Chapter 22 - Diaphragm Rehabilitation Techniques

22.1 Overview

Diaphragm failures are less commonly observed in earthquakes, and the disruption caused by strengthening the diaphragm can be quite significant, so diaphragm rehabilitation is less commonly employed than adding global strength and stiffness, or improving connection paths. Some diaphragms are inherently less likely to be an issue, such as cast-in-place concrete flat slabs or waffle slabs; others like straight sheathed wood or poorly connected precast floors are of greater concern.   This chapter provides examples of various diaphragm systems and their strengthening techniques. They are organized here in a single chapter for convenience and because many of the diaphragms can be found in different building types. For discussion of diaphragm-to-wall connection issues, see individual building type chapters.

22.2 Detailed Description of Diaphragm Rehabilitation Techniques

22.2.1 Wood Diaphragm Strengthening

Deficiency Addressed by Rehabilitation Technique

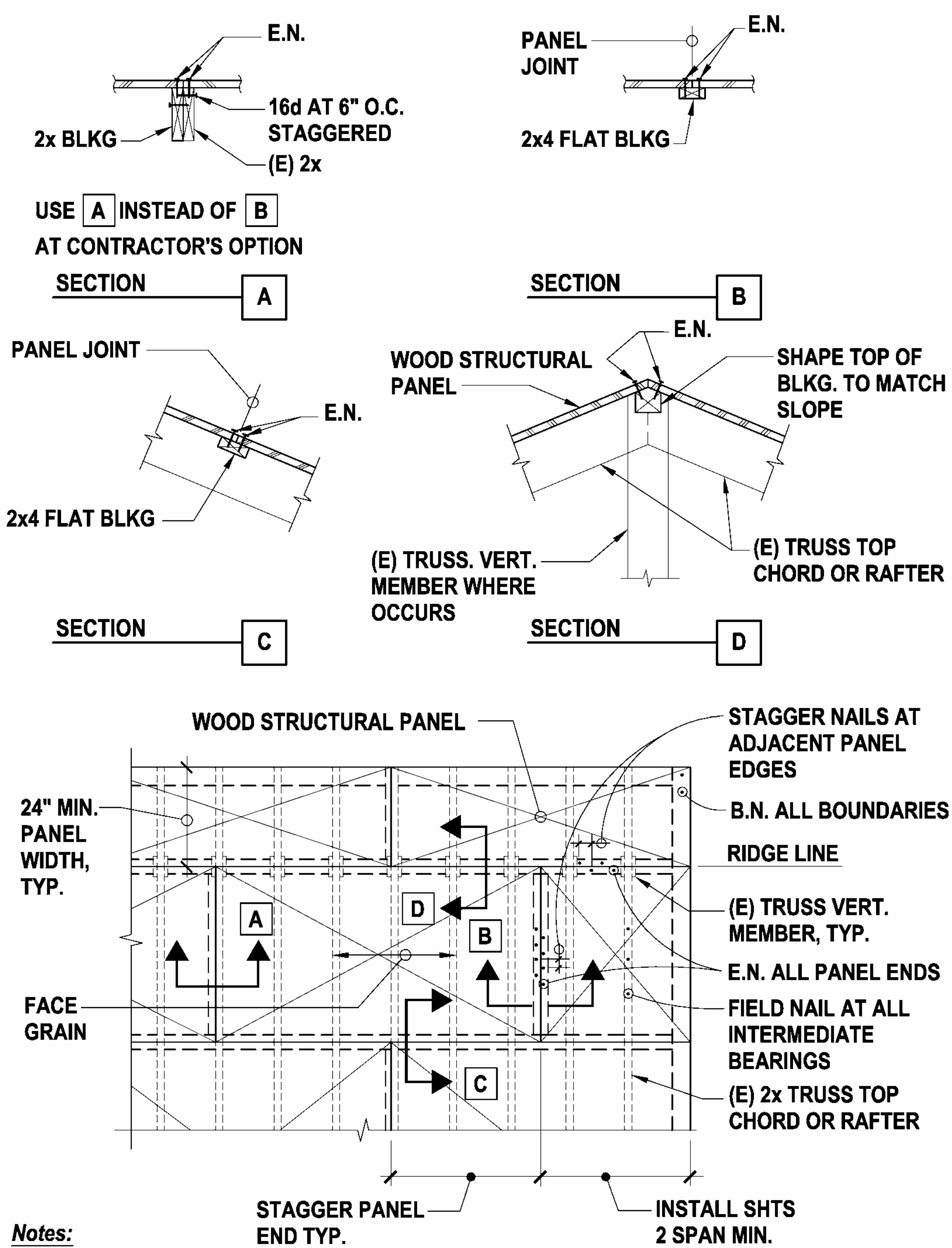
Inadequate diaphragm strength and/or stiffness

Description of the Rehabilitation Techniques

The addition of new wood structural panel sheathing is a traditional and common approach to diaphragm strengthening. Adding fastening and blocking to existing wood structural panel sheathing can also be done. Specifically, this section covers:

Replacing existing sheathing with new wood structural panel sheathing
Wood structural panel sheathing overlays with new blocking
Wood structural panel sheathing overlays without new blocking
Improving strength and stiffness of an existing wood structural panel sheathed diaphragm

Each of these techniques aims to improve the shear strength and lateral stiffness of the existing diaphragm. Figure 22.2.1-1 shows the replacement of existing sheathing with new sheathing directly onto the existing joists. Figure 22.2.1-2 shows a wood structural panel overlay on existing straight sheathing floors or roofs when new blocking is added below the existing sheathing. Figure 22.2.1-3 shows an overlay when blocking is not added, and Figure 22.2.1-4 shows a similar overlay to use when the bottom of the existing sheathing is to remain exposed to view and penetrations through it would not be acceptable. Figure 22.2.1-5 shows how shear transfer can be made to get past an existing partition sill that is to remain in place.



Notes:

1. *Remove existing sheathing.*
2. *Use 3x and 4x blocking for higher capacity if necessary.*
3. *Use of A requires wood structural panels to be field cut to fit existing joist spacing. Detail B permits nominal wood structural panel size.*

Figure 22.2.1-1: Remove and Replace Existing Wood Sheathing with Wood Structural Panel at a Roof

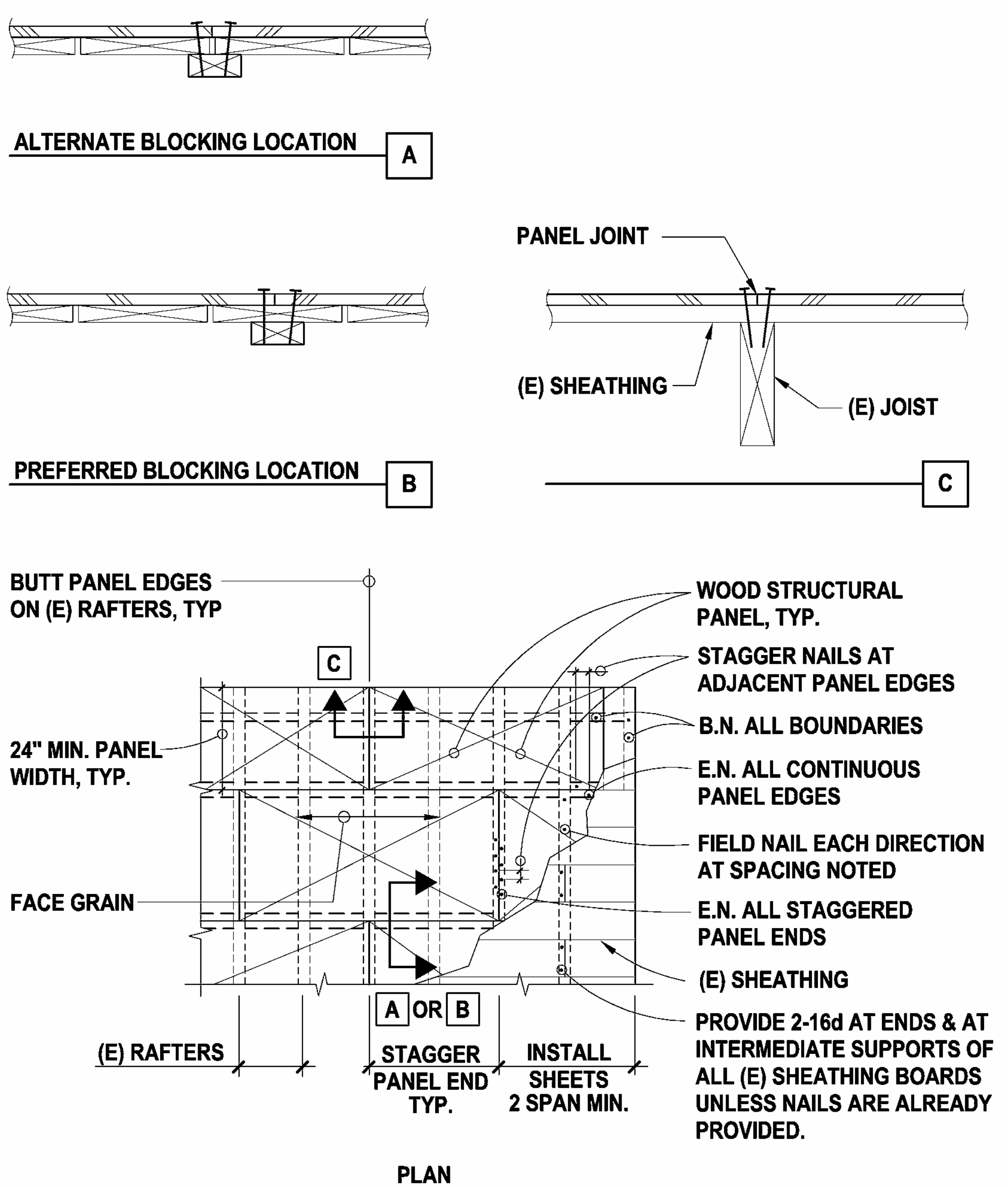


Figure 22.2.1-2: Wood Panel Overlay with Blocking Over Existing Sheathing

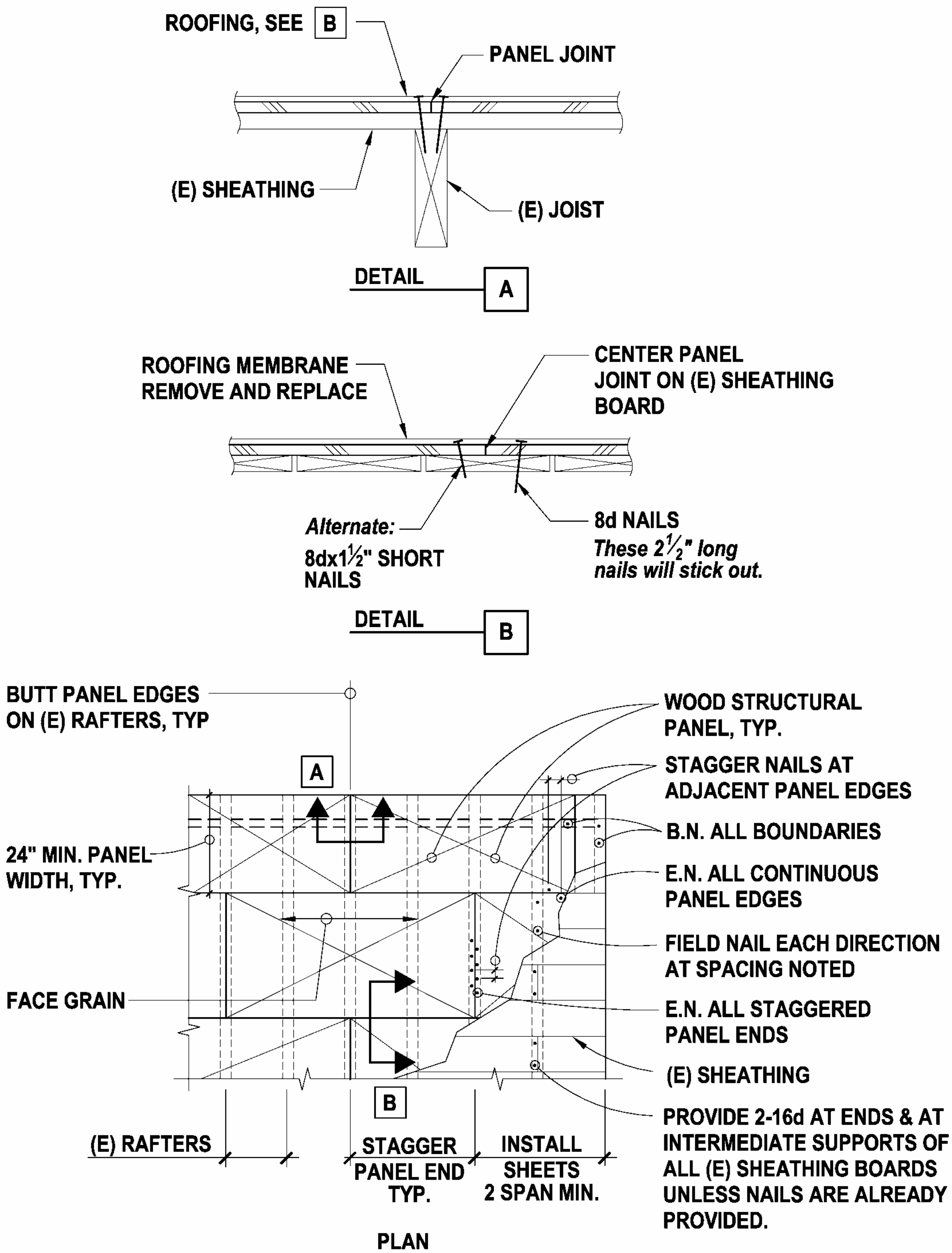


Figure 22.2.1-3: Wood Panel Overlay without Blocking Over Existing Sheathing

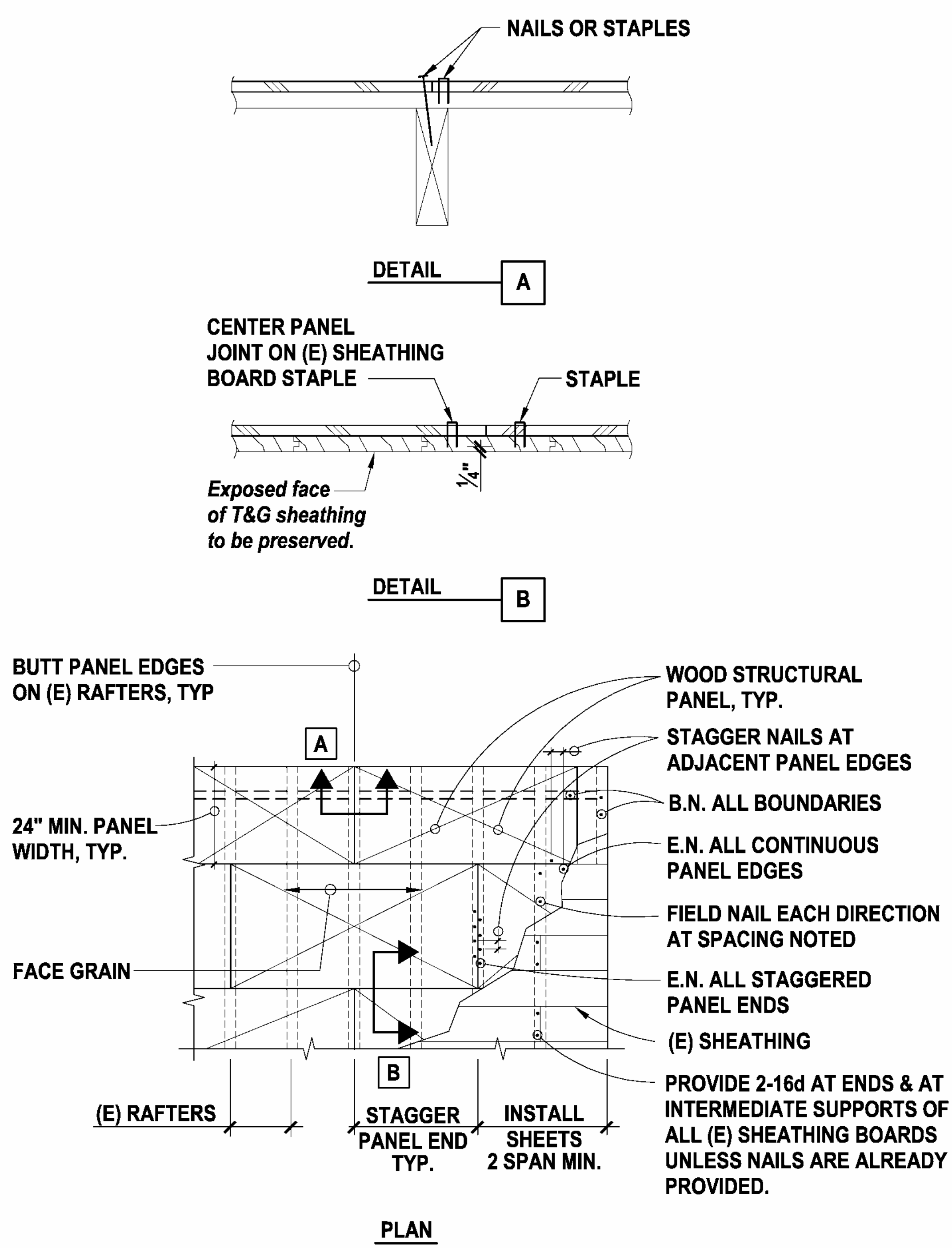


Figure 22.2.1-4: Wood Panel Overlay without Blocking Over Existing Sheathing When the Bottom of the Existing Sheathing is Visible

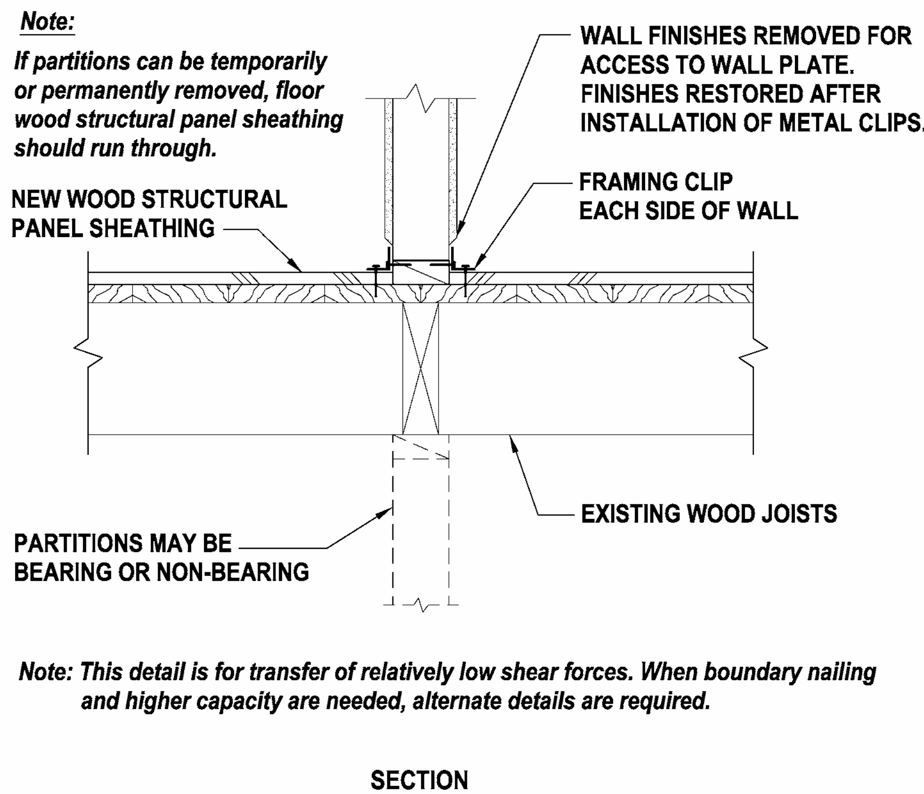


Figure 22.2.1-5: Shear Transfer in New Overlay at Existing Partitions

Design Considerations

Research basis: When new wood structural panel sheathing replaces existing sheathing, then the basic research for panel sheathing used to develop diaphragm capacities is applicable, and values would be taken from the relevant building code. When structural panel sheathing is used as an overlay, there is less research available. Values that have made it into model codes such as the UCBC (ICBO, 1997) and IEBC (ICC, 2003b) are based in part on the ABK research program for URM bearing wall strengthening, including ABK (1981). In this program, a series of 14 full-scale, 20’x60’ horizontal diaphragm specimens were subjected to quasi-static, cyclic, in-plane displacements and dynamic, in-plane earthquake shaking. Specimens include filled and unfilled steel deck, blocked and unblocked plywood, and straight and diagonal sheathing with and without plywood overlays and with roofing material. More recent tests include Peralta, Bracci, and Hueste (2004) where a series of twelve 12’x24’ horizontal diaphragm specimens were subjected to quasi-static, reversed cyclic in-plane displacements. Specimens included tongue groove sheathing retrofit with strapping and with an underlying steel truss, straight sheathing with and without openings retrofit with a steel truss and with blocked and unblocked plywood overlays. Results were compared with both FEMA 273 (1997a) and FEMA 356 (2000).

Types of diaphragms: Approaches to diaphragm rehabilitation can be categorized as follows:

Structural wood panel sheathing where the existing sheathing is replaced: This is the approach typically used when high capacities are needed.

- "High load" diaphragms where 3x and 4x blocking is added and multiple lines of nailing are used: This may be done in accordance with provisions in the IBC; additional detailing information in ICC-ES Legacy Report 1952 (ICC-ES, 2004) is highly recommended. See APA (2000) for testing results.
- Traditional diaphragms with 3x and 2x blocking and various panel layouts: The relevant building code capacities are used. An issue that often arises is whether existing joists, which are typically thicker than the code assumed 1-1/2", can count as 3x blocking. Some engineers ratio values between 2x and 3x code capacities.
- Unblocked diaphragms: It is relatively unusual to remove existing sheathing only to replace it with unblocked wood structural panels as the capacities are not substantially different.

Wood structural panel sheathing overlays over existing 1x nominal sheathing: In the 1997 UCBC, there are values given for the following three approaches. The 2003 IEBC only lists the first type. Inherent in these approaches is the assumption that existing lumber sheathing is one-inch nominal (commonly 5/8-inch to 7/8-inch actual) thickness.

- Wood structural panel overlays nailed directly over existing straight sheathing with ends of the panels bearing on joists or rafters and edges of the panels located on center of individual sheathing boards: The lack of blocking makes this a relatively weak diaphragm.
- Wood structural panel overlays nailed directly over existing diagonal sheathing with ends of wood structural panel sheets bearing on joists or rafters: Diagonal sheathing provides increased strength compared to the overlay of straight sheathing.
- Wood structural panel overlays nailed directly over existing straight or diagonal sheathing with ends of panels bearing on joists or rafters with edges of panels located over new blocking and nailed to provide a minimum nail penetration into framing and blocking of 1-5/8": The 1997 UCBC limits this to 75% of code values for wood structural panel overlays without the existing sheathing, due in part to the potential for bending of the nail in the existing sheathing before it reaches the main member blocking and the risk of the nailing being near the edges of the existing sheathing.

Wood structural panel sheathing overlays over existing lumber planking (2-inch nominal or thicker) or laminated decking; the IBC (ICC, 2003a) and the AF&PA (2005) permit wood structural panel diaphragm sheathing to be fastened over solid lumber planking or laminated decking using full tabulated values for new construction. Inherent is the assumption that the sheathing nail will have a penetration of not less than 10 diameters (1-3/8 inches for 8d common and 1-1/2" for 10d common) into the planking or decking. Special attention is needed at all diaphragm boundaries to ensure shear transfer from the sheathing, through the planking or decking to the boundary members below.

Wood structural panel sheathing overlays over existing spaced (or skip) sheathing: A common roof framing system is to span 1x nominal boards across rafters. Building paper is placed on top of the boards and under the final roofing layer such as shakes or shingles. Wide spaces of several inches are left between the 1x boards both to save sheathing material and to permit air flow to help dry the roofing sandwich. This construction is the most flexible and the weakest type of existing wood diaphragm and has no code values. Wood structural panel overlays can be placed across the skip sheathing. Care should be taken to align the panel edges atop the spaced sheathing. Due to the 1x thickness of the spaced sheathing, full development of the nail will not be achieved. With the gaps between sheathing boards, two edges of the wood structural panels will not be blocked. 1x sheathing or wood structural panel nailing strips with matching thicknesses can be placed atop the rafters in the gap to serve as "blocking" at these edges. Direct code values for these overlays are not available, though some engineers use code values reduced down by the amount of actual vs. full nail development length. Alternatively, staples can be used to help address the shallow sheathing depth.

Wood structural panel sheathing overlays over existing wood structural panel sheathing: Two layers of wood structural panel sheathing have been tested and documented in APA (2000). The tested configuration used overlays at panel ends in high-load regions.

Existing wood structural panel diaphragm enhancement without overlays: A wide variety of rehabilitation measures are available for existing wood structural panel diaphragms that do not involve new overlays. These include:

- Addition of 2x wood blocking to an unblocked diaphragm (Dolan et al., 2003)
- Addition of sheet steel blocking to an unblocked diaphragm (APA, 2000)
- Addition of nailing to existing blocked diaphragm (allows limited improvement because framing member requirements change at closer nail spacing)
- Adding staples to existing wood structural panel diaphragm. Staples are designed to carry entire seismic unit shear
- Stapling of tongue and groove sheathing joints (APA, 2000).
- Addition of a wood structural panel soffit in local areas of high diaphragm shear (see Section 22.2.2)

Existing diaphragms without overlays: In the 1997 UCBC and 2003 IEBC, there are values for the following existing materials:

- 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 sheathing
- Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular: Values are relatively high for this combination
- Floors with diagonal sheathing and finish wood flooring: Values are also relatively high for this combination

FEMA 356 has its own extensive listing of diaphragm types, and there are examples and even tests in the literature exploring the influence of glue, double layers of panel sheathing, herringbone panel overlays. IBC, APA (2000), and ICC-ES (2004) provide techniques for calculating code level values, including stapled diaphragms.

In order to select and properly detail diaphragm rehabilitation measures, it is important to determine the layout and thickness of existing sheathing and framing. Significant attention is needed to transfer of shear at all diaphragm boundaries. This includes diaphragm chords (Section 22.2.2), subdiaphragms and cross-ties for flexible diaphragm/rigid wall buildings (Section 22.2.3), and collectors (Sections 6.4.5 and 7.4.2).

Condition assessment of the existing roof structure is important. It is common to find decay damage to existing framing and sheathing in the vicinity of roof drains.

Detailing and Construction Considerations

Detailing and construction considerations for wood diaphragm strengthening include the following.

Aligning panel edges: When the existing sheathing is removed, the joists or rafters typically remain in place. Their spacing will vary. To align the edges of new 4'x8' sheets of structural wood panels on top of the supporting framing requires field measuring and cutting the sheets. Alternatively, new blocking can be added between existing framing to reduce the need to cut the structural wood panels. See Figure 22.2.1-1 for examples of each approach.

Missing sheathing edges: To reduce the risk of splitting during installation or later during the earthquake, nailing through the center of existing joists is desirable. This can take considerable field effort, however, due to the need to field measure and cut the structural wood panels. See Figure 22.2.1-2 and 22.2.1-3 for examples.

Staples, short nails, regular length nails: When the existing sheathing is removed and the structural wood panel is placed directly on the framing, regular length nails are commonly used. When the structural wood panel is applied to the existing lumber sheathing without blocking, 8d and 10d nails will go well through the underside of the sheathing. "Short" or "diaphragm" nails can be used to reduce the amount of nail protrusion. See Figure 22.2.1-3. When the overlay is on a diaphragm that is architecturally exposed from below, nail penetrations are not desirable. Staples can also be used, such as shown in Figure 22.2.1-4; per IBC, 16 gage staples require one-inch penetration into framing for tabulated values.

The nail penetration into diaphragm framing members required to achieve code and standard tabulated allowable shear values has changed recently. In the past, a nail penetration of 1-5/8 inches was required to obtain full diaphragm capacity. As a result, allowable shear reductions were applied when only 1-1/2 inch penetration was provided, as commonly occurs with 2x flat blocking in diaphragms or engineered joist top chords. The 2003 IBC only requires 1-3/8-inch penetration for 8d common nails and 1-1/2-inch penetration for 10d common nails. APA T98-22 (APA, 1998) provides one explanation, based on calculation using yield-mode equations. The nail penetration requirements are stated specifically in the diaphragm tables, and methods to adjust for reduced penetration are not suggested. Reduction in penetration below the IBC minimums is not recommended; because considerable slip can occur between sheathing and framing as a diaphragm takes up load, reduced embedment may lead to premature withdrawal failure. These APA and IBC penetration requirements are applicable to sheathing-to-framing fastening.

Nail penetration requirements have also been changing in the NDS (AF&PA, 2005), where a nail penetration of 10 diameters is now adequate to develop tabulated nail capacities. This number has been 12 and 11 diameters in previous provisions. Nails with a penetration of less than six diameters are not permitted to be used. These NDS penetration requirements are applicable to framing-to-framing fastening.

Gluing of diaphragms: Adding glue between a wood structural panel and supporting framing in a diaphragm or shear wall assembly where inelastic behavior is anticipated is strongly recommended against, as glued sheathing has limited ductility or energy dissipation capacity. This applies whether or not nailing is provided in addition to the glue. Dolan et al. (2003) evaluated the effect of diaphragm gluing on strength and stiffness.

Partitions: A diaphragm that is continuous between walls provides the stiffest and most direct load path. In an existing building, however, there are almost always existing partitions on the floor. If they are to remain during the rehabilitation, Figure 22.2.1-5 shows a detail for shear transfer from one side to the other of the partition sill in an overlay. This approach is adequate when the value of the load transfer is relatively low; when higher capacities are needed such as for boundary nailing or double rows of nails, alternative details will need to be developed and typically include blocking down and around the partition.

Weight: Adding structural wood panel sheathing over existing sheathing adds weight to diaphragm. This rarely poses a problem, but the engineer should consider the issue.

Location of diaphragm: Figures 22.2.1-1 through 22.2.1-5 all show the structural wood panel added to the top of the floor. In many situations, due to finishes on the top of the floor or usage of a particular story, enhancing the underside of the diaphragm is a less disruptive approach.

Cost/Disruption

Adding structural wood panel overlays can be a significant disruption to occupants, just from the need for access to either the top or underside of the floor, as well as from the noise of sawing and hammering. If the building is to remain occupied during rehabilitation, work is sometimes phased by floor or wing to minimize the number of impacted occupants at any one time. Many existing buildings have had roof strengthening done from above with the occupants in place. Sometimes the work is limited to certain hours that are considered less disruptive. When improvements or overlays are installed on top of the roof, it may be necessary to develop detailing to allow work around existing roof top equipment platforms and curbs, skylights, etc.

Proprietary Issues

There are typically no proprietary concerns with wood diaphragm strengthening.

22.2.2 Add or Enhance Chord in Existing Wood Diaphragm

Deficiency Addressed by Rehabilitation Technique

This rehabilitation technique addresses inadequate, incomplete or missing chords in buildings with reinforced concrete or masonry shear walls; also addressed is inadequate shear transfer into chord members. Provision of chord members is specifically not required for diaphragms in unreinforced masonry buildings, where wall bed joint shear is thought to provide some chord member capacity. See Chapter 21 for additional discussion of URM buildings.

Rehabilitation approaches discussed may also be applicable to collectors and detailing at re-entrant corners. While systematic evaluation may identify the need for chord enhancement, it is also often provided in conjunction with diaphragm enhancement, as discussed in Section 22.2.1.

Description of the Rehabilitation Technique

The purpose of a diaphragm chord is to act as a tension or compression member resisting diaphragm flexural forces; this requires both an adequate member and adequate transfer of shear from the diaphragm to the chord member along the full member length. In buildings with wood diaphragms and reinforced concrete or masonry walls, the most common chord members are reinforcing steel placed in the wall at or near the roof diaphragm elevation and a structural steel angle bolted to the wall.

Where the existing chord member is adequate, rehabilitation may be limited to enhancing shear transfer. Figures 22.2.2-1A and 1B show added fastening at the roof diaphragm boundary and added adhesive anchors to the concrete or masonry wall, where the existing reinforcing steel is adequate. The reader is cautioned to check the adequacy of the reinforcing as-built conditions at tilt-up concrete walls and reinforced masonry walls with movement joints. See Chapters 16 and 19 for further discussion.

Where additional chord capacity is needed, it is most practical to add a new steel angle on the surface of the existing concrete wall, as shown in Figures 22.2.2-2A and 22.2.2-2B.

Diaphragm chords may be incomplete when vertical offsets occur in the roof diaphragm. When this occurs, it may be possible to use a tilt-up panel to resolve the vertical offset, as shown in Figure 22.2.2-3. Where chords are not occurring at the roof diaphragm level, care should be taken in assessing the unsupported length for compression design.

Design Considerations

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

Enhancement to an existing chord member must be compatible with existing chord behavior. It is unlikely that any chord enhancement applied to the wall face can be compatible with an existing reinforcing steel chord, because of fastener slip required to develop forces in the new chord member. Where an existing reinforcing steel chord is being enhanced, it is suggested that the capacity of the existing reinforcing be neglected.

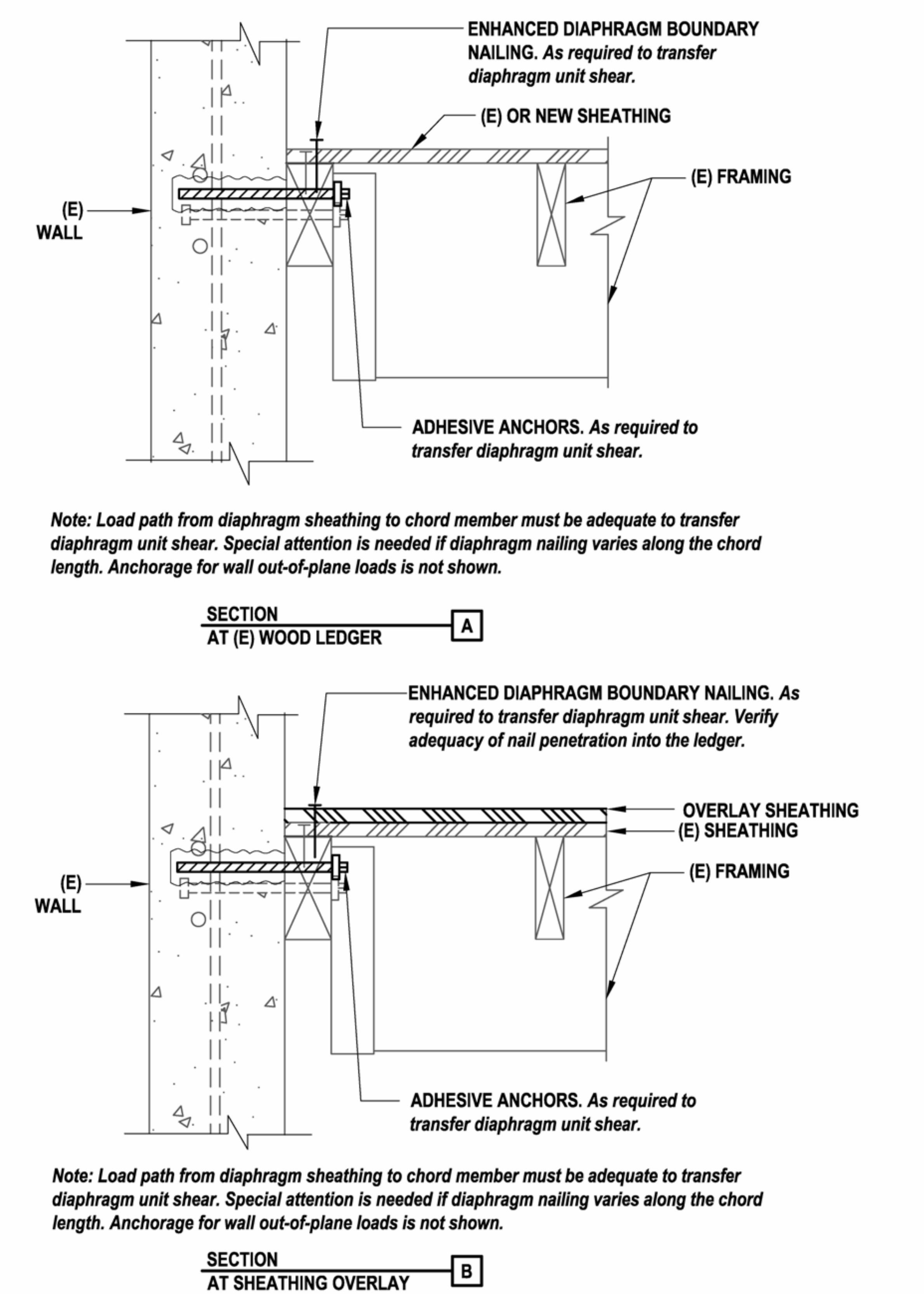


Figure 22.2.2-1: Enhanced Chord Member Fastening at Wood Diaphragm

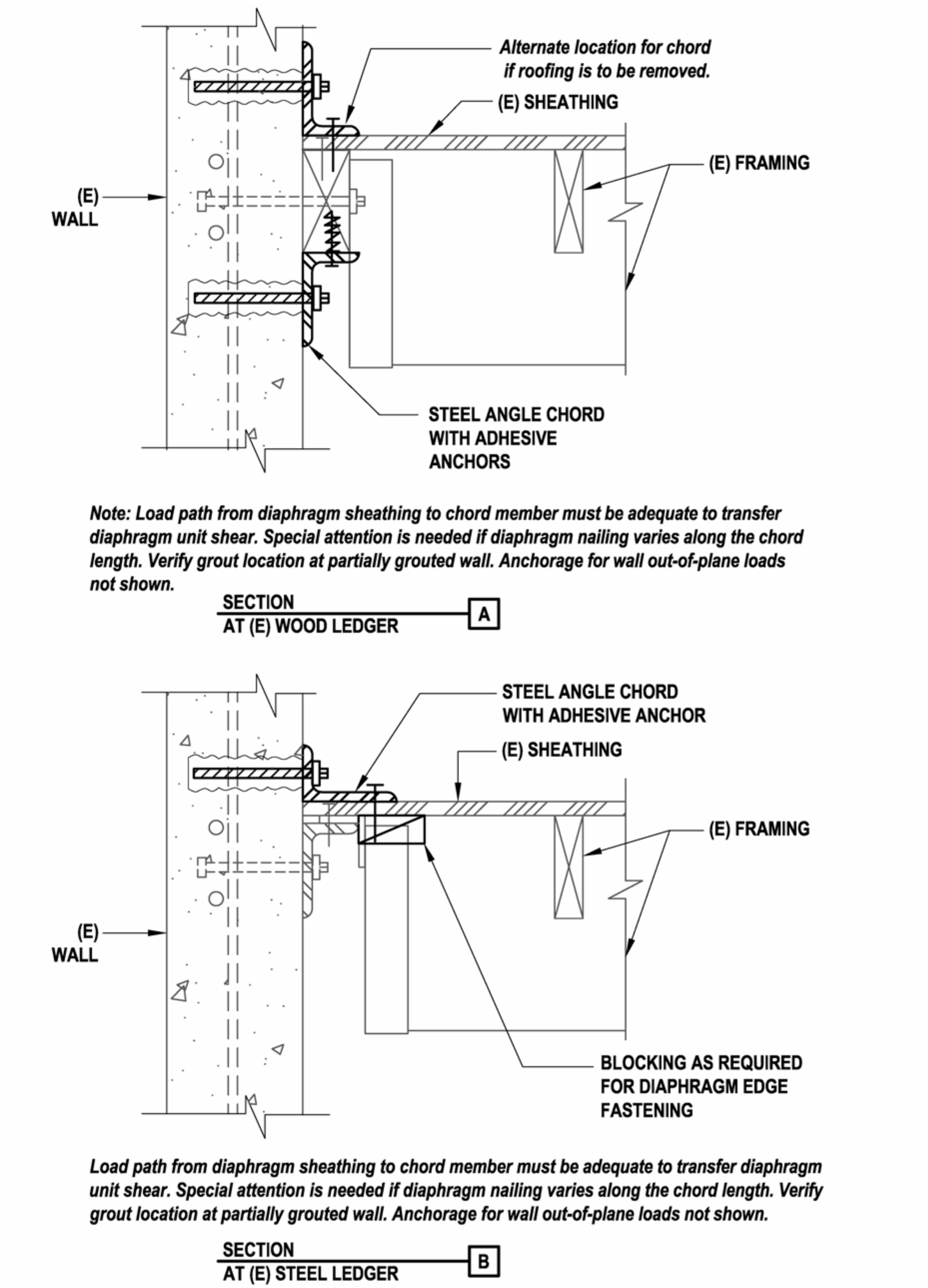


Figure 22.2.2-2: Enhanced Chord Member and Fastening at Wood Diaphragm

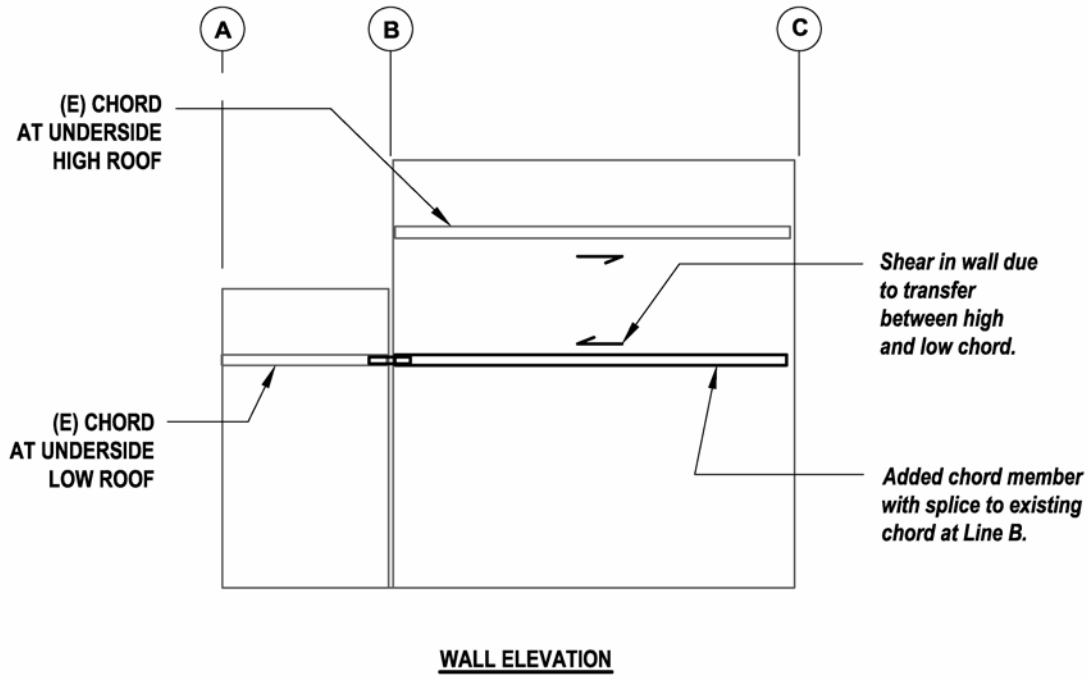


Figure 22.2.2-3: Elevation of Wall Panels with Incomplete Chord Due to Vertical Offset in Roof Diaphragm

The new or enhanced chord member must be anchored into the diaphragm for unit shear transfer. Anchorage for shear transfer is also discussed in Section 22.2.3. As a wood sheathed diaphragm is loaded, slip will occur between the perimeter framing member and the sheathing. Fastening of the chord or chord enhancement should not inhibit this slip. If the slip is not permitted, premature failure at the opposite side of the sheathing panel could occur. This is not a concern with a welded steel deck diaphragm, which has limited slip.

