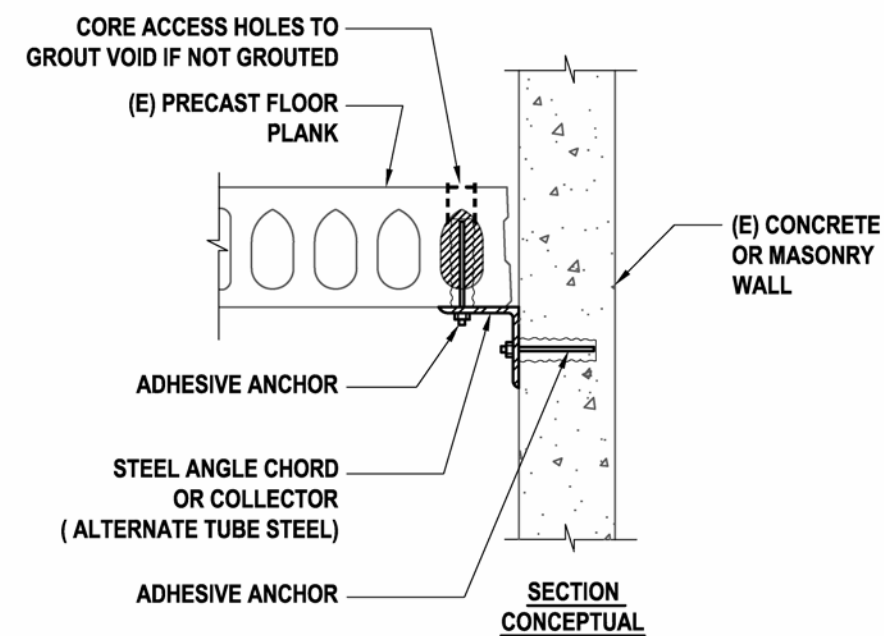


**Figure 17.4.1-6: Steel Chord or Collector at Floor Perimeter without Cast-In-Place Topping**

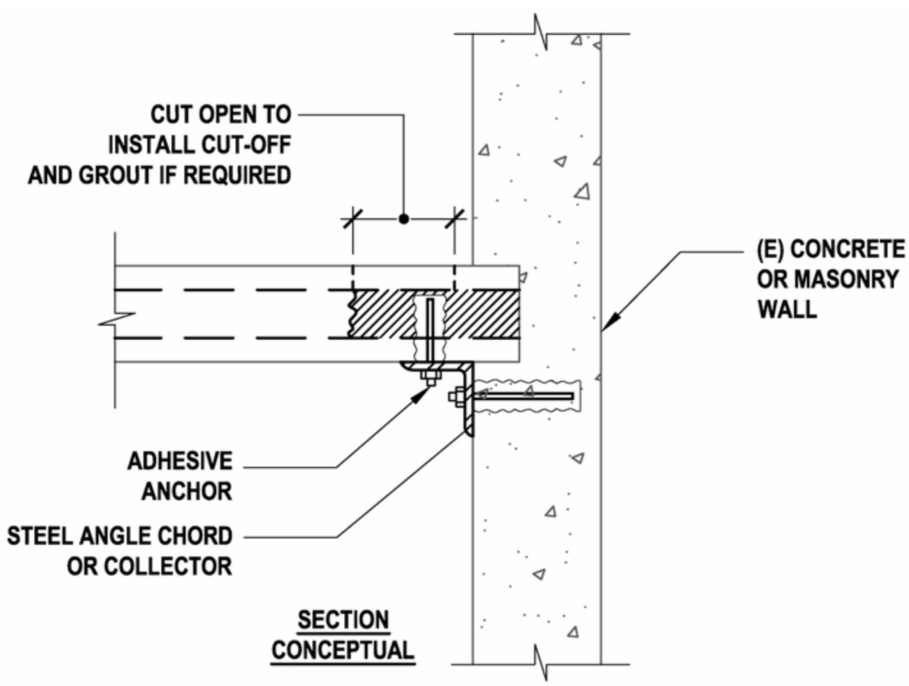


**Figure 17.4.1-7: Steel Chord or Collector at Floor Perimeter without Cast-In-Place Topping**

The deformation compatibility of the existing and added connections needs to be carefully considered. It may be desirable to design new connections to carry all forces, neglecting the contribution of the existing connections. In some cases it might even be desirable to cut the weld on the original connection to ensure that it will not carry load.

### *Detailing Considerations*

Use of reinforcing dowels to anchor new chord or collector reinforcing to existing concrete, as shown in Figures 17.4.1-1 and 17.4.1-2, relies on shear friction. Roughening of the existing concrete to the amplitude required by ACI 318 requires a very heavy sandblasting or chipping with a jackhammer, in order to expose the aggregate. This is expensive and messy. It is often preferable to use a reduced $\mu$ coefficient of less than 1.0 and add more dowels, rather than roughening the surface. Removal of finishes and cleaning of the concrete surface is still required. Use of shear friction also requires that the yield capacity of the dowel be developed on both sides of the joint. This forces use of smaller dowels and a curb dimension adequate to develop standard hooks per ACI 318 requirements. Adhesive anchor embedment requirements for development of the bar yield are generally available from the manufacturer.

Figure 17.4.1-2 places new chord or collector reinforcing at the underside of the floor so that vehicle or foot traffic above is not disrupted. Where chord or collector reinforcing runs parallel to the T-beams, use of reinforcing bars is likely feasible. Where the chord or collector runs perpendicular, reinforcing bars would have to be installed in short lengths threaded thru the T-beam webs. An unstressed tendon that can be more easily placed might be a preferable alternative.

Figure 17.4.1-3 uses the existing girder as the chord or collector member and adds angles for shear transfer between the slab and collector. This must be used in combination with enhanced connections at the girder ends. The figure illustrates the possible pre-earthquake cracks in the topping slab described by Wood, Stanton and Hawkins (2000). The shear transfer angles bypass this potentially weak location. Where steel splice angles are used, splice details will be required. A field welded splice would be common in concrete rehabilitation. Where yielding of the steel chord/collector member can be anticipated, it may be desirable to proportion the splice plates and welds to develop the anticipated strength of the chord/collector member.

Figure 17.4.1-5 illustrates considerations when collector steel is to be doweled into the end of an existing shear wall. This is only possible when there is adequate reinforcing within the wall in the vicinity of the collector anchorage to distribute the collector force over the wall length. Anchoring two No. 9 bars to the end of walls with only two No. 4 bars should not occur. Further, it is important that the collector lap adequately with reinforcing in the shear wall that can distribute the collector force over the wall length, as shown in the upper two figures. If embedment were limited to the requirements of the anchor manufacturer, as shown in the bottom figure, failure of the collector may occur due to inability of the wall reinforcement to develop.

### *Cost/Disruption and Construction Considerations*

The work required to add or enhancement chords and collectors is generally spread out over considerably over the building area. This distribution of work is reasonably easy in parking garages because there are generally no finishes to remove and replace, all areas are reasonably accessible for materials and equipment, and user relocation does not involve a big effort. In contrast, the distribution of work will create significantly greater cost and coordination in other building types such as commercial or residential. In these building types, it may well be easier and more cost effective to add vertical elements, rather than enhancing chords and collectors, because this work may be located in one or more local areas.

***Proprietary Concerns***

There are no proprietary concerns with this rehabilitation technique other than the use of proprietary anchors as part of the assemblage.

### 17.4.2 Enhance Connections Between Existing Precast Diaphragm, Shear Wall and Foundation Elements

***Deficiency Addressed by Rehabilitation Technique***

This rehabilitation technique addresses rehabilitation of inadequate connections between precast concrete elements including: diaphragm to shear wall, shear wall above to shear wall below, shear wall to foundation, and shear wall panel to panel connections.

***Description of the Rehabilitation Technique***

Existing connections of precast wall and diaphragm elements are generally either welded connections between cast-in embed plates, or connections involving mechanical, grouted or welded rebar splices. Where precast elements are combined with cast-in-place elements, Figures 17.4.2-1 and 17.4.2-2 show connections between precast wall panels and foundation and diaphragms. The connections depict one possible configuration for existing welded connections and rehabilitation approaches for enhancing connection shear capacity. The rehabilitation measures involve adding new steel angles and adhesive anchors as required to carry seismic forces. Figures 17.4.2-1B shows added anchorage to a hollow core wall section using grout, which provides a high connection capacity. Alternately an anchor specifically designed for attachment to hollow masonry could be used, resulting in a lower connection capacity.

***Design Considerations***

*Research basis:* FRP composite connections between wall panels have been tested at the University of Utah (Pantelides, Volnyy, Gergeley, and Reaveley, 2003).

Existing precast shear wall connections most often use cast-in embed plates and welded plate connections. The capacity of anchors embedded in the concrete and welds would have been sized to meet load requirements (wind or earthquake) applicable at time of construction. Even if earthquake loading were considered, the need to allow for forces in excess of design levels, ductility and energy dissipation likely would not likely have been considered. Because inelastic behavior in the precast walls and diaphragms is very unlikely, it must be anticipated that inelastic behavior will concentrate in the connections between members. With many existing connections, evaluation would likely identify failure of the anchors embedded in concrete as the weak link. This is an undesirable weak link due to lack of ductility. Unless extreme overstrength has been provided, allowing the connection to remain elastic, rehabilitation of the connections is needed in order to avoid this weak link. ACI 318 Appendix D (ACI, 2005) provisions for anchorage to concrete require that the design be governed by tensile or shear strength of a ductile steel element rather the concrete capacity. This requirement is particularly appropriate for rehabilitation of connections in precast wall buildings. The deformation compatibility of the existing and added connections needs to be carefully considered. It may be desirable to design new connections to carry all forces, neglecting the contribution of the existing connections. In some cases, it might even be desirable to cut the weld on the original connection to ensure that it will not carry load.

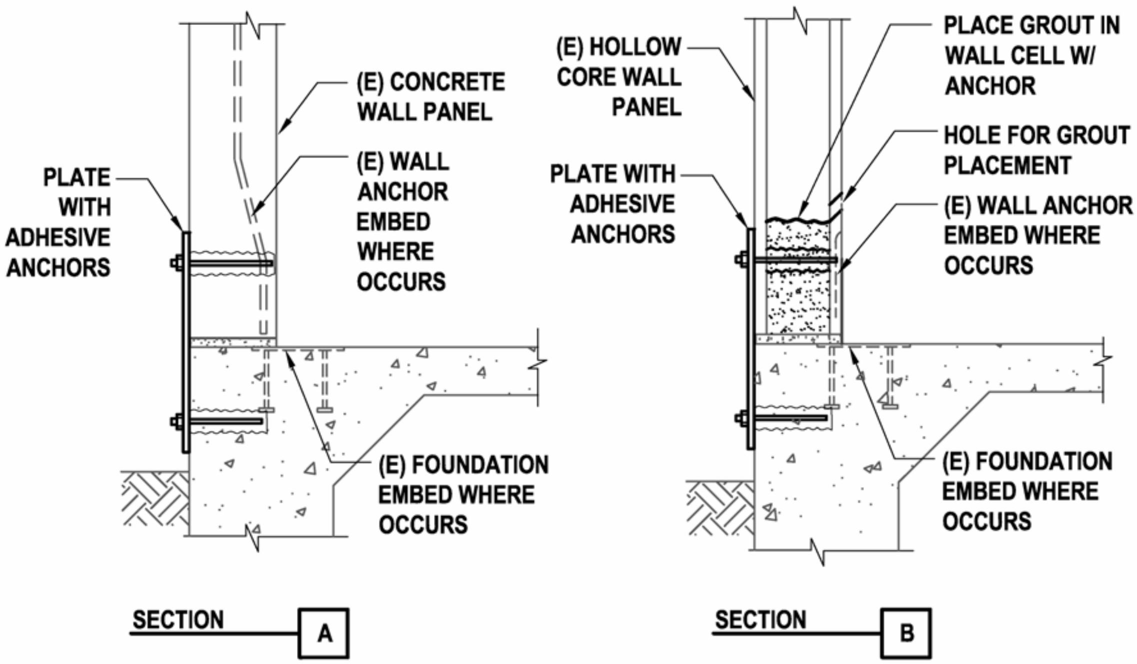


**Figure 17.4.2-1: Added or Enhanced Precast Wall-to-Foundation Connection**

Each precast wall panel tends to have two existing welded connections at the wall top and two at the wall bottom, and connections are generally not provided panel to panel. As a result, the two connections at the wall bottom must resist both shear and overturning forces from in-plane loads as well as out-of-plane loads. This is illustrated in Figure 17.4.2-3A. Where multiple wall panels are in line, as shown in Figure 17.4.2-3B, it may be possible to add panel to panel connections to resist overturning, reducing the demand on the bottom of panel connections. Overturning capacity will still be needed at each end of the panel group. The perpendicular wall panel at the right side of Figure 17.4.2-3 should not be counted on to resist the uplift, since it may also be overturning.

Figure 17.4.2-4 illustrates an FRP composite connection between wall panels developed and tested at the University of Utah (Pantelides, Volnyy, Gergeley, and Reaveley, 2003) that would be applicable to this use. While various surface preparations and FRP applications were investigated, the researchers settled on use of a high-pressure water jet preparation of the concrete surface to expose aggregate, a bonding agent and dry lay-up of carbon fibers, and saturation with an epoxy resin. A lay up of six layers of 12k (12,000 threads per tow) carbon fiber reinforcing in a 16 inch by 48 inch rectangle on one face of the wall panels provided failure loads on the order of 40 kips. The failures were sudden and brittle, indicating that design of this connection type should be considered force-controlled, and ductility should be provided in other connections or members. See Section 13.4.1 for general information on FRP composite overlays.

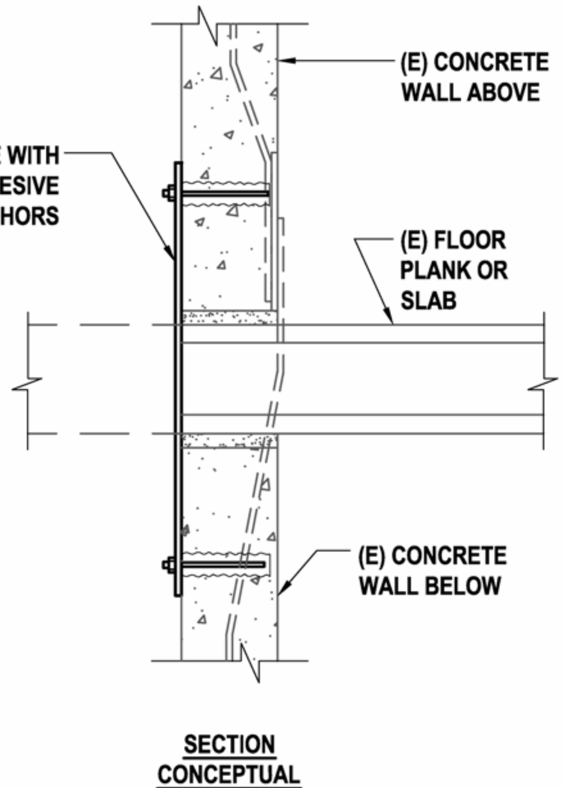


**Figure 17.4.2-2: Precast Wall Connection**

The new connections shown have both vertical and horizontal eccentricities that must be considered in design of the angle section and the anchors.

### *Detailing Considerations*

Anchorage to hollow core wall panels is more difficult than solid precast panels because the voids do not leave solid concrete sections large enough to meet embedment and edge distance requirements for adhesive anchors. For most anchorage, drilling grout access and inspection holes and filling the lower portion of the void with grout can provide a solid cell for anchor placement. Similar grouting of floor planks is sometimes used in new construction. For very light anchorage loads, it may be possible to use screen-tube adhesive anchors specifically designed for attachment to hollow masonry units. Along with low capacity, failure of this anchorage should be expected to have brittle behavior.

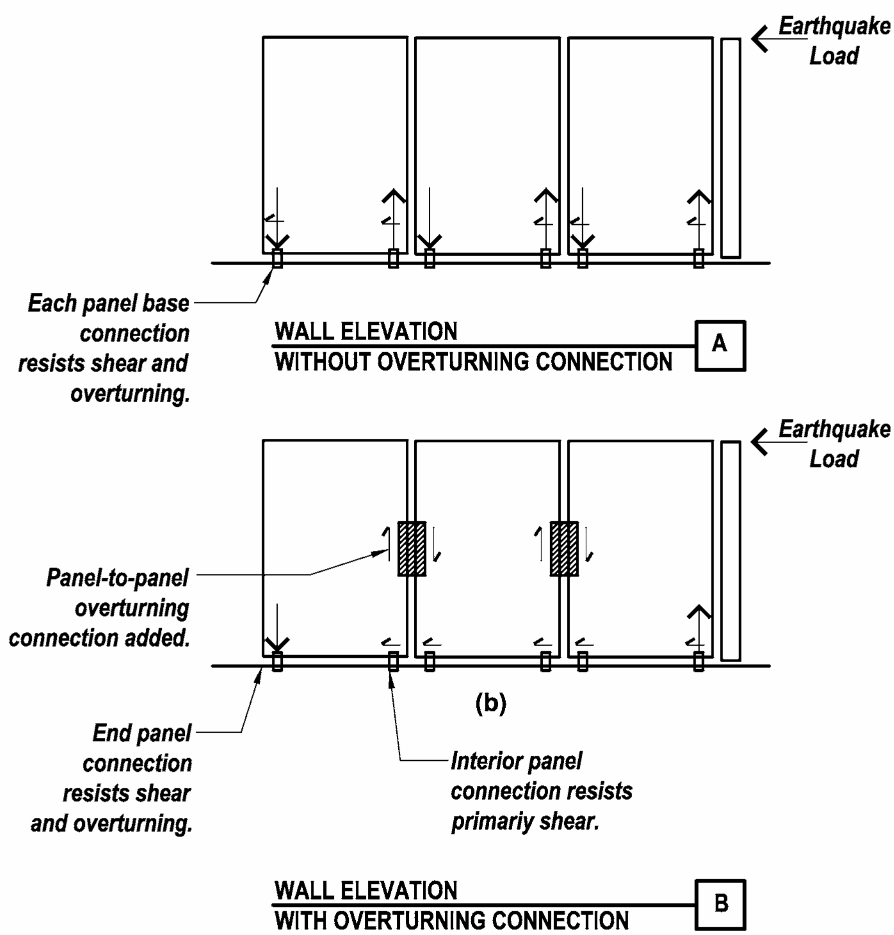


**Figure 17.4.2-3: Modification of Demand on Anchors Through Use of Panel-to-Panel Connections**

### *Cost, Disruption and Construction Considerations*

The cost of retrofitting connections in areas covered by finishes can be very expensive due to finish removal and replacement. The use of high-pressure water jets for surface preparation is not practical for an occupied building with finishes. It may be possible in an open building such as a parking garage.

### *Proprietary Concerns*

FRP systems are proprietary. Manufacturers should be contacted for appropriate uses and limitations. Proprietary adhesives are used as part of connection details.

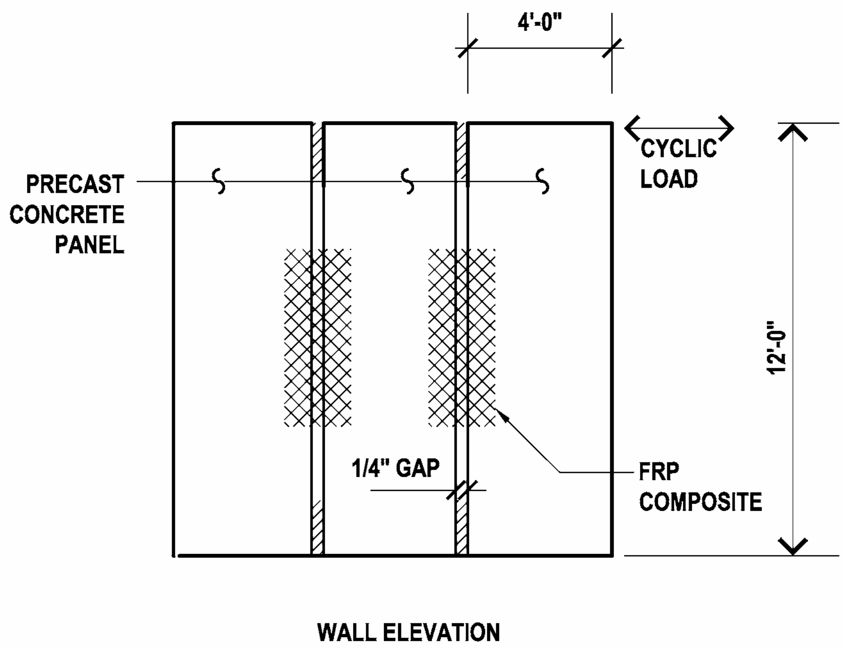


**Figure 17.4.2-4: Panel Setup for Testing of FRP Panel-to-Panel Connections**

### 17.4.3 Mitigate Configurations Creating Short Columns

***Deficiency Addressed by Rehabilitation Technique***

This technique addresses mitigation of unintended shortening and fixity of concrete columns.

***Description of the Rehabilitation Technique***

Figures 17.4.3-1, 17.4.3-2 and 17.4.3-3 illustrate unintended shortening and stiffening of columns, common in parking garages. The condition in Figure 17.4.3-1 is very easily mitigated by sawcutting the required gap between the column and the guardrail. The connection at the base of the guardrail may need to be improved as a result. The condition in Figure 17.4.3-2 can be improved by creating a hinge or joint in the column at the top of slab level. This will require shoring to take the load off of the column during modification, and the development of a shear and tension connection to the pedestal and footing. The condition in Figure 17.4.3-3 is difficult to address. Columns with this configuration are not easily confined. A solution that can minimize column shortening for new construction is to provide separate columns for the ramp and level floors. This approach could also be attempted for a retrofit, but would be costly.

***Design Considerations***

*Research basis:* No research applicable to this rehabilitation measure has been identified.

***Proprietary Concerns***

There are no proprietary concerns with this rehabilitation technique.

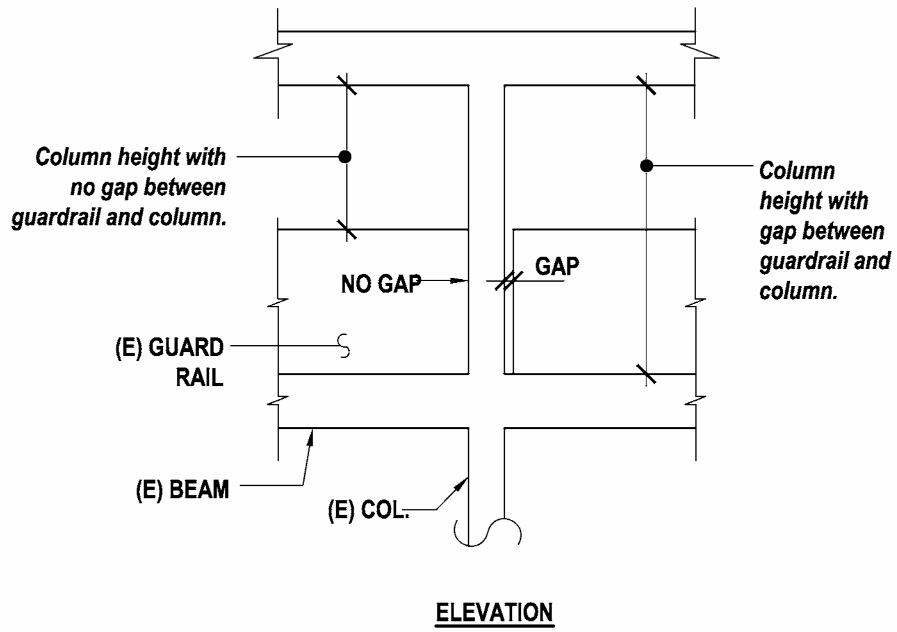


**Figure 17.4.3-1: Accidental Reduction of Column Height Due to Guardrail**

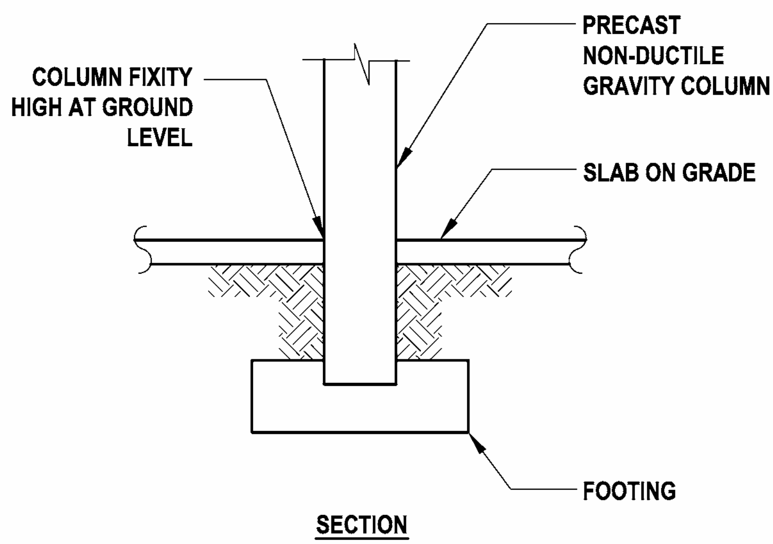


**Figure 17.4.3-2: Accidental Increase in Column Fixity Due to Embedment into Grade**

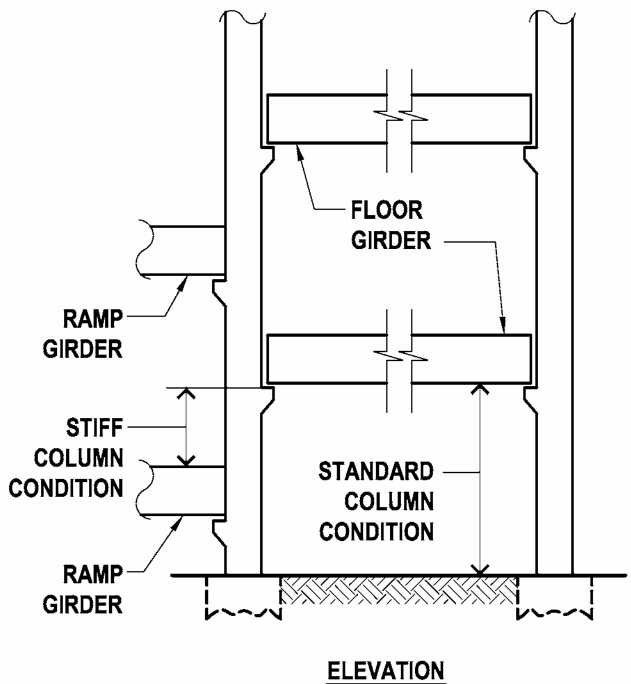


**Figure 17.4.3-3: Accidental Reduction of Column Height at Ramp**

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# Chapter 18 - Building Type RM1t: Reinforced Masonry Bearing Walls (Similar to Tilt-up Concrete Shear Walls)

## 18.1 Description of the Model Building Type

Building Type **RM1** is constructed with reinforced masonry (brick cavity wall or concrete masonry unit) perimeter walls with a wood or metal deck flexible diaphragm. For this document, Building Type **RM1** is separated into two categories. Chapter 19 describes **RM1u**, which is multistory, and typically has interior CMU walls and shorter diaphragm spans. This chapter covers **RM1t**, the large, typically one-story buildings with relatively open interiors that are similar to concrete tilt-ups. The exterior walls are commonly bearing, with an interior post and beam system of steel or wood. Older buildings of this type are generally small and used for a wide variety of occupancies. Recently, the building type has become commonly used for one-story warehouse and wholesale/retail occupancies similar to tilt-up (Building Type **PC1**) buildings. Figure 18.1-1 illustrates one example of the **RM1t** Building Type.

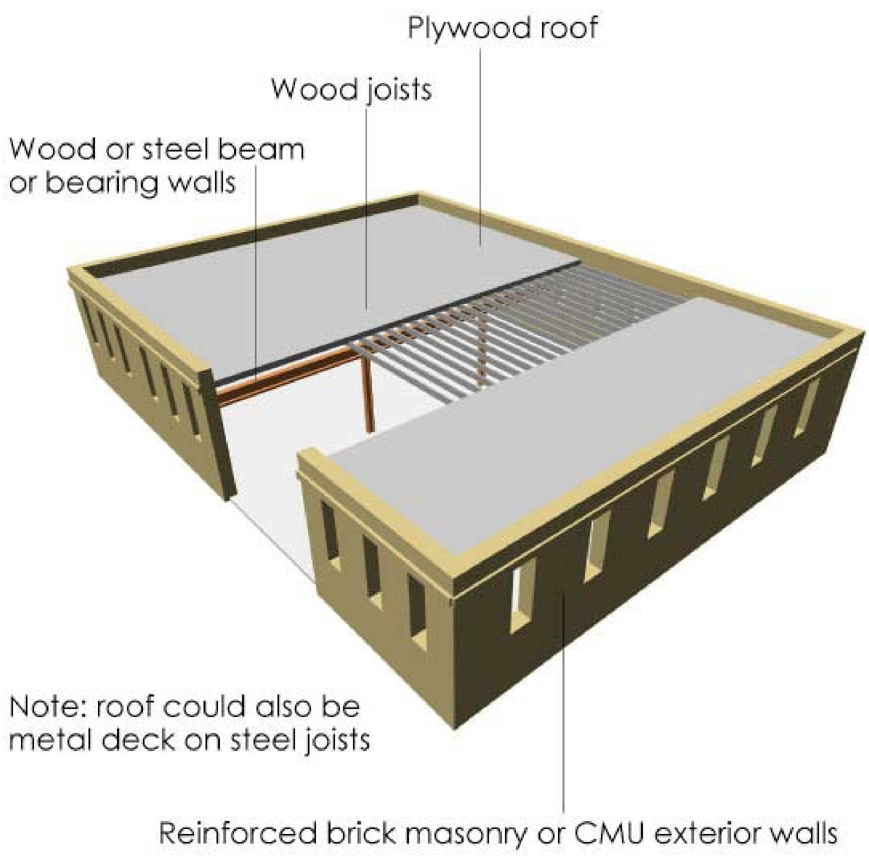



**Figure 18.1-1: Building Type RM1t: Reinforced Masonry Bearing Walls**

*Guidelines for Seismic Evaluation and Rehabilitation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures* or *SEAONC Guidelines* (SEAONC, 2001) provides a substantial collection of information on flexible diaphragm / rigid wall building issues, the West Coast experience with earthquake performance, rehabilitation priorities, and techniques for rehabilitation. This document was a primary source of information for Chapter 16 (**PC1** buildings), and it is also recommended for **RM1t** buildings.

### *Walls*

Exterior reinforced masonry walls are the primary vertical elements in the lateral load-resisting system. Buildings with large plan areas may have interior walls providing additional lateral resistance; however, this is not common. Like tilt-up construction, masonry walls have recently transitioned from use of code-prescribed height to thickness (*h/t*) limits to use of much higher *h/t* ratios, in combination with rational analysis of slenderness effects. Most existing construction, however, will have been designed using prescriptive ratios and allowable stress design methods.

Reinforced masonry walls in **RM1t** buildings share many issues with **PC1** buildings, discussed in Chapter 16. Two distinctive aspects of reinforced masonry walls require discussion: movement control joints and partially grouted masonry.

The masonry industry recommends providing vertical control joints in new masonry wall construction to accommodate masonry wall shrinkage and thermal movement. The currently recommended maximum spacing of control joints is the lesser of 1.5 times the wall height or 25 feet (CMACN, 2003). The inclusion of and spacing of control joints varies significantly, however, with the age and region of the building. Wall-to-diaphragm anchorage remains a primary concern irrespective of whether control joints are provided. Where vertical control joints occur, diaphragm chords and collectors will be provided by either horizontal reinforcing that continues across the control joint (typically provided at the diaphragm level only), or a steel angle or similar member on the face of the masonry. Rehabilitation of chords, collectors, and shear transfer for **RM1t** building will be much the same as **PC1** buildings. Masonry wall construction without control joints will not have to rely as much on discreet chords and collectors; adequacy of shear transfer may still be a focus of rehabilitation.

