CHAPTER 8
REHABILITATION TECHNIQUES FOR STRUCTURAL SYSTEMS

8-1. General

This chapter provides guidance for the selection of upgrading or rehabilitation techniques for the mitigation of identified deficiencies in structural systems. Guidance is also provided for innovative systems that may reduce or preclude the need to rehabilitate an existing deficient structural system.

a. Rehabilitation strategies. The rehabilitation techniques discussed in the following paragraphs are based on one or more of the following optional strategies:

(1) Strengthening or stiffening existing deficient structural components. Included in this category is the necessary strengthening or stiffening of connections to transfer forces to and from adjacent components. Also included are procedures that add ductility without significantly changing strength or stiffness.

(2) Replacing existing deficient components with stronger or stiffer components. The note in preceding paragraph regarding connecting elements also applies to this option.

(3) Providing supplementary structural systems or components. In some cases, the existing structural systems may be adequate for the gravity loads, but may not have adequate strength or stiffness for the required seismic loads (e.g., an older steel or concrete moment frame building). In these cases, a cost-effective retrofit may be to provide a new structural system designed to resist only the lateral loads (e.g., provide new exterior or interior shear walls in a moment frame building). If the new systems can be designed and constructed within the applicable functional and aesthetic restrictions, it may be more efficient, as well as more economical, than the labor-intensive strengthening of the existing frame systems. Selection of an appropriate new structural system must consider the deformation compatibility of the two systems for elastic and post-yield response. For example, new concrete shear walls are a common and effective rehabilitation technique for deficient low-rise moment frame buildings. The greater stiffness of the shear walls significantly reduces the seismic forces and deformations to be resisted by the frames, and a properly designed shear wall will have moderate post-yield capacity for energy dissipation without reliance on the ductility of the frame. New shear walls may not be appropriate for rehabilitating a high-rise (i.e., 10 stories or more) frame building. The cantilever deformation of the shear walls in the upper stories will usually exceed the predominantly shear deformation of the frames, and the total shear resisted by the frames in those stories may exceed the story shear. Similarly, a moment frame system is usually not appropriate to strengthen a deficient low-rise shear wall system.

(4) Modification of the building.

(a) Elimination of vertical or plan irregularities. This can be very effective in improving building response and reducing the probability of damage to peripheral components.

(b) Reduction of mass. This could be accomplished by the removal of one or more stories to effect a reduction in the seismic responses of the remaining stories. If the building has water storage tanks or other heavy nonstructural items on the roof or in upper stories, relocation of these items to the grounds will also reduce the seismic response in the building.

(5) Protective systems.

(a) Base isolation. This reduces the response for some buildings by lengthening the fundamental period, and is most effective for stiff buildings on stiff soils.

(b) Energy dissipation. This reduces the response for some buildings by increased damping of the dynamic response, and is most effective for buildings with fundamental periods close to the natural site period.

b. Rehabilitation techniques. The following paragraphs provide discussion of alternative techniques for rehabilitation of primary structural components. Tables 8-1 through 8-5 illustrate the application of these techniques to representative structural systems. The rehabilitation techniques described in this chapter are representative of current practice in structural rehabilitation. The number of techniques described is by no means inclusive, or meant to be restrictive. Other techniques may be employed provided they possess the necessary strength, stiffness, and if required, energy dissipation capabilities to be compatible with those assumed in the analysis and design of the rehabilitation.

8-2. Rehabilitation Techniques for Structural Components

The rehabilitation techniques described in this paragraph will employ one or more of the first three rehabilitation strategies described in paragraph 8-1a. Modification of structural response with protective systems should be understood to be an option that, for some buildings, could reduce the response of deficient structural components to acceptable limits without rehabilitation, and should always be evaluated; particularly when it is important to avoid alteration of an existing structure, such as an historic building, or disruption of an important function in an essential building.

a. Shear walls. Shear walls are structural walls designed to resist lateral forces parallel to the plane of the wall. Shear walls may consist of cast-in-place reinforced concrete, masonry, precast concrete, and unreinforced masonry. Shear walls that are restrained within a moment frame are classified as in-filled walls, and are discussed in paragraph 8-2c. Shear walls in wood frame buildings and wall panels in light steel frame buildings are beyond the scope of this document.

(1) Cast-in-place reinforced concrete and masonry shear walls. Cavity walls in reinforced masonry are assumed to consist of the inner wythes as a shear wall and an outer wythe of veneer laterally supported by metal ties across the cavity. Strengthening options for reinforced concrete or masonry shear wall buildings are provided in Table 8-1.

(a) Deficiencies. The principal deficiencies of reinforced concrete or masonry shear walls are:

Table 8-1. Strengthening Options for Reinforced Concrete or Masonry Shear Wall Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
a. Shear walls	(1) Inadequate shear capacity	(a) Fill in openings	para 8-2a(1)(b)		3.2.1.2a 3.2.1.2b
		(b) Add reinforced concrete to interior or exterior face	para 8-2a(1)(b)	8-2	3.2.1.2c
		(c) Provide FRP overlay to interior or exterior face	para 8-2a(1)(b)	8-1	
		(d) Provide supplemental vertical resisting elements	para 8-2a(1)(b)		3.4 and 3.4.2
	(2) Inadequate flexural capacity	Same as above	para 8-2a(1)(c)	8-3	Same as above
b. Coupling beams	(1) Inadequate shear or flexural capacity	(a) Fill in openings	para 8-2a(1)(d)		3.2.1.2a 3.2.1.2b
		(b) Remove and Replace	para 8-2a(1)(d)		3.2.1.4
		(c) Accept rotational damage	para 8-2a(1)(d)		
c. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with reinforced concrete	para 8-2f(1)(b)		3.5.2.2
	(2) Inadequate chord capacity	(a) Add new concrete or steel chord	para 8-2f(1)(d)		3.5.2.3 3.5.4.3
	(3) Shear or tensile stresses at openings	(a) Add structural member below the slab	para 8-2f(1)(d)		3.5.2.4a
		(b) Add concrete topping with trim bars	para 8-2f(1)(e)		3.5.2.4b
		(c) Fill in opening	para 8-2f(1)(e)		3.2.1.2b
	(4) Inadequate wall anchorage	(a) Add new concrete or steei doweled into wall	para 8-3a(1)(c)		3.5.2.3 3.5.4.3

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Table 8-1. Strengthening Options for Reinforced Concrete or Masonry Shear Wall Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
d. Steel deck floor or roof diaphragm	(1) Inadequate shear capacity	(a) Additional welding	para 8-2f(4)(b)		
		(b) Add concrete fill or overlay	para 8-2f(4)(b)		3.5.5.2a
		(c) Provide horizontal bracing system	para 8-2f(4)(b)		3.5.5.2b
	(2) Inadequate shear transfer	(a) Add steel member between joists	para 8-2f(4)(b)	8-12	
		(b) Add additional bolts to wall	para 8-3a(4)(b)	8-18	
	(3) Inadequate wall anchorage	(a) Add through bolts to wall	para 8-3a(4)(b)	8-18	
		(b) Add welded straps to decking	para 8-3a(5)(b)		
e. Precast concrete diaphragms	(1) Inadequate shear capacity of connections	(a) Add reinforced concrete overlay	para 8-2f(3)(b)		3.5.4.2
	(2) Inadequate chord capacity	(a) Add continuous steel or concrete member above or below slab	para 8-2f(3)(c)		3.5.4.3
	(3) Excessive shear or tensile stress at openings	(a) Add structural members below the slab	para 8-2f(1)(d)		3.5.2.4a
		(b) Add concrete topping with trim bars	para 8-2f(1)(e)		3.5.2.4b
		(c) Fill-in opening	para 8-2f(1)(e)		3.2.1.2b
	(4) Inadequate wall anchorage	(a) Add reinforced concrete overlay	para 8-2f(3)(b)		3.5.4.2
f. Continuous footings	(1) Excessive soil bearing pressure	(a) Add drilled piers	para 8-2g(1)(b)		3.6.1.2b
		(b) Modify soil properties	para 8-2g(1)(b)		
		(c) Underpin existing footing	para 8-2g(1)(b)		3.6.1.2a
	(2) Excessive uplift forces	(a) Add drilled piers	para 8-2g(1)(c)		3.6.1.2b

- Inadequate shear capacity (shear or shear-compression at the toe of the wall);

- Inadequate flexural capacity;

- Inadequate shear or flexural capacity in the coupling beams between shear walls or piers;

- Vertical discontinuities; and

- Inadequate development lengths for reinforcement at splices or dowels.

(b) Strengthening techniques for shear capacity. Deficient shear capacity of existing concrete or reinforced masonry shear walls can be improved by:

- Increasing the effectiveness of the existing walls by filling in door or window openings with reinforced concrete or masonry (FEMA 172, Figures 3.2.1.2 a and 3.2.1.2 b).

- Providing a fiber-reinforced polymer (FRP) overlay on one or both sides of the existing shear wall (Figure 8-1).

- Providing additional thickness to the existing walls with a cast-in-place or pneumatically applied (i.e., shotcrete) reinforced concrete overlay anchored to the inside or outside face of the existing walls (Figure 8-2 and FEMA 172, Figure 3.2.1.2 c).

- Reducing the shear or flexural stresses in the existing walls by providing supplemental vertical-resisting components (i.e., shear walls, bracing, or external buttresses).