Chord stresses due to shrinkage and temperature change have been identified as a concern for connections between tilt-up panels (SEAOSC, 1979), as discussed in Chapter 16, and these stresses should be considered in chord design.

Detailing Considerations

It is desirable to keep the chord elevation as close as possible to the elevation of the diaphragm in order to minimize secondary stresses and additional deformation. At the edge of the diaphragm this is most easily accomplished by putting a new chord member on the top of the diaphragm (shown as an alternate location in Figures 22.2.2-2A and 22.2.2-2B). This is only possible when re-roofing will occur at the time of rehabilitation work. Otherwise added chord members must be located below existing perimeter members and connections. Splicing of the new or enhanced chord member needs to be specifically detailed.

Diaphragm boundary fastening: Based on observed shear wall test behavior (Gatto and Uang, 2002), providing extra nailing at the diaphragm boundary will likely not provide extra diaphragm capacity, and it may result in premature failure at the first interior joint due to shifting of the center of the fastener group. As a result, sheathing fasteners should be placed symmetrically around the panel edge where possible, and care should be taken to not arbitrarily put extra rows of fasteners at the boundary chord and collector members.

It is preferable to use the same type and size of sheathing fastener at the diaphragm boundary as at the diaphragm interior; however, this may be difficult where new steel chord members are being added on top of the diaphragm, as shown in the alternate location in Figures 22.2.2-2A and 22.2.2-2B. Although graphically shown as a nailed connection from the steel angle chord member to the diaphragm, it may become necessary to use wood screws or lag screws for higher-load diaphragms. Testing of this mix of fasteners has not been identified, so behavior is not known. Behavior of cut-thread wood screws in sheathing to framing fastening has been observed to be problematic, as discussed in Section 6.4.2.

Partially grouted masonry walls: Where shear transfer is being provided into partially grouted masonry walls, it is necessary to verify that the existing wall is grouted at the anchorage location. It is generally anticipated that the existing masonry will be grouted and reinforced at the existing roof ledger location. If, however, anchorage to the wall needs to occur above or below this location, presence of grout will need to be verified. Although methods of anchoring only to the face shell are available, these have very low capacities and should never be mixed with anchors to grouted masonry. So, it is recommended that anchorage to grouted cells be provided. It may be possible to grout cells at desired anchor locations, particularly if just above the roof line and accessible from at the parapet. Care should be taken so that the anchor force in a grouted cell does not exceed the force that can be transferred by the unit bed joint.

Collector connections: Where possible, it is desirable for the collector member to be located at the face of the shear wall and extend the full length of the shear wall, matching the chord detailing shown in Figures 22.2.2-1A and 22.2.2-1B and Figures 22.2.2-2A and 22.2.2-2B. This detailing approach is often but not always possible. Great care should be taken when a significant collector load needs to be transferred into the very end of a concrete or masonry wall. The load needs to be transferred far enough into the wall that wall reinforcing can develop adequate capacity. Edge and center to center spacing requirements need to be met for anchorage to the wall.

Cost, Disruption and Construction Considerations

When rehabilitation work is undertaken on the roof diaphragm, it is important that the cost and the preferred location for work take into account the combination of work, rather than considering one portion 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.

Proprietary Concerns

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

22.2.3 Add or Enhance Diaphragm Cross-ties for Out-of-Plane Wall-to-Diaphragm Loads in Flexible Wood and Steel Diaphragms

Deficiency Addressed by Rehabilitation Technique

This rehabilitation technique addresses inadequate or missing diaphragm cross-tie systems, as part of wall anchorage requirements for flexible diaphragm / rigid wall buildings. This rehabilitation technique is used when diaphragm cross-tie systems have not been provided, or do not provide adequate strength. Both wood and steel flexible diaphragms are addressed. The diaphragm cross-tie system is an extension of wall to diaphragm anchorage for out-of-plane loads, as addressed in Chapter 16 for **PC1** buildings, Chapter 18 for **RM1t** buildings, and Chapter 21 for **URM** buildings.

The addition or enhancement of the diaphragm cross-tie system is recommended as a high priority for rehabilitation for wood diaphragm **PC1, RM1t,** and for **URM** buildings. Due to limited earthquake experience to date, the vulnerability of and need to rehabilitate cross-tie systems in flexible steel diaphragms is not known; however, vulnerabilities similar to wood diaphragms buildings might occur. This section illustrates the basic rehabilitation concepts. *SEAONC Guidelines* (SEAONC, 2001) provides exhaustive treatment of detailing for **PC1** buildings.

Description of the Rehabilitation Technique

A system of continuous ties between exterior walls of flexible diaphragm / rigid wall buildings is now a requirement for new construction in areas of high seismic hazard. The concept is to tie all the way across the diaphragm to opposing walls. The wall anchorage will generally occur at four, six or eight feet on center.

Cross-ties at each wall anchor location can be fairly easily accommodated in new steel deck diaphragm buildings. The steel deck is permitted to be used as the cross-tie in the direction of its span, provided it can be shown to be adequate for tension and compression forces. See Chapter 16 for further discussion. Perpendicular to the decking span, with relatively long-span steel joist members it is practical to provide diaphragm cross-ties at each joist. The number of cross-tie splices required is not excessive, and wall anchorage forces do not greatly change the open web joist design. This is also the preferred approach for rehabilitation of cross-ties in steel deck construction, where the forces can be accommodated by decking and joists.

Cross-ties at each wall anchor location are not as easily accommodated in wood diaphragm systems, particularly in panelized wood diaphragm systems with eight foot subpurlins spans, due to the number of breaks in framing members across which connectors would have to be provided. A cross-tie system using subdiaphragms has been developed for wood diaphragm buildings. This same approach can be used in steel diaphragm buildings. Rather than representing anticipated building behavior, subdiaphragms need to be viewed as a computational tool. Unit shears from subdiaphragm design are not intended to be added to main diaphragm shears. Design in each area of the diaphragm needs to be for the more critical of subdiaphragm or main diaphragm seismic forces.

Figure 22.2.3-1A illustrates a roof plan for a wood diaphragm that uses subdiaphragms as part of the cross-tie system. For loading in the east-west direction, subdiaphragms are provided between Lines A and B and Lines G and H. Similarly, for loading in the north-south direction, subdiaphragms are provided between Lines 1 and 2 and Lines 3 and 4. The depth of the subdiaphragm is selected based on the unit shear at the subdiaphragm reaction, as well as having a member available to act as a subdiaphragm chord. The wall anchor force is transferred into the subdiaphragm over the full subdiaphragm depth. For east-west loads subdiaphragms span between Lines 1 and 2, 2 and 3, and 3 and 4. Subdiaphragm reactions are resisted at the exterior walls at Lines 1 and 4, and interior cross-ties are provided on Lines 2 and 3. Boundary nailing must be provided for each subdiaphragm on Lines 1, 2, 3, 4, A and B. The cross-tie provides a continuous tie between exterior walls with a capacity not less than the subdiaphragm reaction. This pattern is repeated for subdiaphragms between Lines G and H, 1 and 2, and 3 and 4.

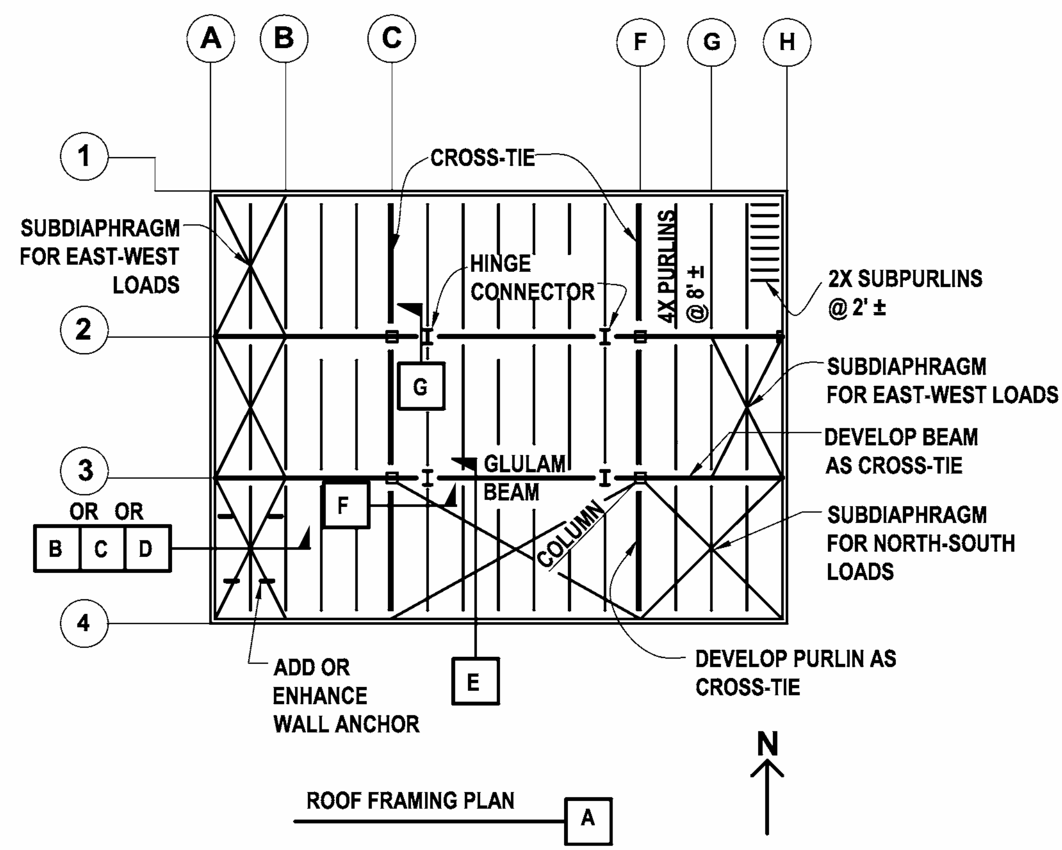


Figure 22.2.3-1A: Roof Plan with Diaphragm Cross-Tie System Using Subdiaphragms, Shown for Wood Diaphragm

Figures 22.2.3-1B, 22.2.3-C, and 22.2.3-D depict sections through the subdiaphragm extending between Lines A and B. Figure 22.2.3-1B shows the assumed subdiaphragm where existing roof sheathing is not being modified. The subdiaphragm depth will be controlled by the capacity of the existing sheathing. The wall anchor engages each wall purlin across the subdiaphragm depth.

Existing subpurlin-to-sheathing nailing must be adequate to transfer the wall anchor force to the subdiaphragm. In Figure 22.2.3-1B the added wall anchor is located between existing subpurlins in order to engage more existing sheathing nailing. Sheathing fastening to subpurlins must be assumed to be field nailing unless edge nailing has been confirmed.

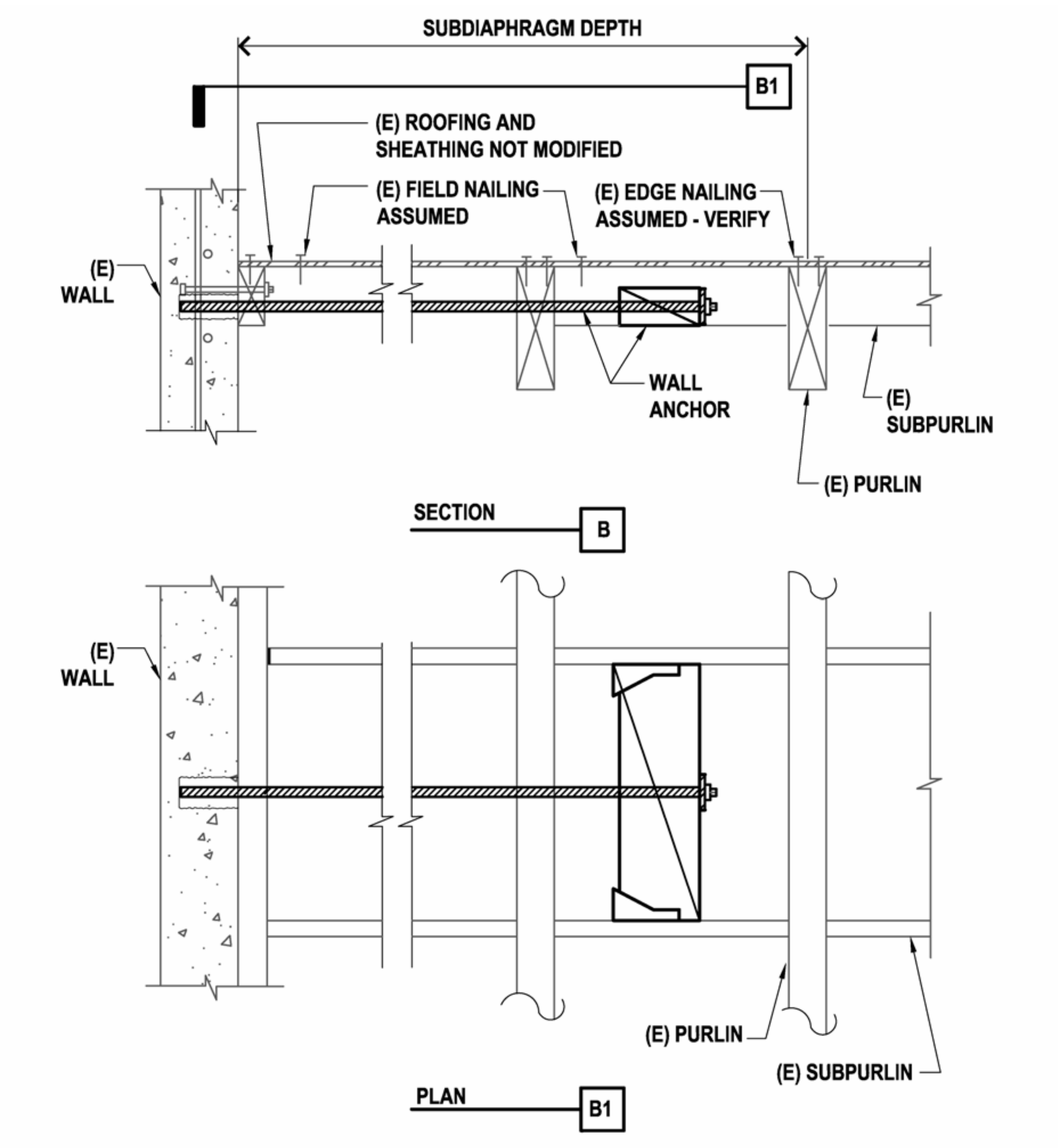


Figure 22.2.3-1B: Subdiaphragm for Flexible Wood Diaphragm – Roofing Not Removed

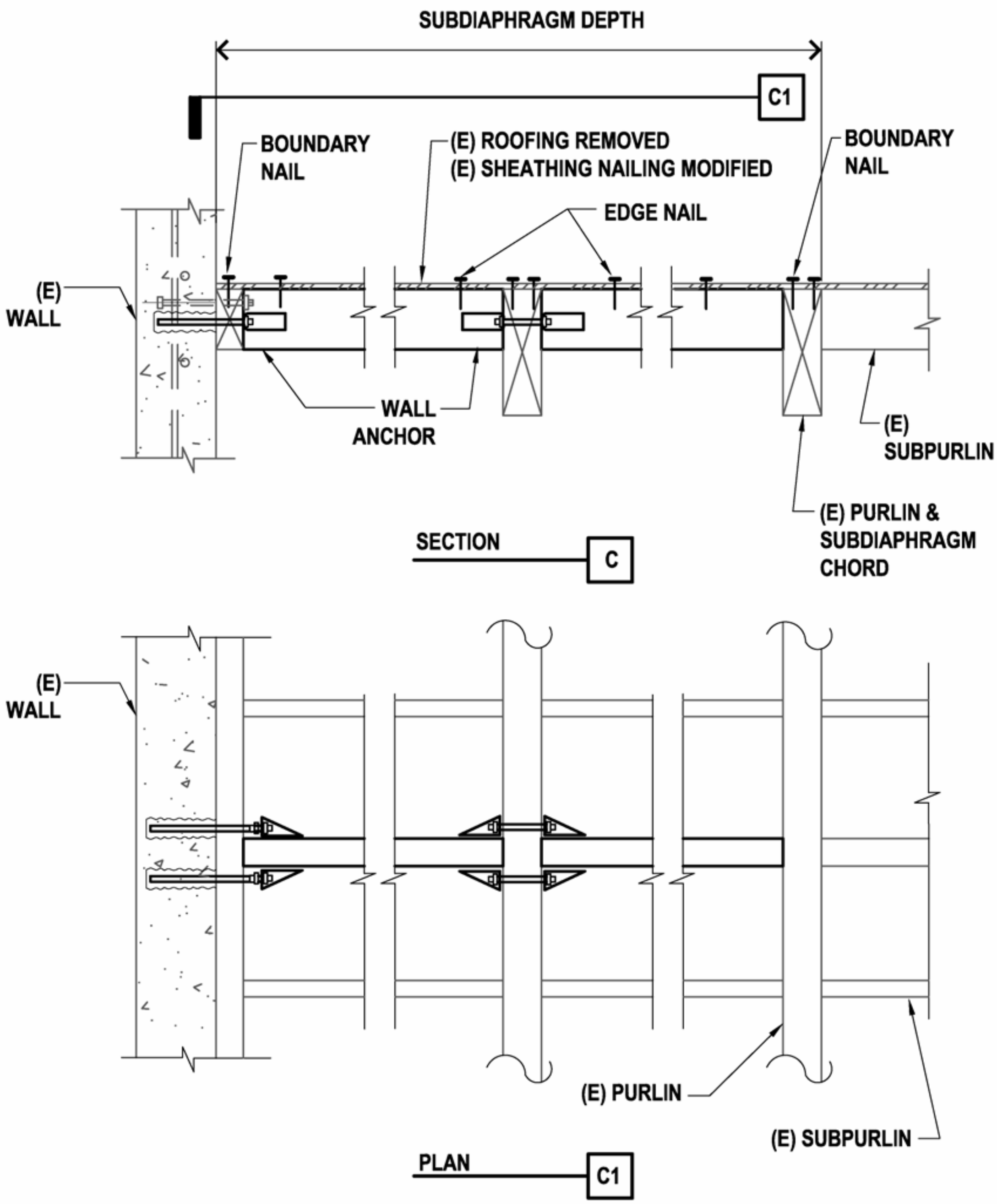


Figure 22.2.3-1C: Subdiaphragm for Flexible Wood Diaphragm – Roofing Removed

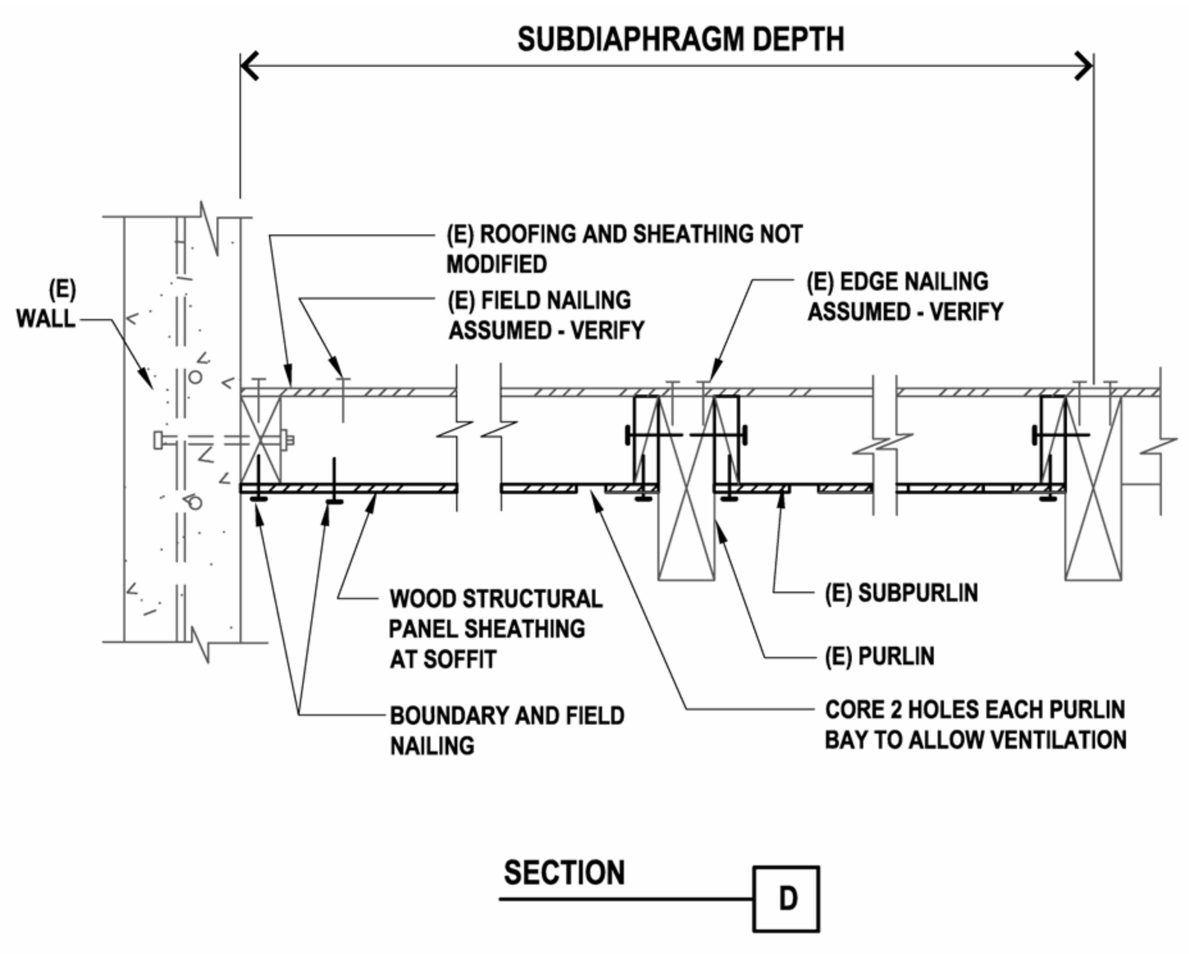


Figure 22.2.3-1D: Enhanced Wood Subdiaphragm with Added Wood Structural Panel Soffit

Figure 22.2.3-1C depicts a subdiaphragm where access from the top is assumed, and the subdiaphragm can be renailed to meet required demands. A new member is provided at the wall anchor. Tie-downs are added to carry the wall anchorage force across the entire subdiaphragm width. Figure 22.2.3-1D illustrates a third subdiaphragm alternative where new subdiaphragm sheathing is provided as a soffit at the underside of the roof framing. Wall out-of-plane anchorage is not shown, but would be similar to Figure 22.2.3-1C. Attention is needed to providing shear transfer into the main diaphragm at all subdiaphragm boundaries. See other chapters for additional discussion of wall anchorage.

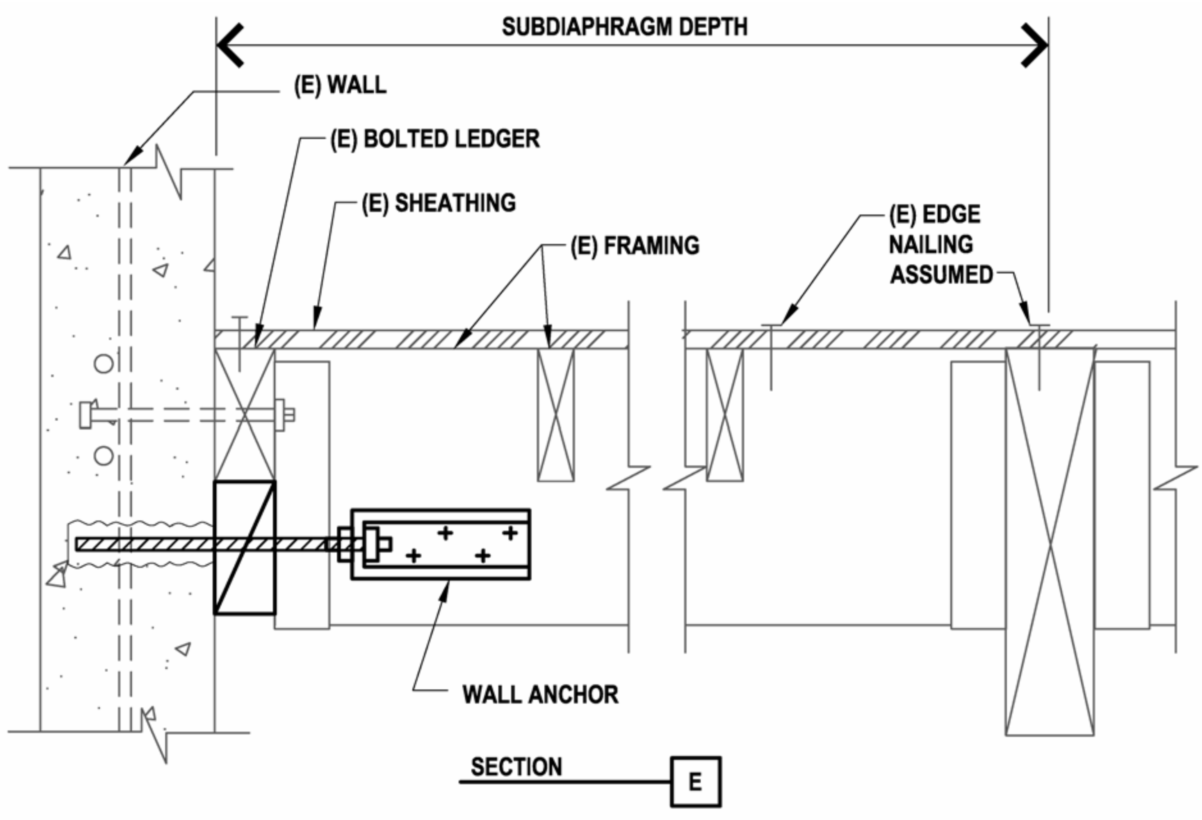


Figure 22.2.3-1E: Subdiaphragm for Flexible Wood Diaphragm at Purlins

Work can be conducted either from the underside or the top of the diaphragm. Location of access needs to be decided early on in the design process and will drive both calculations and detailing of the rehabilitation work. Where the roofing is not going to be removed, it is possible to strengthen the diaphragm in local areas by sheathing the underside of the roof subpurlins, as shown in Figure 22.2.3-1D. This is expensive and tedious work that should not occur over large areas, but may be advantageous for reinforcing of subdiaphragms in combination with wall anchorage.

Figure 22.2.3-1E illustrates anchorage of the north and south walls into subdiaphragms extending between Lines 1-2 and 3-4.

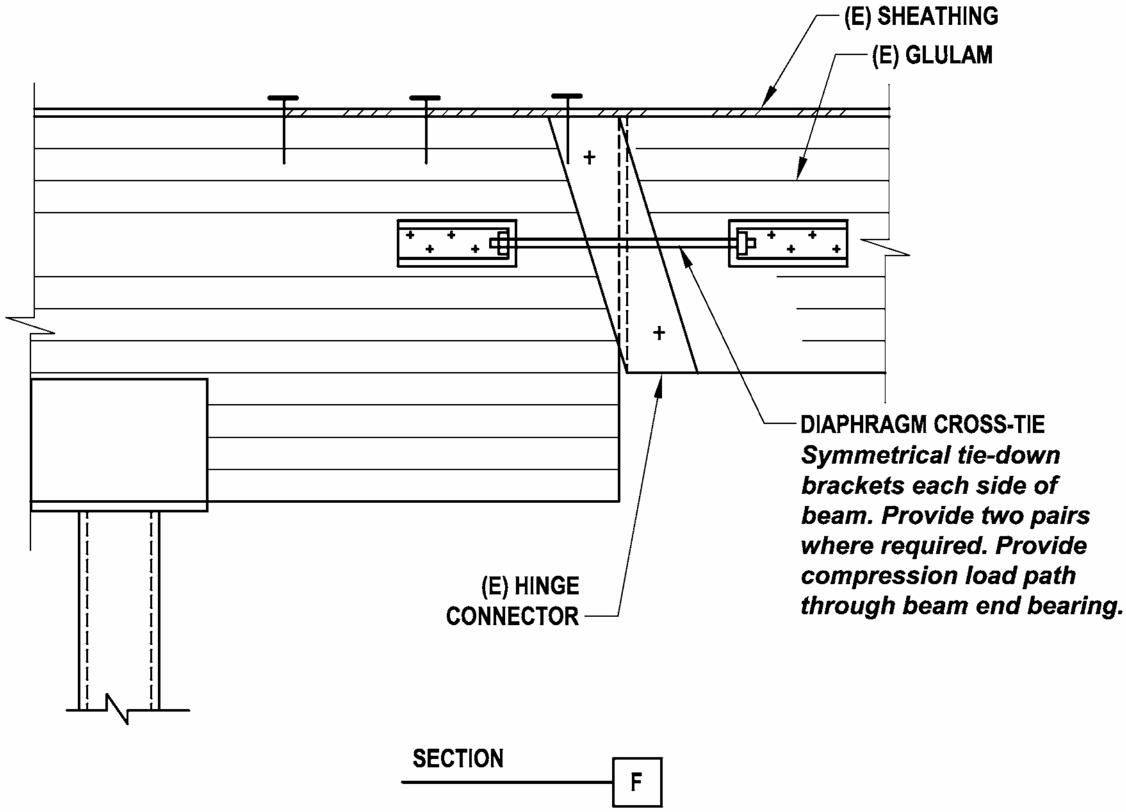


Figure 22.2.3-1F: Cross-Tie for Flexible Wood Diaphragm at Glulam Beams

Figure 22.2.3-2A illustrates a similar roof plan with a steel diaphragm. Figures 22.2.3-2B through 22.2.3-2D provide details. Instead of using subdiaphragms, direct ties are provided. Alternative connections locations for field welded connections between joists (Figures 22.2.3-2C and 22.2.3-2D) include the joist top chord, vertical and horizontal legs. The alignment of joists at support locations will greatly affect the connection detail used, so field determination of detail and alignment should be made. See Section16.4.1 for additional discussion.

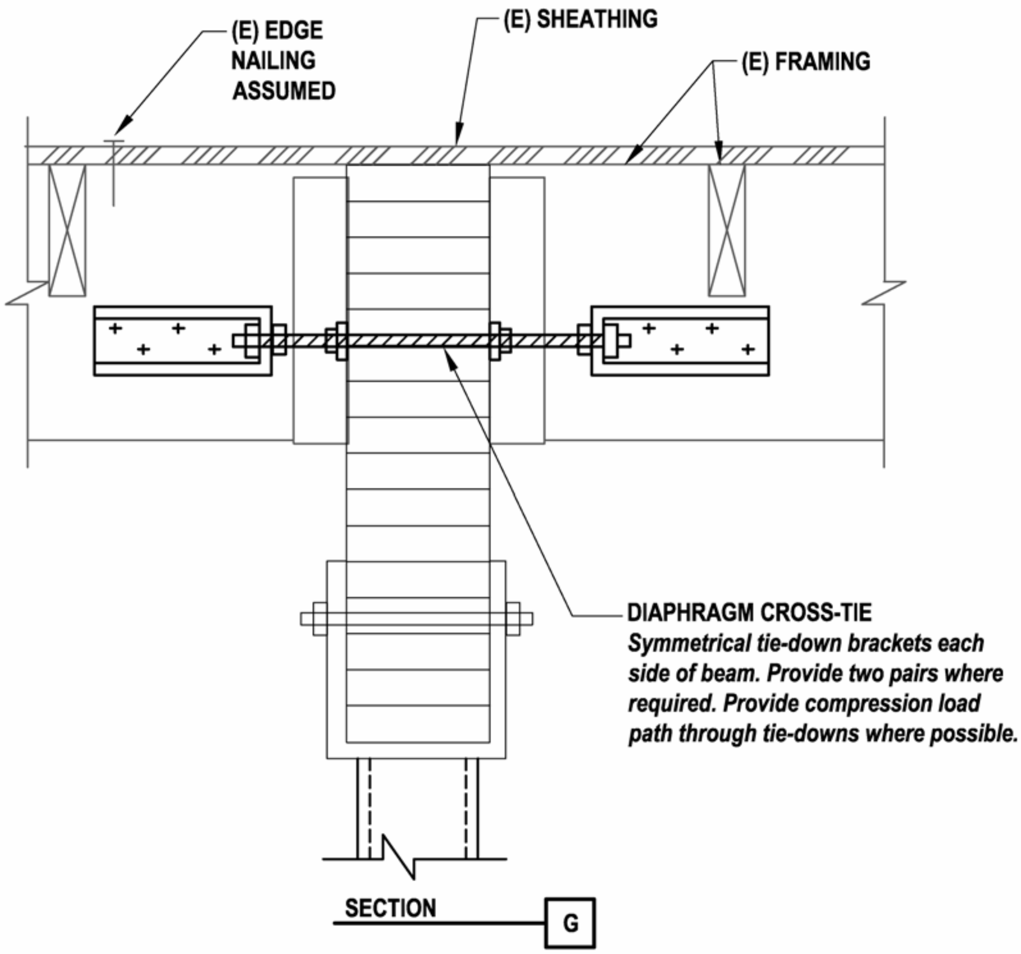


Figure 22.2.3-1G: Cross-Tie for Flexible Wood Diaphragm at Purlins

Design and Detailing Considerations

Research basis: No research relating 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); and Ghosh and Dowty (2000).

The reader is referred to the extensive discussion in the *SEAONC Guidelines* for design and detailing considerations for the wood diaphragm.

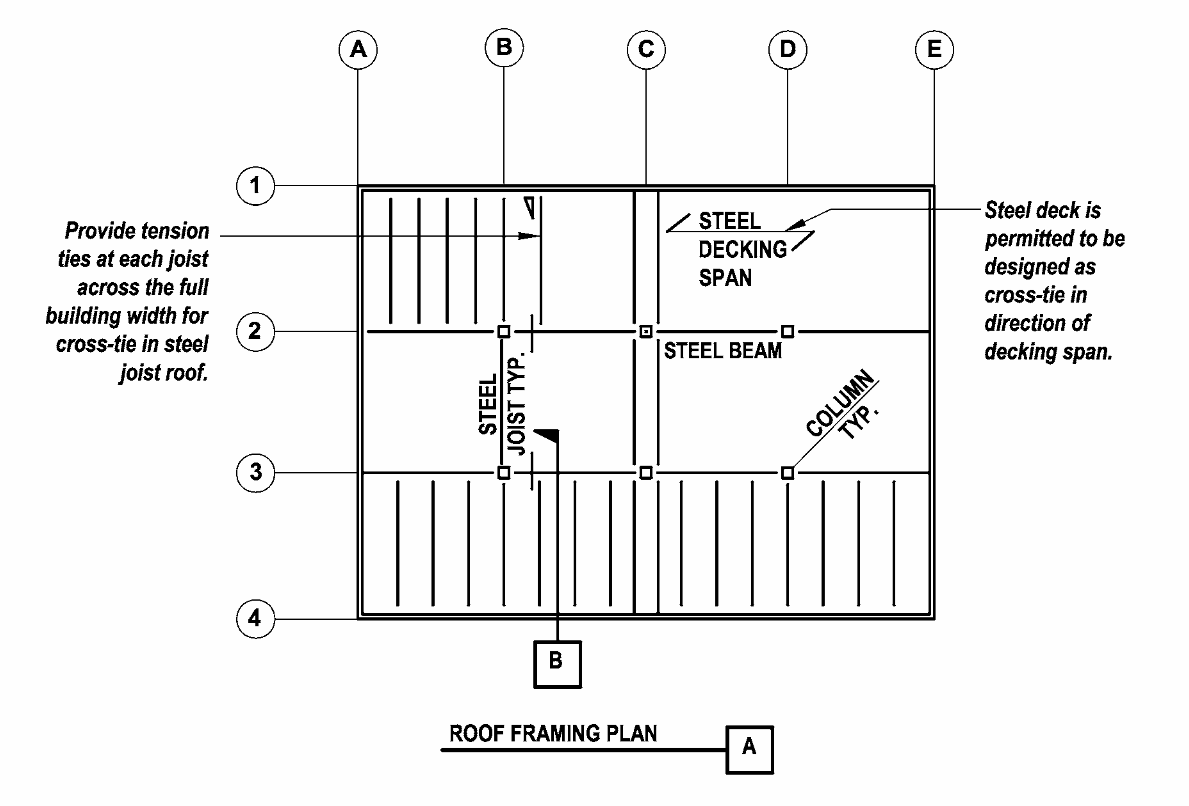


Figure 22.2.3-2A: Roof Plan with Diaphragm Cross-Tie System Using Direct Ties, Shown for Steel Diaphragm

Cost, Disruption and Construction Considerations

When rehabilitation work is undertaken on the roof diaphragm, it is important that the cost and the preferred location for work 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 adhesives as part of the assemblage.