In areas of high seismic hazard, it is most common for masonry walls to be fully grouted. In many other areas, however, grout is only provided at required reinforcing. A typical reinforced masonry wall in a **RM1t** building might have vertical reinforcing and grout alongside window and door openings, and at between four and ten feet on center horizontally and vertically in the piers and spandrels. Partial grouting has a significant effect on the weight of the wall and calculated seismic forces, as well as wall strength for both in-plan and out-of-plane forces. Where partial grouting has been provided, grout locations need to be known in order to design wall to diaphragm anchorage.

### *Gravity Load-Carrying Support at the Building Perimeter*

It is most common for wood girders to be supported on the building exterior walls in **RM1t** buildings. As in **PC1** buildings, girder connections to the exterior walls are required to resist wall out-of-plane loading in addition to gravity loads. Connections of girders to the exterior walls require evaluation and possible rehabilitation. Section 16.4.2 discusses applicable rehabilitation measures. Where the existing masonry is partially grouted, it may be necessary to open up the masonry face and grout at new anchorage locations. Where this is done, cast-in anchors can be provided in lieu of adhesive anchors.

#### *Roof Diaphragms*

Like the **PC1** building, the roof system will generally be either of wood or steel construction. In the western states, roof systems are almost exclusively wood sheathed. Outside of the western states, roof systems are almost exclusively sheathed with steel decking topped with rigid insulation or vermiculite concrete. For both the wood and the steel roof systems, the roof diaphragm in the **RM1t** building is almost always flexible compared to the walls. See Chapter 16 for additional discussion.

#### *Wall-to-Diaphragm Connections*

Like **PC1** buildings, wall-to-diaphragm connections are thought to be the aspect of **RM1t** buildings most vulnerable to earthquake damage, due to the significant force and deformation demands imposed on this connection. The wall-to-diaphragm connections are therefore recommended as the first focus of rehabilitation measures for one-story **RM1t** buildings. See Chapter 16 for a discussion of past performance of these connections in tilt-up buildings. Similar to tilt-up buildings, even in **RM1t** buildings constructed or upgraded recently, it should be assumed that these connections require review and possible rehabilitation.

#### *Interior Additions*

Mezzanines and interior second stories, commonly constructed within large box-like **RM1t** buildings, can restrain building movement under earthquake loading, resulting in unintended load paths and damage. See Chapter 16 for discussion.

#### *Foundations*

**RM1t** buildings are generally constructed on continuous perimeter footings, with dowels to the reinforced masonry walls at vertical reinforcing locations.

## 18.2 Seismic Response Characteristics

**RM1t** buildings, like **PC1** buildings, are distinguished by rigid shear walls and flexible diaphragms. Like **PC1** buildings, amplification of seismic forces near the center of the diaphragm is of concern for both diaphragm capacity and wall anchorage capacity. The in-place construction of the reinforced masonry walls may provide wall continuity that makes **RM1t** buildings somewhat less vulnerable to partial collapse than **PC1** buildings; however, the potential for significant performance problems exists.

## 18.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

Unlike **PC1** buildings, damage to **RM1t** buildings has not been noted as significant or wide spread. In the 1994 Northridge earthquake, little in the way of damage to **RM1t** buildings was reported (EERI, 1996; and Klingner, 1994). One item of interest was damage to masonry walls at building corners near the roof line, attributed to interaction between the masonry wall and flexible wood diaphragm. See below for general discussion and Table 18.3-1 for a detailed

**Table 18.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for RM1t Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| Global Stiffness | | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.2] | Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| Configuration | Torsionally irregular plans | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Concrete wall overlay [21.4.5]<br>Infill openings | | | |
| | Re-entrant corners | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8]<br>Collector [7.4.2] | Enhance existing collector<br>Concrete wall overlay [21.4.5]<br>Infill openings | Collector [7.4.2] | | |
| | Incidental bracing | | | | | Separate component from incidental bracing |
| Load Path | Inadequate or missing wall-to-diaphragm tie for out-of-plane load | | | Wall-to-diaphragm tension anchors plus subdiaphragms and cross-tie [16.4.1] | | |
| | Inadequate anchorage to diaphragms for in-plane forces | | | Wall-to-diaphragm shear anchors [21.4.2] | | |
| | Inadequate collectors | Add collector [7.4.2] | Improve collector member and connections | | | |

**Table 18.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for RM1t Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing | Wall inadequate for out-of-plane bending | | Wall strongback or pilaster [21.4.3] | | | |
| | Inadequate detailing of slender walls | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Concrete wall overlay [21.4.5]<br>Infill openings<br>Add backup vertical supports where bearing might be lost [21.4.11] | | | |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8] | Enhance existing diaphragm [22.2.1]<br>Horizontal braced frame [21.2.10] | | | |
| | Inadequate chord capacity | Enhance chord [22.2.2] | Enhance chord [22.2.2] | | | |
| | Excessive stresses at openings and irregularities | | Enhance diaphragm detailing | | | |
| | Re-entrant corners | | Enhance diaphragm detailing | | | |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation measures. | | | | | | |

compilation of common seismic deficiencies and rehabilitation techniques for Building Type **RM1t**. See also Chapter 16 for similar issues in **PC1** buildings.

### *Global Strength and Stiffness*

Global strength and stiffness are rarely a concern for large box-like **RM1t** buildings, but can be for smaller buildings that have very short walls along a street-front side. Rehabilitation of global strength and stiffness deficiencies will commonly involve adding new vertical elements, enhancing existing elements, or infilling openings in existing walls.

### *Configuration*

Poor distribution of shear walls can result in torsionally irregular behavior of **RM1t** buildings. Common occurrences include street-front walls in commercial buildings. The most direct approach to rehabilitation of this condition is the addition of strength and stiffness in line with the perforated wall. This can be accomplished through addition of new shear walls, enhancing existing shear walls, or addition of steel braced frames.

Rehabilitation at re-entrant corners requires the provision of adequate chords and collectors, shear transfer to the in-set wall panels, and possibly the strengthening of the wall panels and connections to the foundation. The *SEAONC Guidelines* suggest that there may be diaphragm continuity over this interior diaphragm support, increasing the diaphragm reaction to the in-set wall line. See Chapter 16 for illustration and additional discussion.

### *Load Path*

As previously mentioned, load path connections between the masonry walls and the flexible diaphragm are suggested as the first focus of rehabilitation in **RM1t** buildings. Diaphragm cross-ties, as addressed in Chapter 22, are a required continuation of the wall anchorage system. Section 16.4.1 discusses applicable rehabilitation measures. Where the existing masonry is partially grouted, it may be necessary to open up the masonry face and grout at new anchorage locations. Where this is done, cast-in anchors can be provided in lieu of adhesive anchors. Connection between the wall and diaphragm may also be inadequate for in-plane shear loads.

The addition of or enhancement of existing collectors may be required in order to transmit diaphragm forces to the resisting shear walls. This is particularly of concern when a limited length of shear wall intended to carry a significant portion of the building shear. Although not as common, there is also significant concern when vertical offsets in the roof diaphragm result in incomplete chords or collectors. Any breaks or offsets in chords or collectors need to be carefully evaluated.

### *Component Detailing*

Component detailing deficiencies include inadequate out-of-plane wall capacity. Where existing walls are partially grouted concrete masonry, it may be possible to place additional vertical reinforcing and grout in ungrouted cells, accessed by cutting open face shells. Where this approach is taken, doweling of the vertically reinforcing at the top and bottom of the grouted cells would commonly be provided. In addition, the increase in wall weight should be considered in building seismic forces. Where existing masonry wall construction is solid, it is seldom practical to address wall capacity by adding reinforcing and concrete thickness to individual wall sections, so addition of wall pilasters or strongbacks is common. Where pilasters are added to masonry walls, pilaster-to-roof diaphragm anchorage must be provided to accommodate the concentration of wall out-of-plane force.

### *Diaphragm Deficiencies*

Due to changes in building code requirements, it is very common for diaphragms in areas of high seismic hazard to have inadequate in-plane shear capacity. These diaphragms may also have inadequate in-plane stiffness due to high unit shear stresses. Regardless of this, the *SEAONC Guidelines* indicate that diaphragm overstresses have rarely been associated with significant earthquake damage. Diaphragm strength and stiffness deficiencies are most often rehabilitated by enhancing the existing diaphragm.

Other diaphragm deficiencies include inadequate chord capacity and stress concentrations at large diaphragm openings and re-entrant corners. Rehabilitation at re-entrant corners primarily involves the provision of adequate chords and collectors. The same is true at large diaphragm openings.

## 18.4 Detailed Description of Techniques Primarily Associated with This Building Type

No techniques have been developed for this building type. See other chapters for detailed descriptions of relevant rehabilitation techniques.

## 18.5 References

CMACN, 2003, "Movement Control Joints," *Masonry Chronicles Winter 02-03*, Concrete Masonry Association of California and Nevada, Citrus Heights, CA.

EERI, January 1996, *Northridge Earthquake of January 17, 1994, Reconnaissance Report, Volume 2, Earthquake Spectra*, Supplement C to Volume 11, Earthquake Engineering Research Institute, Oakland, CA.

Klingner, R., 1994, *Performance of Masonry Structures in the Northridge California Earthquake of January 17, 1994* (Technical Report 301-94), The Masonry Society, Boulder, CO.

SEAONC (Structural Engineers Association of Northern California), 2001, *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures*, Structural Engineers Association of Northern California, San Francisco, CA.

# Chapter 19 - Building Type RM1u: Reinforced Masonry Bearing Walls (Similar to Unreinforced Masonry Bearing Walls)

## 19.1 Description of the Model Building Type

Building Type **RM1** takes a variety of configurations, but they are characterized by reinforced masonry walls with flexible diaphragms such as wood or metal deck. The walls are commonly bearing, but the gravity system also contains post and beam construction of wood or steel in interior or some façade locations. For this document, Building Type **RM1** is separated into two categories. Chapter 18 describes **RM1t**, the large, typically one-story buildings with relatively open interiors that are similar to concrete tilt-ups. This chapter covers **RM1u**, which is multistory, and typically has interior CMU walls and shorter diaphragm spans. It is similar to Building Type **URM** (Chapter 21) and has many of the same deficiencies. Figure 19.1-1 shows an example of this building type.

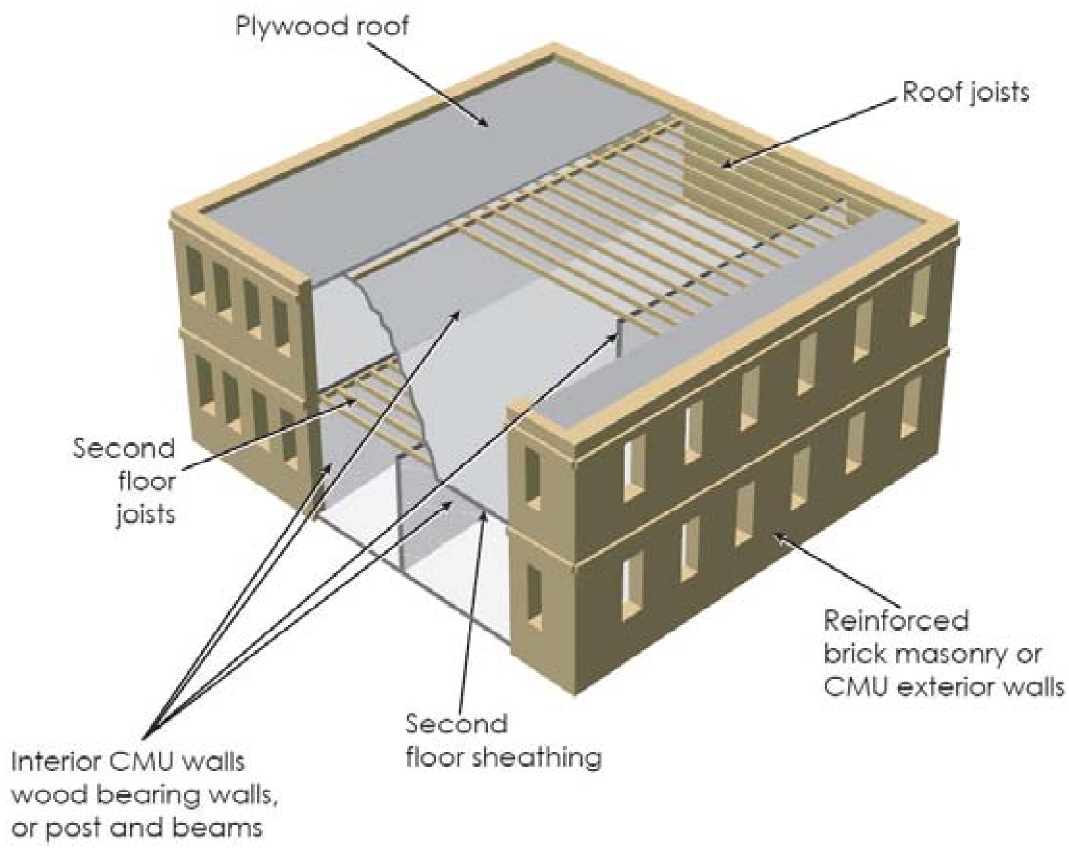



**Figure 19.1-1: RM1u Building Type: Reinforced Masonry Bearing Walls**

### *Masonry Wall Materials*

FEMA 306 (FEMA, 1999) identifies several common reinforced masonry wall types. These are:

Fully-grouted hollow concrete block
Partially-grouted hollow concrete block
Fully-grouted hollow clay brick
Partially-grouted hollow clay crick
Grouted-cavity wall masonry (two wythes of clay brick or hollow units with a reinforced grouted cavity)

Brick veneer facing may be placed on the exterior façade with the above walls used as backing walls.

### *Floor and Roof Diaphragm*

Floor and roof diaphragm construction is similar to those of Building Type **URM**, although unfilled metal deck diaphragms can be found at the roof and occasionally at floors. See Chapter 21.

### *Foundations*

Foundations for Building Type **RM1u** are typically spread footings at interior columns and strip footings under masonry bearing walls. Footings are typically concrete.

## 19.2 Seismic Response Characteristics

As a flexible diaphragm, stiff wall structure Building Type **RM1u** is expected to have dynamic behavior similar to that described for Building Type **URM**. See Chapter 21. Since the walls are reinforced, however, in-plane and out-of-plane wall behavior modes are substantially different from those of unreinforced masonry walls. They are instead more similar to those of reinforced concrete. FEMA 306 provides the most comprehensive categorization of reinforced masonry wall behavior modes.

## 19.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

**RM1u** buildings, while similar to **URM** buildings, are generally considered to be less hazardous. In-plane damage to reinforced masonry walls is much less likely to reach levels compromising life safety. Parapets can still be overstressed, but the risk to life safety is much less than those of unreinforced parapets, and out-of-plane failures of the walls spanning between diaphragms are relatively unlikely. The most significant risk to loss of life is due to inadequate connections between the walls and diaphragms. See below for general discussion and Table 19.3-1 for a detailed compilation of common seismic deficiencies and rehabilitation techniques for Building Type **RM1u**.

### *Global Strength*

As shear wall buildings, global strength in **RM1u** buildings is dependent on the in-plane shear capacity of the walls. Relatively large seismic forces are needed to lead to life safety concerns

**Table 19.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for RM1u Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane wall strength | Wood structural panel shear wall [5.4.1], [6.4.2]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6]<br>Grouting<br>Infill openings [21.4.7] | | Seismic isolation [24.3] | |
| Global Stiffness | | | | | | |
| Configuration | Soft story, weak story, excessive torsion | Wood structural panel shear wall [6.4.2]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9] | | | | |
| Load Path | Inadequate or missing wall-to-diaphragm tie | | | Tension anchors [16.4.1]<br>Shear anchors [21.4.2]<br>Subdiaphragms and cross-ties [22.2.3] | | |
| | Missing collector | Add collector [7.4.2] | | | | |
| | Inadequate girder-to-column connection | | | Improve connection<br>Supplemental vertical supports [21.4.11] | | |
| | Inadequate wall-foundation dowels | | | Wall-to-foundation improvements | | |

| Table 19.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for RM1u Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing | Wall inadequate for out-of-plane | | Exposed interfloor wall supports [21.4.3]<br>Reinforced cores [21.4.4]<br>Concrete wall overlays [21.4.5]<br>Fiber composite overlays [21.4.6] | | | |
| | Poorly anchored veneer or appendages | | Add ties | | | Remove veneer or appendages |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Wood structural panel shear wall [6.4.2]<br>Steel braced frame [7.4.1]<br>Concrete/masonry shear wall [21.4.8]<br>Add wood structural panel or moment frame crosswall [21.4.10]<br>Horizontal braced frame [22.2.9] | Enhance existing diaphragm [22.2.1]<br>Enhance crosswall [21.4.10] | | | |
| | Inadequate chord capacity | Add steel strap or angle | | | | |
| | Excessive stresses at openings and irregularities | Add wood or steel strap reinforcement | | | | |
| | Re-entrant corner | Wood structural panel shear wall [6.4.2]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9] | | Collector [7.4.2] | | |
| Foundations | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

with in-plane wall behavior, though cracking damage will occur in relatively moderate events. When walls are found to be deficient, new vertical lateral force-resisting elements can be added at interior locations or existing walls can be enhanced.

### *Global Stiffness*

Reinforced masonry walls are generally quite rigid, even if punctured with window openings, so global stiffness deficiencies are relatively uncommon.

### *Configuration*

Many commercial **RM1u** buildings will have a fairly open street façade at the ground level, leading to a weak and soft first story and torsional irregularities. This is usually addressed by the addition of a moment frame at the façade or another vertical lateral force-resisting element at some distance back from the façade.

### *Load Path*

As noted above, it is the lack of adequate ties between the walls and diaphragms that is the single most significant deficiency in **RM1u** buildings. Rehabilitation measures include tension ties for out-of-plane forces and shear ties for in-plane forces along the typical wall-diaphragm interface. In many reinforced masonry walls, pilasters are formed by thickening the wall in order to provide support of key girder lines. They may not have adequate bearing length for the girder seat or sufficient reinforcement at the top of the pilaster. Anchor bolts from the ledger to the wall are likely to be present, but cross-grain bending under out-of-plane tension loading will be a common deficiency. Rehabilitation measures include supplemental column supports and connection enhancements.

### *Component Detailing*

When brick veneer is present, it may not be adequately anchored back to the backing masonry, creating a falling hazard. Veneer ties can be added.

### *Diaphragm Deficiencies*

Wood diaphragms may lack both strength and stiffness in **RM1u** buildings. This can be addressed by adding new interior elements to cut the diaphragm span or by enhancing the diaphragm itself with wood structural panel overlays.

### *Foundation Deficiencies*

Foundation deficiencies for existing elements are relatively uncommon in **RM1u** buildings. Foundation rehabilitation work usually is focused on the support for new lateral force-resisting elements that are added to the superstructure.

## 19.4 Detailed Description of Techniques Primarily Associated with This Building Type

Rehabilitation techniques for Building Type **RM1u** are typically similar to those used in **URM** buildings. See Chapter 21 for detailed descriptions of techniques. When metal deck floors are present, techniques in Chapter 16 provide examples of rehabilitation methods for connecting the metal deck to the tilt-up concrete walls; details for connecting to reinforced masonry walls are similar.

## 19.5 References

FEMA, 1999, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings: Basic Procedures Manual*, FEMA 306, May.

# Chapter 20 - Building Type RM2: Reinforced Masonry Bearing Walls (Similar to Concrete Shear Walls with Bearing Walls)

## 20.1 Description of the Model Building Type

This building consists of reinforced masonry walls and concrete slab floors that may be either cast-in-place or precast. In this type of building, all walls usually act as both bearing and shear walls. The building type is similar and often used in the same occupancies as Building Type **C2b**, namely in mid- and low-rise hotels and motels. This system is also commonly used in residential apartment/condominium type buildings.

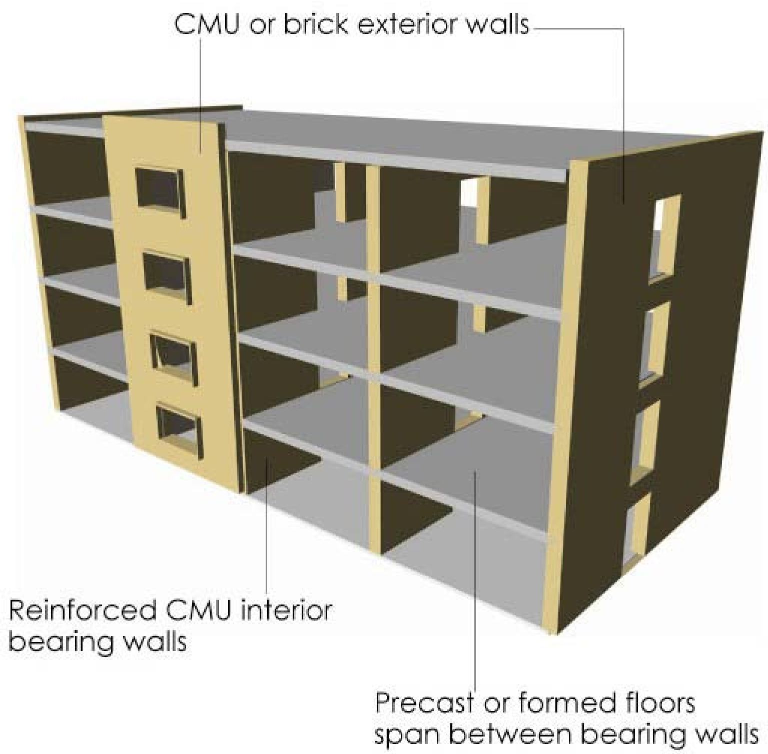



**Figure 20.1-1: RM2 Building Type: Reinforced Masonry Bearing Walls**

### *Variations Within the Building Type*

In order for this framing system to be efficient, a regular and repeating pattern of bearing walls are required to provide support points for the floor framing. In addition, since it is difficult and expensive to make significant changes in the plan during the life of the building, planning flexibility is not normally an important characteristic when this structural system is employed. The occupancy types that most often fit these characteristics are residential buildings, including dormitories, apartments, motels, and hotels. These buildings will often be configured with reinforced masonry bearing walls between rooms which act as shear walls in the transverse direction, and reinforced masonry walls on the interior corridor which act as shear walls in the longitudinal direction. Sometimes the longitudinal lateral system includes the exterior wall system, although this wall is normally made as open as possible. In any case, the wide variation in structural layouts and occupancies that is included in other shear wall buildings such as **C2f** is not seen in **RM2**.

It is seldom possible to plan a building layout that provides complete gravity support with walls, and often local areas are supported with isolated columns, sometimes with beams and girders. However, story heights in these buildings are usually small, and added depth in the floor framing system is difficult to obtain. The extent of such beam and column framing often causes confusion as to the classification of the structure as a bearing wall system. However, if significant plan area is supported solely by walls, the structures are normally classified as **RM2**.

There are important variations in floor framing systems employed in this building type, and their adequacy to act as a diaphragm is an important characteristic of this building type as discussed below.

### *Floor and Roof Diaphragms*

The parallel layouts of supporting walls and the need to minimize story heights normally leads to the use of one-way uniform-depth concrete floor systems. Cast-in-place and precast systems, both conventionally reinforced and prestressed, have been employed. The precast systems are often built up of narrow planks, which may not provide an adequate diaphragm unless a cast-in-place topping is provided. In addition, the precast systems may be placed with only a very narrow bearing area on the supporting walls, almost always on the outer masonry wythe or the CMU shell. When prestressed, the planks may be connected into the wall system only with the tail of the stressing tendon, and this connection may be inadequate to provide vertical support during seismic movements. The adequacy of the shear connection between slab and walls is also often an issue for both cast-in-place and precast systems.

### *Foundations*

The bearing walls obviously require some kind of starter beam at grade for construction purposes, and this often leads to a simple continuous grade beam system. In poor soils, piles or drilled piers may be added below the grade beam. A continuous mat foundation may also be employed due to the short spans and total length of bearing points in this building type.

## 20.2 Seismic Response Characteristics

Due to the extent of wall, bearing wall buildings will be quite stiff. Elastic and early post-elastic response will therefore be characterized with lower-than average drifts and higher-than-average floor accelerations. Damage in this range of response should be minimal.

Overall post-elastic response may often include rocking at the foundation level. If rocking does not occur, the height-to-length ratio of shear walls in these buildings may force shear yielding near the base, which may lead to strength and stiffness degradation.

Global stability may also be compromised by poor connections between floor slab construction and bearing walls.

### *Shear Wall Behavior*

When subjected to every increasing lateral load, individual shear walls or piers will first often force yielding in spandrels, slabs, or other horizontal components restricting their drift, and eventually walls and piers either rock on their foundations, suffer shear cracking and yielding, or form a flexural hinge near the base. Shear and flexural behavior are quite different, and estimates of the controlling action are affected by the distribution of lateral loads over the height of the structure.

Yielding of spandrels, slabs, or other coupling beams can cause a significant loss of stiffness in the structure. Flexural yielding will tend to maintain the strength of the system, but shear yielding, unless well detailed, will degrade the strength of the coupling component and the individual shear wall or pier will begin to act as a cantilever from its base. In this building type, the coupling elements are often slabs, and their lack of bending stiffness may reduce or eliminate significant coupling action.

Rocking is often beneficial, limiting the response of the superstructure. However, the amplified drift in the superstructure from rocking must be considered. In addition, if varying wall lengths or different foundation conditions lead to isolated or sequencing rocking, the transfer of load from rocking walls must be investigated. In buildings with basements, the couple created from horizontal restraint at the ground floor diaphragm and the basement floor/foundation (often termed the “backstay” effect) may be stiffer and stronger than the rocking restraint at the foundation and should be considered in those configurations.

Shear cracking and yielding of the wall itself are generally considered undesirable, because the strength and stiffness will quickly degrade, increasing drifts in general, as well as potentially creating a soft story or torsional response. However, in accordance with FEMA 356 (FEMA, 2000), shear yielding walls or systems can be shown to be adequate for small target displacements. Type **RM2** buildings will often fall into this category.

Flexural hinging is considered ductile in FEMA 356 and will degrade the strength of the wall only for larger drifts. Similar to rocking, the global effect of the loss of stiffness of a hinging wall must be investigated.

## 20.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

See Table 20.3-1 for deficiencies and potential rehabilitation techniques particular to this system. Selected deficiencies are further discussed below by category.

### *Global Strength*

Due to the extensive use of walls, buildings of this type seldom have deficiencies in this category, unless significant degradation of strength occurs due to shear failures.

| Table 20.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for RM2 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane wall shear strength | Concrete/masonry shear wall [12.4.2] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [13.4.1]<br>Steel wall overlay | | Seismic isolation [24.3]<br>Reduce flexural capacity [13.4.4] | |
| | Insufficient flexural capacity | Concrete/masonry shear wall [12.4.2] | Add chords [12.4.3] | | | |
| | Inadequate capacity of coupling beams | Concrete/masonry shear wall [12.4.2] | Strengthen beams [13.4.2]<br>Improve ductility of beams [13.4.2] | | | Remove beams |
| Global Stiffness | Excess drift (normally near the top of the building) | Concrete/masonry shear wall [12.4.2] | Concrete/steel column jackets [12.4.5]<br>Provide detailing of all other elements to accept drifts<br>Concrete wall overlay [21.4.5] | | Supplemental damping [24.4] | |
| Configuration | Discontinuous walls | Add wall or adequate columns beneath [12.4.2] | Fiber composite wrap of supporting columns [12.4.4]<br>Concrete/steel jacket of supporting columns [12.4.5] | Improve connection to diaphragm [13.4.3] | | Remove wall |
| | Soft story or weak story | Add strength or stiffness in story to match balance of floors | | | | |
| | Re-entrant corner | Add floor area to minimize effect of corner | | Provide chords in diaphragm [12.4.3] | | |

| Table 20.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for RM2 Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Configuration (continued) | Torsional layout | Add balancing walls [12.4.2] | | | | |
| Load Path | Inadequate collector | Add steel or concrete collector [12.4.3] | | | | |
| | Inadequate slab bearing on walls | | | Add diagonal dowels [13.4.3]<br>Add steel ledger [13.4.3] | | |
| Component Detailing | Wall inadequate for out-of-plane bending | Add strongbacks [21.4.3] | Concrete wall overlay [21.4.5] | | | |
| | Wall shear critical | | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [13.4.1] | | Reduce flexural capacity [13.4.4] | |
| Diaphragms | Precast components without topping | | Improve interconnection [22.2.11]<br>Add topping | | | |
| | Inadequate in-plane shear capacity | | Concrete slab overlay<br>Fiber composite overlays [22.2.5] | | | |
| | Inadequate shear transfer to walls | | Add diagonal drilled dowels [13.4.3]<br>Add steel angle ledger [13.4.3] | | | |
| | Inadequate chord capacity | New concrete or steel chord member [12.4.3] | | | | |
| | Excessive stresses at openings and irregularities | Add chords [12.4.3] | | | | Infill openings [22.2.4] |
| Foundation | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

### Global Stiffness

Similar to strength, global stiffness is seldom a problem in this building type. However the effect of coupling slabs on initial stiffness and the potential change in stiffness due to yielding of these coupling slabs or wall-beams over doors should be investigated.

### Configuration

The most common configuration deficiencies in this building type are weak or soft stories created by walls that change configuration or are eliminated at the lower floors. It is difficult to provide the needed ductility at the weak story and often strength must be added. Completely discontinuous walls also create a load transfer deficiency for both overturning and shear. In such cases, collectors are often needed in the floor diaphragm, and supporting columns need axial strengthening.

### Load Path

A common deficiency in this building is weakness in the load path from floor to walls, either collector weaknesses or shear transfer weakness immediately at the floor wall interface. Local transfer can be strengthened by adding concrete or steel corbel elements, dowels, or combinations of these components. As indicated above, discontinuous walls also often create load path deficiencies from the wall into the diaphragm at the discontinuity.

### Component Detailing

The most common detailing problem in this building type is an imbalance of shear and flexural strength in the walls, leading to pre-emptive shear failure. This condition may be shown to be acceptable with small displacement demands. Walls can be strengthened in shear with overlays of concrete, steel, or FRP.

The layout of walls often forces coupling between walls through the slab system or across headers of vertically aligned doors. These coupling components are seldom designed for the coupling distortions that they will undergo, particularly in older buildings. Short lengths of slabs between adjacent walls receive damage by coupling action that could compromise the gravity capacity. It is difficult to add strength or ductility to these slab areas, but vertical support at support points can be supplemented by corbels of steel or concrete. Damage to headers over doors often does not contribute to deterioration of overall response and can sometimes be acceptable. Local areas of wall can also be strengthened by overlays of concrete, steel, or FRP.

### Diaphragm Deficiencies

Precast floor systems used in this building type often provide inadequate diaphragm behavior that could lead to bearing failures at the floor wall interface, particularly when no topping slab is present. Some topping slabs used primarily for leveling and smoothing the floor are inadequately tied to the precast elements or the walls, and are too thin or poorly reinforced to act as diaphragms on their own. See Chapter 22.

### *Foundation Deficiencies*

This building type often places large demands on the foundation system.  If rocking is shown to be a controlling displacement fuse for the building, the foundations must be investigated to assure that these displacements can safely occur.  See Chapter 23.

## 20.4 Detailed Description of Techniques Primarily Associated with This Building Type

Most significant recommendations listed in Table 20.3-1 are the same as techniques used in the similar building type, **C2b**, Concrete Shear Walls (Bearing Wall Systems) or general techniques applied to concrete diaphragms.  Details concerning these techniques can be found in other chapters.

## 20.5 References

FEMA, 2000, *Prestandard and Commentary for the Seismic Rehabilitation of* Buildings, FEMA 356, Federal Emergency Management Agency, Washington, D.C., November.

# Chapter 21 - Building Type URM: Unreinforced Masonry Bearing Walls

## 21.1 Description of the Model Building Type

Building Type **URM** consists of unreinforced masonry bearing walls, usually at the perimeter and usually brick masonry. The floors are typically of wood joists and wood sheathing supported on the walls and on interior post and beam construction. This building type is common throughout the United States and was built for a wide variety of uses, from one-story commercial or industrial occupancies to multistory warehouses to mid-rise hotels. It has consistently performed poorly in earthquakes. The most common failure is an outward collapse of the exterior walls caused by loss of lateral support due to separation of the walls from the floor and roof diaphragms. Figure 21.1-1 shows an example of this building type.