The first three techniques discussed above will generally be more economical than the fourth, particularly if they can be accomplished without increasing existing foundations. If adequate additional capacity can be obtained by filling in selected window or door openings without impairing the functional or aesthetic aspects of the building, this alternative will probably be the most economical. The infill should be selected to match the shear modulus of the wall within reasonable limits (i.e., brick or CMU infill should be used in brick or CMU walls). If this is not feasible, the second or third technique should be considered. The optimum application of these alternatives would be when adequate additional capacity could be obtained by an overlay on a selected portion of the outside face of the perimeter walls without unduly impairing the functional or aesthetic qualities of the building, and without the need to increase the footings. In some cases, restrictions may preclude any change in the exterior appearance of the building (i.e., a building with historical significance). In these cases, it will be necessary to consider overlays to the inside face of the exterior shear walls or to either face of interior shear walls. Obviously this is more disruptive, and thus more costly, than restricting the work to the exterior of the building; however, if the functional activities within the building are to be temporarily relocated because of other interior alterations, the cost difference between the overlay to the inside face and the outside face of the building walls is reduced. In some cases, for example, when deficiencies exist

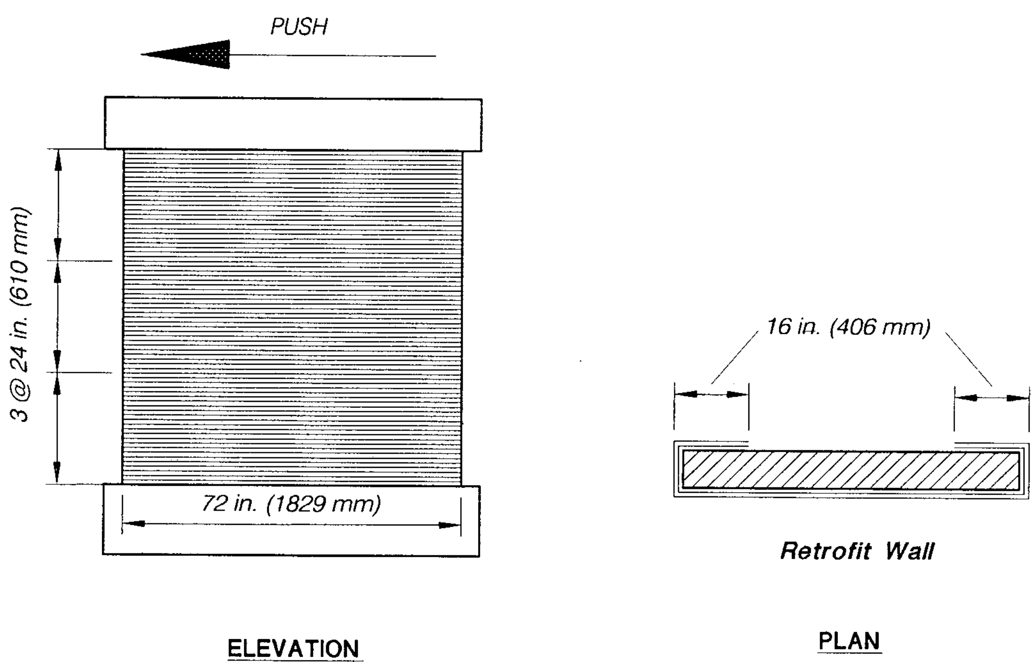
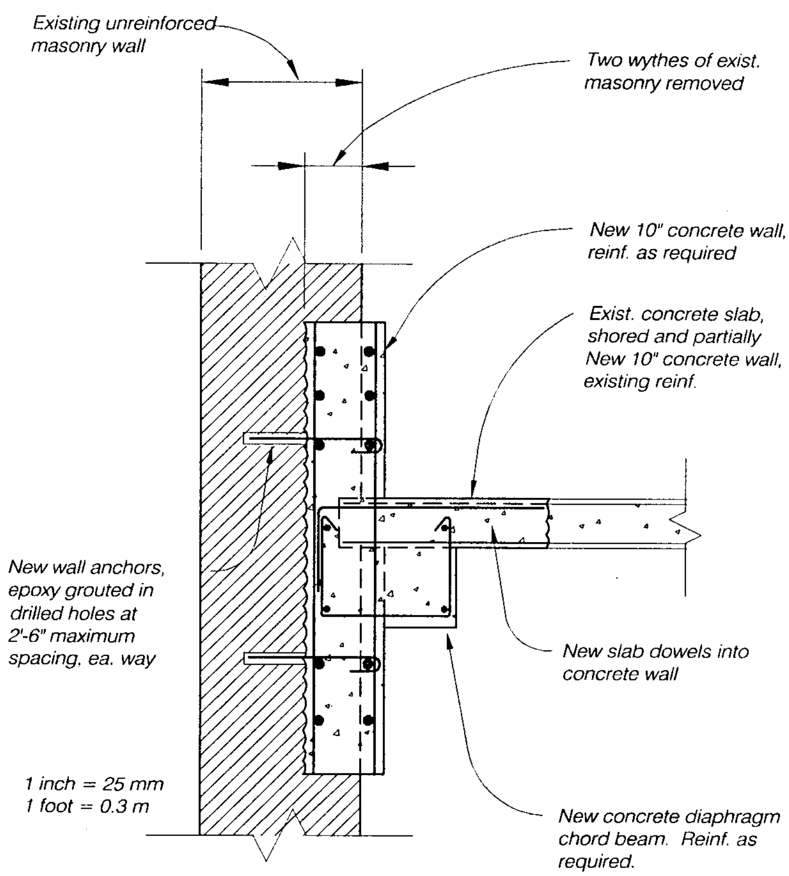


Figure 8-1. Carbon Fiber Overlay to Enhance Shear Capacity of Masonry Wall



1 inch = 25 mm

1 foot = 0.3 m

Figure 8-2. Strengthening of an Unreinforced Masonry Wall

in the capacity of the diaphragm chords or in the shear transfer from the diaphragm to the shear walls, there may be compelling reasons to place the overlay on the inside face and concurrently solve other problems. FRP overlays with either fiberglass or carbon fibers are a comparatively recent procedure, and the application technique and structural preparation depend on the composition of the overlay. Technical data are available from the FRP manufacturers, and consensus guidelines are being developed by CERL and ACI. Figure 8-1 depicts a test specimen of a masonry shear wall with an FRP overlay. When overlays are applied to an existing brick masonry or CMU wall, the masonry in that wall shall be ignored in the distribution of shears by relative rigidities. If the overlay is not applied to all wall panels in the same line of force, only the masonry in the overlay panel shall be ignored. Providing supplemental vertical-resisting components usually involves construction of additional interior shear walls or exterior buttresses. This alternative is generally more expensive than the other two, because of the need for new foundations and for new drag struts or otherconnections to collect the diaphragm shears for transfer to the new shear walls or buttresses. The foundations required to resist overturning forces for an exterior buttress are significant because the dead weight of the building cannot be mobilized to resist the uplift forces on the outer column. Piles or drilled piers may be required to provide tensile hold-down capacity for the footings. Buttresses located on both ends of the wall can be designed to take compression only, minimizing the foundation problems. Buttresses frequently are not feasible due to adjacent buildings or property lines. The advantage of the buttress over a new interior shear wall is that the work can be accomplished with minimal interference to ongoing building functions.

(c) Strengthening techniques for flexural capacity. Deficient flexural capacity of existing reinforced concrete or masonry shear walls can be improved using the same techniques identified to improve shear capacity, ensuring that flexural steel has adequate connection capacity into existing walls and foundations. Shear walls that yield in flexure are more ductile than those that yield in shear. Shear walls that are heavily reinforced (i.e., with a reinforcement ratio greater than about 0.005) are also more susceptible to brittle failure; therefore, care must be taken not to overdesign the flexural capacity of rehabilitated shear walls. FRP overlays are not generally effective to enhance flexural capacity because of the difficulty associated with development of the tensile capacity of the overlay at the bottom or top of the wall. Figure 8-3 depicts two test specimens of flexural masonry walls overlaid with FRP sheets. FRP overlays have also been successfully used to provide confinement that reduces the necessary development length of reinforcement, and also enhances the ductility of a flexure-controlled wall by permitting greater inelastic compressive strains in the concrete.

(d) Strengthening techniques for coupling beams. Deficient shear or flexural capacity in coupling beams of reinforced concrete or reinforced masonry shear wall can be improved by:

- Improving the ductility of the coupling beams with FRP overlays;
- Eliminating the coupling beams by filling in openings with reinforced concrete or masonry;

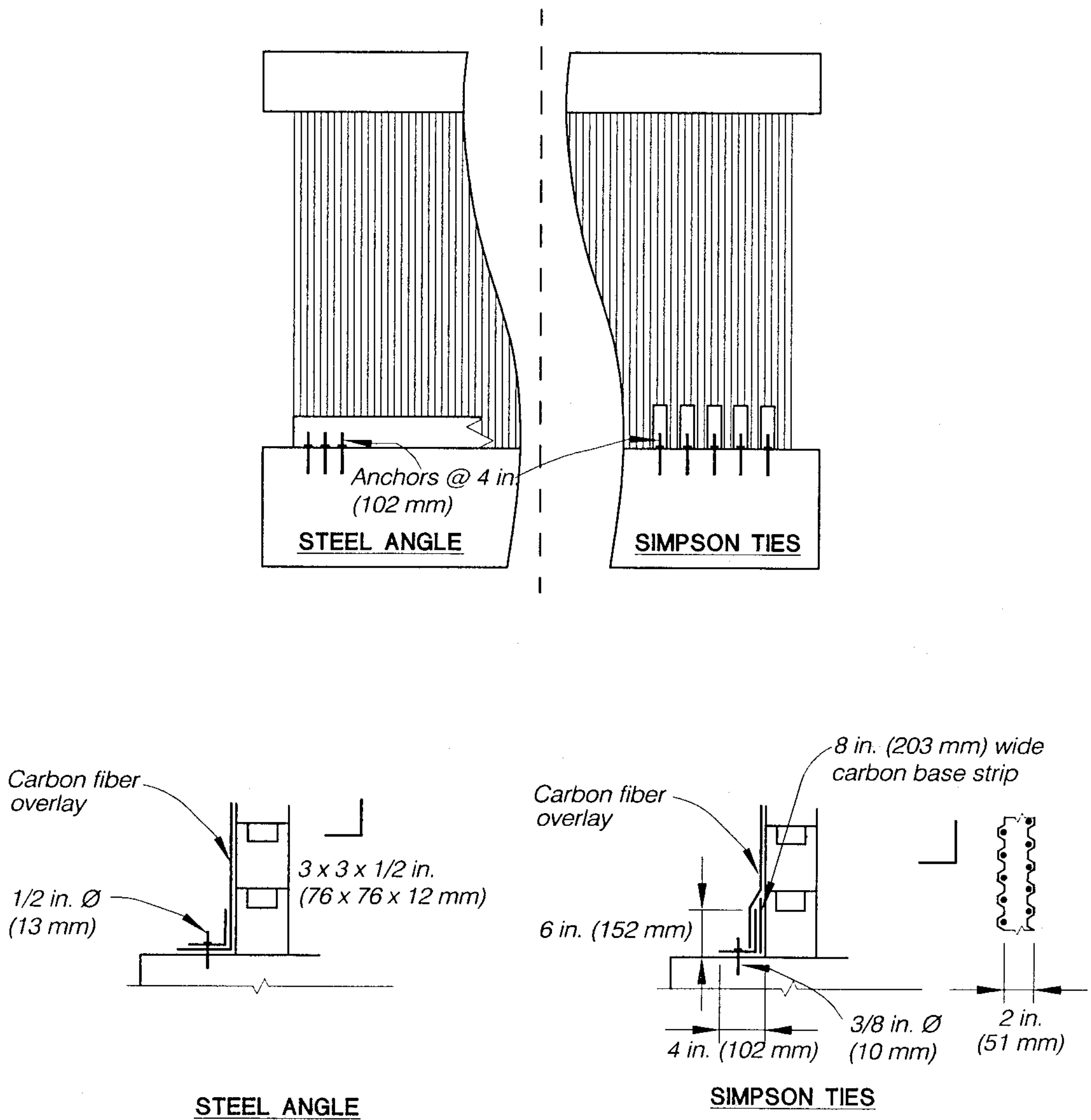


Figure 8-3. Carbon Fiber Overlay to Enhance Flexural Capacity of Masonry Wall

- Removing the existing beams and replacing with new, properly reinforced beams (FEMA 172, Figure 3.2.1.4); and

- Reducing the shear or flexural stresses in the connecting beams by providing additional vertical-resisting components (i.e., shear walls, bracing, or external buttresses).

