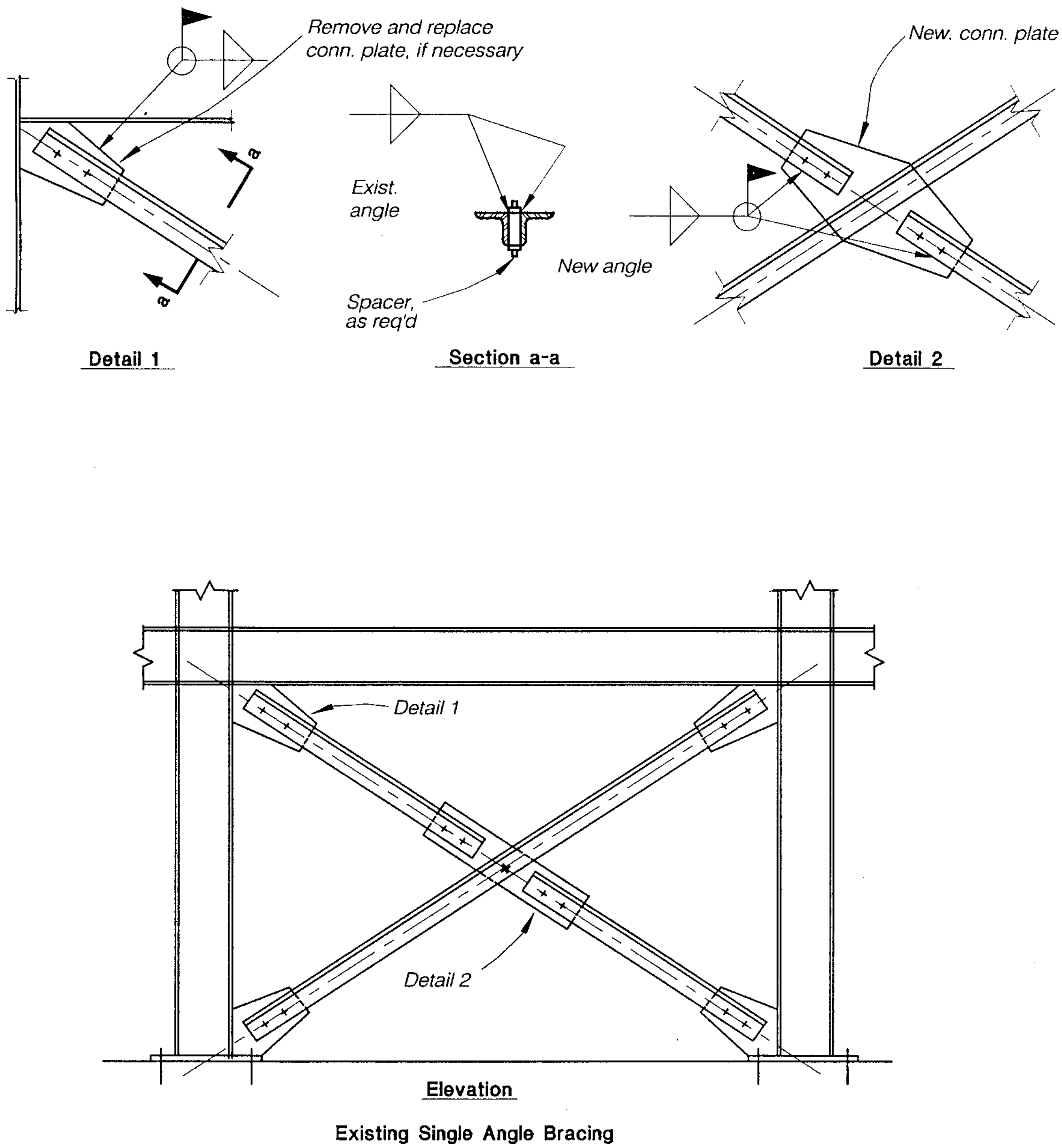


Figure 8-10. Strengthening of Single-Angle Bracing

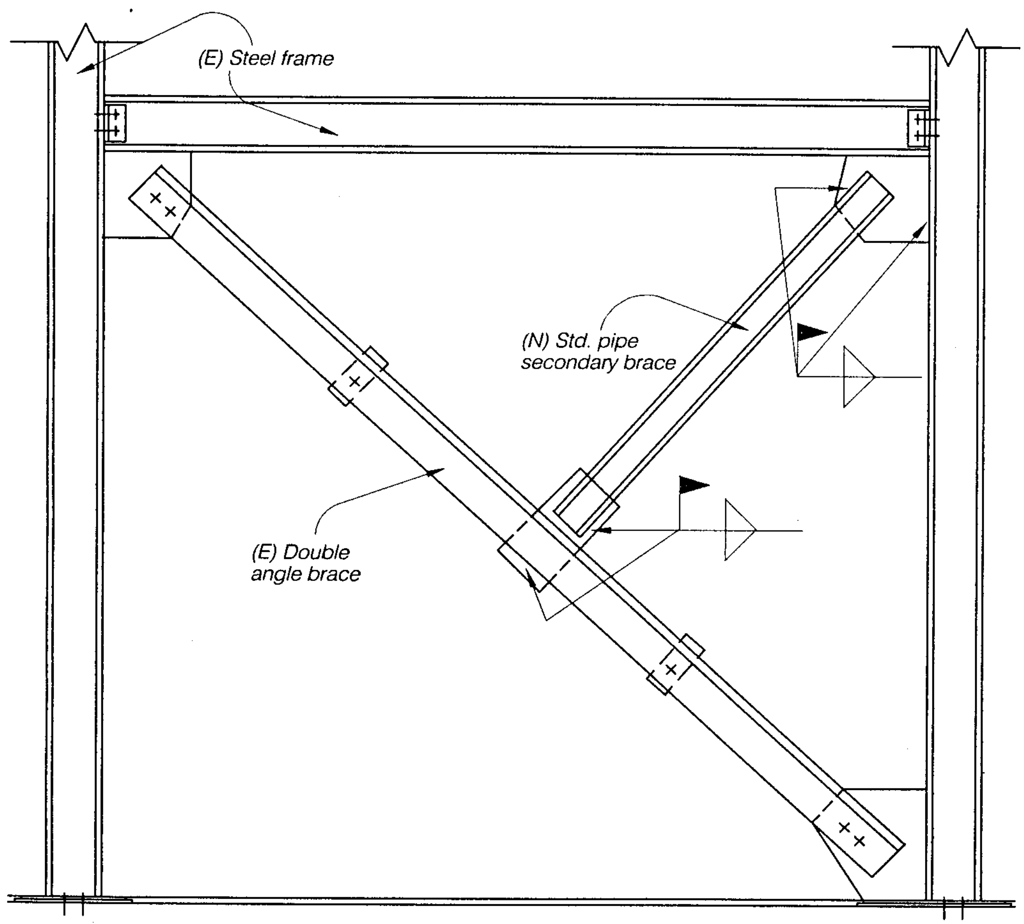


Figure 8-11. Strengthening an Existing Brace with Secondary Bracing

- Increasing the capacity of the connection by additional bolting or welding;

- Increasing the capacity of the connections by removing and replacing the connection with elements of greater capacity; and

- Reducing the loads on the-braces and their connections by providing supplemental vertical-resisting components (i.e., shear walls, bracing, or eccentric bracing).

Adequate capacity of brace connections is essential to the proper performance of the brace. The capacity of the brace is limited by its compression capacity, and the connection may have been designed for this load. When the brace is loaded in tension, however, the brace may transmit significantly higher forces to the connection. If the existing connection members (e.g., gusset plates) have sufficient capacity (TI 809-04, Figure 7-22), the most economical alternative may be to increase the existing connection capacity by providing welding or bolts. If the existing gusset plates have inadequate capacity, the existing configuration and accessibility need to be assessed to determine whether it is more economical to add supplemental connecting members, or replace the existing connecting members with members of greater capacity. If the existing brace members require strengthening or replacement with members of greater capacity, it is probable that new connections would be the most cost-effective alternative. Whether reducing loads by adding supplemental members is a cost-effective alternative is most likely to be a consideration when assessing the capacities of the braces, not the brace connections. The merits of this alternative are discussed above.

(4) Strengthening techniques for inadequate axial load capacity by adding cover plates to the member flanges or by boxing the flanges. Deficient axial load capacity of existing bracing system columns and beams can be improved by:

- Providing additional load capacity by adding cover plates to the member flanges or by boxing the flanges;

- Providing additional axial load capacity by jacketing the existing members with reinforced concrete; and

- Reducing the loads on the beams and columns by providing supplemental vertical-resisting components (i.e., shear walls, bracing, or eccentric bracing).

The most cost-effective alternative for increasing the capacity of the existing beams and columns in a concentrically braced frame system is to add cover plates to the flanges or to box the flanges. The effort involved in adding cover and box plates includes removing the existing fireproofing and nonstructural obstructions. Jacketing of existing members with reinforced concrete would seldom be cost-effective due to the significant forming effort required. The relative merits of reducing the loads by providing supplemental members is discussed in previous paragraphs.

e. Rod or other tension bracing.

(1) Deficiencies. The principal deficiencies of rod or other tension bracing systems are:

- Inadequate tension capacity of the rod, tensile member, or its connection; and

- Inadequate axial capacity of the beams or columns in the bracing system.

(2) Strengthening techniques for inadequate tension capacity of the rod, or other tension member, or its connection. Deficient tension capacity of the rod or other tension member and its connection can be improved by:

- Increasing the capacity by strengthening the existing tension members;

- Increasing the capacity by removing the existing tension members and replacing with new members of greater capacity;

- Increasing the capacity by removing the existing tension member and replacing it with a diagonal or X-bracing capable of resisting compression as well as tension forces; and

- Reducing the forces on the existing tension members by providing supplemental vertical-resisting elements (i.e., additional tension rods).

Tension bracing is commonly found in light industrial steel-frame buildings, including some designed for prefabrication. The most common deficiency is inadequate tensile capacity in the tension rods. These rods generally are furnished with upset ends so that the effective area is in the body of the rod rather than at the root of the threads in the connection. It is therefore rarely feasible to strengthen a deficient rod; hence, correction of the deficiency likely will require removal and replacement with larger rods; removal of existing tension bracing, and replacement with new bracing capable of resisting tension and compression; or installation of additional bracing. When replacing existing tension braces with new braces capable of resisting tension and compression, it is good practice to balance the members (i.e., design the system such that approximately the same number of members act in tension as in compression). Increasing the size of the bracing probably will require strengthening of the existing connection details, and also will be limited by the capacity of the other members of the bracing system or the foundations, as discussed above for ordinary concentric bracing. The effectiveness of replacing the tension bracing with members capable of resisting compression forces depends on the length of the members, and the need for secondary members to reduce the unbraced lengths. Secondary members may interfere with existing window or door openings. The most cost-effective technique for correction of the deficiency probably will be to provide additional bracing, unless functional or other nonstructural considerations (e.g., obstruction of existing window or door openings) preclude the addition of new bracing.

(3) Strengthening techniques for inadequate beam or column capacity.

Deficient axial capacity of the beams or columns of the bracing systems can be improved by:

- Increasing the axial capacity by adding cover plates to or by boxing the existing flanges; and

- Reducing the forces on the existing columns or beams by providing supplemental vertical-resisting components (i.e., braced frames or shear walls).

Reinforcing the existing beams or columns with cover plates or boxing the flanges are generally the most cost-effective alternatives. If supplemental braces or shear walls are required to reduce stresses in other structural components such as the tension rods or the diaphragm, the addition of supplemental vertical- resisting components may be a viable alternative.

f. Diaphragms. Diaphragms are horizontal subsystems that transmit lateral forces to the vertical-resisting elements. Diaphragms typically consist of the floors and roofs of a building. In this document, the term "diaphragm" also includes horizontal bracing systems. There are five principal types of diaphragms: timber diaphragms, concrete diaphragms, precast concrete diaphragms, steel decking diaphragms, and horizontal steel bracing. Inadequate chord capacity is listed as a deficiency for most types of diaphragms. Theoretical studies, testing of diaphragms, and observation of earthquake-caused building damage and failures provide evidence that the commonly used method of determining diaphragm chord force (i.e., comparing the diaphragm to a flanged beam and dividing the diaphragm moment by its depth) may lead to exaggerated chord forces, and thus overemphasize the need for providing an "adequate" boundary chord. Before embarking on the repair of existing chord members or the addition of new ones, the need for such action should be considered carefully, with particular attention to whether the beam analogy is valid for calculating chord forces in the diaphragm under consideration. Since few diaphragms have span-depth ratios such that bending theory is applicable, the capacity of the diaphragm to resist the tensile component of shear stress could be compared with tensile stresses derived from deep beam theory. In analyzing diaphragms by beam theory, chords provided by members outside of the diaphragms, but connected to their edges, may be considered and may satisfy the chord requirement.

(1) Concrete diaphragms.

(a) Deficiencies. The principal deficiencies of monolithic concrete diaphragms (i.e., reinforced concrete or post-tensioned concrete diaphragms) are:

- Inadequate in-plane shear capacity of the concrete diaphragm;

- Inadequate diaphragm chord or collector capacity; and

- Excessive shear or tensile stresses at the diaphragm openings or plan irregularities.