Building Type **URMA** is similar to the Building Type **URM**, but the floors and roof are constructed of materials that form a rigid diaphragm, usually concrete slabs or steel joists with flat-arched unreinforced masonry spanning between the joists. Building Type **URMA** is not covered by this document.

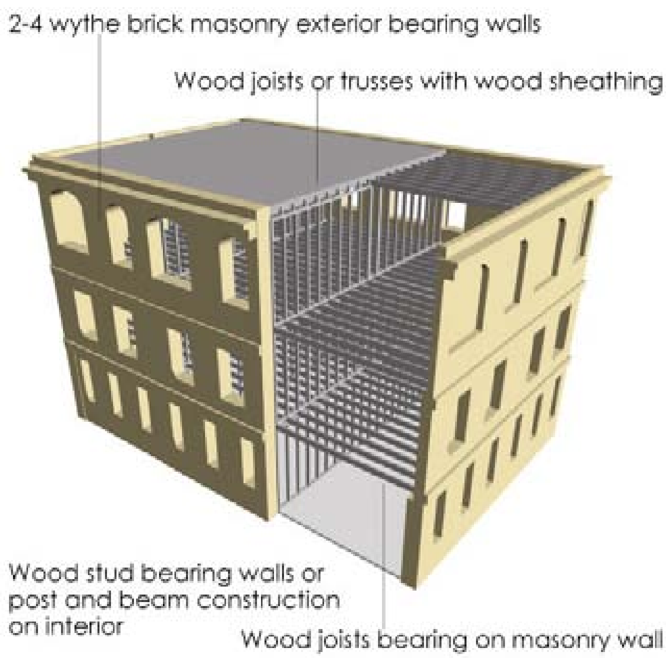



**Figure 21.1-1: Building Type URM: Unreinforced Masonry Bearing Walls**

### *Masonry Wall Materials*

FEMA 306 (FEMA, 1999a) provides an overview of masonry wall material variables. It is paraphrased here. Unreinforced masonry is one of the oldest and most diverse building materials. Important material variables include masonry unit type, wall construction type, and material properties of various constituents.

Solid clay-brick unit masonry is the most common type of masonry unit, but there are a number of other common types, such as hollow clay brick, structural clay tile, concrete masonry, stone masonry, and adobe. Hollow clay tile (HCT) is a more common term for some types of structural clay tile. Concrete masonry units (CMU) can be ungrouted, partially grouted, or fully grouted. Stone masonry can be made from any type of stone, but sandstone, limestone, and granite are common. Other stones common in a local area are used as well. Sometimes materials are combined, such as brick facing over CMU backing, or stone facing over a brick backing.

Wall construction patterns also vary widely, with bond patterns ranging from common running bond in brick to random ashlar patterns in stone masonry to stacked bond in CMU buildings. The variety of solid brick bond patterns is extensive. Key differences include the extent of header courses, whether collar joints are filled, whether cavity-wall construction was used, and the nature of the ties between the facing and backing wythes. In the United States, for example, typical running-bond brick masonry includes header courses interspersed by about five to six stretcher courses. Header courses help tie the wall together and allow it to behave in a more monolithic fashion for both in-plane and out-of-plane demands. The 1997 UCBC (ICBO, 1997) and 2003 IEBC (ICC, 2003) have specific prescriptive requirements on the percentage, spacing, and depth of headers. Facing wythes not meeting these requirements must be considered as veneer and are therefore not used to determine the effective thickness of the wall. Veneer wythes must be tied back to the backing to help prevent out-of-plane separation and falling hazards. Although bed and head joints are routinely filled with mortar, the extent of collar-joint fill varies widely. Completely filled collar joints with metal ties between wythes help the wall to behave in a more monolithic fashion for out-of-plane demands. One form of construction where interior vertical joints are deliberately not filled is cavity-wall construction. Used in many northeastern United States buildings, the cavity helps provide an insulating layer and a means of dissipating moisture. The cavity, however, reduces the out-of-plane seismic capacity of the wall.

Material properties—such as compressive, tensile and shear strengths and compressive, and tensile and shear moduli—vary widely among masonry units, brick and mortar. An important issue for in-plane capacity is the relative strength of masonry and mortar. Older mortars typically used a lime/sand mix and are usually weaker than the masonry units. With time, cement was added to the mix and mortars became stronger. When mortars are stronger than the masonry, strength may be enhanced, but brittle cracking through the masonry units may be more likely to occur, resulting in lower deformation capacity.

Given the wide range of masonry units, construction and material properties, developing comprehensive mitigation techniques for all permutations is not practical. The rehabilitation measures in this document are most directly relevant to solid clay brick masonry laid in running bond with a typical spacing of header courses.

For additional general background on URM materials, see ABK (1981a), FEMA 274 (FEMA, 1997b), FEMA 307 (FEMA, 1999b), and Rutherford and Chekene (1997).

### Floor and Roof Diaphragms

Building Type **URM**, by definition, is built with diaphragms that are considered, relative to the masonry walls, to be flexible. Typically, wood sheathing is attached to wood joists. Several types of wood diaphragm construction are common, including:

Roofs with straight sheathing and roofing applied directly to the sheathing
Roofs with diagonal sheathing and roofing applied directly to the sheathing
Floors with straight tongue-and-groove flooring
Floors with straight tongue-and-groove flooring over straight sheathing
Floors with finished flooring over diagonal sheathing

Wood structural panel overlays may have been added as part of past renovation work, or there may be additional layers of sheathing materials. In some buildings with heavy live loads, like warehouses, 2x or 3x decking may have been used to span between joists.

### Foundations

Foundations for URM buildings typically are spread footings at interior columns and strip footings under masonry bearing walls. Footings are typically either brick or concrete, though stone might be found under older walls, particularly if stone masonry was used in the walls.

## 21.2 Seismic Response Characteristics

In many building types, the horizontal diaphragms are more rigid than the vertical elements of the lateral force-resisting system. Such buildings are often thought of as lumped mass systems with the weight tributary to each diaphragm level lumped along a vertical cantilever with dynamic properties dependent on the stiffness of the vertical lateral force-resisting elements. Ground motion input at the base is dynamically amplified up the cantilever, increasing at each floor level. Each point within a floor has a similar acceleration.

Building Type **URM**, by contrast, has flexible diaphragms and stiff walls. Beginning with the ABK research program in the 1980s (see ABK, 1981a,b,c; and ABK, 1984), a different dynamic model was formulated for **URM** buildings. The ABK model assumes that there is relatively little dynamic amplification between the base and the top of the URM walls in the direction parallel to input motion. Significant amplification instead occurs at the midspan of the flexible diaphragms as they are driven by in-plane motion of the end walls. This generates large out-of-plane forces on the connections between the diaphragm and the coupled masonry walls. In some cases, the diaphragm may yield, limiting the forces that can be transmitted to the in-plane walls. If interior partitions are connected to the partitions, the deformation, cracking damage and resulting energy dissipation can help limit the movement of the diaphragms. Such existing partitions or newly added partitions or moment frames are termed “crosswalls”.

The ABK program identified two modes of behavior for in-plane loading on the piers in the unreinforced masonry walls: shear-critical behavior and rocking-critical behavior. Each of these modes of behavior could be found acceptable if demands were below the capacity. These two modes are included in the UCBC as well. FEMA 273 (FEMA, 1997a) and then FEMA 356 (FEMA, 2000) expanded the characterization of in-plane behavioral modes into the more brittle force-controlled modes (toe crushing and diagonal tension) and the more ductile deformation-controlled modes (bed joint sliding and rocking). FEMA 306 and FEMA 307 identify a number of other in-plane modes and sequences of modes.

The ABK program also established criteria for determining the acceptability of out-of-plane resistance of the unreinforced masonry walls. Important variables are the height-to-thickness (*h/t*) ratio, presence of crosswalls, and overburden (axial compression) pressure on the walls. In short, a stocky lower story wall with overburden pressure from floors and walls above that is driven by a diaphragm damped by crosswalls is less likely to buckle and fail out-of-plane.

It is important to recognize that **URM** buildings vary substantially in structural layout and characteristics, and this can have a significant effect on seismic response. Fairly rectangular multistory residential, office, and commercial buildings often have relatively low story heights and many partitions that can serve as crosswalls. Walls adjacent to other buildings will usually be relatively solid. These buildings typically perform much better than structures like churches, which can have irregular plans, re-entrant corners, tall story heights, heavy walls, offset roofs, few partitions, and many windows. Churches can also be some of the most expensive structures to rehabilitate.

## 21.3 Common Seismic Deficiencies and Applicable Rehabilitation Techniques

**URM** buildings are generally considered to be one of the most hazardous building types. Significant property damage and loss of life have occurred in **URM** buildings during earthquakes around the world and in the United States. The primary deficiencies are due to unbraced parapets which can fall on adjacent pedestrian thoroughfares and poorly connected walls and diaphragms which can lead wall failure and loss of vertical support for diaphragms. See below for general discussion and Table 21.3-1 for a more detailed compilation of common seismic deficiencies and rehabilitation techniques for Building Type **URM**.

### *Global Strength*

As shear wall buildings, global strength in **URM** buildings is dependent on the in-plane shear capacity of the walls. Relatively large seismic forces are needed to lead to life safety concerns with in-plane wall behavior, though cracking damage will occur in relatively moderate events. When walls are found to be deficient, new vertical lateral force-resisting elements can be added at interior locations or existing walls can be enhanced. At interior locations, new elements include wood structural panel shear walls, concrete shear walls, reinforced masonry shear walls, braced frames, and moment frames. At exterior locations, care must be taken to address relative rigidity concerns. Typically, concrete or shotcrete overlays are used to enhance the URM wall capacity. When the wall is highly punctured, braced frames or moment frames may be a viable option. The use of wood structural panel shear walls in buildings with masonry walls is permitted in new construction only in limited situations, such as one-story or two-story buildings with low story heights and no use of diaphragm rotation to resist loads, due to concerns about wood flexibility. Rehabilitation standards such as the 1997 UCBC and 2003 IEBC relax these restrictions significantly, though they do not permit the wood shear walls to resist lateral forces with other materials along the same line of resistance or when there are rigid diaphragms. Use of wood structural panel shear walls in rehabilitating masonry buildings should be carefully considered.

| Table 21.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for URM Bearing Wall Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Global Strength | Insufficient in-plane wall strength | Wood structural panel shear wall [6.4.2], [5.4.1]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9] | Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6]<br>Grouting<br>Infill openings [21.4.7] | | Seismic isolation [24.3] | |
| Global Stiffness | | | | | | |
| Configuration | Soft story, weak story, excessive torsion | Wood structural panel shear wall [6.4.2]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9] | | | | |
| Load Path | Inadequate or missing wall-to-diaphragm tie | | | Tension anchors [21.4.2]<br>Shear anchors [21.4.2]<br>Cross-ties and subdiaphragms [22.2.3]<br>Supplemental vertical supports [21.4.11] | | |
| | Missing collector | Add collector [7.4.2] | | | | |

**Table 21.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for URM Bearing Wall Buildings**

| Deficiency | | Rehabilitation Technique | | | | |
|---|---|---|---|---|---|---|
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| Component Detailing | Wall inadequate for out-of-plane bending | | Exposed interfloor wall supports [21.4.3]<br>Reinforced cores [21.4.4]<br>Concrete wall overlay [21.4.5]<br>Fiber composite wall overlay [21.4.6] | | | |
| | Undesirable wall in-plane behavior mode | | Sawcutting to change shear mode to rocking mode | | | |
| | Unbraced parapet | | Brace parapet [21.4.1] | | | Remove parapet and improve roof-to-wall tie [21.4.1] |
| | Unbraced chimney | | Brace chimney [5.4.6]<br>Infill chimney [5.4.6] | | Reduce chimney height [5.4.6] | Remove chimney [5.4.6] |
| | Poorly anchored veneer or appendages | | Add ties [21.4.12] | | | Remove veneer or appendages |
| Diaphragms | Inadequate in-plane strength and/or stiffness | Add horizontal braced frame [22.2.9]<br>Wood structural panel shear wall [6.4.2]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9]<br>Wood structural panel or steel moment frame crosswall [21.4.10] | Enhance existing diaphragm [22.2.1]<br>Enhance woodframe crosswall [21.4.10] | | | |

| Table 21.3-1: Seismic Deficiencies and Potential Rehabilitation Techniques for URM Bearing Wall Buildings | | | | | | |
|---|---|---|---|---|---|---|
| **Deficiency** | | **Rehabilitation Technique** | | | | |
| **Category** | **Deficiency** | **Add New Elements** | **Enhance Existing Elements** | **Improve Connections Between Elements** | **Reduce Demand** | **Remove Selected Components** |
| | Inadequate chord capacity | Add steel strap or angle | | | | |
| Diaphragms (continued) | Excessive stresses at openings and irregularities | Add wood or steel strap reinforcement | | | | |
| | Re-entrant corner | Wood structural panel shear wall [6.4.2]<br>Concrete/masonry shear wall [21.4.8]<br>Steel braced frame [7.4.1]<br>Steel moment frame [21.4.9] | | Collector [7.4.2] | | |
| Foundation | See Chapter 23 | | | | | |
| [ ] Numbers noted in brackets refer to sections containing detailed descriptions of rehabilitation techniques. | | | | | | |

### *Global Stiffness*

URM bearing walls are generally quite rigid. When walls are solid or lightly punctured with window openings, global stiffness deficiencies are typically not an issue. In some buildings, though, facades facing the street can be highly punctured with relatively narrow piers between openings. In addition to lacking adequate strength, these wall lines may also be too flexible as well.

### *Configuration*

Many commercial **URM** buildings will have a fairly open street façade at the ground level, leading to a weak and soft first story and torsional irregularities. This is usually addressed by the addition of a moment frame at the façade or another vertical lateral force-resisting element at some distance back from the façade.

### *Load Path*

As noted above, it is the lack of adequate ties between the walls and diaphragms that is the single most significant deficiency in **URM** buildings. Rehabilitation measures include tension ties for out-of-plane forces and shear ties for in-plane forces. Bond beams are often employed for connections where the roof runs over the top of the walls. As a back-up vertical support system, supplemental vertical supports are added under trusses or girders where large gravity loads are concentrated on the wall in case the masonry is damaged locally.

### *Component Detailing*

Since the masonry elements in Building Type **URM** are unreinforced by definition, they do not comply with modern ductile detailing requirements. Walls deemed susceptible to out-of-plane bending failures can be strengthened by strongbacks placed against them either on the outside or more commonly on the interior face. When preservation of finishes is critical, reinforced cores can be drilled and installed within the wall. Parapet bracing and chimney bracing are common. In some buildings, the exterior brick wythe will not be anchored back to the backing wall with mechanical ties or sufficient headers, and veneer ties are installed.

### *Diaphragm Deficiencies*

Wood diaphragms may lack both strength and stiffness in **URM** buildings. This can be addressed by adding new interior elements to cut the diaphragm span or by enhancing the diaphragm itself with wood structural panel overlays. In some cases, such as sloped roofs, new horizontal braced frame diaphragms are added, in lieu of strengthening the existing diaphragm.

### *Foundation Deficiencies*

Foundation deficiencies for existing elements are relatively uncommon in **URM** buildings. Foundation rehabilitation work usually is focused on the support for new lateral force-resisting elements that are added to the superstructure.

## 21.4 Detailed Description of Techniques Primarily Associated with This Building Type

### 21.4.1 Brace or Remove URM Parapet

***Deficiency Addressed by Rehabilitation Technique***

Past earthquakes have consistently shown that unreinforced masonry chimneys and parapets are the first elements to fail in earthquakes due to inadequate bending strength and ductility. Parapets tend to have greater damage at midspan of diaphragms due to higher accelerations and displacements from the oscillating diaphragm.

***Description of the Rehabilitation Technique***

URM parapets can be braced or removed to minimize the falling hazard risk. Bracing is usually done with a steel angle brace. The brace is anchored near the top of the parapet and to the roof. The existing roof framing may need localized strengthening to take the reaction from the brace. Roof-to-wall tension anchors are typically part of parapet bracing. See Figure 21.4.1-1 for an example of parapet bracing. If the top of the parapet is removed, the vertical compressive stress on roof-to-wall anchors is reduced, so removing the parapet is often combined with adding a concrete cap or bond beam as part of the roof-to-wall anchorage. See Figure 21.4.1-2 for an example of parapet removal and addition of a concrete cap beam. See Section 21.4.2 for more details on wall-to-diaphragm anchorage.

***Design Considerations***

*Research basis:* No references directly addressing testing of parapet bracing have been identified.

*Parapet height:* Codes such as the 2003 IEBC and 1997 UCBC provide maximum allowable height-to-thickness (*h/t*) ratios for parapets. The height is taken from the lower of the either the tension anchors or the roof sheathing. Requirements are more stringent in higher seismic zones. With the 1997 UCBC, for example, the *h/t* ratio in Seismic Zone 4 is 1.5, so for a typical 13" thick, three-wythe wall, parapets taller than 19.5" above the roof-to-wall anchors require removal or bracing. In the 2003 IEBC, locations with $S_{D1} > 0.4g$ have the same requirements.

*Fire protection:* The original purpose of extending the masonry wall up to form a parapet was to help limit the spread of fire between the wood roofs of adjacent buildings. Removing a parapet must be coordinated with local building code fire safety requirements.

*Guardrails:* Parapets often serve as a guardrail around a roof. Removing a parapet must be coordinated with local building code life safety requirements.

*Load path:* When parapet bracing/roof-to-wall tension anchorage is the only rehabilitation technique, the out-of-plane load path can be incomplete, particularly when the roof joists are perpendicular to the brace. A more complete rehabilitation strategy includes developing the parapet and tension anchorage forces back into the roof diaphragm through the use of subdiaphragms and cross-ties.

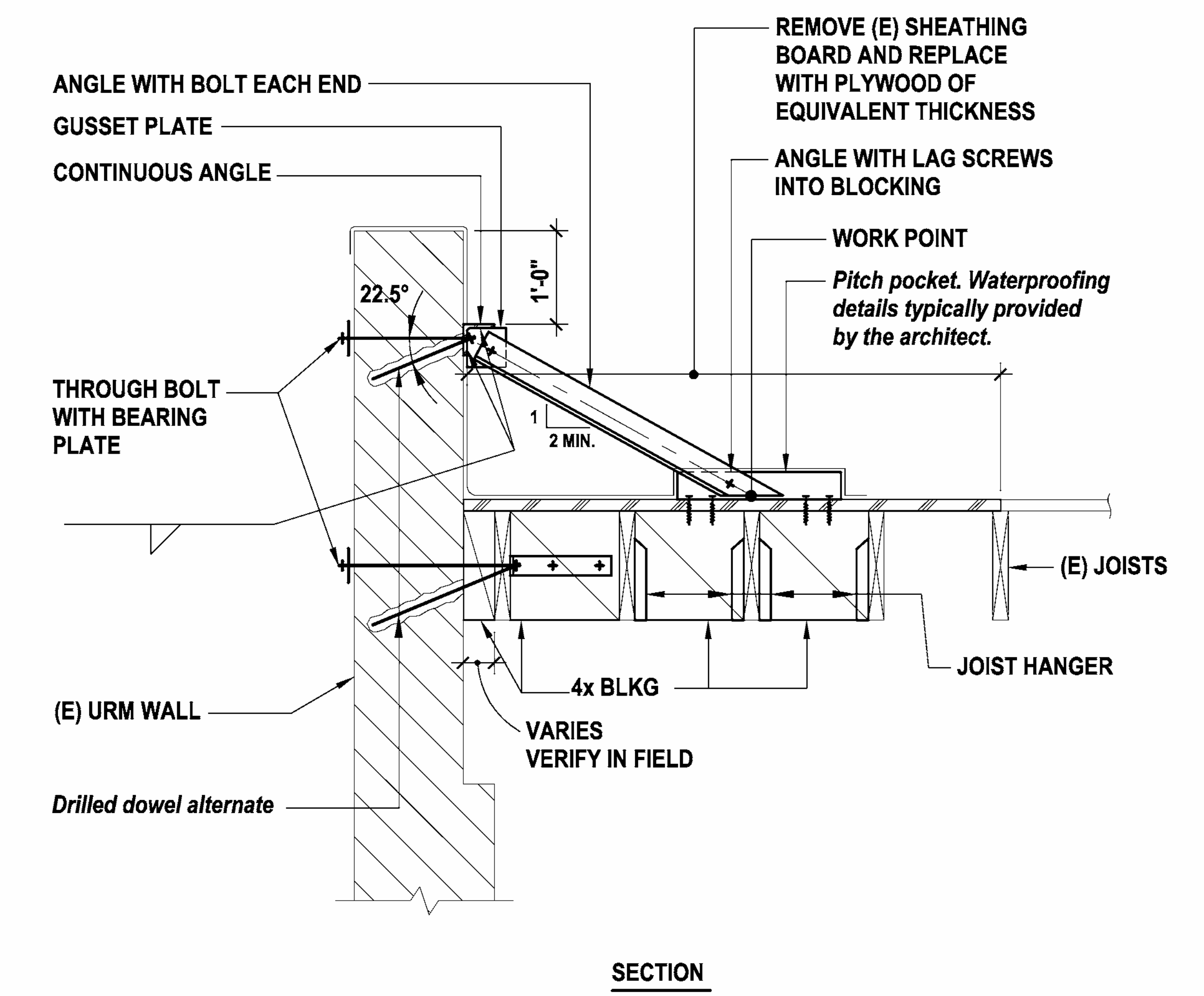



**Figure 21.4.1-1: Parapet Bracing**

### *Detailing and Construction Considerations*

*Parapet anchorage types:* Drilled dowels connecting the top of the bracing to the masonry can be with through bolts or adhesive anchors. See Section 21.4.2 for detailed discussion of drilled dowels.

*Top angle:* Figure 21.4.1-1 shows a continuous angle running between braces in the roof. This angle can be used to span between braces to reduce the number of bracing points. It also increases redundancy over a localized connection of the brace to the parapet.

*Load in the roof framing:* The vertical reactions at the base of the brace are typically resisted by roof framing. In Figure 21.4.1-1 the added blocking beneath the base of brace workpoint helps to engage three joists in resisting vertical loads. Tall parapets can generate substantial brace forces that existing wood roof joists may not be able to resist. Additional joists can be added, or more braces can be used to distribute the load. Horizontal loads from the brace are distributed by the blocking and new wood structural panel.

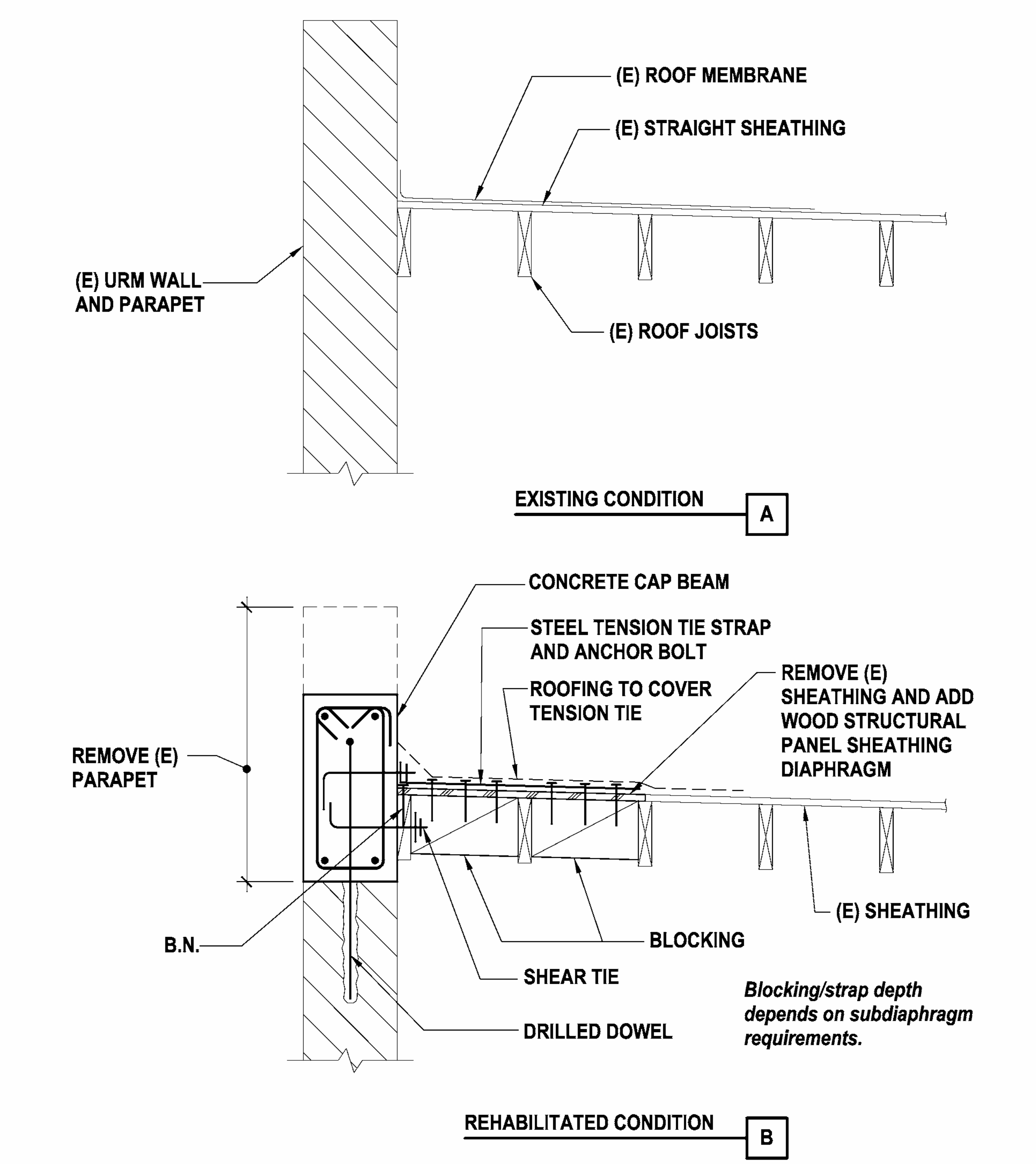



**Figure 21.4.1-2: Parapet Removal and Concrete Cap Beam**

*Waterproofing at the roof:* The brace anchor at the roof needs to attach to the structural framing members, so a penetration through the roof membrane will occur that needs proper waterproofing design by a qualified contractor or design professional. Often, parapet bracing and roof-to-wall ties and even roof diaphragm sheathing rehabilitation activities are combined with roofing replacement given the cost effectiveness of combining the work.

***Cost and Disruption Considerations***

Adding parapet bracing and roof-to-wall tension anchors provide some of the most effective seismic rehabilitation for reducing life safety risks. As a result, some communities—such as San Francisco—passed parapet safety ordinances requiring mandatory mitigation many years ago. Disruption is typically relatively low since occupants can remain in place. Combining parapet bracing and roof-to-wall ties and even roof diaphragm sheathing rehabilitation activities with roofing replacement can significantly reduce the total cost of the work. Disruption can increase noticeably if the roof has to be removed for installation.

***Proprietary Issues***

There are no proprietary concerns with parapet bracing, other than use of proprietary anchors as part of the assemblage. See Section 21.4.2.

### 21.4.2 Add Wall-to-Diaphragm Ties

***Deficiency Addressed by Rehabilitation Technique***

Inadequate or missing shear and tension connections between the unreinforced masonry bearing wall and the wood floor or roof.

***Description of the Rehabilitation Technique***

The most significant deficiency in URM bearing wall buildings is the lack of an adequate positive (i.e. mechanical) tie between the masonry walls and the floor and roof diaphragms. Ties are usually separated into two categories: tension ties and shear ties. Tension ties transfer out-of-plane inertial loads perpendicular to the face of the masonry back into the diaphragm. Shear ties transfer loads from the diaphragm into the wall where they are resisted by in-plane action of the wall. Tension ties help keep the walls from falling away from the diaphragms; shear ties help keep the diaphragm from sliding along parallel to the wall. Ties are assemblages that consist of both the anchorage to the wall (shown in detail in Figures 21.4.2-1 and 21.4.2-2) and the anchorage back into the diaphragm (shown in the subsequent figures).

***Design Considerations***

*Research basis:* The focus of wall-to-diaphragm testing to date has been on the anchorage to the masonry and has been done primarily by manufacturers. Paquette, Bruneau and Brzev (2003) tested a specimen of a small full-scale one-story building with roof-to-wall ties, but the focus of the work was on wall and diaphragm response.

