3

SEISMIC STRENGTHENING OF EXISTING BUILDINGS

3.0 INTRODUCTION

The life-safety hazard posed a building found to be vulnerable to earthquake ground motion can be mitigated in several ways: the building can be condemned and demolished or strengthened or otherwise modified to increase its capacity or the seismic demand on the building can be reduced. Structural rehabilitation or strengthening of a building can be accomplished in a variety of ways, each with specific merits and limitations related to the unique characteristics of the building.

This chapter focuses on the structural considerations of seismic strengthening or upgrading; however, it must be remembered that other factors may influence or even dictate which technique is most appropriate for an individual building. Recommendations for enhancing the seismic resistance of existing structures by eliminating or reducing the adverse effects of design or construction features were presented in Chapter 2. Cost, function, aesthetic, and seismic zone considerations that also influence the selection of a strengthening technique are reviewed briefly below and are elaborated on in the remaining sections if this chapter. It should be noted, however, that seismic strengthening may trigger application of other building rehabilitation requirements such as those related to handicap access, asbestos, fire sprinklers, fire resistance, and egress.

3.0.1 COST CONSIDERATIONS

Cost is very important and often may be the only criterion applied when choosing among equivalent strengthening options. When using relative costs to evaluate two or more feasible strengthening or rehabilitation alternatives, it is important to consider all applicable costs. For example, an existing steel frame building, with steel floor and roof decking and vertical bracing in the exterior walls may have inadequate seismic shear capacity in the diaphragms and vertical bracing. Although it may be feasible to increase the capacity of the existing diaphragms and the bracing, it may be more cost-effective to add bracing to the interior frames to reduce the diaphragm shears to an allowable level. If additional bracing can be installed without additional foundations and without adverse effects on the functional use of the building, it may be significantly more economical than any of the diaphragm strengthening techniques.

3.0.2 FUNCTIONAL CONSIDERATIONS

Most buildings are intended to serve one or more functional purposes (e.g., to provide housing or to enclose a commercial or industrial activity). Since the functional requirements are essential to the effective use of the building, extreme care must be exercised in the planning and design of structural modifications to ensure that the modifications will not seriously impair the functional use. For example, if new shear walls or vertical concentrically braced frames are required, they must be located to minimize any adverse effect on access, egress, or functional circulation within the building. When considering alternative structural modifications for an existing building with an ongoing function, the degree to which construction of the proposed alternative will disrupt that function also must be considered in assessing cost-effectiveness.

3.0.3 AESTHETIC CONSIDERATIONS

In some cases, the preservation of aesthetic features can significantly influence the selection of a strengthening technique. Historical buildings, for example, may require inconspicuous strengthening designed to preserve historical structural or nonstructural features. Other buildings may have attractive or architecturally significant facades, entrances, fenestration, or ornamentation that require preservation.

A decrease in natural light caused by the filling in of window or skylight openings or the installation of bracing in front of these openings may have an adverse effect on the occupants of the building. Also, the need for preservation of existing architectural features may dictate the location and configuration of the new bracing system. In many such cases, the engineer may not be able to assign an appropriate value to these subjective considerations; however, any additional costs involved in preserving aesthetic features can be identified so that the building owner can make an informed decision.

3.0.4 SEISMIC RISK

The *NEHRP Recommended Provisions* contains seismic zonation maps that divide the United States into seven seismic zones ranging from effective seismic accelerations of 0.05*g* to 0.40*g*. Seismic strengthening may be required for older structures built before the advent of seismic codes or built under less stringent requirements (i.e., seismic force levels in most codes have escalated and the seismic zoning in many areas has been revised upward). However, since these structures were designed for and have been tested over time by vertical loads and wind forces, it is safe to assume that they have some inherent capacity for resisting seismic forces.* Obviously, older existing structures located in a lower seismic zone have a higher probability of requiring little or no strengthening than do similar structures in a higher zone. Further, some strengthening techniques for existing structures with moderate seismic deficiencies in the lower seismic zones are not appropriate for use in higher zones.

In lower seismic zones it sometimes can be demonstrated that a building does not require seismic strengthening because it can resist wind loads in excess of the code-prescribed seismic forces. For other buildings in low seismicity zones, more detailed structural evaluations may be warranted if there is a probability that the seismic adequacy of the structure can be demonstrated.

3.1 VERTICAL-RESISTING ELEMENTS--MOMENT RESISTING SYSTEMS*

Moment resisting systems are vertical elements that resist lateral loads primarily through flexure. There are four principal types of moment resisting systems: steel moment frames, concrete moment frames, precast concrete moment frames; and moment frames with infill walls.

3.1.1 STEEL MOMENT FRAMES

3.1.1.1 Deficiencies

The principal seismic deficiencies in steel moment frames are:*

- Inadequate moment/shear capacity of beams, columns, or their connections;
- Inadequate beam/column panel zone capacity; and
- Excessive drift.

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*The American Iron and Steel Institute has written a minority opinion concerning the footnoted sentence in Sec. 3.0.4 and the organization of Sec. 3.1 and the American Institute of Steel Construction has written a minority opinion concerning the first sentence in Sec. 3.1.1.1; see page 193.

3.1.1.2 Strengthening Techniques for Inadequate Moment/Shear Capacity of Beams, Columns, or Their Connections

Techniques. Deficient moment/shear capacity of the beams, columns, or the connections of steel moment frames can be improved by:

1. Increasing the moment capacity of the members and connections by adding cover plates or other steel sections to the flanges (Figure 3.1.1.2a) or by boxing members (Figure 3.1.1.2b).

2. Increasing the moment/shear capacity of the members and connections by providing steel gusset plates or knee braces.

3. Reducing the stresses in the existing frames by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls) as discussed in Sec. 3.4.

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

5. Encasing the columns in concrete.

Relative Merits. If the existing steel frame members are inaccessible (e.g., they are covered with architectural cladding), Techniques 1 and 2 usually are not cost-effective. 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 moment stresses by providing supplemental resisting elements (Technique 3) 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) reduces stresses on the existing moment frames. Providing supplemental bracing or shear walls also can reduce frame stresses. Concentric frames and bracing may pose relative rigidity problems where a rigid diaphragm is present. 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. In addition to being compatible with the rigidity of the moment

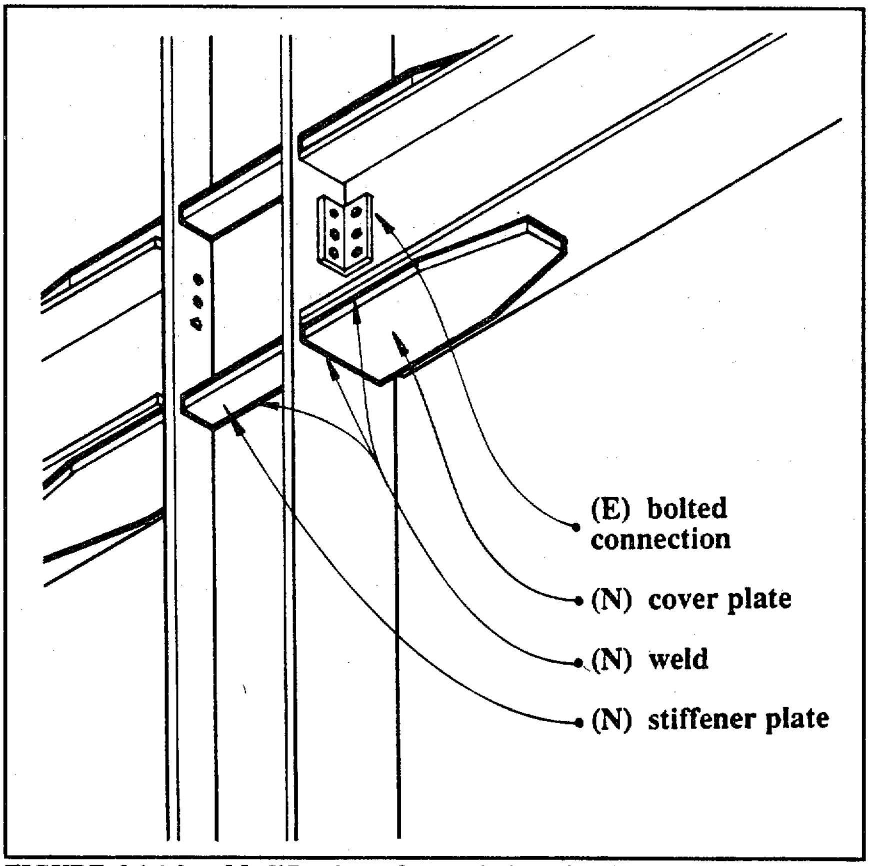

FIGURE 3.1.1.2a Modification of an existing simple beam to a moment connection.

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 (Technique 1) may be practical. The addition of cover plates to beam flanges (Figure 3.1.1.2a) can increase the moment capacity of the existing connection, and the capacity of columns can be increased by boxing (Figure 3.1.1.2b). Since the capacity of a column is determined by the interaction of axial plus bending stresses, the addition of box plates increases the axial capacity, thus permitting the column a greater bending capacity. 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 flanged gussets to form haunches below and/or above the beam or the use of knee braces (Technique 2) may be effective for increasing the moment capacity of a deficient moment frame. The effects of the haunches or knee braces 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 gusset or brace and the beam or column.