(b) Strengthening techniques for inadequate shear capacity. Deficient in-plane shear capacity of monolithic concrete diaphragms can be improved by:

- Increasing the shear capacity by overlaying the concrete diaphragm with a new reinforced concrete topping slab (FEMA 172, Figure 3.5.2.2); and

- Reducing the shear in the existing concrete diaphragm by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

Concrete diaphragms usually are strengthened with a concrete overlay. This will require removal and replacement of the existing partitions and floor finishes, and will be disruptive to ongoing operations even though the work can be limited to one floor or a portion of a floor at a time. Adding the concrete overlay also will increase the dead weight of the structure; therefore, existing members, connections, and foundations must be checked to ensure that they are capable of resisting these added loads. It may be possible to avoid strengthening a concrete diaphragm by providing additional shear walls or vertical bracing that will reduce the diaphragm shears. This alternative generally is more costly than the overlay, but it may be competitive when it can be restricted to selected areas of the building, and when minimal work is required on the foundations. For shear transfer, new reinforced concrete or masonry shear walls will require dowels grouted in holes drilled in the concrete diaphragms. When the concrete diaphragm is supported on steel framing, shear walls or vertical bracing may be located under a supporting beam. Dowels or other connections for shear walls or bracing may be welded to the steel beam, but it also may be necessary to provide additional shear studs, welded to the steel beam, in holes drilled in the diaphragm slab to facilitate the shear transfer from the concrete slab to the steel beam. When drilling or cutting an existing reinforced concrete slab, care must be taken to avoid damage to the existing reinforcement, unless the result of cutting the reinforcement has been considered, and any required shoring or other necessary measures have been taken. Special care should be exercised to avoid damaging or cutting prestressing tendons. When it is necessary to cut unbonded tendons, in addition to the above precautions, the tendons shall be unloaded at their anchorage prior to being cut.

(c) Strengthening techniques for inadequate in-plane shear transfer and out-of-plane wall anchorage in concrete diaphragms are provided in paragraph 8-3a.

(d) Strengthening techniques for inadequate flexural capacity. Deficient flexural capacity in monolithic concrete diaphragms can be improved by:

- Increasing the flexural capacity by removing the edge of the diaphragm slab and casting a new chord member integral with the slab (FEMA 172, Figure 3.5.2.3);

- Adding a new chord member by providing a new, reinforced concrete or steel member above or below the slab and connecting the new member to the existing slab with drilled and grouted dowels or bolts (similar to FEMA 172, Figure 3.5.4.3); and

- Reducing the existing flexural stresses by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

If the existing concrete slab is supported on steel framing, the most cost-effective means of providing sufficient diaphragm chord capacity is to ensure adequate shear transfer of the diaphragm to the perimeter steel beam by adding drilled and grouted bolts, and to ensure adequate strength and stiffness capacity of the perimeter beam connections. If a new chord is being secured with drilled and grouted anchors to an existing diaphragm containing prestressing strands, drilling must be done very carefully to ensure that strands are not cut. When a portion of an existing diaphragm slab is removed to provide a new diaphragm chord and/or collector member, as well as new dowels for wall anchorage or shear transfer, this technique is recommended only for one-way slabs in the direction parallel to the slab span, because of the potential risk of gravity load failure of the retrofitted portion of the slab. For other conditions, a detail using new concrete above or below the slab is recommended.

(e) Strengthening techniques for inadequate shear or tensile capacity at openings. Deficient shear or tensile stress at diaphragm openings or plan irregularities in monolithic concrete slabs can be improved by:

- Reducing the local stresses by distributing the forces along the diaphragm by means of structural members beneath the slab, and made integral through the use of drilled and grouted bolts (FEMA 172, Figure 3.5.2.4 a);

- Increasing the capacity of the concrete by providing a new concrete topping slab in the vicinity of the opening and reinforcing with trim bars (FEMA 172, Figure 3.5.2.4 b);

- Removing the stress concentration by filling in the diaphragm opening with reinforced concrete as indicated for shear walls (similar to FEMA 172, Figure 3.1.2.2 c); and

- Reducing the shear stresses at the location of the openings by adding supplemental vertical-resisting components (i.e., shear walls or braced frames).

In existing reinforced concrete diaphragms with small openings or low diaphragm shear stress, the existing reinforcement may be adequate. If additional reinforcement is required, new trim bars probably will be the most cost-effective alternative if a new topping slab is required to increase the overall diaphragm shear capacity. Providing new structural steel or reinforced concrete elements requires analysis of the shear and the tensile forces around the opening. The tensile or compressive stresses in the new elements at the opening must be developed by shear forces in the connection to the existing slab. The new elements also must be extended beyond the opening a sufficient distance to transfer the tensile or compressive chord forces back into the existing slab in the same manner. Removing the stress concentration by filling in the opening may be a feasible alternative, provided that the functional requirements for the opening (e.g., stair or elevator shaft or utility trunk) no longer exist or have been relocated.

(2) Poured gypsum diaphragms.

(a) Deficiencies. Poured gypsum diaphragms may be reinforced or unreinforced and have the same deficiencies as cast-in-place concrete diaphragms.

(b) Strengthening techniques for poured gypsum diaphragms. Strengthening techniques for deficiencies in poured gypsum diaphragms are similar to those listed for concrete diaphragms; however, the addition of a new horizontal bracing system may be the most effective strengthening alternative. Poured gypsum has physical properties similar to those of very weak concrete. Tables of allowable structural properties (i.e., shear, bond, etc.) are published in various building codes and engineering manuals. A typical installation is for roof construction using steel joists. Steel bulb tees, welded or clipped to the joists, span over several joists and support rigid board insulation on the tee flanges. Reinforced or unreinforced gypsum is poured on the insulation board to a depth of 2 or 3 inches (50 to 75 mm), embedding the bulbed stems of the tees. While use of the strengthening techniques discussed for reinforced concrete diaphragms (i.e., reinforced overlays, additional chord reinforcement, etc.) is technically feasible, application of these techniques generally is not practical because of the additional weight or low allowable stresses of gypsum. Since dead loads normally constitute a significant portion of the design loads for roof framing members, the addition of several inches (approximately 75 mm) of gypsum for a reinforced overlay probably will overstress the existing light steel framing. Similarly, the low allowable stresses for dowels and bolts will allow strengthening of only marginally deficient diaphragms. For these reasons, gypsum diaphragms found to have significant deficiencies may have to be removed and replaced with steel decking or may be strengthened with a new horizontal bracing system.

(3) Precast concrete diaphragms.

(a) Deficiencies. The principal deficiencies of precast or post-tensioned concrete planks, tees, or cored slabs are:

- Inadequate in-plane shear capacity of the connections between the adjacent units;
- Inadequate diaphragm chord or collector capacity; and
- Excessive in-plane shear stresses at diaphragm openings or plan irregularities.

(b) Strengthening techniques for inadequate connection shear capacity. Deficient in-plane shear capacity of connections between adjacent precast concrete planks, tees, or cored slabs can be improved by:

- Replacing and increasing the capacity of the existing connections by overlaying the existing diaphragm with a new reinforced concrete topping slab (FEMA 172, Figure 3.5.4.2); and

- Reducing the shear forces on the diaphragm by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

The capacity of an existing diaphragm composed of precast concrete elements (i.e., cored slabs, tees, planks, etc.) generally is limited by the capacity of the field connections between the precast elements. It may be possible to modify these connections for a moderate increase in diaphragm capacity; however, it usually is not feasible to develop the full shear capacity of the precast units except with an adequately doweled and complete poured-in-place connection. This usually is very costly. Overlaying the existing precast system with a new reinforced concrete topping is an effective procedure for increasing the shear capacity of the existing diaphragm. Because of the relatively low rigidity of the existing connections, the new topping should be designed to resist the entire design shear. Existing floor diaphragms with precast concrete elements may have a 2- or 3-inch (50 to 75 mm) poured-in-place topping with mesh reinforcement to compensate for the irregularities in precast elements, and such toppings may constitute an adequate diaphragm. Where mechanical connections between units exist along with a topping slab, the topping slab generally will resist the entire load (until it fails) because of the relative rigidities; therefore, the addition of mechanical fasteners generally is ineffective. Applying an additional topping slab over the existing slab may be prohibitive because of the additional gravity and seismic loads that must be resisted by the structure. For the above reasons, the most cost-effective alternative may be reducing the diaphragm shear forces through the addition of supplemental shear walls or braced frames.

(c) Strengthening techniques for inadequate chord or collector capacity. Deficient diaphragm chord capacity of precast concrete planks, tees, or cored slabs can be improved by:

- Providing a new continuous steel member above or below the concrete slab, and connecting the new member to the existing slab with bolts (FEMA 172, Figure 3.5.4.3);

- Removing the edge of the diaphragm and casting a new chord member integral with the slab; and

- Reducing the diaphragm chord forces by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

Providing a new steel chord member generally is the most cost-effective approach to rehabilitating a deficient diaphragm chord for precast concrete elements. When this approach is used, adequate shear transfer between the existing planks or slabs and the new chord member must be provided. Grouting under the new steel chord member may be necessary to accommodate uneven surfaces. Although typically more costly, casting a new chord into the diaphragm may be considered a viable alternative where the projection caused by a new steel chord member is unacceptable for architectural reasons. The second technique may be a feasible option only when the chord is required in the direction parallel to the precast elements. The third technique generally would be viable only if it is being considered to improve other deficient conditions.

(d) Strengthening techniques for excessive shear stresses at openings. Deficient diaphragm shear capacity at diaphragm openings or plan irregularities can be improved by:

- Reducing the local stresses by distributing the forces along the diaphragm by means of steel members beneath the slab, and made integral with the existing slab with drilled and grouted bolts (FEMA 172, Figure 3.5.2.4 a);

- Increasing the capacity by overlaying the existing slab with a new reinforced concrete topping slab with reinforcing trim bars in the vicinity of the opening (FEMA 172, Figure 3.5.2.4 b),

- Removing the stress concentration by filling in the diaphragm opening with reinforced concrete (similar to FEMA 172, Figure 3.5.2.4 c); and

- Reducing the shear stresses at the location of the openings by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

The relative merits for rehabilitating excessive shear stresses at openings in precast concrete planks, tees, or cored slabs are similar to those discussed for cast-in-place concrete diaphragms.

(4) Steel deck diaphragms.

(a) Deficiencies. The principal deficiencies in steel deck diaphragms are inadequate in-plane shear capacity, which may be governed by the capacity of the welding to the supports, or the capacity of the seam welds between the deck units; inadequate diaphragm chord capacity; and excessive in-plane shear stresses at diaphragm openings or plan irregularities.

(b) Strengthening techniques for inadequate shear capacity. Deficient in-plane shear capacity of steel deck diaphragms can be improved by:

- Increasing the steel deck shear capacity by providing additional welding;

- Increasing the deck shear capacity of unfilled steel decks by adding a reinforced concrete fill or overlaying a new topping slab for concrete-filled steel decks;

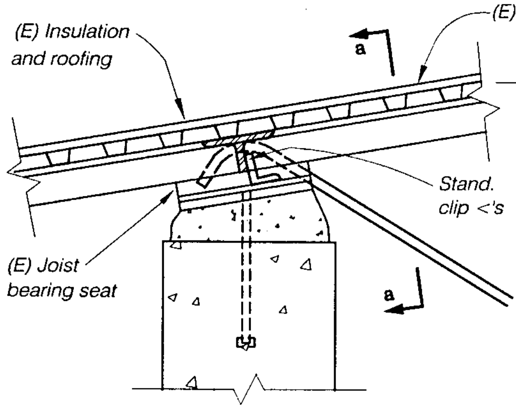
- Increasing the diaphragm shear capacity by providing a new horizontal steel bracing system under the existing diaphragm; and

- Reducing the diaphragm shear stresses by providing supplemental vertical-resisting elements to reduce the diaphragm span.

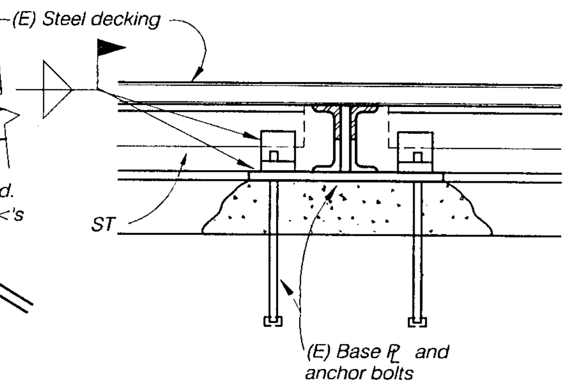
Steel decking, with or without an insulation fill (e.g., vermiculite or perlite), may be used as a diaphragm whose capacity is limited by the welding to the supporting steel framing, and crimping or seam welding of the longitudinal joints of the deck units. The shear capacity of this type of diaphragm may be increased modestly by additional welding if the shear capacity of the existing welds is less than the allowable shear of the steel deck itself. Significant increases in capacity may be obtained by adding a reinforced concrete fill and shear studs welded to the steel framing through the decking. This procedure will require the removal of any insulation fill and the removal and replacement of any partitions and floor or roof finishes. The shear capacity of steel deck diaphragms supported on open-web joists often is limited by the lack of adequate connection from deck to shear wall or other vertical element. The lack of intermediate connectors between joists is common. Frequently, the joist bearing ends themselves are not well connected to transfer diaphragm shear. Addition of supplemental steel members connected to wall and diaphragm is illustrated in Figure 8-12. The capacity of steel decking with an existing reinforced concrete fill may be increased by adding a reinforced concrete overlay. Although this is an expedient alternative for increasing the shear capacity of an existing composite steel deck, providing adequate shear transfer to the vertical-resisting members or chord elements through the existing composite decking may require special details (e.g., additional shear studs). Since the addition of a concrete overlay will increase the dead weight of the structure, the existing members, connections, and foundation must be checked to determine whether they are capable of resisting the added loads. The above alternatives provide positive, direct methods for strengthening an existing steel deck diaphragm. Both alternatives require complete access to the top of the diaphragm, and the removal and replacement of partitions and floor finishes or roofing.