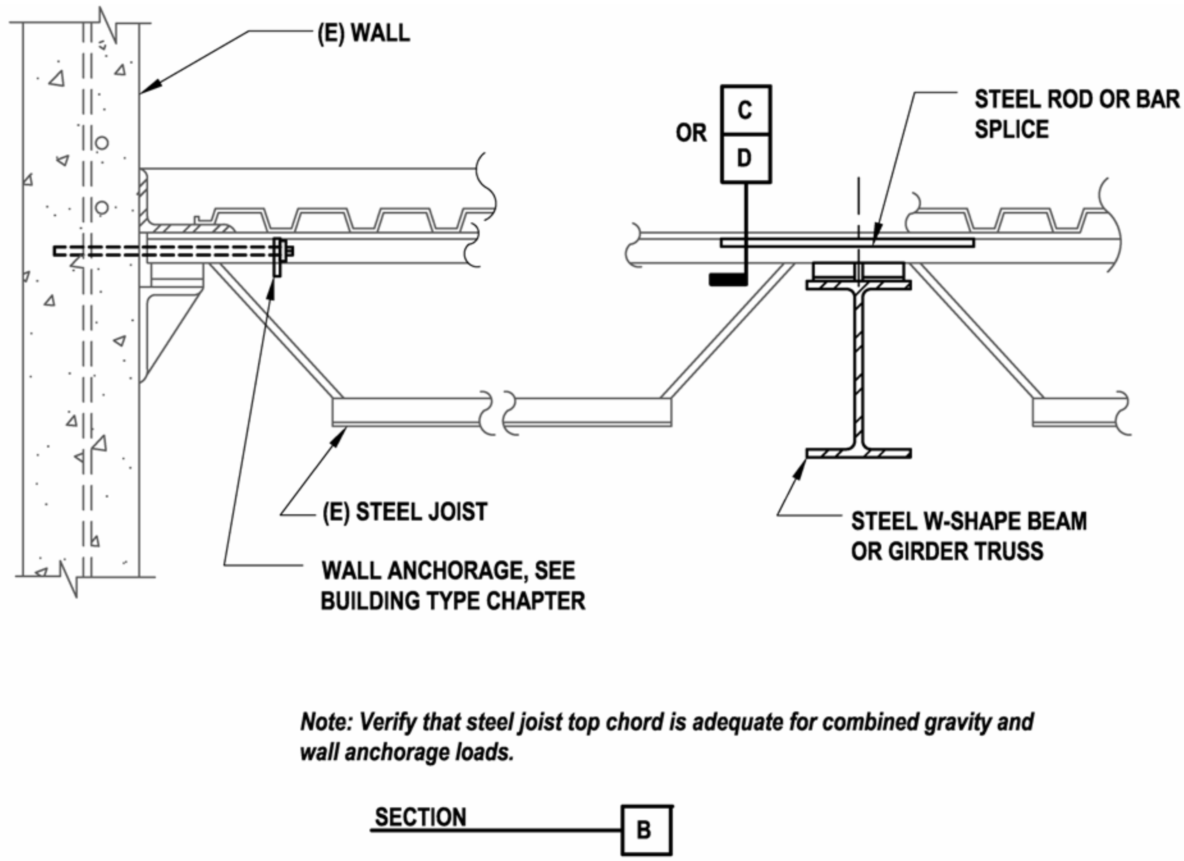


Figure 22.2.3-2B: Cross-Tie for Flexible Steel Diaphragm

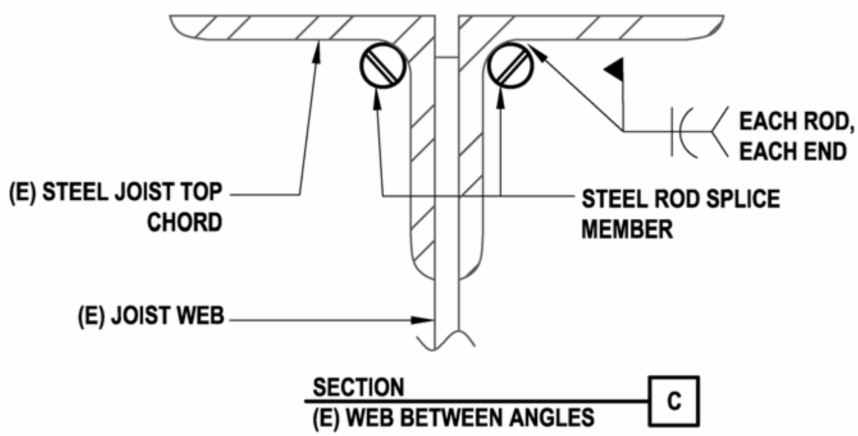


Figure 22.2.3-2C: Steel Open Web Joist Connection for Diaphragm Cross-Ties

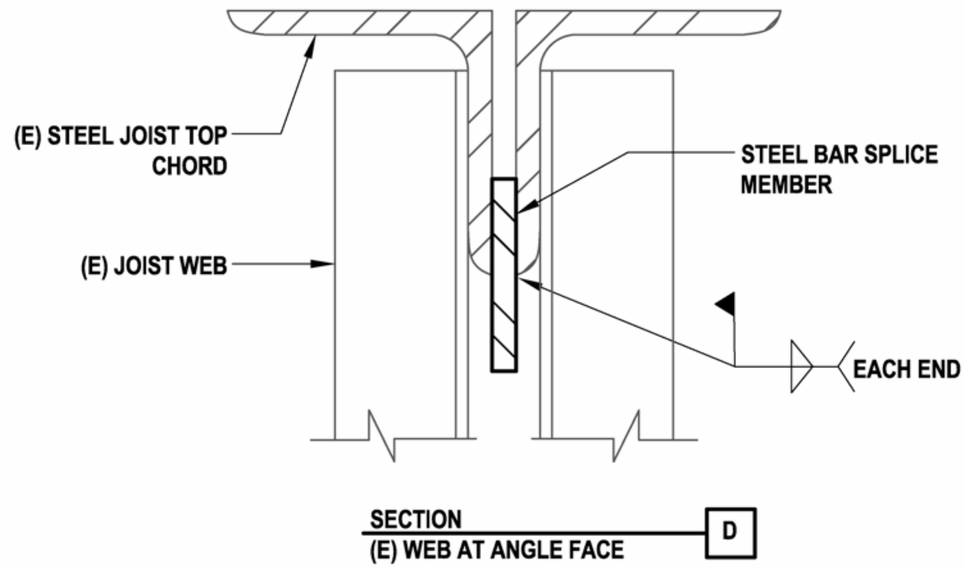


Figure 22.2.3-2D: Steel Open Web Joist Connection for Diaphragm Cross-Ties

22.2.4 Infill Opening in a Concrete Diaphragm

Deficiencies Addressed by the Rehabilitation Technique

Inadequate diaphragm shear or chord capacity at existing opening.

Description of the Rehabilitation Technique

Addition of a structural infill to close an existing opening is a relatively simple method of correcting this type of local diaphragm deficiency in a concrete diaphragm. The new infill will reduce concentrated shear and chord force demand in the surrounding diaphragm and eliminate the need for often nonexistent local chords around the edges of the opening. In almost all cases, the new infill will be made with cast-in-place reinforced concrete or shotcrete. While it is conceivable, and perhaps possible in some unusual cases, to close the opening with steel plate or a precast concrete "plug," the connections to the surrounding slab are very problematic, and their effectiveness as a mitigation measure is doubtful.

Design Considerations

Gravity load support: In addition to diaphragm shear demand, a new infill of an existing opening will create new floor or roof area which must be designed to support its self weight and the associated live load. In addition, the surrounding floor or roof system must be capable of supporting the gravity loads delivered from the newly infilled area. For larger infills, new beams may be required, both in the infill area and at the affected surrounding slabs, to provide this capacity.

Detailing Considerations

Connection to existing concrete floor and roof diaphragms: Typical details of a reinforced concrete (cast-in-place or shotcrete) infill are indicated in Figure 22.2.4-1. Sufficient dowels must be placed into the existing diaphragm slab on all sides of the opening to transfer the required shear demand to and from the infill section. Forms may be supported from the floor below or suspended from the surrounding floor or roof. This latter option is much more common for smaller openings or for openings surrounded by waffle ribs, pan joists or beams. Since the concrete infill will shrink relative to the surrounding slab, some care should be given to use shrinkage compensated mix.

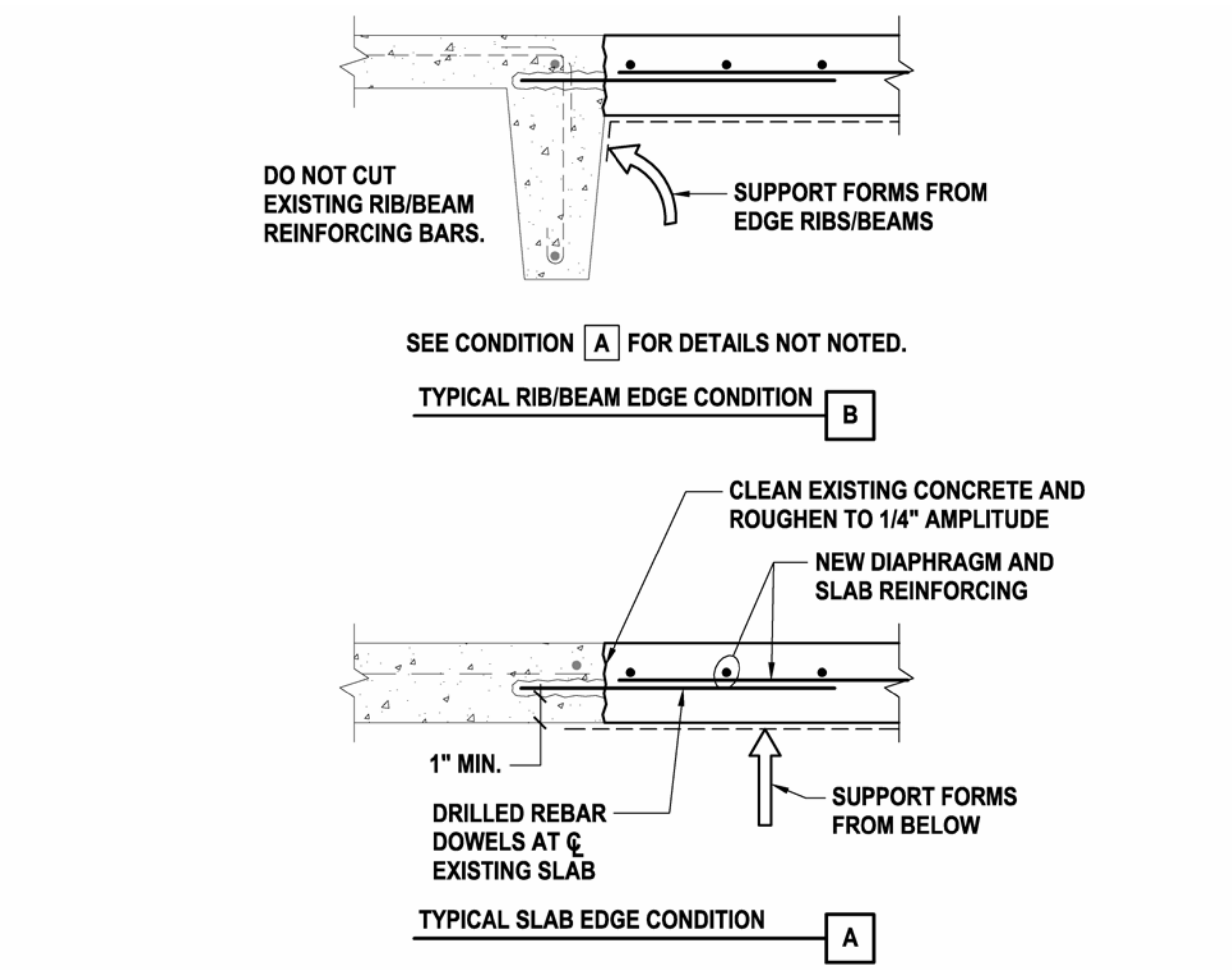


Figure 22.2.4-1: Typical Infill Opening in a Concrete Diaphragm

Cost/Disruption Considerations

The cost of this type of infill is very modest and will generally be a very small component in the overall retrofit project. Except for the noise and vibration associated with the dowel drilling, disruptions associated with this type of infill will be very localized, affecting only the immediate surrounding floor area and the area on the floor below.

Construction Considerations

The existing concrete surfaces around the entire perimeter of the existing opening to be in contact with the new concrete infill should be thoroughly cleaned of all finishes, paint, dirt, or other substances and then be roughened to provide ¼” minimum amplitude aggregate interlock at joints and bonded surfaces. Alternatively, a lower μ-factor and more dowels can be used with less roughening.

For shotcrete applications, separate test “panels” should be made to represent the slab infill work in addition to the normal test panels for shear walls. Nozzle operators should have several years experience with similar structural seismic improvement applications.

22.2.5 Add Fiber-Reinforced Polymer Composite Overlay to a Concrete Diaphragm

Deficiencies Addressed by the Rehabilitation Technique

Inadequate shear capacity in a slab

Description of the Rehabilitation Technique

The use of an FRP overlay with slabs for in-plane shear strength (diaphragm shear) enhancement is a very new technique that has had limited implementation. For shear enhancement of monolithic slab construction, the fibers are oriented parallel to the applied shear direction. The technique is also used for precast floor systems, where the shear plane is the joint between panels. Joint strengthening usually employs bi-directional fibers orientated at 45 degrees to the shear plane.

Design Considerations

Research basis: Although there has been a significant amount of research conducted on flexural strengthening of concrete slabs or strengthening of bridge decks using FRP overlays, published research focused specifically on strengthening of concrete diaphragms using FRP overlays has not been identified. Designers have typically considered results of tests performed on FRP composite strengthened shear walls relevant for diaphragm strengthening applications.

Chord and collector considerations: The diaphragm usually resists seismic loads in both directions, which requires bi-directional fiber orientation.

While shear transfer between two concrete elements has been tested and proved to be reliable, there are diaphragm internal forces termed *chord* and *collector* forces. This rehabilitation technique, which may have been intended solely as a shear enhancement may, in fact, have chord and collector force demands.

Chord actions, which develop from in-plane flexing of the full diaphragm depth, are developed in boundary elements gradually over the span length. These forces can be very high. The limited bond capacity and difficulty of anchoring the FRP composite may prohibit development of such large forces. Further, the strain limitations of the FRP composite prevent significant yielding; hence, the diaphragm chord forces should be based on the diaphragm forces required to yield the vertically-oriented elements of the lateral force-resisting system. This force level would be similar to a code level force multiplied by omega, an over-strength factor, which is the same force level used to design diaphragm collectors.

The use of FRP composite overlay to provide collector type load transfer is more difficult than that for chords. The collector force is usually being transferred from the diaphragm to a concentrated location, such as a brace frame or shear wall element. Strain compatibility and anchorage issues discussed with the bond-critical application (see Section 13.4.1, “Enhance Shear Wall with Fiber-Reinforced Polymer Composite Overlay, Fiber-Reinforced Polymer Overview, Requirements at the FRP-to-Substrate Interface”) prevent reliable transfer of the collector force to the frame of wall element.

If, however, this technique must be used, then the bond, load transfer, strain compatibility, uncertainty in diaphragm demand forces, etc. must be carefully considered and reflected in the design and details.

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

Detailing Considerations

Given the high dependence on the bond strength of the FRP overlay to the substrate, in situ bond testing is recommended as part of the contract documents. A testing program will verify the design assumptions and assist in providing quality assurance. The vertical offset between the two slabs should be minimized. This can be achieved by removing surface projections and applying leveling compound to ensure that the FRP composite overlay does not exceed the 1-2% out-of-plane angle. Offsets exceeding this limit or lack of bond between the leveling compound or substrate and the polymer may cause premature delamination.

In many situations, improvement in shear transfer capacity at the edge of the diaphragm will be needed in addition to enhancement of the capacity of the diaphragm itself. Transfer details from the slab to the wall using FRP need careful consideration. See Figure 22.2.5-1. Typically, the fiber is lapped from the slab to the wall, and fibers are oriented at 45 degrees (in plan view) to the length of the wall. The 90 degree bend in the fiber at the turn to the wall creates several issues. First, preparation of the existing sharp corner with resin putty is needed to allow a reasonable radius for the fiber. Second, when shear forces develop, they create tensile forces in the fiber. Because of the bend in the fiber, a substantial out-of-plane component is developed which must be resisted. The bond stress of the fiber has limited capability to take this force, usually leading to the need to reinforce the bend with mechanical means. A cut pipe placed against the corner, matching the radius of the curve, can be anchored with drilled dowels through the fiber to the wall or slab. Finally, testing to date of slab-to-wall shear transfer details is limited, necessitating increased caution.

Construction Considerations

Should underside of slab strengthening be used, the utilities at this location may need to be removed and reinstalled. This could impact building function during the construction period, and will add to the construction cost. For above slab strengthening architectural finishes, thresholds, and slopes will need to be considered.

Proprietary Concerns

See Section 13.4.1 for brief discussion of proprietary concerns.

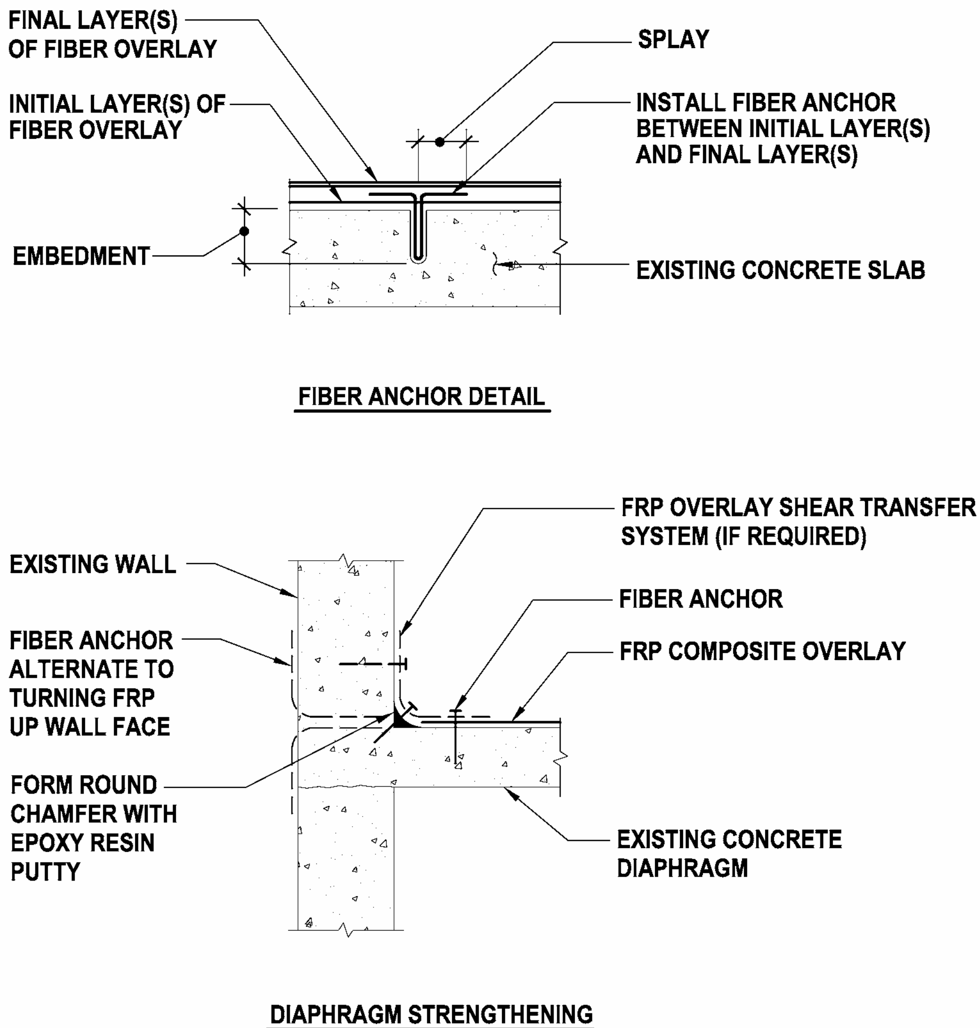


Figure 22.2.5-1: Shear Strengthening of Concrete Diaphragm Using FRP Composite

22.2.6 Infill Opening in a Concrete Fill On Metal Deck Diaphragm

Deficiency Addressed by Rehabilitation Technique

Increase diaphragm shear and/or chord capacity by infilling opening.

Description of the Rehabilitation Technique

Adding infill to an existing opening is a simple method of reducing local stresses around the opening as well as the demand on the diaphragm. However, this technique can only be employed if an existing opening is no longer necessary for the function of the building. Thus, it would likely have to coincide with other building renovations that eliminate the function of the opening. The opening may have been used for stairs, an elevator shaft, a pipe and conduit shaft, or an atrium. The infill should be constructed in a similar manner as the existing diaphragm when possible, using similar types of metal deck and concrete as well as reinforcing steel layout. This ensures that the infill matches the strength and stiffness of the surrounding diaphragm. The new metal deck can be connected to the existing deck with welds or fasteners while the new reinforcing steel bars are doweled into the edges of the opening. The edges of the opening should be roughened to ensure adequate bond between the new and existing concrete. For smaller openings, it may be acceptable to span the opening with a flat piece of gauge steel instead of metal deck, provided that proper measures are taken to fill the openings between the deck flutes.

Design and Detailing Considerations

Gravity loads: The infill has to support its self-weight and additional dead and live loads. The surrounding floor system should also be evaluated for these new loads. At larger infills, new steel framing may be required either directly below or at the edge of the infill.

Metal deck attachment: The new metal deck should overlap the existing metal deck around the perimeter of the opening. The deck can be attached to one another with puddle or seam welds, or mechanical fasteners, which may include expansion anchors, screws, or shot pins.

Bar development: Details of the reinforcement are similar to that for infilling an opening in a concrete diaphragm shown in Figure 22.2.4-1. Development lengths for the same size reinforcing bar will vary depending on the grout or adhesive product used to dowel the bar into the existing concrete. Bars on opposite sides of the openings should be spliced inside the opening. At smaller openings, the splice lengths will be limited by the size of the opening. The bars can be hooked in these cases for development. Adding bars to thin slabs will be difficult, particularly in the direction perpendicular to the metal deck flutes. Existing bars that are parallel to the flutes may be damaged while drilling holes for the new dowels. As an alternative, it may be easier to use welded wire fabric (WWF) instead of reinforcing steel. The slab would have to be chipped back around the opening to allow for development of the WWF.

Cost/Disruption

The cost associated with this technique is minimal compared to other diaphragm strengthening techniques, such as adding concrete overlays or horizontal braced frames. Since the infilling of an opening is likely related to other changes to a building, the disruption caused by the other changes are often more significant.

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

Many grout and adhesive products are available.

22.2.7 Increase Shear Capacity of Unfilled Metal Deck Diaphragm

Deficiency Addressed by Rehabilitation Technique

Strengthen inadequate bare metal deck diaphragm.

Description of the Rehabilitation Technique

Metal deck diaphragms are governed by either the capacity of the deck or its connection to other components of the lateral force-resisting system. Connection capacity is limited by the strength of the welds or other mechanical fasteners. At locations where welds or fasteners cannot be directly added, such as concrete walls, the addition of a steel angle connected with expansion anchors or adhesive dowels to a wall and diaphragm is often feasible. The capacity of a longitudinal joint between deck units is limited by the strength of the crimps or seam welds. These connections should be upgraded to the strength of the metal deck to achieve ductile diaphragm behavior during an earthquake. If the connections can develop the metal deck capacity, but the deck is found to be inadequate, significant increases in capacity may be obtained by adding a reinforced concrete fill or horizontal braced frame (Section 22.2.9).

Design and Detailing Considerations

Connections: In order to enforce deformation compatibility, new connections should be constructed similarly to the existing connections. Thus, puddle welds should be used if the existing diaphragm is welded to the steel framing. Similarly, the same types of mechanical fasteners should be used to match the existing fasteners when screws, shot pins, or expansion anchors are found at the connections.

Deck stiffeners: Some deck manufacturers fabricate stiffeners specifically intended for use with unfilled metal decks. The stiffeners are constructed to match the profile of the decks, which provide additional stiffness at the supports and in turn, increase the strength of the diaphragm. The stiffeners are typically welded to the deck and the steel beams.

Concrete fill: When reinforced concrete is added over metal deck, a shear transfer mechanism from the concrete to the lateral force-resisting system is required, e.g. welded shear studs at steel beams and drilled dowels at concrete members. Since the addition of a concrete overlay will increase the dead weight of the structure, the existing forces, members, connections, and foundation must be checked to determine whether they are capable of resisting the added loads.

Cost/Disruption

Diaphragm connection upgrades can be performed efficiently to minimize disruption and are cost effective if upgrades to other parts of the lateral force-resisting system are not required. If concrete fill is added, cost and disruption could increase significantly if upgrades are required to other parts of the lateral force-resisting system. Also, nonstructural elements such as insulation fill, roofing, and partitions would all require temporary removal.

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

Metal deck stiffeners are only provided by some manufacturers for use with their decks.

22.2.8 Enhance Masonry Flat Arch Diaphragm

Deficiency Addressed by Rehabilitation Technique

A relatively common type of floor in a masonry building or steel frame infill building, particularly outside the West Coast, uses narrowly spaced steel beams to support shallow or "flat" arches of masonry. The masonry can be made of hollow clay tile or brick. It is usually bearing on the bottom flange of the steel beam and supports nonstructural and acoustic fill above it. The horizontal kick from the base of the arch is balanced in the diaphragm interior by the adjacent arch. At the exterior, this kick either goes into the wall, or a tension tie of steel is provided at the bottom of the beams. In some cases, the steel strapping or bars run the full width of the diaphragm. When a tension tie is missing at the base of the arch and the diaphragm vibrates and expands, localized gravity failure can result when loss of arching action occurs. At the exterior of the diaphragm, the unbalanced kick of the arch can add to out-of-plane demands on the wall, contributing to out-of-plane wall failure and loss of vertical support. See Figure 22.2.8-1 for examples of failure scenarios.

Description of the Rehabilitation Techniques

There are several rehabilitation techniques for masonry flat arches floors. They can be combined for economy of scale.

Wall-to-diaphragm tension ties: Figure 22.2.8-2 shows the addition of tension ties from the wall to the steel beams for conditions when the beams are perpendicular to the wall and when they are parallel. When beams are perpendicular, an angle and drilled dowel is sufficient. When beams are parallel, strapping back to joists inside the floor is necessary. Figure 22.2.8-3 shows an example of placing the strapping on top of the beams, in case this is the preferred location for work.

Wall-to-diaphragm shear ties: The drilled dowels in Figure 22.2.8-2 also serve as ties for transferring shear forces from the edge of the diaphragm into the wall.

Chord: If the angle in Figure 22.2.8-2 is continuous, it can serve as a diaphragm chord.

Interior tension: While providing a tension tie for the case when the beams are parallel to the wall next to the wall is the most critical priority, it is desirable as well to continue the strapping all the way across the floor so local interior failure does not occur. Figure 22.2.8-4 shows the straps, plus notes the tension and shear ties and the chord.

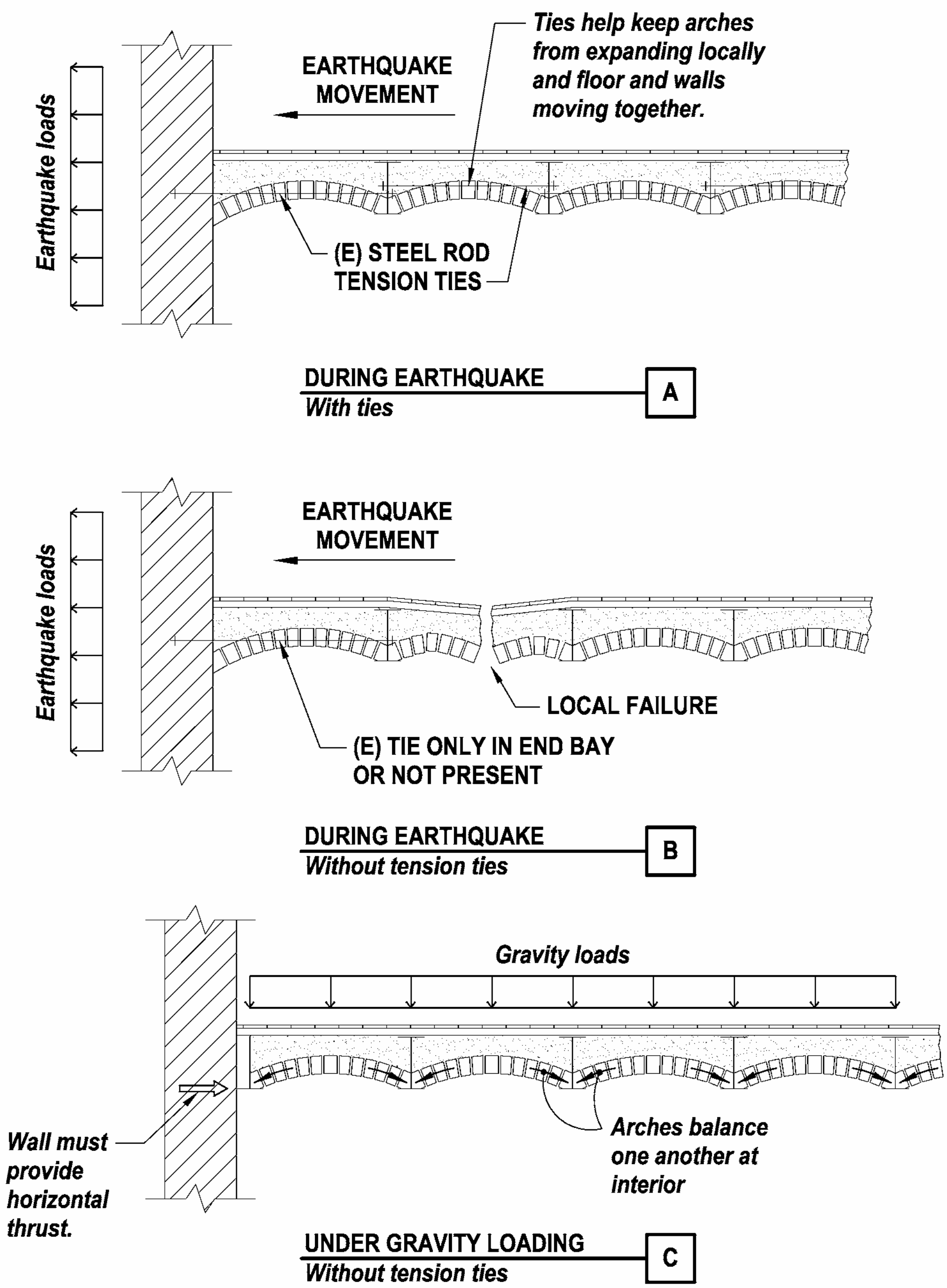


Figure 22.2.8-1: Failure Scenarios for Masonry Flat Arch Floors

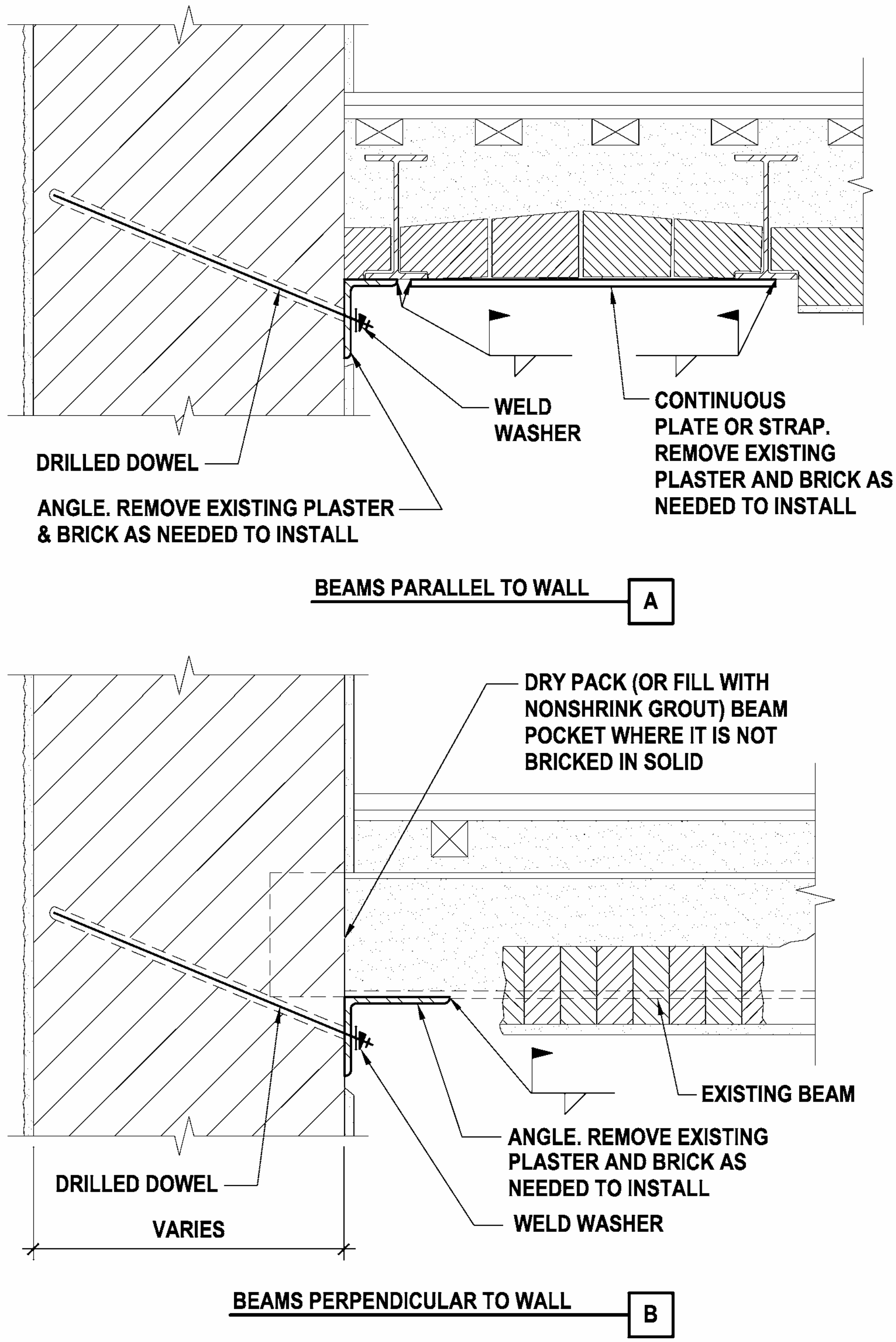


Figure 22.2.8-2: Add Wall-to-Diaphragm Ties and Chord for Masonry Flat Arch Floor - Access from Below the Floor

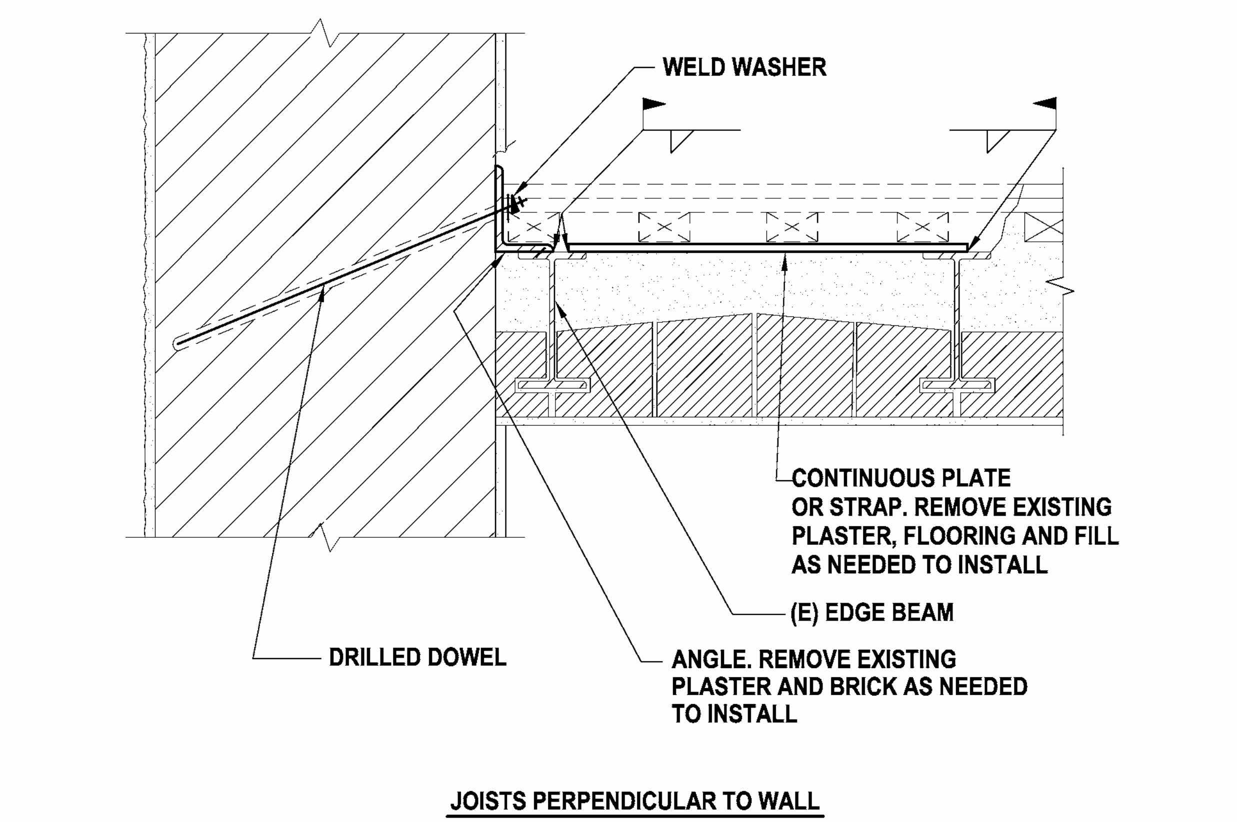


Figure 22.2.8-3: Add Wall-to-Diaphragm Ties and Chord for Masonry Flat Arch Floor - Access from Above the Floor

Diaphragm strengthening: Figure 22.2.8-4 also shows how adding diagonal bracing can be combined with existing beams and straight to create a horizontal braced frame diaphragm.

Topping slab: Theoretically, part of the flooring substrate can be replaced with a reinforced concrete diaphragm, though the vertical capacity of the floor would need to be sufficient and the weight of the new concrete adds to the inertial weight of the building.

Design Considerations

Research basis: No research specific to seismic rehabilitation of flat arch floors has been identified. There is also very limited information about how the floors have performed in actual earthquakes. There was some damage in the 1906 San Francisco Earthquake reported for these floors (Himmelwright, 1906) though much of the damage was due to fire. There are photos of the flat arch roof failures and reports of significant damage in Iranian earthquakes when tension ties are not present (Alimoradi, 2005).

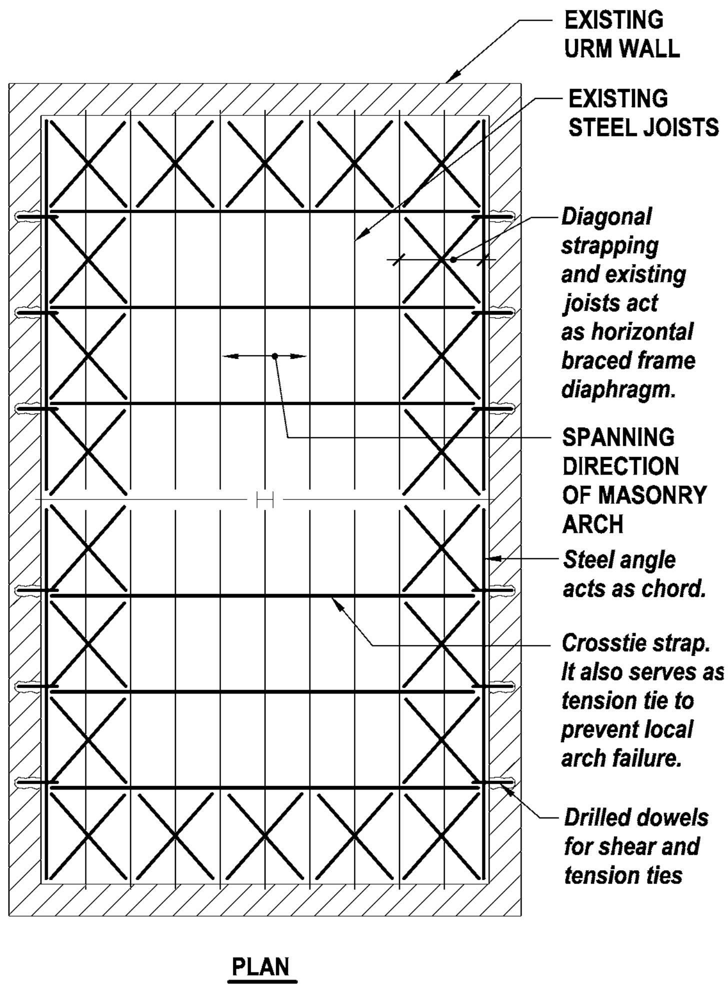


Figure 22.2.8-4: Masonry Flat Arch Floor Strengthening

Shear capacity: This type of floor has not been addressed by recent evaluation publications like FEMA 273 (FEMA, 1997a), FEMA 274 (FEMA, 1997b), FEMA 356 (FEMA, 2000), or ASCE 31-03 (ASCE, 2003), so capacity evaluations are from first principles. One strategy is to take all of the lateral force resistance in the new diaphragm strengthening due to the lack of interconnections in the diaphragm. Another approach is to develop strut-and-tie models in the diaphragm with the new and existing steel as ties and the masonry as a strut.

Stiffness: Although this floor lacks interconnections, it is likely to be quite stiff, as well as extremely heavy.

Detailing and Construction Considerations

Floor types: Lavicka (1980) is a reprint of an 1899 textbook on turn-of-the-century construction techniques and has an excellent summary of masonry flat arch variations. The system was intended to provide improved fireproofing and acoustic benefits. Flat tile arches were popular and had flat top and bottom surfaces to the tile, but beveled edges to create internal arching action. Side method arches had the voids in the hollow clay tile parallel to the beams; end method arches oriented the voids perpendicular to the beams. There were combinations of the orientations as well. The tile at the steel beam was usually notched around the bottom flange to provide masonry cover of the bottom of the bottom flange. Tile depths range from 6” to 12” with beams spaced from 3’6” to 7’6”. Segmental tile arches had shallow arches of several inches at the crown, the voids were parallel to the beams, and the end tile would bear on top of the bottom flange. Other systems have been observed to include clay bricks oriented with the long direction of the brick perpendicular and parallel to the beams. The masonry arches often supported a fill of cinders, sometimes mixed with mortar. This in turn would support wood sleepers spanning over the top of the steel beams and a wood floor. Tension ties were recommended; they were to be ¾” diameter rods placed near the bottom of the steel beam web and at about a spacing of 7’-8’.

Bottom cover: Figure 22.2.8-2 shows clay tile floors covering the bottom of the bottom flange. There is typically plaster adhering to the masonry. To install steel strapping, the plaster and masonry must be notched. Figure 22.2.8-3 shows an alternative to avoid damaging the underside by adding steel plate or straps, but working from the top. Of course, this is quite disruptive to occupants as well. In some arch types, though, the bottom flange is not covered and adding steel from below is much less disruptive.

Cost/Disruption

Rehabilitation of a masonry flat arch floor can be quite disruptive and expensive, particularly when ties are necessary at the building interior and if plaster ceilings and masonry or floors must be temporarily removed and patched.

Proprietary Issues

There are no proprietary concerns with diaphragm improvements in masonry flat arch floors.