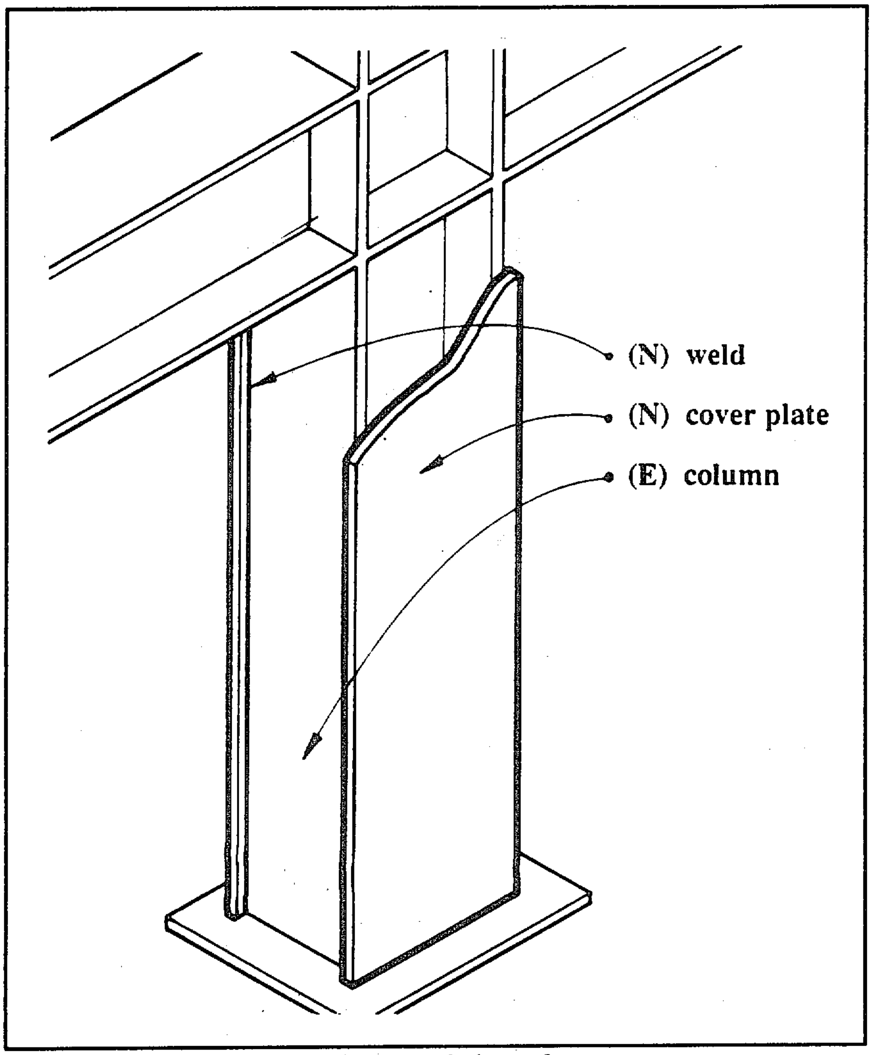

FIGURE 3.1.1.2b Strengthening an existing column.

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.) However, if 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 as shown in Figure 3.1.1.2c to avoid an abrupt change in section modulus beyond the point where the additional section modulus is required. Where composite action is not an alternative, increasing the top flange thickness can be achieved by adding tapered plates to the sides of the top flange and butt-welding these plates to the beam and column flanges.

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.

Encasing the columns in concrete (Technique 5) can increase column shear capacity in addition to increasing stiffness. This alternative may be cost-effective when both excessive drift and inadequate column shear capacity need to be addressed.

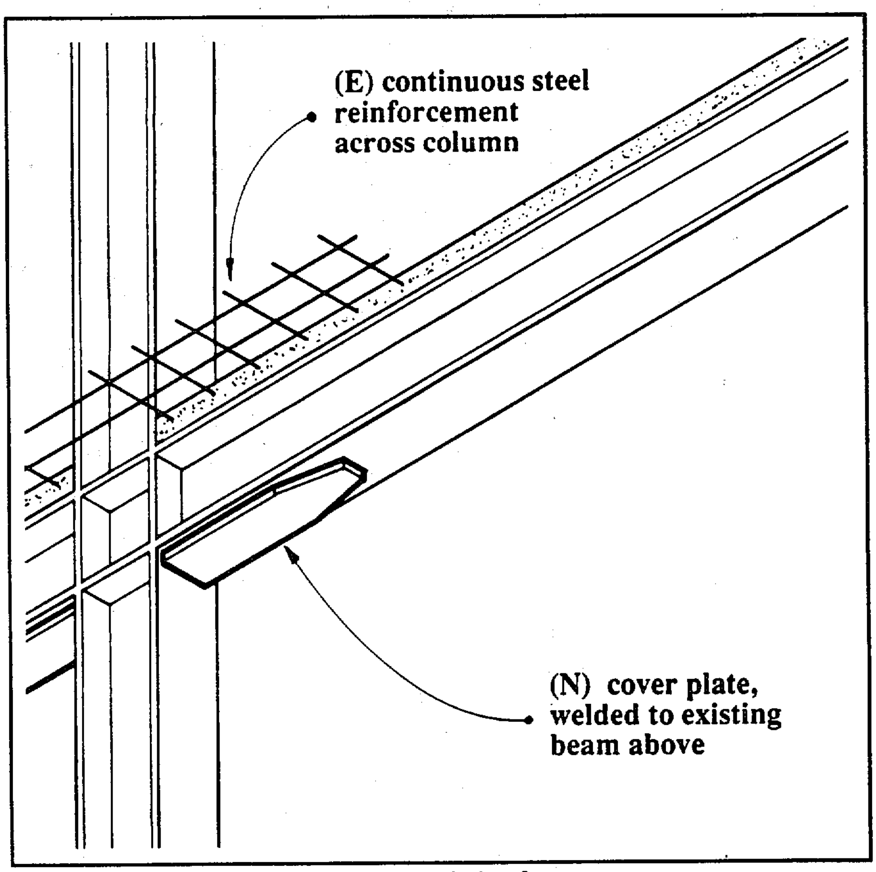

FIGURE 3.1.1.2c Strengthening an existing beam.

3.1.1.3 Strengthening Techniques for Inadequate Panel Zone Capacity

Techniques. 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), 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 flange (i.e., shear yielding or buckling of the column web). Deficient panel zones can be improved by:

1. Providing welded continuity plates between the column flanges.

2. Providing stiffener plates welded to the column flanges and web.

3. Providing web doubler plates at the column web.

4. Reducing the stresses in the panel zone by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls) as discussed in Sec. 3.4.

Relative Merits. Technique 2 (i.e., 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. The cost for removal and replacement of existing architectural cladding and fireproofing associated with these alternatives needs to be considered in assessing cost-effectiveness.

3.1.1.4 Techniques for Reducing Drift

Techniques. Drift of steel moment frames can be reduced by:

1. Increasing the capacity and, hence, the stiffness of the existing moment frame by cover plates or boxing.

2. Increasing the stiffness of the beams and columns at their connections by providing steel gusset plates to form haunches.

3. Reducing the drift by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls) as discussed in Sec. 3.4.

4. Increasing the stiffness by encasing columns in reinforced concrete.

5. Reducing the drift by adding supplemental damping as discussed in Sec. 4.

Relative Merits. 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 frame by the addition of bracing or shear walls. However, increasing the rigidity of the frame also may increase the demand load by lowering the fundamental period of vibration of the structure, and this potential adverse effect must be assessed.

Providing steel gusset plates (Technique 2) to increase stiffness and reduce drift may be cost-effective in some cases. This technique however, must be used with caution since new members may increase column bending stresses and increase the chance for a nonductile failure. Thus, column and beam stresses must be checked where beams and columns interface with gussets 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 (Technique 4) 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 be only partially effective and is therefore only cost-effective when a building has relatively stiff beams and flexible columns.

Reducing drift by adding supplemental damping is an alternative that is now being considered 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. This alternative is further discussed in Sec. 4.3.2.

3.1.2 CONCRETE MOMENT FRAMES

3.1.2.1 Deficiency

The principal deficiency in concrete moment frames is inadequate ductile bending or shear capacity in the beams or columns and lack of confinement, frequently in the joints.

3.1.2.2 Strengthening Techniques for Deficiency in Concrete Moment Frames

Techniques. Deficient bending and shear capacity of concrete moment frames can be improved by:

1. Increasing the ductility and capacity by jacketing the beam and column joints or increasing the beam or column capacities (Figures 3.1.2.2a and 3.1.2.2b).

2. Reducing the seismic stresses in the existing frames by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls) as discussed in Sec. 3.4.

3. Changing the system to a shear wall system by infilling the reinforced concrete frames with reinforced concrete (Figure 3.1.2.2c).

Relative Merits. Improving the ductility and strength of concrete frames by jacketing (Technique 1) generally is not cost-effective because of the difficulty associated with providing the necessary confinement and shear reinforcement in the 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 (Technique 2) or filling in the frames with reinforced concrete (Technique 3). Either of these alternatives will tend to make the frames ineffective for lateral loads. 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 decision regarding whether the new walls should be in the interior of the building or at its perimeter or exterior buttresses usually will depend on nonstructural considerations such as aesthetics and disruption or obstruction of the functional use of the building.