*Anchor types and capacities:* The 1997 UCBC and 2003 IEBC provide prescriptive values for tension and shear bolts meeting certain requirements. These are for a 2-1/2” diameter hole filled with nonshrink grout approach that is typically no longer used. The ICBO and now ICC evaluation report process has standardized procedures for vendors supplying adhesive ties for use in brick masonry. Three installations are included in most vendors’ ICC Evaluation Service reports, and they have standardized installation techniques and capacities. Adapted versions of these installations are shown in Figures 21.4.2-1 and 21.4.2-2. Figure 21.4.2-1A shows a

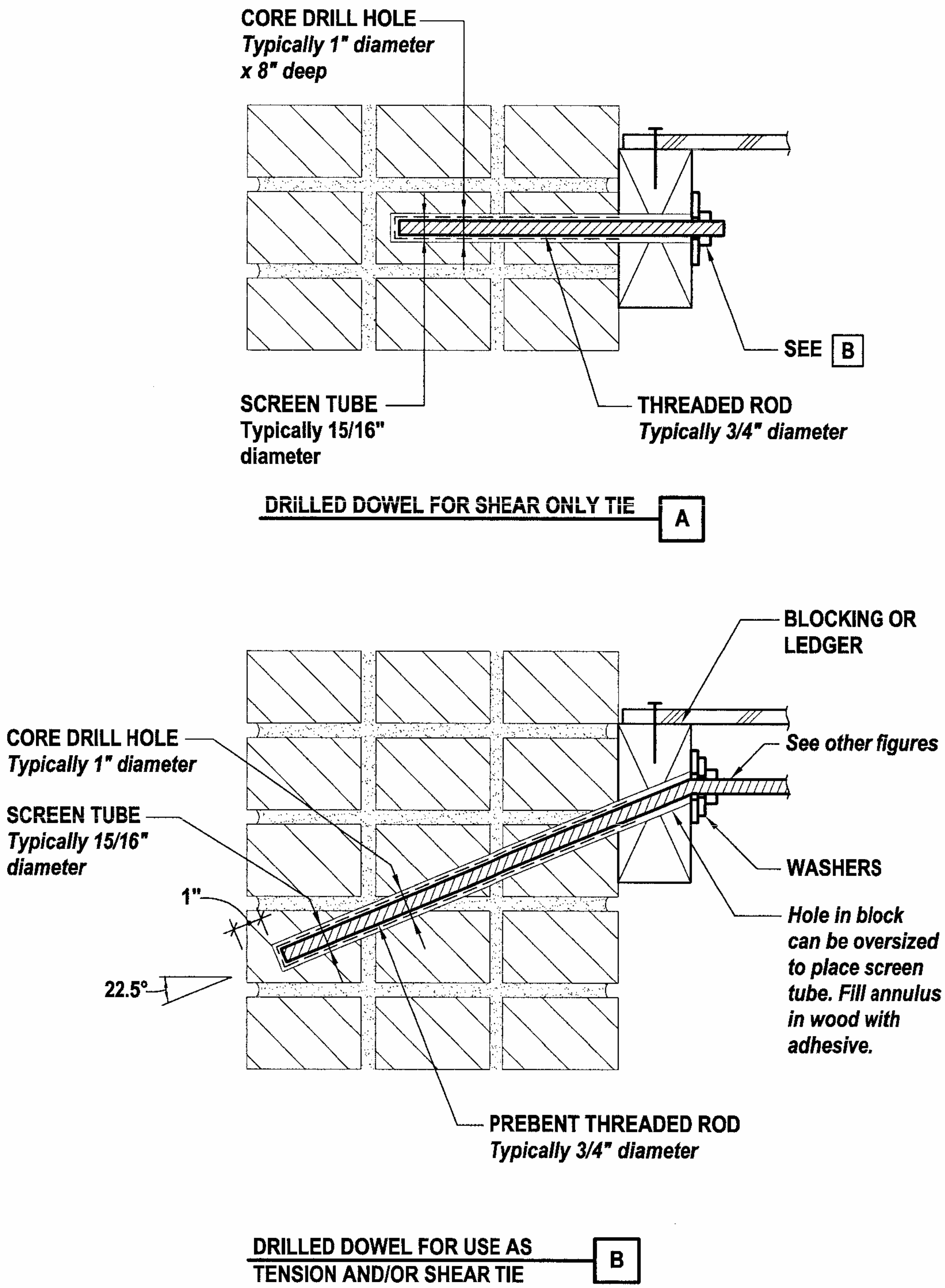
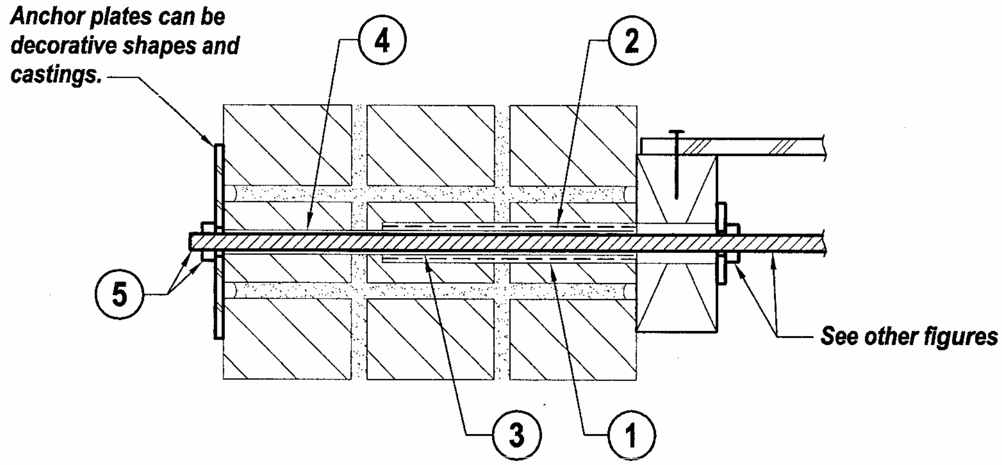



**Figure 21.4.2-1: Drilled Dowels**




**SEQUENCE OF INSTALLATION**

1. **CORE DRILL HOLE.**
   *Typically 1" diameter x 8" deep.*
2. **PLACE SCREEN TUBE WITH ADHESIVE.**
   *Typically 15/16" diameter x 8" deep with plug at end.*
3. **INSERT STEEL SLEEVE.**
   *Typically 13/16" outside diameter.*
4. **AFTER CURING, DRILL HOLE THROUGH PLUG AND REMAINING MASONRY.**
5. **PLACE THREADED ROD AND ANCHOR PLATE.**
   *Typically 5/8" diameter and 6"x6"x3/8" respectively.*

**Figure 21.4.2-2: Through Bolt Anchor**

"combination" drilled dowel that can be used for resisting both tension and shear forces. It is drilled into the wall at a 22.5 degree angle from horizontal at least 13" into the wall. The angle allows the dowel to engage more courses of brick, theoretically improving the reliability. At the allowable stress design (ASD) force level, it is good for 1200 lbs in tension and 1000 lbs in shear. Figure 21.4.2-1B shows a drilled dowel used only for resisting shear forces. It goes in 8" deep into the masonry and is good for 1000 lbs at the ASD level. Figure 21.4.2-2 shows a special through bolt anchor using a steel sleeve in the first 8" that can take tension and shear and has the same values as the combination anchor. These ICC capacities are typically used in design; they come with a number of restrictions and requirements such as quality of masonry. When higher values are needed, proof testing can be undertaken. In the ICC standards for both shear and tension testing (ICC-ES, 2005) of adhesive anchors that manufacturers must use to obtain ICC qualification, allowable stress design capacities are the lower of prescriptive values and the average ultimate test value divided by a safety factors of 5.

It should be appreciated that the prescriptive values in the UCBC, the IEBC, and ICC Evaluation Service reports are based on tests of the drilled dowel itself, not the full elements of the detail.

Capacities for nails, wood structural panels, bolts in wood and straps come from typical code provisions.

### *Detailing and Construction Considerations*

There are many issues to consider in detailing for tension and shear ties. These include the following:

*Aesthetics:* Anchors that go all the way through the wall have a visible bearing plate on the exterior face, such as shown in Figure 21.4.2-2. There are simple circular or octagonal plates that can be purchased or fabricated. Some manufacturers make plates with a countersunk hole and use flathead bolt heads to reduce the surface projection. When the exterior face is stucco, a plate with a countersunk hole can be recessed into the stucco or just into the masonry and refinished with stucco so it is hidden. Special cast anchors can be made if there is a desire to match an historic exposed cast iron anchor. When the anchor plate approach cannot be used, drilled dowels are used such as those shown in Figure 21.4.2-1.

*Nonshrink grout vs. chemical adhesive:* Early ties used cementitious nonshrink grout. They required larger diameter holes (such as 2-1/2") to be cored in the masonry to place the grout. A number of vendors have now created special chemical adhesives and tools that have optimized the process. Standard details use ¾" diameter threaded rods in 1" diameter holes, though other sizes can be used, depending on manufacturer requirements. The typical installation approach is to drill the hole; clean it with a brush and compressed air; fill a nylon, carbon, or stainless steel screen tube (which looks like a test tube made out of wire mesh) with adhesive; place the screen tube into the hole; and then push the rod into the screen tube forcing the adhesive out of the tube into the annulus between the tube and the masonry. Figures 21.4.2-1 and 21.4.2-2 show the anchorage using chemical adhesives and screen tubes.

*Chemical adhesive types:* There are many different types of chemical adhesives, though most are epoxy. Epoxy products have the longest track record. Some vendors have begun to produce other types of chemicals. Key issues when considering an adhesive are the length of time the adhesive has been in use, the extent and quality of the testing, the ability to bond to damp or water filled surfaces, setting time, cost, the heat deflection temperature (an ASTM test method for quantifying the loss of strength as ambient temperature rises), and the capacities shown by test results. Most modern adhesives use two-component pre-packaged assemblies, rather than bulk products used in the past. This reduces the risk of improper mixing and not developing the adhesive to its proper strength. When adhesives are curing, the off-gassing can be unpleasant, and proper ventilation procedures are necessary.

*Dowel material type:* Threaded rod is commonly specified as ATSM A36 all-thread rod. It is a relatively ductile material, with a minimum yield strength of 36 ksi and ultimate strength of 60 ksi. When higher strength material is needed (which is rare), ASTM A193, Grade B7 threaded rod can be used with a minimum yield strength of 105 ksi and ultimate strength of 125 ksi. Rebar can be used as well, but this is not typically done in ties that connect to the wood diaphragms since the threaded connection is needed. Threaded rod is sometimes supplied with oil on it. This must be solvent cleaned, so that proper bonding with adhesives can occur.

*Access:* Installation of ties can be done either from below the diaphragm or above. Figure 21.4.2-3 shows installation of floor-to-wall tension ties from below. Figure 21.4.2-4 shows installation occurring from above the floor. Figure 21.4.2-5 shows installation of floor-to-wall shear dowels from above. Similar details are contained in Rutherford & Chekene (1990), SEAOSC (1982) and SEAOSC (1986). The choice of whether to install from above or below depends on whether there are finishes that need to be avoided, whether diaphragm strengthening is being done, and what type of diaphragm strengthening is planned. If there is a special plaster ceiling to be avoided, then access and installation would proceed from above. If there is no plaster ceiling and the floor or roof diaphragm is not being modified or is being enhanced by adding a wood structural panel overlay from above, then access and installation for wall-diaphragm ties would be from below. Angled dowels (see section below) installed from below can be angled upwards rather than the typical downward angle, provided non-sag adhesives are used.

*Joist direction:* Framing in most *buildings* is orthogonal so that joists or rafters are either perpendicular or parallel to the in-plane direction of the wall. Installations where the joists are perpendicular to the wall are easier to make; installations where the joists are parallel involve blocking and more complicated details. Figures 21.4.2-3 to 21.4.2-5 show variations for joist orientation.

*Special issues at the top of the wall:* In most **URM** buildings, the wall continues up past the roof forming a parapet that provides fire protection and serves as a guardrail during roof maintenance, as described in Section 24.4.1. In some buildings, though, the roof continues over the top of the wall. In these situations, the roof might be relatively flat or sloped. As a result, special issues arise. First, there is reduced overburden pressure at the top of the wall, reducing the reliability of drilled dowels. Second, eccentricities become more significant, such as the vertical eccentricity between the roof diaphragm and the top of the masonry. Making reliable connections between walls in these situations can be particularly challenging and is usually dependent on the specific geometry and characteristics of the existing details. A common strategy is to employ a concrete bond beam at the top of the wall. This ties the wall together, serves as a collector and chord, increases redundancy and often simplifies details. Figure 21.4.2-6 shows a bond beam placed on top of an existing wall under the roof framing. This is possible when the wall is wide, and there is sufficient distance between the masonry and rafter. Figure 21.4.2-7 shows an alternative when there is insufficient clearance between the rafter and top of wall that involves removing the top two courses of masonry to gain room for the bond beam.

*Eccentricity:* It is desirable to minimize the eccentricities in a connection. Figure 21.4.2-8 illustrates the issue and some alternate approaches with floor-to-wall tension ties. Figure 21.4.2-8A shows a common tension tie detail in plan view where a tie-down anchor is connected to the side of an existing joist. The plan offset between the drilled dowel at the center of the tie-down where load is applied and the center of the joist where it is resisted times the force is a moment that must be resisted by the joist in weak way bending. This stress can be quite significant. Figure 21.4.2-8B shows an alternative where two tie-downs are used to make a connection that is

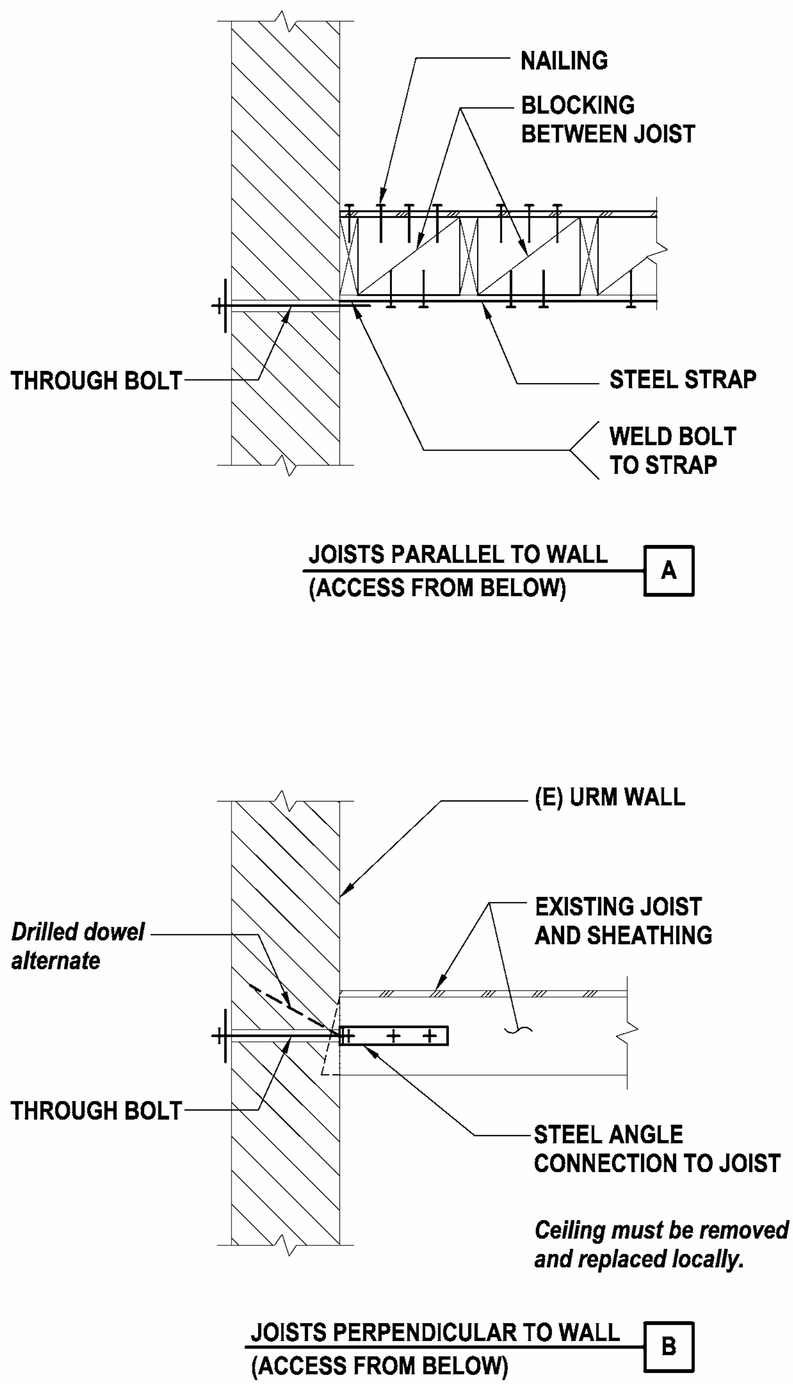



**Figure 21.4.2-3: Tension Anchors Installed from Below the Floor**

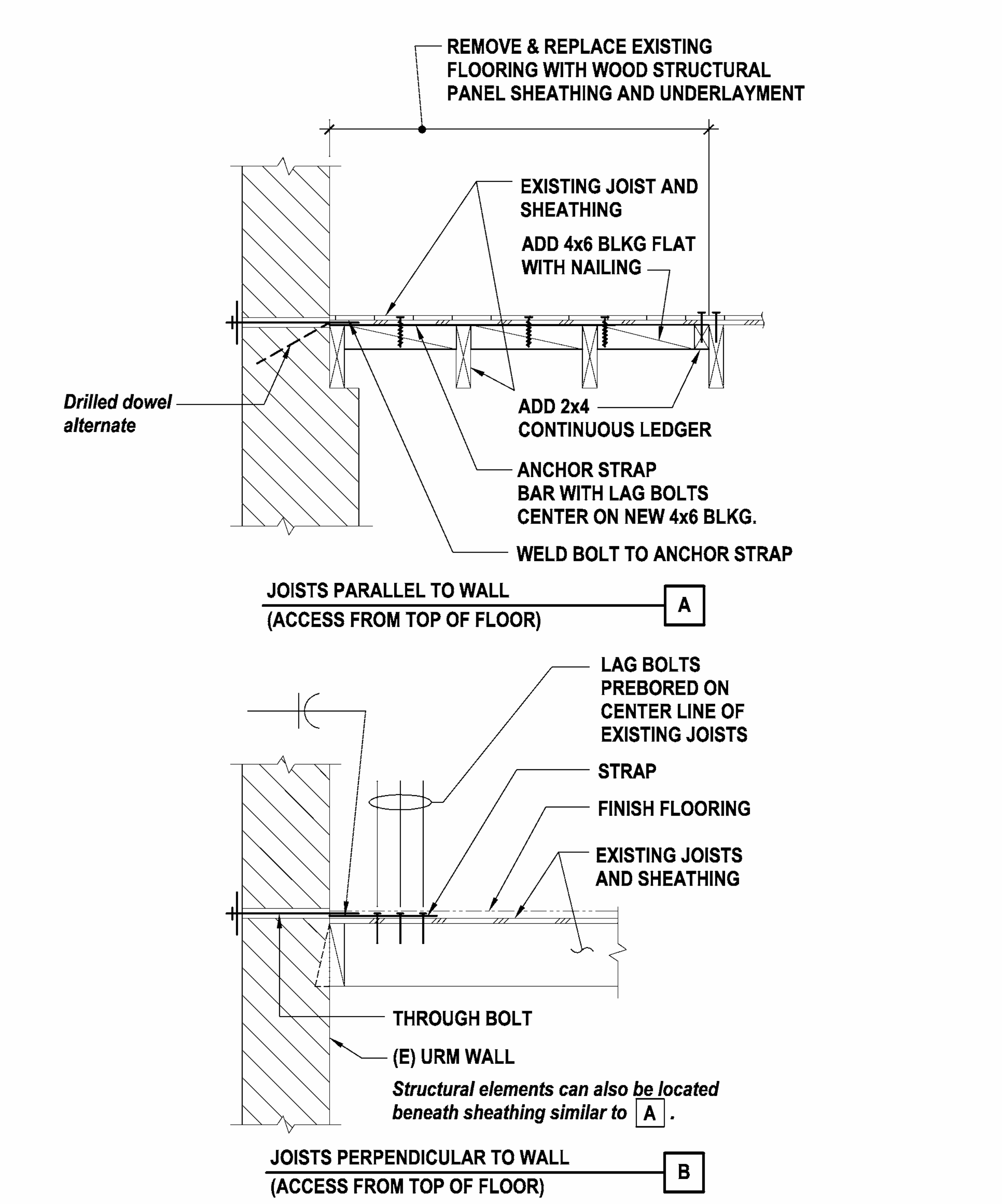



**Figure 21.4.2-4: Tension Anchors Installed from Above the Floor**

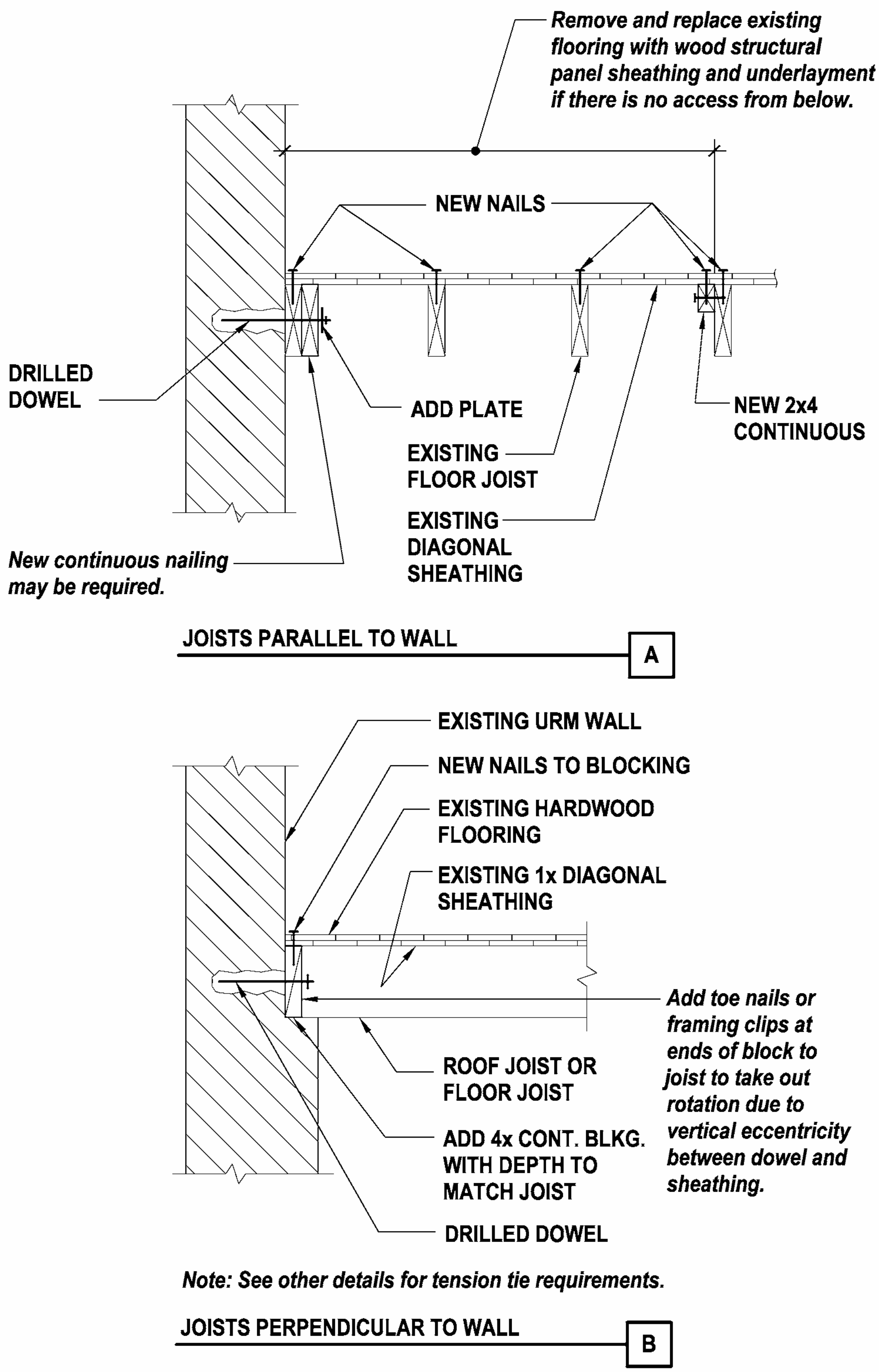



**Figure 21.4.2-5: Floor-to-Wall Shear Anchors**

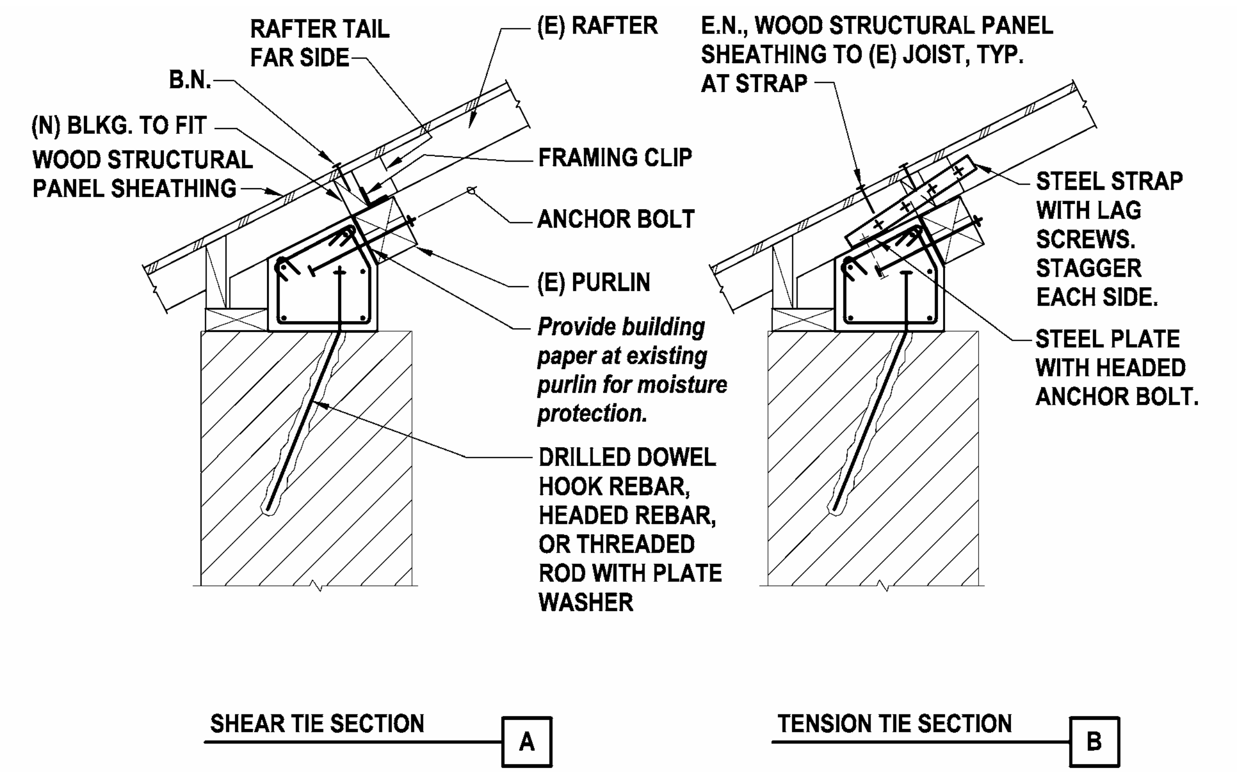



**Figure 21.4.2-6: Bond Beam at a Sloping Roof**

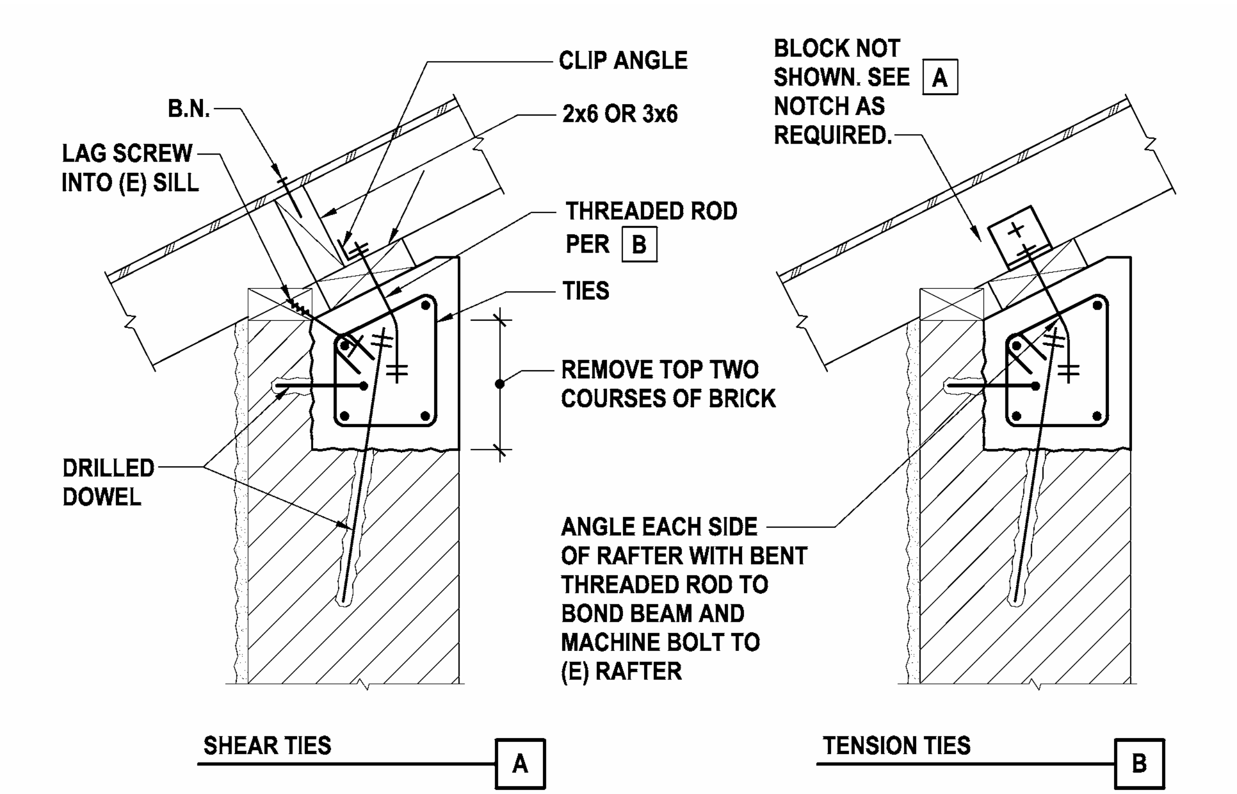



**Figure 21.4.2-7: Bond Beam at a Sloping Roof with Limited Clearance**

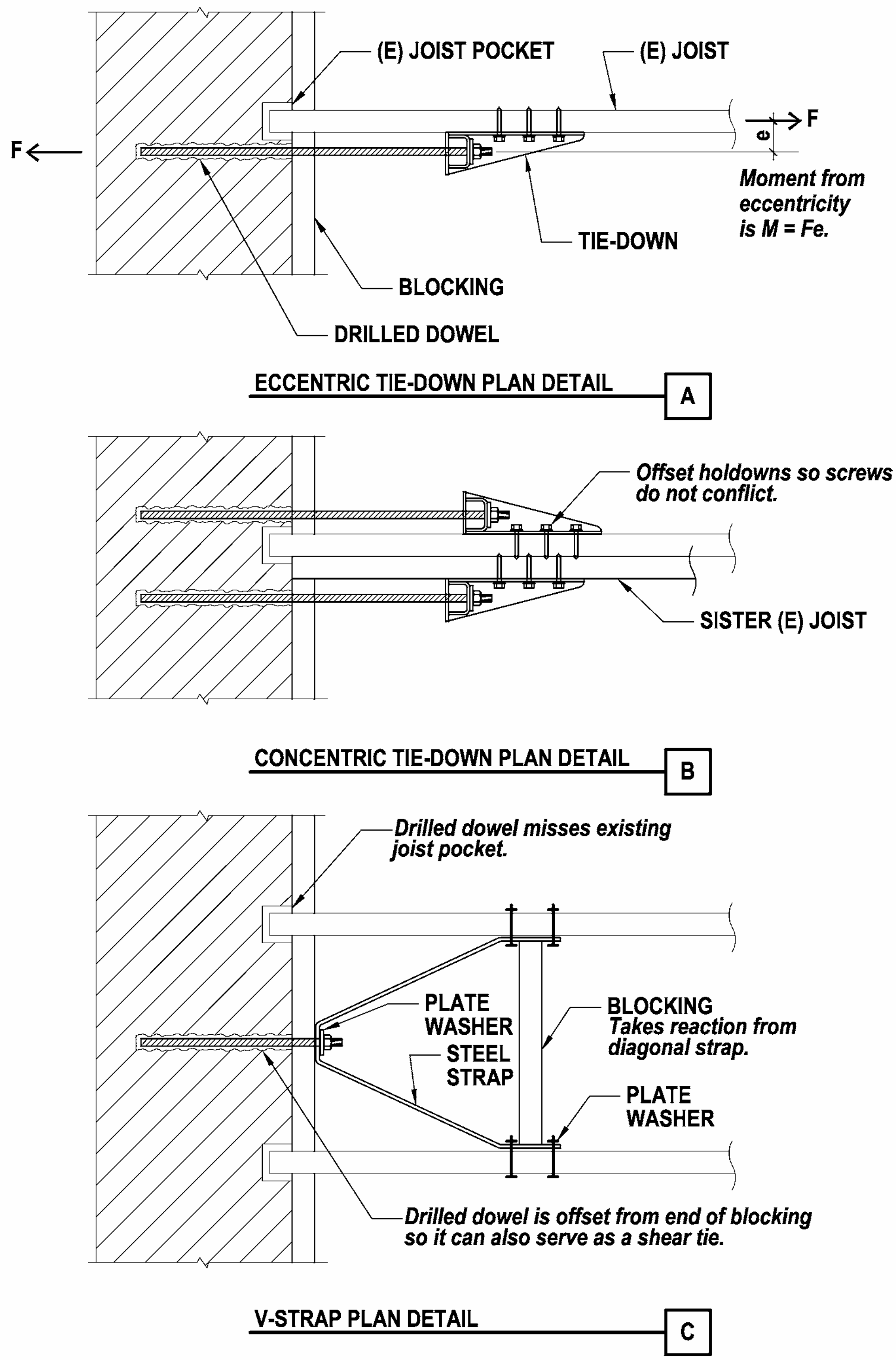



**Figure 21.4.2-8: Tension Tie Connection Issues**

more concentric. This detail, however, puts a large number of screws into the existing joist, so a sistered joist is shown. Adding the sister also permits the nailing into the diaphragm to be into each joist, reducing the nailing demand on the joists. Bolted tie-downs, instead of tie-downs with screws, can be used with through bolts placed in double shear. Traditional bolted tie-downs have greater slip than the more recent tie-downs using screws. There are proprietary connectors using tubes as tie-downs on each side without oversize holes that bolt eliminate eccentricity and reduce bolt slip. Both Figure 21.4.2-8A and 21.4.2-8B have dowels adjacent to the joist. This means the dowel will enter the wall next to or in the weakened area of joist pocket and at the end of new blocking used for shear transfer, where there is insufficient end distance to use the dowel as a shear tie. Figure 21.4.2-8C shows a V-strap detail where the drilled dowel is placed between joists, away from the joist pocket and with plenty of end distance. When the strap is in tension, forces perpendicular to the joists are produced that are resisted by the added blocking and plate washers.