Topping over an existing concrete fill will change the finished floor elevation by several inches, and will therefore require a number of nonstructural adjustments to the new elevation (e.g., to stairs, elevators, floor electrical outlets, etc.).

1. New horizontal bracing. An additional alternative for strengthening steel decking without concrete fill is to add new horizontal bracing under the decking. Since steel decking generally is supported on structural steel framing, the existing framing with new diagonal members forms the horizontal-bracing system. The diaphragm shears are shared with the existing decking in proportion to the relative rigidity of the two systems. This alternative requires access to the underside of the floor or roof framing, and may require relocation of piping, ducts, or electrical conduit, as well as difficult and awkward connections to the existing framing. These costs must be weighed against the costs for a concrete overlay. It should be noted that this alternative may not be feasible for steel decking with a composite concrete fill because of the much greater rigidity of the existing composite diaphragm compared with that of the bracing system. For the bracing system to be effective in this case, the diaphragm shears would be distributed on the basis of the bracing system and the steel decking without the concrete fill (i.e., failure of the concrete fill in shear would be assumed to be acceptable). The new horizontal bracing system will require continuous chord or collector members to receive the bracing forces and transfer them to shear walls or other vertical-resisting elements. A tubular steel member is a preferred section for the new bracing members, as is the tee section for the chord or collector members connected to shear walls. Where existing construction does not permit the use



Detail 1



Section a-a

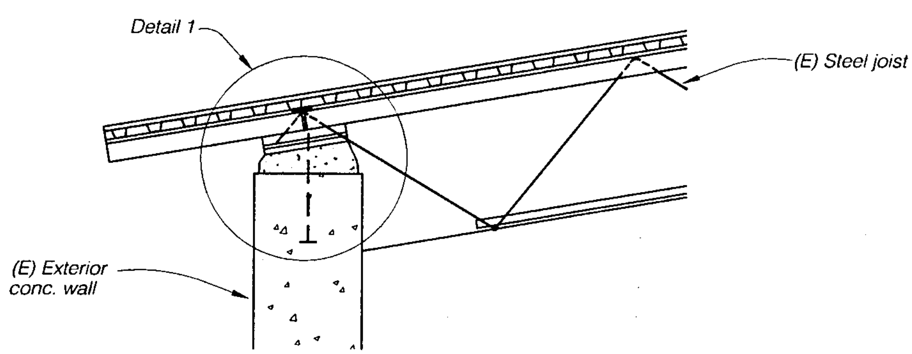


Figure 8-12. Providing Shear Transfer for Steel Decking on Steel Joists at Exterior Wall

of the tee section, an angle may be used. In the latter case, bending of the angle and prying action on the anchor bolts may need to be investigated.

2. Additional shear walls or vertical bracing. Reduction of the existing diaphragm stresses to acceptable levels by providing additional shear walls or vertical bracing also may be a feasible alternative. The choice between shear walls or bracing will depend on compatibility with the existing vertical-resisting elements (i.e., additional shear walls should be considered for an existing shear wall system and additional bracing for an existing bracing system). The appropriateness of this technique (as discussed above) depends on the extent to which new foundations will be required, and potential interference with the functional use of the building.

(c) Strengthening techniques for inadequate chord capacity. Deficient chord capacity of steel-deck diaphragms can be improved by:

- Increasing the chord capacity by providing welded or bolted continuity splices in the perimeter chord steel framing members (Figure 8-13);

- Increasing the chord capacity by providing a new continuous steel member on top or bottom of the diaphragm; and

- Reducing the diaphragm chord stresses by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) such that the diaphragm span is reduced.

Steel decking generally is constructed on steel framing. The perimeter members of the steel framing typically will have sufficient capacity to resist the diaphragm chord stresses, provided the shear capacity of the connections between the decking and the chord member and the tensile capacity of the steel framing connections are adequate to transfer the prescribed loads. Increasing the capacity of these connections by providing additional plug welds to the decking or adding steel shear studs in the case of concrete-filled metal decking may be required. The first technique generally is the most cost-effective. Increasing the chord capacity by providing a new steel chord member to the perimeter of the diaphragm would be appropriate only if it was impractical to use an existing member. If new concrete fill is to be added to increase the shear capacity of the steel decking, the chord requirements can be satisfied by designing reinforcements at the perimeter of the fill to resist the chord forces. Reducing the diaphragm chord stresses by providing supplemental shear walls or braced frames generally would not be cost-effective to correct a chord capacity problem, unless it is being seriously considered to improve other component deficiencies as well.

(d) Strengthening techniques for excessive shear stresses at opening. Excessive shear stresses at diaphragm openings or plan irregularities can be improved by:

- Reducing the local stress concentrations by distributing the forces into the diaphragm by means of steel drag struts (FEMA 172, Figure 2.2.2.4 b);

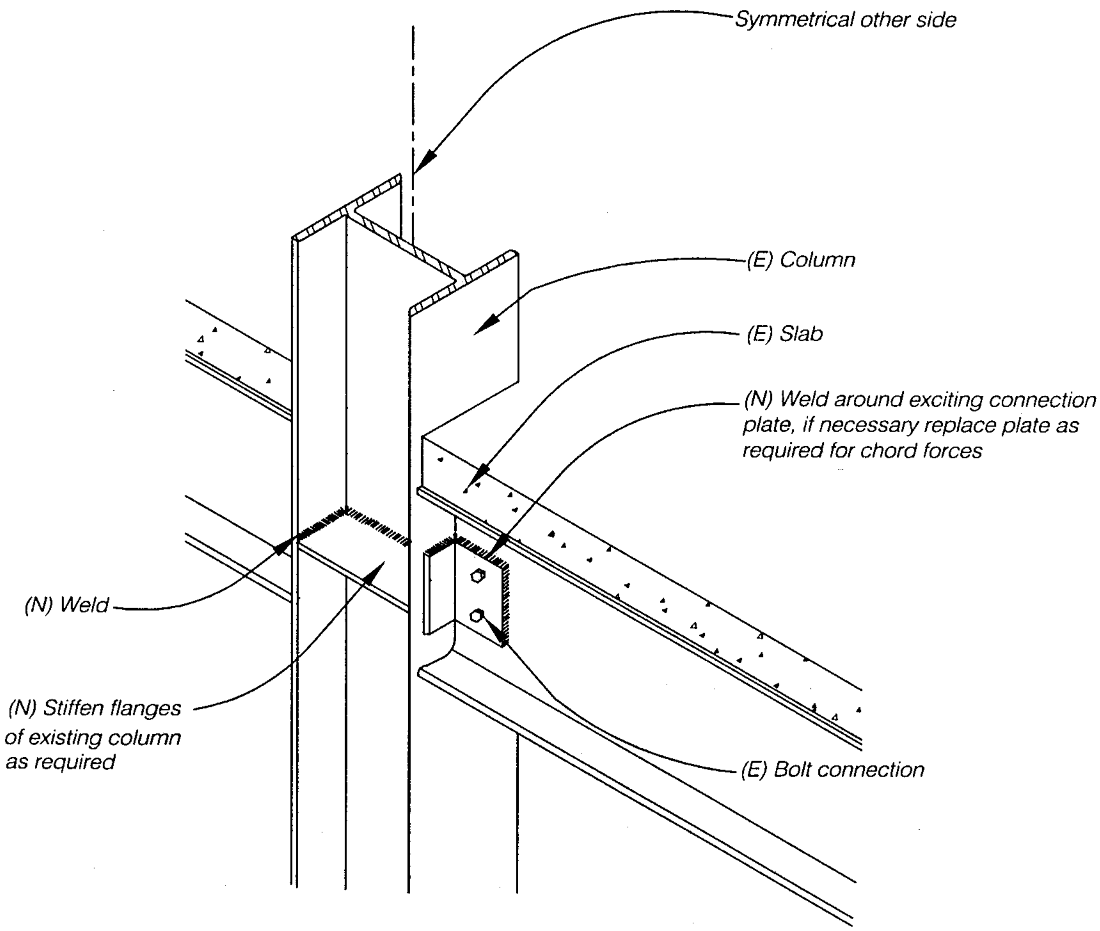


Figure 8-13. Modifying Simple Beam Connection to Provide Chord Tension Capacity

- Increasing the capacity of the diaphragm by reinforcing the edge of the opening with a steel-angle frame welded to the decking (similar to FEMA 172, Figure 3.5.2.4 a); and

- Reducing the diaphragm stresses by providing supplemental vertical-resisting elements (i.e., shear walls, braced frames, or new moment frames) such that the diaphragm span is reduced.

Openings and plan irregularities in steel deck diaphragms generally are supported along the perimeter by steel beams designed to support the gravity loads. If continuous past the corners of the openings or irregularities, these beams can distribute the concentrated stresses into the diaphragm, provided the capacity of the connections between the decking and the steel beams is adequate to transfer the prescribed loads. If inadequate, the connections can be reinforced by adding plug welds or shear studs. If beams are not continuous beyond an opening or irregularity, new beams can be provided to act as drag struts. Adequate connection of the new beams to the diaphragm and to the existing beams will be required to distribute loads.

Correcting the diaphragm deficiency by providing a steel frame around the perimeter of the opening or along the sides of the irregularity is similar to providing drag struts. The connection between the new steel members and the diaphragm must be sufficient to adequately distribute the local stresses into the diaphragm. The dimensions of the opening or irregularity will dictate whether this can be achieved solely with the use of a perimeter steel frame. Reducing the diaphragm stresses by providing supplemental shear walls or braced frames generally would not be cost-effective to correct a diaphragm opening deficiency unless it also was being considered to improve other component deficiencies.

(5) Timber diaphragms.

(a) Deficiencies. Timber diaphragms can be composed of straight-laid or diagonal sheathing or plywood. The principal deficiencies in the seismic capacities of timber diaphragms are:

- Inadequate shear capacity of the diaphragm;

- Inadequate chord capacity of the diaphragm;

- Excessive shear stresses at diaphragm openings or at plan irregularities; and

- Inadequate stiffness of the diaphragm resulting in excessive diaphragm deformations.

(b) Strengthening techniques for inadequate shear capacity. Deficient shear capacity of existing timber diaphragms can be improved by:

- Increasing the capacity of the existing timber diaphragm by providing additional nails or staples with due regard for wood-splitting problems;

- Increasing the capacity of the existing timber diaphragm by means of a new plywood overlay (Figure 8-14); and

- Reducing the diaphragm span through the addition of supplemental vertical-resisting elements (i.e., shear wall or braced frames).

Adding nails and applying a plywood overlay requires removal and replacement of the existing floor or roof finishes, as well as removal of existing partitioning, but is generally less expensive than adding new walls or vertical bracing. If the existing system consists of straight-laid or diagonal sheathing, the most effective alternative is to add a new layer of plywood, since additional nailing of the existing diaphragm typically is not feasible because of limited spacing and edge distance. Additional nailing is usually the least expensive alternative, but the additional capacity is still limited to the number and capacity of the additional nails that can be driven (i.e., with minimum allowable end distance, edge distance, and spacing). The additional capacity that can be developed by plywood overlays usually depends on the capacity of the underlying boards or plywood sheets to develop the capacity of the nails from the new overlay. Higher shear values are allowed for plywood overlay when adequate nailing and blocking (i.e., members with at least 2 inches [50 mm] of nominal thickness) can be provided at all edges where the plywood sheets abut. Adequate additional capacity for most timber diaphragms can be developed using this technique unless unusually large shears need to be resisted. When nailing into existing boards, care must be taken to avoid splitting. If boards are prone to splitting, pre-drilling may be necessary. The addition of shear walls or vertical bracing in the interior of a building may be an economical alternative to strengthening the diaphragms, particularly if the additional elements can be added without the need to strengthen the existing foundation. When additional bracing or interior shear walls are required, relative economy depends on the degree to which ongoing operations can be isolated by dust and noise barriers, and on the need for additional foundations.

(c) Strengthening techniques for inadequate chord capacity. Deficient diaphragm chord capacity can be improved by:

- Providing adequately nailed or bolted continuity splices along joists or fascia parallel to the chord (Figure 8-15);

- Providing a new continuous steel chord member along the top of the diaphragm (Figure 8-16); and

- Reducing the stresses on the existing chords by reducing the diaphragm's span through the addition of new shear walls or braced frames.