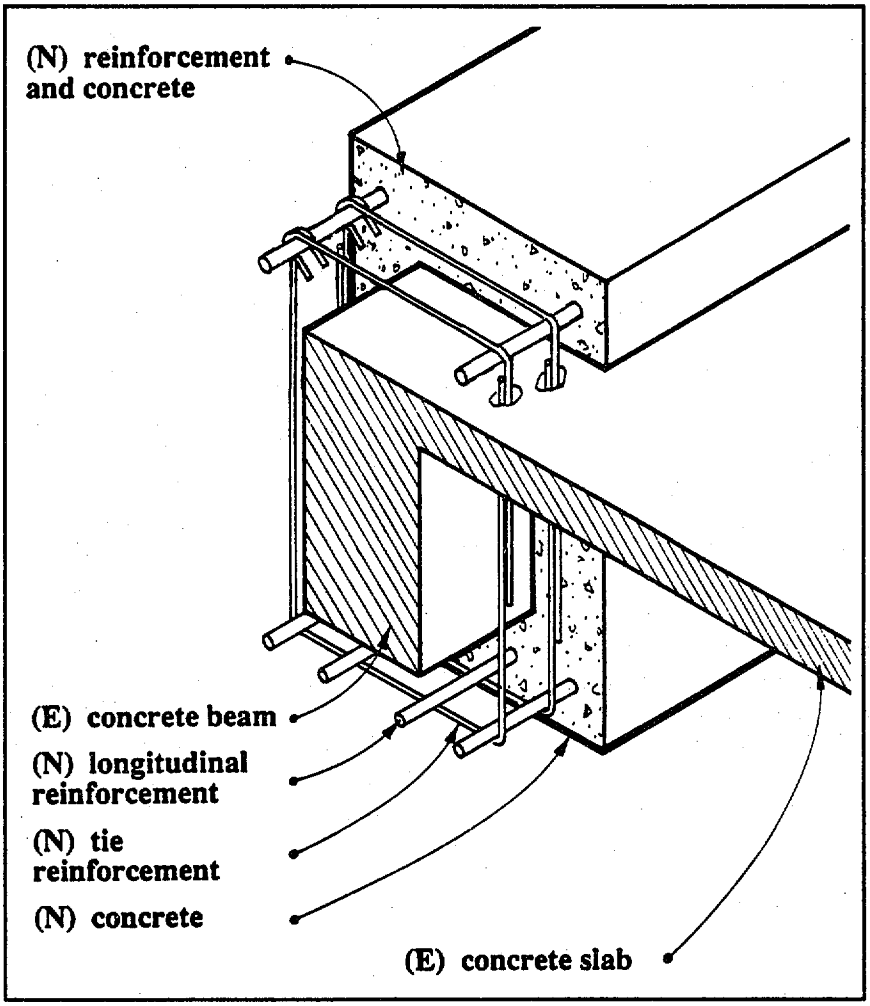

FIGURE 3.1.2.2a Encasing an existing beam in concrete.

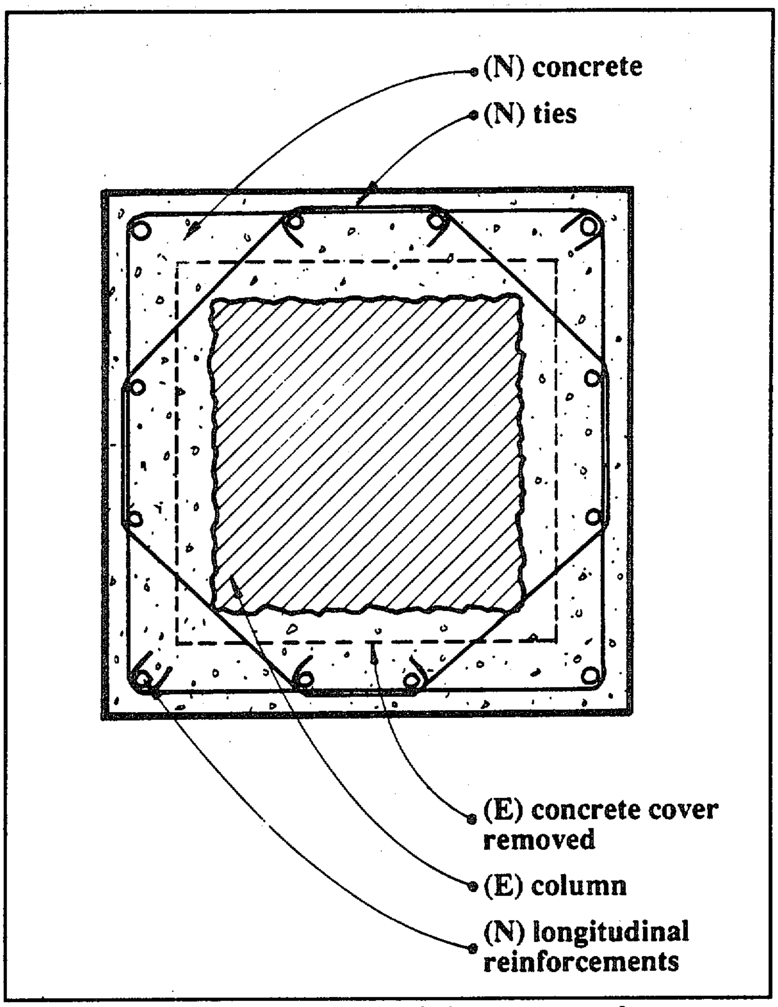

Figure 3.1.2.2b Strengthening an existing concrete column.

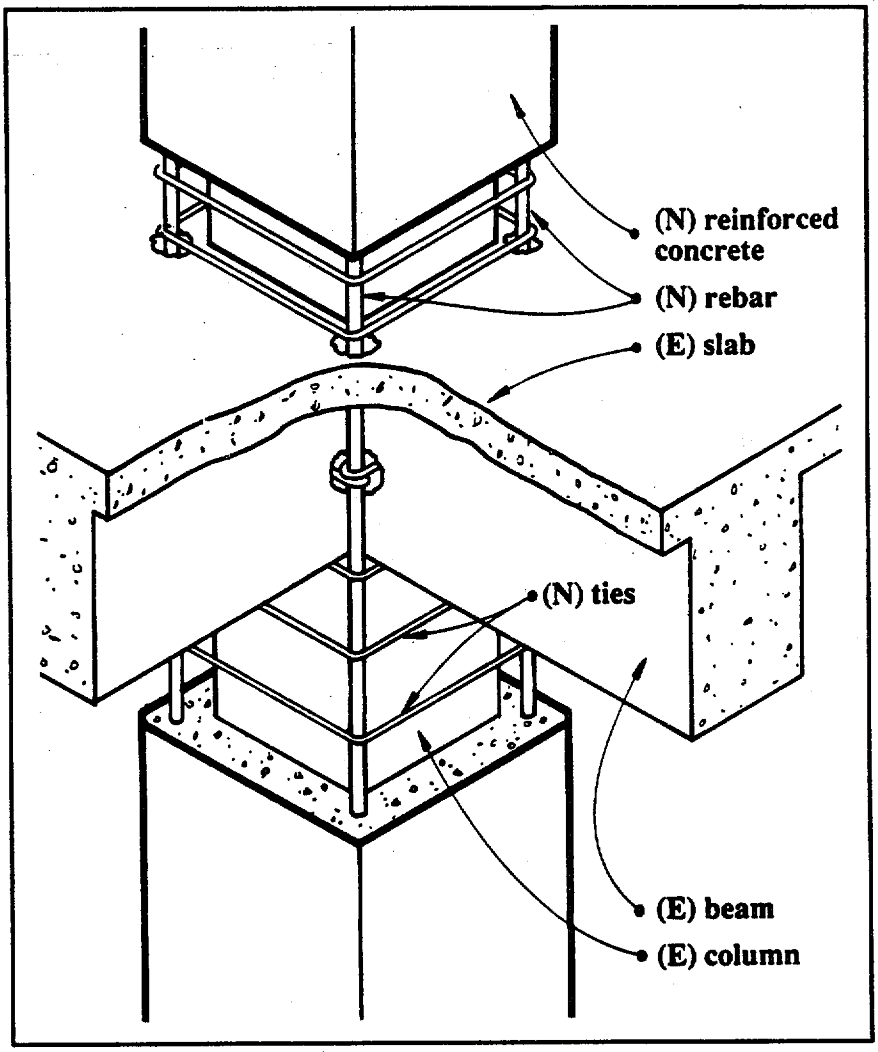

FIGURE 3.1.2.2b continued.