*Truss anchorage:* In some **URM** buildings, there will be large gravity elements that bear on the wall, such as girders or trusses. These also become concentrated points of stiffness in the diaphragm. Since the relative rigidity of the elements cannot be easily quantified, it is usually prudent to use an enveloping or “belt and suspenders” approach of assigning demand, so that typical anchors between trusses take the uniform load and the ties connecting the wall and trusses take additional load.

*New ties vs. reuse of existing ties:* In many older **URM** buildings, there are existing ties called government or “dog” anchors. These anchors typically only occur in the direction where the joists are perpendicular to the face of the wall, and they may not be at sufficient spacing. The 1997 UCBC and 2003 IEBC permit use of these anchors as wall-to-diaphragm tension anchors if tested in accordance with certain standards and capacities are sufficient.

*Dowel spacing and edge distance:* The 1997 UCBC and 2003 IEBC have maximum spacing requirements on shear and tension dowels. When walls become thick, the out-of-plane demands and the relatively low ICC Evaluation Service report capacity values can lead to fairly tight spacing of dowels. The UCBC and IEBC do not have minimum spacing requirements. From a practical point of view, dowels should not be placed closer than 12” o.c. Some ICC reports provide minimum spacing limits as well, like those commonly employed for drilled dowels in concrete. For one vendor, these spacing limits are 16” o.c. in the horizontal and vertical direction, and there is 16”minimum for edge distance as well.

*Corrosion considerations:* Drilled dowels are typically installed from the interior. The masonry cover and epoxy serve as corrosion protection, so mild steel anchors are typically considered sufficient. For increased corrosion protection, stainless steel dowels and screen tubes can be used. When through bolted connections are installed, there is a more direct path for moisture intrusion. The anchor plate can be painted with exterior grade paint, galvanized or be made from stainless steel, and the through bolt can be made from stainless steel as well.

*Screen tubes:* The purpose of the screen tube is to prevent loss of epoxy into cracks or unfilled collar joint voids within the wall. Screen tubes vary somewhat from vendor to vendor and should be considered part of the manufacturer's assembly. Nylon screen tubes have begun to be supplied by many vendors as they are more economical than stainless steel and more corrosion resistant than carbon steel. They do have a much larger coefficient of thermal expansion than both steel screen tubes and masonry.

*Hollow masonry:* Anchorage of hollow clay tile, ungrouted concrete masonry units and other hollow masonry systems to diaphragms is particularly challenging. When forces are large, grouting in the region of the anchor is usually required. When forces are small, use of screen tubes may be acceptable. The screen tube is filled with adhesive, inserted into the wall and as the dowel is pushed into it, the adhesive seeps through the screen tube forming a key behind the face shell of the masonry. Capacities are small and the connection is nonductile. This type of connection may be viable for out-of-plane wall strengthening (see Section 21.4.3) where the demands are lower, but it is not recommended for wall-to-diaphragm connections. Figure 21.4.2-9 shows a method of connecting a floor to an ungrouted CMU wall. Even in ungrouted CMU, a grouted bond beam is usually found beneath the floor, and it helps provide bearing support for the floor joists. Figure 21.4.2-9 involves locally grouting the courses at and just above the floor to install a new anchor. Figure 21.4.2-10 shows an alternative that avoids working from above and uses the existing bond beam. Sistering and a nailer help get the new anchor to the proper elevation. If a grouted bond beam is not present, it may be necessary to create one to make the proper anchorage, similar to the top courses in Figure 21.4.2-9.

*Drilling:* Holes need to be drilled with a rotary drill or a rotohammer drill with the percussion setting turned off to limit vibration into the wall. This can slow drilling significantly. In some cases, coring with a diamond tipped blade is more efficient. This may be the only way some hard masonry, like granite, can be drilled. Sometimes water is used to cool the bit, and the slurry produced by the water, mortar and masonry can stain the face of the wall.

### *Cost/Disruption*

Considerations for cost depend on the number, type and depth of dowels; the difficulty of access; and the extent of finishes that are impacted. Through bolts are usually less expensive than adhesive anchors.

Drilling is loud and can be disruptive to occupants. Typically, either the floor or ceiling has to be removed to install the dowels. Thus, it is usually not practical to install dowels in occupied rooms, though the work can be phased by building area so disruption is minimized.

### *Proprietary Issues*

Values for anchor capacity come from individual vendors, but there are no known concerns with use of a properly procured product.

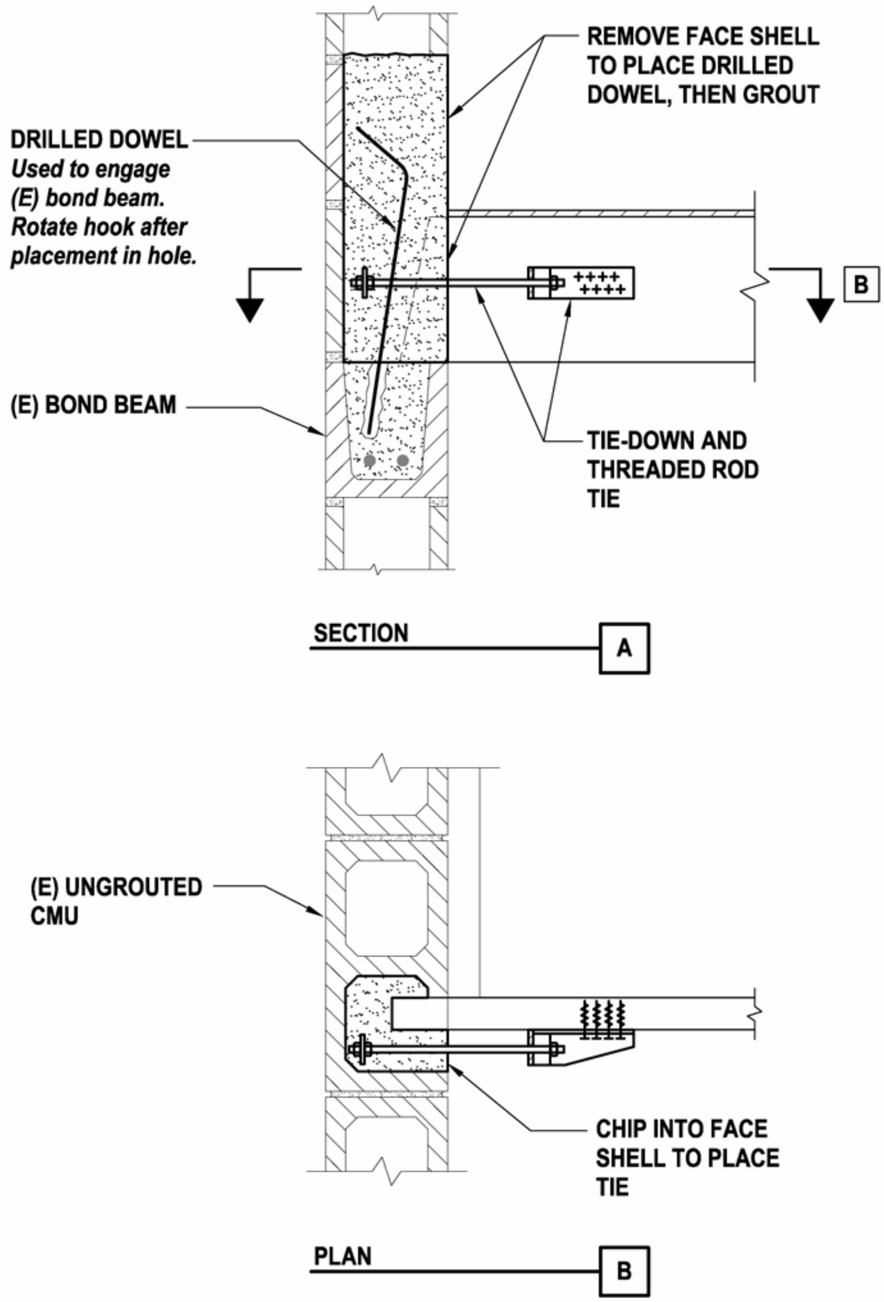



**Figure 21.4.2-9: Wall-to-Floor Tension Tie in Hollow Masonry**

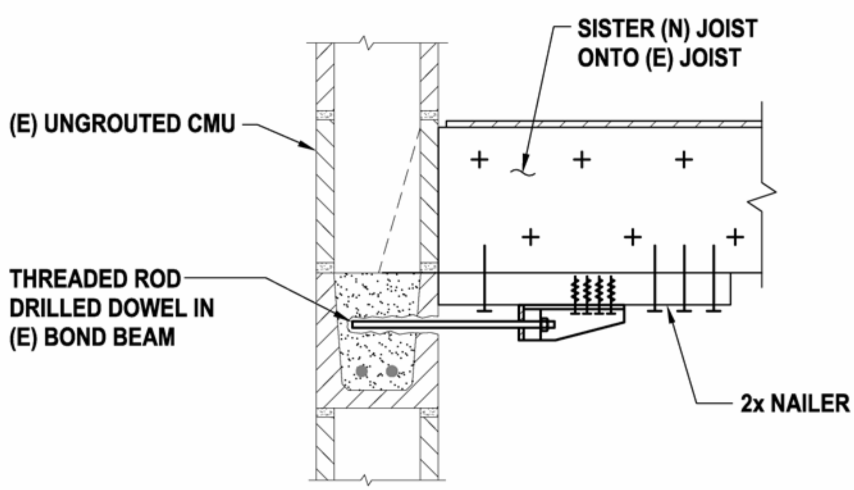



**Figure 21.4.2-10: Wall-to-Floor Tension Tie in Hollow Masonry Alternate**

## 21.4.3 Add Out-of-Plane Bracing for URM Walls

***Deficiency Addressed by Rehabilitation Technique***

Inadequate out-of-plane bending resistance of an unreinforced masonry wall.

***Description of the Rehabilitation Technique***

Two types of bracing can be used: diagonal braces that reduce the effective height of the masonry wall (Figure 21.4.3-1A) and vertical braces or strongbacks that span the full height of the inside face of the wall (Figure 21.4.3-1B). Vertical braces can be surface mounted or, when aesthetic considerations are paramount, recessed into the wall; see Figure 21.4.3-2.

***Design Considerations***

*Research basis:* The most comprehensive set of testing done to date on out-of-plane response of URM walls was part of the ABK research program in the 1980s, and it is documented in ABK (1981c). Full-scale, dynamic testing of 20 wall specimens was conducted. Specimens were 6' wide, 10' to 16' tall, and had height-to-thickness (*h/t*) ratios that varied from 14 to 25. Superimposed axial loads were varied; and materials included brick, grouted CMU, and ungrouted CMU.

*H/t limits:* It is tall, narrow walls that have been found to be susceptible to out-of-plane wall demands. The 1997 UCBC and 2003 IEBC provide maximum *h/t* requirements. Walls with larger *h/t* ratios must be braced.

*Spacing:* For strongbacks, such as shown in Figures 21.4.3-1B and 21.4.3-2A, the maximum spacing requirements are set by the 1997 UCBC or 2003 IEBC at the minimum of 10 feet or half the unsupported height of the wall. For diagonal braces, the maximum spacing is set at 6 feet.

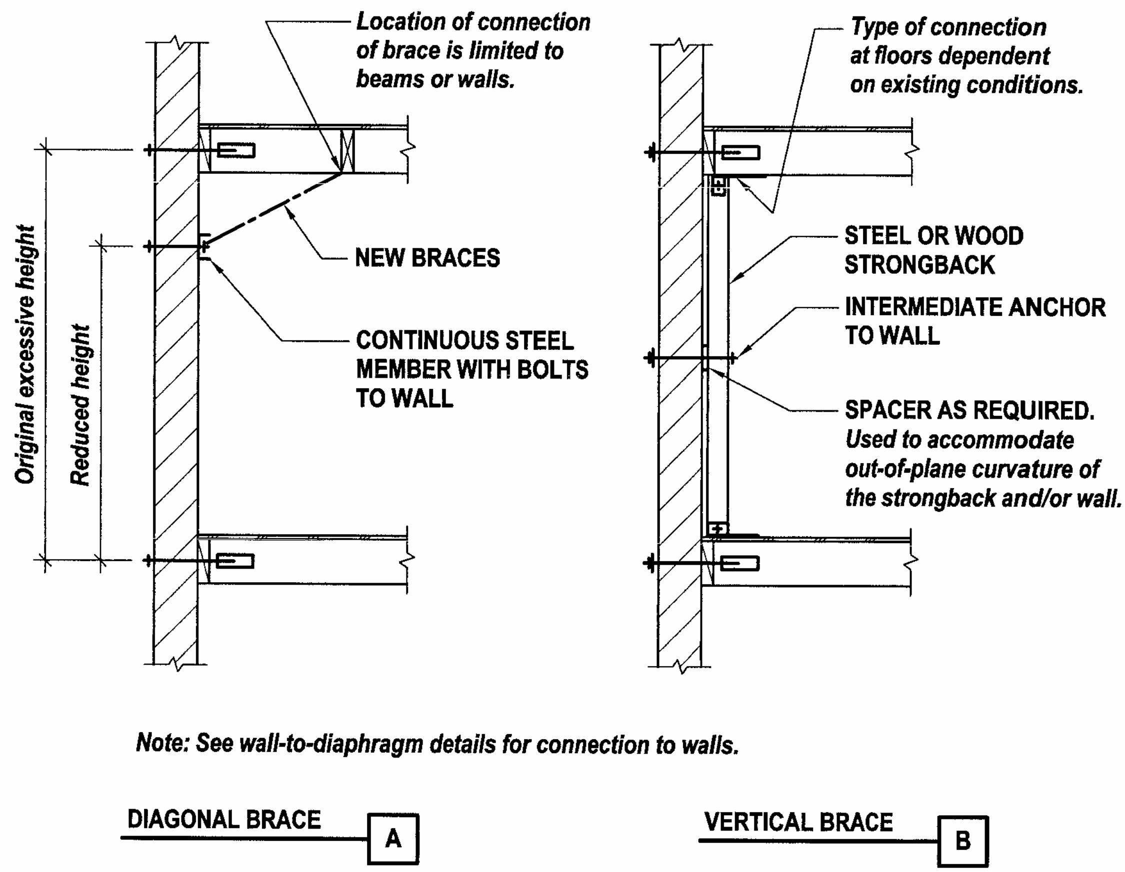



**Figure 21.4.3-1: Exposed Out-of-Plane Wall Bracing**

*Stiffness:* For strongbacks, such as shown in Figures 21.4.3-1B and 21.4.3-2A, the 1997 UCBC limits deflection of the wall at ASD demands to one tenth of the wall thickness. This is not a particularly stringent requirement. Say that the first story of a multistory building in Seismic Zone 4 is 13” thick and 18’ tall and its resulting *h/t* ratio of 16.7 exceeds the *h/t* limit of 16 in the UCBC. Bracing would be need to be stiff enough to keep deflections down to 10% of 13” or 1.3”. This is L/166, which is comparatively low to most masonry design requirements, which are typically L/360 or higher, up to even L/600. Kariotis (1982) notes that the goal of a flexible vertical brace is to keep the brace elastic and provide a predictable restoring force during cracked excursions of the masonry wall. For diagonal braces, the UCBC encourages detailing to minimize vertical deflections.

*Diagonal braces loading vs. bracing the wall:* If the roof deflects downward on a diagonal brace, a horizontal reaction is imparted to the wall. One concern with diagonal braces is that vertical vibration of the roof in an earthquake can contribute to the out-of-plane inertial forces on the wall. This concern, combined with the difficulty of making the roof stiff enough for against vertical deflections, makes vertical bracing a preferred engineering choice over diagonal bracing.

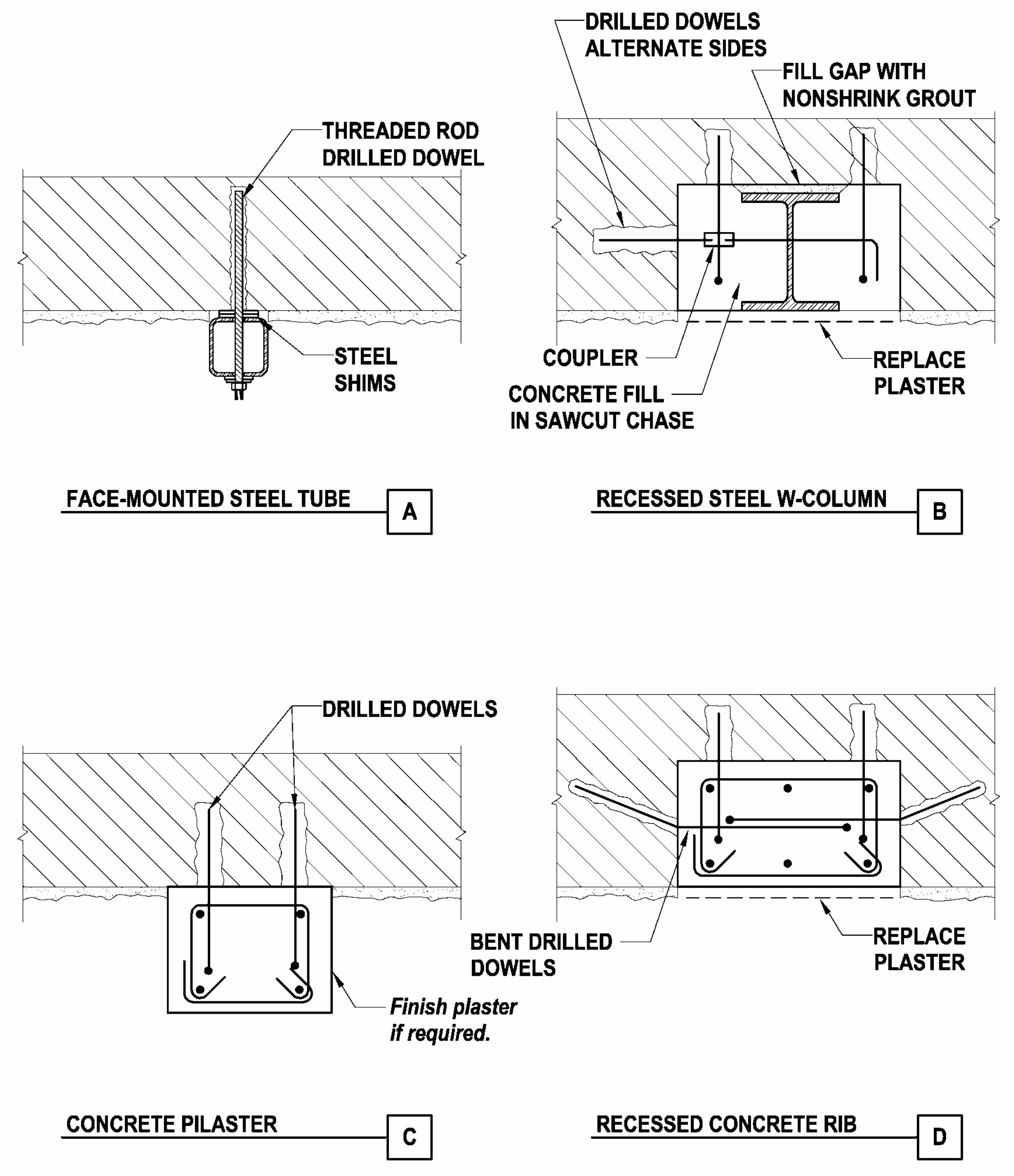



**Figure 21.4.3-2: Vertical Bracing Alternatives**

*Recessed steel and concrete and surface-mounted concrete:* Provisions in the 1997 UCBC and 2003 IEBC do not explicitly consider the approaches shown in Figures 21.4.3-2B, 21.4.3-2C and 21.4.3-2D. These approaches are unusual, but they can be used when a more sensitive aesthetic approach or higher loads are needed.

### *Detailing and Construction Considerations*

*Materials:* Braces are typically done with steel as shown in Figures 21.4.3-1 and 21.4.3-2A, but strongbacks can also be done with wood posts or with concrete pilasters (Figure 21.4.3-2C).

*Aesthetics:* Figure 21.4.3-1 shows exposed braces. This is the least expensive approach and is appropriate for certain occupancies. When there is architectural desire to hide the steel, the bracing can be furred at added cost and impact on the usable space. To minimize the impact on the space, the vertical brace can recessed into a cavity cut in the wall with either a steel or a concrete member. See Figure 21.4.3-2. Recessing the steel or concrete requires significantly more work and raises the potential for cracking to propagate from the inside of the recess to the masonry face.

*Strongback anchor spacing:* Figure 21.4.3-1B shows only a central anchor at midheight of the wall. Often demand/capacity ratios for anchorage to the wall with through bolts or drilled dowels (see Section 21.4.2) will dictate a tighter spacing of anchors.

*Floor/roof framing capacity:* Figure 21.4.3-1 shows anchorage to joists oriented perpendicular to the wall. When joists are parallel to the wall, the horizontal anchorage force must be developed out into the diaphragm. In Figure 21.4.3-1A, the existing roof beams may need to be strengthened to provide adequate strength to resist downward loading.

*Hollow masonry:* Figures 21.4.3-1 and 21.4.3-2 apply to solid masonry. When the existing masonry is hollow, alternative connection methods are needed. Figure 21.4.3-3 shows use of vertical concrete ribs. A chase is created by removing the face shell on one side of the wall. Reinforcing steel is added and then grout or concrete fill. There is typically insufficient space for ties. This approach is messy and noisy. Figure 21.4.3-4 shows an alternative where steel strongbacks are bolted to the wall with either drilled dowels or through bolts. The screen tube anchor of Figure 21.4.3-4A relies on mechanical keying action from the spreading adhesive to engage the face shell. The capacity is limited to the face shell of the masonry and can be quite low, in the low hundreds of pounds at allowable stress design levels. It is also nonductile as the failure mechanism is spalling of the face shell. The through bolt in Figure 21.4.3-4B provides increased capacity and locally grouting in the anchor provides additional capacity.

### *Cost/Disruption*

Diagonal bracing is usually less expensive, but is considered less reliable than vertical bracing. Furring can be used to cover the braces at added cost. Exposed braces are typically less expensive than more architecturally sensitive alternatives like recessed vertical braces or reinforced cores (See Section 21.4.4). Installation of bracing is fairly disruptive since it must occur around the entire perimeter; and it involves drilled dowels, and accessing and connecting to horizontal diaphragms.

### *Proprietary Issues*

There are no known proprietary concerns with bracing of URM walls.

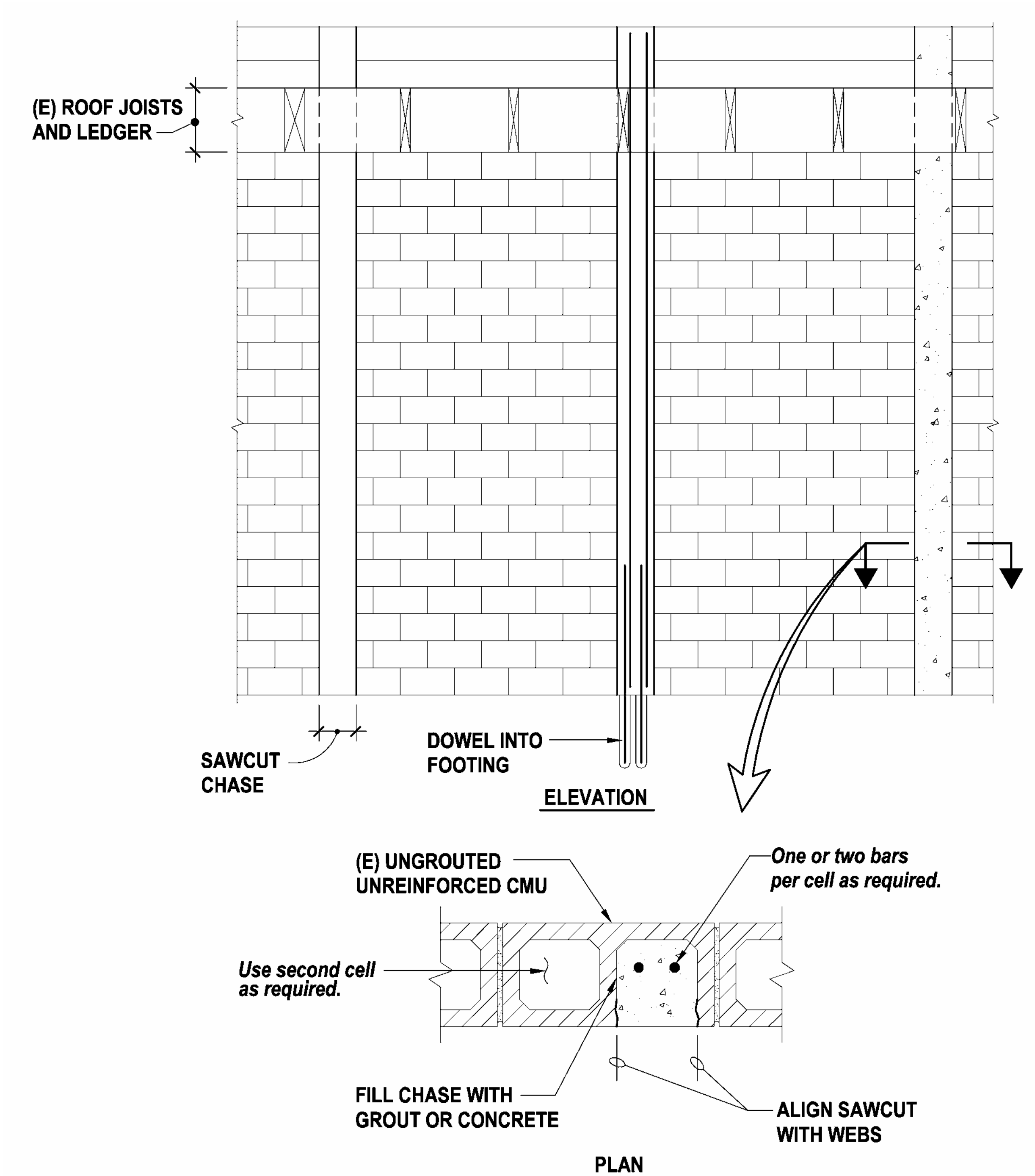



**Figure 21.4.3-3: Concrete Ribs in Hollow Masonry**

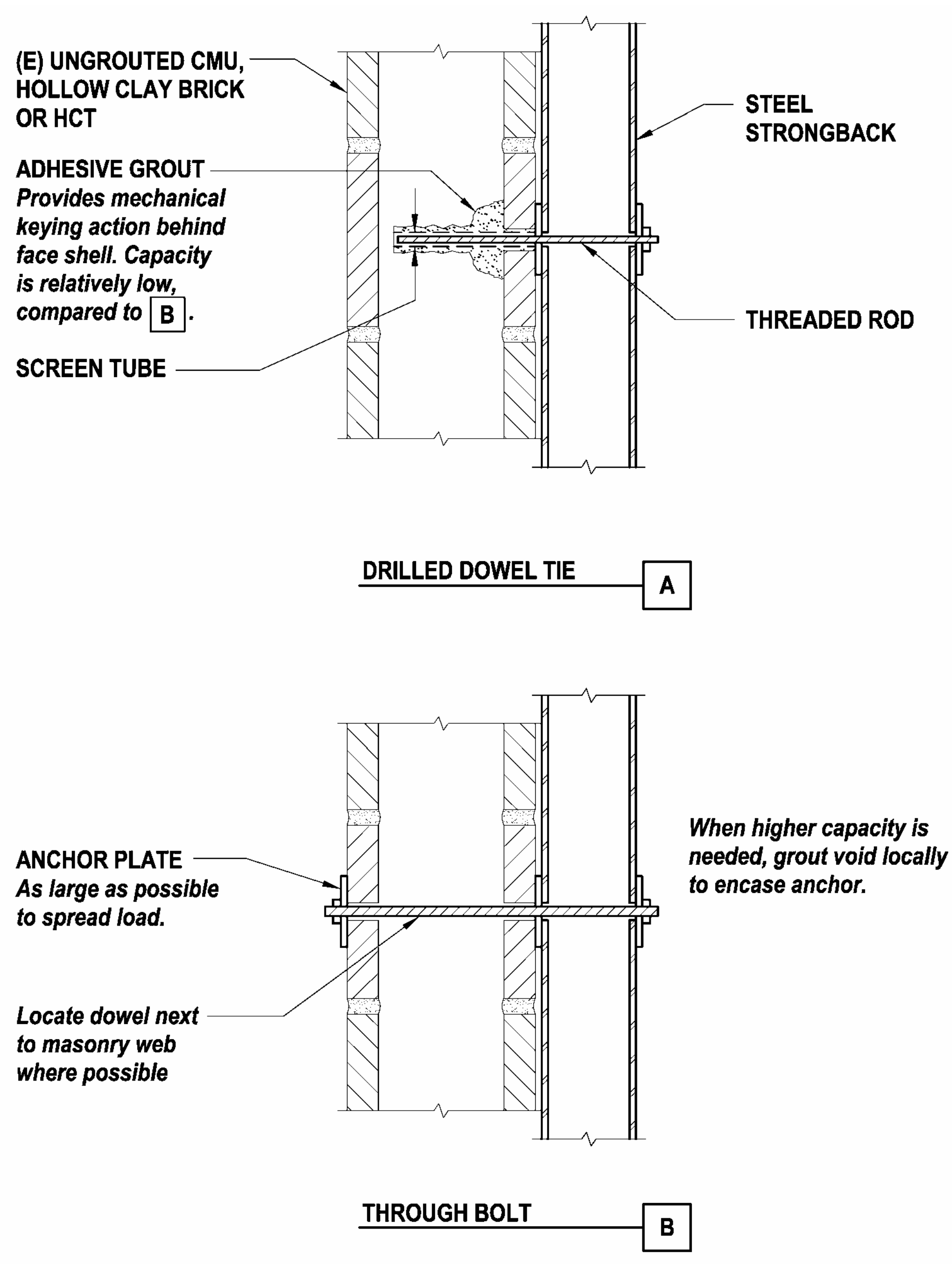



**Figure 21.4.3-4: Connection of Strongback to Hollow Masonry**

### 21.4.4 Add Reinforced Cores to URM Walls

***Deficiencies Addressed by Rehabilitation Technique***

Inadequate out-of-plane capacity and in-plane capacity of unreinforced masonry wall.

***Description of the Rehabilitation Technique***

Installing the reinforced core involves drilling a core from the roof down the inside of an unreinforced masonry wall. A steel reinforcing bar and grout are placed inside the hole to increase the wall strength. See Figures 21.4.4-1A and 21.4.4-1B. This process is used to avoid the aesthetic impact of exposed bracing described in Section 21.4.3.

***Design Considerations***

*Research basis:* The original research at CSU Long Beach and North Carolina State University for reinforced cores is summarized in (Plecnik, Cousins, and O'Conner, 1986) and Plecnik (1988). It covered both out-of-plane and in-plane loading. Subsequent vendor tests for in-plane loading were done at UC Irvine but have not been published. More recent in-plane testing is summarized in Abrams and Lynch (2001).

*Out-of-plane capacity:* When reinforced cores are used for enhancing out-of-plane bending capacity, the wall is analyzed as a reinforced masonry element. Some engineers have used a traditional allowable stress design code format for masonry. Another common approach is to use factored design methods. Plecnik, Cousins and O'Conner (1986) provided an ultimate strength design formulation. As with concrete design or typical reinforced masonry design, the compressive strength of the masonry, $f'_m$, is needed. Default values are available in FEMA 356, but it is important not to over-reinforce the masonry section and cause a brittle failure of the masonry, so it is often prudent to obtain the masonry strength.

*In-plane capacity:* Plecnik, Cousins and O'Conner (1986) tests showed significant increase in in-plane loading from the reinforced cores, but a design methodology was not provided. Breiholtz (1987) suggested using the test results as well, but did not provide a complete design methodology. One design approach is simply to extrapolate the test results on a per lineal foot basis. Another is to consider the vertical bars as the tie element in a strut-and-tie methodology, with diagonal struts in the masonry connecting the cores.