22.2.9 Add Horizontal Braced Frame as a Diaphragm

Deficiency Addressed by Rehabilitation Technique

Strengthen inadequate diaphragm.

Description of the Rehabilitation Technique

Providing a horizontal braced frame as a diaphragm strengthening technique is useful if the existing floor cannot be disturbed for functional reasons or the cost of replacing the existing diaphragm is more expensive (e.g., a sloped roof). This is also an alternative when concrete overlays add too much mass or lead to other construction complications. The existing diaphragm could be constructed of concrete filled or unfilled metal deck, or wood. The new horizontal bracing is added under the existing diaphragm, in which the existing framing with new diagonal members forms the horizontal bracing system. The diaphragm shears are shared with the existing diaphragm in proportion to the relative rigidity of the two systems. The design philosophy is generally to have the diaphragm remain essentially elastic, with the goal of achieving ductile inelastic behavior in the vertical lateral force-resisting elements. See Chapter 9 for a general discussion of braced frames.

Design Considerations

Force distribution: The diaphragm strength could be evaluated by considering boundary solutions. First, its capacity including both the existing diaphragm and the horizontal braced frame is determined based on their relative rigidities. This alternative may not be always be fully effective if the existing diaphragm has much greater rigidity of that of the bracing system, such as metal deck with heavily reinforced concrete fill. Thus, an evaluation should also be performed assuming failure of the concrete fill. The diaphragm strength would only include that of the braced frame with minimal contribution from the metal deck without the concrete fill. If the latter solution yields a greater value, extensive cracking of the concrete fill and greater diaphragm displacements would be assumed to be acceptable.

Sloped roofs: The horizontal braced frames could be sloped to match the roof slopes, which would require proper consideration of the slopes and their effects on the diaphragm forces. Alternatively, the braced frames could have a flat layout, but this may affect the functional space as well as aesthetics.

Brace members: Similar to the selection of members in braced frames, compact and non-slender sections are preferred for their ductility. Installation of the braces should be factored into their selection due to the logistics associated with delivering and attaching the braces to their final locations. Note the self-weight of the braces adds a component to the flexural forces that may be reduced by adding hanger rods.

Chords and collectors: The new horizontal bracing system requires continuous chord and collector members to receive the brace forces and transfer these forces to the lateral force-resisting elements. The existing members that serve this purpose should be used when possible, as shown in Figure 22.2.9-1.

Detailing Considerations

Connections: For steel structures, the braces can be welded or bolted with or without gusset plates to the existing framing. An example of a welded connection is shown in Figure 22.2.9-2. Bolting eliminates welding issues that include space restrictions and venting weld fumes while welding may permit smaller and more compact connections. In concrete structures, connection of the new horizontal bracing system to the existing vertical system is accomplished by welding braces to plates that connect to the walls or frames with mechanical fasteners, such as threaded dowels and expansion anchors.

Cost/Disruption

These costs of adding horizontal bracing must be weighed against that of a concrete overlay. Temporary removal or relocation of nonstructural elements such as piping and partition walls are required and should be included in the cost evaluation for both options. The horizontal braced frame requires connection modifications, which are locally very disruptive.

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:

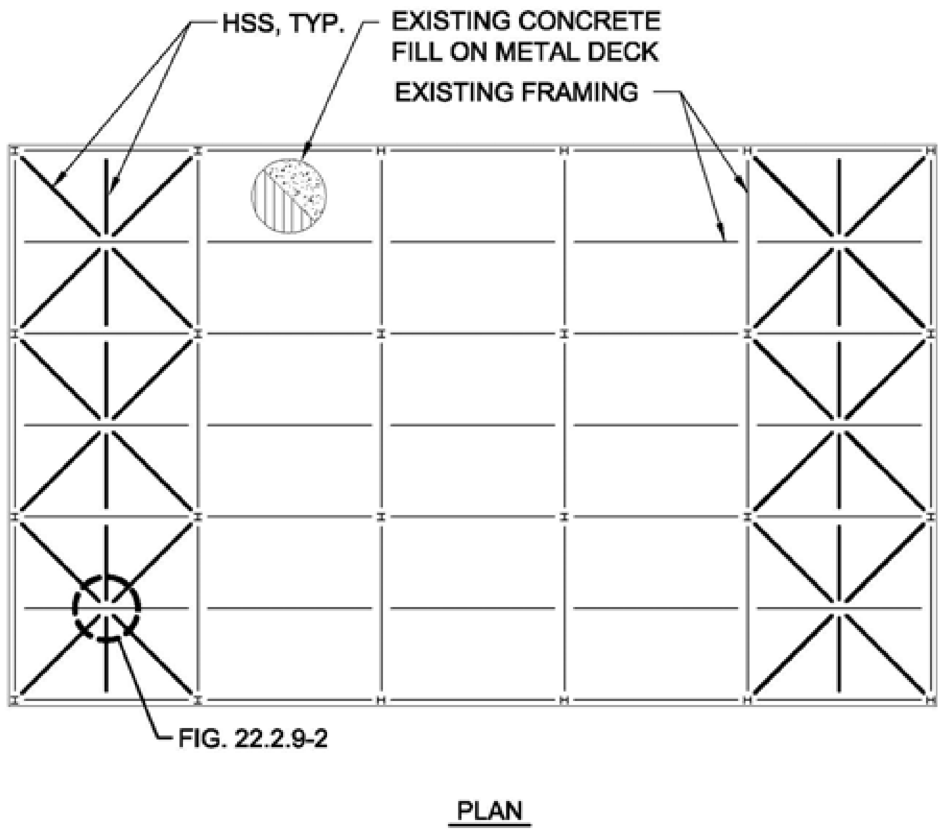


Figure 22.2.9-1: Diaphragm Strengthening using Horizontal Braced Frame

Welding/bolting issues: See general discussion in Section 8.4.1. Primary issues associated with bolting consist of typical field bolting issues such as set up, fit-up, and alignment.

Removal of existing nonstructural elements: This technique requires access to the underside of the floor or roof framing and may require relocation of piping, ducts, or electrical conduits as well as difficult and awkward connections to the existing framing. See Section 8.4.1 for discussions of fireproofing, asbestos, and concrete encasement.

Removal of existing structural elements: Existing structural elements do not typically have to be removed to add horizontal steel bracing. However, if required, shoring and temporary bracing may be necessary.

Construction loads: See general discussion in Section 8.4.1.

Proprietary Concerns

There are no known proprietary concerns with this technique.

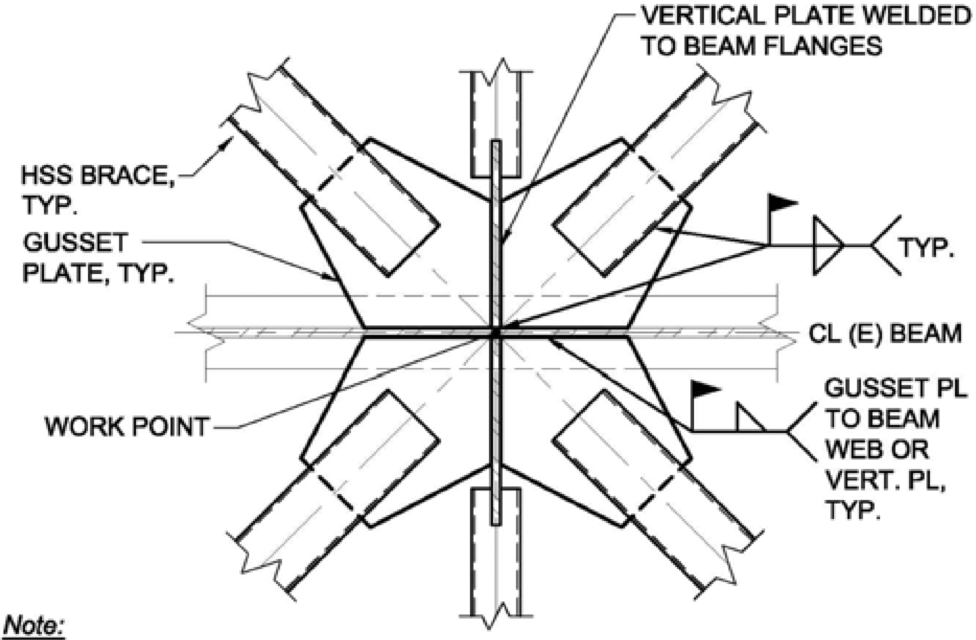


Figure 22.2.9-2: Horizontal Braced Frame Connection

22.2.10 Improve Tension Rod Horizontal Steel Bracing

Deficiency Addressed by Rehabilitation Technique

Repair nonductile tension rod bracing and/or connections

Description of the Rehabilitation Technique

Tension rod bracing consist of rods that are spliced together by turnbuckles and connected to clevis pins at the ends. The clevis pins are bolted to typical gusset plates. Tension rods that are inadequate for the seismic demands should be replaced entirely since it would probably be more complicated to upgrade existing rods. Increasing the rod size also requires replacing the turnbuckles and clevis pins. Connections that are inadequate can be upgraded similarly as typical braced frame connections. An example of a typical rod connection to a concrete or CMU wall is shown in Figure 22.2.10-1. The connection to the wall should develop the strength of the rod.

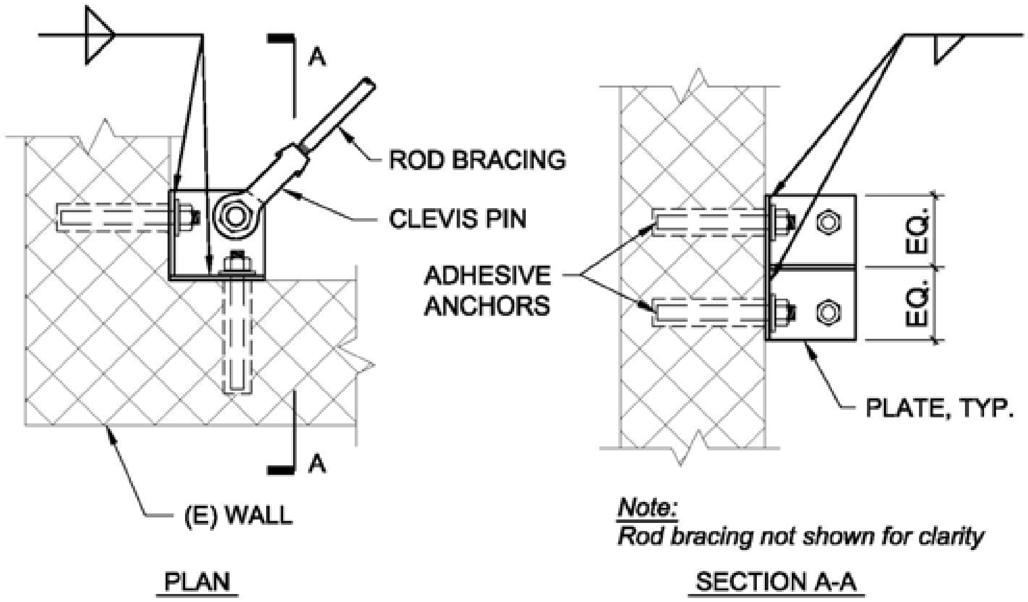


Figure 22.2.10-1: Tension Rod Connection at Wall

Design and Detailing Considerations

Tension rod bracing is used in applications where seismic forces are relatively low. It would be most appropriate for unfilled metal deck or wood diaphragms. The rod upgrades may increase the stiffness of the existing diaphragm and the total diaphragm force. Thus, all other elements of the lateral force-resisting system—connections, chords, collectors, frames or walls, and foundations—should be evaluated and upgraded accordingly.

Cost/Disruption

Replacing tension rods is fairly efficient on both a cost and time basis compared to other types of diaphragm upgrades. Connection modifications will only be locally disruptive and can be performed rapidly.

Construction Considerations

See Section 8.4.1 for general discussions of welding issues, removal of existing nonstructural and structural elements, and construction loads. Also see Section 22.2.9 for a discussion of construction issues related to modification of horizontal steel bracing.

Proprietary Concerns

There are no known proprietary concerns with this technique.

22.2.11 Improve Shear Transfer in Precast Concrete Diaphragm

Deficiency Addressed by Rehabilitation Technique

Inadequate diaphragm strength and/or stiffness

Precast diaphragm deficiencies and observed behavior have been discussed in some detail in Chapter 17; the reader is referred to these discussions. To date, construction of precast buildings in areas of high seismic hazard in the U.S. has been of limited quantity, resulting in limited opportunities to observe earthquake performance. The poor performance of some long-span precast diaphragms in parking structures in the 1994 Northridge earthquake has raised questions about shear capacity in diaphragms with topping slabs, excessive diaphragm deformation due to performance of chords and collectors and the interaction of shear and flexure. 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).

Description of the Rehabilitation Techniques

There are three types of precast concrete diaphragms commonly used: topped precast tee-beam, untopped precast tee-beam, and untopped precast hollow-core plank. Topped precast hollow-core may be used on occasion, but is not as common.

To date, very little rehabilitation of precast diaphragms has occurred in the U.S. As a result, the following discussion of rehabilitation measures draws from limited available research, suggested details for new precast construction, and application of rehabilitation techniques for concrete buildings to the specific configurations of precast elements.

Fiber-reinforced polymer (FRP) composite overlays provide one possible approach to shear connections between adjacent precast diaphragm members, and overlays could be used for any of the three common systems noted above. For parking structures, attention to both ultraviolet (UV) ray exposure and wearing under vehicle loads would be important to performance. The FRP overlay could be applied continuously over the area of high diaphragm shear and then used to transfer loads into supporting shear walls or frames, or applied locally at each member joint. Research by Pantelides, Volnyy, Gergeley, and Reaveley, (2003) on FRP composite connection between wall panels may be of interest; however, the reader is cautioned to consider the effects of simultaneous shear and tension at joints. See Section 17.4.2.

For untopped hollow-core diaphragms it may be possible to add construction roughly equivalent to that used for new construction. In new design, where diaphragm shear stresses exceed those allowed for grout key shear transfer, the cast concrete beams at the diaphragm perimeter or interior are used as flexural elements, resisting horizontal diaphragm forces. New beams could be added to serve this purpose. Connection between the precast sections and the new beams, either by bearing or mechanical connection is required. Attention to adequate strength and stiffness is also required.

Bolted steel plate connections providing shear connections from panel to panel are another possible approach. This involves use of a continuous plate or series of plates crossing the precast panel joint, with adhesive or expansion anchors on each side of the joint. Steel plate thickness must be selected in order to avoid plate bucking between connections. See Chapter 17 for discussion of anchors.

Design Considerations

Research basis: No research applicable to rehabilitation of precast diaphragm strength and stiffness has been identified; however, the following research for new construction may provide some guidance for rehabilitation:

> 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.
>
> *Shear Diaphragm Capacity of Untopped Hollow-Core Floor Systems* (Concrete Technology Associates, 1981) describes testing of grouted hollow-core joints. Note that issues raised by the Northridge earthquake might imply modification of testing approach.
>
> Research by Pantelides, Volnyy, Gergeley, and Reaveley, (2003) on FRP composite connection between wall panels.
>
> Research by K.S. Elliott, University of Nottingham, on untopped hollow-core diaphragms.

Basic design approach: The *PCI Handbook* (PCI, 1999) and *Design and Typical Details of Connections for Precast and Prestressed Concrete* (PCI, 1988) are basic references for design and detailing of precast concrete structures. These documents discuss the use of grouted keys for diaphragm-to-diaphragm connections, and they also recognize use of friction connections, without positive anchorage for wall-to-diaphragm connections. Use of these mechanisms must be given very careful consideration for possible inelastic seismic demands.

Shear and flexure interaction: One of the issues identified from the performance of parking structures in the Northridge earthquake is the interaction of shear and flexural deformations. Diaphragm deformations will result in tension and compression forces between adjacent diaphragm members. As a result, shear connections between members will need to accommodate simultaneous tension or compression plus shear. It is recommended that the diaphragm chord and collector members also be evaluated, and rehabilitated if necessary to control tension forces. It is also recommended that the rehabilitation measure chosen be capable of withstanding anticipated simultaneous forces. Methods of estimating diaphragm demands are proposed in Nakaki (1998) and Naito and Cao (2004).

Transfer into and out of diaphragm reinforcing: Transfer of loads is key to the use of fiber composites or steel plate for connecting between precast diaphragm segments. Where panel-to-panel connections are made, it is necessary to transfer the full design load in and out at each connection. In some cases it may become more practical to provide reinforcing over the entire diaphragm or highly loaded sections of the diaphragm; the load portions of load carried in the existing diaphragm and the reinforcing would need to be determined by deflection compatibility.

Topping slabs: The addition of a topping slab is seldom a practical approach because of the added weight for vertical loads and mass for seismic loads. In rare cases where additional vertical load capacity has been provided, this may be possible. The additional capacity is needed not only in the diaphragm slab and beam system, but in all of the vertical support system through the foundation. The removal and replacement of a topping slab could permit the addition of reinforcing and connections without increasing gravity or seismic loads. This, however, is a costly process.

Proprietary Concerns

Fiber composite materials and adhesive and mechanical anchors are proprietary and must be used in accordance with manufacturer and ICC-ES requirements.

22.3 References

ABK, 1981, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing*, A Joint Venture of Agbabian Associates, S.B. Barnes and Associates, and Kariotis and Associates (ABK), Topical Report 04, c/o Agbabian Associates, El Segundo, CA.

ASCE, 2003, *Standard for the Seismic Evaluation of Buildings*, ASCE 31-03, Structural Engineering Institute of the American Society of Structural Engineers, Reston, VA.

AF&PA, 2005, *National Design Specification for Wood Construction, ASD/LRFD*, American Forest and Paper Association, Washington, D.C.

Alimoradi, A., 2005, "Steel Frame with Semi-Rigid 'Khorjini' Connections and Jack Arch Roof 'Taagh-e-Zarbi'," *EERI World Housing Encyclopedia*, www.world-housing.net; Report 25.

APA, 2000, *Research Report 138, Plywood Diaphragms*, APA The Engineered Wood Association, Tacoma, WA.

Concrete Technologies Associates, 1981, *Shear Diaphragm Capacity of Untopped Hollow-Core Floor Systems* (TCA 38), Concrete Technology Associates, Tacoma, WA.

Dolan, J.D., D. Carradine, J. Bott, and W. Easterling, 2003, *Design Methodology of Diaphragms*, CUREE Publication No. W-27, CUREE, Richmond, CA.

EERI, August 1989, *Armenia Earthquake Reconnaissance Report*, Earthquake Spectra Special Edition, Earthquake Engineering Research Institute, Oakland, CA.

FEMA, 1997a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* FEMA 273. Federal Emergency Management Agency, Washington, D.C.

FEMA, 1997b, *NEHRP Commentary on the Guidelines for Seismic Rehabilitation of Buildings,* FEMA 274, Federal Emergency Management Agency, Washington, D.C.

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

Fonseca, F., S. Wood and N. Hawkins, 1996, "Measured Response of Roof Diaphragms and Wall Panels in Tilt-Up Systems Subject to Cyclic Loading," *Earthquake Spectra,* Volume 12, Number 4, Earthquake Engineering Research Institute, Oakland, CA.

Ghosh, S.K. and S. Dowty, 2000, *Anchorage of Concrete or Masonry Walls to Diaphragms Providing Lateral Support*, Draft 2000, Not Published.

Hamburger, R.O. and D. McCormick, 1994, "Implications of the January 17, 1994 Northridge Earthquake on Tilt-up Wall and Masonry Wall Buildings with Wood Roofs," *Proceedings of the 1994 Convention of the Structural Engineers Association of California,* Sacramento, CA.

Himmelwright, A., 1906, *The San Francisco Earthquake and Fire, A Brief History of the Disaster, A Presentation of Facts and Resulting Phenomena, with Special Reference to the Efficiency of Building Materials, Lessons of the Disaster,* The Roebling Construction Company, New York, NY.

ICBO, 1997, *Uniform Code for Building Conservation*, 1997 Edition, International Conference of Building Officials, Whittier, California.

ICC, 2003a, *International Building Code*, International Code Council, Country Club Hills, IL.

ICC, 2003b, *International Existing Building Code,* International Code Council, Country Club Hills, IL.

ICC-ES, 2004a, *303 Siding and High-Load Diaphragms,* ICC-ES Legacy Report ER-1952, ICC Evaluation Service, Inc, Whittier, CA.

Lavicka, W., 1980, *Masonry, Carpentry, Joinery,* reissued version of 1899 textbook of the same name, Chicago Review Press: Chicago, IL.

Naito, C. and L. Cao, 2004, "Precast Diaphragm Panel Joint Connector Performance," *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, B.C. Canada.

Nakaki, S., 1998, *Design Guidelines: Precast and Cast-in-Place Concrete Diaphragms,* Earthquake Engineeering Research Institute, Oakland, CA.

Pantelides, C. P., V. A. Volnyy, J. Gergeley, and L.D. Reavely, *Seismic Retrofit of Precast Concrete Panel Connections with Carbon Fiber Reinforced Polymer Composites*, PCI Journal, Vol. 48, No. 1, January/February 2003, Precast/Prestressed Concrete Institute, Chicago, IL.

PCI, 1988, *Design and Typical Details of Connections For Precast and Prestressed Concrete*, Prestressed Concrete Institute, Chicago, IL.

PCI, 1999, *PCI Design Handbook, Precast and Prestressed Concrete*, Fifth Edition, Prestressed Concrete Institute, Chicago, IL.

Peralta, David, Joseph Bracci and Mary Beth Hueste, 2004, "Seismic Behavior of Wood Diaphragms in Pre-1950s Unreinforced Masonry Buildings," *Journal of Structural Engineering*, ASCE: Reston, VA, Volume 130, Number 12, December, pp. 2040-2050.

SEAOSC (Structural Engineers Association of Southern California), 1979, *Recommended Tilt-up Wall Design* (Yellow Book), SEAOSC, Los Angeles, CA.

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*, SEAONC, San Francisco, CA.

Wan, G., R. Fleishchman, C. Naito, J. Restrepo, R. Sause, L. Cao, M. Schoettler, and S.K. Ghosh, 2002, "Integrated Analytical and Experimental Research Program to Develop a Seismic Methodology for Precast Diaphragms," *SEAOC 2004 Convention Proceedings*, Structural Engineers Association of California, Sacramento, CA.

Chapter 23 - Foundation Rehabilitation Techniques

23.1 Overview

While the need to add or supplement existing foundations for new superstructure elements such as shear walls and braced frames is relatively common in seismic rehabilitation, rehabilitation of existing foundation deficiencies is comparatively less common. There are two basic reasons for this: foundation work in existing buildings is quite expensive, and there has been relatively little note in earthquake reconnaissance reports of life loss and property damage resulting from foundation failures in buildings.

Foundation analysis can be one of the most challenging areas of seismic rehabilitation. Different assumptions regarding base conditions of restraint, soil properties, and locations and types of potential nonlinearity can lead to widely varying results. For many buildings, it can take significant analytical effort in modeling and evaluating interim results to understand how the foundation interacts with the superstructure and surrounding soil under earthquake loading. Often, the weakest link or governing mechanism may be a foundation element or soil yielding, but it is only after looking at the substructure and superstructure as a whole that the sequence and nature of element behavior can be determined.

In the past, force-based analytical techniques placed emphasis on strength capacity and whether the foundation and underlying soils were "overstressed". With the advent of displacement-based analytical techniques, the extent of soil movement is acknowledged as more critical. Due to the cost and disruption of foundation rehabilitation work, the consequences of foundation deflection should be carefully evaluated to determine if there are actually going to be unacceptable movements. Large soil movements from rigid body rotation of a shear wall, for example, may have minimal consequences if the entire structure rotates, but they may have significant consequences to attached adjacent elements which are not rotating in phase or at all.

When careful analysis reveals that new foundations must be added or that existing foundations must be enhanced, the structural engineer must have a good understanding of soil engineering issues; rehabilitation goals, performance criteria, and assumptions; and construction techniques and limitations. Obviously, it is usually much more difficult to perform work inside an existing structure than it is in a new building when the site is open. Because of the cost of foundation rehabilitation, other options should be fully explored, and the need for foundation modification should be thoroughly investigated.

There are relatively few, if any, proprietary issues associated with foundation rehabilitation, though some equipment used to install new elements in limited access areas may have been developed by a specialty contractor and thus not widely available.

This chapter provides a short discussion of general goals for foundation rehabilitation, brief mention of some key analytical considerations, and general construction issues; then provides discussion of structural rehabilitation techniques for foundations; reviews common ground improvement techniques; and ends with a short discussion of other ground hazards such as fault rupture, lateral spreading, and seismic-induced landsliding.

23.2 General Goals for Seismic Rehabilitation of Foundations

The goal of any seismic evaluation is to identify deficiencies, their relative likelihood of occurrence, and the hazards they pose. The foundation must not be ignored during the evaluation, and foundation behavior response must be placed in the context of the overall performance of the building. If the foundation is identified as the weak link, the type of foundation mechanism needs to be identified A shear wall might be overstressed in shear or bending if assumed to have a fixed base, but when its small foundation is considered, rocking or overturning might be the governing mechanism. A braced frame might have adequate strength and stiffness, but the pile caps its columns sit on may not have any reinforcing to take uplift forces that occur beyond code level forces. Existing drilled piers may lack adequate confining ties in the top of the pier or insufficient lateral resistance in general or their connections to the pier cap may be insufficient.

Consideration of the foundation is an integral part of the overall rehabilitation strategy for the structure. It may be possible to change the building behavior response by superstructure rehabilitation to preclude undesirable foundation modes. When foundation work is necessary, goals for rehabilitation design include providing sufficient strength, stiffness, and ductility for compression, tension, and lateral loading; identifying a defined and ductile mechanism of energy dissipation; and minimizing gravity stress redistribution within the existing foundation system. New foundations should not undermine existing foundations, either during construction or over the long-term. Moreover, the relative lower stiffness of unconsolidated soil under new foundations versus the higher stiffness under existing older foundations needs to be considered.

23.3 Construction Issues

Construction issues are quite critical during foundation work in existing buildings and will often drive the systems and techniques being considered. Issues include:

> *Access and height restrictions:* Installing shallow foundations, such as spread footings or grade beams, is usually done with hand methods or small excavation equipment and will rarely be a problem, though it will take longer than it would in a new building. Installing deep foundations, however, can run into several construction limitations. Drill rigs for piers, for example, are much more efficient when they are larger. Getting a drill rig into a building may require enlarging existing openings. Once inside, story heights will usually significantly limit the size of the drill rig that can be used. Special drills have been developed for use in existing buildings, but they often require at least 9 feet to 12 feet of vertical clearance. Drilling next to adjacent walls may limit the size of the pier or lead to shifting it inboard of the wall creating a horizontal eccentricity to be addressed.
>
> *Noise and vibrations limits:* Pile driving imparts significant noise and vibration. Even if there were clearance outside the building for a pile driving rig, the vibration is usually too significant. Drilled piers impart less vibration, though the noise requires consideration. Micropiles have even less vibration and noise, so they are a common rehabilitation technique.
>
> *Restrictions imposed by existing utilities:* Most buildings will have utilities beneath the existing ground level suspended floor or slab-on-grade. The locations and depths may

not be fully known. Excavating below grade requires careful effort, often with hand methods, so that utilities are not damaged.

Restrictions associated with ongoing operations: As with any rehabilitation work in the superstructure, if the building is occupied with people or equipment, foundation demolition, drilling, and excavation work will have to be coordinated.

Contaminated soil: There can be contaminated soil underneath the existing building, particularly if it has or had industrial uses. Removal of contaminated soil requires special techniques and must be taken to special landfills, increasing costs.

23.4 Analytical Issues

This document's focus is on detailing of rehabilitation techniques, not on analyzing the existing or rehabilitated structure, but it still worth pointing out a few analytical considerations in foundation modeling that often arise in seismic rehabilitation since codes and design guidelines provide limited guidance.

Modeling the base of the building: The most basic question to be established is: Where is the dynamic base of the building? If there is no basement, this is straightforward. When there is a basement, partial basement, or sloped site, this is not a simple issue. Say that the building is four stories above grade and has a one-story full basement. Figure 23.4-1 shows several possible modeling approaches. Model A is probably the most common approach—to stop the model of the superstructure at grade on a fixed base and take the results and impart them separately to the foundation walls and other elements. Model B is to ignore the ground entirely and put the base of the building at the bottom of the basement. When this is done, the inertial loads of the ground floor are usually not included. Model C is the same as Model B, except the ground floor loads are conservatively included. Model D changes the base conditions to account for vertical flexibility of the soil under the building. Significant modeling effort and variability have to be considered when springs are used. None of these models captures the "backstay" effect caused by the embedded foundation and the potential for shear reversals in the basement shear walls from soil pressures. To evaluate this effect, horizontal springs must be added to simulate the strength and stiffness of the surrounding soil, as shown in Model E. Note that this type of effect is similar but not the same as the backstay effect resulting from upper levels landing on larger, stiffer podium bases, which distribute local overturning loads out to other resisting elements using the diaphragms at the top and bottom of the podium. Nonlinearity can be added to the superstructure, substructure, and soil springs in these models as well. See below.

Modeling soil stiffness: With displacement-based analytical seismic rehabilitation methodologies, understanding and quantifying displacements has becoming increasingly necessary. In the past, when displacement was considered, it usually was in the form of construction and long-term differential settlements between columns or the modulus of subgrade reaction for gravity loading under a mat or grade beam on soft soil. During seismic loading, we need stiffness values relevant to the short-term nature of earthquake demands. ATC-40 and FEMA 356 provide detailed advice on these issues, but there

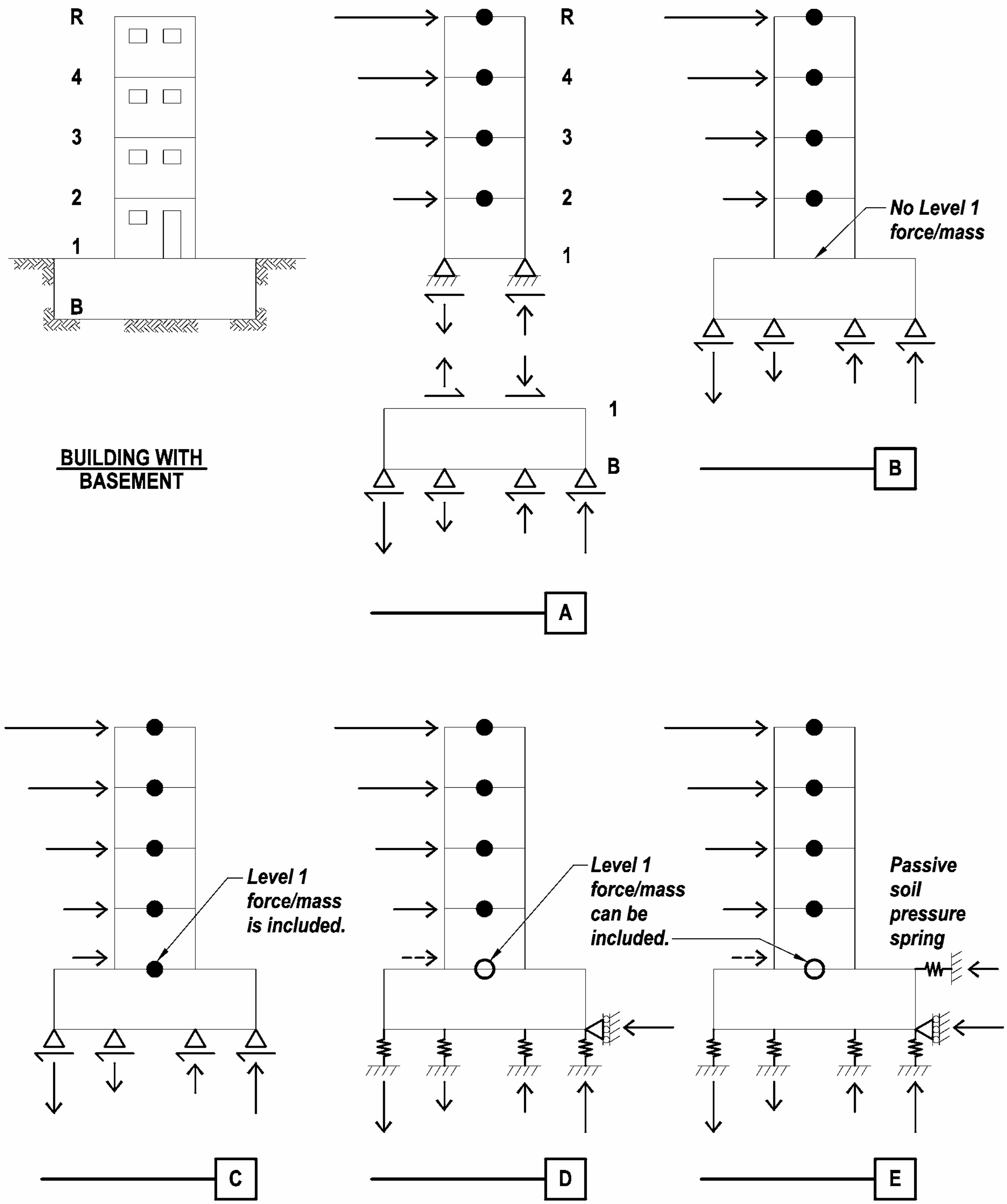


Figure 23.4-1: Modeling Approaches for Buildings with Basements

remains relatively limited data on short-term stiffnesses, particularly under high loads, and wide ranges of potential properties must be considered. These documents recommend taking half and twice the target stiffness estimates (i.e., a factor of four on the range). Key issues include whether to model soils springs with initial high stiffness relevant before yielding, a lower secant stiffness for some larger displacement, or to use nonlinear models that account for the expected nonlinear force-displacement curve of the soil. While this is the most accurate, it can take a significant analytical effort in any moderate to large building. Quantitative information on soil nonlinearity at high strains is limited. Some geotechnical engineers continue to use linear models, even in soils like clay. Significantly different results can occur if strains are sufficient to reach the point of nonlinearity. In fact, nonlinearity in the soil can lead to the accumulation of permanent deformation.

Damping, basement embedment and base slab averaging: Soil-structure interaction generally tends to reduce the input motion to the building as does an embedded basement and a slab or other foundation system that can distribute or average peak motions over the site. The input motion reduction is higher for buildings with a fundamental period below 0.7-1.0 seconds and not that significant for longer period buildings. These effects are now being considered in seismic evaluation and rehabilitation, and they are the subject of FEMA 440 (FEMA, 2005).

Second opinions: In some situations, the lower bound and higher bound of geotechnical strength and stiffness properties that are being provided can lead to significantly different results. Alternative opinions or geotechnical peer review can be advantageous in identifying alternative sources of information, narrowing the range of assumptions, or increasing the strength and displacement capability.

For detailed information on evaluation, analytical and design for foundation elements see ATC 40 (ATC, 1996), FEMA 274 (FEMA, 1997), FEMA 356 (2000), and ASCE 31-03 (ASCE, 2003).

23.5 Increasing Estimates of Capacity by In-Situ Testing

Existing shallow and deep foundations might have as-built capacities that exceed their design capacities. If these higher capacities can be confirmed, additional loads can be imposed on these foundations without any modifications to the existing foundations. Alternatively, the estimated capacity of new micropiles or drilled piers installed as part of a rehabilitation project can be verified or increased by performing in-situ load tests on them. The most common direct method for confirming these higher capacities is by performing in-situ load tests of the foundation elements.

The plate bearing test is probably the most common direct in-situ test for estimating the capacity of an existing shallow foundation. It involves the determination of the load-deformation characteristics of the soil directly below the shallow foundation. It is worth noting that indirect methods involving in-situ (instead of laboratory determination) of the settlement characteristics of foundation soils are sometimes employed. These indirect methods, which are not covered in this document, allow one to more accurately estimate settlement associated with additional loads to be imposed on the existing foundation. Examples of such methods include 1) the use of dilatometer tests to define the in-situ deformation characteristics of sandy and clayey soils and 2) the use of pore pressure dissipation techniques during cone penetration tests to estimate in-situ settlement characteristics of soft clays.

The most common direct method for estimating the load-deformation characteristics of deep foundation elements is static load tests in tension or compression.

23.5.1 Plate Bearing Tests

Preparation

Plate bearing tests are generally performed on existing shallow foundations to determine their capacities. Access to the bottom of the existing foundation, which is used as a reaction element, is required for the test to be performed. Access to the bottom of the foundation is facilitated via an access pit, which is at least 3 feet by 3 feet in plan view and extends at least 18 inches below the bottom of the foundation. The access pit is located in such a way that the exterior edge of the foundation is exposed in the pit. This access pit can be dug with a backhoe. Depending on the depth of the pit and the materials that are exposed in the pit, shoring may or may not be required.

From the bottom of the pit, a rectangular mini-tunnel that extends from the exposed to the opposite edge of the foundation is dug underneath the foundation using handmining techniques. The tunnel has to be at least 18 inches wide in cross section to facilitate the placement of bearing plates and hydraulic jacks for the test. Sometimes, it is necessary to chip off excess concrete from the bottom of the foundation to create a flat surface for the placement of the upper bearing plate. Also, the bottom of the mini-tunnel must be prepared to create a flat surface for the lower bearing plate. The access pit and the mini-tunnel are depicted in Figure 23.5.1-1. It must be noted that sometimes it is more economical to dig the access pit from the crawl space side of the existing foundation in Figure 23.5.1-1.

Equipment, Set-Up and Testing

The minimum required equipment includes the following:

> A hydraulic ram that is capable of imposing load exceeding the design capacity of the existing foundation. The pressure gage of the ram must be calibrated to allow the load imposed by the ram to be estimated. A load cell can be used in addition to the pressure gage for more accurate determination of imposed loads.
> One-inch thick steel 12-inch square bearing plate.
> Minimum of four dial gages for measuring soil deformation. Linear variable displacement transducers or transformers (LVDT)s could be used in lieu of dial gages for measurement of deformations.

The hydraulic ram and bearing plates are set up as depicted in Figure 23.5.1-1. The test is performed by imposing load incrementally on the soil below the lower bearing plate and measuring the corresponding deformations. The procedure has been standardized as ASTM D1194.

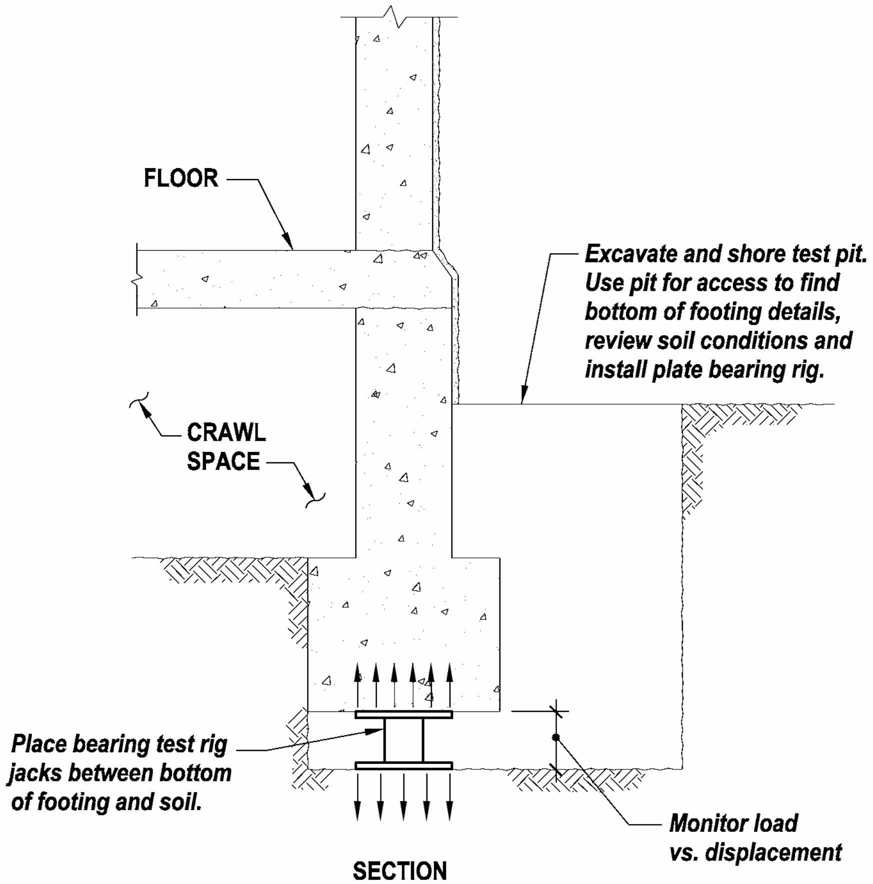


Figure 23.5.1-1: Plate Bearing Tests for In-Situ Bearing Capacity Determination

After the test is completed, the mini-tunnel and access pit are usually partially backfilled with lean concrete or controlled density fill to the top of the foundation, and the balance of the pit is backfilled with either the same material or compacted native soil.

Test Results

The load-deformation data that are recorded are applicable to the 12-inch square lower bearing plate. The data must be corrected for scale effects to apply them to the prototype foundation. Reasonable results are usually obtained when plate bearing tests are performed on very stiff clays or sandy or gravelly soil. Poor results are usually obtained when tests are performed on soft to stiff clays. Refer to Bowles (1996) for discussion on extrapolating test results.

Cost/Disruption/Challenges

Digging the access pit and mini-tunnel can be somewhat disruptive and costly, though much less expensive than the cost of foundation rehabilitation. If the pit and mini-tunnel are dug from the crawl space side, usually only handmining techniques can be employed, and the hauling of excavated spoils becomes time-consuming.