Simplified calculations to determine stresses in diaphragm chords conservatively consider the diaphragm as a horizontal beam and ignore the flexural capacity of the web of the diaphragm, as well as the effect of the perimeter shear walls that reduce the chord stresses. However, even though the chord requirements in some buildings may be overstated, in most buildings, a continuous structural element is required at diaphragm boundaries to collect the diaphragm shears and transfer them to the individual

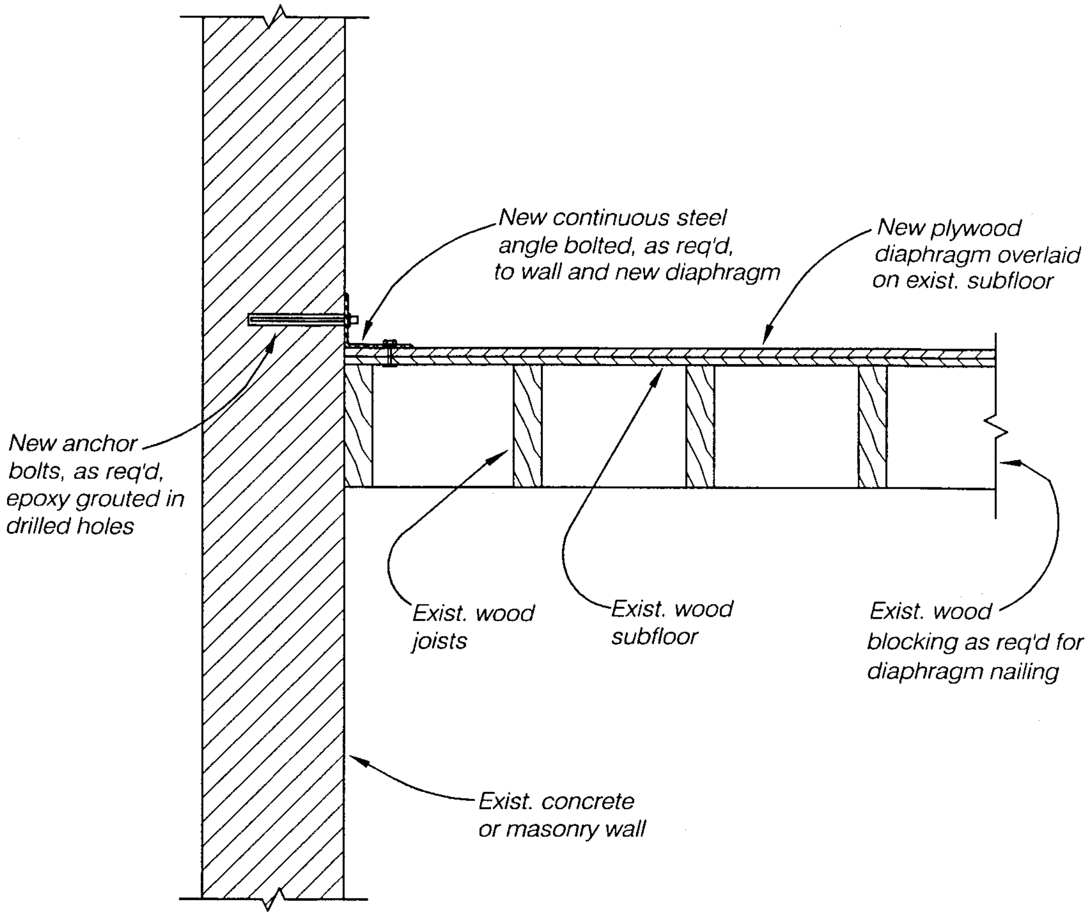


Figure 8-14. Strengthening of a Timber Diaphragm

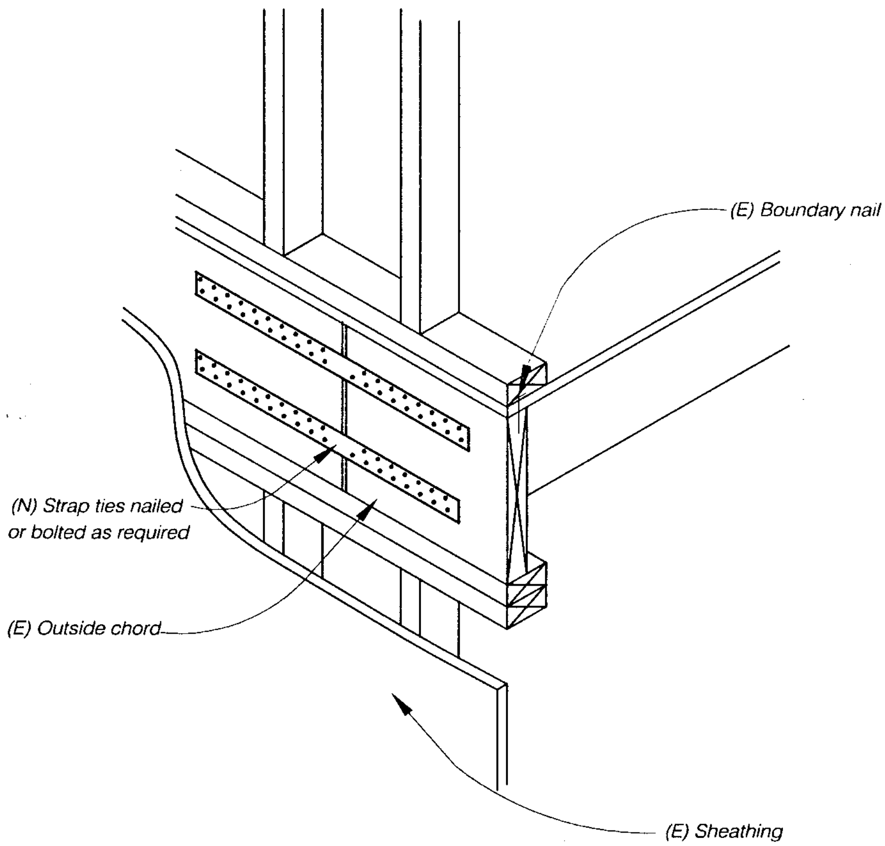


Figure 8-15. Chord Splice for Wood Diaphragm

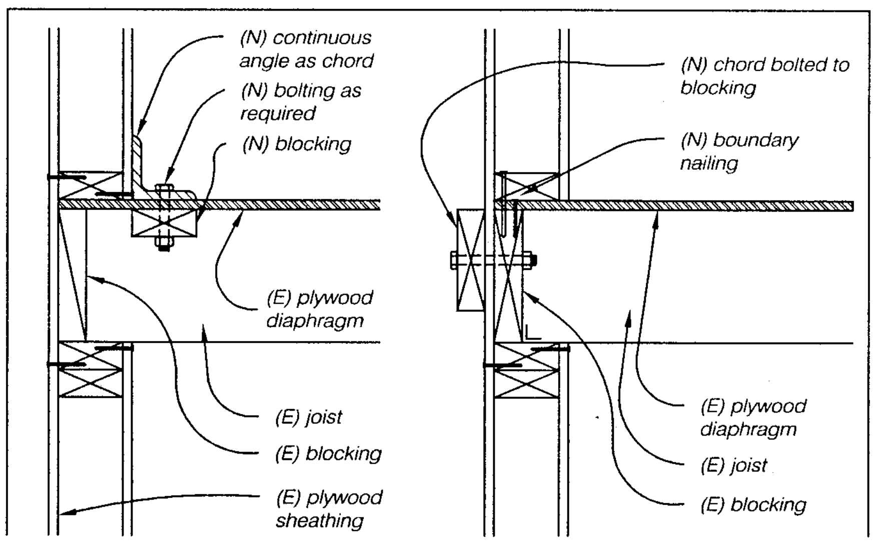


Figure 8-16. Providing New Chords for Wood Diaphragms

resisting shear walls along each boundary. A continuous steel member along the top of the diaphragm may be provided to function as a chord or collector member. For existing timber diaphragms at masonry or concrete walls, the new steel members may be used to provide wall anchorage, or as a chord or collector member for the diaphragm shear forces. The lack of adequate chord capacity is seldom the reason why new shear walls or braced frames would be considered to reduce the diaphragm loads. Reducing the diaphragm span and loads through the introduction of new vertical-resisting elements, however, may be considered to address other member deficiencies, and if so, the chord inadequacy problem may also be resolved.

(d) Strengthening techniques for excessive shear stresses at openings or plan irregularities.

Excessive shear stresses at diaphragm openings or other plan irregularities can be improved by:

- Reducing the local stresses by distributing the forces along the diaphragm by means of drag struts (FEMA 172, Figure 2.2.2.4 b);

- Increasing the capacity of the diaphragm by overlaying the existing diaphragm with plywood, and appropriate nailing of the plywood through the sheathing at the perimeter of the sheets adjacent to the opening or irregularity; and

- Reducing the diaphragm stresses by reducing the diaphragm spans through the addition of supplemental shear walls or braced frames.

The most cost-effective way to reduce large local stresses at diaphragm openings or plan irregularities is to install drag struts to distribute the forces into the diaphragm. Proper nailing of the diaphragm into the drag struts is required to ensure adequate distribution of forces. Local removal of roof or floor covering will be required to provide access for nailing. The analysis for the design of the drag strut and the required additional nailing is similar to that for the reinforcement of an opening in the web of a steel plate girder. The opening divides the diaphragm into two parallel horizontal beams, and the shear in each beam causes moment that induces tension or compression in the outer fibers of each beam. For small-opening or low-diaphragm shears, these bending forces may be adequately resisted as additional stresses in an existing diaphragm. For larger openings and/or larger diaphragms, tension or compression "flanges" may have to be developed at the opening. In a timber diaphragm, these "flanges" may be assumed to be the joists or headers that frame the opening, but to preclude distress due to stress concentration at the corners, the joists or headers must be continuous beyond the edge of the opening in order to transfer the flange forces back into the diaphragm by additional nailing. Applying a plywood overlay to increase the local diaphragm capacity, or providing supplemental vertical-resisting elements to reduce the local stresses generally will be viable alternatives only if they are being considered to correct other structural deficiencies.

(e) Strengthening techniques for inadequate stiffness. Excessive seismic displacement of an existing timber diaphragm can be prevented by:

- Increasing the stiffness of the diaphragm by the addition of a new plywood overlay; and

- Reducing the diaphragm span, thus reducing the displacements by providing new supplemental vertical-resisting elements such as shear walls or braced frames.

The addition of new shear walls or braced frames may be the most cost-effective alternative for reducing excessive displacements of plywood diaphragms (as is also the case for reducing excessive shear stresses as discussed above) if the additional elements can be added without strengthening the existing foundations, and when the existing functional use of the building permits it. The spacing of new vertical elements required to limit the deflection of straight or diagonal sheathing to prescribed limits may be too close to be feasible. In these cases, overlaying with plywood may be the most cost-effective alternative. It should be noted that the Special Procedure for URM bearing wall buildings identifies flexibility as the primary diaphragm deficiency, and special "cross walls" are prescribed rather than diaphragm strengthening to reduce deflections.

(6) Horizontal steel bracing. Existing horizontal steel bracing systems may be in the plane of the roof or floor framing (e.g., rod tension bracing or light angles using some of the framing members as chords or compression sheets, or in the case of existing roof trusses, existing bracing may occur to provide lateral support for the lower chord. New bracing may be installed in a similar manner, but for some existing systems, such as open-web joist framing, it is usually easier to install the new bracing below the lower chord of the joists. In any event, for either new or existing bracing to resist seismic forces, there must be a positive and direct path to transfer the floor or roof inertia forces to the bracing, and from the bracing to the walls or other vertical-resisting elements.

(a) Deficiency. The principal deficiency in horizontal steel bracing systems is inadequate force capacity of the members (i.e., bracing and floor or roof beams) and/or the connections.

(b) Strengthening techniques for inadequate bracing systems. Deficient horizontal steel bracing system capacity can be improved by:

- Increasing the capacity of the existing bracing members, or removing and replacing them with new members and connections of greater capacity;

- Increasing the capacity of the bracing system by adding new horizontal bracing members to previously unbraced panels (if feasible);

- Increasing the capacity of the bracing system by adding a steel deck diaphragm to the floor system above the steel bracing; and

- Reducing the stresses in the horizontal bracing system by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

Horizontal bracing systems to resist wind or earthquake forces have been in common use for many years in steel-framed industrial buildings. These bracing systems generally are integrated with the existing floor or roof framing systems, and the capacity of the bracing system generally is governed by the diagonal braces and their connections. If the structural analysis indicates that the existing floor or roof-framing members in the bracing systems do not have adequate capacity for the seismic loads, providing additional bracing or other lateral-load-resisting elements may be a cost-effective alternative to strengthening these members. Simple strengthening techniques include increasing the capacity of the existing braces and their connections (e.g., single-angle bracing could be doubled, double-angle bracing could be "starred"[i.e., two pairs of angles back-to-back]) as well as removing existing braces and replacing them with stronger braces and connections. The existing connections must be investigated, and if found to be inadequate, the connections will need to be strengthened. Providing horizontal braces in adjacent unbraced panels, if present, may be a very cost-effective approach to increasing the horizontal load capacity. Existing horizontal bracing systems often do not have an effective floor diaphragm, and a new floor or roof diaphragm consisting of a reinforced concrete slab or steel decking with or without concrete fill can be provided to augment or replace the horizontal bracing systems. A steel deck diaphragm may be designed to augment the horizontal bracing, but a concrete slab probably would make the bracing ineffective because of the large difference in rigidities. The concrete slab therefore would need to be designed to withstand the entire lateral load. As with other diaphragms, it may be possible to reduce diaphragm stresses to acceptable limits by providing additional shear walls or vertical bracing. Unlike true diaphragm systems, however, a horizontal bracing system may not have been designed with the same shear capacity at any section (e.g., a simple bracing system between two end walls may have increasing shear capacity from the center towards each end). In some cases, additional vertical-resisting elements can increase the stresses in some of the elements of the existing bracing systems.

g. *Foundations*. Deficient foundations occasionally are a cause for concern with respect to the seismic capacity of existing buildings. Because the foundation loads associated with seismic forces are transitory and of very short duration, allowable soil stresses for these loads, combined with the normal gravity loads, may be permitted to approach ultimate stress levels. Where preliminary analysis indicates that there may be significant foundation problems, recommendations from a qualified geotechnical engineer should be requested to establish rational criteria for the foundation analysis.