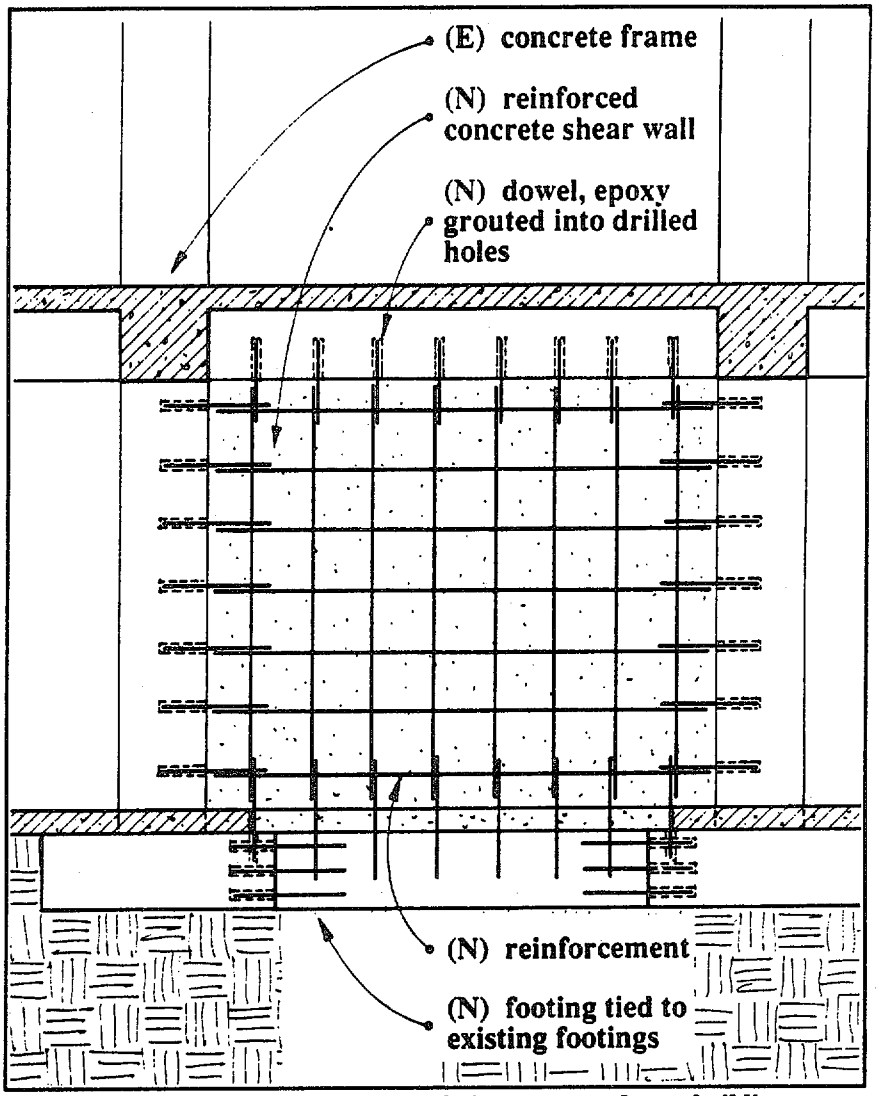

FIGURE 3.1.2.2c Strengthening an existing concrete frame building with a reinforced concrete shear wall.

3.1.3 MOMENT FRAMES WITH INFILLS

3.1.3.1 Deficiencies

When reinforced concrete or steel moment frames are completely infilled, the frame action may be inhibited by the rigidity of the infill wall. Rigid infill walls (e.g., reinforced concrete, reinforced masonry, or clay tile) will resist lateral forces predominantly as shear walls and the frames will be relatively ineffective. Reinforced concrete or steel frames completely infilled with less rigid walls (e.g., unreinforced masonry) will tend to resist lateral forces as braced frames with a diagonal compression "strut" forming in the infill. The principal deficiencies in moment frames with infill walls are:

- Crushing of the infill at the upper and lower corners due to the diagonal compression strut type action of 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 or their connections 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 anchorage or excessive slenderness of the infill wall.

If the infill walls have inadequate capacity to resist the prescribed forces, the deficiencies may be corrected as described below for shear walls.

Partial height infills or infills with door or window openings also will tend to brace concrete or steel frames, but 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 or flexural failure of the column could occur in addition to the other potential deficiencies indicated above for completely infilled frames.

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. 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 a wire mesh. Basketing will not prevent the infill from failing but will prevent debris from falling.

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. 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 in the preceding paragraph.

3.1.3.2 Rehabilitation Techniques for the Infill Walls of Moment Frames

Techniques. Inadequate shear transfer of the infill walls of moment frames can be improved by:

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

2. Treating the infill frame as a shear wall and correcting the deficiencies as described in Sec. 3.2.

Relative Merits. If the frame, without the infill wall, has adequate capacity for the prescribed forces, the most expedient correction is to provide a resilient joint between the column, upper beam, and wall to allow the elastic deformation of the column to take place without restraint (Technique 1). This may be accomplished by cutting a gap between the wall and the column and the upper beam 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, then proper connection of the infill to the frame may result in an adequate shear wall. 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.

3.1.4 PRECAST CONCRETE MOMENT FRAMES

3.1.4.1 Deficiency

The principal deficiency of precast concrete moment frames is inadequate capacity and/or ductility of the joints between the precast units.

3.1.4.2 Strengthening Techniques for the Precast Concrete Moment Frames

Techniques. Deficient capacity and ductility of the precast concrete moment frame connections can be improved by:

1. 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.

2. Reducing the forces on the connections by providing supplemental vertical-resisting elements (i.e., additional moment frames, braces, or shear walls) as discussed in Sec. 3.4.

Relative Merits. Reinforcing the existing connections as indicated in Technique 1 generally 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 (Technique 2) generally is more cost-effective; however, the two alternatives may be utilized in combination.

3.2 VERTICAL-RESISTING ELEMENTS--SHEAR WALLS

Shear walls are structural walls designed to resist lateral forces parallel to the plane of the wall. There are four principal types of shear walls: cast-in-place reinforced concrete or masonry shear walls; precast concrete shear walls; unreinforced masonry shear walls; and shear walls in wood frame buildings.

3.2.1 REINFORCED CONCRETE OR REINFORCED MASONRY SHEAR WALLS

3.2.1.1 Deficiencies

The principal deficiencies of reinforced concrete or masonry shear walls are:

- Inadequate shear capacity,

- Inadequate flexural capacity, and

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

3.2.1.2 Strengthening Techniques for Shear Capacity

Techniques. Deficient shear capacity of existing reinforced concrete or reinforced masonry shear walls can be improved by:

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

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

3. Reducing the shear or flexural stresses in the existing walls by providing supplemental vertical-resisting elements (i.e., shear walls, bracing, or external buttresses) as discussed in Sec. 3.4.

Relative Merits. Techniques 1 and 2 generally will be more economical than Technique 3, 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 probably will the most economical. If this is not feasible, Technique 3 should be considered.

The optimum application of this alternative would be when adequate additional capacity could be obtained by a reinforced concrete 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 footing. In some cases, restrictions may preclude any change in the exterior appearance of the building (e.g., 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 concrete overlay to the inside face and the outside face of the building walls is reduced. In some cases, for example, when deficiencies exist 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.

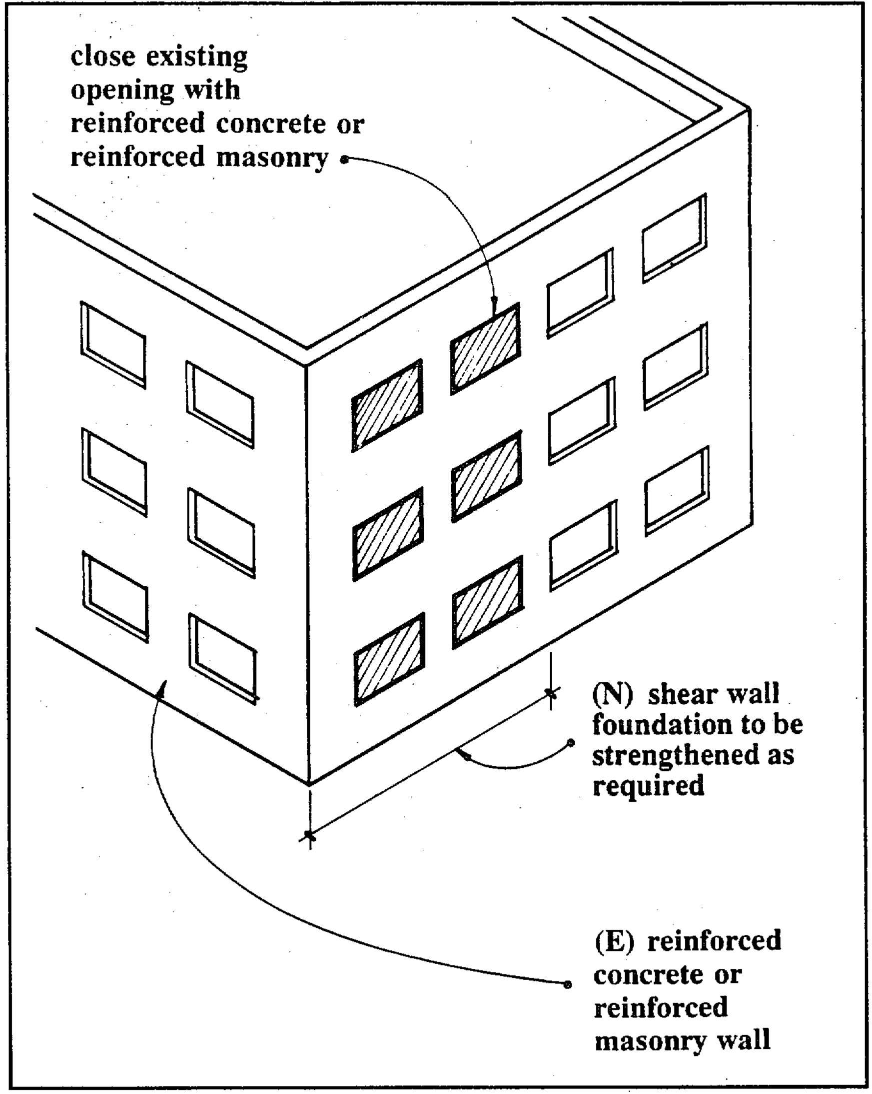

FIGURE 3.2.1.2a Strengthening an existing shear wall by filling in existing openings.

Technique 3 (i.e., providing supplemental 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 new drag struts or other connections to collect the diaphragm shears for transfer to the new shear walls or buttresses. The foundation required to resist overturning forces for an exterior buttress usually is significant because the dead weight of the building cannot be mobilized to resist the overturning forces. 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 advantages of the buttress over a new interior shear wall is that the work can be accomplished with minimal interference to ongoing building functions.