*Post-tensioned masonry:* Post-tensioned masonry has been used in a few instances. The goal of post-tensioning the bars can be to add compressive stress to the masonry wall to increase the effective shear stress, since unreinforced masonry shear capacity formulations, such as those in FEMA 356 or the ICBC, provided increased shear strength with higher compressive strength. Recent research by Rosenboom and Kowalsky (2004) provides some results from cyclic testing, but the specimens have central cavities filled with grout rather than solid multi-wythe brick, and design equations are not provided.

***Detailing and Construction Considerations***

Detailing and construction considerations for reinforced cores include the following.

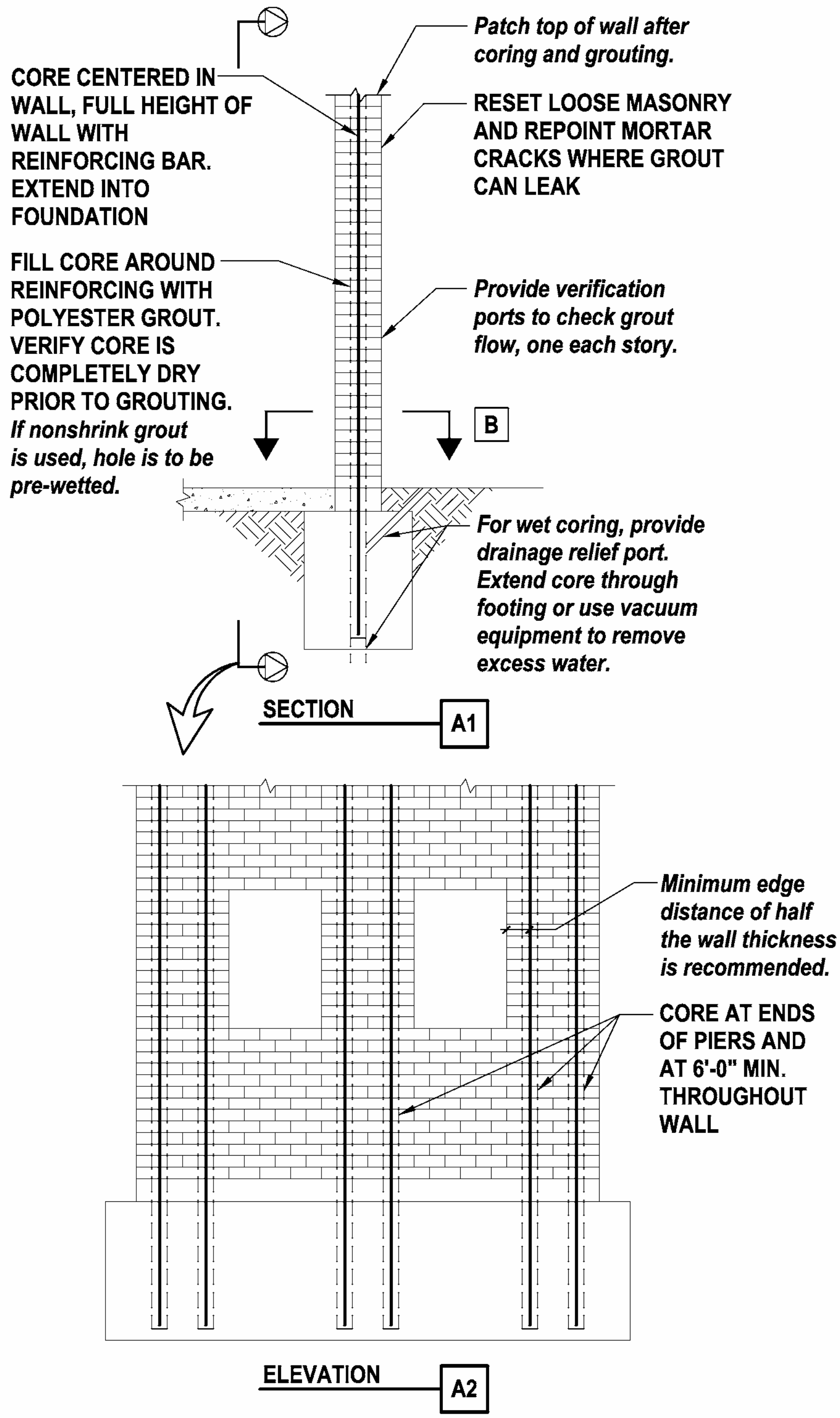



**Figure 21.4.4-1A: Reinforced Cores**

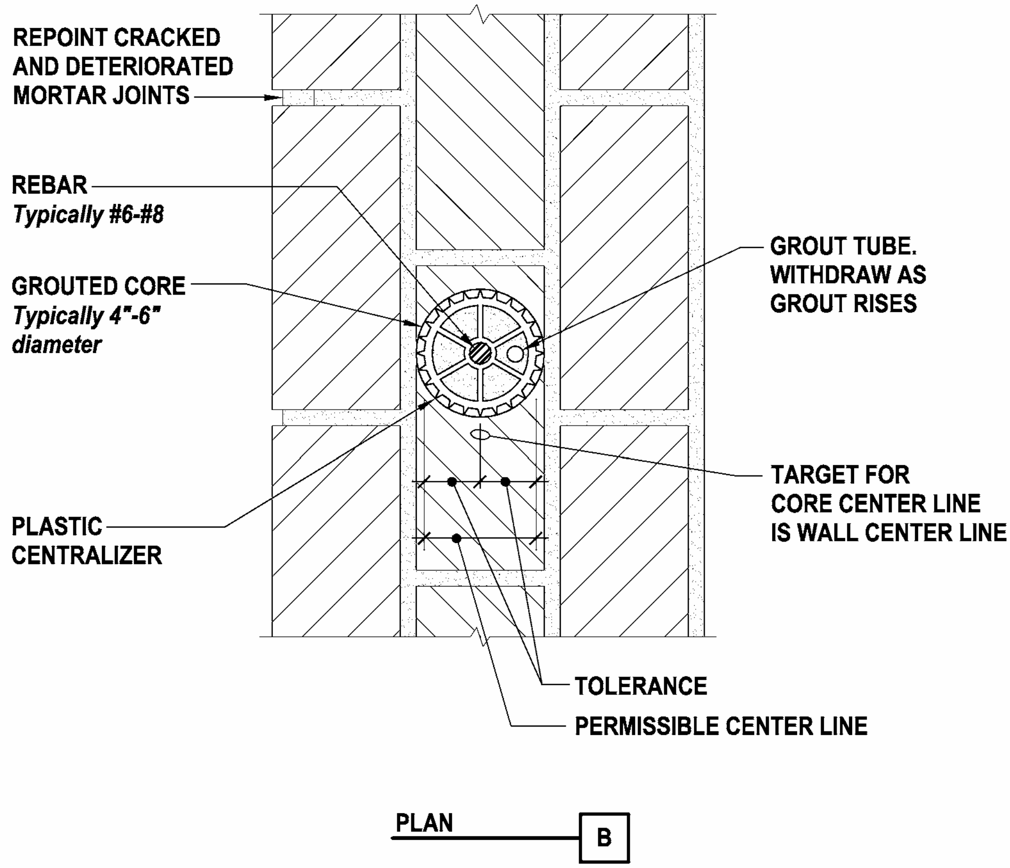



**Figure 21.4.4-1B: Plan Detail of Reinforced Core in Masonry Wall**

*Wet vs. dry drilling:* Traditionally, holes cored in masonry were done similarly to those in concrete, using diamond tipped coring bits cooled by water. The slurry created by the water and brick dust can lead to staining of sensitive surfaces. Reinforced cores gained popularity when drilling companies developed specialized drilling equipment that did not need water to cool the bit. In many cases, this involves coring drills that rotate quite slowly compared to traditional coring equipment. The material within the core comes out in cylindrical chunks or in small pieces of debris or dust that are vacuumed into debris containers.

*Reducing leaks:* To minimize leaks during wet drilling and during grouting, loose masonry should be repaired and cracks repointed. To limit the extent of repointing, consideration can be given to monitoring the location of dust clouds that escape cracks during dry drilling and repointing those locations.

*Obstructions:* Drilling progresses most rapidly if the masonry is neither too hard nor too soft and is relatively homogeneous. Encountering wood debris inside walls or metal veneer ties can slow or stop the drilling.

*Angled drilling:* Drilling is typically done from the top and straight down. Occasionally, special situations arise where angled drilling might be necessary. This can be done, but requires much greater skill from the driller.

*Hole diameter:* Hole diameters are typically about four inches, but can range from three to six inches.

*Drilling tolerances:* The wider and shorter the wall, the easier it is to drill because it provides better tolerance against drilling inaccuracy. Tolerances of about ±2” in reasonably tall walls are usually achievable. A simple way to check this is as follows. Say the hole is four inches in diameter. Attach a small penlight flashlight to the end of a string, creating a lighted plumb bob. Drop the plumb bob down from the center of the hole at the top. If it does not hit the side of the cored hole at any point on the way down, a two-inch tolerance has been met.

*Bar material type:* The reinforcing bars used in reinforced cores are typically regular ASTM A615 mild steel, as they are protected by the grout. For increased corrosion protection, stainless steel or epoxy coated rebar can be used. If post-tensioned center coring is done, high strength ASTM A722 threaded bars can be used.

*Bar size:* Bar size depends on the demand/capacity ratios, but typically ranges from #5 to #8.

*Centralizers:* In order to keep the rebar centered in the hole, a plastic centralizing wheel is used by some engineers. Others consider this an obstruction limiting the flow of grout.

*Grout type:* The original research by Plecnik, Cousins and O’Conner (1986) evaluated several grouts and concluded that a formulation using polyester grout provided the best dispersion into the masonry. Polyester has some offgassing concerns from styrene vapors and requires the hole to be dry before installing the grout. It also does not have the long-term track record of other more widely used grout materials. Some engineers, as a result, use a high quality nonshrink cementitious grout. With cementitious grouts, the hole needs to be prewetted prior to grout placement.

*Grouting process:* Center coring can be done in multistory buildings, so the depth of holes can get quite large. A tremie grouting technique can be used to assure placement of grout. A grout tube can be tied loosely to the bar and/or centralizer(s) as the bar is lowered into the hole. As grout is pumped into the hole, the tube is slowly withdrawn as the level of grout rises.

*Verification ports:* Grout will leak into voids in the masonry. To confirm that the grout is rising in the core, horizontal holes can be drilled into the wall. When the grout reaches the port, the port is plugged, and the grout is allowed to continue to rise to the top of the core.

*Bottom of holes:* Coring usually goes into the foundation. In a concrete foundation, there will not be any place for water used to cool the bit in wet coring to escape. The hole can be vacuumed out, or the core can be continued all the way down to the bottom of the foundation.

*Post-tensioning:* When reinforced cores are post-tensioned, several additional issues come into play. First, grouting is done in two stages. The first stage is in the foundation where the post-tensioning is anchored. After the grout cures and the bar is stressed, the second stage of grouting occurs up to the top of the hole. Post-tensioning vendors provide proprietary anchorage hardware for the bar at the top of the wall. A concrete cap or bond beam may be necessary or desirable to distribute the load on the top of the wall to reduce the stress on the masonry.

*Access to top of wall:* The top of the wall must be accessible to drilling equipment. Temporary scaffolding or work platforms will usually need to be erected adjacent to the hole. Bracing of drilling equipment back to the wall and a point in the roof is necessary to keep the drill plumb.

*Spacing:* Reinforced core spacing will depend on demand/capacity ratios, but a minimum spacing of six to ten feet is desirable.

#### *Cost/Disruption*

Adding reinforced cores can be considerably more expensive than exposed bracing, so it is usually only performed in historically and architecturally sensitive buildings. Since the core is placed inside the wall, the disruption to interior and exterior faces is limited to sealing cracks, access to the roof to place the drilling equipment, and drilling noise and vibration.

#### *Proprietary Issues*

Research for reinforced cores is in the public domain. Some drilling contractors reportedly have patents on certain types of proprietary drills. Some of the terms used with the process have been trademarked by some of the first engineers to implement the technique. As a result, the generic term “reinforced core” is used in this document.

### 21.4.5 Add Concrete Overlay to Masonry Wall

#### *Deficiency Addressed by Rehabilitation Technique*

Improving inadequate in-plane wall capacity is the primary purpose of a new concrete overlay, but the concrete can also improve inadequate out-of-plane bending capacity.

#### *Description of the Rehabilitation Technique*

New concrete is applied against an existing unreinforced masonry wall to increase the shear capacity of the wall. The new concrete is attached to the old wall with adhesive anchors and can either be cast-in-place concrete or sprayed-in-place. In rehabilitation work, sprayed concrete, known as shotcrete, is more commonly employed than cast-in-place construction, since the existing wall provides the back-side form. The thickness of the new concrete varies with strength requirements, but it is usually from four to 12 inches. See Figure 21.4.5-1.

#### *Design Considerations*

*Research basis:* A fair amount of research in the use of shotcrete overlays on masonry has been done. A summary is given in El Gawady, Lestuzzi and Badoux (2004). Early diagonal tension testing was done by Kahn (1984), and more recently static cyclic tests were done by Abrams and Lynch (2001). Kahn (1984) showed significant increases in strength from the shotcrete and that adding drilled dowels between the shotcrete and masonry or an epoxy bonding agent did not lead

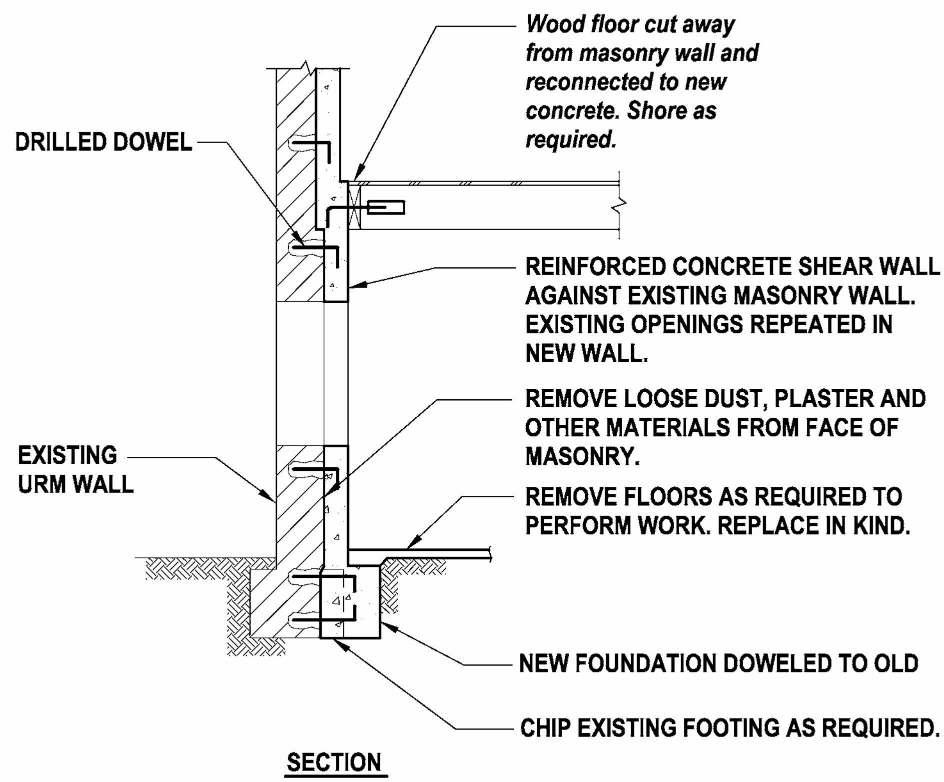



**Figure 21.4.5-1: Concrete or Shotcrete Wall Overlay**

to significant improvements. A saturated masonry surface was recommended. Testing by Abrams and Lynch (2001) aimed at increasing the shear capacity to lead to flexural yielding of the tension bars in the shotcrete. Strength increased by about a factor of 3, but displacement capacity did not increase.

*Design criteria:* When a concrete overlay is used, there are several common force-based design approaches for the wall, due to the relatively high strength of the concrete compared to the masonry. One is to take 100% of the demand tributary to the strengthened wall line in the concrete overlay itself and ignore the masonry. While this may sound conservative, it can mean that the masonry will be significantly damaged before the concrete ever sees the majority of its design load. Another approach is to share the load, by relative rigidity, between the masonry and the concrete. When this is done, both the masonry and the concrete must be checked to confirm they are not overstressed. The most conservative approach is to use the overlay to resist 100% of the tributary load, but to also check that the masonry can resist the loads it will actually attract. Displacement-based design approaches inherently consider the relative rigidity of the concrete and masonry, but they are less commonly employed.

*Discretely applied overlays:* The URM walls to which the overlay is applied are typically punctured with window and door openings. It can be tempting to apply the concrete to wide piers. The comparatively high strength of the concrete means it can take high loads, but it is unlikely to have sufficient stiffness to actually attract the load it was intended to take. Eventually, though, when the masonry cracks, the load will find the concrete, but this can lead to significant cracking at the ends of the concrete in the masonry spandrels. See Figure 21.4.5-2A. This can be addressed by spreading out the influence of the overlay by using top and/or bottom spandrels or grade beams, such as shown in Figure 21.4.5-2B. Alternatively, drilled piers can be placed at the ends of the new walls to add stiffness, as in Figure 21.4.5-2C. Or most simply, a continuous overlay can be used, enabling a reduced thickness and consistent finish surface, as shown in Figure 21.4.5-2C.

*Collector load transfer pathways:* With discretely applied overlays in the field of the masonry wall, the question arises of how the load in the floors and the masonry wall will reach the overlay. Some engineers ignore this issue and assume the masonry wall will serve as the collector. Others provide an explicit steel collector in the edge of the diaphragm or a bond beam on top of the wall.

*Out-of-plane load resistance:* When the overlay is added to the wall, its additional inertial load must be considered in the seismic weight of the structure. Out-of-plane anchorage requirements at the overlays are thus larger, and wall-to-diaphragm tie spacing often decreases there.

### *Detailing and Construction Considerations*

Detailing and construction considerations for concrete overlays include the following.

*Drilled dowel spacing:* It is common practice to connect the overlay to the masonry wall with drilled dowels. The drilled dowels transfer shear between the two materials, and they also serve as out-of-plane ties for the masonry. Spacing of two feet to three feet on center is typical. Good detailing involves drawing an elevation and showing the location of dowels around openings since the nominal spacing will typically change there.

*New foundation at the base of wall:* The base of the shotcrete can be set on the ledge of the existing footing if conditions permit or a new footing can be provided. The added load from the shotcrete and distribution of stresses on the existing footing must be considered. See Section 23.6.2 for issues involved in adding a new footing next to an existing footing.

*Interface between the wall and diaphragms:* Where the new wall meets the existing floors is usually the location where special consideration must be given to detailing. Conditions where the floor joists are parallel to the wall are the easiest to address. The first and/or second joists in from the masonry are removed to install the overlay, a ledger is placed back, and the floor sheathing run up to the ledger. If shotcrete is used, sufficient clearance must be provided to avoid shadowing during spraying. This should be checked as part of the preconstruction test panel.

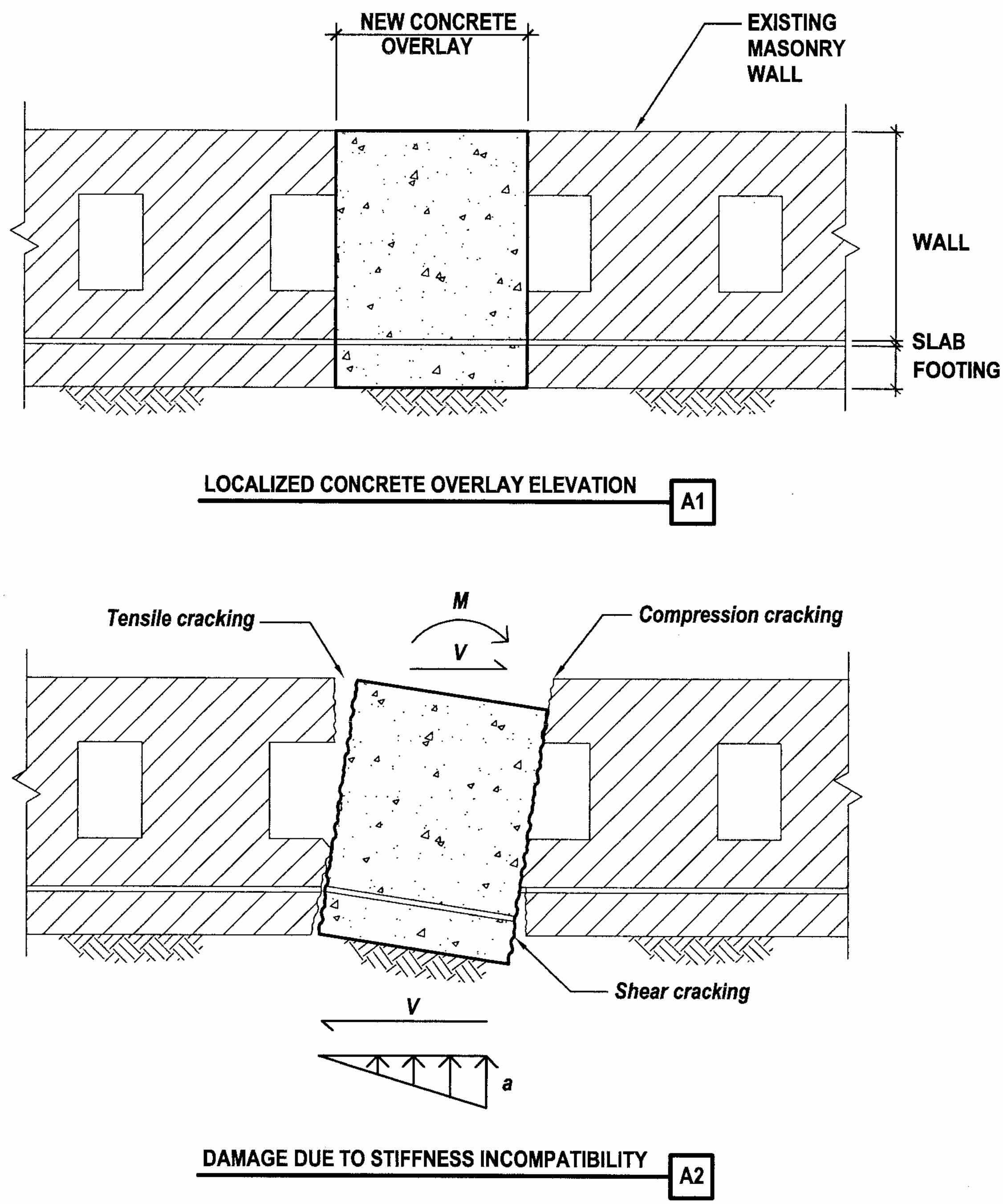



**Figure 21.4.5-2A: Potential Damage at Ends of Narrow Concrete Overlay**

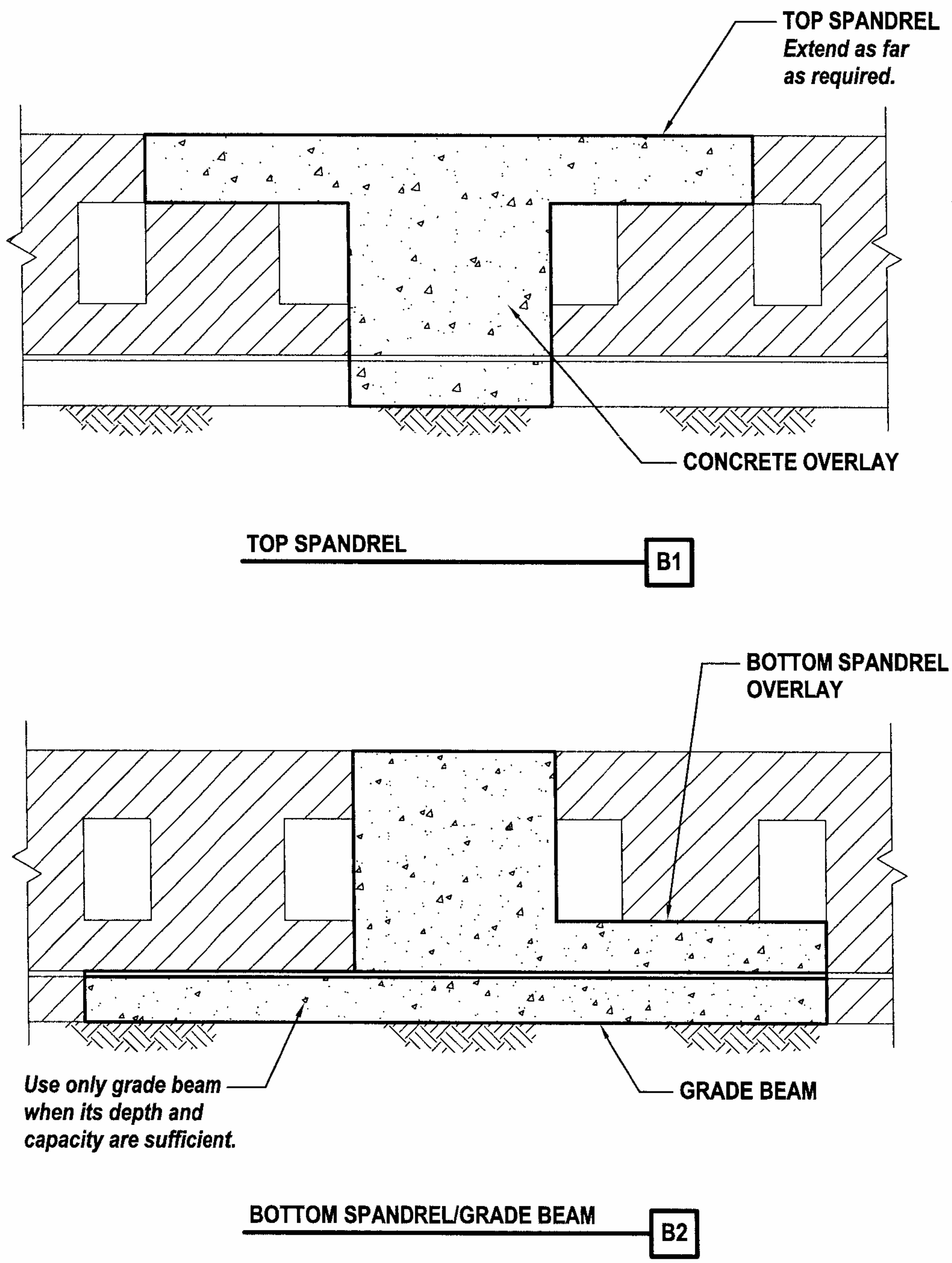



**Figure 21.4.5-2B: Alternatives to Distribute Overturning Loads in Concrete Overlay**

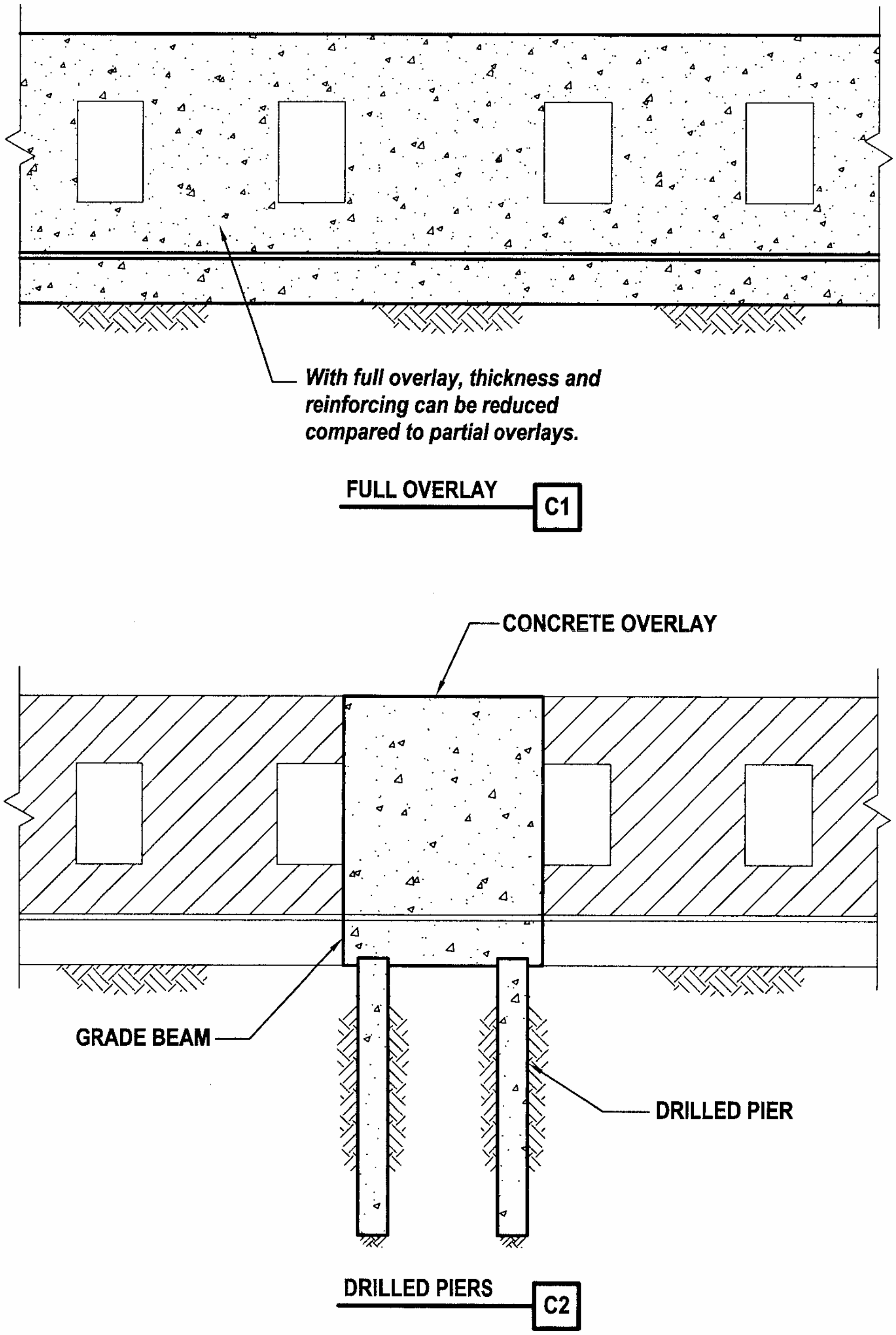



**Figure 21.4.5-2C: Alternatives to Distribute Overturning Loads in Concrete Overlay**

When the joists are perpendicular to the wall, they can be cast into the wall, but when this is done, the joists should be treated with preservative, an air gap provided on the top and sides of the joists and building paper placed on the bottom to minimize moisture entering the lumber. Special rebar detailing will be needed to transfer shear from above the floor to below the floor through the weakened area of the joists. Given these issues, the perpendicular joists are often headed off and supported off a ledger on the face of the new wall. This requires shoring, but simplifies the remaining construction and provides for a better wall. This approach is shown in Figure 21.4.5-1.

*Pre-wetting the masonry wall:* Cast-in-place walls and, to a lesser extent, shotcrete walls have moisture in them during placement that will be absorbed by the masonry. Some engineers require wetting the masonry wall, just prior to placement.

*Curing considerations in existing building:* Curing concrete emits moisture. If the building has finishes that are sensitive to moisture emission, precautions will need to be taken to protect the finishes. This can be particularly critical if shotcrete is used. Curing of the face of the concrete is either done with curing compound, continuous spraying or a moisture-retaining cover. Finding a curing compound that will later be acceptable for certain adhered finishes can be difficult. Continuous spraying, however, adds substantial moisture to the interior space.

*Efflorescence concerns from additives and moisture:* Like all concrete, overlays can be susceptible to alkali salts leaching to the surface, usually leading to white streaks or spots. These stains can come from additives within the concrete or from salts within the masonry wall. Use of low-alkali concrete is recommended, and additives should be limited to those known not to lead to efflorescence.