If the deficiency is in both the piers and the connecting beams, the most economical solution is likely to be adding reinforced concrete on one or both sides of the existing wall and replacing the beams with properly reinforced concrete. The new concrete may be formed and poured in place or may be placed by the pneumatic method. If the identified deficiency exists only in the flexural capacity of the connecting beams, consideration should be given to improving the ductility of the coupling beams with FRP overlays or the acceptance of some minor damage in the form of cracking or spalling by repeating the structural evaluation with the deficient beams modeled as pin-ended links between the piers. If this condition is unacceptable, the second technique may be the most economical, and the beams should be removed and replaced with properly designed reinforced concrete. Depending on functional and architectural, as well as structural considerations, filling in selected openings may be practical. If the first two techniques are not feasible or adequate to ensure the proper performance of the wall, the third technique, reducing the stress by adding supplemental new structural components, should be considered. This alternative is likely to be the most costly because of the need for new foundations, vertical members, and collectors.

(e) Strengthening techniques for vertical discontinuities. Discontinuous shear walls (i.e., shear walls that do not extend to the foundation) are often supported in a lower story by concrete columns or piers. These supporting structural components are vulnerable to possible overstrength in the supported shear walls, particularly from the overturning forces due to lateral loads. For Seismic Use Group I buildings, these components shall be analyzed and strengthened or replaced, if necessary, in accordance with the provisions of the 9.6.2 of FEMA 302 using the special load combinations (and with the Ω_o overstrength factor). For buildings with enhanced performance objectives, the supporting columns or piers shall be considered to be force-controlled components in accordance with paragraph 7-2f(5).

(2) Precast concrete shear walls.

(a) Deficiencies. The principal deficiencies of precast shear walls are:

- Inadequate shear or flexural capacity in the wall panels;

- Inadequate interpanel shear or flexural capacity;

- Inadequate out-of-plane flexural capacity; and

- Inadequate shear or flexural capacity in coupling beams.

(b) Strengthening techniques for inadequate shear or tensile capacity. Deficient in-plane shear or flexural capacity of precast concrete panel walls can be improved by:

- Increasing the shear capacity of walls with significant openings for doors or windows by infilling the existing openings with reinforced concrete;

- Increasing the shear or flexural capacity by adding reinforced concrete (cast-in-place or shotcrete) at the inside or outside face of the existing walls;

- Providing a fiber-reinforced polymer (FRP) overlay on one or both sides of the existing shear wall;

- Increasing the flexural capacity by positive interpanel connections for vertical shear; and

- Adding interior shear walls to reduce the flexural or shear stress in the existing precast panels.

Precast concrete shear walls generally only have high in-plane shear stress when there are large openings in the wall, and the entire shear force tributary to the wall is carried by a few panels. The most cost-effective solution generally is to infill some of the openings with reinforced concrete. In the case of inadequate interpanel shear capacity, the panels will act independently and can have inadequate flexural capacity. Improving the vertical shear capacity of the connection between panels can improve the overall wall flexural capacity. The last two techniques are generally not cost effective unless a significant overstress condition exists.

(c) Strengthening techniques for inadequate interpanel capacity. Deficient interpanel shear connection capacity of precast concrete wall panels can be improved by:

- Making each panel act as a cantilever to resist in-plane forces (this may be accomplished by adding or strengthening tie-downs, edge reinforcement, footings, etc.); and

- Providing a continuous wall by exposing the reinforcing steel in the edges of adjacent units, adding ties, and repairing with concrete.

The two techniques can be equally effective. The installation of tie-downs and possibly surface-mounted wall reinforcement that will make each panel act as a cantilever is a cost-effective way to compensate for inadequate interpanel capacity, where panels have adequate flexural capacity, and operational and aesthetic requirements for the space can accommodate such installation. Where this is not acceptable, creating a continuous wall by exposing horizontal reinforcing steel and weld-splicing them across panel joints is a viable, although more costly, option. A commonly used technique to increase interpanel capacity is to bolt steel plates across panel joints; however, observations of earthquake damage indicate this technique may not perform acceptably due to insufficient ductility, and its use is not recommended in regions with $S_{DS} \geq 0.25$.

(d) Strengthening techniques for inadequate out-of-plane flexural capacity. Deficient out-of-plane flexural capacity of precast concrete shear walls can be remediated by:

- Providing pilasters at and/or in-between the interpanel joints;
- Providing FRP overlay on both sides of walls; and
- Adding horizontal beams between the columns or pilasters at mid-height of the wall.

The reinforcing in some precast concrete wall panels may have been placed to handle lifting stresses without concern for seismic out-of-plane flexural stresses. A single layer of reinforcing steel, for example, may be placed adjacent to one face of the wall. If this condition exists, new and/or additional pilasters can be provided between the diaphragm and the foundation at a spacing such that the wall will adequately span horizontally between pilasters. FRP overlays on both sides of a precast concrete wall can also significantly enhance the out-of-plane flexural capacity of the wall. In addition, horizontal beams can be provided between the pilasters at a vertical spacing such that the wall spans vertically between the diaphragm and the horizontal beam, or between the horizontal beam and the foundation.

(e) Strengthening techniques for inadequate shear or flexural capacity in coupling beams. Deficient shear or flexural capacity in coupling beams in precast concrete walls can be improved using the techniques identified for correcting the same condition in concrete shear walls. The relative merits of the alternatives for improving the shear or flexural capacity of connecting beams in precast concrete coupling beams are similar to those discussed for concrete shear walls.

(3) Unreinforced masonry shear walls. Masonry walls include those constructed of solid or hollow units of brick or concrete. Hollow clay tile is also typically classified as masonry. The use of hollow tile generally has been limited to nonstructural partitions, and is discussed in Chapter 9. Strengthening options for unreinforced concrete or masonry buildings are provided in Table 8-2.

(a) Deficiencies. The principal deficiencies of unreinforced masonry shear walls are:

- Inadequate in-plane shear or flexural capacity;
- Inadequate out-of plane flexural capacity of the walls; and
- Inadequate out-of-plane anchorage.

A secondary deficiency is inadequate shear or flexural capacity of the coupling beams.

(b) Strengthening techniques for inadequate in-plane shear and out-of-plane flexural capacity. Deficient in-plane shear or flexural capacity and out-of-plane flexural capacity of unreinforced masonry walls can be improved by:

- Providing additional shear capacity by placing reinforcing steel on the

Table 8-2. Strengthening Options for Unreinforced Concrete or Masonry Buildings

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
a. Shear walls	(1) Inadequate shear or flexural capacity	(a) Fill-in openings	para 8-2a(1)(b)		3.2.1.2a 3.2.1.2b
		(b) Add reinforced concrete to interior or exterior face	para 8-2a(1)(b)	8-2	3.2.1.2c
		(c) Provide FRP overlay to interior or exterior face	para 8-2a(1)(b)	8-1	
		(d) Center coring technique	para 8-2a(3)(b)		3.2.3.2
		(e) Provide supplementary vertical resisting elements	para 8-2a(1)(b)		3.4 and 3.4.2
b. Coupling beams	(1) Inadequate shear or flexural capacity	(a) Fill-in openings	para 8-2a(1)(d)		3.2.1.2a
		(b) Remove and replace	para 8-2a(1)(d)		3.2.1.4
		(c) Accept rotational damage	para 8-2a(1)(d)		
c. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with reinforced concrete	para 8-2f(1)(b)		3.5.2.2
	(2) Inadequate chord capacity	(a) Add new concrete or steel chord	para 8-3a(1)(c)		3.5.2.3 3.5.4.3
	(3) Shear or tensile stresses at openings	(a) Add structural member below the slab	para 8-2f(1)(d)		3.5.2.4a
		(b) Add concrete topping with trim bars	para 8-2f(1)(e)		3.5.2.4b
		(c) Fill-in opening	para 8-2f(1)(e)		3.2.1.2b
	(4) Inadequate wall anchorage	(a) Provide reinforced concrete overlay with dowels	para 8-2f(1)(e)		3.5.2.2

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Table 8-2. Strengthening Options for Unreinforced Concrete or Masonry Buildings

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
		(b Provide new steel member with bolting to wall	para 8-2f(1)(e)		3.5.4.3
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Provide additional fasteners	para 8-2f(5)(b)		
		(b) Provide plywood overlay	para 8-2f(5)(b)	8-14	
	(2) Inadequate chord capacity	(a) Provide continuity splice for joists or fascia	para 8-2f(5)(c)	8-15	
		(b) Provide continuous steel member	para 8-2f(5)(c)	8-16	
	(3) Shear or tensile stresses or openings	(a) Provide new drag struts	para 8-2f(5)(d)		2.2.2.4b
		(b) Provide new plywood overlay with appropriate nailing	para 8-2f(5)(d)	8-14	
	(4) Inadequate stiffness	(a) Provide plywood overlay	para 8-2f(5)(b)	8-14	
		(b) Provide "cross" walls	para 8-2a(3)(d)		
e. Continuous footings	(1) Excessive soil bearing pressures	(a) Add drilled piers	para 8-2g(1)(b)		3.6.1.2b
		(b) Modify soil properties	para 8-2g(1)(b)		
		(c) Underpin existing footing	para 8-2g(1)(b)		3.6.1.2a
	(2) Excessive uplift forces	(a) Add drilled piers	para 8-2g(1)(c)		3.6.1.2b

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inside or outside face of the wall and applying new reinforced concrete overlay (Figure 8-2);

- Providing a fiber-reinforced polymer (FRP) overlay on one or both sides of the existing shear wall (Figure 8-1);

- Providing additional capacity only for out-of-plane lateral forces by adding reinforcing steel to the wall utilizing the center coring technique (FEMA 172, Figure 3.2.3.2);

- Providing additional capacity for out-of-plane lateral forces by adding thin surface treatments (e.g., plaster with wire mesh and Portland cement mortar) at the inside and outside face of existing walls;

- Filling in existing window or door openings with unreinforced concrete or masonry; and

- Providing additional shear walls or a steel bracing system at the interior or perimeter of the building, or providing external buttresses (FEMA 172, Figure 3.1.2.2 c).