If groundwater is encountered, the conditions in the pit are mucky. Even without groundwater, the conditions in the pit are damp and cramped for the individual who has to set up the plates, hydraulic ram and dial gages in the mini-tunnel as well as for the person who has to crouch in the pit to read the dial gages while a test partner applies the load and records readings from a position near the edge of the access pit. It is often necessary to provide a plastic covering at the bottom of the pit.

Plate bearing tests on clayey soils can be time consuming because it takes a longer time for the deflection under each load increment to level off.

23.5.2 Static Tests on New Deep Foundations

Preparation

Static load tests are usually performed on new micropiles to determine their axial capacities because of the potential effects of installation procedures on the capacities. The tests can be performed in compression or tension. For a compressive type load test, reaction micropiles must be installed at a distance of at least three times the diameter of the test or reaction micropile, whichever is greater, to minimize the potential for group effects between the test pile and the reaction elements. For tension tests, timber mats could be used as reaction elements in lieu of reaction micropiles. The test and reaction micropiles should be allowed to cure for at least seven days after installation before the load test is performed.

Static load tests can also be performed on new drilled piers installed as part of a rehabilitation project as a means of increasing their estimated axial capacities. The preparatory work described above for micropiles also applies to drilled piers. Because of the size of drilled piers in comparison to micropiles, the spacing between the test pier and the reaction piers is much larger. This implies that a much larger reaction beam is required for tests on drilled piers.

Equipment, Set-Up and Testing

The minimum required equipment includes the following:

A reaction beam spanning between the reaction elements and capable of sustaining the maximum test load without excessive deflection.

A hydraulic ram that is capable of imposing load exceeding the design capacity of the existing foundation. The pressure gage of the ram must be calibrated to allow the load imposed by the ram to be estimated. Usually, a load cell is used in addition to the pressure gage for more accurate determination of imposed loads.

An independent reference beam with supports that are located away from the test or reaction micropiles.

Minimum two dial gages for measuring the deflection of the pile head. LVDTs could be used in lieu of dial gages for measurement of deformations. Whether LVDTs or dial gages are used, a secondary system of deflection measurement is required as a back-up.

The reaction beam, the reference beam, hydraulic ram and dial gages are set-up as depicted in Figure 23.5.2-1 for compression tests and in Figure 23.5.2-2 for tension tests. The test is performed by imposing load incrementally on the soil below the lower bearing plate and measuring the corresponding deformations. The maximum test load is usually about 1-1/2 to 2 times the design load. The test can be performed in accordance with ASTM D1143 (for compression tests) and ASTM D3689 (for tension tests).

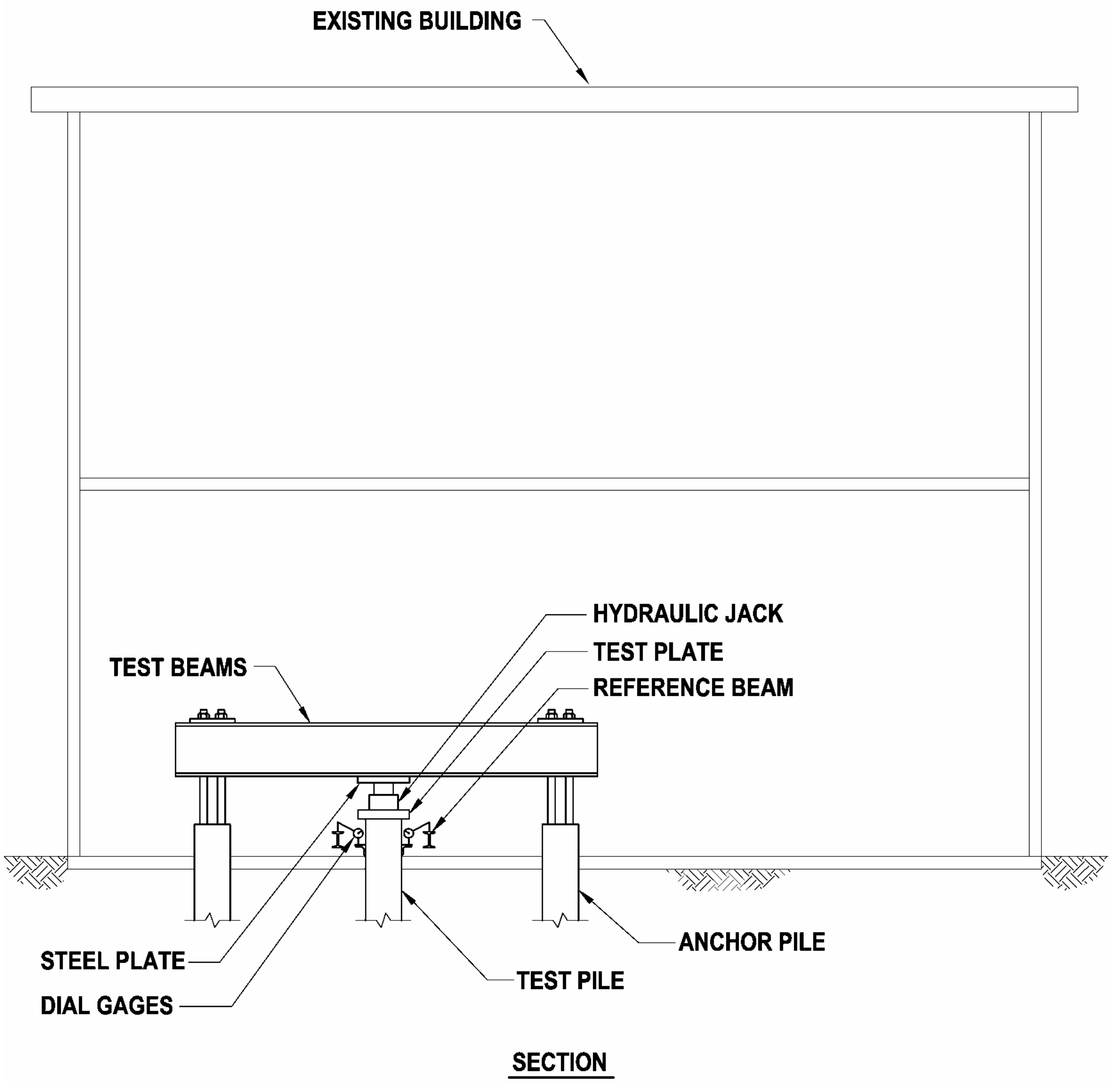


Figure 23.5.2-1: Static Pile Load Test in Compression

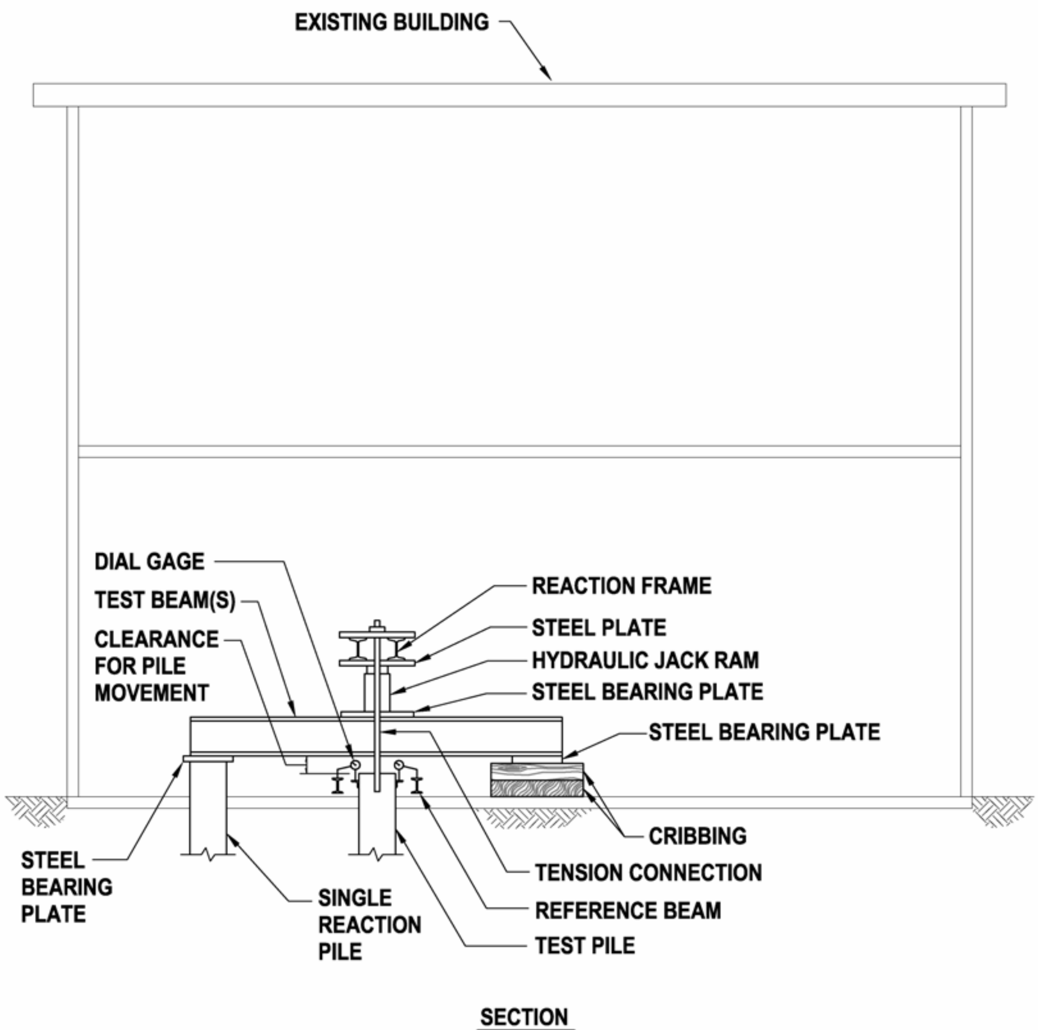


Figure 23.5.2-2: Static Pile Load Test in Tension

Test Results and Interpretation

The load-deformation data that are recorded must be interpreted in two phases. The first phase involves the determination of the axial capacity in tension or compression of the test foundation element. The next phase involves interpreting the axial capacity relative to the known foundation conditions, such as the applicability of the axial capacity to a group of deep foundation elements or the applicability of the observed settlement from the load tests, given its short duration, to a production deep foundation element bearing in clayey soil that could consolidate.

Cost/Disruption/Challenges

Setting up and performing a load test can be quite costly. Setting up can be time consuming because there is more manual labor involved in transporting test equipment from one test location to another in cramped situations. In the case of drilled piers, it might be impossible because of access related issues to set up an adequate reaction beam for a static load test. It may be possible, however, to find locations on site that are not within the building, such as parking lots or landscaped areas, with similar underlying soils and perform the test on elements that will not be used under the building.

23.5.3 Static Load Tests on Existing Deep Foundations

While theoretically possible, this approach is so disruptive and costly that it is generally not implemented in practice except where the existing foundation consists of timber piles or where the existing pier is located on the exterior of the building. This kind of testing would require temporary shoring of the column supported by the pier or pile to be tested and removal by cutting of the structural connection between the deep foundation element and the column. The top of the pier or pile must be accessible to allow for a load test set up. For a compressive type load test, reaction micropiles or piers must also be installed.

23.6 New Foundations

23.6.1 Types of New Foundations Commonly Used in Seismic Rehabilitation

Foundation elements can be broadly classified into two basic categories: shallow and deep foundations. Shallow foundations include continuous strip footings, isolated spread footings, grade beams, and mats. Deep foundations include drilled piers and micropiles. Driven piles are rarely used in existing construction due to access and vibration limitations. Figure 23.6.1-1 shows examples of these foundation types. Several excavation approaches are shown in the figures. In cohesive soils, the soil may be able to be cut without it sloughing into the hole. Metal stayforms (expanded metal lath forms) are sometimes used when there is some risk of the soil sloughing after the intial excavation. The stayforms are left in place when the concrete is poured. When the excavation gets to a certain depth, however, shoring can be required due to safety regulations or an open cut excavation can be used. In cohensionless soils, like sand, an open cut excavation will be necessary. A form can be placed, the concrete poured, the form removed, and then soil backfilled into the remaining open cut. Alternatively, the form can be left out and the concrete for the footing “overpoured” in the full open cut. The eccentricity of the overpour should be evaluated.

23.6.2 Add Shallow Foundation Next to Existing Shallow Foundation

Description of the Rehabilitation Technique

When a concrete overlay is placed against an existing wall, a new footing is typically needed. A common situation is the existing footing is a continuous strip footing and the new footing is either a strip footing or a grade beam. Figure 23.6.2-1 shows an example of a new concrete wall and footing against and existing unreinforced masonry wall and concrete strip footing.

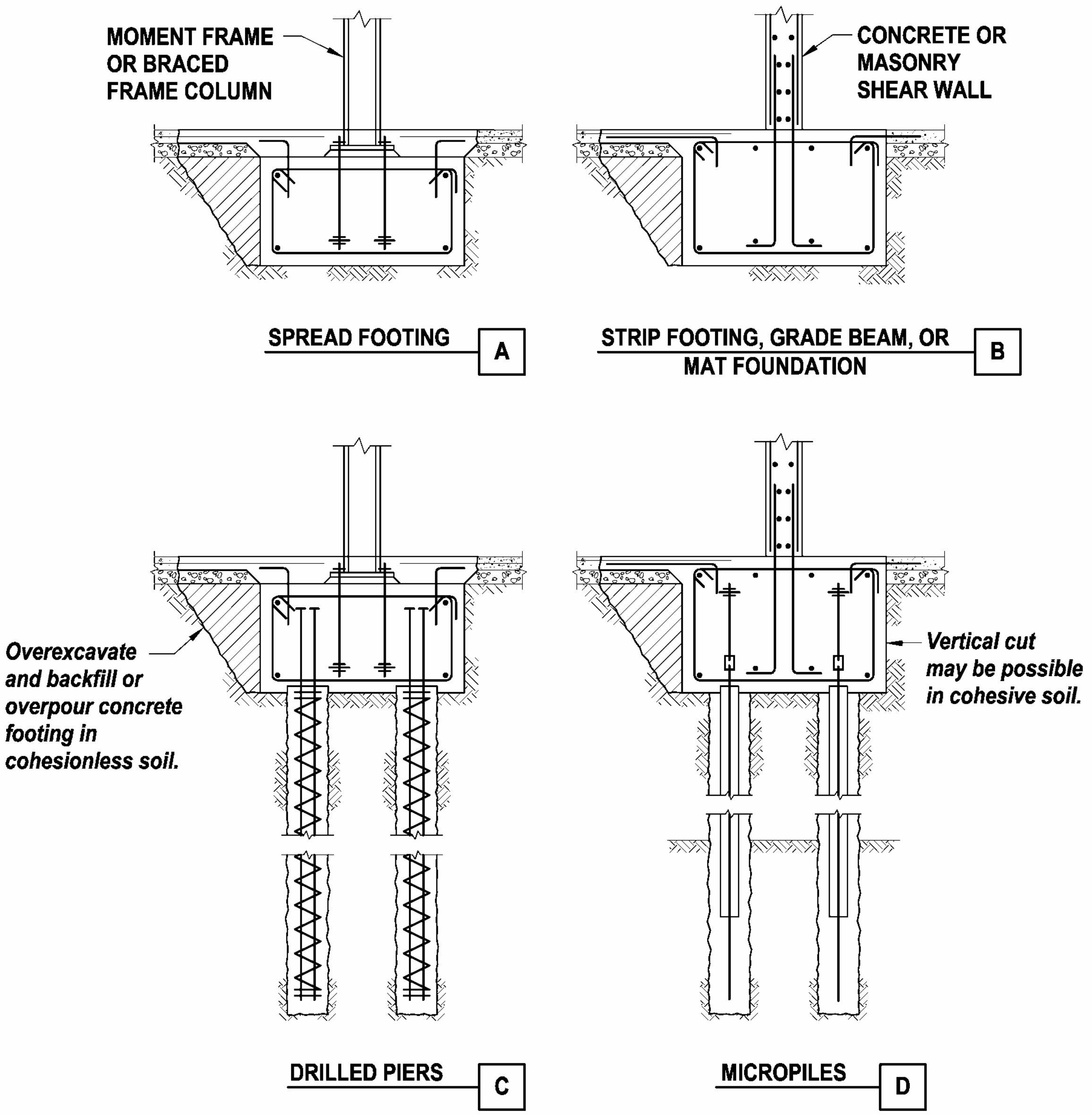


Figure 23.6.1-1: Types of New Foundations Commonly Used in Seismic Rehabilitation

Design Considerations

Effective footing width: Several approaches to footing design are used. One is to assume only the new footing resists the loads under the new overlay. Another is to share loads between the new and existing footing simply on the basis of area. The most sophisticated approach is to recognize the potentially different stiffness between the soil under the existing footing which has been consolidated already and the soil under the new footing which is likely to be more flexible since loading is likely to be lighter and only newly applied. Sometimes jacking is employed to transfer loads to new foundations.

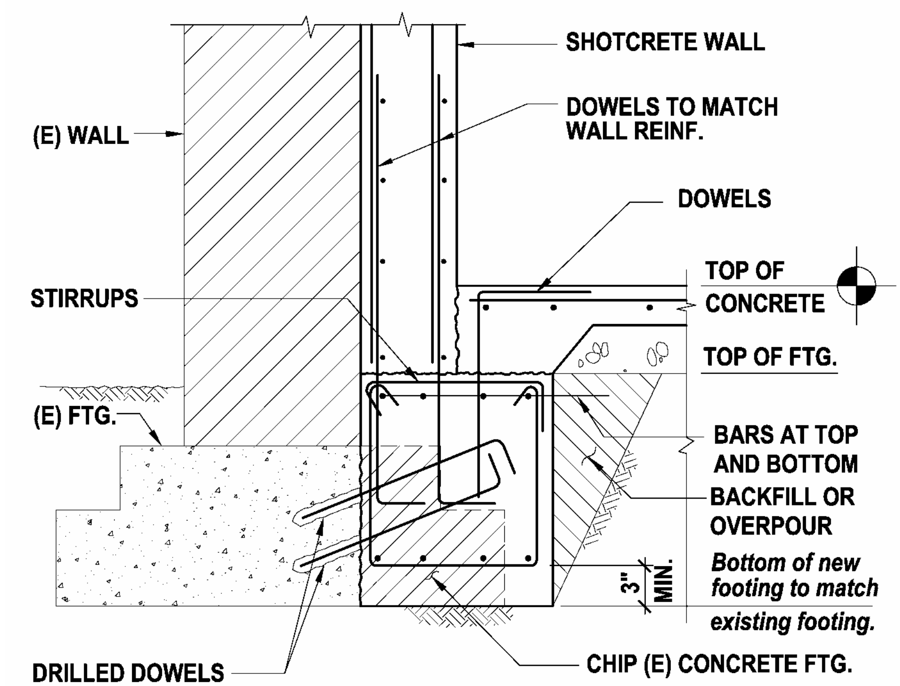


Figure 23.6.2-1: New Concrete Strip Footing Next to Existing Strip Footing

Shear transfer: It is standard practice to connect the new and existing footings with drilled dowels, though it useful to consider whether the dowels are actually necessary elements. Dowels in the footing and wall above should be designed to be sufficient to transfer the force intended to be resisted under the existing footing.

Unreinforced existing footings: The existing footing may be unreinforced masonry or poorly reinforced concrete. If the footing is wide enough so that so beam action will result under bearing pressure, the bottom drilled dowels can be extended deep into the existing footing near the base of the footing to serve as positive reinforcing.

Detailing and Construction Considerations

New footing is deeper than existing footing: A key goal when adding a new footing is not to surcharge or undermine an existing footing. The best approach, then, is to match the new and old footing depths. This is, of course, not always possible. Figure 23.6.2-2 shows the situation when the new footing needs to be deeper than an existing footing. If excavation proceeds without underpinning, particularly in soils with minimal cohesion, soil can slough away from under the existing footing into the new excavation leading to damaging footing movement. Underpinning is used to address this situation. Underpinning means digging a series of short length pits separated by a sufficient distance, digging under the existing footing adjacent to the pit, adding concrete to the base of the final excavation depth, and then going back and completing the underpinning in between the initial pits. An alternative underpinning approach is to place long underpinning piers intermittently beneath the new footing to derive support at depth so that the typical new footing need not be deeper than the existing footing.

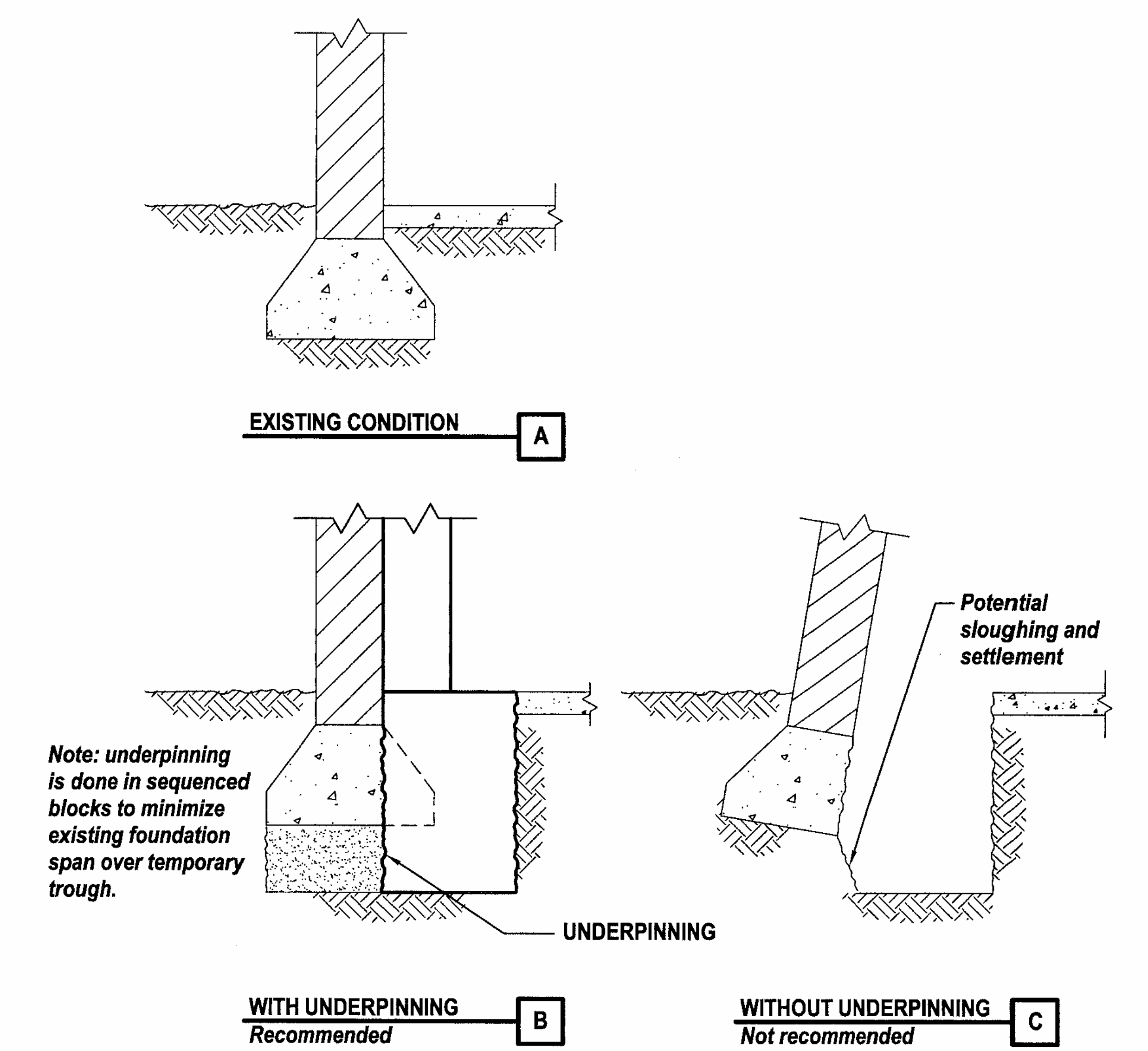


Figure 23.6.2-2: New Shallow Footing is Deeper than Existing Shallow Footing

New footing does not need to be a deep as existing footing: Figure 23.6.2-3 shows the situation when the new footing does not need to be as deep. If the excavation is kept shallow, the new footing when loaded can impart additional and eccentric loads into the existing footing that may not be desirable. As a result, it is common to extend the bottom of the new footing down to match the depth of the existing footing. The extension is often lightly reinforced.

Existing footing is in the way: As Figures 23.6.2-1 to 23.6.2-3 show, the existing footing will often extend inboard from the existing wall underneath the new wall. To place the new footing, the existing footing often must be chipped away to develop a properly reinforced footing. This can be done with jackhammering or sawcutting. The capacity of the existing footing during the temporary condition where it is smaller and eccentrically loaded should be verified as adequate.

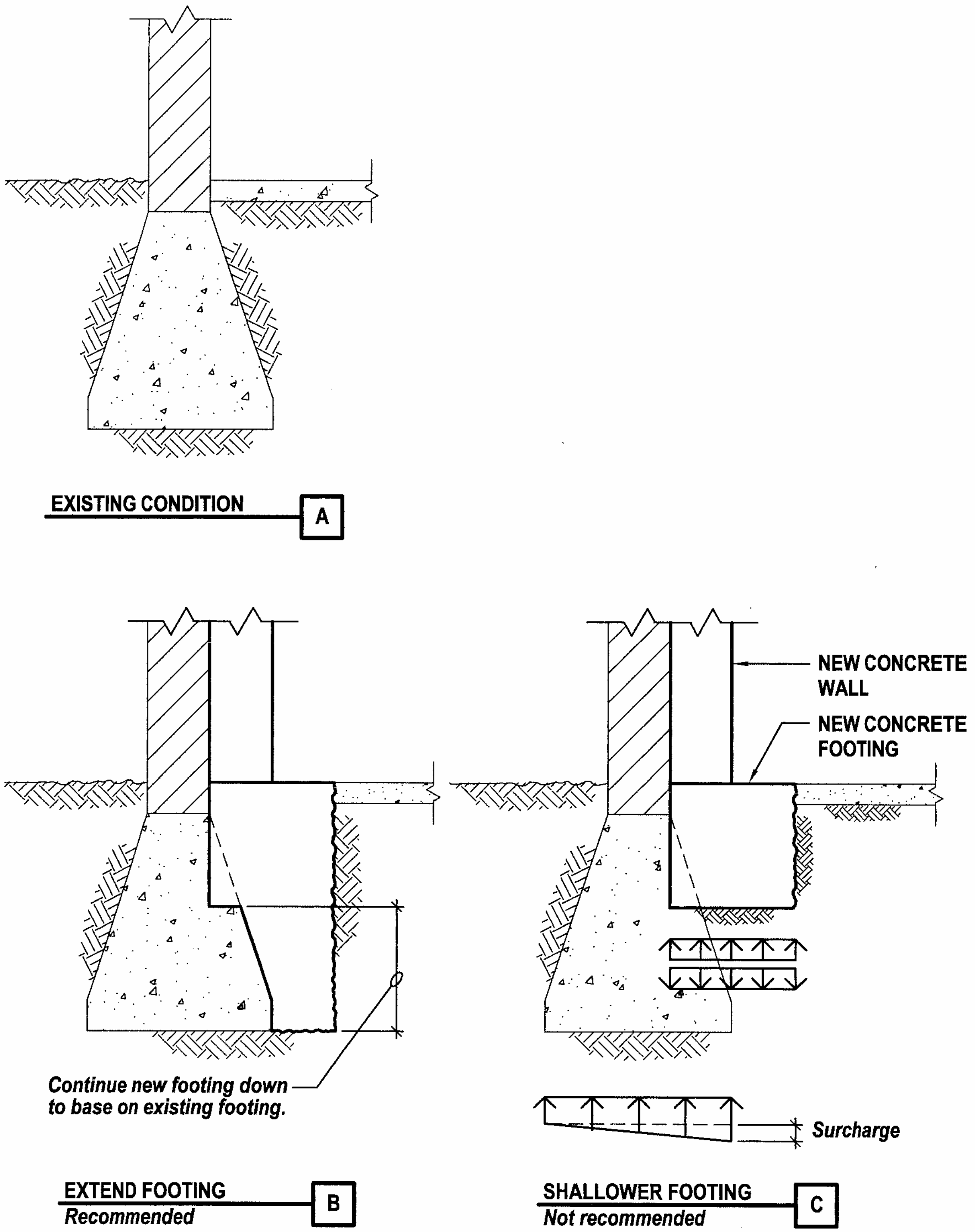


Figure 23.6.2-3: New Shallow Footing Does Not Need to be as Deep as Existing Shallow Footing

Cost/Disruption

Adding a new footing is quite disruptive and costly. The existing slab-on-grade must be sawcut and removed, then the trench excavated, drilled dowels installed, rebar laid, debris in the footing removed, and concrete placed. This is all time-consuming, messy, and noisy.

23.6.3 Add Shallow Foundation Next to Existing Deep Foundation

Adding a new shallow foundation next to an existing deep foundation is relatively rare for two reasons. First, the existing foundation was deep because soil or structural loading conditions would not permit a shallow foundation. Without ground remediation, a new foundation would have the same issue. Second, as noted in Section 23.6.2, if the new foundation is higher than the existing foundation, the new foundation will impart gravity and earthquake loads to the existing foundation which is usually undesirable. With careful study of relative rigidity considerations, there can be situations where a new shallow foundation can be added adjacent to an existing deep foundation, such as a mat next to drilled piers. See Section 23.8.2.

There can be cases, though, when adding new deep foundations are very disruptive or not economical practicable due to existing access limitations. Sometimes a shallower foundation is added, such as a new mat next to an existing drilled pier foundation. The relative stiffness of each foundation then becomes the key consideration.

23.6.4 Add Deep Foundation Next to Existing Shallow Foundation

Adding a new deep foundation next to an existing foundation is occasionally done, such as drilled piers under a new wall next to an existing strip footing. Figure 23.6.4-1 shows an example of this technique. Drilling limitations can be significant, and they include access requirements for the drill rig, height restrictions for the drill rig, the offset needed to get the edge of the drill up against the existing wall, vibration during drilling, and utilities in the way of the drilling. Sometimes when the exterior face of the building is accessible, slanted drilling is done under the existing footing. Usually, the drilled piers are spaced at a sufficient distance that the existing footing and walls can span around or over the open hole. After the pier and new wall are installed and dowelled into the existing wall and footing, a composite system has been created. While many engineers simply take gravity in the existing spread footing, and overturning in the piers, live loads and earthquake loads are of course actually distributed throughout the system by relative rigidity.

23.6.5 Add Deep Foundation Next to Existing Deep Foundation

There will also be situations where new deep foundations are added next to existing deep foundations. New deep foundations include drilled piers and micropiles. See Section 23.8 for examples.

23.7 Structural Rehabilitation for Existing Shallow Foundations

23.7.1 Goals

Typical structural improvements to existing shallow foundations can be simplified into two basic categories: enhancing compression capacity and enhancing tension capacity.

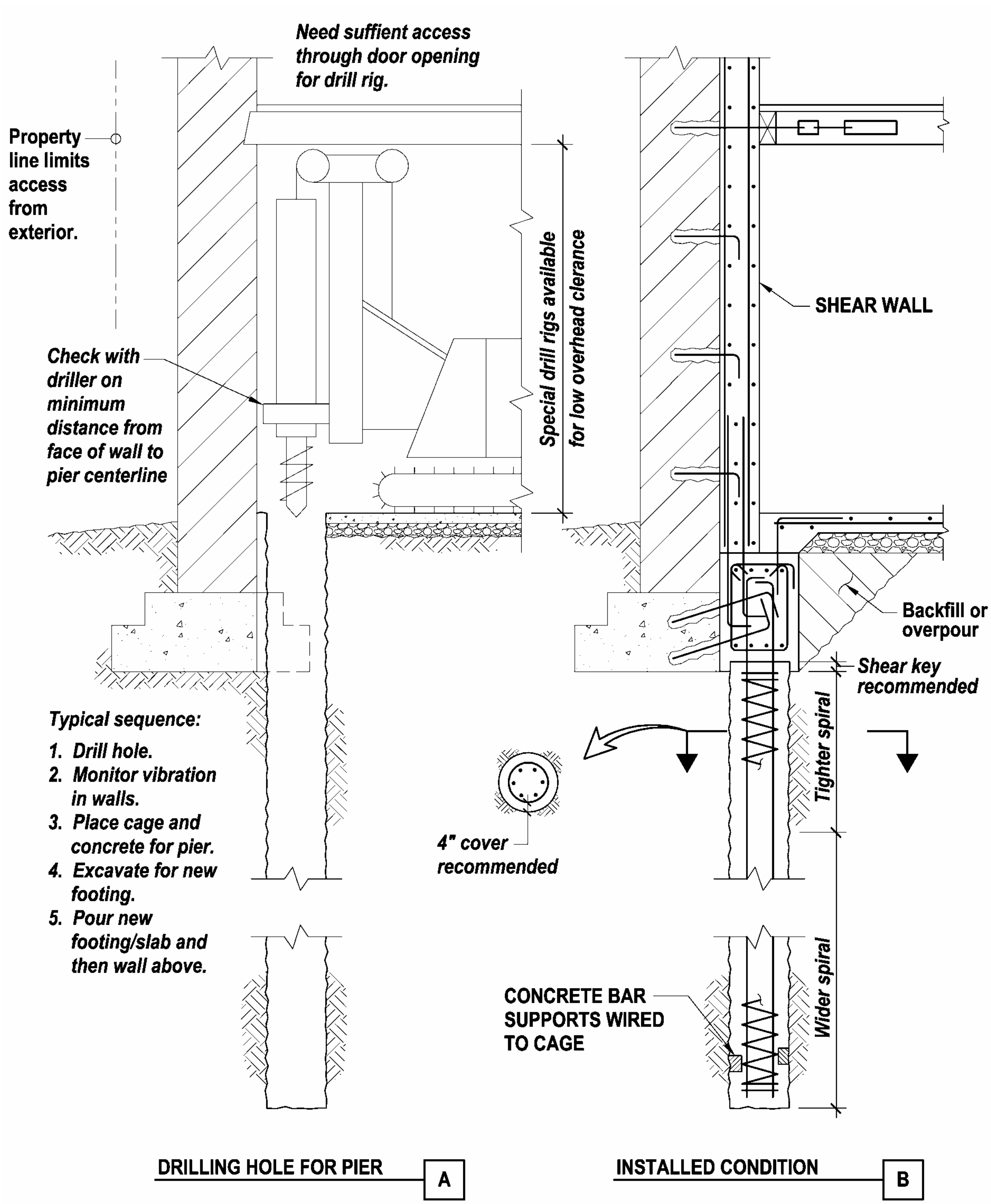


Figure 23.6.4-1: New Drilled Pier Next to Existing Strip Foundation

General techniques for improving inadequate compression capacity: Compression strength capacity of existing spread and strip footings can be addressed by widening the footing base; replacing the footing with an enlarged foundation; adding micropiles, screw anchors or drilled piers adjacent to the existing footing; adding micropiles through the existing footing; or adding grade beams to connect isolated spread footings together.

General techniques for improving inadequate tension capacity: Improving inadequate tension capacity of existing spread and strip footings uses similar techniques to those for improving compression capacity, including widening the footing base to increase the dead load; replacing the footing with an enlarged foundation; adding micropiles, screw anchors, or tie-downs adjacent to or through the existing footing; or adding grade beams to adjacent footings and columns to pick of dead load to resist uplift.

The following sections provide some examples of rehabilitation techniques for existing shallow footings.

23.7.2 Add Micropiles Adjacent to an Existing Strip Footing

Deficiencies Addressed by Rehabilitation Technique

Inadequate compression capacity at the toe of strip footing beneath a wall
Inadequate tension capacity at the heel of a strip footing beneath a wall

Description of the Rehabilitation Technique

To improve the compression and/or tension capacity of the existing footing, the footing is widened and micropiles, also known as pin piles, are added. Figure 23.7.2-1 provides an example.

Design Considerations

Research basis: FHWA (2000) provides guidelines for the design and construction of micropiles.

Compression strength and stiffness: When micropiles are added together with the strip footing, resistance is shared between the two different elements, depending on their relative rigidity. Micropile strength and stiffness are given in the geotechnical report. Governing strength depends on both the soil capacity and the structural capacity of the pile, including the pipe, grout, and reinforcing bar. Compression stiffness considers the pile elements and surrounding soil movement.

Tension strength and stiffness: Uplift resistance is taken by the micropiles. Structural tension strength is lower than compression strength in the micropiles and is usually based on just the reinforcing bar, unless special details are used to engage the top of the casing in tension. Tension stiffness is also usually lower; tension flexibility comes from the reinforcing bar elongation and surrounding soil movement.

Corrosion effects: Permanent casing associated with micropiles is typically uncoated. Depending on the corrosivity of the soil, corrosion of the permanent casing can occur over time. Techniques are available for estimating the extent of thickness of the steel pipe lost to corrosion;

with the estimates, a reduced thickness and reduced lateral and buckling capacities of the pile can be calculated.

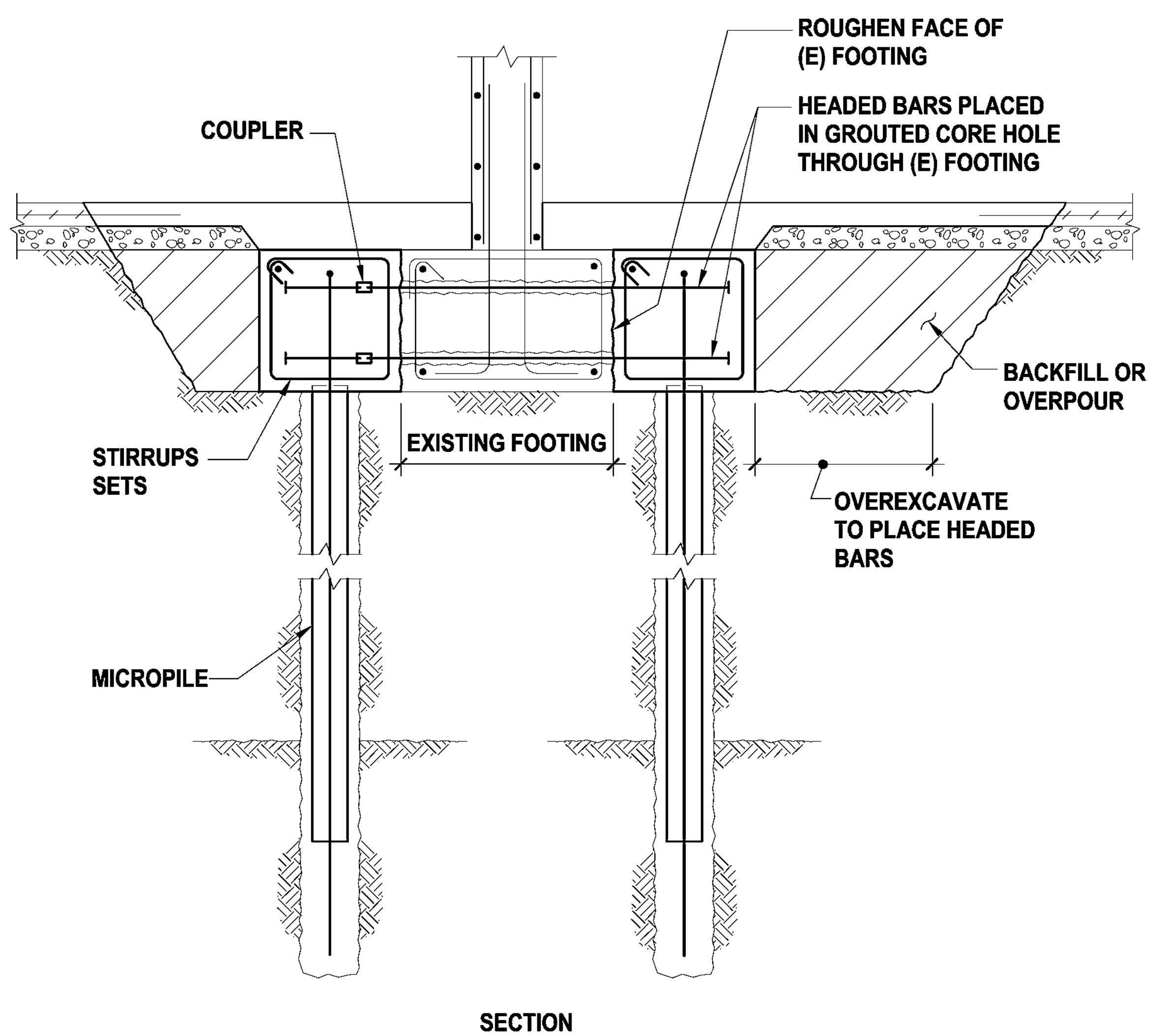


Figure 23.7.2.-1: Micropile Enhancement to Existing Strip Footing

Testing: Performance and proof load testing are performed at the start of and periodically during construction to verify that specified design capacities will be achieved. During performance testing, the test piles are usually loaded to 2.0 to 2.5 times the design load. Proof testing, on the other hand, involves testing the pile to 1.33 to 1.67 times the design load. Proof testing is usually limited to a percentage of the production piles. Creep tests are typically performed as part of the performance and proof tests, especially if the micropiles are to be bonded in clayey soils that are susceptible to creep. PTI (1996) provides guidelines on performing and evaluating performance, proof, and creep tests on foundation elements.

End bearing vs. friction: Because of its small size, micropiles generally derive most of their capacities from friction. The geotechnical compression capacity of the micropile is therefore generally equal to the axial tensile capacity.