*Protection of existing masonry substrate:* The main advantage of shotcrete is that the existing masonry serves as the backside form and forming the front is unnecessary. In typical stone masonry and brick masonry situations, the wall will be adequate to serve as the backside form. When the wall is particularly thin or in poor condition, the contractor will have to take care to brace the masonry to resist the force of the applied shotcrete.

***Cost/Disruption***

Adding new concrete, particularly with shotcrete, can be quite disruptive. Where access is sufficient, shotcrete is typically chosen as it is less expensive than cast-in-place work which requires front-side formwork.

*Shotcrete:* Placing shotcrete requires access for the hose and concrete truck and sufficient room (several feet) to spray the concrete. It is desirable to shoot downward, so scaffolding is needed at upper portions of walls to achieve the necessary angle. Spraying is noisy and very dusty. If an indoor wall is being shot, the room will usually be sealed off with plastic sheeting to control dust. During shooting, residue—known as rebound—forms at the base of the shoot and must be cleaned away so that it does not become part of the overlay. Protection against rebound on existing floor and wall surfaces is needed.

*Cast-in-place concrete:* Placing cast-in-place concrete also requires access for the hose and concrete truck. Less front-side access is needed than shotcrete, but a front-side form is required with the associated sawing and hammering noise of construction. Concrete placement is noisy, and in addition to workmen and concrete truck noise, there is the vibrator used to consolidate the concrete.

***Proprietary Issues***

There are no known proprietary concerns with shotcrete or cast-in-place overlays on existing masonry walls.

### 21.4.6 Add Fiber-Reinforced Polymer Overlay to Masonry Wall

***Deficiency Addressed by Rehabilitation Technique***

Improving inadequate in-plane wall strength is the primary purpose of a new fiber-reinforced polymer (FRP) overlay, but the overlay can also improve out-of-plane bending capacity.

***Description of the Rehabilitation Technique***

An FRP overlay, typically made of glass or carbon fibers in an adhesive matrix, is applied against an existing unreinforced masonry wall to increase the shear strength of the wall. The existing wall surface must be prepared to receive the new material, and after application the fiber composite must be protected against ultraviolet rays.

***Design Considerations***

*Research basis:* Research in fiber composites is extensive, but has primarily been focused on enhancement of concrete elements or reinforced concrete masonry. There is, nonetheless, a growing body of research for unreinforced masonry strengthening. A partial listing of some papers is given here.

For out-of-plane strengthening of unreinforced masonry, tests include Reinhorn and Madan (1995a) on clay brick; Portland State University (1998) on hollow clay tile; Vandergrift, Gergely, and Young (2002) on hollow concrete masonry; Tumialan, Galati, Namboorimadathil, and Nanni (2002) on surface applied fiber reinforced bars to hollow concrete masonry; Tumialan, et al. (2002) on glass and aramid fiber reinforced polymer composites on both concrete and clay brick; Tumialan, Galati, and Nanni (2002) on situ field tests in an infill frame building being demolished of brick and clay tile walls strengthened using glass fiber strips; Ehsani, Saadatmanesh, and Velazquez-Dimas (1999); and Velazquez-Dimas, Ehsani, and Saadtmanesh (2000) on half-scale clay masonry.

In-plane testing for clay brick masonry includes Reinhorn and Madan (1995b); Ehsani and Saadatmanesh (1996); Ehsani, Saadatmanesh, and Al-Saidy (1997); Haroun and Mosallam (2002); Senescu and Mosalam (2004). Elgwady, Lestuzzi and Badoux (2003) performed dynamic tests on slender and squat hollow clay masonry piers with and without aramid, glass, and carbon fiber composite overlays. Vandergrift, Gergely, and Young (2002) performed tests on half-scale hollow concrete masonry. Schwegler and Ketterborn (1996) discusses in-plane masonry strengthening with carbon fiber, including straps at various angles of orientation. Reinhorn and Madan (1995b) found about a 120% increase in strength from the composite and some additional displacement capacity in a one-cycle reversed cyclic test. After the fiber ruptured, however, masonry cracks immediately widened in a brittle manner. Haroun and Mosallam (2002) found a minor increase (about 20%) in strength capacity and about a 30% increase in displacement capacity. Cracking in the masonry was the ultimate limit which occurred after the composite debonded. Senescu and Mosalam (2004) used monotonic diagonal tension tests with mixed results, some showing improvements in strength and displacement capacity, others actually showing reductions in strength and displacement capacity. Paquette, Bruneau, and Brzev (2004) and Paquette and Bruneau (2004) investigated strengthening or repairing a one-story building with rocking-critical piers using fiber composite chord strips at ends of piers.

*Design basis:* There are no code guidelines or FEMA 356 provisions explicitly addressing FRP overlays on unreinforced masonry. Information can be obtained from manufacturer literature and can be used in conjunction with criteria in the ICC-ES interim standard (ICC-ES, 2003). They focus only on the design of the fiber itself, not the fiber and masonry combined performance. Velazquez-Dimas and Ehsani (2000) provide modeling and design recommendations for out-of-plane strengthening.

*Behavioral mode:* It is important to understand the underlying governing behavioral mode of both the unstrengthened unreinforced masonry wall and the strengthened wall. Fiber composites have little ductility. Adding an FRP overlay to a rocking critical wall pier may be able to reduce cyclic degradation of the pier, but will not change the strength or behavior mode if the fiber is only applied to the pier. If the fiber crosses the top and bottom of the pier into the spandrel, in can inhibit or prevent formation of a rocking mode. Strength will be increased, but ductility will be reduced from that of a rocking-critical mode to that of a shear critical mode.

### *Detailing and Construction Considerations*

Detailing and construction considerations for FRP overlays include the following.

*Surface preparation:* The surface of the masonry needs to be cleaned of loose material and finishes that prevent proper adhesion. Sandblasting of the masonry is not usually necessary; a wire brush is used instead. See Figure 24.4.6-1.

*Complete overlay vs. strips:* Both in research and in practice, both complete overlays over the full surface of the wall and use of strips are found. Vertical strips are used when only improvement to out-of-plane resistance is needed. Diagonal strips have been used to resist diagonal tension stresses from in-plane shear.

*One side or both sides of wall:* Applying fiber to both sides of the wall improves performance, particularly for out-of-plane resistance, but testing has been performed with fiber on only one side.

*Continuity of fiber at top and bottom:* Providing load transfer with the fiber can challenging, particularly at floor-to-wall interfaces. If the fiber is being used to resist out-of-plane loads and is transferring these loads back into the floor diaphragm, special details may be needed to turn the vertical fiber overlay into the horizontal diaphragm. Fiber cannot be bent at 90 degrees;

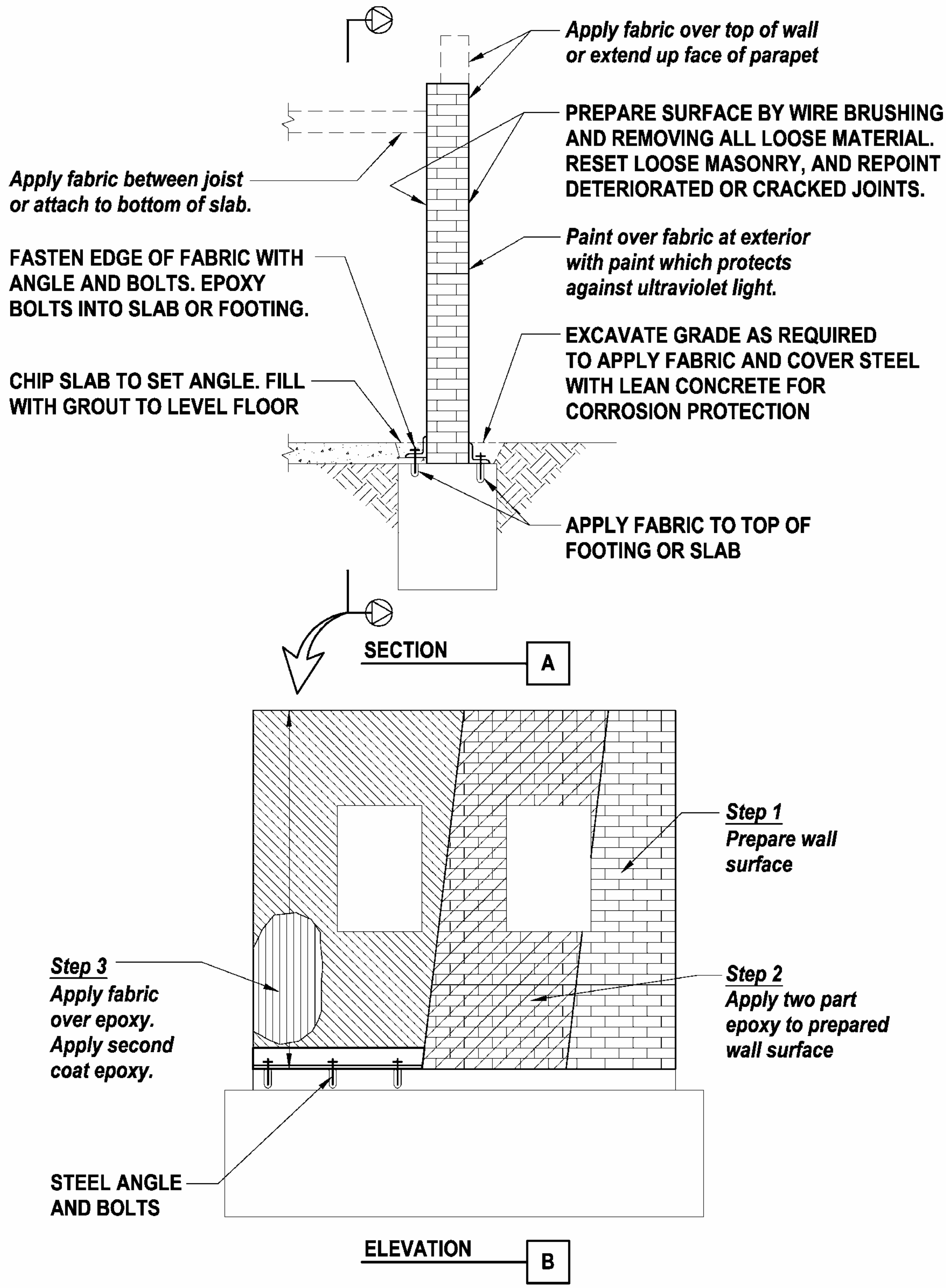



**Figure 21.4.6-1: Fiber Composite Wall Overlay on URM Wall**

rather a radius is needed. Sometimes, steel reinforcing plates are used to stiffen the turns. If the fiber is being used to transfer in-plane loads from one story to the next, continuity past the floor is needed and will require special details as does shear transfer out of the diaphragm into the wall

*Moisture barrier:* Fiber composites are impermeable. If continuous overlays are used, moisture transmission through the masonry wall will be stopped at the fiber. Eventually, the concern would be the moisture would build up and begin to delaminate the fiber bond and lead to general building concerns with excessive moisture.

*Additional information:* See Section 13.4.1 for more detailed discussion on FRP issues including:

Composite makeup and application
Mechanical properties
Fiber and mechanical anchors
Durability
Constructability

***Cost/Disruption***

Fiber composites are relatively expensive as an application for masonry wall strengthening and have not seen significant use. Disruption comes from the sandblasting and surface preparation of the masonry wall, the fumes from the adhesives used in application of the fiber composite and removal and access requirements where the fiber transitions from story to story at the floor levels.

***Proprietary Issues***

Fiber composite materials are supplied by vendors. Capacities and design methods vary depending on the vendors.

### 21.4.7 Infill Opening in a URM Wall

***Deficiency Addressed by Rehabilitation Technique***

Inadequate unreinforced masonry in-plane wall strength.

***Description of the Rehabilitation Technique***

Window and door openings are filled to increase the shear capacity and reduce the shear stresses on the unreinforced masonry wall. The opening is typically filled with concrete, reinforced concrete masonry units, or reinforced clay brick, rather than with unreinforced masonry due to code concerns with adding unreinforced masonry. To provide adequate shear transfer between the existing wall and the new infill, the interface can be toothed, but more typically, drilled dowels are used. See Figure 21.4.7-1.

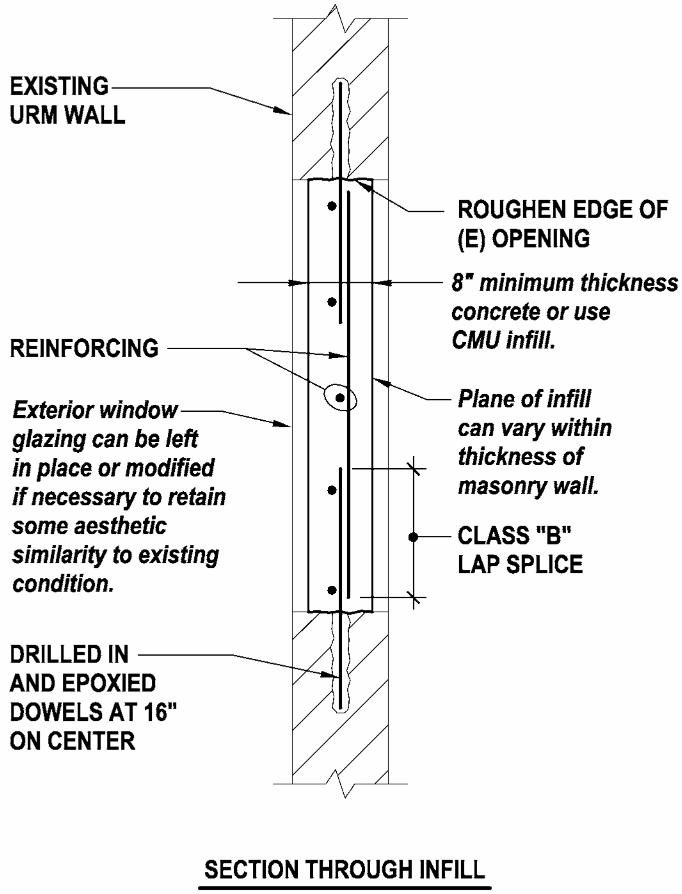



**Figure 21.4.7-1: Infilling an Opening in a URM Wall**

### *Design Considerations*

*Research basis:* Research that directly addresses testing of infilled openings has not been identified.

*Capacity of infill:* The typical infill materials are stronger and potentially stiffer than the surrounding masonry. It is typical, however, to consider the infilled wall as solid unreinforced masonry in determining shear capacity of the composite wall. Since the increase in shear capacity from the infill is often not substantial, it is done when only moderate increases in capacity are needed.

*Behavioral mode:* It is important to understand the underlying governing behavioral mode of both the unstrengthened unreinforced masonry wall and the infilled wall. Infilling openings could change a rocking-critical wall line to a shear-critical wall line, which may be less desirable.

***Detailing and Construction Considerations***

*Aethestics:* Infilling openings can obviously have a significant visual impact. Sometimes new concrete is used and is set back from the exterior face so that a window can be placed. Lighting can be added between the glazing and infill to mitigate the opacity of the infill.

***Cost/Disruption***

Infilling an opening is relatively inexpensive if no architectural treatment is done to the face. Disruption is also more localized compared to other in-plane wall strengthening methods like concrete and fiber reinforced polymer overlays.

Noise will occur during drilling holes for drilled dowels and placing the infill.

***Proprietary Issues***

There are no proprietary concerns with infilling masonry wall openings.

### 21.4.8 Add Concrete or Masonry Shear Wall (Connected to a Wood Diaphragm)

***Deficiency Addressed by Rehabilitation Technique***

A new concrete or masonry wall provides additional global strength and stiffness, reduces demands on existing masonry walls and can reduce demands on diaphragms by cutting tributary spans.

***Description of the Rehabilitation Technique***

The new wall should be properly designed to meet current code detailing provisions. This section focuses on detailing at the interface between the new wall and the existing wood diaphragm. Figure 21.4.8-1 shows sample concepts both for joists parallel to the wall and joists perpendicular to the wall at an interior floor location; Figure 21.4.8-2 shows similar concepts at a roof.

***Design Considerations***

*Research basis:* Research specific to ledger connections between wood diaphragms and concrete or masonry walls has not been identified.

*Capacity:* The connection can be designed either for the code level demands or to develop the diaphragm, depending on where inelastic action is intended to occur.

***Detailing and Construction Considerations***

Detailing and construction considerations for connecting a new wall to an existing wood diaphragm include the following.

*Masonry vs. concrete:* Masonry is usually considered quicker to install and less expensive; concrete (or shotcrete) is stronger and stiffer and usually considered to have better earthquake performance. Making connections from concrete to wood diaphragms can be easier than with masonry.

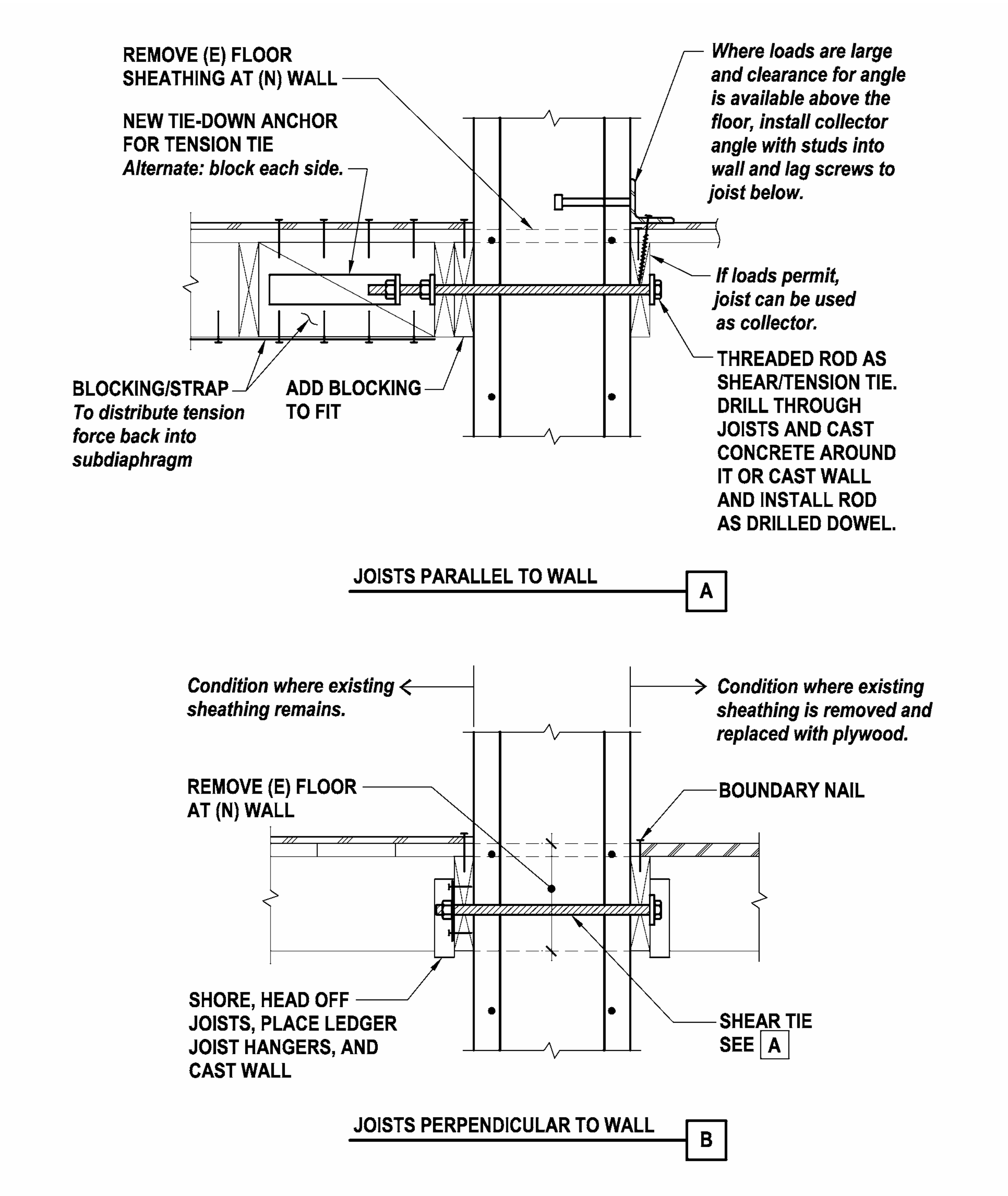



**Figure 21.4.8-1: Connecting a New Concrete Wall to an Existing Wood Floor**

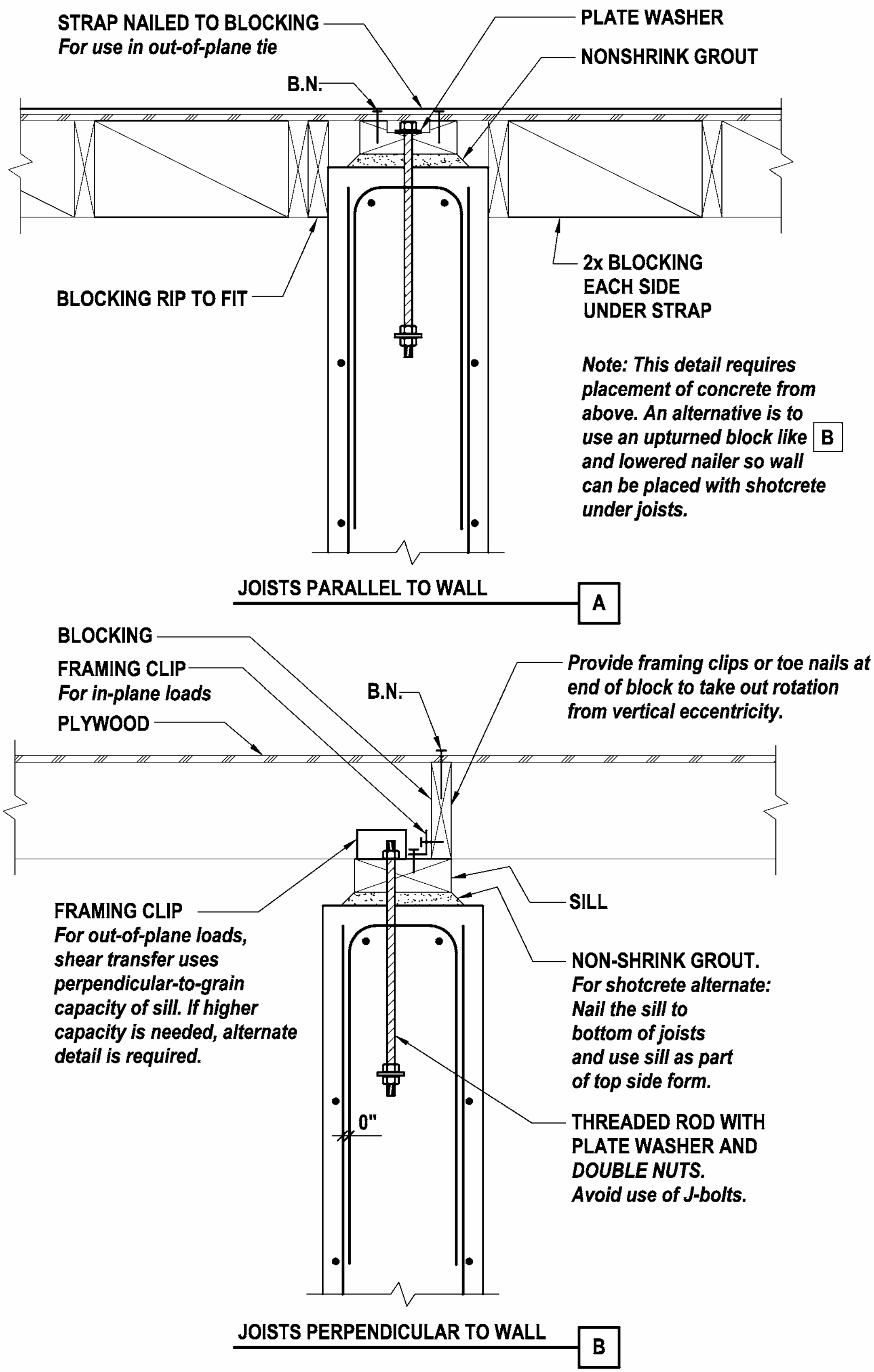



**Figure 21.4.8-2: Connecting a Concrete Wall to an Existing Wood Roof at Interior**

*In-plane shear transfer:* In Figure 21.4.8-1, shear transfer from the diaphragm to the wall goes from the diaphragm boundary nailing to the ledger and through the threaded rod into the wall. A tight fit on the rod and ledger is needed. The ledger should be dry dimensional lumber or glulam material to minimize vertical shrinkage of the ledger. When the wall is not as long as the diaphragm (a very common occurrence), a collector attachment into the wall will be needed. Figure 21.4.8-1A shows a steel angle with headed studs cast into the wall and diaphragm-to-collector connections using lag screws. The steel could go above or below the floor. When loads are relatively low, wood members such as the ledger can be used as the collector.

*Out-of-plane tension transfer:* In Figure 21.4.8-1, tension transfer of wall loads goes into the tie-down anchor, into the blocking, through straps in the blocking to additional blocks as required and eventually back into the diaphragm. Alternatively, blocking for a bay or two can be placed on both sides and out-of-plane resistance accomplished by compression bearing on the diaphragm joists.

*Joist direction:* When the wall can be fit in between existing joists, the amount of labor is reduced. When joists are perpendicular to the wall, the joists are typically headed off on each side of the wall to allow the wall to pass through. This requires temporary shoring of the floor around the wall. At the top of the wall, the wall can stop just under the joists and be blocked up to the diaphragm for shear transfer.

*Shotcrete vs. cast-in-place concrete:* See Section 21.4.5 for discussion of shotcrete vs. cast-in-place concrete issues.

#### *Cost/Disruption*

See Section 21.4.5 for discussion of cost and disruption issues.

#### *Proprietary Issues*

There are no proprietary concerns with connecting a concrete or masonry wall to a wood diaphragm.

### 21.4.9 Add Steel Moment Frame (Connected to a Wood Diaphragm)

#### *Deficiency Addressed by Rehabilitation Technique*

A new moment frame provides additional global strength, reduces demands on existing masonry walls and can reduce demands on diaphragms by cutting tributary spans.

#### *Description of the Rehabilitation Technique*

When a moment frame is added into a **URM** building, it typically goes either just behind a highly punctured street front façade or at an interior location within the diaphragm. Figure 21.4.9-1 shows the perimeter condition; Figure 21.4.9-2 shows interior conditions. A moment frame retrofit at a **W1A** building with a soft story is discussed in Chapter 6.

#### *Design Considerations*

*Research basis:* New steel moment frame issues are covered by FEMA 350 (FEMA, 2000). The CUREE woodframe project report on tuckunder building testing (Mosalam, et al., 2002) documents quasistatic component testing of moment frame to wood diaphragm connections and full-scale testing of a three-story tuckunder apartment building rehabilitated with a ground story moment frame on the open front side.

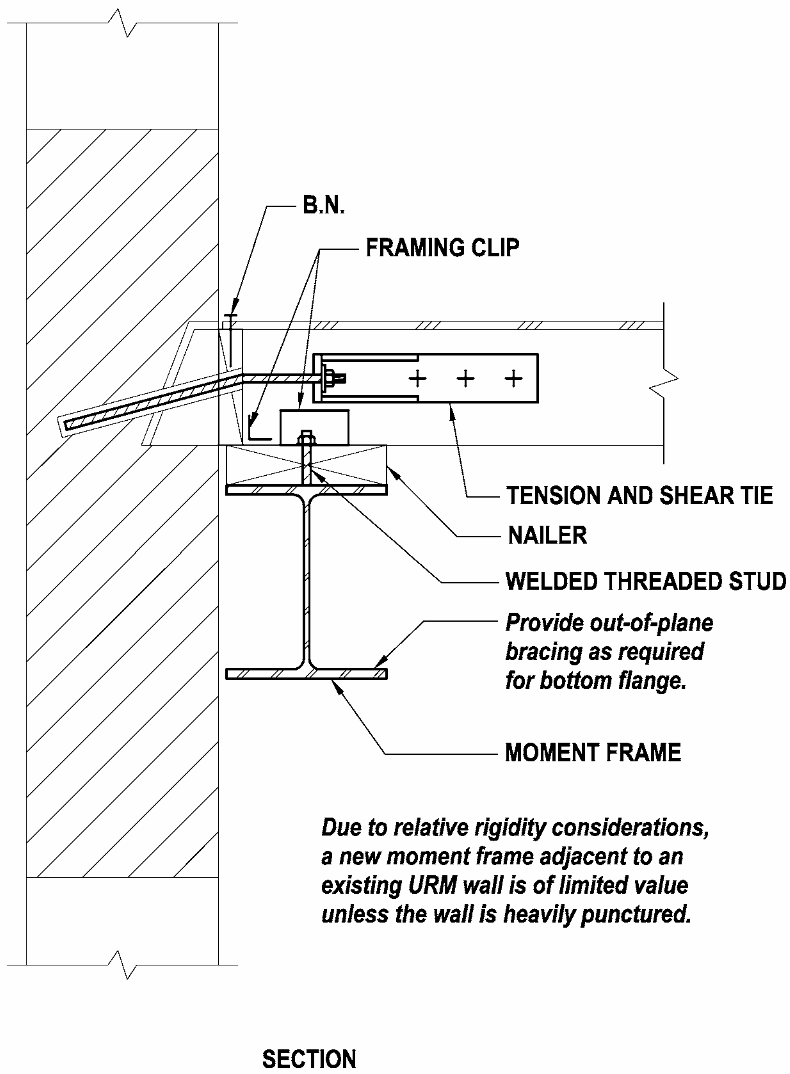



**Figure 21.4.9-1: New Perimeter Steel Moment Frame to an Existing Wood Floor**

*Stiffness considerations:* At either the perimeter or interior condition, reasonable stiffness of the frame is desirable. At the perimeter, minimizing the amount of drift and resulting masonry façade cracking is desirable. At the interior, if the moment frame does not have sufficient stiffness, the diaphragm will span between the end walls with the moment frame taking out relatively small loads due to its flexibility.