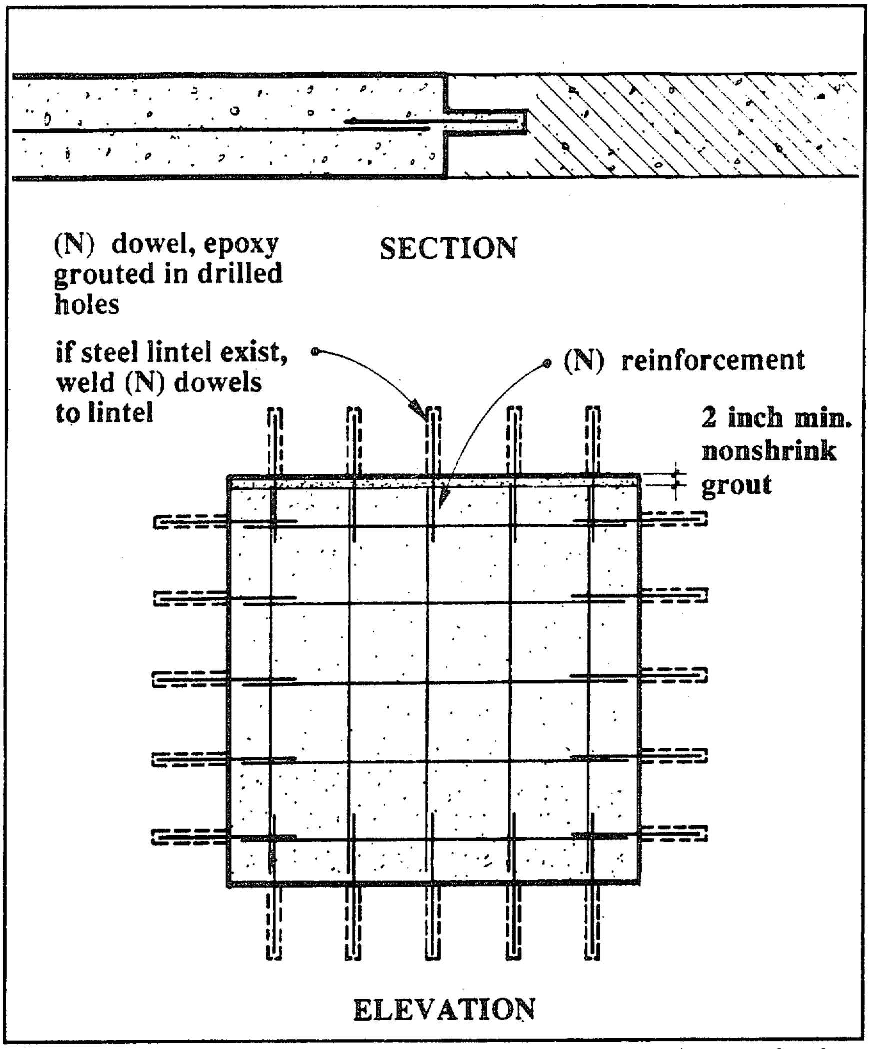

FIGURE 3.2.1.2b Example of details for enclosing an existing opening in a reinforced concrete or masonry wall.

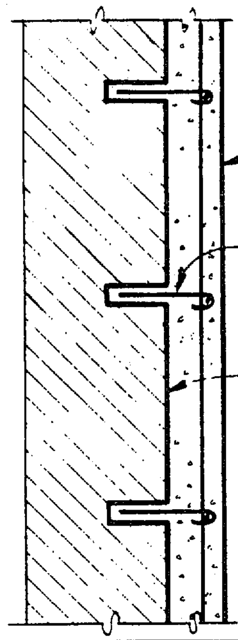

FIGURE 3.2.1.2c Strengthening an existing reinforced concrete or masonry wall.

3.2.1.3 Strengthening Technique 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 capacities 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) also are more susceptible to brittle failure; therefore, care must be taken not to overdesign the flexural capacity of rehabilitated shear walls.

3.2.1.4 Rehabilitation Technique for Coupling Beams

Deficient shear or flexural capacity in coupling beams of reinforced concrete or reinforced masonry shear walls can be improved by:

1. Eliminating the coupling beams by filling in openings with reinforced concrete (Figure 3.2.1.2b).

2. Removing the existing beams and replacing with new stronger reinforced beams (Figure 3.2.1.4).

3. Adding reinforced concrete to one or both faces of the wall and providing an additional thickness to the existing wall (Figure 3.2.1.2c).

4. Reducing the shear or flexural stresses in the connecting beams by providing additional vertical-resisting elements (i.e., shear walls, bracing, or external buttresses) as discussed in Sec. 3.4.

Relative Merits. If the deficiency is in both the piers and the connecting beams, the most economical solution is likely to be the Technique 3 (i.e., adding reinforced concrete on one or both sides of the existing wall). Shallow, highly stressed connecting beams may have to be replaced with properly reinforced concrete as part of the additional wall section. 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 connecting beams, consideration should be given to 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, Technique 2 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, Technique 1 (i.e., filling in selected openings) may be practical. If Techniques 1 through 3 are not feasible or adequate to ensure the proper performance of the wall, reducing the stresses by adding supplemental new structural elements (Technique 4) should be considered. This alternative is likely to be the most costly because of the need for new foundations, vertical members, and collectors.

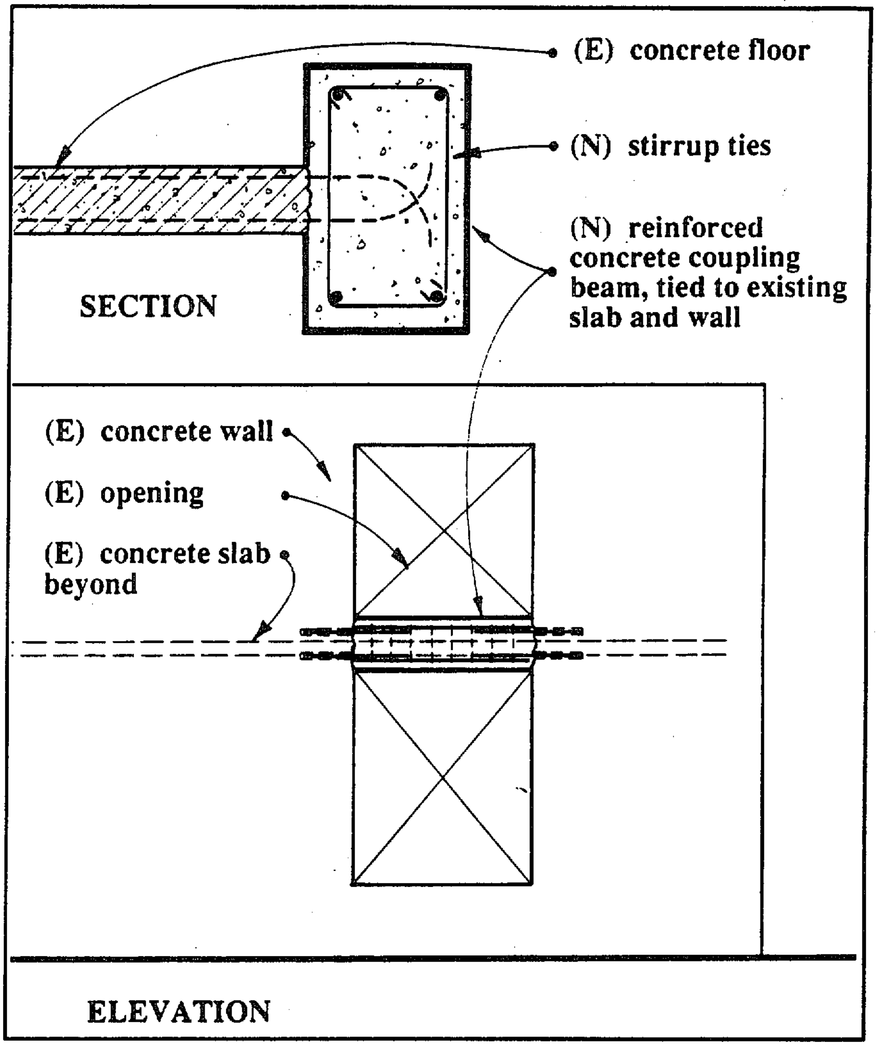

FIGURE 3.2.1.4 Example of strengthening an existing coupling beam at an exterior wall.

3.2.2 PRECAST CONCRETE SHEAR WALLS

3.2.2.1 Deficiencies

The principal deficiencies of precast concrete 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.

3.2.2.2 Strengthening Techniques for Inadequate Shear or Flexural Capacity

Techniques. Deficient in-plane shear or flexural capacity of precast concrete panel walls can be improved by:

1. Increasing the shear and flexural capacity of walls with significant openings for doors or windows by infilling the existing openings with reinforced concrete.

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

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

Relative Merits. Precast concrete shear walls generally only have high in-plane shear or flexure 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 (Technique 1). In the case of inadequate interpanel shear capacity, the panels will act independently and can have inadequate flexural capacity. Improving the connection capacity between panels can improve the overall wall capacity. Techniques 2 and 3 generally not cost-effective unless a significant overstress condition exists.

3.2.2.3 Strengthening Techniques for Inadequate Interpanel Capacity

Techniques. Deficient interpanel shear connection capacity of precast concrete wall panels can be improved by:

1. 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.).