(1) Continuous or strip footings.

(a) Deficiencies. The principal deficiencies in the seismic capacity of existing continuous or strip wall footings are:

- Excessive soil-bearing pressure due to overturning forces; and

- Excessive uplift conditions due to overturning forces.

(b) Strengthening techniques for excessive soil-bearing pressure. The problem of excessive soil-bearing pressure caused by seismic overturning forces can be mitigated by:

- Decreasing the soil-bearing pressure by underpinning and enlarging the footing at each end (FEMA 172, Figure 3.6.1.2a);

- Increasing the vertical capacity of the footing by adding new drilled piers adjacent and connected to the existing footing (FEMA 172, Figure 3.6.1.2 b);

- Increasing the soil-bearing capacity by modifying the existing soil properties; and

- Reducing the overturning forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames).

The most effective procedure for correcting excessive soil pressure due to seismic overturning forces is to provide a drilled pier on each side and at each end of the wall. The reinforced concrete piers should be cast-in-place in uncased holes so as to develop both tension and compression. Each pier should extend above the bottom of the footing and be connected by a reinforced concrete "needle" beam through the existing wall above the footing. The above techniques are costly and disruptive. For this reason, when seismic upgrading results in increased forces that require foundation strengthening, it may be cost-effective to consider other seismic upgrading schemes. Soil conditions may be such that modifying the capacity of existing soils is the most viable alternative. The soil beneath structures founded on clean sand can be strengthened through the injection of chemical grouts. The bearing capacity of other types of soils can be strengthened by compaction grouting. With chemical grouting, chemical grout is injected into clean sand in a regular pattern beneath the foundation. The grout mixes with the sand to form a composite material with a significantly higher bearing capacity. With compaction grouting, grout also is injected in a regular pattern beneath the foundation, but it displaces the soil away from the pockets of injected grout rather than dispersing into the soil. The result of the soil displacement is a densification of the soil, and hence, increased bearing capacity. Some disruption of existing floors adjacent to the subject foundations may be required in order to cut holes needed for uniform grout injection. Alternatively, seismic forces on the footing can be reduced by adding other vertical-resisting components such as bracing, shear walls, or buttresses.

(c) Strengthening techniques for excessive uplift conditions. Deficient capacity of existing foundations to resist prescribed uplift forces caused by seismic overturning moments can be improved by:

- Increasing the uplift capacity of the existing footing by adding drilled piers or soil anchors (FEMA 172, Figure 3.6.1.2b); and

- Reducing the uplift forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames).

Any seismic rehabilitation alternative that requires significant foundation work will be costly. Access for heavy equipment (e.g., drilling rigs, backhoes, and pile drivers), ease of material handling, and the need to minimize the disruption of the functional use of the building are a few of the reasons why exterior foundation rehabilitation work will be significantly less costly than interior work. Providing a significant increase in the uplift capacity of an existing foundation generally is most effectively achieved by adding drilled piers or soil anchors. Reinforced concrete piers can be provided adjacent to the footing and connected to the existing footing with steel or concrete beams. Locating the piers symmetrically on both sides of the footing will minimize connections that must transfer eccentric loads. The details for eccentric connections may not always be feasible; however, providing concentric drilled piers almost ensures that interior foundation work will be needed. Soil anchors similar to those used to tie-back retaining walls also can be used instead of drilled piers. Hollow core drill bits from 4 inches to 2 feet (100 mm to 0.6 m) in diameter can be used to drill the needed deep holes. After drilling, a deformed steel tension rod is placed into the hole through the center of the bit. As the bit is withdrawn, cement grout is pumped through the stem of the bit, bonding to the tension rod and the soil. These types of soil anchors can provide significant tensile capacity. Drilling rigs are available that can drill in the interior of buildings even with low headroom; however, this is more costly. As with other rehabilitation techniques, reducing the overturning forces by providing additional vertical-resisting components such as braced frames, shear walls, or buttresses may be viable. The addition of buttresses may transfer loads to the exterior of the building, where foundation work may not be so costly. Some engineers believe that uplifting of the ends of rigid shear walls is not a deficiency, and may actually be beneficial in providing a limit to the seismic base shear. Others design the structure for the overturning forces but ignore the tendency of the foundation to uplift. If the foundations are permitted to uplift, the engineer must investigate the redistribution of forces in the wall and in the soil due to the shift in the resultant soil pressure, and also the potential distortion of structural and nonstructural elements framing into the wall.

(2) Individual pier or column footings.

(a) Deficiencies. The principal deficiencies in the seismic capacity of existing individual pier or column footings are:

- Excessive soil-bearing pressure due to overturning forces;
- Excessive uplift conditions due to overturning forces; and
- Inadequate friction and passive soil pressure to resist lateral loads.

(b) Strengthening techniques for excessive bearing pressure. The problem of excessive soil-bearing pressure due to overturning forces can be mitigated by:

- Increasing the bearing capacity of the footing by underpinning the footing ends and providing additional footing area (FEMA 172, Figure 3.6.1.2a);

- Increasing the vertical capacity of the footing by adding new piers drilled through the existing footing (Figure 8-17);

- Reducing the bearing pressure on the existing footings by connecting adjacent footings with deep reinforced concrete tie beams;

- Increasing the soil-bearing capacity by modifying the existing soil properties; and

- Reducing the overturning forces by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

The considerations in selecting alternatives to correcting excessive soil-bearing pressure due to overturning forces in individual pier or column footings are similar to those discussed above for continuous or strip footings. Underpinning existing footings to increase the bearing area is an ancient technique that is still employed because of its simplicity. The end result is brick or concrete underpinning under the existing footing. The new bearing area is increased by extending the underpinning down and out at 45 degrees from the bottom edge of the footing. The work is generally done progressively in quadrants or smaller sections, and preloaded by jacking to minimize settlement (FEMA 172, Figure 3.6.1.2a). The second alternative presumes that the existing footing is large enough to accommodate four drilled piers of about 1 foot (0.3 m) in diameter. The third alternative of tying adjacent footings together with a deep reinforced concrete beam may be a feasible means of distributing the forces resulting from the overturning moment to adjacent footings.

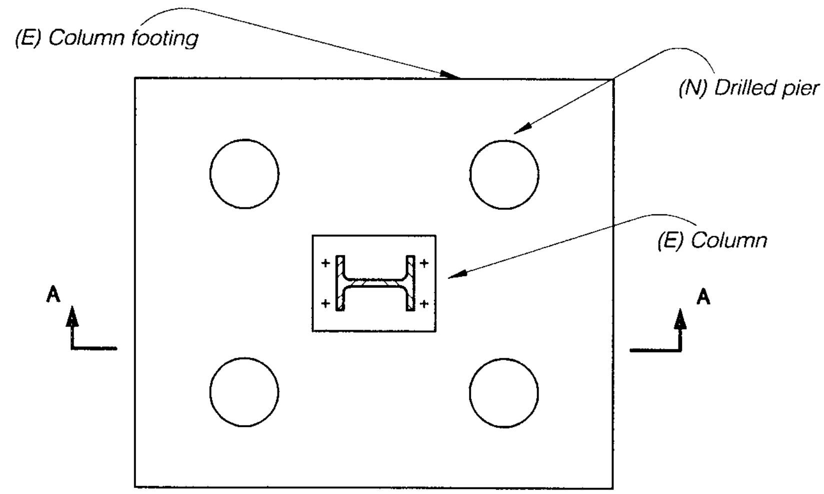
(c) Strengthening techniques for excessive uplift conditions. Deficient capacity of existing foundations to resist the prescribed uplift forces caused by seismic overturning moments can be improved by:

- Increasing the uplift capacity of the existing footing by adding drilled piers or soil anchors (similar to Figure 8-17);

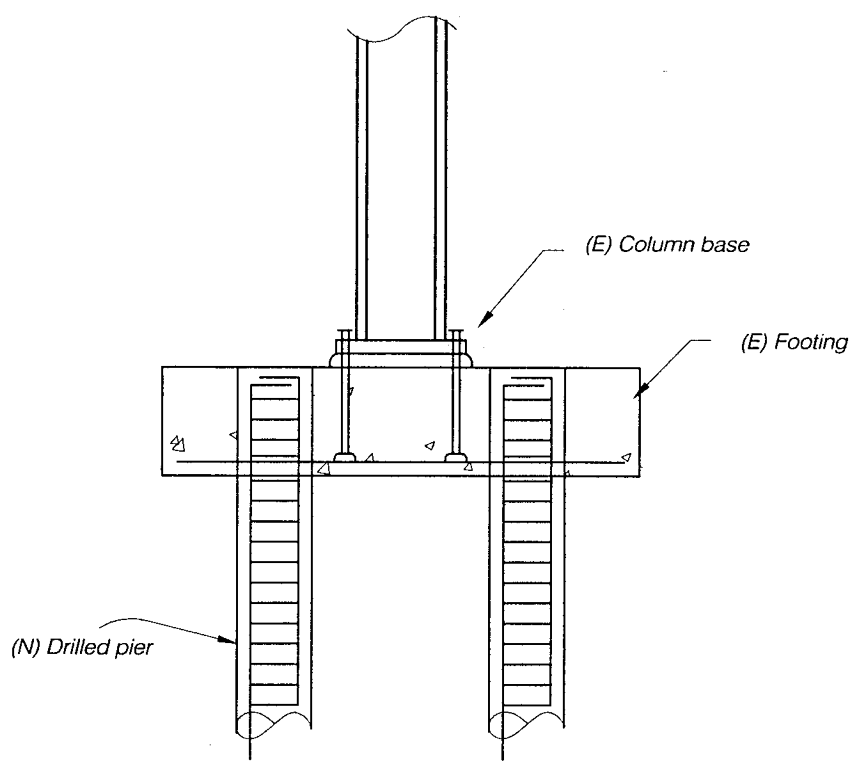
- Increasing the uplift capacity by providing a new deep reinforced concrete beam to mobilize the dead load on an adjacent footing; and

- Reducing the uplift forces by providing supplemental vertical-resisting components (i.e., shear walls or braced frames).

The first technique is similar to the second technique described in the previous paragraph to reduce excessive bearing pressure. The drilled piers can be designed to provide additional bearing and uplift capacity. For uplift capacity, a reinforced concrete overlay may be required to resist the flexural stresses in the footing. If the drilled piers are for uplift only, the diameter may be smaller (i.e., 4 to 6 inches [100 to 150 mm]) if a post-tensioned soil anchor is used for the uplift resistance. The second technique is also similar to the third technique in the previous paragraph, and is used here as a feasible means for:



Plan



Section A-A

Figure 8-17. Strengthening an Existing Spread Footing

mobilizing the existing mass supported by an adjacent footing.

(d) Strengthening techniques for inadequate passive pressure. The problem of excessive passive soil pressure caused by seismic loads can be mitigated by:

- Providing an increase in vertical bearing area by enlarging the footing;

- Providing an increase in vertical bearing area by adding new tie beams between existing footings;

- Improving the existing soil conditions adjacent to the footing to increase the allowable passive pressure; and

- Reducing the bearing pressure at overstressed locations by providing supplemental vertical-resisting components such as shear walls or braced frames at selected locations.

As noted above, foundation rework generally is relatively costly. The foundation strengthening technique that is the most cost-effective generally is the technique that can resolve more than one concern. The addition of a new deep tie beam between adjacent footings, if required to resist overturning forces, will likely address inadequate passive soil pressure concerns. As the above discussion indicates, the most cost-effective alternative to the strengthening of an existing foundation usually is not readily apparent. Several alternative schemes may have to be developed to the point where reasonable cost estimates can be made to evaluate the tangible costs (i.e., the total actual work that needs to be accomplished), as well as the architectural considerations and the disruption or relocation of an on-going function. The third alternative provides the same results as enlarging the footing and can be very cost-effective if the foundations are accessible for in-situ strengthening of the soil (e.g., construction of vane-mixed soil/cement piers adjacent to the footing. As indicated above, the second alternative will distribute loads between foundation elements, as well as provide additional surface area to mobilize passive pressure. In specific situations, the other alternatives may be more cost-effective, depending upon accessibility, as well as the impact each alternative may have on the ongoing functional use of the building.

(3) Piles or drilled piers.

(a) Deficiencies. The two principal deficiencies in the seismic capacity of piles or drilled piers are:

- Excessive tensile or compressive loads on the piles or piers due to seismic forces combined with gravity loads;

- Inadequate capacity to transfer tensile forces to the pile or pier cap; and

- Inadequate lateral-force capacity to transfer the seismic shears to the soil.

(b) Strengthening techniques for excessive vertical force. The deficient tensile or compression capacity of piles or piers can be improved by one or more of the following techniques:

- Increasing the capacity of the foundation by removing the existing pile cap, driving additional piles, and providing new pile caps of larger sizes (FEMA 172, Figure 3.6.3.2); and

- Reducing the load on overstressed piles by distributing the seismic forces to adjacent pile caps with deep tie beams.

Although it may be possible to drive additional piles to correct the deficiency, it is usually very difficult to utilize the existing pile cap to distribute the loads effectively to both old and new piles. It may be necessary to consider temporary shoring of the column, or other structural members supported by the pile caps, or that the pile caps can be removed and replaced with a new pile cap that will include the new piles. The use of deep tie beams to distribute seismic overstressing forces is similar to that discussed above for spread footings.