Strengthening techniques for inadequate in-plane shear capacity are similar to those discussed above for reinforced concrete or masonry walls, but there is an important difference because of the very low allowable stresses normally permitted for unreinforced masonry. Inadequate in-plane flexural capacity frequently occurs in slender URM piers between window openings when the pier flexural strength is attained prior to the shear strength. Because non-reinforced masonry has minimal tensile strength, URM walls are also very susceptible to flexural failure caused by out-of-plane forces. A common strengthening technique for this deficiency is to construct reinforced concrete pilasters or steel columns anchored to the masonry wall and spanning between the floor diaphragms. The spacing of the pilasters or columns is such that the masonry wall can resist the seismic inertia forces by spanning as a horizontal beam between the pilasters or columns. FRP overlays on both sides of a precast concrete wall can also significantly enhance the out-of-plane flexural capacity of the wall. A recent innovation that has been used on several California projects is the seismic strengthening of unreinforced masonry walls by the center coring technique. This technique consists of removing 4-inch (102 mm) -diameter (+/-) vertical cores from the center of the wall at regular intervals (about 3 to 5 feet [0.90 to 1.53 m]apart) and placing reinforced steel and grout in the cored holes. Polymer cement grout has been used because of its workability, low shrinkage, and penetrating characteristics. The reinforcement has been used with and without post-tensioning. This technique provides a reinforced vertical beam to resist flexural stresses, and the infusion of the polymer grout strengthens the mortar joint in the existing masonry, particularly in the vertical collar joints that generally have been found to be inadequate. This method is a developing technology, and designers contemplating its use should obtain the most current information on materials and installation techniques. The third technique for strengthening out-of-plane capacity of existing walls is to apply thin surface treatments of plaster or Portland cement over welded wire mesh. These treatments should be applied on both faces of existing walls. Filling in existing window and/or door openings can be a cost-effective means of increasing in-plane shear capacity if the architectural and functional aspects of the building can be accommodated. To maintain strain compatibility around the perimeter of the opening, it is desirable that the infill material has physical properties similar to those of the masonry wall.

1. Strain compatibility. Research indicates that it is difficult to maintain strain compatibility between uncracked masonry and cracked reinforced concrete. As a result, when there is a significant deficiency in the in-plane shear capacity of unreinforced masonry walls, some structural engineers prefer to ignore the participation of the existing masonry; to provide out-of-plane support for the masonry; and to design the concrete overlay to resist the total in-plane shear in the overlay panel.

2. Redistribution. New reinforced concrete shear walls may be provided in an existing building to reduce the in-plane shear stresses in the unreinforced masonry walls by redistributing the seismic forces by relative rigidities. It should be noted that this redistribution is most effective when the walls are in the same line of force, and connected by a competent spandrel beam or drag strut. When the new concrete walls are not in the same line of force, and when the diaphragm is relatively flexible with respect to the wall, the redistribution may be by tributary area rather than by relative rigidity, and the benefit of the additional shear wall may not be entirely realized.

(c) Strengthening techniques for inadequate out-of-plane anchorage. Deficient out-of-plane anchorage can be improved only by providing proper anchorage to the floor and roof diaphragms. Proper anchorage details are discussed in paragraph 8-3a(1) and 8-3a(7) for concrete and wood diaphragms.

(d) Special Procedure for unreinforced masonry bearing wall buildings. An alternative methodology has been developed for the evaluation and design of unreinforced masonry bearing wall buildings with flexible wood diaphragms. Initially designated as the "ABK Methodology," it is based on research funded by the National Science Foundation and performed by Agbabian Associates, S. B. Barnes and Associates, and Kariotis and Associates. The procedure for evaluation of unreinforced masonry (URM) bearing wall buildings presented in Section 4.2.6 of FEMA 310 is based on this methodology. Some of the principal differences between this methodology and conventional code provisions are as follows:

- The in-plane masonry walls are assumed to be rigid (i.e., there is no dynamic amplification of the ground motion in walls above ground level);

- The diaphragms and the tributary masses of the out-of-plane walls respond to ground motion through their attachments to the in-plane walls;

- The maximum seismic force transmitted to the in-plane walls by the diaphragm is limited by the shear strength of the diaphragms;

- The diaphragm response is controlled within prescribed limits by cross walls (i.e., existing or new wood sheathed stud walls) or shear walls; and

- Maximum height-to-thickness (h/t) ratios are specified in lieu of flexural calculations for out-of-plane response of the walls.

This Special Procedure is permitted only for buildings in Seismic Use Group I.

b. Moment frames. Moment-resisting systems are vertical components that resist lateral loads primarily through flexure. There are three principal types of moment-resisting systems: steel moment frames, concrete moment frames, and precast concrete moment frames. All of these frames may occur with reinforced or unreinforced concrete or masonry walls.

(1) Steel moment frames. Strengthening options for steel moment-resisting frame buildings are provided in Table 8-3.

(a) Deficiencies. The principal seismic deficiencies in steel moment frames are:

1. Beams
- Inadequate moment capacity.
- Inadequate stiffness (drift).

2. Columns
- Inadequate moment capacity.
- Inadequate stiffness (drift).
- Inadequate tensile capacity at splices.
- Inadequate anchorage at foundation.

3. Beam/column joint
- Inadequate rotation capacity (ductility).
- Inadequate vertical shear capacity.
- Inadequate panel joint shear capacity.

(b) Strengthening techniques for inadequate moment/shear capacity of beams or columns. Deficient moment/shear capacity of the beams or columns can be improved by:

- Increasing the moment capacity of the beams by adding cover plates or other steel sections to the flanges (FEMA 172, Figure 3.1.1.2 c);

- Reducing the stresses in the existing frames by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls); and

- Providing lateral bracing of unsupported flanges to increase capacity limited by tendency for lateral/torsional buckling.

If the existing steel frame members are inaccessible (e.g., they are covered with architectural cladding), the first two techniques usually are not cost-effective.

Table 8-3. Strengthening Options for Steel Moment-Resisting Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
a. Columns and beams	(1) Inadequate moment capacity	(a) Add cover plates to flanges	para 8-2b(1)(b)		3.1.1.2c
		(b) Provide lateral bracing of unsupported flanges	para 8-2b(1)(b)		
		(c) Provide supplemental vertical-resisting elements	para 8-2b(1)(b)		3.4 and 3.4.2
	(2) Inadequate axial load capacity	(a) Box column flanges	para 8-2b(1)(b)		3.1.1.2b
	(3) Inadequate stiffness	(a) Provide boxing or cover plates	para 8-2b(1)(e)		3.1.1.2b 3.1.1.2c
		(b) Provide haunches to beams	para 8-2b(1)(e) and 8-2b(1)(c)		
		(c) Encase columns in concrete	para 8-2b(1)(e)		
		(d) Provide supplemental vertical-resisting elements	para 8-2b(1)(e)		
	(4) Inadequate column splice capacity	(a) Provide additional splice plates and welding	para 8-2c(3)(b)	8-9	
b. Beam/column joints	(1) Inadequate moment capacity	(a) Add cover plates to beam flanges	para 8-2b(1)(c)		3.1.1.2a
		(b) Provide haunches to lower beam flanges	para 8-2b(1)(c)	8-4 and 8-5	
		(c) Add ribs to beam flanges	para 8-2b(1)(c)	8-6	
		(d) Provide supplemental vertical-resisting elements	para 8-2b(1)(b)		
	(2) Inadequate shear capacity	(a) Provide additional welding to shear connection	para 8-2b(1)(b)	8-4 and 8-8	

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Table 8-3. Strengthening Options for Steel Moment-Resisting Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	FEMA 172
	(3) Inadequate panel zone capacity	(a) Provide web doubler plate	para 8-2b(1)(d)		
		(b) Provide stiffener or continuity plates	para 8-2b(1)(d)	8-4 and 8-5	
c. Concrete floor and roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with reinforced concrete	para 8-2f(1)(b)		3.5.2.2
	(2) Inadequate chord capacity	(a) Provide continuity in existing beams	para 8-2f(4)(c)	8-8	
	(3) Shear or tensile stresses at openings	(a) Add structural member below the slab	para 8-2f(1)(e)		3.5.2.4a
		(b) Add concrete topping with trim bars	para 8-2f(1)(e)		3.5.2.4b
		(c) Fill-in opening	para 8-2f(1)(e)		3.2.1.2b
d. Steel deck floor and roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding	para 8-2f(4)(b)		
		(b) Add concrete fill or overlay	para 8-2f(4)(b)		3.5.5.2a
		(c) Provide horizontal bracing	para 8-2f(4)(b)		
	(2) Inadequate shear transfer	(a) Add steel member between joists	para 8-2f(4)(b)	8-12	
	(3) Inadequate chord capacity	(a) Provide continuity in existing beams	para 8-2f(4)(c)	8-8	
e. Spread footings	(1) Excessive soil bearing pressure	(a) Underpin footing	para 8-2g(2)(b)		3.6.1.2a
		(b) Add new piers drilled through footing	para 8-2g(2)(b)	8-17	
		(c) Modify existing soil	para 8-2g(2)(b)		
		(d) Provide tie beams	para 8-2g(2)(b)		

Table 8-3. Strengthening Options for Steel Moment-Resisting Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	FEMA 172
	(2) Excessive uplift forces	(a) Add new piers drilled through footing	para 8-2g(2)(b)	8-17	
		(b) Provide tie beams	para 8-2g(2)(b)		
	(3) Inadequate passive pressure	(a) Enlarge footing	para 8-2g(2)(b)		
		(b) Modify existing soil	para 8-2g(2)(b)		
		(b) Provide tie beams	para 8-2g(2)(b)		
f. Pile or drilled pier foundations	(1) Inadequate tensile or compression capacity	(a) Drive additional piles, remove and replace pile cap	para 8-2g(3)(b)		3.6.3.2
		(b) Provide tie beams	para 8-2g(2)(b)		
	(2) Inadequate lateral load capacity	(a) Modify existing soil	para 8-2g(2)(b)		
		(b) Provide tie beams	para 8-2g(2)(b)		
		(c) Drive additional piles, remove and replace pile cap	para 8-2g(3)(b)		3.6.3.2

The majority of the columns, beams, and connections would need to be exposed; significant reinforcement of the connections and members would be required, and the architectural cladding would have to be repaired. Reducing the excessive stresses by providing supplemental resisting elements usually will be the most cost-effective approach. Providing additional moment frames (e.g., in a building with moment frames only at the perimeter, selected interior frames can be modified to become moment frames, as indicated in Figure 3.1.1.2a of FEMA 172) reduces the stresses on the existing moment frames. Providing supplemental bracing or shear walls also can reduce frame stresses. Concentric bracing in a moment frame system with a rigid diaphragm will attract lateral shear forces from the moment frames because of its greater relative rigidity. Shear walls have the additional disadvantage of requiring additions to or modifications of the existing foundations. The addition of eccentric bracing may be an efficient and cost-effective technique to increase the lateral-load capacity of the deficient frame, provided existing beam sizes are appropriate, and the resulting overturning forces can be resisted. In addition to being compatible with the rigidity of the moment frames, eccentric bracing has the advantage of being more adaptable than concentric bracing or shear walls in avoiding the obstruction of existing door and window openings. If architectural cladding is not a concern, reinforcement of existing members may be practical. The addition of cover plates to beam flanges can increase the moment capacity of the existing beams. Cover or box plates also may increase the moment capacity of the columns at the base, and thereby require that the foundation capacity also be increased. Increasing the moment capacity of columns with cover plates at the beam/column connection usually is not feasible because of the interference of the connecting beams. The addition of haunches below and/or above the beam may be effective for increasing the moment capacity of a deficient moment frame. The effects of the haunches will require a re-analysis of the frame, and the designer must investigate the stresses and the need for lateral bracing at the interface between the haunch and the beam or column. In many cases, it may not be feasible to increase the capacity of existing beams by providing cover plates on the top flange because of interference with the floor beams, slabs, or metal decking. (Note that for a bare steel beam, a cover plate on only the lower flange may not significantly reduce the stress in the upper flange.)