Filling the annulus with grout: Where the cutting tool of micropile drilling equipment creates a hole slightly larger than the permanent casing, an annulus is created around the casing. Typically, this annulus is not grouted.

Detailing and Construction Considerations

Detailing and construction considerations for adding micropiles to an existing footing include the following.

Connecting to the new footing: Figure 23.7.2-1 shows bars drilled all the way through the existing footing. If this not done, the existing capacity of the footing for bending and the center where the moment is largest must be checked; it is unlikely to be acceptable. In the figure, the through dowels are installed from the right and coupled on the left. Headed bars are shown for ease in installation. Hooked bars could be used, but they would trigger a position coupler (one that eliminates the need to rotate the bar), at least for the bottom row of bars. To install the longer dowels, over excavation of the adjacent soil is needed. This needs to be understood during detailing, as their may be existing elements on top of that portion of the slab.

Access and height limitations: Adequate clearance must be available for the equipment used to install micropiles inside existing buildings.

Anchorage to the footing: Figure 23.7.2-2 shows a micropile and some of its details. In this figure, tension is taken by threaded rod and the plate at the top of the rod. Sufficient embedment of the plate above the base of the footing is needed to develop the strength of the rod. Similarly, the bottom plate is designed to take the compression and deliver it to the pipe. In some cases, the bottom plate may not be necessary as the grout diameter or top plate can be sufficient. If the top plate is used, it must be sufficiently deep below the top of the footing so it is not the weak link.

Bar types and size: Bar types include ASTM A722 high strength threadbar, with F_y = 150 ksi, with common sizes of 1”, 1-1/4”, 1-3/8” and 1-3/4” diameter. CALTRANS has typical details using #18 bars in ASTM A615 steel, where ends needing nuts or couplers are threaded.

Pipe types and sizes: API casing with F_y = 80 ksi is commonly used, with 7” diameter and 9-5/8” diameter pipes being common.

Depth of pipe and grouting: The pipe typically goes down into the bearing layer the requisite depth. The reinforcing bar usually continues deeper. Grouting fills up the hole at the base, the annulus around the pipe and the inside of the pipe. Post-grouting or secondary grouting can be used at the base to increase the bar capacity.

Strain limits at the top: To increase the length over which the bar is strained in tension, the top of bar below the anchorage plates are sometimes debonded with a greased PVC pipe.

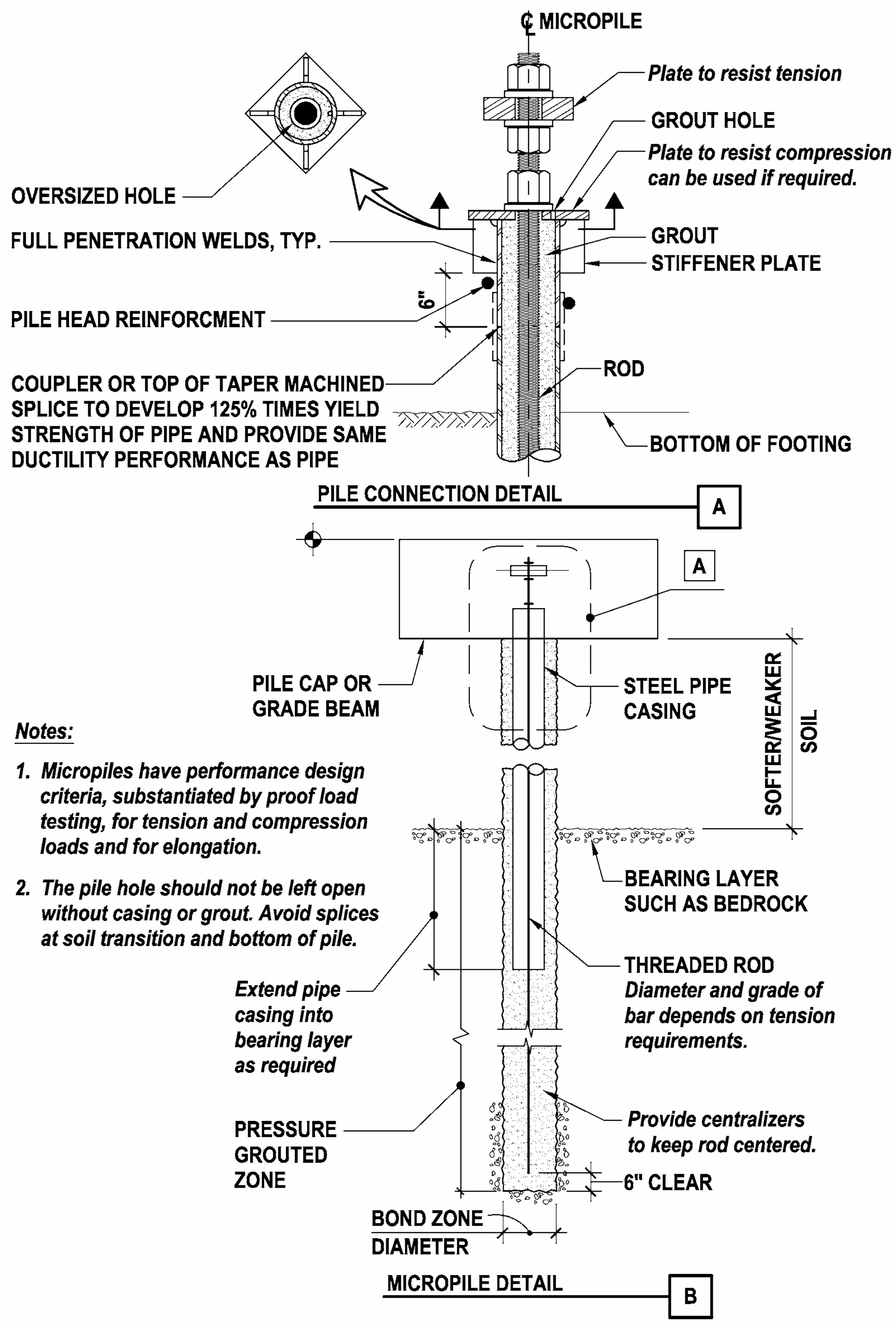


Figure 23.7.2.-2: Micropile Details

Cost/Disruption

Micropiles are typically less expensive than drilled piers, unless very large capacities are required. They require less headroom and smaller footings to receive the bars and pipe. Excavation noise and dust, and drilling and grouting noise must be considered as part of the rehabilitation strategy.

Proprietary Issues

Micropile specifications are often written like tiebacks, so that the contactor must design and build the micropile to meet performance requirements. Figure 23.7.2-2 shows a generic type of pile. There are other proprietary piles that use a pointed pipe casing as the drill.

23.7.3 Enlarge or Replace an Existing Spread Footing

Deficiency Addressed by Rehabilitation Technique

Inadequate compression capacity at a spread footing
Inadequate tension capacity at a spread footing

Description of the Rehabilitation Technique

An existing spread footing may be under a braced frame, moment frame or a concrete column below a discontinuous shear wall and be subjected to compression or tension forces that exceed the footing capacity. The existing footing can be enlarged or replaced to increase compression capacity or the dead load for resisting tension.

Design Considerations

Research basis: No research specific to enlarging or replacing existing footings has been identified.

Bending moment and shear checks in enlarged footing: Gaining large increases in compression capacity by enlarging an existing footing is often difficult given the limits of the existing footing. In Figure 23.7.3-1, reinforcing is drilled in from the sides, but does not go through to the other side. The shear capacity of the footing is not increased. The bending capacity has to be checked at critical locations "A" and "B". Location A will typically govern. If sufficient capacity cannot be achieved, the footing can be replaced as shown in Figure 23.7.3-2.

Tension capacity: Tension capacity can be quite limited if the existing spread footing only has bottom reinforcing bars which would be typical. Drilled dowels can be added to the top of the footing and top steel added in the slab-on-grade level. See Section 23.9 for a similar example in a pile cap.

Detailing and Construction Considerations

Detailing and construction considerations for enlarging or replacing an existing spread footing include the following.

Existing reinforcing: Existing reinforcing should be preserved in the footing. This will typically require placing new drilled dowels at a higher elevation, with a resulting lower moment capacity.

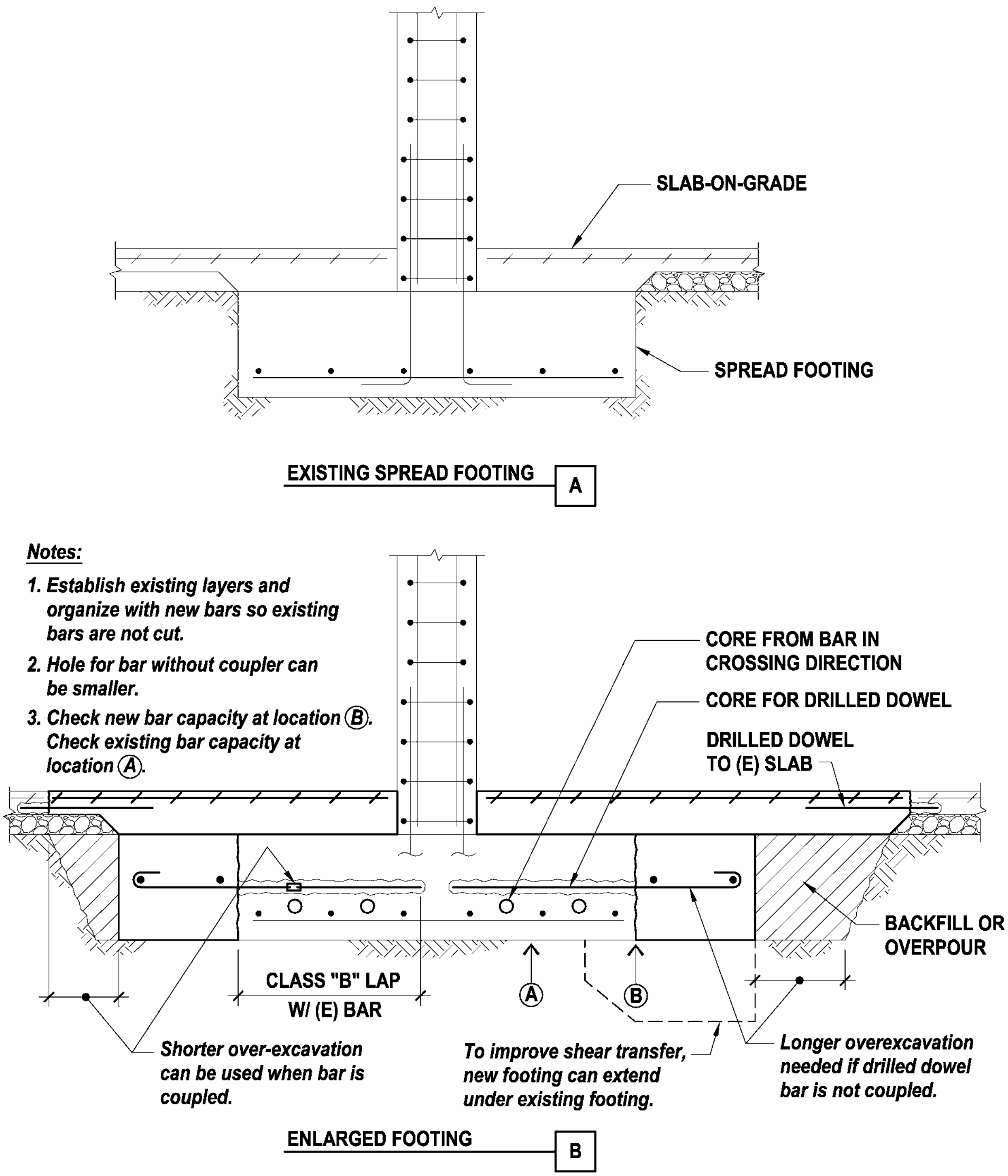


Figure 23.7.3-1: Enlarge Existing Spread Footing

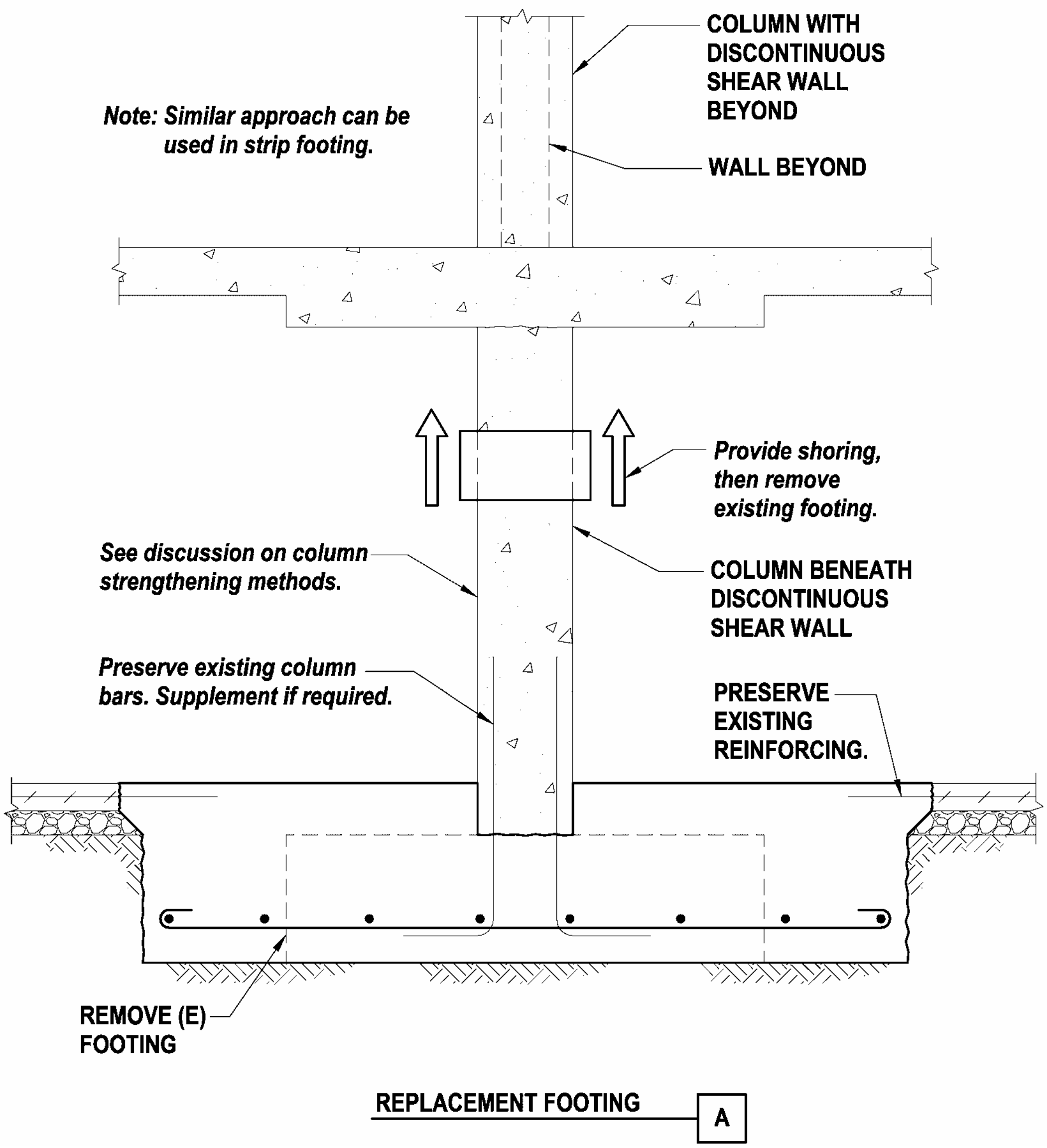


Figure 23.7.3-2: Replace Existing Spread Footing

Installing drilled dowels: Figure 23.7.3-1 shows two approaches for installing bars. On the left a coupler is used, permitting a smaller overexcavation past the footing, but triggering a hole large enough to accommodate the coupler. This is likely to force pressure grouting with nonshrink grout as the annulus will be too large for most adhesives like epoxy. On the right, a larger over-excavation is used and a single piece bar is installed.

Lapping with the existing slab-on-grade: The existing slab is likely to have wire mesh. To minimize vertical offsets the new and existing slabs should be dowelled together. Either the mesh in the existing slab-on-grade can be preserved when the slab is demolished or drilled dowels can be drilled into the edge of the existing slab.

Shear transfer between the new and existing footings: Transferring shear between the existing footings is necessary. This can be accomplished by roughening the existing footing face. Some engineers bevel the existing face as well with the top wider than the bottom, so outward pressure is exerted under compression loading. Some engineers dig the new footing slightly deeper than the existing footing and undercut the soil at the edge of the existing footing, so that the new footing acts as a corbel to resist downward pressure from the existing footing.

Shoring: If the existing footing is replaced, shoring will be needed. It is critical that the base of footing be properly compacted and the new concrete be tightly placed beneath the existing column to minimize or eliminate any settlement when the shores are removed.

Cost/Disruption

Enlarging or replacing an existing footing is a localized but disruptive process, involving excavation, dust, mud, drilling/jackhammering noise and concrete placement. Protection of existing finishes in the vicinity and in the working path is necessary.

23.8 Structural Rehabilitation for Existing Deep Foundations

23.8.1 Goals

Typical structural improvements to existing deep foundations can be simplified into several basic categories: enhancing the overall compression capacity, tension capacity, or lateral capacity of the foundation; and improving the ductility and detailing of specific elements or connections within the system.

General techniques for improving overall inadequate compression, tension and lateral capacity: Inadequate strength and deformation capacity of existing pile and pier foundations can be addressed by adding new shallow adjacent shallow foundations, and new piers or micropile foundations, either in vertical or battered orientations.

General techniques for improving inadequate improving ductility and detailing: Inadequate confinement can be improved with enlarged or replacement pier and pile caps; lack of top steel in pier and pile caps can be addressed with new concrete overlays on top of the cap.

The following sections provide some examples of rehabilitation techniques for existing deep footings.

23.8.2 Add a Mat Foundation, Extended Pile Cap or Grade Beam

Deficiency Addressed by the Rehabilitation Technique

The deficiency addressed by this technique is inadequate compression capacity of an existing deep foundation element. The technique involves taking advantage of the contributions of shallow foundation elements that are part of the overall foundation system.

Description of the Rehabilitation Technique

When existing piers or piles have inadequate capacity, the usual approach to increasing their capacity is to install new micropiles or piers connected by grade beam to the existing adjacent piles or piers. Where competent bearing soil is within five feet of building ground floor, an alternative approach to installing new piles or piers is to widen and deepen the cap or grade beams atop the pier or pile or connecting adjacent piles or piers. Figure 23.8.2-1 shows an example of existing piers whose capacities are augmented by installing a mat between the piers.

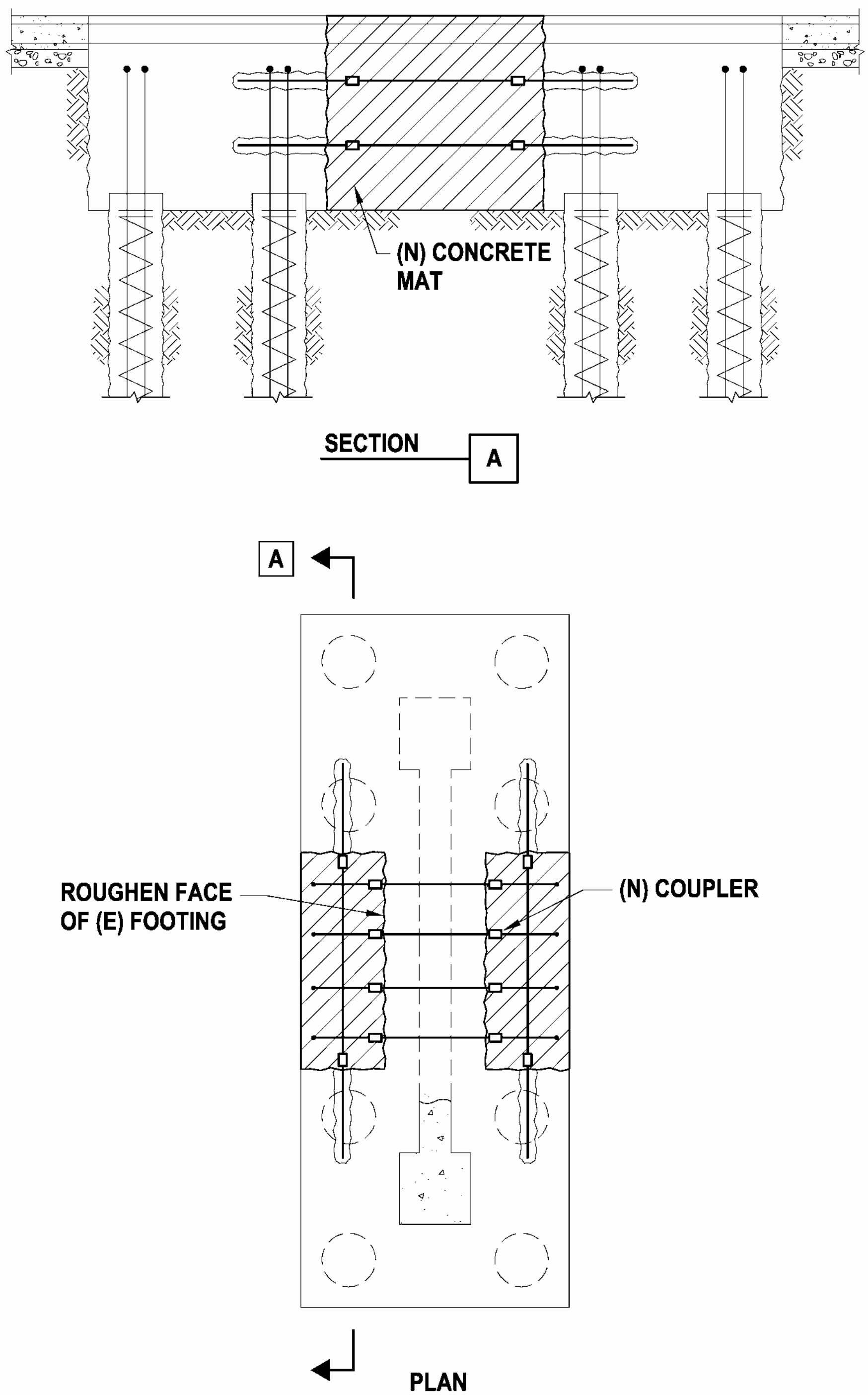


Figure 23.8.2-1: New Mat Foundation Between Existing Drilled Piers

Design Considerations

Analysis: Several approaches to design are used. One is to assume that only the new mat foundation resists the new loads imposed by the retrofit scheme. This assumption is inaccurate. The most sophisticated approach is to model the soil under the new mat as a spring with stiffness that is different from that of the spring representing the existing piers. The analysis would show that new loads are supported by both the new mat foundation and the existing piers based on the relative stiffnesses of the two sets of foundation elements.

Shear transfer: It is standard practice to connect the new mat or cap or grade beam and existing piers with drilled dowels. Dowels in the piers and grade beams above should be designed to be sufficient to transfer the force intended to be resisted under the new arrangement.

Detailing and Construction Considerations

The best approach is to match the new mat and old pier cap and grade beam depths. This is, of course, not always beneficial especially if the soils that the caps or grade beams are bearing on have low bearing characteristics. Sometimes the new mat needs to be deeper than the existing grade beams or pier.

Cost/Disruption

Adding a new mat or cap or grade beam is quite disruptive and costly. The existing slab-on-grade must be sawcut and removed, then the foundation excavation completed, drilled dowels installed, rebar laid, debris in the excavation removed, and concrete placed. This is all time-consuming, messy and noisy.

23.8.3 Add Drilled Piers to an Existing Drilled Pier Foundation

Deficiency Addressed by Rehabilitation Technique

Inadequate compression capacity of a drilled pier foundation
Inadequate tension capacity of a drilled pier foundation
Inadequate lateral capacity or ductility of a drilled pier foundation

Description of the Rehabilitation Technique

An existing drilled pier footing beneath a shear wall may lack sufficient compression capacity at the toe, tension capacity at the heel, or the existing pier reinforcing may be inadequate for lateral demands. Adding new, well detailed drilled piers provides supplemental capacity to reduce the demands on existing elements or increase the overall capacity and ductility. Figure 23.8.3-1 shows an example where the “web” or center of the footing is widened or replaced and new drilled piers are added.

Design Considerations

Research basis: No research specific to supplementing existing drilled pier footings has been identified.

Relative rigidity: In Figure 23.8.3-1, all of the piers—new and existing—will participate in resisting axial and lateral demands and should be considered in modeling efforts. Demands in existing piers should be confirmed as adequate.

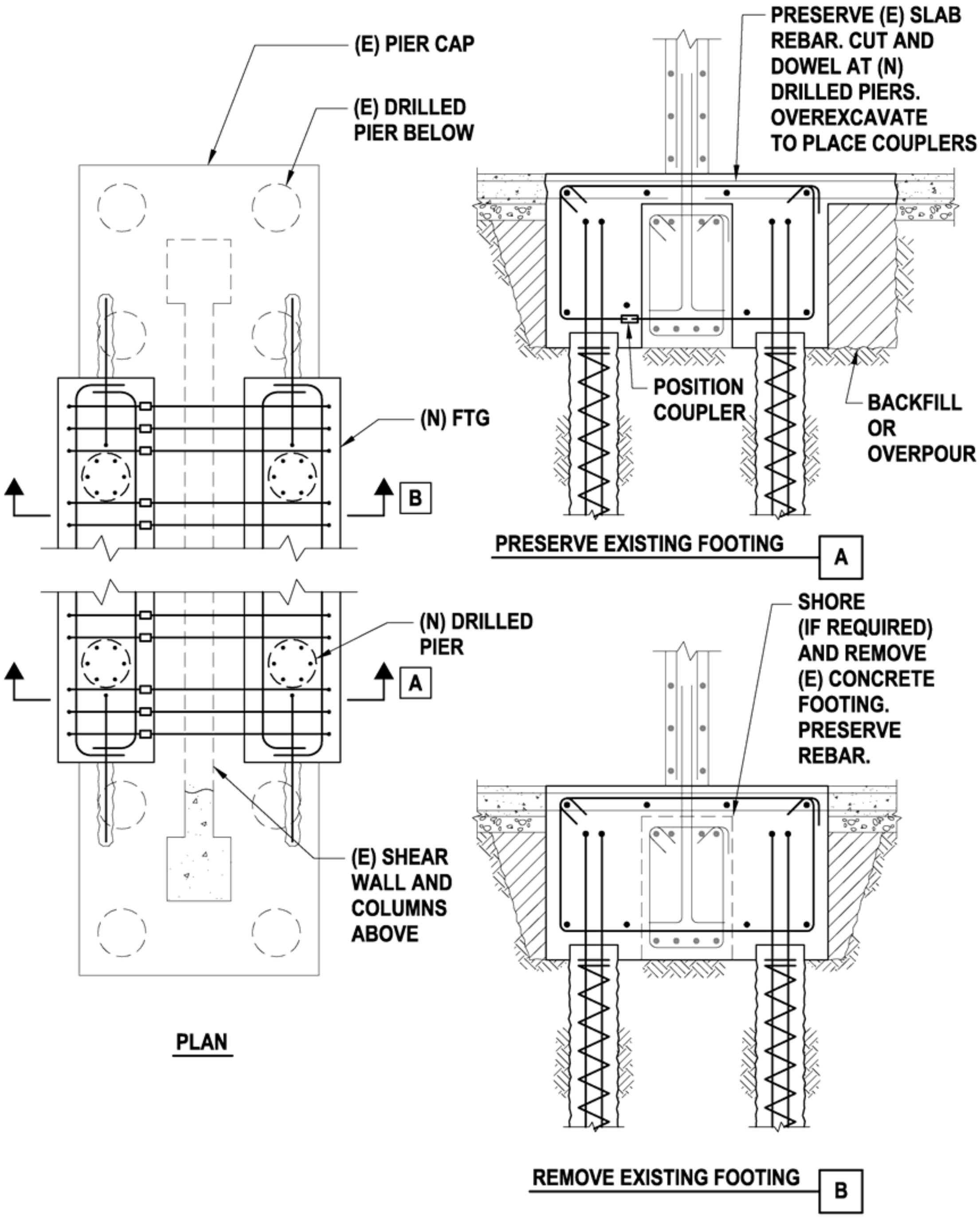


Figure 23.8.3.-1: Adding Drilled Piers to an Existing Drilled Pier Foundation

Detailing and Construction Considerations

Detailing and construction considerations for adding new drilled piers to an existing drilled pier footing include the following.

Access and height limitations: All of the drilling limitations noted in Section 23.8 for drilled piers apply here as well.

Spacing: Drilled piers typically have spacing limits of three times the pier diameter to avoid group effect reductions. This can limit the number of piers that can be installed.

Pier cap/thickened footing: The concrete above the piers will likely require widening as shown in Figure 23.8.3-1. Either drilled dowels can be installed in the existing footing, or the footing can be demolished and replaced. With a shear wall above, the wall may be able to bridge across to the belled ends of the footing without any shoring.

Cost/Disruption

Adding new drilled piers in an existing building is very disruptive, involving excavation, dust, mud, drilling/jackhammering noise and concrete placement. Protection of existing finishes in the vicinity and in the working path is necessary.

23.8.4 Add Micropiles to an Existing Drilled Pier Foundation

Deficiencies Addressed by Rehabilitation Technique

Inadequate compression capacity of a drilled pier foundation
Inadequate tension capacity of a drilled pier foundation
Inadequate lateral capacity or ductility of a drilled pier foundation

Description of the Rehabilitation Technique

An existing isolated drilled pier footing supporting a braced frame column, moment frame column or concrete column under a discontinuous shear wall may lack sufficient compression capacity, tension capacity, or the existing pier reinforcing may be inadequate for lateral demands. Adding new micropiles provides supplemental capacity to reduce the demands on the existing drilled pier. Figure 23.8.4-1 shows an example.

Design Considerations

Research basis: No research specific to supplementing existing drilled pier footings with adjacent micropiles has been identified.

Relative rigidity: In Figure 23.8.4-1, both the new micropiles and existing drilled pier will participate in resisting axial and lateral demands and should be considered in modeling efforts. Demands in the existing piers should be confirmed as adequate. While the axial strength of the new micropiles may be comparable to the drilled pier, they will much lower lateral stiffness.

Detailing and Construction Considerations

Detailing and construction considerations for adding new micropiles to an existing drilled pier footing include the following.

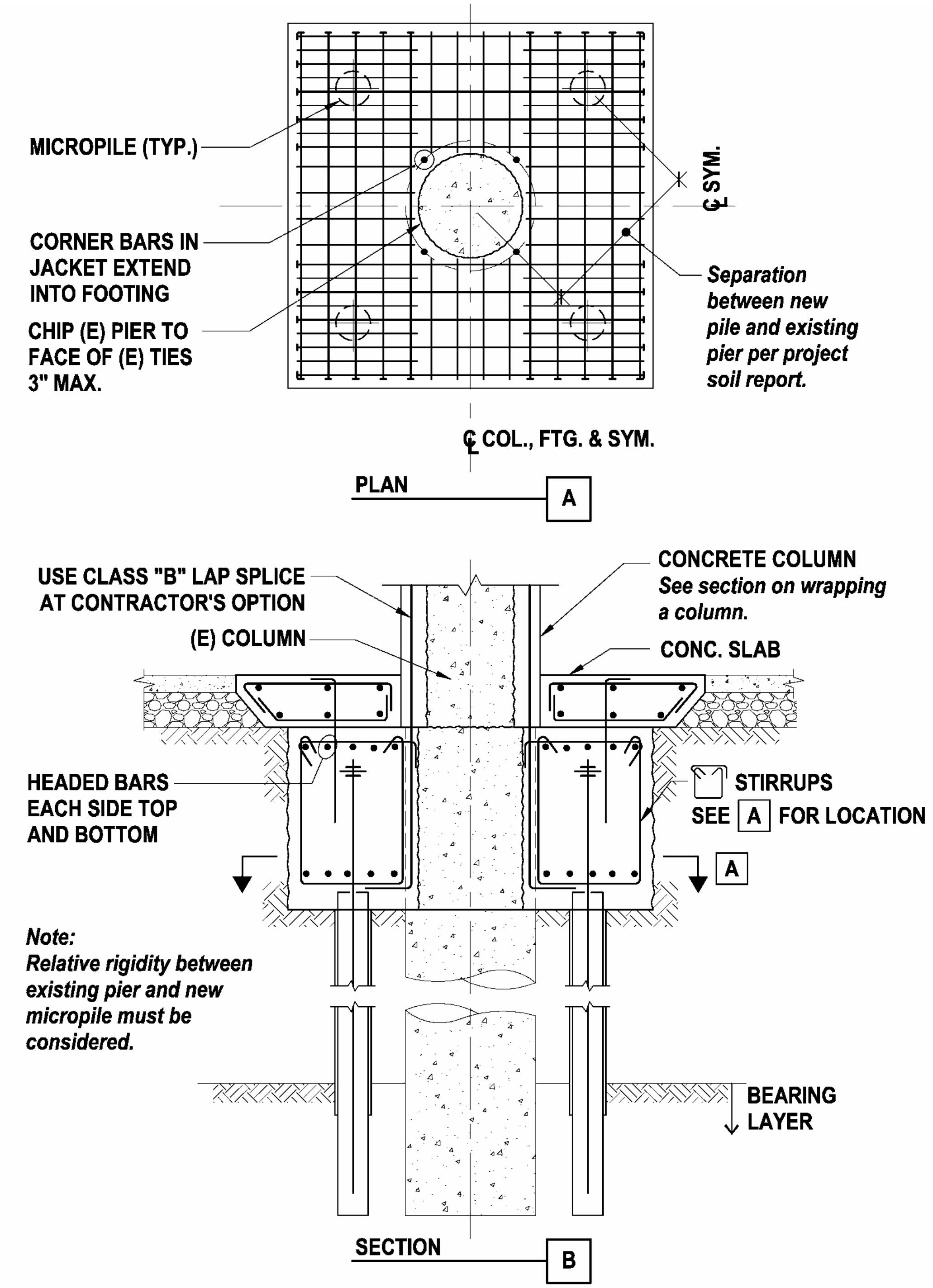


Figure 23.8.4.-1: Micropile Enhancement of an Existing Drilled Pier Footing

Access and height limitations: All of the drilling limitations noted in Section 23.7 for micropiles apply here as well.

Spacing: Spacing limits between the drilled pier and micropile to avoid group effects should be addressed.

Collar around drilled pier: In Figure 23.8.4-1, a concrete collar wraps the top of the drilled pier and provides the termination point for the pile anchors. It can also provide the starter bar location for the concrete jacket used to wrap the concrete column above.

Cost/Disruption

Adding new micropiles in an existing building is less disruptive than new drilled piers, but it still involves excavation, dust, mud, drilling/jackhammering noise and concrete placement. Protection of existing finishes in the vicinity and in the working path is necessary.

23.8.5 Add Top Bars to an Existing Pile Cap

Deficiency Addressed by Rehabilitation Technique

Inadequate bending capacity of the top of the pile cap to resist uplift forces

Description of the Rehabilitation Technique

An existing pile cap or pier cap may lack top reinforcing bars because the original design showed no net uplift. When a capacity design approach to evaluation or a pushover is conducted, it is likely that uplift will occur at some point and trigger the need for top bars in the pile cap to resist bending. Sometimes there is sufficient capacity in the reinforcing of the slab and its nominal connection to the top of the pile cap that it can serve the function of top steel. If not, top bars can be added as shown in Figure 23.8.5-1.

Design Considerations

Research basis: No research specific to adding top bars to existing pier or pile caps has been identified.

Anchorage of the pile to the pile cap: In Figure 23.8.5-1, it is assumed that the anchorage of the pile to the pile cap is adequate for uplift. If the foundation was not originally designed for uplift, only nominal anchorage between the pile and pile cap is likely to be found. This could be a single large bar or a bundle of two bars placed in a grouted hole in the top of the pile. It could be the pile reinforcing extended up into the pile cap. This would be more likely in end bearing piles where refusal is hit early and the top of the pile must be chipped down to the right elevation. It is important to realize that even a pile designed for a pin top with a central bar will resist moment unless special design considerations such as neoprene pads are added on top of the pile. This is highly unlikely in an older building. The tension in the pile anchorage under lateral loading has to be added to the uplift from the superstructure.

Anchorage of the column to the pile cap: In Figure 23.8.5-1, it is assumed that the anchorage of the column to the pile cap is adequate for uplift. If the foundation was not originally designed for uplift, anchor bolt embedments and diameters may not be sufficient.

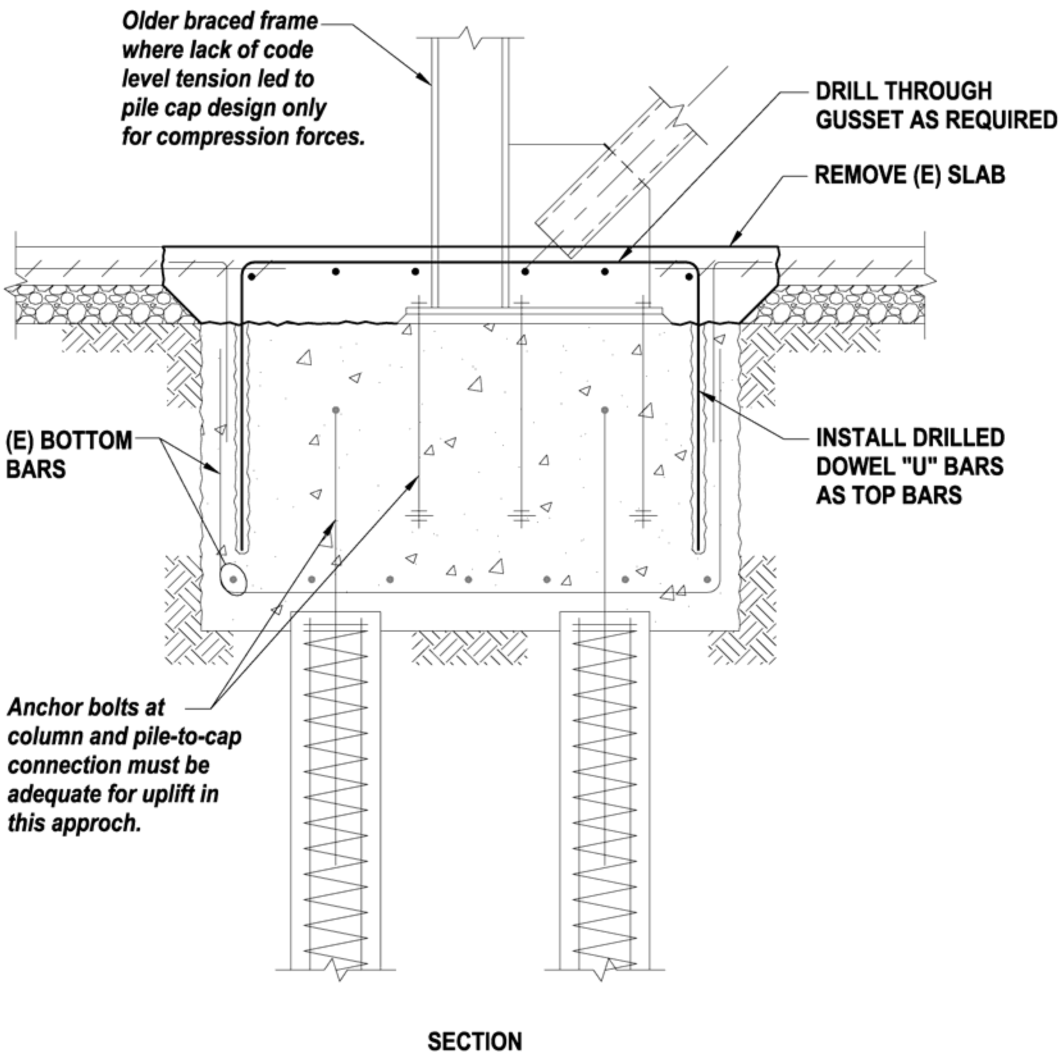


Figure 23.8.5-1: Adding Top Bars to an Existing Pile Cap

Pile cap modeling: The pile cap is typically fairly deep and may behave more as a deep beam. It can also be analyzed using strut-and-tie models.

Detailing and Construction Considerations

Detailing and construction considerations for adding top bars to an existing pile cap include the following.

Existing reinforcing: Existing reinforcing should be preserved in the footing. This will typically require placing new drilled dowels inboard of the edges of the existing footing. Added edge distance is also desirable if there or no or minimal side bars.

U bars or lapped L bars: Figure 23.8.5-1 shows new U bars, so that each leg of the U must be lowered simultaneously into the pile cap. L bars lapping over the top of the cap may make installation easier.

Confinement: The existing slab is likely to have wire mesh. To minimize vertical offsets the new and existing slabs should be dowelled together. Either the mesh in the existing slab-on-grade can be preserved when the slab is demolished or drilled dowels can be drilled into the edge of the existing slab.

Cost/Disruption

Adding top bars to the pile cap is a localized and less disruptive process than many foundation retrofits. It does involving dust, drilling/jackhammering noise and concrete placement. Protection of existing finishes in the vicinity is necessary.

23.9 Ground Improvement for Existing Shallow and Deep Foundations

23.9.1 Goals

Typical goals for ground improvement under existing shallow and deep foundations can be classified into two categories: mitigating the potential impacts of an identified geologic hazard, and enhancing the capacity of the foundation by changing the load-deformation characteristics of the foundation soil. The general techniques used to achieve these two goals separately or in combination include compaction grouting and permeation grouting. Warner (2004) is a good resource for both types of grouting.