(c) Strengthening techniques for excessive lateral forces. The deficient lateral-force capacity of piles or piers can be improved by one or more of the following:

- Reducing the loads on overstressed pile caps by adding tie beams to distribute the loads to adjacent pile caps;

- Increasing the allowable soil pressure adjacent to the pile cap by improving the soil;

- Increasing the capacity of the foundation by removing the existing pile cap, driving additional piles, and providing a pile cap of larger size (FEMA 172, Figure 3.6.3.2); and

- Reducing the load on the piles or piers by providing supplemental vertical-resisting components (i.e., braced frames or shear walls) to transfer the forces to other foundation elements with reserve capacity.

Damage to concrete piles or piers (particularly that resulting from shear fracture) is unacceptable and should be avoided. Transfer of seismic shear forces to the soil at the pile cap level, rather than by the piles or piers, is preferable. Thus, the first two alternatives, which are similar to those described above for spread footings, are also preferred for pile caps.

(4) Mat foundations.

(a) Deficiencies. Seismic deficiencies in mat foundations are not common; however, the following deficiencies can occur:

- Inadequate moment capacity to resist combined gravity plus seismic overturning forces;

- Inadequate passive soil pressure to resist sliding; and

- Inadequate capacity to resist hydrostatic uplift pressure due to groundwater.

(b) Strengthening techniques for inadequate moment capacity. Deficient mat foundation moment capacity due to concentrated loads can be corrected by increasing the mat capacity locally by providing additional reinforced concrete (i.e., an inverted column capital) doweled and bonded to the existing mat to act as a monolithic section. If the inadequacy is due to concentrated seismic overturning loads, it may be possible to provide new shear walls on the mat to distribute the overturning loads, and also to locally increase the section modulus of the mat.

(c) Strengthening technique for inadequate lateral resistance. Deficient mat foundation lateral resistance (e.g., the possibility of a mat sliding when founded at shallow depth in the soil) can be corrected by the construction of properly spaced shear keys at the mat perimeter. The shear keys would be constructed by trenching around the perimeter of the mat to provide concrete buttresses with a base extending below the bottom of the mat.

(d) Strengthening technique for excessive hydrostatic pressure. Excessive hydrostatic pressure can be resisted by providing internal soil anchors for the mat. This can be accomplished by drilling and casing holes through the mat and into the soil below. A high-strength steel rod is placed in the hole and anchored by grouting in the soil below the casing. After post-tensioning, the rod is grouted in the casing and anchored to a bearing plate on the mat. If the groundwater is seasonal, the technique can be implemented during the dry season, when the groundwater is below mat level. If the groundwater is not seasonal, it would need to be lowered temporarily with well points to permit drilling through the mat.

8-3. Rehabilitation Techniques for Connections

a. Diaphragm connections. Seismic inertial forces originate in all elements of buildings and are delivered through structural connections to horizontal diaphragms. The diaphragms distribute these forces to vertical components that transfer the forces to the foundation. An adequate connection between the diaphragm and the vertical components is essential to the satisfactory performance of any structure. The connections must be capable of transferring the in-plane shear stress from the diaphragms to the vertical elements, and of providing support for out-of-plane forces on the vertical elements. The following types of diaphragms are discussed below: concrete, precast concrete, steel deck without concrete fill, steel deck with concrete fill, and timber.

(1) Connections of concrete diaphragms.

(a) Deficiencies. The principal deficiencies of the connections of concrete diaphragms to vertical-resisting elements such as shear walls or braced frames are:

- Inadequate in-plane shear transfer capacity; and
- Inadequate anchorage capacity for out-of-plane forces in the connecting walls.

(b) Strengthening techniques for in-plane shear wall connections. Deficient in-plane shear transfer capacity of a diaphragm to a shear wall or braced frame can be improved by:

- Reducing the local stresses at the diaphragm-to-wall interface by providing collector members or drag struts under the diaphragm, and connecting them to the diaphragm and the wall (FEMA 172, Figure 3.5.4.3); and

- Reducing the shear stresses in the existing connection by providing supplemental vertical-resisting elements.

Inadequate in-plane shear capacity of connections between concrete diaphragms and vertical-resisting elements usually occurs where large openings in the diaphragm exist adjacent to the shear wall (e.g., at stairwells or an exterior wall with discontinuous shear piers between full-height window openings) or where the shear force distributed to interior shear walls or braced frames exceeds the capacity of the connection to the diaphragm. If the walls and the diaphragm have sufficient capacity to resist the prescribed loads, the addition of collector members is likely to be the most cost-effective alternative. As previously discussed, reducing the forces in the deficient connection by providing supplemental vertical-resisting components is not likely to be the most cost-effective alternative (due to the probable need for new foundations and drag struts) unless it is being considered to correct other component deficiencies.

(c) Strengthening techniques for out-of-plane anchorage capacity. Deficient out-of-plane anchorage capacity of concrete diaphragm connections to concrete or masonry walls can be improved using one or both of the following techniques:

- Increasing the capacity of the connection by providing additional dowels grouted into drilled holes; and/or

- Increasing the capacity of the connection by providing a new member above or below the slab connected to the slab and the wall with drilled and grouted bolts similar to that indicated for providing a new diaphragm chord (FEMA 172, Figure 3.5.4.3).

The most cost-effective alternative generally is to provide additional dowels grouted into drilled holes. The holes are most efficiently drilled from the exterior through the wall and into the slab. Access to the exterior face of the wall is obviously required. When the exterior face is not accessible (e.g., when it abuts an adjacent building), providing a new member connected to the existing wall and slab is likely to be preferred.

(2) Connections of poured gypsum diaphragms.

(a) Deficiencies. The principal deficiencies of poured gypsum diaphragms are similar to those for concrete diaphragms:

- Inadequate in-plane shear transfer capacity; and

- Inadequate anchorage capacity for out-of-plane forces in the connecting walls.

(b) Strengthening techniques for poured gypsum diaphragms. If the gypsum diaphragm is supported by the shear wall, it will be possible to improve the in-plane shear transfer by providing new dowels from the diaphragm into the shear wall. Alternative strengthening techniques for the deficiencies also include removal of the gypsum diaphragm and replacement with steel decking, or the addition of a new horizontal bracing system designed to resist all of the seismic forces. Allowable structural stresses for gypsum are very low, and the additional strengthening that can be achieved is very limited. Further, the typical framing details (e.g., steel joist, bulb tee, and insulation board) are such that it is difficult to make direct and effective connections to the gypsum slab. For these reasons, the techniques involving removal and replacement, or a new horizontal bracing system, are likely to be the most cost-effective solutions, except when the existing diaphragm is only marginally deficient.

(3) Connections of precast concrete diaphragms.

(a) Deficiencies. The principal deficiencies of the connections of precast concrete diaphragms to the vertical-resisting elements are:

- Inadequate in-plane shear transfer capacity; and

- Inadequate anchorage capacity at the exterior walls for out-of-plane forces.

(b) Strengthening techniques for precast concrete diaphragm connections. Deficient shear transfer or anchorage capacity of a connection of a precast concrete diaphragm to a concrete or masonry wall or a steel frame can be improved by:

- Increasing the capacity of the connection by providing additional dowels placed in drilled and grouted holes;

- Increasing the capacity of the connection by providing a reinforced concrete overlay that is bonded to the precast units and anchored to the wall with additional dowels placed in drilled and grouted holes;

- Providing a supplemental connection element, such as a steel angle, bolted to the diaphragm and the wall or welded to the steel frame; and

- Reducing the forces at the connections by providing supplemental vertical-resisting components.

Precast concrete plank or tee floors that have inadequate connection capacity for transferring in-plane shear to vertical elements such as shear walls or braced frames can be strengthened by drilling intermittent holes in the precast units at the vertical element. When the floors are supported on steel framing, welded inserts (or studs) can be added and the holes grouted. When the floors are supported on concrete or masonry units, dowels can be inserted and grouted into the drilled holes. If the diaphragm contains prestressing strands, extreme care must be taken prior to drilling to avoid cutting the strands. A more costly alternative is to provide a reinforced concrete overlay that is bonded to the precast units, and additional dowels grouted into holes drilled in the wall. This will require the stripping of the existing floor surface and raising the floor level by 2 to 3 inches, which will necessitate adjusting of nonstructural elements to the new floor elevations (e.g., stairs, doors, electrical outlets, etc.). Providing a supplemental steel connection element (similar to Figure 3.5.4.3 in FEMA 172) may be a cost-effective alternative that can provide in-plane and out-of-plane additional connection capacity. As previously discussed, reducing the shear forces in the deficient connection by providing supplemental vertical-resisting components is not likely to be the most cost-effective alternative (due to the probable need of new foundations and drag struts) unless it is being considered to correct other component deficiencies. This alternative also is not effective in reducing the out-of-plane forces unless the new vertical-resisting elements can be constructed so as to form effective buttresses for the existing walls.

(4) Connections of steel deck diaphragms without concrete fill.

(a) Deficiencies. For steel deck diaphragms without concrete fill, the principal deficiencies of their connections to the vertical-resisting elements such as shear walls, braced frames, or moment frames are:

- Inadequate in-plane shear capacity; and

- Inadequate anchorage capacity for out-of-plane forces in walls.

(b) Strengthening techniques for steel deck connections. Deficient shear transfer or anchorage capacity of a connection of a steel deck diaphragm to a shear wall, braced frame, or moment frame can be improved by:

- Increasing the capacity of the connection by providing additional welding at the vertical element;

- Increasing the capacity of the connection by providing additional anchor bolts;

- Increasing the capacity of the connection by providing concrete fill over the deck with dowels grouted into holes drilled into or through the wall;

- Increasing the capacity of the connection by providing new steel members to effect a direct transfer of diaphragm shears to a shear wall or steel frame; and

- Reducing the local stresses by providing additional vertical-resisting components, such as shear walls, braced frames, or moment frames.

Steel decking is typically supported by metal framing, by steel angles, or by channel ledgers bolted to concrete or masonry walls. If the deficiency is in the connection and not the diaphragm, the most cost-effective alternative is to increase the welding of the decking to the steel member or ledger to at least the capacity of the diaphragm. If supported by a ledger, the capacity of the ledger connections to the concrete or masonry wall also may have to be improved, this is most effectively done by providing additional bolts in drilled and grouted holes (Figure 8-18). If the decking is being reinforced by filling with reinforced concrete, the most effective alternative will be to drill and grout dowels into the adjacent concrete or masonry wall and lap with reinforcing steel in the new slab. In some cases, it may be feasible to use the existing steel support member at the wall as a collector. The capacity of the existing decking can be increased by additional welding to the ledger angle and the addition of a reinforced concrete fill. Reinforcement dowels are welded to the angle that functions as a collector member, and the shear forces are transferred to the wall by the existing and new anchor bolts, as required. Steel deck roof diaphragms may be supported on open-web steel joists that rest on steel bearing plates at the top of concrete or masonry walls. In existing buildings that have not been properly designed for resisting lateral loads, there may not be a direct path for the transfer of diaphragm shears to the vertical walls, particularly when the decking span is parallel to the wall. New steel elements, as indicated in Figure 8-12, can be provided between the joists for direct connection to the decking. A continuous member also can be provided to function as a chord or collector member. As noted above, strengthening a steel deck diaphragm connection to the vertical-resisting component is effective only if the body of the diaphragm has adequate capacity to resist the design lateral forces. If the diaphragm does not have adequate capacity, it needs to be strengthened. As previously discussed, reducing the shear transfer forces in the deficient connection by providing supplemental vertical-resisting components is not likely to be the most cost-effective alternative (due to the probable need of new foundations and drag struts) unless it is being considered to correct other component deficiencies. Further, in order to reduce out-of-plane wall forces, the new vertical components would be required to act as buttresses to the existing walls.

(5) Connections of steel deck diaphragms with concrete fill.

(a) Deficiencies. The principal deficiencies of a connection of a steel deck diaphragm with concrete fill to the vertical-resisting component, such as shear walls, braced frames, or moment frames, are the in-plane shear capacity, or anchorage capacity for out-of-plane forces in walls.