If, however, an existing concrete slab is adequately reinforced and detailed for composite action at the end of the beam, it may be economically feasible to increase the moment capacity by providing cover plates on the lower flanges at each end of the beam. Cover plates should be tapered to avoid an abrupt change in section modulus beyond the point where the additional section modulus is required. In some cases, the capacity of steel beams in rigid frames may be governed by lateral stability considerations. Although the upper flange may be supported for positive moments by the floor or roof system, the lower flange must be checked for compression stability in regions of negative moments. If required, the necessary lateral support may be provided by diagonal braces to the floor system.

(c) Strengthening techniques for inadequate beam/column joints. Current building code provisions for special moment frames require that the vertical shear capacity of the frame/column connection be capable of resisting the gravity loads plus the shear associated with the plastic moment connections were generally designed with shear connections sized for the actual design loads. Strengthening of these deficient connections may be accomplished by welding the connection angles as indicated in Figure 8-4. The 1994 Northridge, California earthquake disclosed the vulnerability of steel-frame beam-to-column joints with full-penetration welds connecting the beam flange to the column flange. This joint detail, which was recommended by the steel industry and accepted as a prequalified detail by many of the building codes, was found to have failed in many of the buildings near the epicenter of this moderately severe earthquake. The failures consisted of cracks in the welds, predominantly in the lower beam flange, and occasionally extending into the beam web and/or into the column flange. Research pertaining to the evaluation, repair, modification, and design of steel moment frames, funded by FEMA, is currently in progress by the SAC joint venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering. Interim Guidelines (FEMA 267), published in 1995, provide guidance for repair of damaged connections and for modification of damaged or undamaged connections. For new buildings, FEMA 302 refers to the AISC Seismic Provisions that require testing of proposed joint details to confirm the required inelastic rotational capability. The steel industry has compiled a number of joint assemblies and welding specifications that have been tested and accepted by building departments. Pending resolution of the indicated uncertainties with the beam flange to column flange weld in existing buildings, when upgrading or strengthening of this connection is required, this document prescribes modification of the connection such that the flexural yielding will occur in the beam rather than at the connection. Several details have been developed to accomplish this objective, including:

- Tapered cover plates welded to the beam flanges and to the column flange (FEMA 172, Figure 3.1.1.2 a);

- Steel tee or wide-flange sections welded to the lower flange at the ends of the beam to form a haunched member (Figures 8-4 and 8-5); and

- Tapered upstanding ribs, welded to one or both flanges of the beam to form a haunched member (Figure 8-6).

As indicated above, the objective of these details is to restrict the stresses in the full penetration welds to the column flange by forcing the yield hinge to form in the beam beyond the strengthened portion. Restricting the strengthening to the lower flange eliminates the significant cost and disruption of removing the floor finish and the structural slab or decking to expose the top flange. In this regard, the second technique described above is considered to be the most effective in reducing the stress in both flange welds.

(d) Strengthening techniques for inadequate panel zone capacity. Beam/column panel zones can be overstressed due to seismic forces if the tensile capacity in the column web opposite the beam flange connection is inadequate (i.e., tearing of the column web); if the stiffness of the column flange where beam flange or moment plate weld occurs is inadequate (i.e., lateral bowing of the column flange);

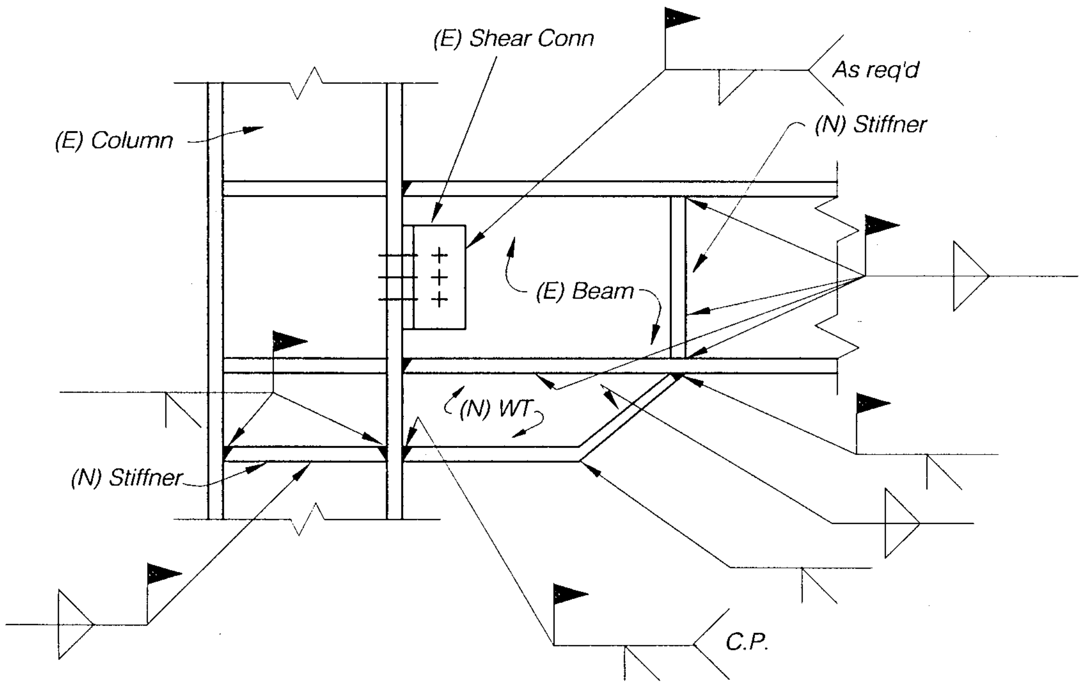
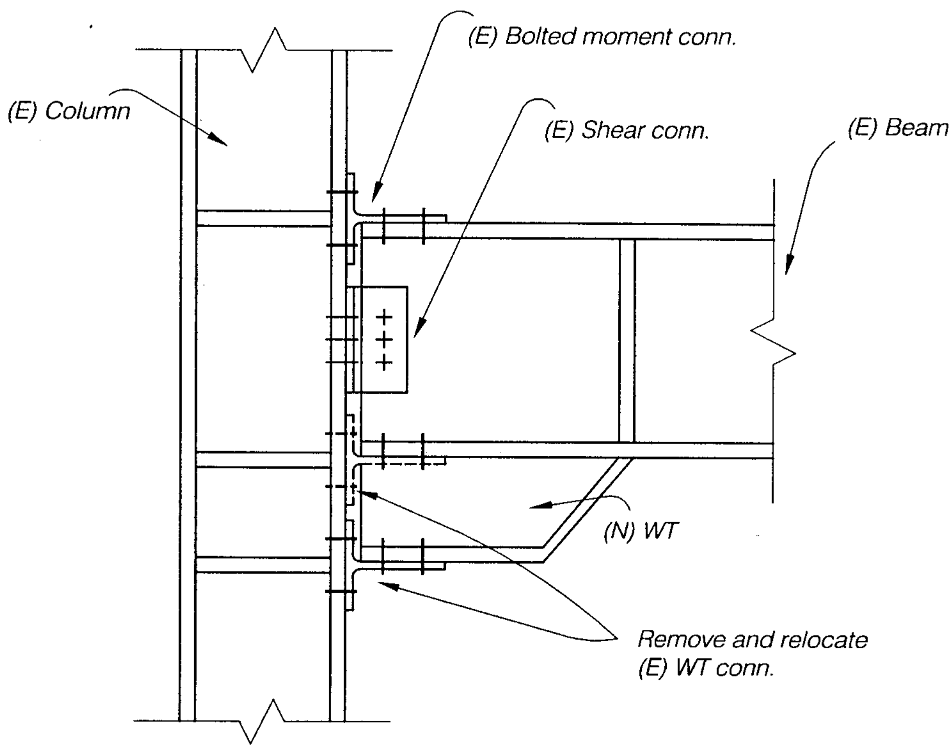


Figure 8-4. Rehabilitation of a Welded Moment Connection



Note: See Figure 8-4 for welding of new WT haunch and stiffener.

Figure 8-5. Rehabilitation of a Bolted Moment Connection

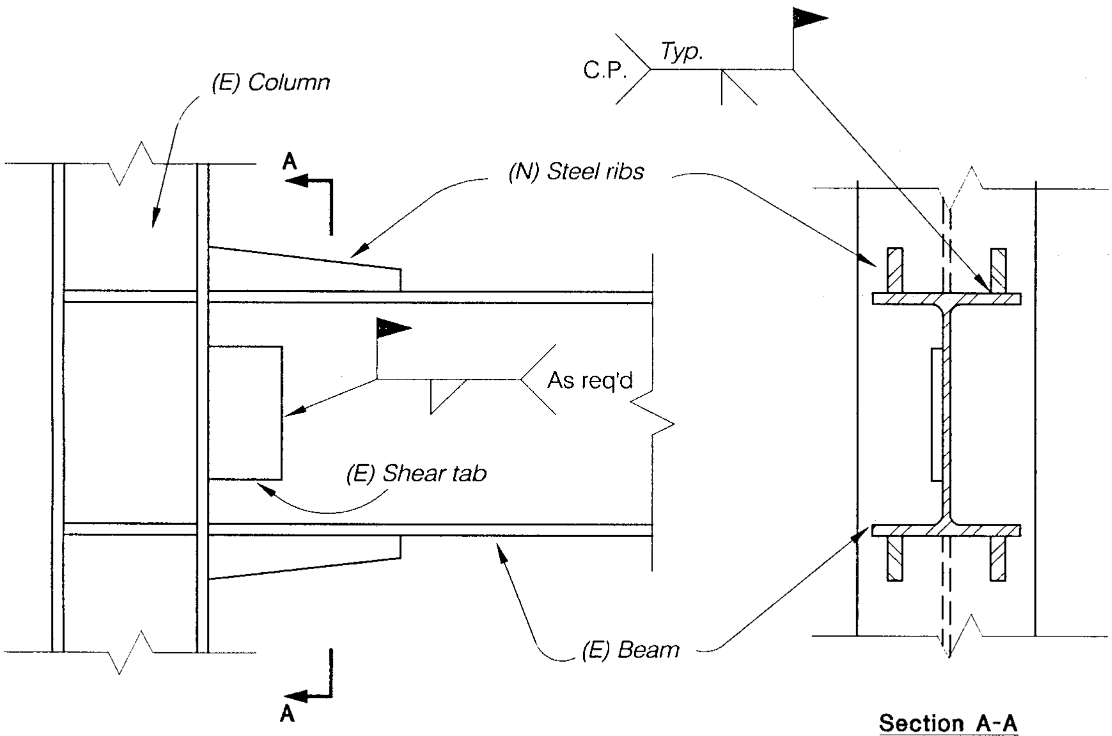


Figure 8-6. Strengthening a Pre-Northridge Moment Connection

capacity of the frame (i.e., $V_{D+LL}+2Mp/L$). Older steel moment frames with bolted or riveted if the capacity for compressive forces in the column web is inadequate (i.e., web crippling or buckling of the column web opposite the compression flange of the connecting beam); or if there is inadequate shear capacity in the column web (i.e., shear yielding or buckling of the column web). Deficient panel zones can be improved by:

- Providing welded continuity plates between the column flanges;

- Providing stiffener plates welded to the column flanges and web;

- Providing web doubler plates at the column web; and

- Reducing the stresses in the panel zone by providing supplemental vertical-resisting components (i.e., additional moment frames, braces, or shear walls).