Typical geological hazards that are mitigated using ground improvement include liquefaction and compaction settlement. These hazards have to be established by a Geotechnical Engineer who will define the recommended zone of geologic hazard mitigation.

23.9.2 Compaction Grouting

Description of the Rehabilitation Technique

Compaction grouting involves the injection of a very stiff grout at a high pressure into a layer of soil to force the individual soil particles into a tighter packing. The resulting increase in the density of the soil substantially increases its resistance to liquefaction as well as its bearing capacity. Compaction grouting can be performed in a wider range of soil types than other grouting methods. It can be performed in various types of sands, and clayey materials, but has limited effectiveness in clean coarse sands and gravels and in high plasticity soils.

The grout is required to have low flowability. This low flowability is necessary because the most important characteristic for effective densification is for the grout to form a controlled mass, which is columnar or tear-shaped, when injected. If it behaves instead like a fluid in the ground, it can create fractures in the soil, through which the grout can flow. Since the effectiveness of the grout is based on its ability to stay as a mass pushing soil particles together, that effectiveness is lost when the grout flows.

The grout—which consists of mostly of sand, cement, and water—is injected through grout holes that are drilled in a grid pattern of between 4 and 12 feet. Casing that typically has an internal diameter of 2 to 4 inches is usually installed in the grout holes. The injection pressure is directly proportional to the pumping rate, the optimal pumping rate being between 1 – 2 cubic feet per minute. Grout is usually injected in a strict primary-secondary pattern. Alternate primary holes are drilled and grouted first, followed by the secondary holes.

Grout is usually injected in stages. Staging involves the injection of only a few feet of grout hole at a time. Staging can proceed from top-down or bottom-up, the latter approach being the most commonly used.

The bottom-up grouting approach involves the following:

1. A hole is drilled to the bottom of the zone to be grouted.
2. Casing is installed to within a few feet of the bottom of the hole.
3. Grout is injected until refusal is reached. Refusal is assumed to have been reached if

    A slight movement of the ground surface or overlying improvement occurs.

    A predetermined amount of grout is injected.

    A given maximum pressure is reached at a given pumping rate.

4. The casing is raised one to two feet.
5. Grout injection is resumed until refusal is reached.
6. Steps 4 and 5 are repeated until the top of the grout zone is reached.

The level of densification achieved is verified by performing a cone penetration test or standard penetration test.

Specific Issues Relating to Grouting Under Shallow Foundations

Compaction grouting can be performed under shallow foundations in the manner described above except that the grout holes tend to be vertical rather than inclined. This is because inclined grout holes result in large horizontal areas that increase the likelihood of surface heave. The grout zone usually extends from the bottom of the existing shallow foundation to a dense or very stiff layer below the foundation.

Unlike compaction grouting performed in an open undeveloped area, the level of densification achieved in the soil below a shallow foundation cannot be verified using cone penetration or standard penetration tests. The level of densification is verified instead through monitoring the volume of grout injected in the holes.

To establish the relationship between the volume of grout and the level of densification, a pilot test program is performed in an open area adjacent to the existing building which will have compaction-grouted footings. The pilot test site is divided into segments where injection points at different spacings are laid out in a grid format as shown in Figure 23.9.2-1. In each hole,

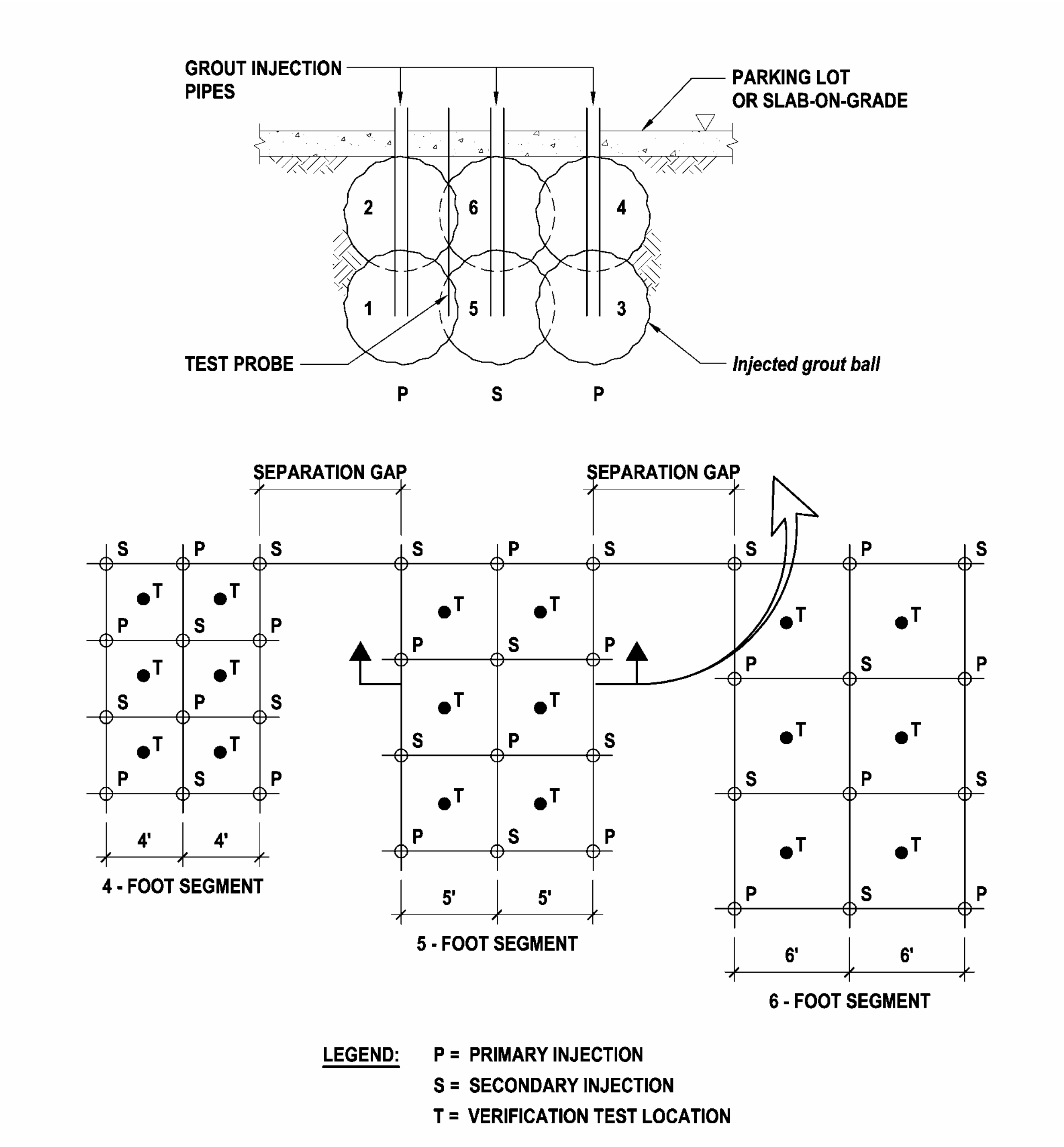


Figure 23.9.2-1: Compaction Grouting Under Existing Shallow Foundations – Pilot Test Program

grout is injected within the upper and lower limits of the zone to be grouted. The volume of grout injected in each hole is recorded. After grout injection is completed, the level of densification achieved in each segment of the pilot test area is verified by performing cone penetration or standard penetration tests at the test locations as depicted in Figure 23.9.2-1. The spacing and the corresponding volume of grout injected in each hole that produced the acceptable level of compaction is selected for production grouting underneath the shallow foundations. The injection point grid pattern will be similar to the pattern in a segment in Figure 23.9.2-1 with the test location coinciding with the center of a square footing or the centerline of a continuous footing. Note that a separation gap equal to three times the minimum spacing of four feet is placed between the segments to minimize the impact of one segment on the other.

Specific Issues Relating to Grouting Under Deep Foundations

The goal of compaction grouting around deep foundations is to enhance the skin friction contribution from the soils surrounding the deep foundation element. As in the case of shallow foundations, compaction grouting around deep foundation elements is performed using vertical rather than inclined grout holes. The grouting zone extends from the top to the tip of the deep foundation element. The injection points are usually set up at least six feet away from the center of the deep foundation element. A pilot test program, similar to the one described above for shallow foundations is performed in an open area adjacent to the existing building, which will have compaction grouted deep foundation elements. See Figure 23.9.2-2. Verification tests are performed at the location marked “T/DF”. The spacing and corresponding volume of grout injected in each hole that produced the acceptable level of compaction is selected for production grouting around the deep foundation elements. The injection point grid pattern for production grouting is set up similar to the pilot test program in Figure 23.9.2-2, except that the deep foundation element location will correspond to a location marked “T/DF.”

Cost/Disruption

Compaction grouting is quite disruptive and costly especially if the creation of injection holes includes drilling through existing pile or pier caps, grade beams, or concrete footings and slab. This grouting process could also be time-consuming and messy. Disruption to the current operations of the building is usually minimized by performing the compaction grouting at night and cleaning up the work area before the start of work the next morning. Compaction grouting is generally less costly than permeation grouting for a given scope of work.

23.9.3 Permeation Grouting Under Existing Shallow and Deep Foundations

Description of the Rehabilitation Technique

Permeation grouting involves the injection of chemical or cement grout into the pore spaces of soils and aggregates without displacing the materials. This helps solidify the usually sandy soils that are amenable to this technique. The resulting increase in shear strength of the soil substantially increases its resistance to liquefaction as well as its bearing capacity. Permeation grouting can be performed in sands and sandy soils that contain minor amounts of fine particles. The structure and the size of voids in the soil structure dictate the type of grout that can be effectively used. In general, either micro-fine cement grout or a chemical grout. The use of chemical grouts has been diminishing for environmental reasons.

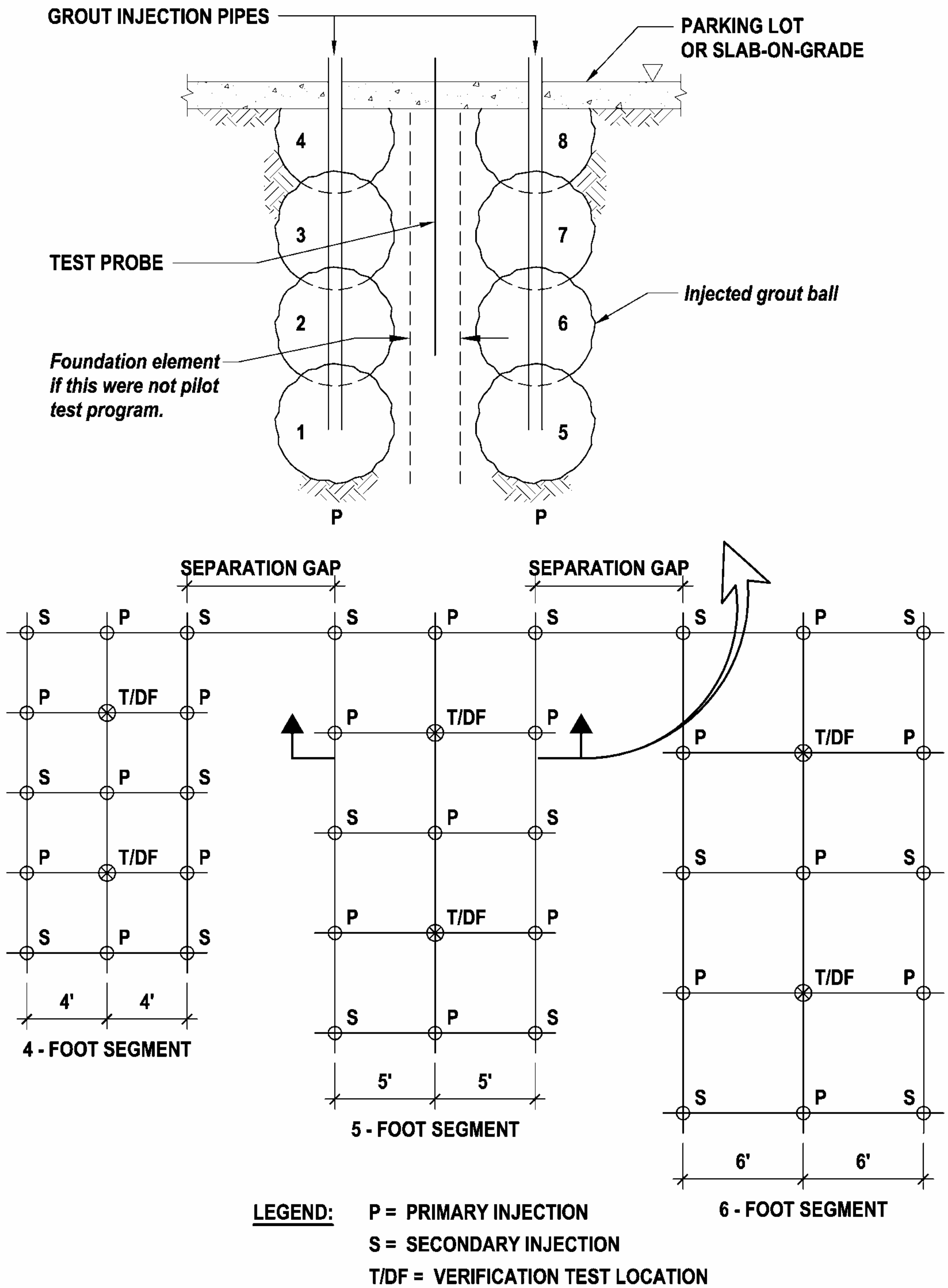


Figure 23.9.2-2: Compaction Grouting Under Existing Deep Foundations – Pilot Test Program

The grout is injected through grout holes that are drilled in a grid pattern of between 2 and 6 feet. Casing that typically has an internal diameter of 2 to 4 inches is usually installed in the grout holes. Grout is usually injected in a strict primary-secondary pattern. Alternate primary holes are drilled and grouted first followed by the secondary holes. The level of solidification achieved is verified by exhuming grouted soil bulbs, taking samples of the grouted soil and performing unconfined compression tests on the samples.

Shallow foundations: The goal for the shallow foundation elements is to create a solidified mass of sandy soil below the footprint of the footing as a minimum. The solidified mass should extend from the bottom of the footing to the top of the dense sand layer as shown in Figures 23.9.3-1 and 23.9.3-2.

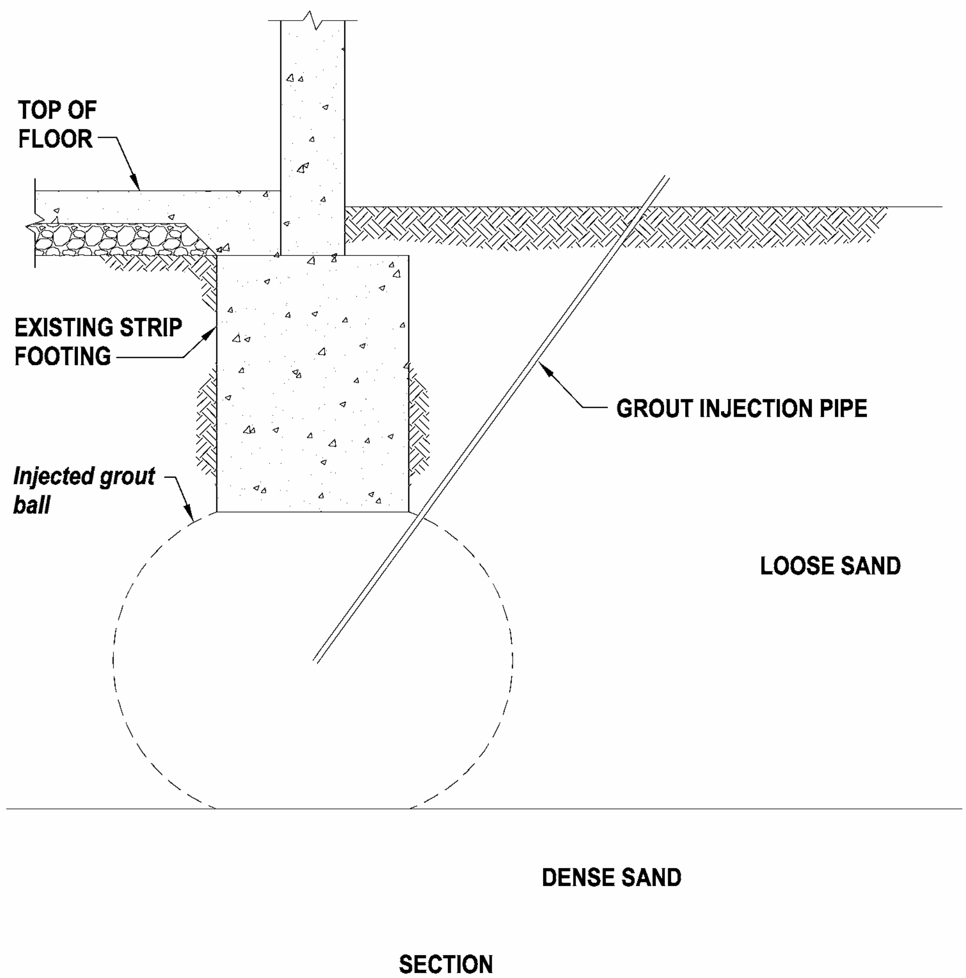


Figure 23.9.3-1: Permeation Grouting of Loose Sand Under Existing Shallow Foundation

Deep foundations: The goal for the deep foundation element is to create a zone of solidified sand around it. The injection points can be as close as three feet to the foundation elements. The zone of grouting should extend from the bottom of the grade beam or cap atop the deep foundation element to the top of the dense sand layer shown in Figure 23.9.3-3.

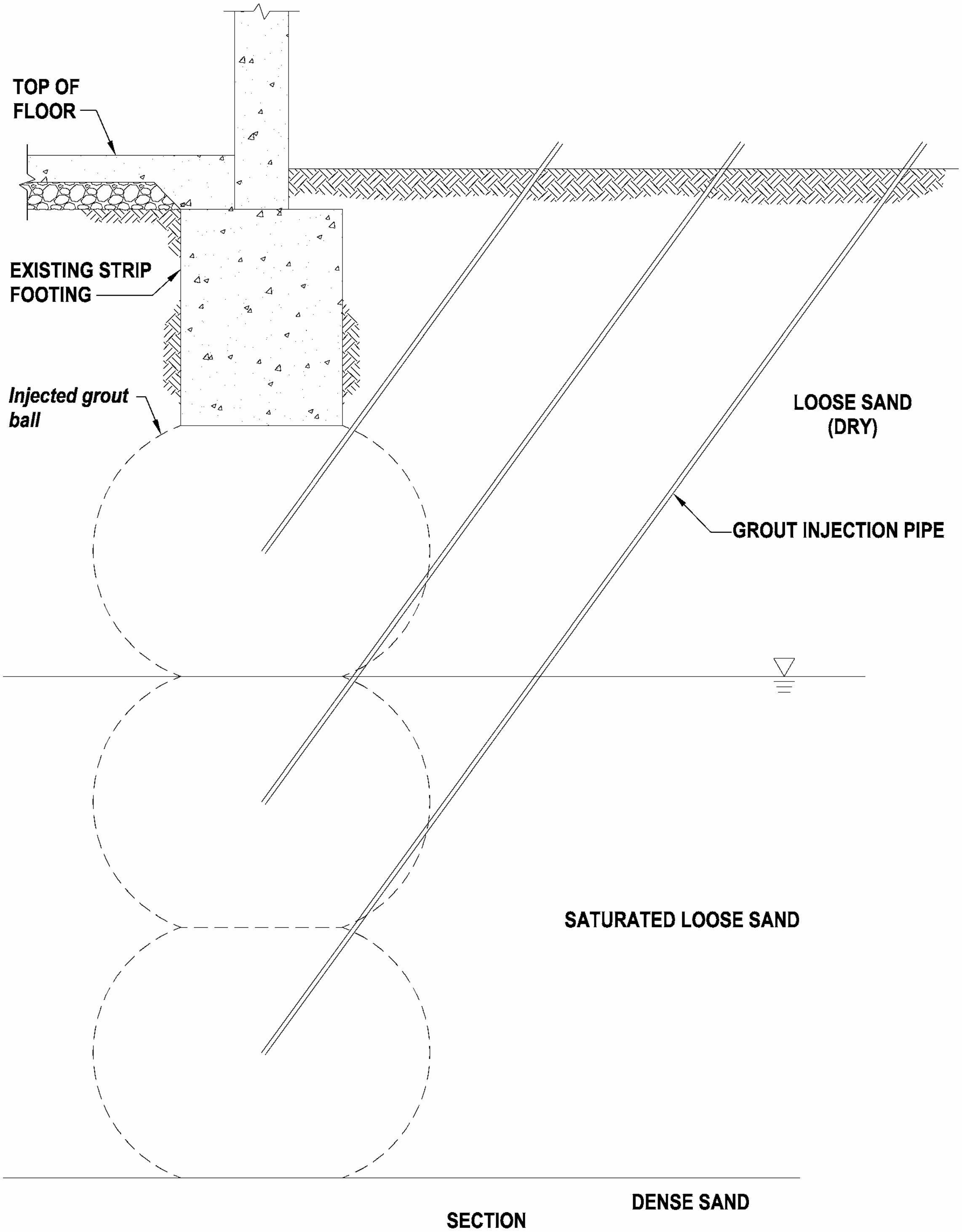


Figure 23.9.3-2: Permeation Grouting of Liquefiable Layer Under Existing Shallow Foundation

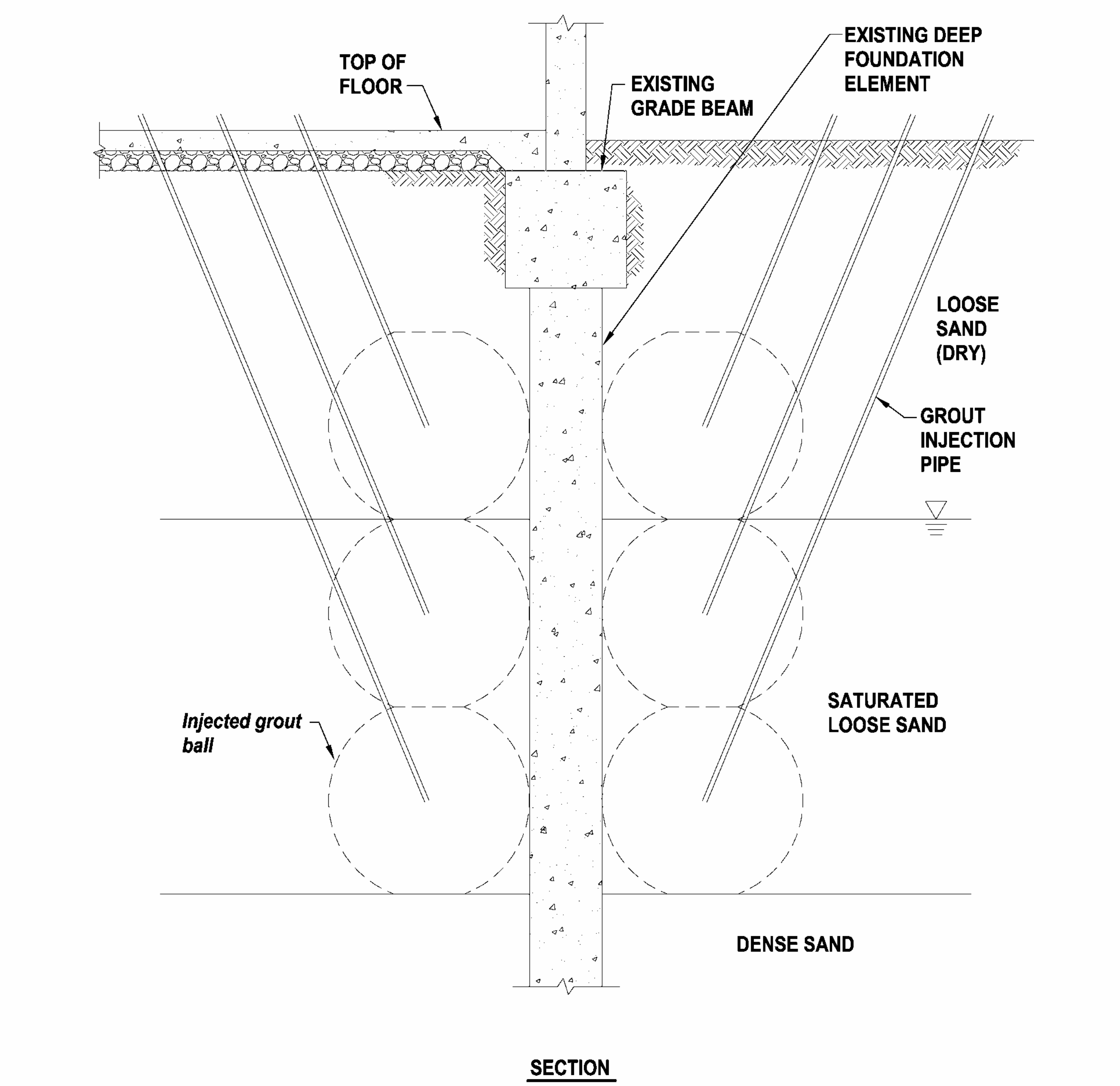


Figure 23.9.3-3: Permeation Grouting of Liquefiable Layer Around Existing Deep Foundation

Cost/Disruption

Permeation grouting can be quite disruptive and costly especially if injection holes have to be drilled through existing concrete footings and slabs. If current operations in the building are to continue, the usual approach is to do the permeation grouting at night and clean up the work area before the start of work the next morning.

In the case of grouting under shallow foundations, there is a tendency for grout to migrate down, resulting in a weakly cemented lens of sand immediately below the shallow foundation elements. The tendency can be minimized by ensuring that grouting is performed in a strictly primary-secondary sequence.

23.10 Mitigating the Impacts of Other Ground Hazards on Existing Foundations

23.10.1 Issues to be Addressed

The mitigation measures described in the preceding sections of this chapter deal primarily with individual foundation elements in a building. Even if these mitigation measures are implemented, their usefulness can be negated by other ground hazards that tend to have global stability effects on the behavior of the entire foundation system and could lead to the collapse of the building to be rehabilitated. These other hazards must therefore be mitigated if they exist, for the intent of the mitigation methods described in the previous sections to be realized. This class of ground hazards includes fault rupture, lateral spreading, and seismic-induced landslide. The potential for these hazards, the level of severity, and the necessary mitigation measures to be implemented must be established by a geotechnical engineer and/or an engineering geologist. See FEMA 274 for additional information. Mitigating the potential impacts of these hazards can be very costly. Disruption to the existing building, however, should be minimal since the work is external to the building.

23.10.2 Fault Rupture

The potential for fault rupture exists when an engineering geologist establishes through fault trenching that an active fault trace traverses the footprint of the existing building that is to be rehabilitated as depicted in Figure 23.10.2-1. The rupture results in displacement along the fault trace, which depending on the type of fault, could be lateral or vertical movement. The magnitude of earthquake-induced displacement in the ground along the fault trace can range from a few inches to several feet. Such displacements can have the effect of tearing a building apart when they occur under or adjacent to a building.

It is difficult to upgrade a building straddling an active fault to accommodate such displacements without collapse. Options for mitigating the hazard include:

- Change the occupancy level from the current to a much lower level in an effort to minimize the potential for loss of life.
- Move the affected structure to a location at least 50 feet from the mapped fault trace, if feasible.

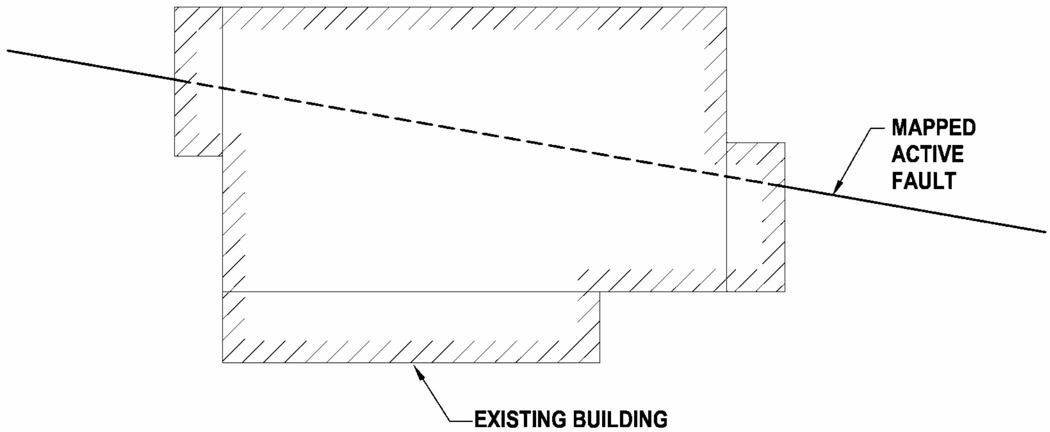


Figure 23.10.2-1: Active Fault Traversing the Footprint of Existing Building

23.10.3 Lateral Spreading

Definition of Hazard and Potential Impacts

Lateral spreading is one of the phenomena associated with liquefaction. It occurs when the blocks of non-liquefiable surface material above a layer of liquefiable soil move laterally towards an open face as depicted in Figure 23.10.3-1.   The magnitude of lateral movement can range from a few inches to several feet.

Lateral spreading has the effect of globally moving the building laterally, or tearing it apart, if the building is supported on shallow foundations or on deep foundations that do not extend into stable material below the liquefiable layer. If the building is supported on deep foundations that extend into the stable material below the liquefiable layer, lateral spreading could result in loss of lateral capacity of the deep foundation elements.

Mitigation

The potential for lateral spreading can be mitigated by creating a stable mass of material near the open face. This can be accomplished by either densifying the layer of potentially liquefiable soil or solidifying the soil to prevent liquefaction. Techniques used to achieve this goal include compaction grouting (densifying) and permeation grouting (solidifying), which were described in Section 23.9. Alternately, vibrocompaction methods involving the installation of stone columns can be used to densify the potentially liquefiable layer as shown in Figure 23.10.3-2.

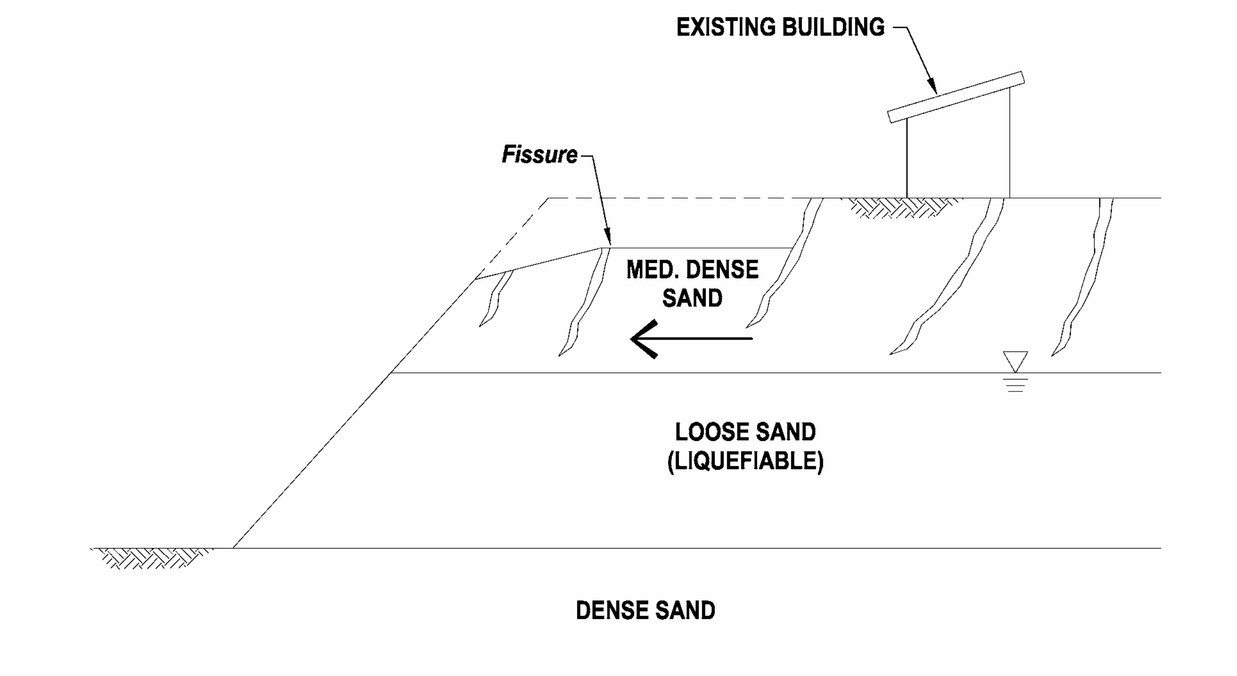


Figure 23.10.3-1: Lateral Spreading of Soil Layer Underlying Existing Building

23.10.4 Seismic-Induced Landslide

Definition of Hazard and Potential Impacts

Seismic-induced landslides result from the failure of an existing slope under earthquake loading. The landslide can occur under various sscenarios as shown in Figure 23.10.4-1. In Figure 23.10.10-1A, the landslide could undermine the building, causing it to collapse. In Figure 23.10.4-1B, the landslide displacement along the failure surface would be much greater than the displacement under the building at the ground surface. Extensional fissures, however, can occur under the building due to the displacement at depth. The geotechnical engineer and engineering geologist can estimate the magnitude of these fissures. In Figure 23.10.4-1C, the landslide could result in debris flow impact on the building.

Mitigation

Various mitigation schemes developed by the geotechnical engineer and engineering geologist can be implemented in each of the three scenarios. In the first scenario, potential mitigation measures include the construction of a stabilizing berm at the toe of the slope as shown in Figure 23.10.4-2A or the construction of a soil nail wall as shown in Figure 23.10.4-2B. In the second scenario, the existing foundation system can be enhanced to span or accommodate the estimated extensional fissures. In the third scenario, a debris wall, as shown in Figure 23.10.4-2C, can be built to protect the building or techniques in Figure 23.10.4-2B can be used to stabilize the slope.

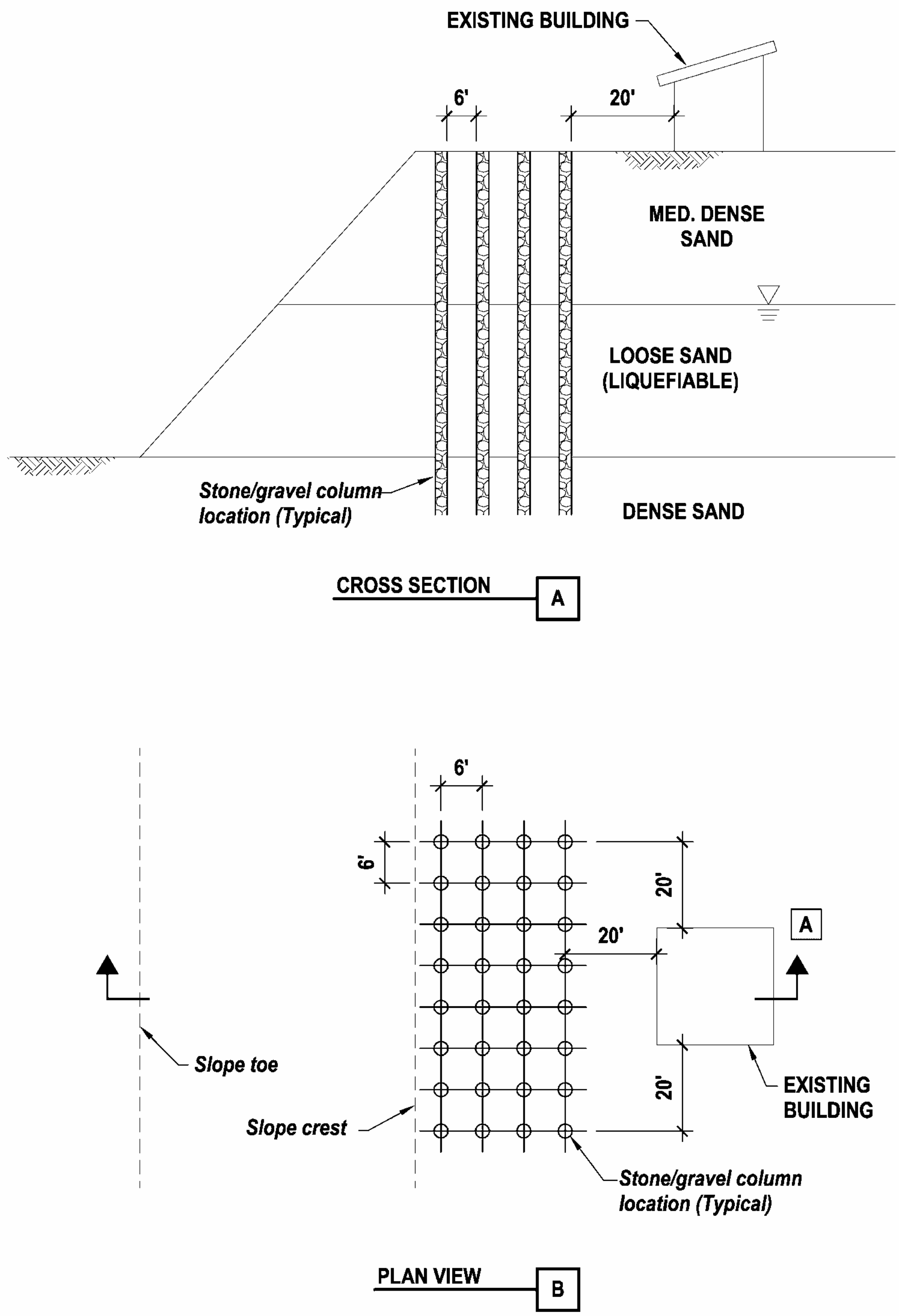


Figure 23.10.3-2: Mitigation Measure for Lateral Spreading – Stone/Gravel Column

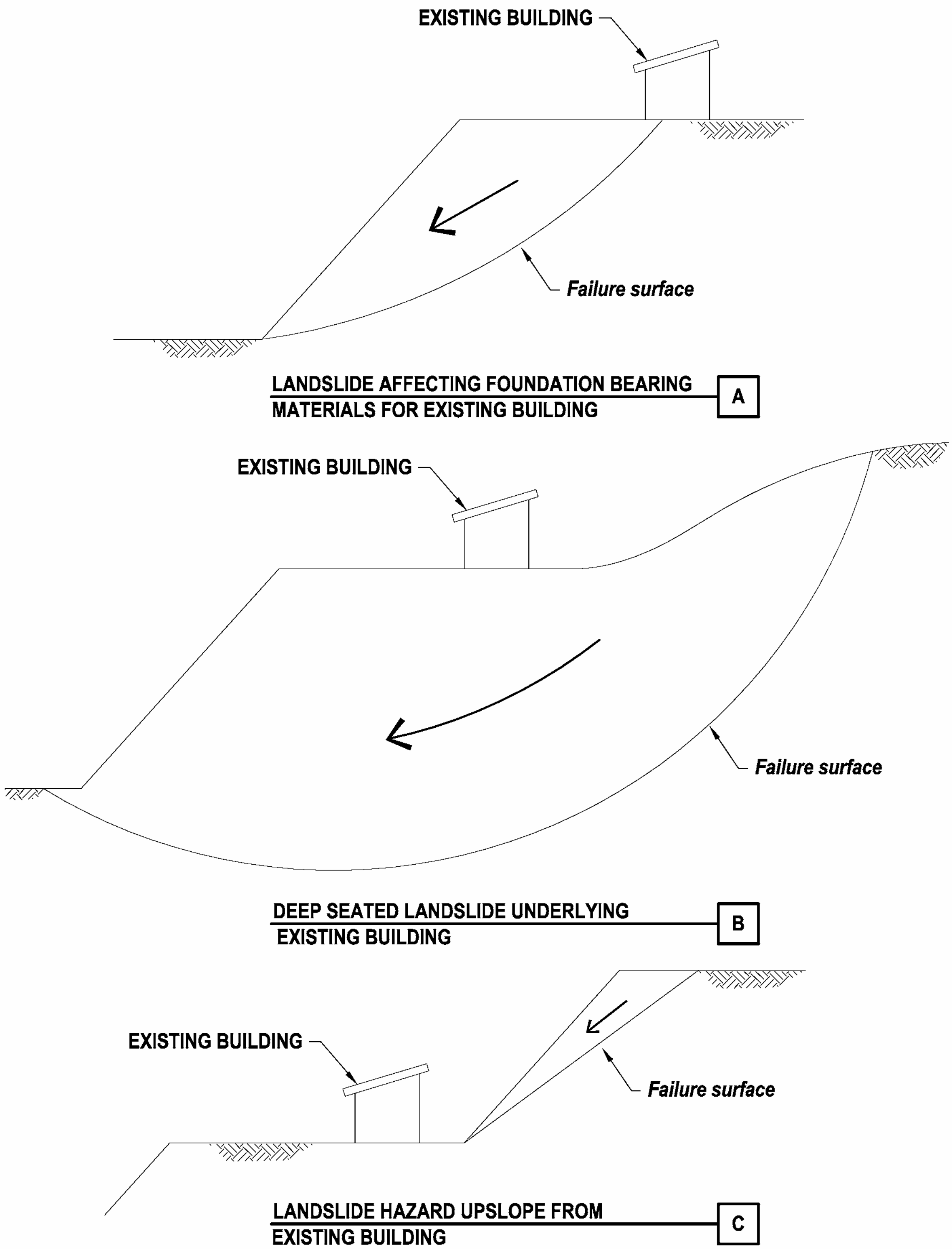


Figure 23.10.4-1: Landslide Hazards

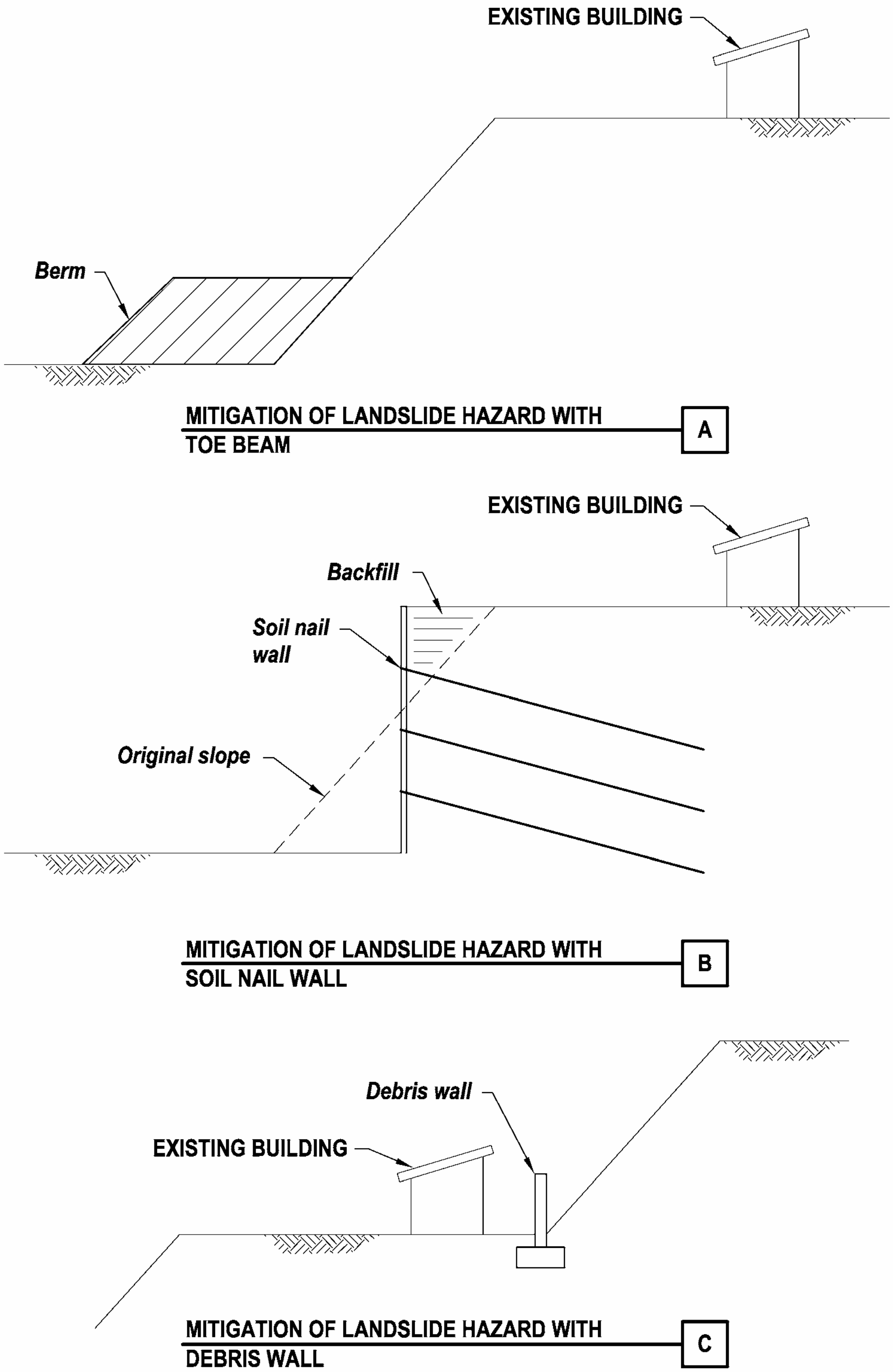


Figure 23.10.4-2: Mitigation of Landslide Hazards

23.11 References

ASCE, 2003, *Standard for the Seismic Evaluation of Buildings,* ASCE 31-03, Structural Engineering Institute of the American Society of Structural Engineers, Reston, VA.