(b) Strengthening techniques for steel deck connections. Deficient shear capacity or anchorage capacity of a connection of a steel-deck diaphragm to a shear wall, braced frame, or moment frame can be improved by:

- Increasing the shear capacity by drilling holes through the concrete fill, and providing additional shear studs welded to the vertical elements through the decking;
- Increasing the capacity of the connection by providing additional

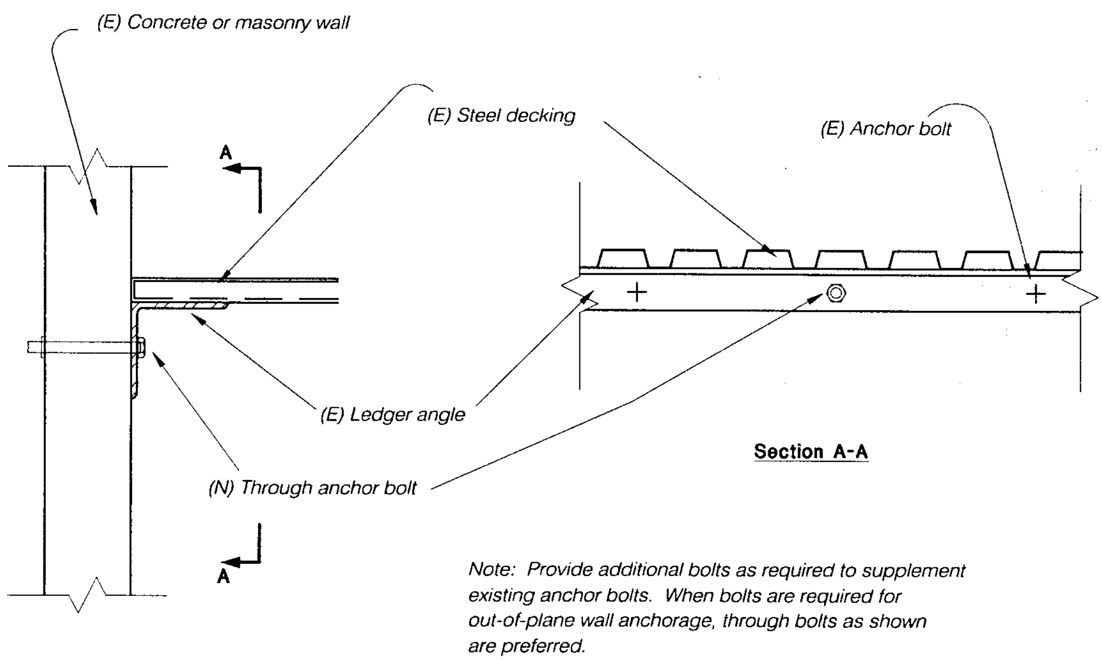


Figure 8-18. Strengthening Steel Decking Support for Shear Transfer and Wall Anchorage

anchor bolts (drilled and grouted) connecting the steel support to the wall (Figure 8-18);

- Increasing the capacity of the connection by providing additional dowels between the existing wall and diaphragm slab; and

- Reducing the local stresses by providing additional vertical-resisting components such as shear walls, braced frames, or moment frames.

If the deficiency is in both the connection of the diaphragm to the ledger and the ledger to the shear wall, the most cost-effective alternative may be to provide a direct-force transfer from the slab to the wall by installing dowels. This is accomplished by removing the concrete to expose the diaphragm slab reinforcement, drilling holes in the wall, laying in dowels, and grouting and reconstructing the diaphragm slab. If the deficiency is in the deck-to-supporting steel member connection, the first technique is preferred. If the deficiency is in the steel ledger to the wall connection, the second technique is preferred. Figure 8-18 illustrates a technique for strengthening a steel deck diaphragm connection to a concrete or masonry wall. In this figure, it is assumed that the existing decking with concrete fill has adequate capacity for the design loads, but the connection to the wall is deficient for in-plane shear and out-of-plane anchorage forces. In the figure, the in-plane shear is assumed to be transferred from the decking to the existing ledger angle with additional welding (if required). Supplementary bolts are installed to connect the ledger angle to the wall for the required in-plane and out-of-plane capacity. When the decking is spanning parallel to the wall, new steel straps, welded to the ledger angle and to the underside of the decking, can provide the additional out-of-plane anchorage capacity. When the new dowels or anchor bolts are to be attached to existing thin concrete walls (e.g., precast tees or other thin-ribbed concrete sections), through-bolts or threaded rods are required to provide adequate anchorage or doweling to the diaphragm. If the vertical-resisting elements are steel-braced frames or steel moment frames, the increase in connection capacity obviously would be achieved through additional welding and supplemental reinforcing members, as required.

(6) Connections of horizontal steel bracing.

(a) Deficiencies. The two primary deficiencies in the connection capacity of horizontal steel braces to vertical-resisting components such as shear walls or braced frames are:

- Inadequate in-plane shear transfer capacity; and

- Inadequate anchorage capacity when supporting concrete or masonry walls for out-of-plane forces.

(b) Strengthening techniques for in-plane shear transfer capacity. Deficient shear transfer of connections of horizontal steel bracing systems to shear walls or braced frames can be improved by:

- Increasing the capacity by providing larger or more bolts or by welding; and

- Reducing the stresses by providing supplemental vertical-resisting components such as shear walls or braced frames.

The first alternative of providing larger or more bolts between the horizontal brace members and the concrete or masonry shear wall, or providing additional welding when connecting to a steel-braced frame, generally will be the most cost-effective. This alternative assumes that the individual member connections at the joints of the bracing system are adequate, and only the connections to the shear walls or braced frames are deficient. Collectors along the wall may be required to distribute the concentrated brace shear along the wall to allow for adequate bolt spacing. As previously discussed, reducing the forces in the deficient connection by providing supplemental vertical-resisting components is not likely to be the most cost-effective alternative, unless it is being considered to correct other component deficiencies.

(c) Strengthening techniques for out-of-plane anchorage. Deficient out-of-plane anchorage capacity of connections between horizontal steel bracing systems and concrete or masonry shear walls can be improved by increasing the capacity of the connection by providing additional anchor bolts grouted in drilled holes, and by providing more bolts or welding to the bracing members.

(7) Connections in timber diaphragms.

(a) Deficiencies. The principal connection deficiencies in timber diaphragms are:

- Inadequate capacity to transfer in-plane shear at the connection of the diaphragm to interior shear walls or vertical bracing;

- Inadequate capacity to transfer in-plane shear at the connection of the diaphragm to exterior shear walls or vertical bracing; and

- Inadequate out-of-plane anchorage at the connection of the diaphragm to exterior concrete or masonry walls.

(b) Strengthening techniques for internal shear wall connections. Deficient shear transfer capacity of a diaphragm at the connection to an interior shear wall or braced frame can be improved by:

- Increasing the shear transfer capacity of the diaphragm local to the connection by providing additional nailing to existing or new blocking, and additional bolting to the wall or frame (similar to FEMA 172, Figures 3.7.1.2 a and 3.7.1.2 b);

- Reducing the local shear transfer stresses by distributing the forces from the diaphragm by providing a collector member to transfer the diaphragm forces to the shear wall; and

- Reducing the shear transfer stress in the existing connection by providing supplemental vertical-resisting elements.

If the shear transfer deficiency is governed by the existing nailing, the most cost-effective alternative probably will be to provide additional nailing; however, stripping of the flooring or roofing surface is required. If it is not feasible to provide adequate additional nailing within the length of the shear wall, the installation of a collector probably will be the most cost-effective alternative. If the nailing of the diaphragm to the new blocking is inadequate to transfer the desired shear force over the length of the shear wall, a drag strut or collector member should be provided, and the new blocking extended as a required beyond the end of the shear wall. The shear force is collected in the drag strut and transferred to the shear wall with more effective nailing or bolting. The new lumber must be dimensionally stable and cut to size. Providing additional vertical-resisting elements usually involves construction of additional interior shear walls or exterior buttresses. This alternative generally is more expensive than the other two because of the need for new foundations and for drag struts or other connections to collect the diaphragm shears for transfer to the new shear walls or buttresses.

(c) Strengthening techniques for in-plane shear transfer capacity to exterior walls. Deficient in-plane shear transfer capacity of a diaphragm to exterior shear walls or braced frames can be improved by:

- Increasing the capacity of existing connections by providing additional nailing and/or bolting;
- Reducing the local shear transfer stresses by distributing the forces from the diaphragm by providing chords or collector members to collect and distribute shear from the diaphragm to the shear wall or bracing; and
- Reducing shear stress in the existing connections by providing supplemental vertical-resisting components.

Inadequate in-plane shear transfer capacity at an exterior shear wall typically is a deficiency when large openings along the line of the wall exist. In this case, the shear force to be resisted per unit length of wall may be significantly greater than the shear force per unit length transferred from the diaphragm by the existing nailing or bolting. If the diaphragm and the shear walls have adequate shear capacity, the solution requires transfer of the diaphragm shear to a collector member for distribution to the discontinuous shear walls. For timber shear walls parallel to the joists, the exterior joist usually is doubled-up at the exterior wall and extended as a header over openings. This doubled joist can be spliced for continuity and used as drag strut with shear transfer to the wall by means of metal clip anchors and nails or lag screws. If the resulting unit shears in the walls on either side of the opening are larger than the existing shear transfer capacity of the roof diaphragm (e.g., in this case, the capacity is governed by the existing nailing to the perimeter blocking or double joists), a collector member is required to collect the diaphragm shears and transfer them, at a higher shear stress, to the shear walls. For steel frame buildings with discontinuous braced panels, the spandrel supporting the floor or roof framing may be used as a chord or collector member. For discontinuous masonry, concrete, or precast concrete shear walls parallel to the joists, the sheathing typically is nailed to a joist, or ledger-bolted to the wall. The joist or ledger can be spliced for continuity and supplementary bolting to the shear wall provided as required. For shear walls perpendicular to the joists, the sheathing may be nailed to discontinuous blocking between the ends of the joists. In this case, the chord or collector member may have to be provided on top of the diaphragm. This new member may be a continuous steel member bolted to the wall and nailed or lag screwed, with proper edge distance, to the diaphragm, and also could be designed to provide out-of-plane anchorage with welded steel straps nailed to the diaphragm. As discussed above with respect to interior wall connection deficiencies, providing additional vertical-resisting components is likely to be the most costly alternative, unless it is being considered to correct other component deficiencies.

(d) Strengthening techniques for inadequate out-of-plane anchorage. Deficient out-of-plane anchorage capacity of wood diaphragms connected to concrete or masonry walls with wood ledgers can be improved by:

- Increasing the capacity of the connection by providing steel straps connected to the wall (using drilled and grouted bolts or through-bolts for masonry walls), and bolted or lagged to the diaphragm or roof or floor joists (FEMA 172, Figures 3.7.1.4 a and 3.7.1.4 b);
- Increasing the capacity of the connections by providing a steel anchor to connect the roof or floor joists to the walls (FEMA 172, Figures 3.7.1.4 c and 3.7.1.4 d); and
- Increasing the redundancy of the connection by providing continuity ties into the diaphragm.

An important condition to be addressed in retrofitting any existing heavy walled structure with a wood diaphragm is the anchorage of the walls for out-of-plane forces. Prior to the mid-1970s, it was common construction practice to bolt a 3x (75 mm) ledger to a concrete or masonry wall; install metal joist hangers to the ledger; drop in 2x (50 mm) joists; and sheath with plywood. The plywood that lapped the ledger would be nailed into the ledger, providing both in-plane and out-of-plane shear transfer. The 1971 San Fernando earthquake caused many of these connections to fail. Out-of-plane forces stressed the ledgers in their weak cross-grain axis and caused many of them to split, allowing the walls to fall out and the roof to fall in. When retrofitting a masonry or concrete structure, this condition should be remedied by providing a positive connection between the concrete or masonry wall and wood diaphragm. The first two techniques are, in general, equally cost-effective. In addition to correcting the ledger concerns, continuity ties need to be provided between diaphragm chords in order to distribute the anchorage forces well into the diaphragm. Joist hangers and glulam connections frequently have no tensile capacity, but this tensile capacity can be provided by installing tie rods bolted to adjacent joist or glulam framing (FEMA 172, Figure 3.7.1.4 e). These continuity ties provide a necessary redundancy in the connection of heavy-walled structures to timber diaphragms.

b. Foundation connections. Seismic inertial forces originate in all elements of buildings and are delivered through structural connections to horizontal diaphragms. The diaphragms distribute these forces to vertical components that transfer the forces to the foundation, and the foundation transfers the forces into the ground. An adequate connection between the vertical components and the foundation is essential to the satisfactory performance of a strengthened structure. The connections must be capable of transferring the in-plane lateral inertia forces from the vertical components to the foundations, and of providing adequate capacity for resisting uplift forces caused by overturning moments.

(1) Connections of cast-in place concrete walls.

(a) Deficiencies. The principal deficiency in the connection of cast-in-place cement walls to the foundation is inadequate development length in the dowels for the vertical reinforcement ("starter" bars).

(b) Strengthening techniques for inadequate development length in the foundation dowels are:

- Provide adequate confinement in the lap area to make existing development lengths effective;
- Provide new boundary members at each end of the wall;
- Expose and lap-weld reinforcement; and
- Permit bond slip of reinforcement, and induce "yield" stress based on actual development length.

Development lengths for reinforcement can be reduced to the minimum values prescribed in ACI 318 with adequate confinement of other concrete. This can be achieved by casting a bolster (i.e., 3 or 4 inches of reinforced concrete) on each side of the wall in the lap area at each end of the wall, and providing transverse cross-ties through the existing wall. As indicated for shear walls in paragraph 8-2a(1)(c), the use of fiber-reinforced polymer (FRP) sheets to provide confinement in walls may be a possibility, but consensus guidelines for this application are currently unavailable. New boundary members, with vertical reinforcement properly anchored to the foundation, are an effective means to compensate for inadequate development lengths in the existing reinforcement. The boundary members will substantially increase the rigidity of the wall and will affect the distribution of the story shears. Lap welding of the reinforcement can be very effective if the reinforcement, when exposed, is in close contact. For double-curtain reinforcement, the reinforcement is exposed and welded only on one side for each curtain.

(2) Connections of precast concrete shear walls.

(a) Deficiencies. The principal deficiencies of the connections of precast concrete shear walls to the foundation are:

- Inadequate capacity to resist in-plane or out-of-plane shear forces; and

- Inadequate uplift capacity to resist seismic overturning forces.

(b) Strengthening techniques for inadequate shear capacity. Deficient shear capacity of the connections of precast shear walls to the foundation can be improved by:

- Increasing the capacity of the connection by adding a new steel member connecting the wall to the foundation or the ground-floor slab.