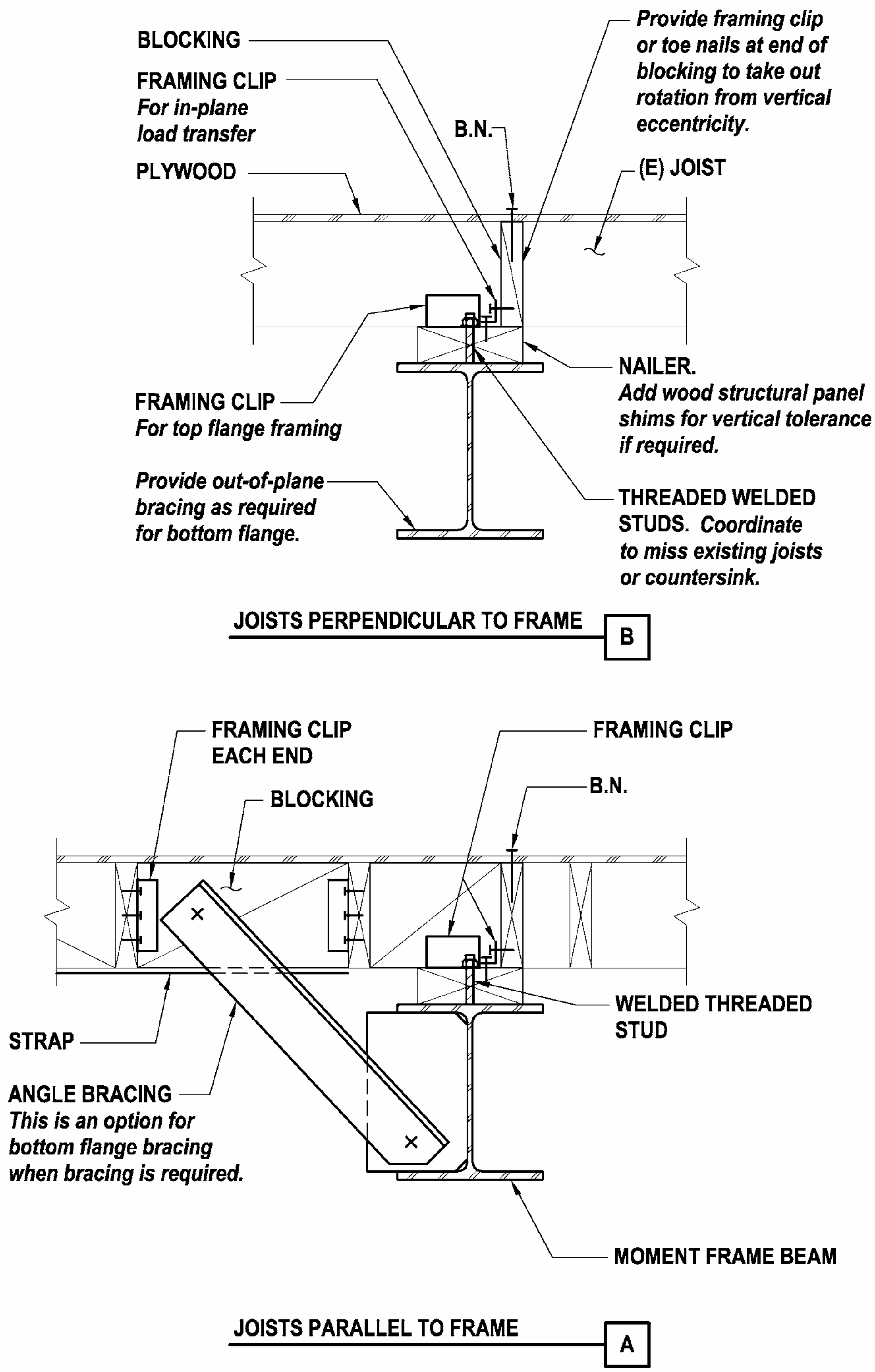



**Figure 21.4.9-2: New Interior Steel Moment Frame to an Existing Wood Floor**

*Design forces:* The new moment frame design can be governed by either stiffness or strength. Strength demands can either be minimum design loads or in some cases the moment frame can be designed to be stronger than the diaphragm so inelastic action happens in the diaphragm. For connection design of the frame to the diaphragm, it is particularly desirable to make sure the connections are stronger than the weaker of the diaphragm or the moment frame.

*Pinned base:* To minimize foundation demand requirements, new moment frames in retrofits are often designed with pinned bases.

### *Detailing and Construction Considerations*

Detailing and construction considerations for connecting a new moment frame to an existing wood diaphragm include the following.

*Welding vs. bolting:* Welding adjacent to wood framing poses a very real fire hazard. Specifications and common sense usually dictate various fire watch provisions in these situations. Cases of hot welding slag lost from view and later reigniting wood material after the welding for the day was finished have been observed and are particularly troublesome. Where possible, detailing with shop welded connections, and then field bolting, is desirable. See Chapter 8 for additional comments on welding.

*Connecting directly to the masonry:* In Figure 21.4.9-1, the moment frame is connected to both the masonry façade and the diaphragm to take out load from the punctured wall into the frame and from the diaphragm into the frame. In alternative details, the load can be taken from the wall into the diaphragm and then through the diaphragm to the frame.

### *Cost/Disruption*

Installation of a new moment frame can be fairly disruptive, though it is usually less disruptive than a new wall. The frame is chosen when existing window or door openings need to be preserved, but head height and visual issues must be considered. Adding new structural steel members can be comparatively expensive, but if the choice is to provide a wood structural panel overlay on a floor or add a new moment frame, the new moment frame can often be less expensive.

### *Proprietary Issues*

There are no proprietary concerns with connecting a steel moment frame to a wood diaphragm. Certain moment frame beam-to-column connections may have proprietary considerations. See Chapter 8.

## 21.4.10 Add or Enhance Crosswalls

### *Deficiency Addressed by Rehabilitation Technique*

Inadequate diaphragm strength and/or excessive diaphragm displacement.

### *Description of the Rehabilitation Technique*

The ABK research program (ABK, 1984) showed that partition walls, called crosswalls, serve as energy-absorbing, displacement-limiting damping elements during seismic loading. The 2003

IEBC and 1997 UCBC permit certain qualifying buildings to use the "Special Procedure" with crosswalls as an integral part of the procedure. There are three basic types of crosswalls: existing partitions with various sheathing materials, new partitions, and new steel moment frames. Existing partitions may be adequate without any rehabilitation or they may need strengthening if they are not connected to the diaphragms or have insufficient capacity. New wood structural panel partitions must also be connected to the diaphragm and meet certain minimum capacities. New moment frames must meet minimum strength and stiffness criteria. See Figures 21.4.10-1 to 21.4.10-3.

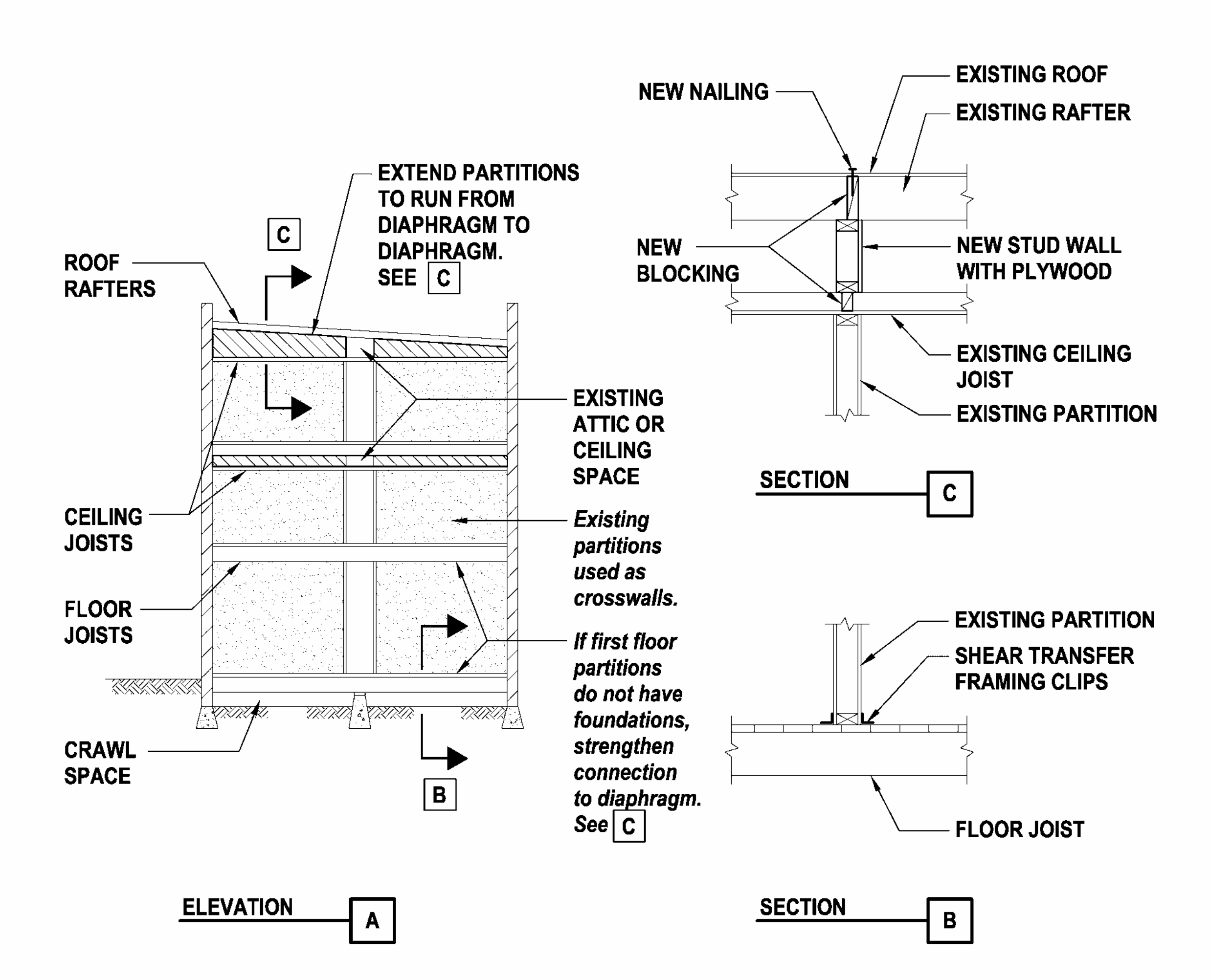



**Figure 21.4.10-1: Strengthen Existing Crosswalls**

### *Design Considerations*

*Research basis:* ABK (1984) provides background for the basis of the crosswall concept.

*Qualifying buildings:* In the 2003 ICBC, the Special Procedure can be used only with buildings have flexible diaphragms at all levels and meet certain requirements regarding open fronts and number of wall lines in each direction. For the 1997 UCBC, these requirements apply, and buildings must be a maximum of six stories, and they cannot be essential or hazardous facilities. The crosswall concept and the Special Procedure were not adopted in FEMA 356.

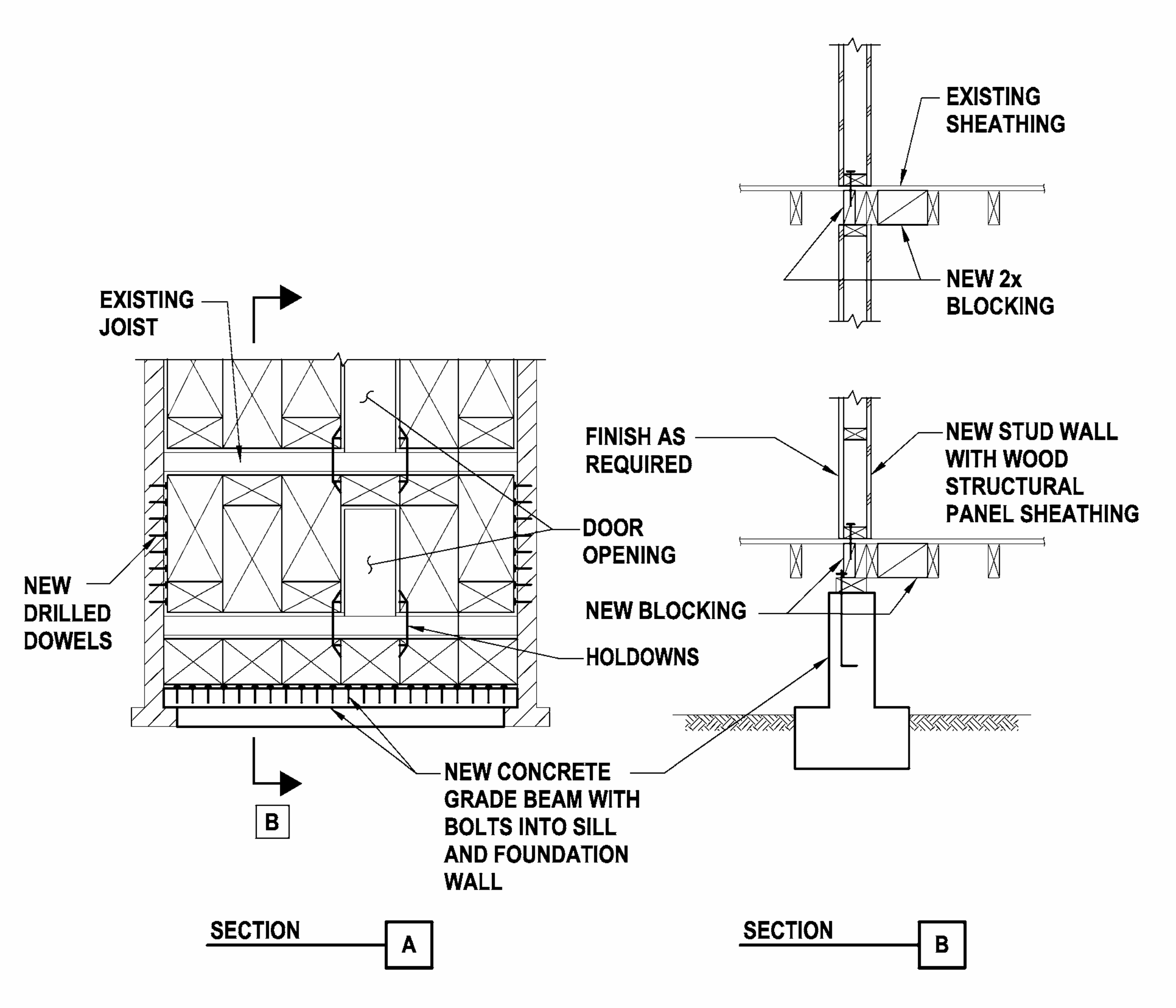



**Figure 21.4.10-2: Add New Crosswalls**

*Crosswall requirements:* The 1997 UCBC and the 2003 IEBC have a number of requirements on crosswall locations, aspect ratios, connection strength, spacing limits that need to be satisfied.

*New crosswalls:* New crosswalls will typically be done with structural wood panels and are similar to adding new wood structural panel shear walls. See Chapters 5 and 6 for additional information.

*Steel moment frames:* Adding moment frames for use as crosswalls is very similar to adding moment frames for use as new lateral force-resisting elements. See Section 21.4.9 for additional information.

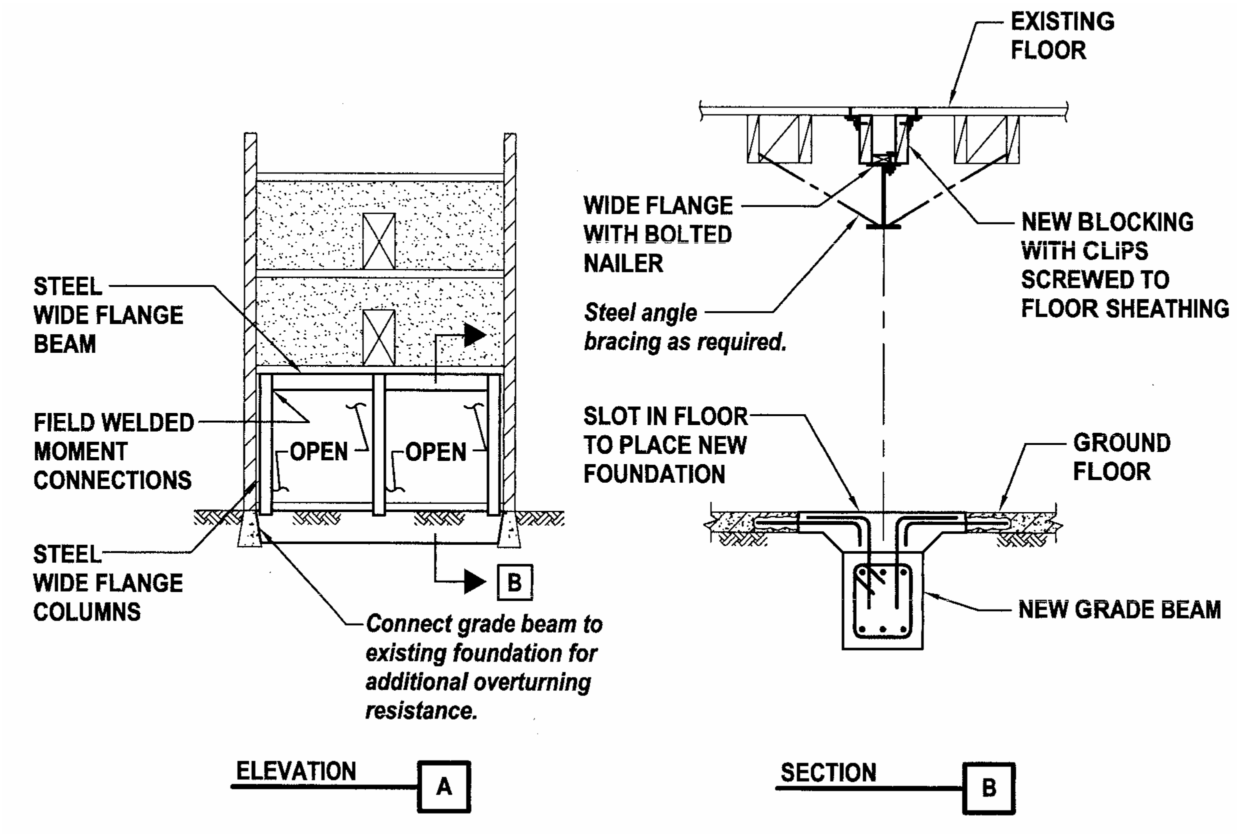



**Figure 21.4.10-3: Add New Moment Frame as Crosswall**

### *Detailing and Construction Considerations, and Cost/Disruption*

Adding or enhancing woodframe crosswalls is similar to adding or enhancing woodframe shear walls; see Chapters 5 and 6. Adding a moment frame as a crosswall is similar to adding a moment frame as a lateral force-resisting element; see Section 21.4.9.

### *Proprietary Issues*

There are no proprietary concerns with adding crosswalls. Certain steel moment frame beam-to-column connections are proprietary.

## 21.4.11 Add Supplemental Vertical Support for Truss or Girder

### *Deficiency Addressed by Rehabilitation Technique*

Supplemental vertical supports provide a secondary load path for concentrated gravity loads on unreinforced masonry walls.

### *Description of the Rehabilitation Technique*

A steel or wood post is added under existing trusses and girders.

### *Design Considerations*

*Research basis:* There are no known tests of supplemental supports.

*Purpose of the support:* To some engineers, the goal of adding a supplemental support is to provide a back-up gravity load path if there is local deterioration of the masonry underneath a concentrated load like a truss bearing point. To others, it provides support if more wholesale failure of the wall occurs.

*Independence of the support:* The 1997 UCBC and 2003 IEBC both use the term "independent secondary columns" when referring to supplemental vertical supports. To some engineers, "independent" means separated from the wall. A post an inch or two away from the wall satisfies this requirement. To other engineers, "independent" simply means an alternative support, so that a ledger or pilaster on the wall is sufficient. In this scenario, the wall just beyond the damaged area is assumed to remain intact enough that gravity load resistance is not compromised. In Figure 21.4.11-1, a gap is shown.

*Use as out-of-plane brace also:* Some engineers like to take advantage of supplemental vertical support posts to serve as out-of-plane braces for the walls as well. Combined bending and axial demands must be considered.

*Continuity of support:* The 1997 UCBC and 2003 IEBC both do not specify whether the posts need to continue down to the next story. It is common to transfer loads at the base of upper story supplemental supports back to existing framing. This framing must be adequate to take the loads that would occur if they supplemental support began to take load.

*Foundation support for posts:* The 1997 UCBC and 2003 IEBC both do not explicitly specify whether the posts need a compliant new foundation or whether the posts can simply bear on an existing slab-on-grade. SEAOC (1992), however, states that a foundation is not required for the posts. Nonetheless, some engineers believe it is prudent to check the slab for bearing support and provide additional support if needed.

*Triggering elements:* The 1997 UCBC and 2003 IEBC only indicate that triggering elements are "trusses and beams, others than rafters and joists". The implication is that elements that support other structural members are the primary focus.

### *Detailing and Construction Considerations*

Detailing and construction considerations for supplemental vertical supports include the following.

*Steel vs wood:* Supplemental support posts can either be of steel or wood. Steel members are smaller; wood members are less expensive.

*Finish the new elements are leave bare:* For certain architectural approaches, leaving the supplemental posts bare is compatible with the existing aesthetic. If it is not, the posts can be furred at added cost.

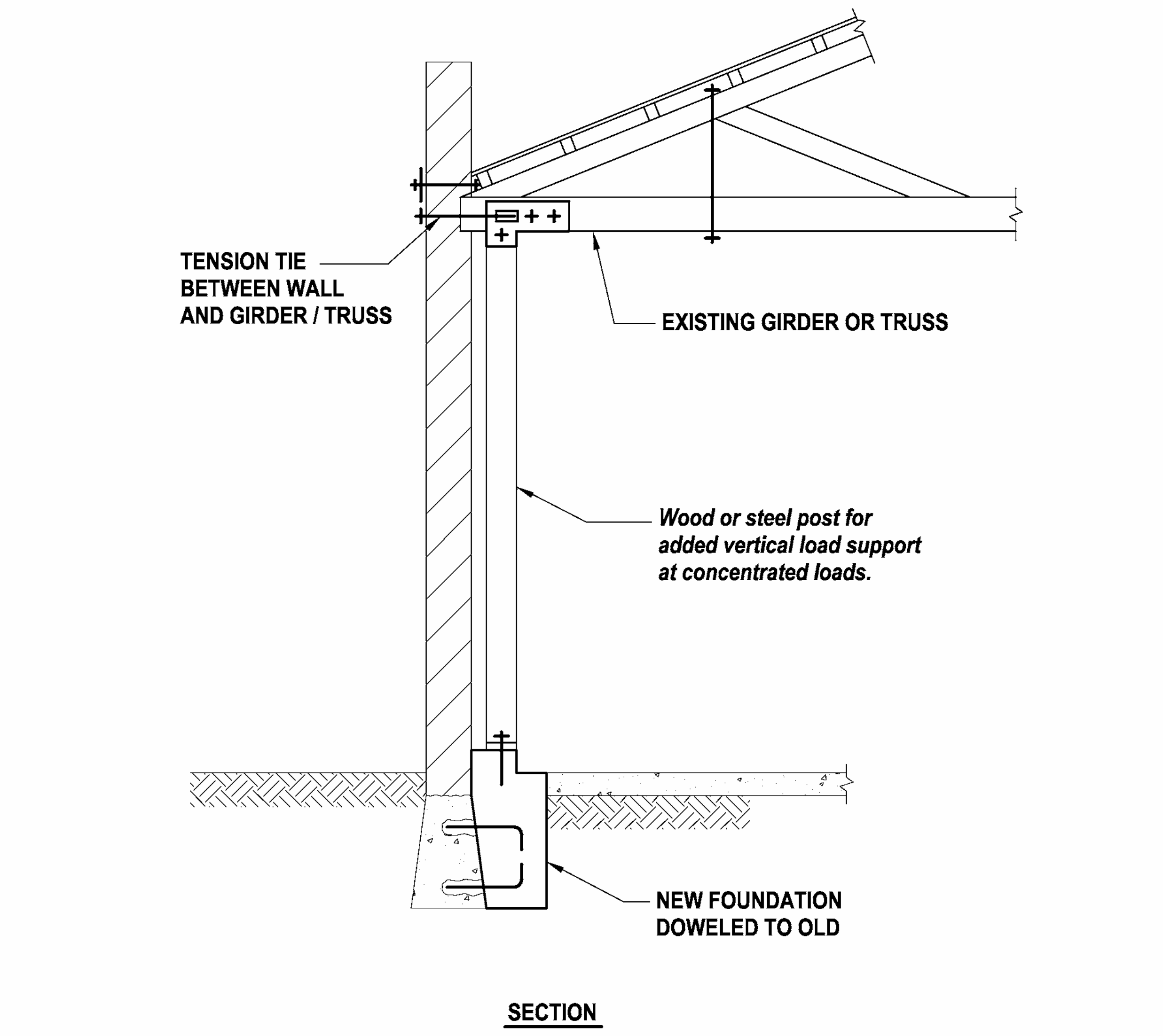



**Figure 21.4.11-1: Supplemental Vertical Support**

### *Cost/Disruption*

The relative cost of adding supplemental supports depends on the number used, whether they continue down to the ground and whether a new foundation is installed. Interior occupants will be disrupted locally as the posts are installed, and the usable space in the vicinity of the posts will be reduced.

### *Proprietary Considerations*

There are no known proprietary concerns with employing supplemental vertical supports.

### 21.4.12 Add Veneer Ties in a URM Wall

*Deficiency Addressed by Rehabilitation Technique*

Missing or inadequate ties between a masonry veneer wythe and the backing wythes can lead to delamination of the veneer and a falling hazard.

*Description of the Rehabilitation Technique*

If the front or facing wythe of brick is not integrally tied into the interior wythes with header courses, it is considered a veneer. If sufficient metal veneer ties are not present to anchor the veneer to the backing wythe, new ties can be provided. Figure 21.4.12-1 shows anchorage using drilled dowels to connect the wythes, either from the exterior or interior, in a typical brick wall. Figure 21.4.12-2 shows anchorage between stone facing and brick backing.

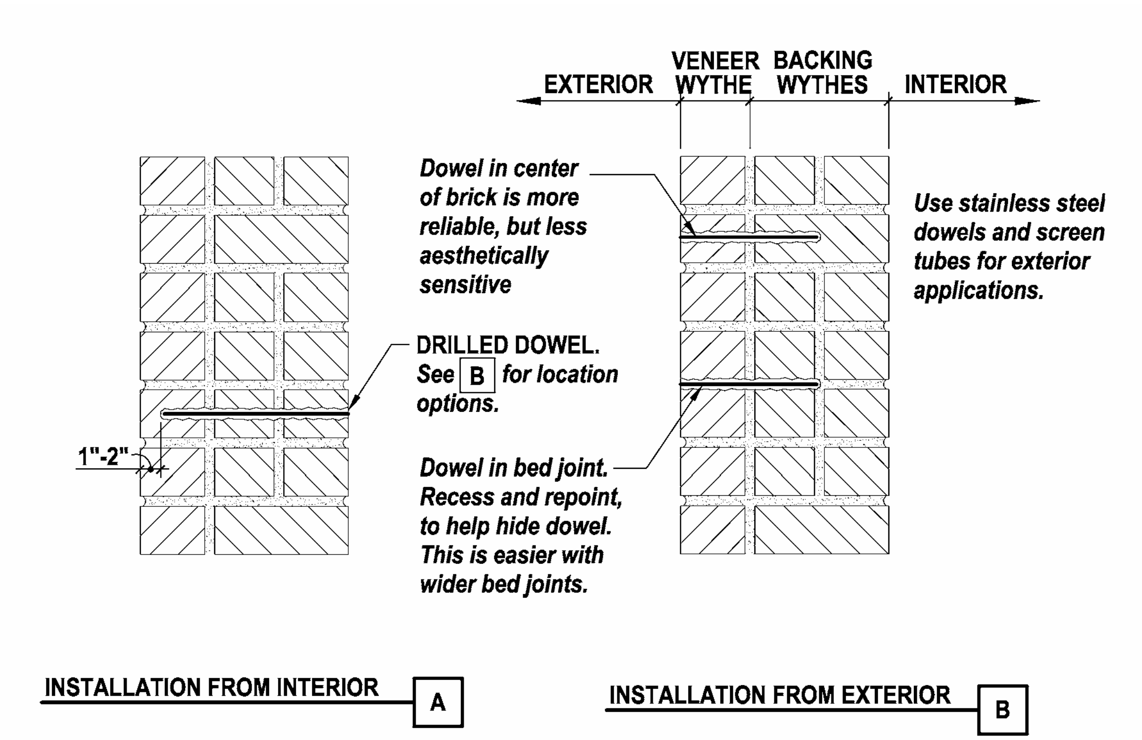



**Figure 21.4.12-1: Veneer Ties in Brick Masonry**

*Design Considerations*

*Research basis:* No academic research on veneer ties has been identified. Individual vendors have performed internal testing of their own products.

*Veneer definition:* Codes such as the 2003 ICBC and 1997 UCBC give minimum lay-up requirements for multi-wythe solid brick. The facing and backing wythes are to be bonded so than not less than 10 percent of the exposed face area is composed of solid headers extending less than 4 inches into the backing. The clear distance between adjacent full-length headers shall not exceed 24 inches vertically or horizontally. Facing wythes that do not meet these lay-up requirements are to be considered as veneer. Veneer wythes are not used in resisting shear forces in the wall and do not count in the thickness used for determining out-of-plane bending resistance.

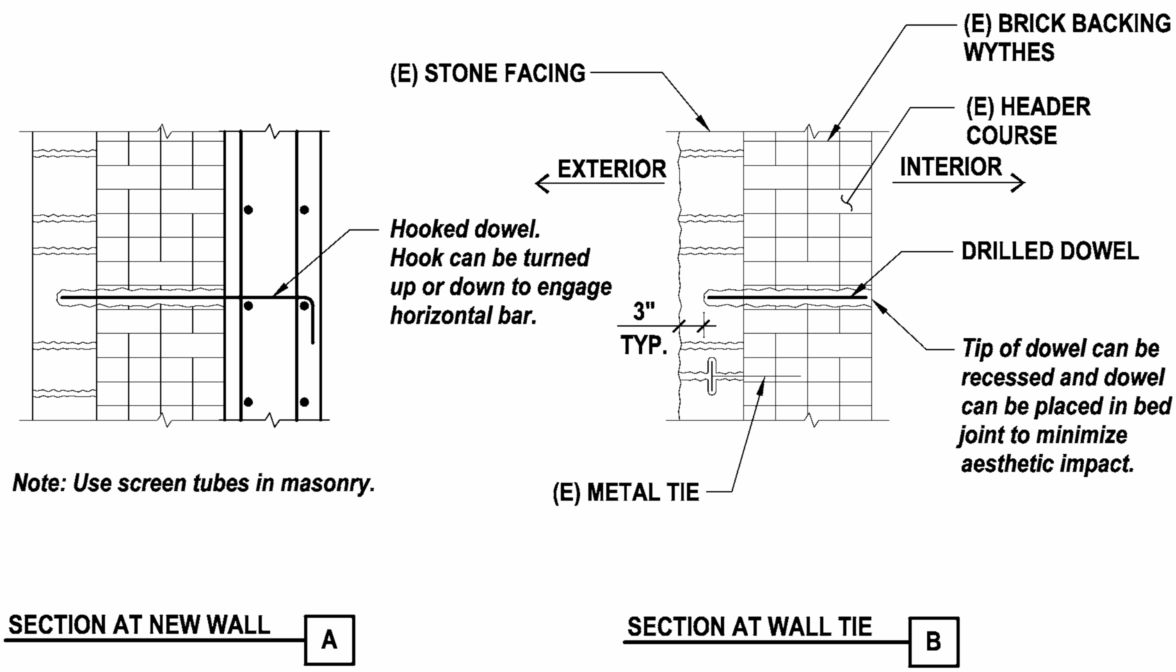



**Figure 21.4.12-2: Veneer Ties for Stone Masonry Facing**

*Veneer tie requirements:* Codes such as the 2003 ICBC and 1997 UCBC provide acceptance criteria for existing ties. They are to be corrugated galvanized iron strips shown to be in good conditions, with dimensions no less than 1” wide, 8” long and 1/16” thick and with a maximum spacing of 24” on center and a maximum supported area of four square feet. Veneer ties not meeting these requirements are to be strengthened.

### *Detailing and Construction Considerations*

Detailing and construction considerations for veneer ties include the following.

*Types of ties:* The ties used in new construction are typically inappropriate for rehabilitation since they are installed as the wall is built up. There are, however, a fair number of proprietary products made by masonry accessory manufacturers that can be used in retrofit applications. Some involve expansion anchors in the backing wythes. Others involve helical anchors or spiral ties that “screw” into the backing wythes. Figures 21.4.12-1 and 21.4.12-2 show traditional drilled dowels. When drilled dowels are used, the tie diameter does not need to be large since loads are low and minimizing the size of the hole is important.

*Drilled dowels installation face:* Drilled dowels can be installed from the interior or exterior depending on which face is more sensitive.

*Location of drilled dowel:* The most reliable location of the drilled dowel is in the center of the brick, but this is will cause the largest aesthetic impact. The dowel can be placed in the bed joist or bed and head joint intersection to minimize the impact. Recessing the tip of the dowel and covering the end with repointing mortar is recommended.

*Brick veneer vs. stone veneer:* Anchoring thick stone is usually easier to do from the interior because the thickness of the stone permits greater cover on the face, reducing the likelihood of spalling. With random ashlar layout and interior installation, however, trying to locate the dowel away from the edges of the stone is not practical.

*Corrosion considerations:* Any dowel installed from the exterior should be done in stainless steel to minimize corrosion.

***Cost and Disruption Considerations***

Veneer anchorage can be relatively expensive depending on the number of new ties added. Ties installed from the inside are much more disruptive. See Section 21.4.2 for additional information on drilled dowels.

***Proprietary Considerations***

Many of the veneer tie anchorage systems are proprietary.

## 21.5 References

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