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

Relative Merits. The two techniques can be equally effective. Where operational and aesthetic requirements for the space can accommodate the installation of tie-downs and possibly surface-mounted wall edge reinforcement that will make each panel act as a cantilever is a cost-effective way to compensate for inadequate interpanel capacity. 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.

3.2.2.4 Strengthening Techniques for Inadequate Out-of-Plane Flexural Capacity

Techniques. Deficient out-of-plane flexural capacity of precast concrete shear walls can be by:

1. Providing pilasters at and/or in-between the interpanel joints.

2. Adding horizontal beams between the columns or pilasters at mid-height of the wall.

Relative Merits. The reinforcing in some precast concrete wall panels may be 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. Also, 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. It should be noted that the problem of inadequate out-of-plane flexural capacity often is caused by wind design, particularly in the lower seismic zones.

3.2.2.5 Strengthening Techniques for Inadequate Shear or Flexural Capacity in Coupling Beams

Techniques. 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.

Relative Merits. 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 in Sec. 3.2.1.4 for concrete shear walls.

3.2.3 UNREINFORCED MASONRY SHEAR WALLS

3.2.3.1 Deficiencies

Masonry walls include those constructed of solid or hollow units of brick or concrete. Hollow clay tile also is typically classified as masonry. The use of hollow tile generally has been limited to nonstructural partitions and is discussed in Sec. 5.4. Unreinforced concrete, although not classified as masonry, may be strengthened by techniques similar to those described below for masonry.

The principal deficiencies of unreinforced masonry shear walls are:

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

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

3.2.3.2 Strengthening Techniques for Inadequate In-plane Shear and Out-of-Plane Flexural Capacity of the Walls

Techniques. Deficient in-plane shear and out-of-plane flexural capacity of unreinforced masonry walls can be improved by:

1. Providing additional shear capacity by placing reinforcing steel on the inside or outside face of the wall and applying new reinforced concrete (Figure 3.2.1.2c).
2. Providing additional capacity for only out-of-plane lateral forces by adding reinforcing steel to the wall utilizing the center coring technique (Figure 3.2.3.2).
3. 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.
4. Filling in existing window or door openings with reinforced concrete or masonry (Figures 3.2.1.2a and 3.2.1.2b).
5. Providing additional shear walls at the interior or perimeter of the building or providing external buttresses.

Relative Merits. 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. These stresses generally are based on the ultimate strength of the masonry determined from core tests or in-situ testing. A very large safety factor commonly is used in establishing allowable shear stress because of the potential variation in workmanship and materials, particularly in masonry joints.

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. However, 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. Since new concrete shear walls can delaminate from the masonry substrate, such walls should have adequate height to thickness ratios (h/t) independent of the masonry wall.

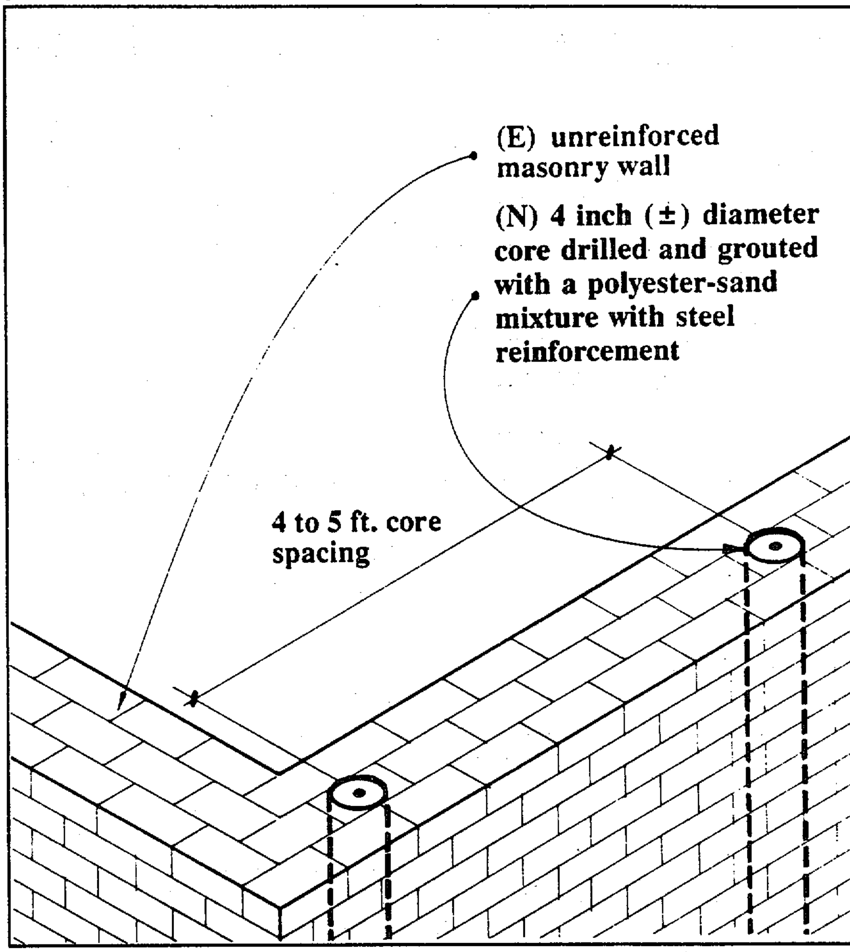

FIGURE 3.2.3.2 Example of center coring technique.

Unreinforced masonry buildings often lack adequate wall anchorage and diaphragm chords. To correct these deficiencies as well as inadequate in-plane shear capacity, it may be desirable to place the concrete overlay on the inside face of the exterior walls (Figure 3.2.1.2c). Foundations, however, may be inadequate to carry the additional weight of the concrete overlay; see the *NEHRP Evaluation Handbook* for further discussion of this subject.

Because unreinforced masonry has minimal tensile strength, these walls are 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.

A recent innovation that has been used on several California projects is the seismic strengthening of unreinforced masonry walls by the center coring technique (Technique 2). This technique consists of removing 4 inch (±) diameter vertical cores from the center of the wall at regular intervals (about 3 to 5 feet apart) and placing reinforcing 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.

Technique 3 for strengthening the 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 (Technique 4) 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 have physical properties similar to those of the masonry wall.

3.2.3.3 Alternative Methodology for Evaluation and Design of 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 ABK methodology was the basis for the City of Los Angeles' Rule of General Application (RGA) that was developed in cooperation with the Hazardous Buildings Committee of the Structural Engineers Association of Southern California and approved in 1987 as an alternate to the conventional design method in Division 88 of the *Los Angeles City Building Code*. Code provisions for the "ABK Methodology" now have been developed jointly by the Structural Engineers Association of California (SEAOC) and the California Building Officials (CALBO) and are published in the 1991 Edition of the Uniform Code for Building Conservation (available from the International Conference of Building Officials). The procedure for evaluation of unreinforced masonry (URM) bearing wall buildings presented in Appendix C of the *NEHRP Evaluation Handbook* is based on this methodology.

Some of the principal differences between the new methodology and conventional code provisions are as follows:

1. 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).

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

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

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

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

The ABK Methodology and the more conventional evaluation and design methods, prescribed in building codes such as the City of Los Angeles' Division 88 for unreinforced masonry have been prescribed in California with the objective of preservation of life safety rather than prevention of damage. Several moderate earthquakes in Southern California have provided limited testing of the methodology and, although the results are not conclusive, very few of the retrofitted buildings suffered total or partial collapse and the degree of structural damage was less than occurred in nonretrofitted buildings.

3.2.4 SHEAR WALLS IN WOOD FRAME BUILDINGS

3.2.4.1 Deficiencies

The principal deficiencies of wood or metal stud shear wall buildings are:

- Inadequate shear capacity of the wall and
- Inadequate uplift or hold-down capacity of the wall.

3.2.4.2 Strengthening Techniques for Inadequate Shear Capacity

Techniques. Deficient shear capacity of the wood or metal stud walls can be improved by:

1. Increasing the shear capacity by providing additional nailing to the existing finish material.
2. Increasing the shear capacity by adding plywood sheathing to one or both sides of the wall.
3. Reducing the loads on the wall by providing supplemental shear walls to the interior or perimeter of the building.

Relative Merits. Seismic forces in existing wood frame buildings generally are moderate and, in many cases, the existing walls may be adequate. Tabulated allowable shear values are available for existing finishes such as lath and plaster and gypsum wallboard. In the latter case, existing nailing may dictate the allowable shear value and higher allowable values may be obtained by additional nailing. Similarly, the allowable shear value for walls with existing plywood sheathing may be increased within limits by additional nailing. New plywood sheathing may be nailed onto existing gypsum wallboard. Longer nails are required and the allowable shear values are comparable to plywood nailed directly to the studs, but the existing finish need not be removed.

Existing metal stud shear walls may be evaluated like wood stud walls. The fasteners generally are self-threading sheet metal screws and corresponding allowable shear values are available for the finishes discussed in the preceding paragraph.

Where the shear capacity of an existing wall is increased, the shear transfer capacity at the foundation and the capacity of the foundation connection to resist overturning forces must be checked. Techniques for increasing the foundation shear connection and overturning capacities are discussed in Sec. 3.8.1.

As with other shear wall strengthening techniques, the most economical scheme will be the one that minimizes the total cost, including removal and replacement of finishes and other nonstructural items, disruption of the functional use of the building, and any necessary strengthening of foundations or other structural supports. Under normal circumstances, sheathing the exterior face of the perimeter walls should have the lowest cost, but in some circumstances (e.g., if extensive interior alterations are planned) strengthening existing interior shear walls or adding new interior shear walls will be more economical.