ASTM D1143, 1994, *Standard Test Method for Piles Under Static Axial Compressive Load*, ASTM.

ASTM D1194, 1994, *Standard Test for Bearing Capacity of Soil for Static Load and Spread Footings,* ASTM.

ASTM D3689, 1995, *Standard Test Method for Piles Under Static Axial Tensile Load*, ASTM.

ATC, 1996, *The Seismic Evaluation and Retrofit of Concrete Buildings,* ATC-40, Applied Technology Council, Redwood City, CA.

Bowles, J.E., 1996, *Foundation Analysis and Design*, McGraw-Hill, New York, NY.

FEMA, 1997, *NEHRP Commentary on the Guidelines for Seismic Rehabilitation of Buildings*, FEMA 274, Federal Emergency Management Agency, Washington, D.C., November.

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

FEMA, 2005, *Improvement of Nonlinear Static Seismic Procedures*, FEMA 440, Federal Emergency Management Agency, Washington, D.C.

FHWA, 2000, *Micropile Design and Construction Guidelines Implementation Manual*, Report No. FHWA-SA-97-070., June.

PTI, 1996, *Post-Tensioning Institute Recommendations for Prestressed Rock and Soil Anchors*, Post-Tensioning Institute (PTI), Phoenix, AR.

Warner, J., 2004, *Practical Handbook of Grouting*, John Wiley & Sons, Hoboken, NJ.

Chapter 24 - Reducing Seismic Demand

24.1 Overview

Most seismic rehabilitation projects utilize rehabilitation strategies involving adding strength, stiffness, ductility, and/or improvement load path details. Another approach, less commonly employed, is to reduce the seismic demand on the structure. This chapter covers three methods of reducing seismic demand on the structure: reducing the effective seismic weight, seismic isolation, and passive damping.

24.2 Reduction of Seismic Weight

Reduction of seismic weight may reduce the seismic demand on an existing structure in certain cases; however, the engineer must carefully evaluate the dynamic effects of such an approach before adopting it as part of a retrofit scheme. Techniques may include replacing heavy cladding with a curtain wall system, removing high permanent live loads, or removing upper stories. Since removing upper stories results in a loss of usable floor area, this approach is usually considered after an owner has built a new adjacent building that provides replacement space.

While the reduction of seismic weight may potentially improve performance by changing a structure's yielding sequence, reducing story drifts, reducing global overturning, or reducing base shear, these reductions in demand, particularly base shear, may not be directly proportional to the decrease in seismic weight. For example, the removal of a building's upper stories will typically shorten the structure's fundamental period of vibration, often leading to an increased spectral acceleration. If this increase in spectral acceleration is greater than the corresponding decrease in seismic weight, the demand base shear on the structure will increase. Buildings with periods in the velocity-sensitive region of the response spectrum should be evaluated for this effect early in the development of a rehabilitation strategy. In particular, tall buildings in which much of the seismic weight is concentrated in the lower stories are likely to have very limited benefits in base shear reduction associated with the removal of upper stories.

The following examples in Figures 24.2-1, 24.2-2 and 24.2-3 illustrate how the removal of a building's upper stories may not significantly decrease the calculated base shear demand. All three examples assume a concrete moment frame system and employ ASCE 7-05 (ASCE, 2005) base shear equations. The first examines a generic model building with uniform story heights and masses, using the ASCE 7-05 approximate period calculation. The second examines a similar model building, in which additional weight is concentrated in the lower stories. The third examines a real building, using fundamental periods calculated from computer analysis. In all three examples, the removal of upper stories decreases global overturning demands but does not significantly decrease base shear demands.

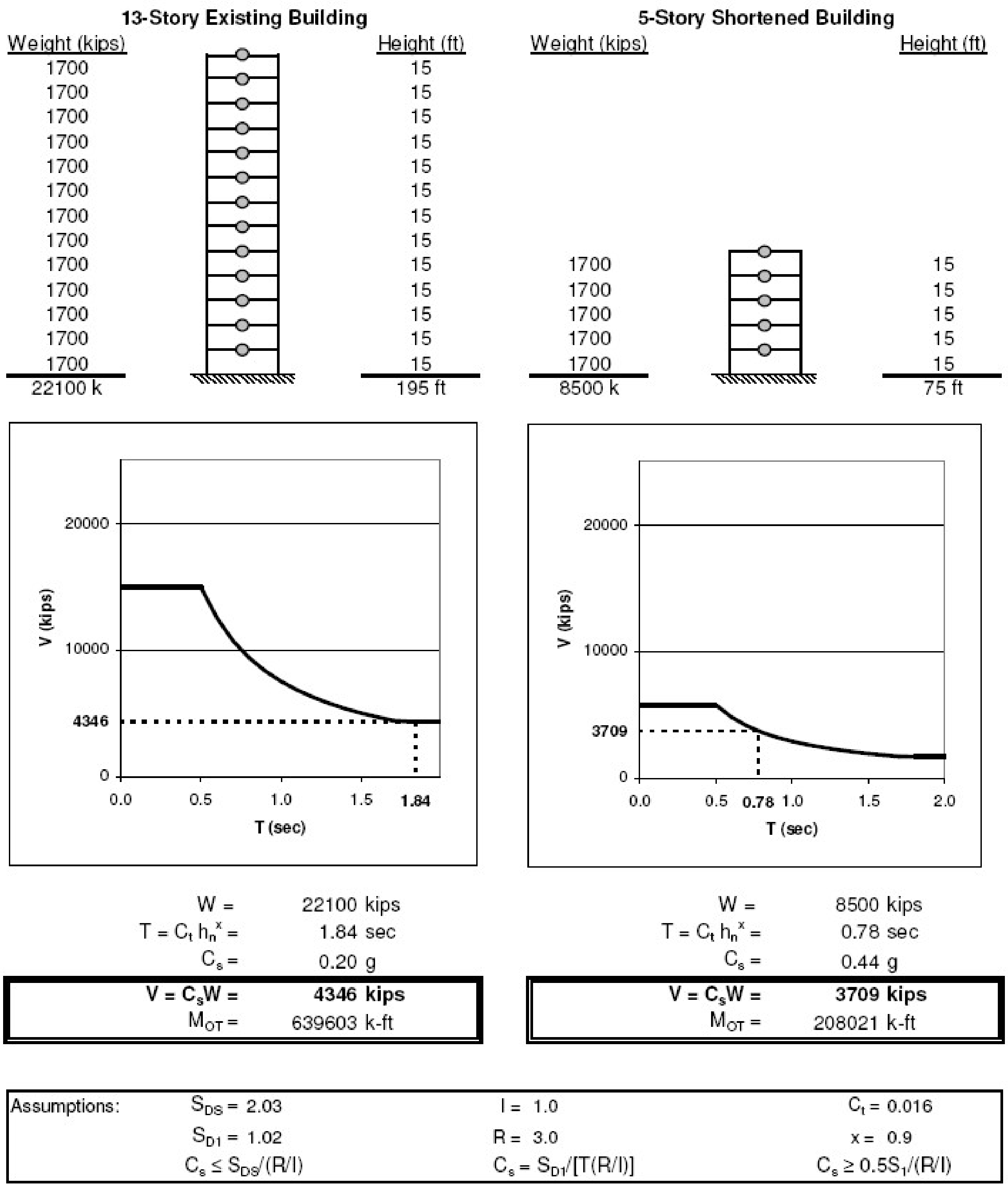


Figure 24.2-1: Generic Building Example of Decreasing Base Shear with Decreasing Height

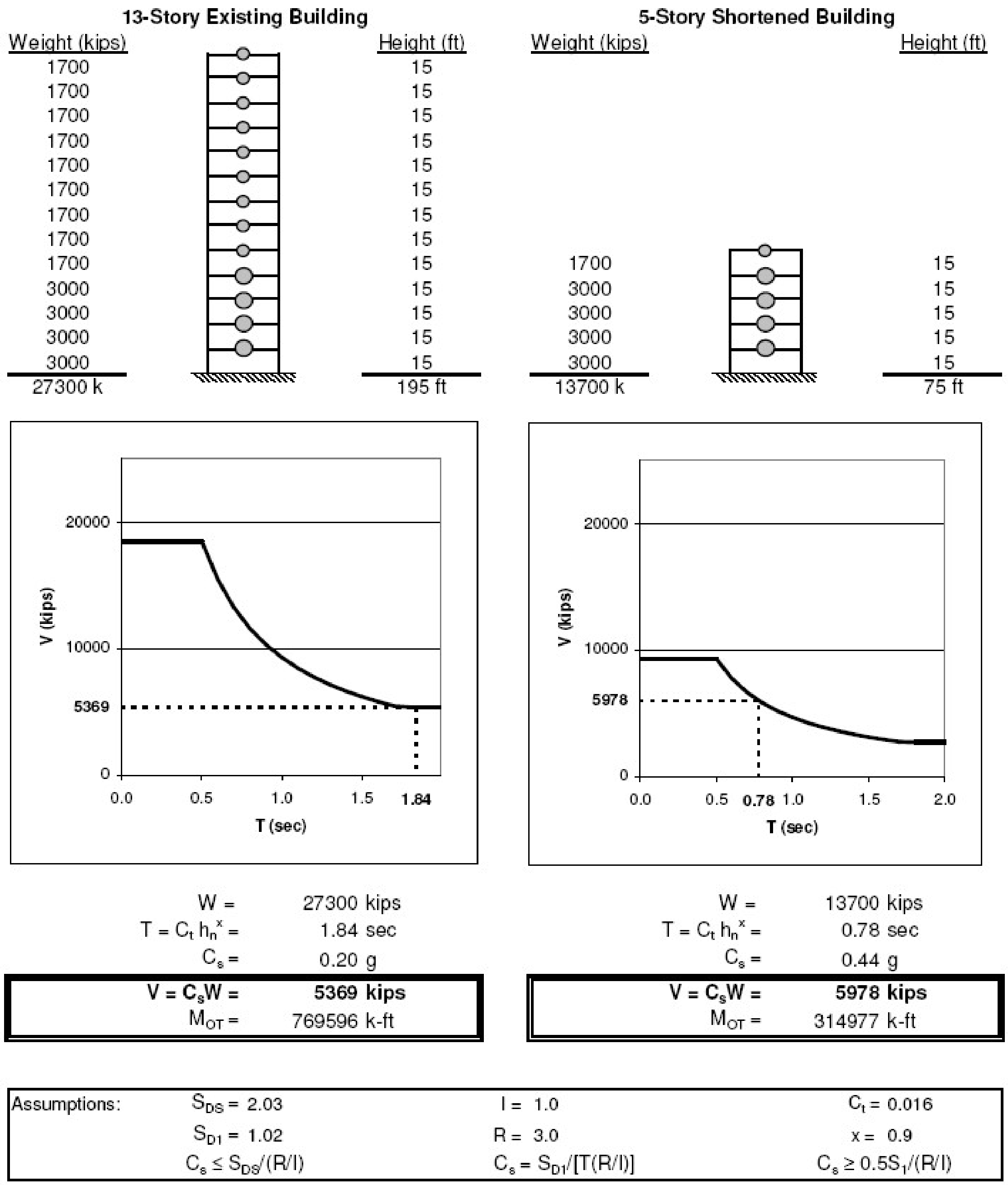


Figure 24.2-2: Generic Building Example of Increasing Base Shear with Decreasing Height

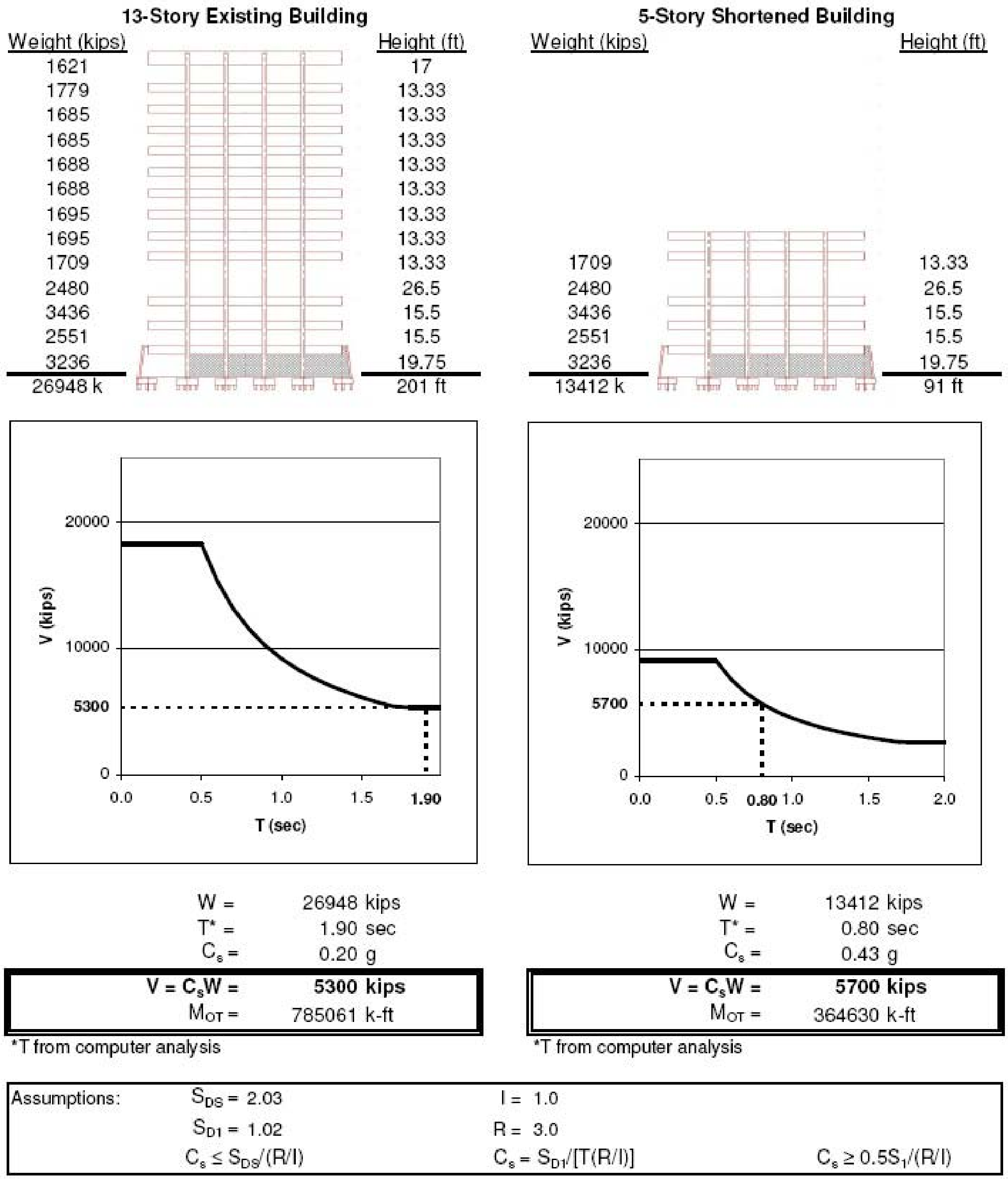


Figure 24.2-3: Real Building Example of Increasing Base Shear with Decreasing Height

24.3 Seismic Isolation

Seismic or base isolation involves lengthening a building's fundamental period of vibration to reduce the seismic demand transmitted from the ground to the building. It has been more commonly used in new building design, but it has also been employed in the United States for several high profile existing buildings as the key strategy in the rehabilitation design.

Types of isolation components: Isolation components include elastomeric bearings and sliding bearings. Elastomeric bearings include high damped rubber, low damped rubber, and low damped rubber with lead cores. Sliding bearings include the friction pendulum system. Dampers are often part of the isolation system to limit displacements. See Section 24.4 for some discussion on dampers.

Applicable buildings: The period range for isolated buildings is from about 2 seconds to 4 seconds. As such, buildings on very soft soils and very tall, flexible buildings may not achieve much benefit from isolation. Seismic isolation is usually a very expensive rehabilitation strategy and has been primarily applied in the United States to important historic structures, usually as a way of minimizing the amount of superstructure strengthening an impact on the historic fabric of the building. Isolation displacements are highly site specific, but in high seismic zones the Maximum Capable Earthquake (MCE) displacements are often on the order of 30" or more. Sufficient clearance from adjacent structures is necessary to avoid pounding during seismic response

System elements: In addition to the bearings and dampers, a complete isolation system will require a number of other special elements, including a moat around the building to accommodate the displacements. The moat has to go down past the plane of isolation. There is usually a complete or partial moat cover at the top of the moat for aesthetic or security considerations. Elevators are typically hung from the superstructure, as they cannot cross the isolation plane without special detailing. Utilities entering the building need to be able to accommodate the isolation displacements; this often triggers special vaults outside the building or areas under the building for joint details. A foundation is needed below the isolators to take the forces they impart, and a structural system is needed above the isolators as well to deliver forces to the isolators and resist the moments that are induced. All of these elements add to the cost of isolating the building.

Analysis and design requires special expertise: The analytical and design effort for an isolated building is typically much more extensive than in fixed base rehabilitations. Time history analysis, where at least the isolators are modeled nonlinearly, is standard practice. Material properties must be achievable with components in the marketplace and must account for material variability from manufacturing, loading, temperature, velocity, wear, aging, and other effects. Experience with this type of work and the properties of the various vendor's components and associated issues is quite useful.

Determining the plane of isolation: Selecting the plane of isolation is a critical design choice. It is usually near the base of the building, though there are examples of isolation elements placed at the top of columns under heavy roofs to limit the forces in the columns. Isolation at the base is either done either in the basement level, leading to some loss of use of the basement or under the basement, leading to additional excavation to place the isolators, the foundation below them and the superstructure assembly above them. The elements directly above and below the isolators are designed to take the Design Basis Earthquake (DBE) motions, without force reductions, which is a comparatively severe demand. The different types of isolation components have different sizes and means of transferring moments. With rubber bearings, P-delta moments are assumed to be split with half of the moment going up and half going down. With the traditional friction pendulum system, all of the P-delta moment goes up or down depending on which way the dish is oriented. How moments are resisted can lead to the selection of specific types of isolators.

Reducing tension: Rubber bearings are much less stiff and have much less strength to resist tension forces. Lead cores in low damped rubber bearings also have limited capacity for resisting tension. Many engineers have concerns about sliding bearings under tension, though sliding bearings that resist some tension have recently come into the market. As a result, isolation layout and superstructure design is often aimed at minimizing tension in bearings.

Transferring load from the existing building foundations to the new bearings: When a new isolated building is built, the columns and remaining elements of the superstructure can be erected directly on top of the isolation bearings. In an existing building, the superstructure is already in place. A key issue in design is developing details that delineate and facilitate the load transfer process of shoring the existing building, cutting the base of columns free, installing new foundations and new horizontal structure above the isolator, installing the isolators, and transferring load to the isolators without damaging movements of the superstructure.

Proprietary/bidding considerations: Detailing for rubber and sliding bearings is quite different. If multiple vendors are necessary, vendors of different types of systems are usually considered. Often they are procured in an early package, due to the long lead time. This also permits the design engineer to move into final design knowing which type of system will be used.

24.4 Energy Dissipation

Adding damping to an existing structure, like seismic isolation, is a relatively unusual seismic rehabilitation strategy. The added damping reduces overall building displacement and acceleration response, and local interstory drifts; but it can impart additional localized forces that must be addressed.

Types of damping components: FEMA 356 provides guidance for displacement-dependent devices or velocity-dependent devices. Displacement-dependent devices include devices that exhibit rigid-plastic (friction devices), bilinear (metallic yielding devices) or trilinear hysteresis. Velocity-dependent devices include solid and fluid viscoelastic devices and fluid viscous devices. There are other devices as well, including shape-memory alloys, friction-spring assemblies with recentering capability, and fluid restoring force-damping devices.

Applicable buildings: Most engineers believe that adding damping is most relevant in flexible buildings, such as steel or concrete moment frames. Damping is also a common element in the seismic isolation system, but there it must accommodate very large displacements. See Section 24.3.

System elements: Figures 24.4-1 and 24.4-2 show examples of adding damping devices in an existing steel moment frame building to minimize drifts and demands on the beam-column joints. Other dampers, such as wall dampers, are possible but not shown. The damper must be connected to the existing structure and potentially the foundation. Installing dampers is similar to installing braced frames. See Chapter 8 for detailed discussion on adding braced frames to a steel building and Chapter 12 for adding a braced frame to a concrete building. Some damper devices and orientations require out-of-plane bracing for stability, such as those shown in Figure 24.4-2.

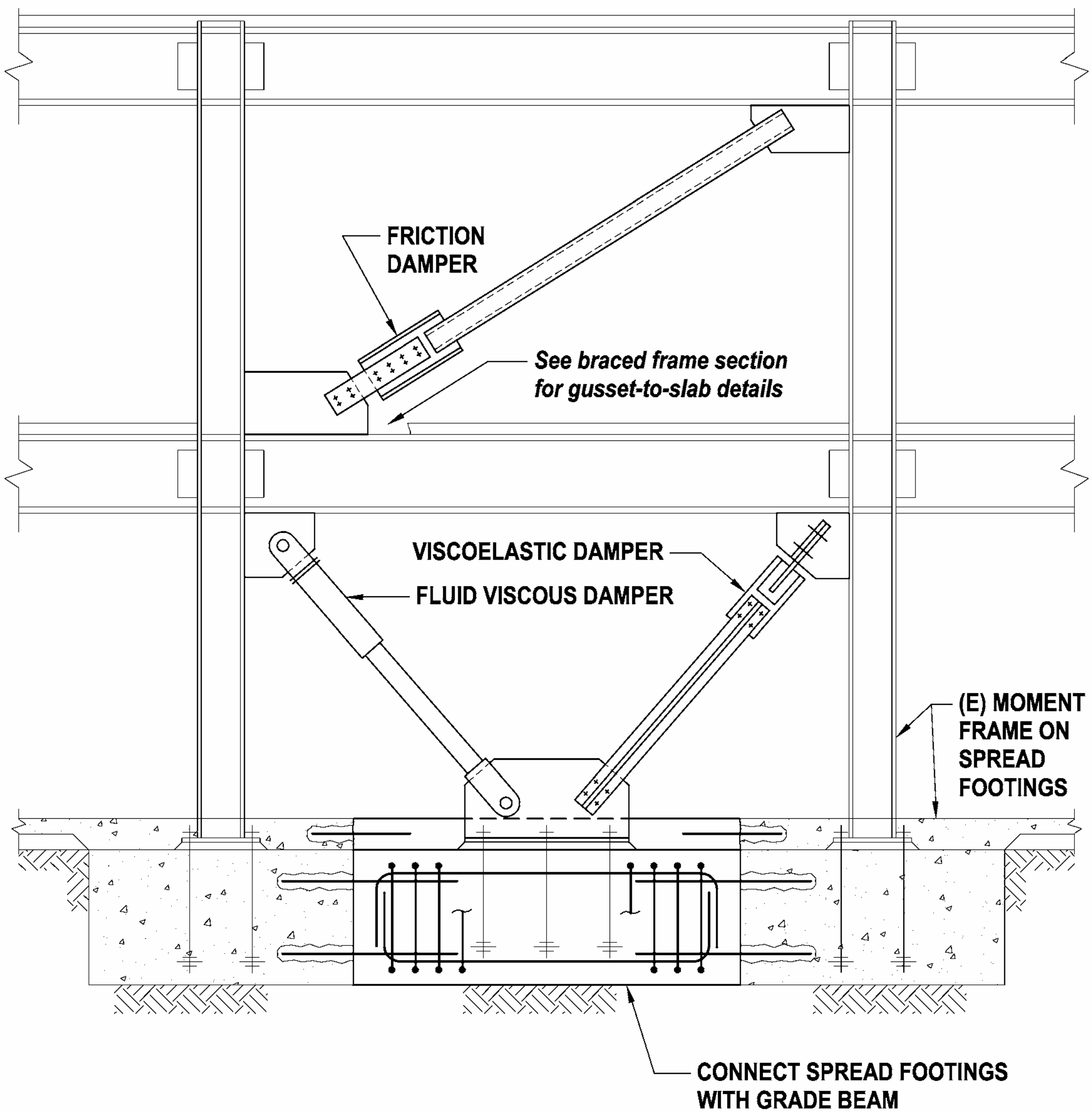


Figure 24.4-1: Damper Alternatives for Rehabilitating an Existing Moment Frame

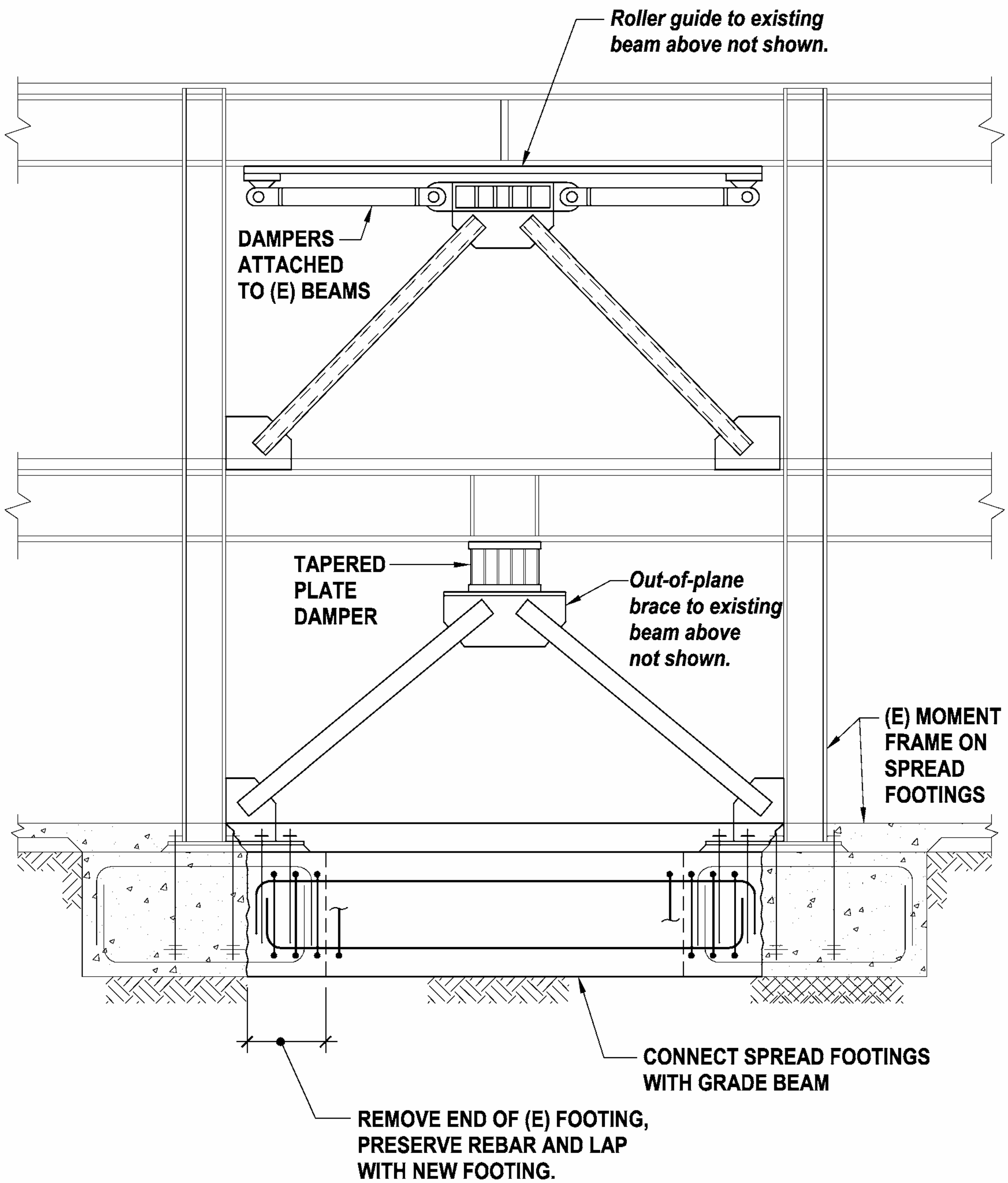


Figure 24.4-2: Additional Damper Alternatives for Rehabilitating an Existing Moment Frame

Analysis can require special expertise: The analytical and design effort for a rehabilitation design involving damping can be more extensive than in fixed base rehabilitations. Time history analysis, where at least the dampers are modeled nonlinearly, is common. Material properties must be achievable with components in the marketplace and must account for material variability from manufacturing, temperature, velocity, wear, aging, and other effects. Experience with this type of work and the properties of the various vendor's components and associated issues is useful. Hanson and Soong (2001) is a comprehensive monograph that covers analysis of buildings with supplemental energy dissipation devices, and it includes several design examples of seismic rehabilitation using damping devices.

Aesthetic impact: Adding dampers looks very similar to adding a braced frame, with the resulting visual and programmatic impacts. Some dampers or their connections can be particularly visually obtrusive.

Checking the existing structure: When dampers are added to the structure, the loads they impart locally must be considered in the design.

Proprietary issues: Most dampers available on the market are proprietary. Material properties, testing histories, limitations and detailing considerations are obtained from the manufacturer. Like seismic isolation components, the particular category of damper such as a fluid viscous damper or a friction damper is usually selected early in the design because the analysis and detailing can be significantly different between categories. There is also a patent regarding certain techniques for connecting bracing and dampers to beams when sliding is employed.

24.5 References

ASCE, 2005, *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-05, American Society of Civil Engineers, Reston, VA.

Hanson, R.D. and T.T. Soong, 2001, *Seismic Design with Supplemental Energy Dissipation Devices*, Monograph MNO-8, Earthquake Engineering Research Institute, Oakland, CA.

Glossary

BLOCKED DIAPHRAGM: A diaphragm in which all sheathing edges not occurring on framing members are supported on and connected to blocking.

BOUNDARY ELEMENT: An element at the edge of an opening or at the perimeter of a shear wall or diaphragm.

BOUNDARY NAILING: Nailing at the perimeter edge of a wood diaphragm to framing members and blocking below.

BRACED FRAME: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a building frame or dual system to resist lateral forces.

CHEVRON BRACING: Bracing where a pair of braces, located either both above or both below a beam, terminates at a single point within the clear beam span.

CHORD: See DIAPHRAGM CHORD.

COLLECTOR: A member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral-force-resisting system (also called a drag strut).

CONCENTRICALLY BRACED FRAME (CBF): A braced frame in which the members are subjected primarily to axial forces.

CONTINUITY PLATES: Steel column stiffeners at the top and bottom of the panel zone. They are also known as transverse stiffeners.

CONTINUITY TIES: Structural members and connections that provide a load path between diaphragm chords to distribute out-of-plane wall loads.

COUPLING BEAM: A structural element connecting adjacent shear walls.

DAMPING: The internal energy absorption characteristic of a structural system that acts to attenuate induced free vibration.

DEMAND: The prescribed design forces required to be resisted by a structural element, subsystem, or system.

DIAPHRAGM: A horizontal, or nearly horizontal, system designed to transmit lateral forces to the vertical elements of the lateral-force-resisting system. The term "diaphragm" includes horizontal bracing systems.

DIAPHRAGM CHORD: The boundary element of a diaphragm or shear wall that is assumed to take axial tension or compression.

DIAPHRAGM STRUT: The element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to vertical-resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression. Also refers to drag strut, tie, or collector.

DOUBLER PLATE: A steel plate added to a panel zone to increase panel zone strength.

DRAG STRUT: See COLLECTOR.

DRIFT: See STORY DRIFT.

DUCTILITY: The ability of a structure or element to dissipate energy inelastically when displaced beyond its elastic limit without a significant loss in load-carrying capacity.

ECCENTRICALLY BRACED FRAME (EBF): A diagonal braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column joint or from another diagonal brace.

EDGE NAILING: Nailing at the perimeter edge of a wood structural panel in a shear wall or diaphragm to framing members and blocking.

FIELD NAILING: Nailing within the interior of a wood structural panel in a shear wall or diaphragm to framing members.

FUNDAMENTAL PERIOD OF VIBRATION: The time it takes the predominant mode of a structure to move back and forth when vibrating freely.

HORIZONTAL BRACING SYSTEM: A horizontal truss system that serves the same function as a diaphragm.

K-BRACING: Bracing where a pair of braces located on one side of a column terminates at a single point within the clear column height.

LAMINATED VENEER LUMBER: An engineered wood product created by layering dried and graded wood veneers with waterproof adhesive into blocks of material. It is also known as structural composite lumber.

LATERAL FORCE-RESISTING SYSTEM: That part of the structural system assigned to resist lateral forces.

LINK BEAM: That part or segment of a beam in an eccentrically braced frame that is designed to yield in shear and/or bending so that buckling or tension failure of the diagonal brace is prevented.

MOMENT RESISTING SPACE FRAME: A structural system with an essentially complete space frame providing support for vertical loads.

NOTCH TOUGHNESS: A measure of material ductility related to the ability to resist fracture. It is typically measured with Charpy V-notch (CVN) test standards.

PANEL ZONE: Area of the beam-to-column connection delineated by beam and column flanges.

REDUNDANCY: A measure of the number of alternate load paths that exist for primary structural elements and/or connections such that if one element or connection fails, the capacity of alternate elements or connections are available to satisfactorily resist the demand loads.

RE-ENTRANT CORNER: A corner on the exterior of a building that is directed inward such as the inside corner of an L-shaped building.

SHEAR WALL: A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall.

SHOTCRETE: Concrete that is pneumatically placed on vertical or near vertical surfaces typically with a minimum use of forms.

SOFT STORY: A story in which the lateral stiffness is less than 70 percent of the stiffness of the story above.

SOIL-STRUCTURE RESONANCE: The coincidence of the natural period of a structure with a dominant frequency in the ground motion.

STORY DRIFT: The displacement of one level relative to the level above or below.

STRUCTURE: An assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

SUBDIAPHRAGM: A portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphagm.

SUBSYSTEMS: One of the following three principle lateral-force-resisting systems in a building: vertical resisting elements, diaphragms, and foundations.

SUPPLEMENTAL ELEMENT: A new member added to an existing lateral-force-resisting subsystem that shares in resisting lateral loads with existing members of that subsystem.

TIE-DOWN: A prefabricated steel element consisting of a tension rod, end brackets and bolts or lags used to transfer tension across wood connections. It is also known as a hold-down.

V-BRACING: Chevron bracing that intersects a beam from above. Inverted V-bracing is that form of chevron bracing that intersects a beam from below.

VERTICAL-RESISTING ELEMENTS: That part of the structural system located in a vertical or near vertical plane that resists lateral loads (typically a moment frame, shear wall, or braced frame).

WEAK STORY: A story in which the lateral strength is less than 80 percent of that in the story above.

WOOD STRUCTURAL PANEL: A wood-based panel product that satisfies the requirements of Voluntary National Product Standard PS-1 or PS-2 and is bonded with waterproof adhesive. Included under this designation are plywood, oriented strand board (OSB) and composite panels.

X-BRACING: Bracing where a pair of diagonal braces crosses near mid-length of the bracing members.

Abbreviations

The following abbreviations are commonly used by structural engineers and have been used in figures and/or text throughout the document.

B.N.	Boundary nailing
BRBF	Buckling-restrained braced frame
CIP	Cast-in-place
C.J.	Construction joint
CJP	Complete joint penetration weld
CL	Centerline
CP	Complete penetration weld
(E)	Existing
EA.	Each
E.N.	Edge nailing
F.N.	Field nailing
FTG.	Footing
MC	Moisture content
M/E/P	Mechanical/electrical/plumbing
(N)	New
PL	Plate
PP	Partial penetration weld
RBS	Reduced beam section (in a beam-to-column moment frame connection)
SPSW	Steel plate shear wall
TYP.	Typical
WSMF	Welded steel moment frame

Workshop Participants

A workshop was held on September 20-21, 2005 in Oakland, California to solicit comment on a draft of the final document. Workshop participants included the following individuals.

ICSSC Members

Krishna Banga
Department of Veteran Affairs
Washington, D.C.

Cathleen Carlisle
FEMA
Washington, D.C.

James A. Caulder
Air Force Civil Engineer Support Agency
Tyndall AFB, FL

James Farasatpour
Federal Aviation Administration
Inglewood, CA

Jack Hayes
U.S. Army
Champaign, IL

Richard Kahler
U.S. Navy
Norfolk, VA

H.S. Lew
NIST
Gaithersburg, MD

Dai H. Oh
U.S. Department of State
McLean, VA

Raymond F. Schuler
NASA Ames Research Center
Moffett Field, CA

Academics and Industry

Daniel Abrams
University of Illinois
Urbana, IL

Melvyn Green
Melvyn Green & Associates
Torrance, CA Richard Howe
Memphis, TN

Lawrence Reaveley
University of Utah
Salt Lake City, UT

Andrew W. Taylor
KPFF
Seattle, WA

James Parker
Simpson, Gumpertz & Heger
Waltham, MA

Barry H. Welliver
Draper, UT

Project Review Panel

Daniel Dolan
Terry Dooley
Kurt Gustafson
Robert D. Hanson
Neil Hawkins
Jim Harris
Bela I. Palfalvi
Daniel Shapiro

Technical Update Team and Contributing Staff

Kelly Cobeen
William T. Holmes
Jack Hsueh
Bret Lizundia
James Malley
Karl Telleen