Adding stiffener plates to the panel zone usually is the most cost-effective alternative. It should be noted that this technique corrects three of the four deficiencies identified above. Also, by confining the column web in the panel zone, shear buckling is precluded, and shear yielding in the confined zone may be beneficial by providing supplemental damping if the additional drift is acceptable. The cost for removal and replacement of existing architectural cladding and associated fireproofing needs to be considered in assessing cost-effectiveness.

(e) Techniques for reducing drift.

- Increasing the capacity, and therefore the stiffness, of the existing moment frame by cover plates or boxing (FEMA 172, Figure 3.1.1.2 b);

- Increasing the stiffness of the beams and the columns at their connections by providing haunches;

- Reducing the drift by providing supplemental vertical-resisting components (i.e., additional moment frames, braces, or shear walls);

- Increasing the stiffness by encasing columns in reinforced concrete; and

- Reducing the drift by adding supplemental damping.

Excessive drift generally is a concern in the control of seismic damage; however, for steel frames, there also may be cause for concern regarding overall frame stability. If the concern is excessive drift and not frame capacity, the most cost-effective alternative typically is increasing the rigidity of the system by the addition of bracing or shear walls; however, the critical elements in the system now will probably be the bracing or shear walls because of their greater relative rigidity, as compared to the moment frames. Providing steel gusset plates to increase stiffness and reduce drift may be cost- effective in some cases. This technique, however, must be used with caution, since the gussets may increase column-bending stresses, and increase the chance for a nonductile failure. Column and beam stresses must therefore be checked where beams and columns interface with gussets or haunches, and column stability under a lateral displacement associated with the design earthquake should be verified. Increasing the stiffness of steel columns by encasement in concrete may be an alternative for reducing drift in certain cases. The principal contributing element to excessive story drift typically is beam flexibility; hence, column concrete encasement will only be partially effective, and is therefore only cost-effective when a building has relatively stiff beams and flexible columns. Boxing a column or cover-plating a beam is an effective technique to increase column and beam stiffness. The additional stiffness achieved in the beams or columns need not increase connection moment capacities if the boxing and cover plates are terminated a distance of at least one-half the member depth from the face of the joint. Reducing drift by adding supplemental damping is an alternative that is now being implemented in some seismic rehabilitation projects. Typically, bracing elements need to be installed in the moment frame so that discrete dampers can be located between the flexible moment frame elements and the stiff bracing elements.

(2) Concrete moment frames. Strengthening options for reinforced concrete moment frame buildings are provided in Table 8-4.

(a) Deficiencies. The principal deficiencies in concrete moment frames are:

- Inadequate shear or flexural capacity in the columns or beams;
- Inadequate joint shear capacity; and
- Inadequate development length at reinforcement splices or anchorages.

(b) Strengthening techniques for inadequate shear or flexural capacity in columns or beams. Deficient shear or flexural capacity in columns or beams of concrete moment frames can be improved by:

- Increasing the shear and flexural capacity by providing concrete jackets with additional transverse and flexural reinforcement (FEMA 172, Figures 3.1.2.2 a and 3.1.2.2 b);
- Increasing the shear and/or flexural capacity of beams or columns by confinement with reinforced concrete, steel, or fiber-reinforced polymer (FRP) sheets (Figure 8-7);
- Reducing the seismic stresses in the existing frames by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls); and
- Changing the system to an infilled shear wall system by infilling the reinforced concrete frames with reinforced concrete (FEMA 172, Figure 3.1.2.2c).

Improving the ductility and strength of concrete frames by jacketing with additional concrete generally is not cost-effective because of the difficulty associated with providing the necessary confinement and shear reinforcement in the beams,

Table 8-4. Strengthening Options for Reinforced Concrete Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure: This Document	Applicable Figure: FEMA 172
a. Frames	(1) Inadequate shear or flexural capacity	(a) Provide reinforced concrete jackets	para 8-2b(2)(b)		3.1.2.2a 3.1.2.2b
		(b) Confine with steel jackets	para 8-2b(2)(b)		
		(c) Overlay with FRP sheets	para 8-2b(2)(b)	8-7	
		(d) Provide supplemental vertical resisting elements	para 8-2a(1)(b)		3.4 and 3.4.2
	(2) Inadequate joint shear capacity	(a) In-fill the frames with reinforced concrete walls.	para 8-2b(2)(b)		3.1.2.2c
		(b) Provide reinforced concrete jackets	para 8-2b(2)(b)		3.1.2.2b
		(c) Provide supplemental vertical resisting elements	para 8-2a(1)(b)		3.4 and 3.4.2
	(3) Inadequate development lengths	(a) Provide reinforced concrete jackets	para 8-2b(2)(b)		3.1.2.2b
		(b) Confine with steel or FRP	para 8-2b(2)(b)		
		(c) Reduce allowable capacity	para 8-2b(2)(d)		
b. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with reinforced concrete	para 8-2f(1)(b)		3.5.2.2
	(2) Inadequate chord capacity	(a) Add new concrete or steel chord	para 8-2f(1)(d)		3.5.2.3 and 3.5.4.3
	(3) Shear or tensile stresses at openings	(a) Add new structural member below the slab	para 8-2f(1)(d)		3.5.2.4.a
c. Steel deck floor or roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding	para 8-2f(4)(b)		
		(b) Add concrete fill or overlay	para 8-2f(4)(b)		3.5.5.2.a
		(c) Provide horizontal bracing system	para 8-2f(4)(b)		3.5.5.2.b

Table 8-4. Strengthening Options for Reinforced Concrete Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure: This Document	Applicable Figure: FEMA 172
	(2) Inadequate shear transfer	(a) Add new steel member between supports	para 8-2f(4)(b)	8-12	
d. Spread footings	(1) Excessive soil bearing pressure	(a) Underpin footing	para 8-2g(2)(b)		3.6.1.2a
		(b) Add new piers drilled through footing	para 8-2g(2)(b)	8-17	
		(c) Modify existing soil	para 8-2g(2)(b)		
		(d) Provide tie beams	para 8-2g(2)(b)		
	(2) Excessive uplift forces	(a) Add new piers drilled through footing.	para 8-2g(2)(b)	8-17	
		(b) Provide new tie beams	para 8-2g(2)(b)		
	(3) Inadequate passive pressure	(a) Enlarge footing	para 8-2g(2)(b)		
		(b) Modify existing soil	para 8-2g(2)(b)		
		(c) Provide new tie beams	para 8-2g(2)(b)		
e. Pile or drilled pier foundations	(1) Inadequate tensile or compression capacity	(a) Provide new tie beams	para 8-2g(2)(b)		3.6.3.2
		(b) Drive additional piles, remove and replace pile cap	para 8-2g(3)(b)		
	(2) Inadequate lateral load capacity	(a) Modify existing soil	para 8-2g(2)(b)		
		(b) Provide new tie beams	para 8-2g(2)(b)		
		(c) Enlarge the pile cap	para 8-2g(3)(b)		
		(d Drive additional piles, remove and replace pile cap	para 8-2g(3)(b)		3.6.3.2

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Figure 8-7. FRP Overlay to Enhance the Shear and Flexural Capacity of Concrete Beams

columns, and beam-column connection zones. When deficiencies are identified in these frames, it will probably be more cost-effective to consider adding reinforced concrete shear walls or filling in the frames with reinforced concrete. Either of these alternatives will tend to reduce the lateral load resisted by frame action. This is because the greater rigidity of the walls will increase the percentage of the lateral load to be resisted by the walls (i.e., lateral forces will be attracted away from the relatively flexible moment frames and into the more rigid walls). This is especially true for buildings with rigid diaphragms. These alternatives also typically will require upgrading of the foundations, which may be costly. The seismic performance of frames with infilled walls is discussed in paragraph 8-2c. The decision regarding whether the new walls should be in the interior of the building or at its perimeter, or as exterior buttresses, usually will depend on nonstructural considerations such as aesthetics and disruption or obstruction of the functional use of the building. Recent developments in California, associated with seismic retrofit of elevated freeway structures, have promoted the use of steel jackets to increase the shear capacity and confinement of concrete columns. Circular and oval steel jackets have been successfully used by Caltrans based on the results of prior research. FRP wrapping of concrete beams and columns in buildings (Figure 8-7) has also had limited application and testing. While test results indicate reasonable effectiveness of the wrapping, resistance to long-term weathering or deterioration has not been fully established.

(c) Strengthening techniques for inadequate joint shear capacity. Techniques for improving the shear capacity of concrete moment frame joints are similar to those for improving the shear or flexural capacity of columns and beams. Jacketing of the joint area is even more difficult and labor-intensive than the jacketing of the frame members because of the need to drill holes to install new transverse reinforcement through the existing beams framing into the joint. FRP wrapping has also been used to increase joint shear capacity. Limited testing has been performed for specific applications, but consensus details and analytical procedures are not yet available. The other two alternative techniques described above for columns and beams will usually also be more cost-effective for the frame joints.

(d) Strengthening techniques for inadequate development length for reinforcement splices and anchorages. ACI 318 specifies minimum development lengths for reinforcement splices and anchorages with factors to increase the minimum length if confinement by transverse reinforcement or the concrete cover over the bar is inadequate. If the existing bar development lengths conform to the minimum ACI 318 requirements, the adverse effects of inadequate confinement or concrete cover can be mitigated by jacketing as described above for columns and beams. FRP wrapping has been successfully used to provide additional confinement for longitudinal reinforcement splices in columns. As indicated in the preceding paragraph, the use of FRP for beams and beams/column joints has been based on limited testing of specific applications. If the development lengths do not conform to the minimum ACI 318 requirements, the reinforcement must be considered as nonconforming, and the allowable stresses reduced proportionately.