If the loads are so large that the above alternatives are not practical, it may be possible to reduce the forces on the wall by strengthening other existing shear walls or by adding supplemental walls (Technique 3).

3.2.4.3 Strengthening Techniques for Inadequate Uplift or Hold-Down Capacity

Techniques. Strengthening techniques for inadequate uplift or hold-down capacity are discussed in Sec. 3.7.1.5 and are illustrated in Figures 3.7.1.5 (a, b, c, and d).

3.3 VERTICAL-RESISTING ELEMENTS--BRACED FRAMES

Braced frames are vertical elements that resist lateral loads through tension and/or compression braces. There are two principal types of braced frames: concentric bracing consisting of diagonals, chevrons, K-bracing, or tension rods and eccentric bracing (Figure 3.3).

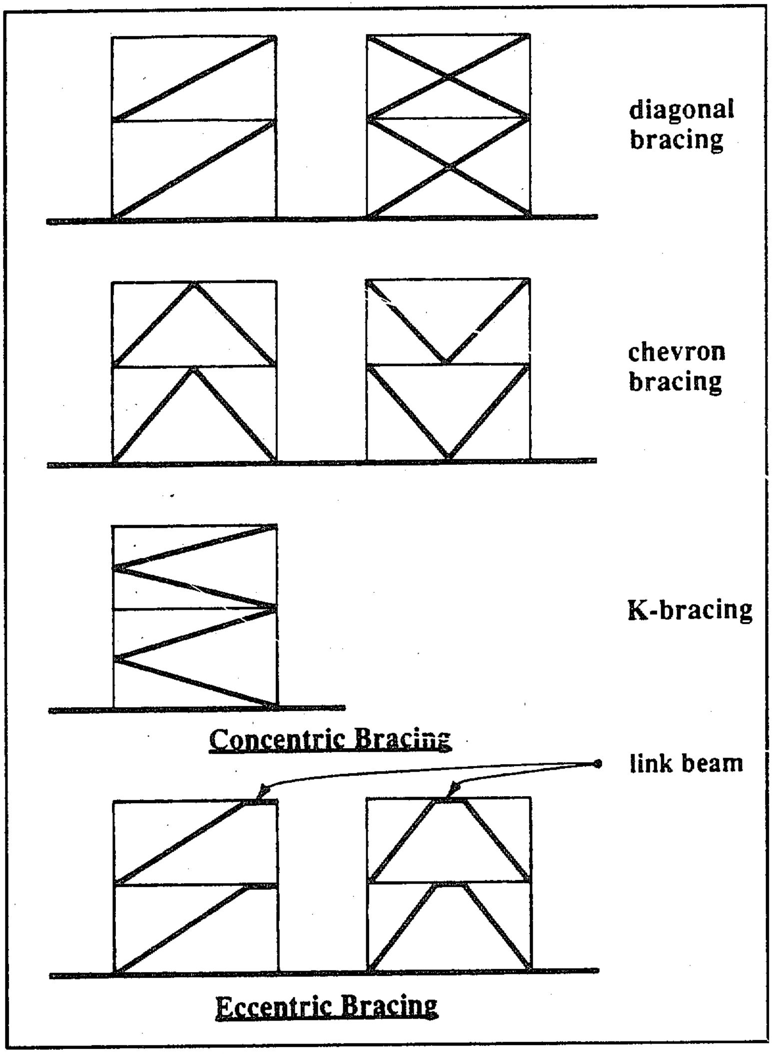

FIGURE 3.3 Bracing types.

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 only a small probability of exceedance 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 at all 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 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.

3.3.1 STEEL CONCENTRICALLY BRACED FRAMES

3.3.1.1 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,

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*The American Institute of Steel Construction has written a minority opinion regarding this sentence; see page 193.

- 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.

3.3.1.2 Strengthening Techniques for Inadequate Brace Capacity

Techniques. Deficient brace compression capacity can be improved by:

1. Increasing the capacity of the braces by adding new members thus increasing the area and reducing the radius of gyration of the braces.
2. Increasing the capacity of the member by reducing the unbraced length of the existing member by providing secondary bracing.
3. Providing greater capacity by removing and replacing the existing members with new members of greater capacity (Figure 3.3.1.2).
4. Reducing the loads on the braces by providing supplemental vertical-resisting elements (i.e., shear walls, bracing, or eccentric bracing) as discussed in Sec. 3.4.

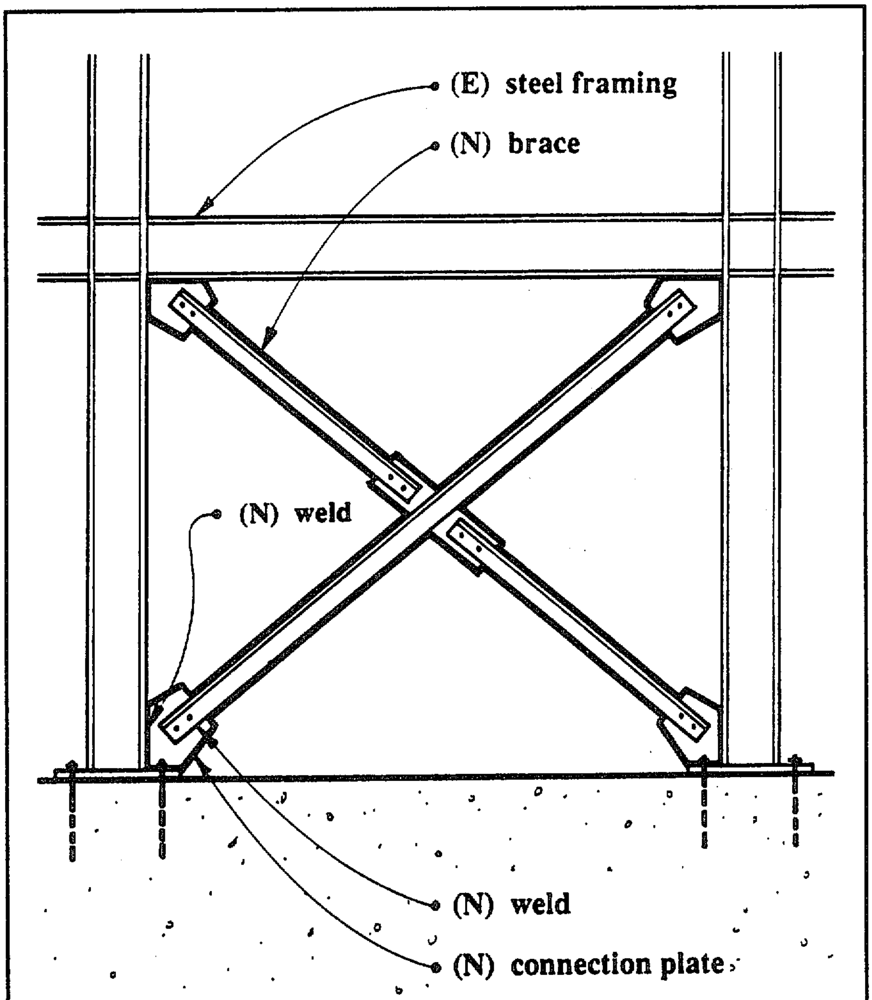

FIGURE 3.3.1.2 Addition to or replacement of an existing X-brace.

Relative Merits. A brace member is designed to resist both tension and compression forces, but its capacity for compression stresses is limited by potential buckling and is therefore less than the capacity for tensile stresses. Since the design of the system generally is based on the compression capacity of the brace, some additional capacity may be obtained by simply reducing the unsupported length of the brace by means of secondary bracing (Technique 3) 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 (Technique 1) or replacement (Technique 3) 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 usually is 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. The rigidity of eccentric bracing, however, can be "tuned" to be compatible with that of the existing concentric bracing, but the advantages of the eccentric bracing may be offset by its greater construction cost. Thus, strengthening the existing bracing or providing additional concentric bracing are considered to be the most cost-effective alternatives.

3.3.1.3 Strengthening Techniques for Inadequate Capacity of the Brace Connection

Techniques. Deficient brace connection capacity can be improved by:

1. Increasing the capacity of the connections by additional bolting or welding.

2. Increasing the capacity of the connections by removing and replacing the connection with members of greater capacity.

3. Reducing the loads on the braces and their connections by providing supplemental vertical-resisting elements (i.e., shear walls, bracing, or eccentric bracing) as discussed in Sec. 3.4.

Relative Merits. 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, the most economical alternative may be to increase the existing connection capacity by providing additional welding or bolts. If the existing gusset plates have inadequate capacity, the existing configuration and accessibility need to be assessed to determine whether adding supplemental connecting members or replacing the existing connecting members with members of greater capacity (Technique 3) is more economical. 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 Technique 1 (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.

3.3.1.4 Strengthening Techniques for Inadequate Axial Load Capacity in the Columns or Beams of the Bracing System

Techniques. Deficient axial load capacity of existing bracing system columns and beams can be improved by:

1. Providing additional axial load capacity by adding cover plates to the member flanges or by boxing the flanges.

2. Providing additional axial load capacity by jacketing the existing members with reinforced concrete.

3. Reducing the loads on the beams and columns by providing supplemental vertical-resisting elements (i.e., shear walls, bracing, or eccentric bracing) as discussed in Sec. 3.4.