Early precast concrete wall construction frequently had minimal lateral connection capacity at the foundation. These connections usually can be strengthened most economically by attaching a steel member to the wall and the floor slab or foundation with drilled and grouted anchors or expansion bolts. Care must be taken to place bolts and/or dowels a sufficient distance away from concrete edges to prevent spalling under load. Figure 3.8.3.2 in FEMA 172 illustrates one option for this technique. In regions of low seismicity, the new steel angle with anchorage to the ground floor slab may be adequate. For more significant lateral forces, the steel plate alternative in Figure 8-19 provides a stronger and more positive connection.

(c) Strengthening techniques for inadequate hold-down capacity. Deficient hold-down capacity of the foundation can be improved by:

- Increasing the hold-down capacity by adding a bolted steel plate as indicated in Figure 8-19 at each end of the wall.

- Reducing the uplift forces by providing supplemental vertical-resisting components such as shear walls or braced frames.

Deficient hold-down capacity of precast units usually will occur when one unit or a part of one unit is required to resist a significant share of the seismic load. If the wall has sufficient bending and shear capacity, then increasing the hold-down capacity using the first technique is usually the most cost-effective. When a wall is composed of a number of solid (i.e., no significant openings) precast panels, the overturning forces generally will be minimal, provided there is adequate vertical shear capacity in the connections between the edges of adjacent panels. In this case, the connections must be checked, and if necessary, strengthened. The second technique usually is a viable approach only if it is being considered to correct other component deficiencies. When excessive uplift forces are due to inadequate vertical shear capacity in the vertical connections between adjacent precast units, strengthening of those connections will reduce the uplift forces.

(3) Connections of braced frames.

(a) Deficiencies. The principal deficiencies of the connections of steel braced frames to the foundation are:

- Inadequate shear capacity; and

- Inadequate uplift resistance.

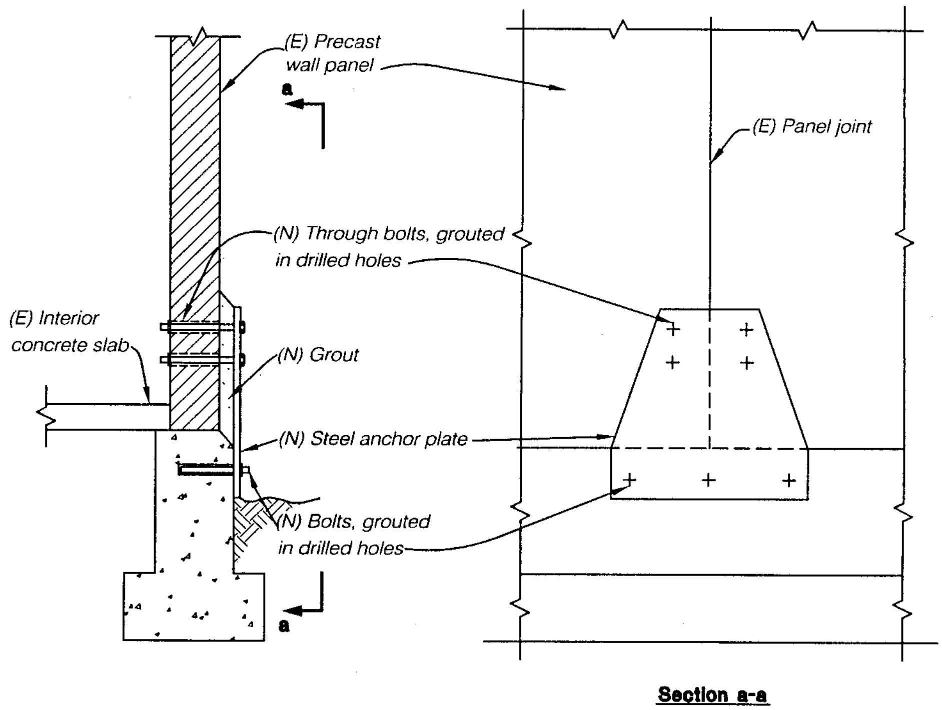


Figure 8-19. Steel Plate Anchorage for Precast Concrete Wall Panels

(b) Strengthening techniques for inadequate shear capacity. Deficient shear capacity of the connections of steel-braced frames to the foundations can be improved by:

- Increasing the capacity by providing new steel members welded to the braced-frame base plates, and anchored to the slab or foundation with drilled and grouted anchor bolts (Figure 8-20); and

- Reducing the shear loads by providing supplemental steel-braced frames.

The first alternative generally will be the most cost-effective, provided the existing slab or foundation can adequately resist the prescribed shear. Steel collectors welded to the existing steel base plates can distribute the shear forces into the slab or foundation. If the existing foundation requires strengthening to provide adequate shear capacity, determining the most cost-effective alternative requires comparing the effort necessary to construct a reinforced concrete foundation to the effort and disruption of functional space required to install supplementary shear walls and their associated foundations and collectors.

(c) Strengthening techniques for inadequate uplift resistance. Deficient uplift resistance capacity of the connections of steel-braced frames to the foundations can be improved by:

- Increasing the capacity by providing new steel members welded to the base plate and anchored to the existing foundation (Figure 8-20); and

- Reducing the uplift loads by providing supplemental steel-braced frames.

Inadequate uplift resistance capacity of a steel-braced frame seldom results just because of deficient connection to the foundation, but is typically a concern reflecting the uplift capacity of the foundation itself. If the foundation is the concern, the techniques discussed in paragraph 8-2g can be considered to correct the problem. If, in fact, the deficiency is the connection, providing new connecting members will be the most economical.

(4) Connection of steel moment frames.

(a) Deficiencies. The principal deficiencies of the connection of a moment frame column to the foundation are:

- Inadequate shear capacity;

- Inadequate flexural capacity; and

- Inadequate uplift capacity.

(b) Strengthening techniques for inadequate shear, flexural, or uplift capacity. The techniques for strengthening steel moment frame column base connections to improve shear and flexural capacity also will likely improve the uplift capacity. For this reason, a combination of the following alternatives may be utilized to correct a deficient column base connection:

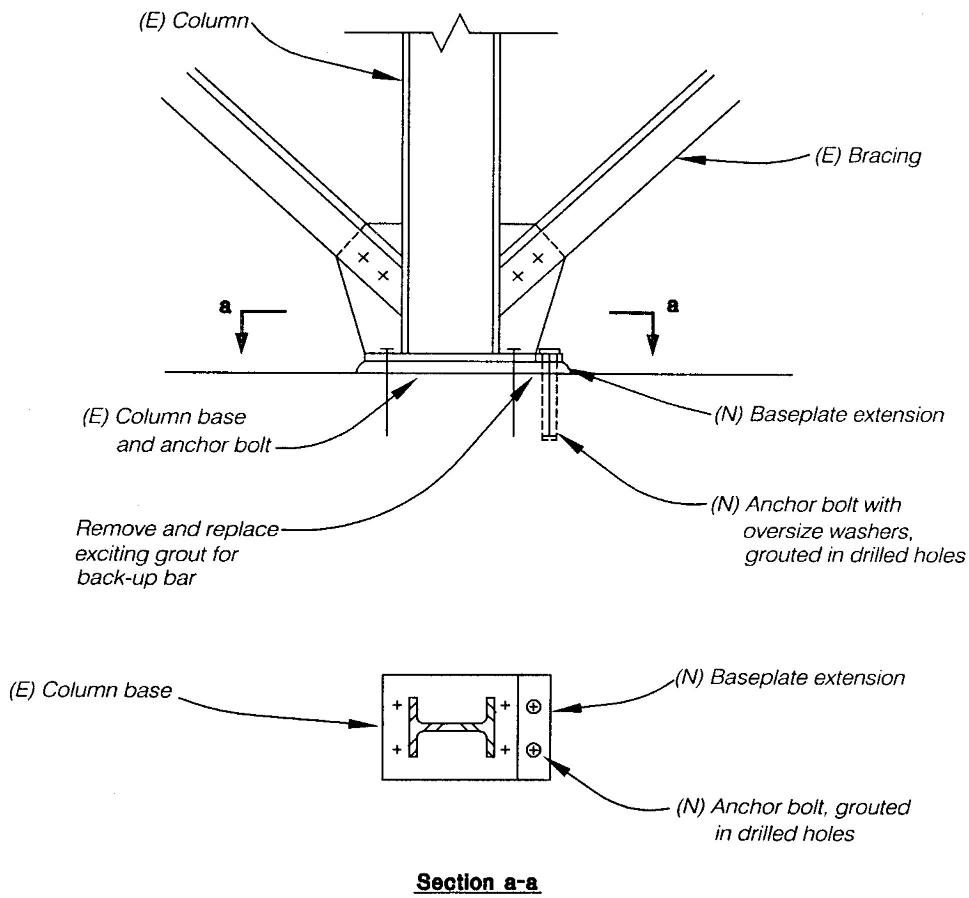


Figure 8-20. Strengthening of a Column Base Plate in a Braced Frame

- Increasing the shear and tensile capacity by enlarging the base plate and installing additional anchor bolts into the foundation (Figure 8-20); and

- Increasing the shear capacity by embedding the column in a reinforced concrete pedestal that is bonded or embedded into the existing slab or foundation.

If the above deficiencies occur only in the column base connection, it is possible to strengthen the connection by enlarging and stiffening the base plate and adding additional anchor bolts. If the column base connection is embedded in a monolithic concrete slab, the slab may be considered for distribution of the shear to the ground by means of any additional existing footings that are connected to the slab. If the column is not embedded in the slab, the same effect can be achieved by adding a concrete pedestal. The interference of this pedestal with the function and operations of the area is an obvious drawback.

8-4. Rehabilitation with Protective Systems

a. General. Although protective systems (i.e., seismic isolation or energy dissipation) can be efficiently used for new construction, most of the installations in the present decade have been used to retrofit existing buildings. The advantages of these systems, for suitable candidate buildings, is that significant reduction in the seismic demand can be achieved, thereby minimizing the structural rehabilitation and functional disruption to the existing building. Seismic isolation has been successfully utilized in the seismic retrofit of historic buildings where other retrofit procedures would have altered the historic structural fabric of the building.

b. Seismic isolation. The design of a seismic isolation system depends on many factors, including the period of the fixed-base structure, the period of the isolated structure, the dynamic characteristics of the soil at the site, the shape of the input response spectrum, and the force-deformation relationship for the particular isolation device. The primary objective of the design is to obtain a structure such that the isolated period of the building is sufficiently longer than both the fixed-base period of the building (i.e., the period of the superstructure), and the predominant period of the soil at the site. In this way, the superstructure can be decoupled from the maximum earthquake input energy. The spectral accelerations at the isolated period of the building are significantly reduced from those at the fixed-base period. The resultant forces on structural and nonstructural elements of the superstructure will be significantly reduced when compared with conventional fixed-base design. The benefits resulting from base isolation are attributed primarily to a reduction in spectral acceleration demand due to a longer period, as discussed in this paragraph. Additional benefits may come from a further reduction in the spectral demand attained by supplemental damping provided by high-damped rubber components or lead cores in the isolation units. Guidelines for the selection and design of these systems are provided in TI 809-04. Figures 8-2, 8-3, and 8-4 in that document indicate the potential reduction in seismic demand for buildings with initial fundamental periods of 0.3, 0.7, and 1.2 secs., founded on these different soil profiles and retrofitted with an isolation period of 2.5 secs. The isolators generally are installed immediately above the foundation level, and a rigid diaphragm or horizontal bracing system is necessary above the isolators to provide displacement compliance for the structural elements (i.e., columns or walls) above the isolators. The anticipated maximum displacement of the isolators must be accommodated by flexible and/or expansion joints in all utility services, stairs, and ramps entering the building, and by a structural gap or moat around the perimeter of the building. Rehabilitation with base isolation will concentrate most of the construction work at the base of the building; however, most existing buildings in which this technique has been utilized have also required some measure of structural rehabilitation in the building above the isolators. Base isolation is significantly more expensive than simple structural rehabilitation, but its use has been justified by minimizing disruption of function, precluding rehabilitation of historic structural features, and protection of fragile nonstructural components or essential equipment.

c. Energy dissipation. These systems are designed to provide supplemental damping in order to reduce the seismic input forces. Most conventional buildings are designed assuming 5% equivalent viscous damping for structures responding in the elastic range. For structures that include viscous dampers or metallic yielding devices, the equivalent viscous damping may be increased to between 15% and 25%, depending on the specific characteristics of the device. In this way, seismic input energy to the structure is largely dissipated through the inelastic deformations concentrated in the devices, reducing damage to other critical elements of the building. The benefits resulting from the use of displacement-dependent or velocity-dependent energy dissipation devices are attributed primarily to the reduction in spectral demand due to supplemental damping provided by the devices. Unlike seismic isolation, where structural alterations can be essentially confined to the base of an existing building, these systems require that the energy dissipation devices be distributed throughout the building. Guidelines for the selection and design of energy dissipation systems are provided in Chapter 8 of TI 809-04. Figures 8-5 and 8-6 in that document indicate the potential reduction in seismic demand for the same three buildings described in paragraph 8-4b above, as the effective damping is increased from 5 percent to 20 percent. The effectiveness of these devices is dependent on the relative displacement and/or velocity of the two ends of each device; therefore, these devices are not generally effective for shear wall buildings or reinforced concrete frames with limited ductility.