(3) Flat slab/column frames. Flat slab systems, with or without column capitals or drop

panels, are a common structural system in older commercial buildings. Many of these systems were designed primarily for gravity loads.

(a) Deficiencies. Common deficiencies include:

- Inadequate shear capacity, usually inadequate punching shear capacity adjacent to columns; and
- Inadequate flexural capacity, usually inadequate flexural reinforcement for positive seismic moments at columns.

(b) Strengthening techniques for the above deficiencies are provided in Moehle, Nicoletti, and Lehman, 1994; however, these techniques are very invasive and labor-intensive. The most cost-effective strengthening technique for these systems will generally be the addition of supplemental shear walls or bracing, rather than strengthening the slab/column system to resist seismic forces.

(4) Precast concrete moment frames.

(a) Deficiencies. Existing precast concrete moment frames may exhibit the same deficiencies as the cast-in-place moment frames. The principal additional deficiency of precast concrete moment frames is inadequate capacity and/or ductility of the joints between the precast units.

(b) Strengthening techniques for the additional deficiencies in precast concrete moment frames. Deficient capacity and ductility of precast concrete moment frame connections can be improved by:

- Removing existing concrete in the precast elements to expose the existing reinforcing steel; providing additional reinforcing steel welded to the existing steel (or drilled and grouted); and replacing the removed concrete with cast-in-place concrete.
- Reducing the forces on the connections by providing supplemental vertical-resisting components (i.e., additional moment frames, braces, or shear walls) as discussed in paragraph 8-2b(4)(b).

Reinforcing the existing connections as indicated in the first technique is not cost-effective because of the difficulty associated with providing the necessary confinement and shear reinforcement in the connections. Providing supplemental frames or shear walls generally is more cost-effective; however, the two alternatives may be utilized in combination.

c. *Frames with infills.* Reinforced concrete or structural steel moment frames that are monolithic with or completely encased in reinforced concrete walls will tend to perform as shear walls with boundary members. In older existing buildings, the infill materials generally consist of reinforced or unreinforced masonry (i.e., brick, concrete blocks, or hollow clay tile). The performance of these infilled frames, as discussed in this paragraph, has been found to be best represented with the development of assumed diagonal compression struts, as indicated in Figure C7-1 of FEMA 274. The resulting mechanism is similar to that of a braced frame, and the stiffness is greater than the sum of the infill and frame stiffnesses considered separately. Modeling and analysis of the infill and the frame are provided in Section 7.5.2 of FEMA 273, and illustrated in Figures C7-1 through C7-5 of FEMA 274.

(1) Deficiencies. The principal deficiencies in moment frames with infill walls are:

- Inadequate shear strength of the infill;

- Crushing of the infill at the upper and lower corners due to the diagonal compression strut action in the infill wall;

- Shear failure of the beam/column connection in the steel frames, or direct shear transfer failure of the beam or column in concrete frames;

- Tensile failure of the columns, their connections, or lap splices due to the uplift forces resulting from the braced-frame action induced by the infill;

- Splitting of the infill due to the orthogonal tensile stresses developed in the diagonal compressive strut; and

- Loss of infill by out-of-plane forces due to loss of bearing or excessive slenderness of the infill wall.

(a) Partial-height infills or infills with door or window openings will tend to brace concrete or steel frames, and the system will resist lateral forces in a manner similar to that of a knee-braced frame. The lateral stiffness of the shortened columns is increased so that, for a given lateral displacement, a larger shear force is developed in the shortened column compared to that in a full-height column. If the column is not designed for this condition, shear failure of the column could occur, particularly in concrete frames, in addition to the other potential deficiencies indicated above for completely infilled frames.

(b) Delamination. In some cases, the exterior face of the infill may extend beyond the edge of the concrete or steel frame columns or beams. For example, an unreinforced brick infill in a steel frame may have one wythe of brick beyond the edge of the column or beam flange to form a uniform exterior surface. This exterior wythe is particularly vulnerable to delamination or splitting at the collar joint (i.e., the vertical mortar joint between the wythes of brick), as the infilled frame deforms in response to lateral loads. In the modeling and analysis of these walls, only the portion of the wall bounded by the frame shall be considered as the effective thickness of the infill. The masonry wythes that are beyond the plane of the frame shall be considered as veneer, and shall be adequately anchored to the infill. Because the in-plane deformation of completely infilled frames is very small, the potential for delamination is greater for partial infills or those with significant openings. The potential life-safety hazard for this condition should be evaluated, and may be mitigated as described above.

(c) Loss of infill. Falling debris resulting from the failure of an existing infill wall also poses a life-safety hazard. Frames may be infilled with concrete or various types of masonry such as solid masonry, hollow clay tile, or gypsum masonry. These infills may be reinforced, partially reinforced, or unreinforced. Infills (particularly brittle unreinforced infills such as hollow clay tile or gypsum masonry) often become dislodged upon failure of the wall in shear. Once dislodged, the broken infill may fall and become a life-safety hazard, or may preclude safe egress from the building. Mitigation of this hazard can be accomplished by removing the infill and replacing it with a nonstructural wall as described above. The infill can also be "basketed" by adding a constraining member such as wire mesh. Basketing will not prevent the infill from failing, but will prevent the debris from falling. Unreinforced infills that comply with the h/t ratio for URM walls in Table 4-2 of FEMA 310 may be considered to be adequate for out-of-plane forces, provided that the top of the infill is in full contact with the soffit of the frame beams. Infills that have excessive h/t ratios may be enhanced with FRP sheets on both sides of the wall.

(2) Strengthening techniques for inadequate shear capacity of infill walls. Inadequate shear capacity in the infilled walls of moment frames can be improved by:

- Eliminating the hazardous effects of the infill by providing a gap between the infill and the frame and providing out-of-plane support; and

- Correcting the deficiencies as prescribed for inadequate shear capacity of reinforced concrete or masonry shear walls in paragraph 8-2a(1)(b).

If the frame with a partial-height infill wall has adequate capacity for the prescribed forces without the infill wall, the most expedient correction is to provide a resilient joint between the column and wall to allow the deformation of the column to take place without restraint. This may be accomplished by cutting a gap between the wall and the column, and filling it with resilient material (out-of-plane restraint of the infill still must be provided), or by removing the infill wall and replacing it with a nonstructural wall that will not restrain the column. If the frame has insufficient capacity for the prescribed forces without the infill, consideration should be given to completely in-filling an adequate number of framed bays, or providing supplemental vertical-resisting elements. For the infill to be effective, it must be in tight contact with the frame columns and beam soffits. The relative rigidities of the shear wall and moment frames in other bays must be considered when distributing the lateral loads, and evaluating the wall and frame stresses.

(3) Strengthening techniques for other deficiencies. Deficiencies pertaining to concrete frame members may be rehabilitated as described for concrete moment frames in paragraph 8-2b(2); however, the presence of the infill makes it more difficult to access the frame for the remedial work. It may be more expedient and cost effective to consider the addition of supplemental shear walls to reduce the forces on the deficient infill walls. Deficiencies in steel frames are more easily addressed.

(a) Inadequate shear capacity in a steel-beam web shear connection can be improved by welding the connection angles as indicated in Figure 8-8. If the connecting bolts are ASTM A325, the bolt capacity may be combined with weld capacity. If the bolts are ASTM A307, which are more prevalent in the older buildings, the welds must be designed to resist the entire shear.

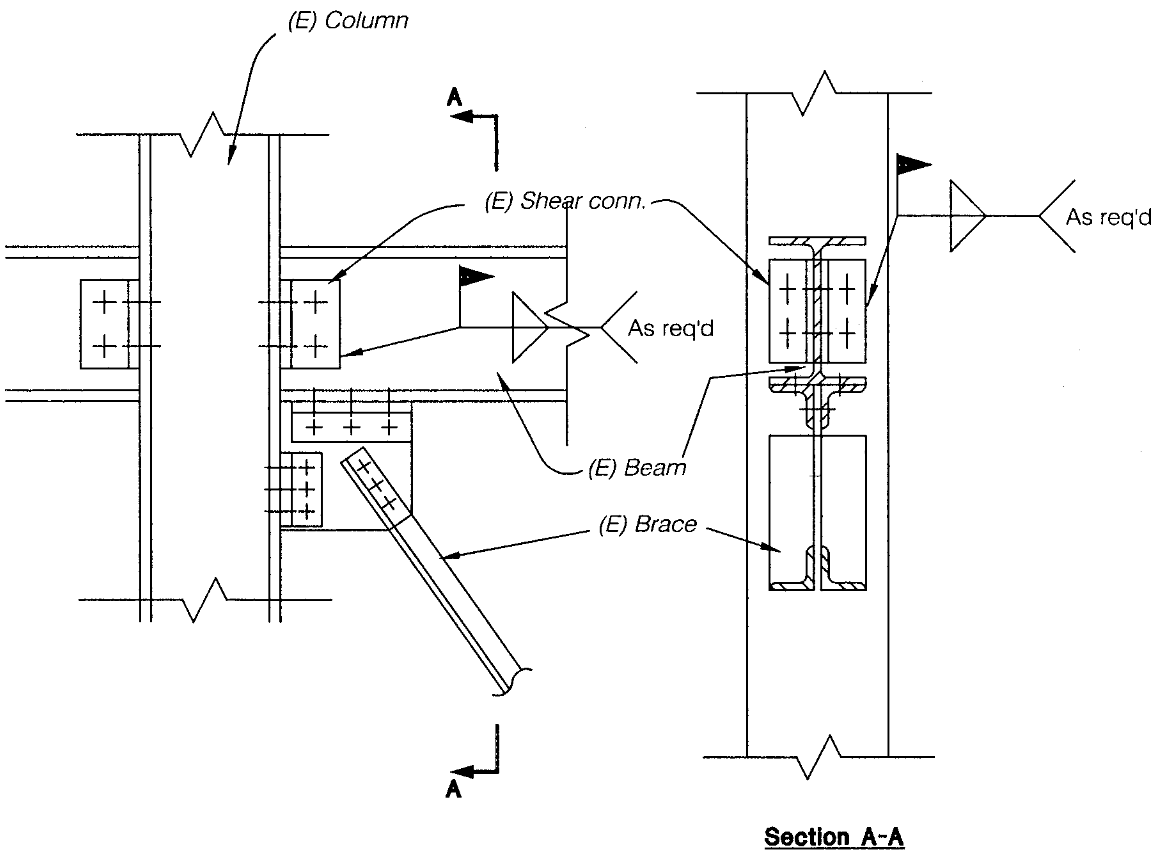

Figure 8-8. Strengthening a Beam Shear Connection

(b) Inadequate tensile capacity in a column splice. Steel columns in older buildings designed for minimal seismic force were typically spliced by milling of the ends for direct bearing with bolted splice plates, connecting the web and/or the flanges. Columns in buildings designed for more significant seismic forces may have milled webs with bolted web splice plates and flanges spliced with partial penetration welds. In either case, additional tensile capacity can be achieved by welding the existing splice plates, or by adding new welded splice plates as indicated in Figure 8-9. Inadequate tensile capacity in a concrete frame column may be mitigated by exposing the reinforcement and welding the lap splices, or by jacketing with reinforced concrete or FRP.

d. Braced frames. Braced frames are vertical elements that resist lateral loads through tension or compression braces. Braced frames are classified as having either concentric or eccentric bracing. The use of eccentric braced frames in seismic design is rather recent, and it is unlikely that those buildings would be candidates for retrofit in the military seismic hazard mitigation program. Concentric bracing may consist of single or double diagonals, chevrons, or K-braces. K-bracing has undesirable performance characteristics for seismic loads in that buckling of the compression brace results in an unbalanced horizontal force on the column from the remaining tension brace. Some building codes permit K-bracing only in low seismic zones, where there is a small probability of exceedence for the design seismic forces. In the higher seismic zones, these braces should be removed and the system modified to one of the other bracing configurations; further, this should be done in all other seismic zones if possible.