Relative Merits. 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 or box the flanges (Technique 1). The effort involved in adding cover and box plates includes removing the existing fireproofing and nonstructural obstructions. Jacketing of existing members with reinforced concrete (Technique 2) 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 Sec. 3.3.1.2.

3.3.2 ROD OR OTHER TENSION BRACING

3.3.2.1 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.

3.3.2.2 Strengthening Techniques for Tension Capacity

Techniques. Deficient tension capacity of the rod or other tension member and its connection can be improved by:

1. Increasing the capacity by strengthening the existing tension members.

2. Increasing the capacity by removing the existing tension members and replacing with new members of greater capacity.

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

4. Reducing the forces on the existing tension members by providing supplemental vertical-resisting elements (i.e., additional tension rods) as discussed in Sec. 3.4.

Relative Merits. 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 therefore is rarely feasible to strengthen a deficient rod (Technique 1); hence, correction of the deficiency likely will require removal and replacement with larger rods (Technique 2), removal of existing tension bracing and replacement with new bracing capable of resisting tension and compression (Technique 3), or installation of additional bracing (Technique 4). 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 (Technique 4) unless functional or other nonstructural considerations (e.g., obstruction of existing window or door openings) preclude the addition of new bracing.

3.3.2.3 Strengthening Techniques for Beam or Column Capacity

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

1. Increasing the axial capacity by adding cover plates to or by boxing the existing flanges.

2. Reducing the forces on the existing columns or beams by providing supplemental vertical-resisting elements (i.e., braced frames or shear walls) as discussed in Sec. 3.4.

Relative Merits. Reinforcing the existing beams or columns with cover plates or boxing the flanges generally is the most cost-effective alternative. 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 elements may be a viable alternative.

3.3.3 ECCENTRIC BRACING

3.3.3.1 Deficiency

The primary deficiency of an eccentrically braced frame system is likely to be nonconformance with current design standards because design standards for such elements did not exist earlier than about 1980. Eccentric bracing in older buildings may not have the desired degree of ductility.

3.3.3.2 Strengthening Techniques for Eccentric Braced Frames

Techniques. An existing eccentrically braced frame system can be brought into conformance with current design standards by ensuring that the system is balanced (i.e., there is a link beam at one end of each brace), the brace and the connections are designed to develop shear or flexural yielding in the link, the connection is a full moment connection where the link beam has an end at a column, and lateral bracing is provided to prevent out-of-plane beam displacements that would compromise the intended action. Alternatively, the loads on the existing eccentrically braced frame can be reduced by providing supplemental vertical-resisting elements such as additional eccentrically braced frames.

Relative Merits. The use of engineered eccentric bracing is a relatively recent innovation (within about 10 years) that can provide the rigidity associated with concentric bracing as well as the ductility associated with moment frames. The recommended design of these frames precludes compressive buckling of the brace members by shear yielding of a short portion of the horizontal beam (the link beam). If the brace is in a diagonal configuration, the yielding occurs in the horizontal beam between the brace connection and the adjacent column; if it is in a chevron configuration, the yielding occurs in the beam between the two brace connections.

Because this system is relatively new, a deficiency in the lateral load capacity reflects either improper design or upgraded design criteria. A properly designed eccentric bracing system balances the yield capacity of the horizontal link beam against the buckling capacity of the brace beam. It usually is not cost-effective to strengthen the members of this bracing system unless it is necessary to correct a design defect (e.g, if the brace has been over designed, the shear capacity of the horizontal beam can be increased by adding doubler plates to the beam web provided other members of the system have adequate additional capacity). Usually it will be necessary to add additional bracing. It should be noted, however, that although eccentric bracing is a desirable supplement to an existing concentric bracing system, concentric bracing is not desirable as a supplement to an existing eccentric bracing system. The proper functioning of an eccentric bracing system requires inelastic deformations that are not compatible with concentric bracing; the introduction of a ductile element (eccentric bracing) into an existing "brittle" system (concentric bracing) is beneficial, but the reverse procedure is not the case. The addition of shear walls to an existing eccentric bracing system also is usually not effective because of their greater rigidity. Thus, the most cost-effective procedure for increasing the capacity of an existing eccentric bracing system probably will be to provide additional eccentric bracing.

3.4 VERTICAL-RESISTING ELEMENTS--ADDING SUPPLEMENTAL MEMBERS

The lateral seismic inertial forces of an existing building are transferred from the floors and roofs through the vertical-resisting elements (i.e., shear walls, braced frames and moment frames) to the foundations and into the ground. The forces in the individual shear walls, braced frames, and moment frames are a function of the weight and height of the building plus the number, size, and location of the elements. By adding new vertical elements to resist lateral forces, the forces in the existing elements will be modified and generally will be reduced. Thus, the addition of supplemental vertical elements that will resist lateral loads can be a means to correct existing elements that are overstressed. The purpose of this section is to discuss the benefits and the problems associated with adding supplemental vertical-resisting elements to an existing building so that comparisons with other rehabilitation techniques such as strengthening overstressed members or reducing demand can be placed in perspective. The two general categories of supplemental vertical-resisting elements are in-plane supplemental elements and new bay supplemental elements. The two categories are schematically portrayed in Figure 3.4.

The introduction of new in-plane supplemental elements into a building will primarily reduce the forces on the existing vertical elements in the plane where the new element is added. Forces on other vertical-resisting elements, diaphragms, and the connections between them will be modified to a lesser degree depending on the relative rigidities of the vertical elements and the diaphragms. All wood and some steel deck diaphragms may be considered "flexible" when used with masonry or concrete shear walls. Straight laid sheathing may be "flexible" with any type of construction, but plywood sheathed diaphragms may be considered rigid with wood frame walls or light steel frame construction. Where diaphragms are flexible, the addition of a supplemental vertical element in the plane of existing vertical elements will have essentially no effect on the forces in vertical elements located in other bays or on the diaphragms or the connections between the diaphragms and the vertical-resisting elements.

On the other hand, the introduction of new vertical bay supplemental elements, will reduce the forces on all the elements--existing vertical elements, diaphragms, foundations, and the connections between them. The reduction in forces will be proportional to the relative rigidity of the vertical elements when the building has a rigid diaphragm and will be proportional to the tributary areas associated with the vertical-resisting elements when the building has a flexible diaphragm.

The effect of adding in-plane supplemental elements or new bay supplemental elements on the lateral-force distribution of an existing building needs to be evaluated when considering whether to add new vertical elements or to strengthen existing members to reduce demand on bracing elements.

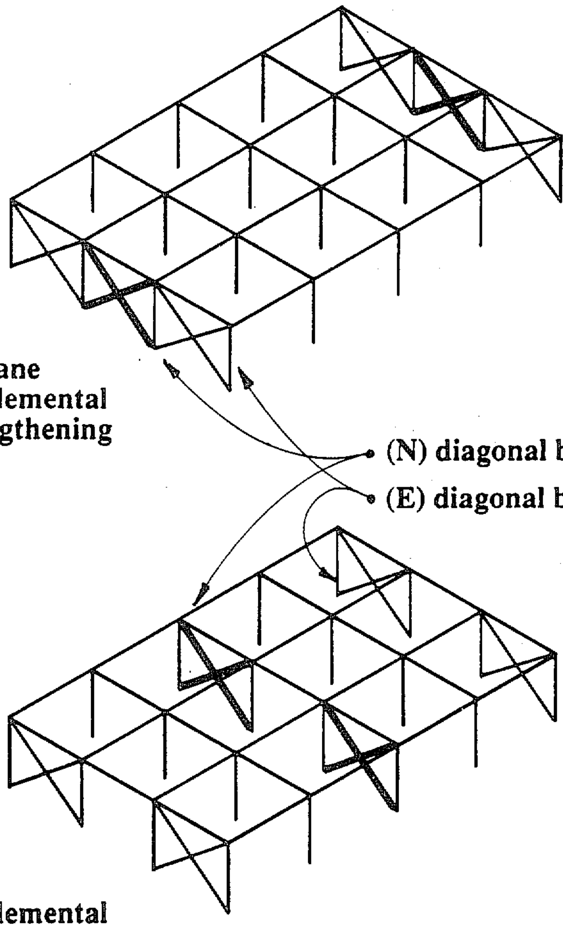

FIGURE 3.4 Examples of supplementary strengthening.

3.4.1 RELATIVE COMPATIBILITY

The effectiveness of supplemental vertical-resisting elements in reducing forces on overstressed components is dependent on the stiffness, strength, and ductility compatibility of the existing vertical-resisting elements relative to the new vertical elements.

Stiffness compatibility is particularly important. A moment frame, for example, is relatively flexible in the lateral direction. New supplemental moment frames, shear walls, or braced frames can be added to an existing moment frame structure. The loads that will be transferred to the supplemental elements will be in proportion to their relative stiffness (for a rigid diaphragm) and, therefore, a shear wall or braced frame added to a moment frame structure will resist a significant portion of the lateral load. If the existing vertical-resisting elements are concrete shear walls, supplemental moment frames generally would be ineffective because of the large degree of wall stiffness.

Structures responding to large earthquakes will behave inelastically, hence the sequence in which different elements yield and the ability of the elements to continue to function in the post yield condition (i.e., their ductility) will affect the dynamic response of the structure. Weaker elements that yield become more flexible resulting in a redistribution of forces. Ductile elements will continue to participate in absorbing energy and resisting forces after yielding. Structures with elements having compatible strengths and ductility will behave better and more predictably than structures with elements of different strength and ductility.