Chevron bracing has similar characteristics in that buckling of one brace in compression results in an unbalanced tensile force from the remaining brace. With chevron bracing, the unbalanced force occurs on the beam rather than on the column. Nonetheless, the unbalanced tensile brace reaction should be considered in the rehabilitation, particularly in the case of the inverted V configuration in which the unbalanced force is additive to the gravity loads supported by the beam. Braced frames are typically of steel construction; however, concrete-braced frames are occasionally constructed. Strengthening options for steel-braced frame buildings are provided in Table 8-5.

(1) Deficiencies. The principal deficiencies of steel concentrically braced frames are:

- Inadequate lateral force capacity of the bracing system governed by buckling of the compression brace;

- Inadequate capacity of the brace connection;

- Inadequate axial load capacity in the columns or beams of the bracing system; and

- Brace configuration that results in unbalanced tensile forces, causing bending in the beam or column when the compression brace buckles.

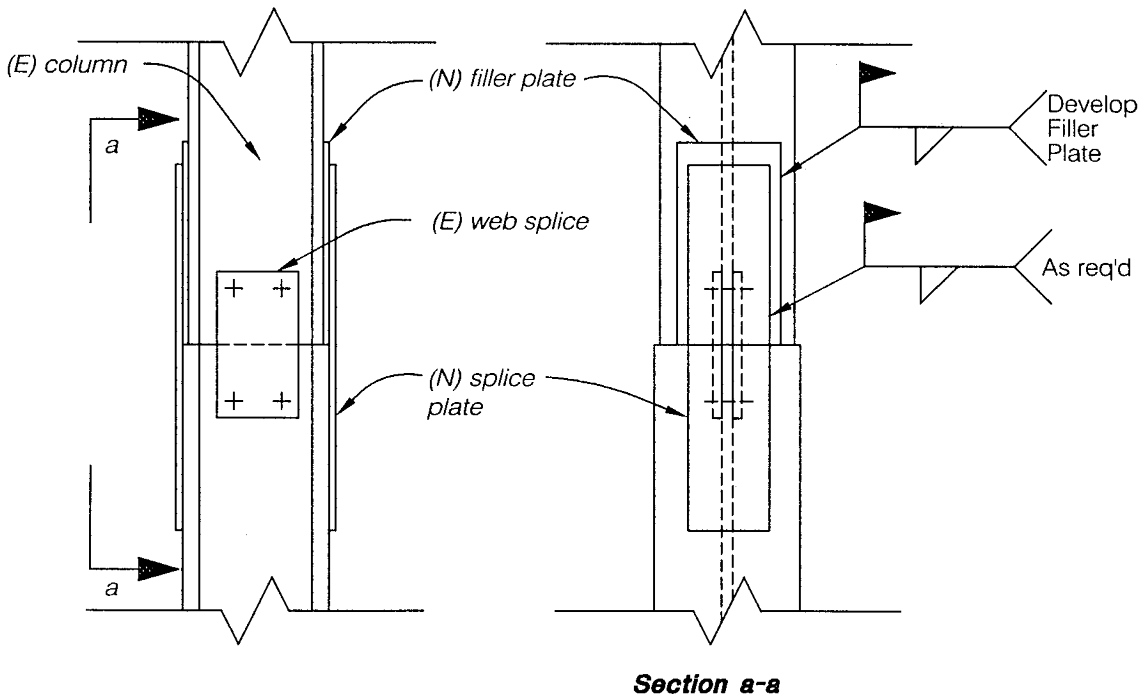

Figure 8-9. Strengthening of a Column Splice

Table 8-5. Strengthening Options for Steel-Braced-Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
a. Steel frame	(1) Inadequate shear capacity in beam connections	(a) Provide additional welding to shear connection	para 8-2b(1)(c)	8-4 and 8-8	
	(2) Inadequate column axial load capacity	(a) Box column flanges	para 8-2b(1)(e)		3.1.1.2b
		(b) Provide supplemental vertical resisting elements.	para 8-2a(1)(b)		3.4 and 3.4.2
	(3) Inadequate column splice capacity	(a) Provide additional splice plates and welding.	para 8-2c(3)(b)	8-9	
	(4) Inadequate shear or uplift capacity at base	(a) Extend base plate and add anchor bolts	para 8-3b(2)(b)	8-20	
b. Bracing	(1) Inadequate in-plane compression capacity	(a) Provide secondary bracing	para 8-2d(2)	8-11	
	(2) Inadequate tensile or compression capacity	(a) Double single member bracing or "star" double angle bracing	para 8-2d(2)	8-10	3.3.1.2
		(b) Provide supplemental vertical resisting elements	para 8-2a(1)(b)		3.4 and 3.4.2
	(3) Inadequate brace connection	(a) Provide additional welding	para 8-2d(3)		
		(b) Remove and replace with stronger connection	para 8-2d(3)		
c. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with reinforced concrete	para 8-2f(1)(b)		3.5.2.2
	(2) Inadequate chord capacity	(a) Add new steel or concrete chord	para 8-2f(1)(c)		3.5.2.3 and 3.5.4.3
	(3) Shear or tensile stresses at openings	(a) Add structural member below the slab	para 8-2f(1)(d)		3.5.2.4a
		(b) Add concrete topping with trim bars	para 8-2f(1)(e)		3.5.2.4b
		(c) Fill-in opening	para 8-2f(1)(e)		3.2.1.2b

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Table 8-5. ... Frame Buildings

Structural Component	Deficiency	Strengthening Technique	Reference	Applicable Figure This Document	Applicable Figure FEMA 172
d. Steel deck floor or roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding	para 8-2f(4)(b)		
		(b) Add concrete fill in overlay	para 8-2f(4)(b)		3.5.5.2a
		(c) Provide horizontal bracing	para 8-2f(4)(b)		3.5.5.2b
	(2) Inadequate shear transfer	(a) Add steel member between joists	para 8-2f(4)(b)	8-12	
e. Spread footings	(1) Excessive soil bearing pressure	(a) Underpin footing	para 8-2g(2)(b)		3.6.1.2a
		(b) Add new piers drilled through footing	para 8-2g(2)(b)	8-17	
		(c) Modify existing soil	para 8-2g(2)(b)		
		(d) Provide tie beams	para 8-2g(2)(b)		
	(2) Excessive uplift forces	(a) Add new piers drilled through footing	para 8-2g(2)(b)	8-17	
		(b) Provide new tie beams	para 8-2g(2)(b)		
	(3) Inadequate passive pressure	(a) Enlarge footing	para 8-2g(2)(b)		
		(b) Modify existing soil	para 8-2g(2)(b)		
		(c) Provide new tie beams	para 8-2g(2)(b)		
f. Pile or drilled pier foundations	(1) Inadequate tensile or compression capacity	(a) Provide new tie beams	para 8-2g(2)(b)		
		(b) Drive additional piles, remove and replace pile cap	para 8-2g(3)(b)		3.6.3.2
	(2) Inadequate lateral load capacity	(a) Modify existing soil	para 8-2g(2)(b)		
		(b) Provide new tie beams	para 8-2g(2)(b)		
		(c) Enlarge the pile cap	para 8-2g(3)(b)		
		(d) Drive additional piles, remove and replace pile cap	para 8-2g(3)(b)		3.6.3.2

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(2) Strengthening techniques for inadequate brace capacity. Deficient brace compression capacity can be improved by:

- Increasing the capacity of the braces by adding new members, thus increasing the area and reducing the radius of gyration of the braces (Figure 8-10);

- Increasing the capacity of the member by reducing the unbraced length of the existing member by providing secondary bracing (Figure 8-11);

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

- Reducing the loads on the braces by providing supplemental vertical-resisting components (i.e., shear walls, bracing, or eccentric bracing) as discussed in paragraph 8-1a(3).

A brace member may be designed to resist both tension and compression forces, but its capacity for compression forces is limited by potential buckling, and is therefore less than the capacity for tensile forces.

Since the design of the system is generally based on the compression capacity of the brace, some additional capacity may be obtained by simply reducing the unsupported length of the brace by means of secondary bracing, as shown in Figure 8-11, provided the connections have adequate reserve capacity, or can be strengthened for the additional loads. If significant additional bracing capacity is required, it will be necessary to consider strengthening or replacement of the brace. Single-angle bracing can be doubled; double-angle bracing can be "starred"; channels can be doubled; and other rolled sections can be cover-plated. New sections should be designed to be compact, if possible, since they will perform with significantly more ductility than noncompact sections. These modifications probably will require strengthening or redesign of the connections. The other members of the bracing system (i.e., columns and beams) must be checked for adequacy with the new bracing loads. Strengthening of existing K- or chevron bracing should be undertaken only after careful evaluation of the additional bending forces following the buckling of the compression bracing. Where the existing bracing in these systems is found to have inadequate capacity, the preferred solution is to replace it with a diagonal or cross-bracing configuration. It is usually a good idea to limit the strengthening of the existing bracing to the capacity of the other members of the bracing system and the foundations, and to provide additional bracing if required. An alternative would be to provide new shear walls or eccentric bracing. Construction of supplemental shear walls may be disruptive, and probably will require new foundations. The greater rigidity of the shear walls as compared with that of the bracing also may tend to make the existing bracing relatively ineffective; thus, the most cost-effective alternatives are considered to be strengthening the existing bracing, or providing additional concentric bracing.

(3) Strengthening techniques for inadequate capacity of the brace connection. Deficient brace connection can be improved by: