10. Simplified Rehabilitation

10.1 Scope

This chapter presents the Simplified Rehabilitation Method, which is intended primarily for use on a selected group of simple buildings being rehabilitated to the Life Safety Performance Level for the level of ground motion specified in FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC, 1992a). In an area of low or moderate seismicity, designing for this level of ground motion may not be sufficient to provide Life Safety Performance if a large infrequent earthquake occurs.

The technique described in this chapter is one of the two rehabilitation methods defined in Chapter 2. It is to be used only by a design professional, and only in a manner consistent with the *Guidelines*. Consideration must be given to all aspects of the rehabilitation process, including the development of appropriate as-built information, proper design of rehabilitation techniques, and specification of appropriate levels of quality assurance. Systematic Rehabilitation is the other rehabilitation method defined in Chapter 2.

The term "Simplified Rehabilitation" is intended to reflect a level of analysis and design that (1) is appropriate for small, regular buildings, and buildings that do not require advanced analytical procedures, and (2) does not achieve the Basic Safety Objective (BSO).

FEMA 178 (BSSC, 1992a), the *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, a nationally applicable evaluation method, is the basis for the Simplified Rehabilitation Method. FEMA 178 is based on the historic behavior of buildings in past earthquakes and the success of current code provisions in achieving the Life Safety Performance Level. It is organized around a set of common construction styles called model buildings. The performance of certain common building types that meet specific limitations on height and regularity can be substantially improved by simply eliminating all of the deficiencies found using FEMA 178. See Section C10.1 in the *Commentary* for further information on FEMA 178 and other introductory comments. FEMA 178 is currently under revision (October, 1997) and the revised version will be available soon. These *Guidelines* refer frequently to FEMA 178 as a pointer to the FEMA 178 references.

Since the preliminary version of FEMA 178 was completed in the late 1980s, new information has become available, which will be added to FEMA 178 in the updated edition of the document now underway. This information has been included in the Simplified Rehabilitation Method, presented as amendments to FEMA 178 (BSSC, 1992a), and includes additional Model Building Types and eight new evaluation statements for new potential deficiencies. They are presented in the same format and style as used in FEMA 178. The set of common Model Building Types has been expanded to separate those buildings with stiff and flexible diaphragms, and to account for the unique behavior of multistory, multi-unit, wood-frame structures. While near-fault effects are also being proposed to amend FEMA 178, they are not expected to affect the buildings eligible for Simplified Rehabilitation and therefore need not be considered.

The evaluation statements and procedures contained in FEMA 178 apply best to low-rise and, in some cases, mid-rise buildings of regular configuration and well-defined building type. Table 10-1 identifies those buildings for which the Simplified Rehabilitation Method can be used to achieve the Life Safety Performance Level for ground motions specified in FEMA 178 (BSSC, 1992a). It is required, however, that the building deficiencies be corrected by strengthening and/or modifying the existing components of the building using the same basic style of construction. Buildings that have configuration irregularities, as defined in the *NEHRP Recommended Provisions for Regulations for New Buildings* (BSSC, 1995), may use this Simplified Rehabilitation Method to achieve the Life Safety Performance Level only if the resulting rehabilitation work eliminates all significant vertical and horizontal irregularities and results in a building with a complete seismic lateral-force-resisting load path.

The Simplified Rehabilitation Method may be used to achieve Limited Rehabilitation Objectives for any building not listed in Table 10-1. (Note that Table 10-1, the remaining Tables 10-2 to 10-22, and Figure 10-1 are at the end of this chapter.)

The Simplified Rehabilitation Method may yield a more conservative result than the Systematic Method. This is due to the variety of simplifying assumptions. Because of the small size and simplicity of the buildings that are eligible for the Simplified Method of achieving the Life Safety Performance Level, the economic consequences of this conservatism are likely to be insignificant. It must be understood, however, that a simple comparison of the design-based shear in FEMA 178 with Chapter 3 of the *Guidelines* will lead to the opposite conclusion. The equivalent lateral forces used in these two documents have entirely different definitions and bases. The FEMA 178 (BSSC, 1992a) values, which are based on the traditional techniques used in building codes, have been developed on a different basis than in Chapter 3 of the *Guidelines*, and have been taken from the 1988 *NEHRP Provisions*. The Chapter 3 values calculated from a "pseudo lateral load," are defined for a component-based analysis and do not include the same reduction factors. As shown in Figure 10-1, while the base and story shear values may vary by approximately six times, the ratios of demand/capacity vary only slightly.

Implementing a rehabilitation scheme that mitigates all of a building's FEMA 178 (BSSC, 1992a) deficiencies using the Simplified Rehabilitation Method does not in and of itself achieve the Basic Safety Objective or any Enhanced Rehabilitation Objective as defined in Chapter 2, since the rehabilitated building may not meet the Collapse Prevention Performance Level for BSE-2. If the goal is to attain the Basic Safety Objective as described in Chapter 2 or other Enhanced Rehabilitation Objectives, this can be accomplished by using the Systematic Rehabilitation Method defined in Chapter 2.

10.2 Procedural Steps

The application of the Simplified Rehabilitation Method first requires a complete FEMA 178 (BSSC, 1992a) evaluation of a building, which results in a list of deficiencies. These deficiencies are then ranked, and common and simple rehabilitation procedures are applied to correct them. Once a full rehabilitation scheme has been devised, the building is reevaluated using FEMA 178 to verify that it fully meets the requirements. A more complete statement of this procedure follows. The procedures are applicable only to buildings that meet the qualification criteria shown in Table 10-1.

1. Identify the model building type. Each is described in Table 10-2 and in more detail in FEMA 178 (BSSC, 1992a). The building must be one of the common building types and satisfy the criteria described in Table 10-1.

2. Identify and rank all potential deficiencies for the building from Tables 10-3 through 10-21. The items in these tables are ordered roughly from highest priority at the top to lowest at the bottom, though this can vary widely in individual cases. Develop as-built information as required in *Guidelines* Section 2.7. Use the procedures in FEMA 178—and also those listed in *Guidelines* Section 10.4 for the eight new potential deficiencies—in order to evaluate fully each potential deficiency and develop a list of actual deficiencies in priority order for correction. If necessary, refer to Section C10.5 of the *Commentary* for a complete list of FEMA 178 deficiencies and their relationship to the deficiency list used here. Table 10-22 provides a cross-reference between all FEMA 178 (BSSC, 1992a) deficiencies and those of this chapter.

3. Develop strengthening details to mitigate the deficiencies using the same basic style and materials of construction. Refer to Section 10.3 and the *Commentary* for rehabilitation strategies associated with each identified deficiency. In most cases, the resulting rehabilitated building must be one of the Model Building Types. For example, adding concrete shear walls to concrete shear wall buildings or adding a complete system of concrete shear walls to a concrete frame building meets this requirement. Some exceptions include using steel bracing to strengthen wood or URM construction. For large buildings, it is advisable to explore several rehabilitation strategies and compare alternative ways of eliminating deficiencies.

4. Design the proposed rehabilitation based on the FEMA 178 (BSSC, 1992a) criteria, including its Appendix C, such that all deficiencies are eliminated.

5. Once rehabilitation techniques have been developed for all deficiencies, perform a complete evaluation of the building in its proposed rehabilitated state, following the FEMA 178 (BSSC, 1992a) procedures. This step should confirm that the strengthening of any one element or system has not merely shifted the deficiency to another.

6. To achieve the BSO, consider the rehabilitated structure's potential performance using the

Systematic Rehabilitation Method. Determine whether the total strength of the building is sufficient, and judge whether the building can experience the predicted maximum displacement without partial or complete collapse.

7. Identify and develop strengthening details for the architectural, mechanical, and electrical components. Refer to the procedures in Chapter 11 for the evaluation and rehabilitation of nonstructural elements related to the Life Safety Performance Level, given the BSE-1 earthquake.

8. Develop the needed construction documents, including drawings and specifications, and include an appropriate quality assurance program as defined in Chapter 2. If only partial rehabilitation is intended, it is recommended that the deficiencies be corrected in priority order and in a way that will facilitate fulfillment of the requirements of a higher objective at a later date. Care must be taken to ensure that a partial rehabilitation effort does not make the building's overall performance worse, such as by unintentionally channeling failure to a more critical element.

10.3 Suggested Corrective Measures for Deficiencies

Tables 10-3 to 10-21 list the potential deficiencies for the various Model Building Types. Each of these may be shown to be a deficiency that needs correction during a rehabilitation effort. (See *Commentary* Section C10.5 for a complete list of evaluation statements for identifying potential deficiencies, both those in FEMA 178 (BSSC, 1992a) and the Amendments to FEMA 178 in these *Guidelines*, Section 10.4. The following sections describe suggested corrective measures for each deficiency. They are organized into deficiency groups similar to those used in FEMA 178, and are intended to assist the thinking of the design professional. Other appropriate solutions may be used. The *Commentary* provides further discussion of the ranking of the deficiencies.

10.3.1 Building Systems

10.3.1.1 Load Path

Load path discontinuities can be mitigated by adding elements to complete the load path. This may require adding new well-founded shear walls or frames to fill in the gaps in existing shear walls or frames that are not carried continuously all the way down to the foundation. Alternatively, it may require the addition of elements throughout the building to pick up loads from diaphragms that have no path into existing vertical elements. (FEMA 178 [BSSC, 1992a], Section 3.1.)

10.3.1.2 Redundancy

The most prudent rehabilitation strategy for a building without redundancy is to add new lateral-force-resisting elements in locations where the failure of a single element will cause an instability in the building. The added lateral-force-resisting elements should be of the same stiffness as the elements they are supplementing. It is not generally satisfactory just to strengthen a nonredundant element (such as by adding cover plates to a slender brace), because its failure would still result in an instability. (FEMA 178 [BSSC, 1992a], Section 3.2.)

10.3.1.3 Vertical Irregularities

New vertical lateral-force-resisting elements can be provided to eliminate the vertical irregularity. For weak stories, soft stories, and vertical discontinuities, new elements of the same type can be added as needed. Mass and geometric discontinuities must be evaluated and strengthened based on Systematic Rehabilitation, if required by Chapter 2. (FEMA 178 [BSSC, 1992a], Sections 3.3.1 through 3.3.5.)

10.3.1.4 Plan Irregularities

The effects of plan irregularities that create torsion can be eliminated with the addition of lateral-force-resisting bracing elements that will support all major diaphragm segments in a balanced manner. While it is possible in some cases to allow the irregularity to remain and instead strengthen those structural elements that are overstressed by its existence, this may require substantial additional analysis, does not directly address the problem, and requires use of the Systematic Rehabilitation Method. (FEMA 178 [BSSC, 1992a], Section 3.3.6.)

10.3.1.5 Adjacent Buildings

Stiffening elements (typically braced frames or shear walls) can be added to one or both buildings to reduce the expected drifts to acceptable levels. With separate structures in a single building complex, it may be possible to tie them together structurally to force them to respond as a single structure. The relative stiffnesses of each and the resulting force interactions must be determined to ensure that additional deficiencies are not created. Pounding can also be eliminated by demolishing a portion of one building to increase the separation. (FEMA 178 [BSSC, 1992a], Section 3.4.)

10.3.1.6 Lateral Load Path at Pile Caps

Typically, deficiencies in the load path at the pile caps are not a life safety concern. However, if the design professional has determined that there is strong possibility of a life safety hazard due to this deficiency, piles and pile caps may be modified, supplemented, repaired, or in the most severe condition, replaced in their entirety. Alternatively, the building system may be rehabilitated such that the pile caps are protected.

10.3.1.7 Deflection Compatibility

Vertical lateral-force-resisting elements can be added to decrease the drift demands on the columns, or the ductility of the columns can be increased. Jacketing the columns with steel or concrete is one approach to increase their ductility.

10.3.2 Moment Frames

10.3.2.1 Steel Moment Frames

A. Drift

The most direct mitigation approach is to add properly placed and distributed stiffening elements—such as new moment frames, braced frames, or shear walls—that can reduce the inter-story drifts to acceptable levels. Alternatively, the addition of energy dissipation devices to the system may reduce the drift, though these are outside the scope of Simplified Rehabilitation. (FEMA 178 [BSSC, 1992a], Section 4.2.1.)

B. Frames

Noncompact members can be eliminated by adding appropriate steel plates. Eliminating or properly reinforcing large member penetrations will develop the demanded strength and deformations. Lateral bracing in the form of new steel elements can be added to reduce member unbraced lengths to within the limits prescribed. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands. (FEMA 178 [BSSC, 1992a], Sections 4.2.2, 4.2.3, and 4.2.9.)

C. Strong Column-Weak Beam

Steel plates can be added to increase the strength of the steel columns to beyond that of the beams, to eliminate this issue. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands. (FEMA 178 [BSSC, 1992a], Section 4.2.8.)

D. Connections

Adding a stiffer lateral-force-resisting system (e.g., braced frames or shear walls) can reduce the expected rotation demands. Connections can be modified by adding flange cover plates, vertical ribs, haunches, or brackets, or removing beam flange material to initiate yielding away from the connection location (e.g., via a pattern of drilled holes or the cutting out of flange material). Partial penetration splices, which may become more vulnerable for conditions where the beam-column connections are modified to be more ductile, can be modified by adding plates and/or welds. Adding continuity plates alone is not likely to enhance the connection performance significantly. (FEMA 178 [BSSC, 1992a], Sections 4.2.4, 4.2.5, 4.2.6, and 4.2.7.)

Moment-resisting connection capacity can be increased by adding cover plates or haunches, or using other techniques as stipulated in the SAC *Interim Guidelines*, FEMA 267 (SAC, 1995).

10.3.2.2 Concrete Moment Frames

A. Frame and Nonductile Detail Concerns

Adding properly placed and distributed stiffening elements such as shear walls will fully supplement the moment frame system with a new lateral-force-resisting system. For eccentric joints, columns and/or beams may be jacketed to reduce the effective eccentricity. Jackets may also be provided for shear-critical columns.

It must be verified that this new system sufficiently reduces the frame shears and inter-story drifts to acceptable levels. (FEMA 178 [BSSC, 1992a], Sections 4.3.1–4.3.15.)

B. Precast Moment Frames

Precast concrete frames without shear walls may not be addressed under the Simplified Rehabilitation Method (see Table 10-1). Where shear walls are present, the precast connections must be strengthened sufficiently to meet the FEMA 178 (BSSC, 1992a) requirements.

The development of a competent load path is extremely critical in these buildings. If the connections have sufficient strength so that yielding will first occur in the members rather than in the connections, the building should be evaluated as a shear wall system (Type C2). (FEMA 178 [BSSC, 1992a] Section 4.4.1.)

10.3.2.3 Frames Not Part of the Lateral-Force-Resisting System

A. Complete Frames

Complete frames, of steel or concrete, form a complete vertical-load-carrying system.

Incomplete frames are essentially bearing wall systems. The wall must be strengthened to resist the combined gravity/seismic loads or new columns added to complete the gravity load path. (FEMA 178 [BSSC, 1992a], Section 4.5.1.)

B. Short Captive Columns

Columns may be jacketed with steel or concrete such that they can resist the expected forces and drifts. Alternatively, the expected story drifts can be reduced throughout the building by infilling openings or adding shear walls. (Section 10.4.2.2.)

10.3.3 Shear Walls

10.3.3.1 Cast-in-Place Concrete Shear Walls

A. Shearing Stress

New shear walls can be provided and/or the existing walls can be strengthened to satisfy seismic demand criteria. New and strengthened walls must form a complete, balanced, and properly detailed lateral-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength such that yielding will first occur in the wall. All shear walls must have sufficient shear and overturning resistance to meet the FEMA 178 load criteria. (FEMA 178 [BSSC, 1992a], Section 5.1.1.)

B. Overturning

Lengthening or adding shear walls can reduce overturning demands; increasing the length of footings will capture additional building dead load. (FEMA 178 [BSSC, 1992a], Section 5.1.2.)

C. Coupling Beams

To eliminate the need to rely on the coupling beam, the walls may be strengthened as required. The beam should be jacketed only as a means of controlling debris. If possible, the opening that defines the coupling beam should be infilled. (FEMA 178 [BSSC, 1992a], Section 5.1.3.)

D. Boundary Component Detailing

Splices may be improved by welding bars together after exposing them. The shear transfer mechanism can be improved by adding steel studs and jacketing the boundary components. (FEMA 178 [BSSC, 1992a], Sections 5.1.4 through 5.1.6.)

E. Wall Reinforcement

The shear walls can be strengthened by infilling openings, or by thickening the walls (see FEMA 172 [BSSC, 1992b], Section 3.2.1.2). (FEMA 178 [BSSC, 1992a], Sections 5.1.7 and 5.1.8.)

10.3.3.2 Precast Concrete Shear Walls

A. Panel-to-Panel Connections

Appropriate Simplified Rehabilitation solutions are outlined in FEMA 172, Section 3.2.2.3. (FEMA 178 [BSSC, 1992a], Section 5.2.1.)

Inter-panel connections with inadequate capacity can be strengthened by adding steel plates across the joint, or by providing a continuous wall by exposing the reinforcing steel in the adjacent units, providing ties between the panels and patching with concrete. Providing steel plates across the joint is typically the most cost-effective approach, although care must be taken to ensure adequate anchor bolt capacity by providing adequate edge distances (see FEMA 172, Section 3.2.2).

B. Wall Openings

Infilling openings or adding shear walls in the plane of the open bays can reduce demand on the connections and eliminate frame action. (FEMA 178 [BSSC, 1992a], Section 5.2.2.)

C. Collectors

Upgrading the concrete section and/or the connections (e.g., exposing the existing connection, adding confinement ties, increasing embedment) can increase strength and/or ductility. Alternative load paths for lateral forces can be provided, and shear walls added to reduce demand on the existing collectors. (FEMA 178 [BSSC, 1992a], Section 5.2.3.)

10.3.3.3 Masonry Shear Walls

A. Reinforcing in Masonry Walls

Nondestructive methods should be used to locate reinforcement, and selective demolition used if necessary to determine the size and spacing of the reinforcing. If it cannot be verified that the wall is reinforced in accordance with the minimum requirements, then the wall should be assumed to be unreinforced, and therefore must be supplemented with new walls, or the procedures for unreinforced masonry should be followed. (FEMA 178 [BSSC, 1992a], Section 5.3.2.)

B. Shearing Stress

To meet the lateral force requirements of FEMA 178 (BSSC, 1992a), new walls can be provided, or the existing walls strengthened as needed. New and strengthened walls must form a complete, balanced, and properly detailed lateral-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength to deliver the actual lateral loads or force yielding in the wall. All shear walls must have sufficient shear and overturning resistance.

C. Reinforcing at Openings

The presence and location of reinforcing steel at openings may be established using nondestructive or destructive methods at selected locations to verify the size and location of the reinforcing, or using both methods. Reinforcing must be provided at all openings as required to meet the FEMA 178 criteria. Steel plate may be bolted to the surface of the section as long as the bolts are sufficient to yield the steel plate. (FEMA 178 [BSSC, 1992a], Section 5.3.3.)

D. Unreinforced Masonry Shear Walls

Openings in the lateral-force-resisting walls should be infilled as needed to meet the FEMA 178 (BSSC, 1992a) stress check. If supplemental strengthening is required, it should be designed using the Systematic Rehabilitation Method as defined in Chapter 2. Walls that do not meet the masonry lay-up requirements should not be considered as lateral-force-resisting elements and shall be specially supported for out-of-plane loads. (FEMA 178 [BSSC, 1992a], Sections 5.4.1, 5.4.2.)

E. Proportions of Solid Walls

Walls with insufficient thickness should be strengthened either by increasing the thickness of the wall or by adding a well-detailed strong back system. The thickened wall must be detailed in a manner that fully interconnects the wall over its full height. The strong back system must be designed for strength, connected to the structure in a manner that: (1) develops the full yield strength of the strong back, and (2) connects to the diaphragm in a manner that distributes the load into the diaphragm and has sufficient stiffness to ensure that the elements will perform in a compatible and acceptable manner. The stiffness of the bracing should limit the out-of-plane deflections to acceptable levels such as $L/600$ to $L/900$ (FEMA 178 [BSSC, 1992a], Sections 5.5.1, 5.5.2.)

F. Infill Walls

The partial infill wall should be isolated from the boundary columns to avoid a “short column” effect, except when it can be shown that the column is adequate. In sizing the gap between the wall and the columns, the anticipated inter-story drift must be considered. The wall must be positively restrained against out-of-plane failure by either bracing the top of the wall, or installing vertical girts. These bracing elements must not violate the isolation of the frame from the infill. (FEMA 178 [BSSC, 1992a], Sections 5.5.3, 4.1.1.)

10.3.3.4 Shear Walls in Wood Frame Buildings

A. Shear Stress

Walls may be added or existing openings filled. Alternatively, the existing walls and connections can be strengthened. The walls should be distributed across the building in a balanced manner to reduce the shear stress for each wall. Replacing heavy materials such as tile roofing with lighter materials will also reduce shear stress. (FEMA 178 [BSSC, 1992a], Section 5.6.1.)

B. Openings

Local shear transfer stresses can be reduced by distributing the forces from the diaphragm. Chords and/or collector members can be provided to collect and distribute shear from the diaphragm to the shear wall or bracing (see FEMA 172, Figure 3.7.1.3). Alternatively, the opening can be closed off by adding a new wall with plywood sheathing. (FEMA 178 [BSSC, 1992a], Section 5.6.2.)

C. Wall Detailing

If the walls are not bolted to the foundation or the bolting is inadequate, bolts can be installed through the sill plates at regular intervals (see FEMA 172 [BSSC, 1992b], Figure 3.8.1.2a). If the crawl space is not deep enough for vertical holes to be drilled through the sill plate, the installation of connection plates or angles may be a more practical alternative (see FEMA 172, Figure 3.8.1.2b). Sheathing and additional nailing can be added where walls lack proper nailing or connections. Where the existing connections are inadequate, adding clips or straps will deliver lateral loads to the walls and to the foundation sill plate. (FEMA 178 [BSSC, 1992a], Section 5.6.3.)

D. Cripple Walls

Where bracing is inadequate, new plywood sheathing can be added to the cripple wall studs. The top edge of the plywood is nailed to the floor framing and the bottom edge is nailed into the sill plate (see FEMA 172, Figure 3.8.1.3). Verify that the cripple wall does not change height along its length (stepped top of foundation). If it does, the shorter portion of the cripple wall will carry the majority of the shear and significant torsion will occur in the foundation. Added plywood sheathing must have adequate strength and stiffness to reduce torsion to an acceptable level. Also, it should be verified that the sill plate is properly anchored to the foundation. If anchor bolts are lacking or insufficient, additional anchor bolts should be installed. Blocking and/or framing clips may be needed to connect the cripple wall bracing to the floor diaphragm or the sill plate. (FEMA 178 [BSSC, 1992a], Section 5.6.4.)

E. Narrow Wood Shear Walls

Where narrow shear walls lack capacity, they should be replaced with shear walls with a height-to-width aspect ratio of two to one or less. These replacement walls must have sufficient strength, including being adequately connected to the diaphragm and sufficiently anchored to the foundation for shear and overturning forces. (*Guidelines* Section 10.4.3.1.)

F. Stucco Shear Walls

For strengthening or repair, the stucco should be removed, a plywood shear wall added, and new stucco applied. The plywood should be the manufacturer's recommended thickness for the installation of stucco. The new stucco should be installed in accordance with building code requirements for waterproofing. Walls should be sufficiently anchored to the diaphragm and foundation. (*Guidelines* Section 10.4.3.2.)

G. Gypsum Wallboard or Plaster Shear Walls

Plaster and gypsum wallboard can be removed and replaced with structural panel shear wall as required, and the new shear walls covered with gypsum wallboard. (*Guidelines* Section 10.4.3.3.)

10.3.4 Steel Braced Frames

10.3.4.1 System Concerns

If the strength of the braced frames is inadequate, more braced bays or shear wall panels can be added. The resulting lateral-force-resisting system must form a well-balanced system of braced frames that do not fail at their joints, and are properly connected to the floor diaphragms, and whose failure mode is yielding of braces rather than overturning. (FEMA 178 [BSSC, 1992a], Section 6.1.1.)

10.3.4.2 Stiffness of Diagonals

Diagonals with inadequate stiffness should be strengthened using the supplemental steel plates, or replaced with a larger and/or different type of section. Global stiffness can be increased by the addition of braced bays or shear wall panels. (FEMA 178 [BSSC, 1992a], Sections 6.1.2 and 6.1.3.)

10.3.4.3 Chevron or K-Bracing

Columns or horizontal girts can be added as needed to support the tension brace when the compression brace buckles, or the bracing can be revised to another system throughout the building. The beam elements can be strengthened with cover plates to provide them with the capacity to fully develop the unbalanced forces created by tension brace yielding. (FEMA 178 [BSSC, 1992a], Section 6.1.4.)

10.3.4.4 Braced Frame Connections

Column splices or other braced frame connections can be strengthened by adding plates and welds to ensure that they are strong enough to develop the connected members. Connection eccentricities that reduce member capacities can be eliminated, or the members can be strengthened to the required level by the addition of properly placed plates. Demands on the existing elements can be reduced by adding braced bays or shear wall panels. (FEMA 178 [BSSC, 1992a], Sections 6.1.5, 6.1.6, and 6.1.7.)

10.3.5 Diaphragms

10.3.5.1 Re-entrant Corners

New chords with sufficient strength to resist the required force can be added at the re-entrant corner. If a vertical lateral-force-resisting element exists at the re-entrant corner, a new shear collector element should be placed at the diaphragm, connected to the vertical element, to reduce tensile and compressive forces at the re-entrant corner. The same basic materials used in the diaphragm being strengthened should be used for the chord. (FEMA 178 [BSSC, 1992a], Section 7.1.1.)

10.3.5.2 Crossties

New crossties and wall connections can be added to resist the required out-of-plane wall forces and distribute these forces through the diaphragm. New strap plates and/or rod connections can be used to connect existing framing members together so they function as a crosstie in the diaphragm. (FEMA 178 [BSSC, 1992a], Section 7.1.2.)

10.3.5.3 Diaphragm Openings

New drag struts or diaphragm chords can be added around the perimeter of existing openings to distribute tension and compression forces along the diaphragm. The existing sheathing should be nailed to the new drag struts or diaphragm chords. In some cases it may also be necessary to: (1) increase the shear capacity of the diaphragm adjacent to the opening by overlaying the existing diaphragm with a wood structural panel, or (2) decrease the demand on the diaphragm by adding new vertical elements near the opening. (FEMA 178 [BSSC, 1992a], Sections 7.1.3 through 7.1.6.)

10.3.5.4 Diaphragm Stiffness/Strength

A. Board Sheathing

When the diaphragm does not have at least two nails through each board into each of the supporting members, and the lateral drift and/or shear demands on the diaphragm are not excessive, the shear capacity and stiffness of the diaphragm can be increased by adding nails at the sheathing boards. This method of upgrade is most often suitable in areas of low seismicity. In other cases, a new wood structural panel should be placed over the existing straight sheathing, and the joints of the wood structural panels placed so they are near the center of the sheathing boards or at a 45-degree angle to the joints between sheathing boards (see FEMA 172 [BSSC, 1992b], Section 3.5.1.2; ATC, [1981]; and FEMA 178 [BSSC, 1992a], Section 7.2.1).

B. Unblocked Diaphragms

The shear capacity of unblocked diaphragms can be improved by adding new wood blocking and nailing at the unsupported panel edges. Placing a new wood structural panel over the existing diaphragm will increase the shear capacity. Both of these methods will require the partial or total removal of existing flooring or roofing to place and nail the new overlay or nail the existing panels to the new blocking. Strengthening of the diaphragm is usually not necessary at the central area of the diaphragm where shear is low. In certain cases when the design loads are low, it may be possible to increase the shear capacity of unblocked diaphragms with sheet metal plates stapled on the underside of the existing wood panels. These plates and staples must be designed for all related shear and torsion caused by the details related to their installation. (FEMA 178 [BSSC, 1992a], Section 7.2.3.)

C. Spans

New vertical elements can be added to reduce the diaphragm span. The reduction of the diaphragm span will also reduce the lateral deflection and shear demand in the diaphragm. However, adding new vertical elements will result in a different distribution of shear demands. Additional blocking, nailing, or other rehabilitation measures may need to be provided at these areas. (FEMA 172, Section 3.4 and FEMA 178 [BSSC, 1992a], Section 7.2.2.)

D. Span-to-Depth Ratio

New vertical elements can be added to reduce the diaphragm span-to-depth ratio. The reduction of the diaphragm span-to-depth ratio will also reduce the lateral deflection and shear demand in the diaphragm. (Typical construction details and methods are discussed in FEMA 172, Section 3.4.) (FEMA 178 [BSSC, 1992a], Section 7.2.4.)

E. Diaphragm Continuity

The diaphragm discontinuity should in all cases be eliminated by adding new vertical elements at the diaphragm offset or the expansion joint (see FEMA 172, Section 3.4). In some cases, special details may be used to transfer shear across an expansion joint—while still allowing the expansion joint to function—thus eliminating a diaphragm discontinuity. (FEMA 178 [BSSC, 1992a], Section 7.2.5.)

F. Chord Continuity

If members such as edge joists, blocking, or wall top plates have the capacity to function as chords but lack connection, adding nailed or bolted continuity splices will provide a continuous diaphragm chord. New continuous steel or wood chord members can be added to the existing diaphragm where existing members lack sufficient capacity or no chord exists. New chord members can be placed at either the underside or topside of the diaphragm. In some cases, new vertical elements can be added to reduce the diaphragm span and stresses on any existing chord members (see FEMA 172, Section 3.5.1.3, and ATC-7). New chord connections should not be detailed such that they are the weakest element in the chord. (FEMA 178 [BSSC, 1992a], Section 7.2.6.)

10.3.6 Connections

10.3.6.1 Diaphragm/Wall Shear Transfer

Collector members, splice plates, and shear transfer devices can be added as required to deliver collector forces to the shear wall. Adding shear connectors from diaphragm to wall and/or to collectors will transfer shear. (See FEMA 172, Section 3.7 for Wood Diaphragms, 3.7.2 for concrete diaphragms, 3.7.3 for poured gypsum, and 3.7.4 for metal deck diaphragms.) (FEMA 178 [BSSC, 1992a], Sections 8.3.1 and 8.3.3.)

10.3.6.2 Diaphragm/Frame Shear Transfer

Adding collectors and connecting the framing will transfer loads to the collectors. Connections can be provided along the collector length and at the collector-to-frame connection to withstand the calculated forces (see FEMA 172, Sections 3.7.5 and 3.7.6). (FEMA 178 [BSSC, 1992a], Sections 8.3.2 and 8.3.3.)

10.3.6.3 Anchorage for Normal Forces

To account for inadequacies identified by FEMA 178 and in Section C10.3.6.3 of the *Commentary*, wall anchors can be added. Complications that may result from inadequate anchorage include cross-grain tension in wood ledgers, or failure of the diaphragm-to-wall connection, due to: (1) insufficient strength, number, or stability of anchors; (2) inadequate embedment of anchors; (3) inadequate development of anchors and straps into the diaphragm; and (4) deformation of anchors and their fasteners that permit diaphragm boundary connection pullout, or cross-grain tension in wood ledgers.

Existing anchors should be tested to determine load capacity and deformation potential including fastener slip, according to the requirements in Appendix C of FEMA 178 (BSSC, 1992a). Special attention should be given to the testing procedure to maintain a high level of quality control. Additional anchors should be provided as needed to supplement those that fail the test, as well as those needed to meet the FEMA 178 criteria. The quality of the rehabilitation depends greatly on the quality of the performed tests. (FEMA 178 [BSSC, 1992a], Sections 8.2.1 to 8.2.6; *Guidelines* Section 10.4.4.1.)

10.3.6.4 Girder-Wall Connections

The existing reinforcing must be exposed, and the connection modified as necessary. For out-of-plane loads, the number of column ties can be increased by jacketing the pilaster, or alternatively, by developing a second load path for the out-of-plane forces. Bearing length conditions can be addressed by adding bearing extensions. Frame action in welded connections can be mitigated by adding shear walls. (FEMA 178 [BSSC, 1992a], Sections 8.5.1 through 8.5.3.)

10.3.6.5 Precast Connections

The connections of chords, ties, and collectors can be upgraded to increase strength and/or ductility, providing alternative load paths for lateral forces. Upgrading can be achieved by such methods as adding confinement ties or increasing embedment. Shear walls can be added to reduce the demand on connections. (FEMA 178 [BSSC, 1992a], Section 4.4.2.)

10.3.6.6 Wall Panels and Cladding

It may be possible to improve the connection between the panels and the framing. If architectural or occupancy conditions warrant, the cladding can be replaced with a new system. The building can be stiffened with the addition of shear walls or braced frames, to reduce the drifts in the cladding elements. (FEMA 178 [BSSC, 1992a], Section 8.6.2.)

10.3.6.7 Light Gage Metal, Plastic, or Cementitious Roof Panels

It may be possible to improve the connection between the roof and the framing. If architectural or occupancy conditions warrant, the roof diaphragm can be replaced with a new one. Alternatively, a new diaphragm may be added, using rod braces or plywood above or below the existing roof, which remains in place. (FEMA 178 [BSSC, 1992a], Section 8.6.1.)

10.3.6.8 Mezzanine Connections

Diagonal braces, moment frames, or shear walls can be added at or near the perimeter of the mezzanine where bracing elements are missing, so that a complete and balanced lateral-force-resisting system is provided that meets the requirements of FEMA 178.

10.3.7 Foundations and Geologic Hazards

10.3.7.1 Anchorage to Foundations

For wood walls, expansion anchors or epoxy anchors can be installed by drilling through the wood sill to the concrete foundation at an appropriate spacing of four to six feet on center. Similarly, steel columns and wood posts can be anchored to concrete slabs or footings, using expansion anchors and clip angles. If the concrete or masonry walls and columns lack dowels, a concrete curb can be installed adjacent to the wall or column by drilling dowels and installing anchors into the wall that lap with dowels installed in the slab or footing. However, this curb can cause significant architectural problems. Alternatively, steel angles may be used with drilled anchors. The anchorage of shear wall boundary components can be challenging due to very high concentrated forces. (FEMA 178 [BSSC, 1992a], Sections 8.4.1 through 8.4.7.)

10.3.7.2 Condition of Foundations

All deteriorated and otherwise damaged foundations should be strengthened and repaired using the same materials and style of construction. Some conditions of material deterioration can be mitigated in the field, including patching of spalled concrete. Pest infestation or dry rot of wood piles can be very difficult to correct, and often require full replacement. The deterioration of these elements may have implications that extend beyond seismic safety and must be considered in the rehabilitation. (FEMA 178 [BSSC, 1992a], Sections 9.1.1 through 9.1.2.)

10.3.7.3 Overturning

Existing foundations can be strengthened as needed to resist overturning forces. Spread footings can be enlarged or additional piles, rock anchors, or piers added to deep foundations. It may also be possible to use grade beams or new wall elements to spread out overturning loads over a greater distance. Adding new lateral-load-resisting elements will reduce overturning effects of existing elements. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

10.3.7.4 Lateral Loads

As with overturning effects, the correction of lateral load deficiencies in the foundations of existing buildings is expensive and may not be justified by more realistic analysis procedures. For this reason, Systematic Rehabilitation is recommended for these cases. (FEMA 178 [BSSC, 1992a], Sections 9.2.1 through 9.2.5.)

10.3.7.5 Geologic Site Hazards

Site hazards other than ground shaking should be considered. Rehabilitation of structures subject to life safety hazards from ground failures is impractical, unless site hazards can be mitigated to the point where acceptable performance can be achieved. Not all ground failures need necessarily be considered as life safety hazards. For example, in many cases liquefaction beneath a building does not pose a life safety hazard; however, related lateral spreading can result in collapse of buildings with inadequate foundation strength. For this reason, the liquefaction potential and the related consequences should be thoroughly investigated for sites that do not satisfy the FEMA 178 statement. Further information on the evaluation of site hazards is provided in Chapter 4 of these *Guidelines*. (FEMA 178 [BSSC, 1992a], Sections 9.3.1 through 9.3.3.)

10.3.8 Evaluation of Materials and Conditions

10.3.8.1 General

Proper evaluation of the existing conditions and configuration of the existing building structure is an important aspect of Simplified Rehabilitation. As Simplified Rehabilitation is often concerned with specific deficiencies in a particular structural system, the evaluation can either be focused on affected structural elements and components, or be comprehensive and inclusive of the complete structure. If the degree of existing damage or deficiencies in a structure has not been established, the evaluation shall consist of a comprehensive inspection of gravity- and lateral-load-resisting systems that includes the following steps.

1. Verify existing data (e.g., accuracy of drawings).

2. Develop other needed data (e.g., measure and sketch building if necessary).

3. Verify the vertical and lateral systems.

4. Check the condition of the building.

5. Look for special conditions and anomalies.

6. Address the evaluation statements and goals during the inspection.

7. Perform material tests that are justified through a weighing of the cost of destructive testing and the cost of corrective work.

10.3.8.2 Condition of Wood

An inspection should be conducted to grade the existing wood and verify physical condition, using techniques from Section 10.3.8.1. Any damage or deterioration and its source must be identified. Wood that is significantly damaged due to splitting, decay, aging, or other phenomena must be removed and replaced. Localized problems can be eliminated by adding new appropriately sized reinforcing components extending beyond the damaged area and connecting to undamaged portions. Additional connectors between components should be provided to correct any discontinuous load paths. It is necessary to verify that any new reinforcing components or connectors will not be exposed to similar deterioration or damage. (FEMA 178 [BSSC, 1992a], Section 3.5.1.)

10.3.8.3 Overdriven Fasteners

Where visual inspection determines that extensive overdriving of fasteners exists in greater than 20% of the installed connectors, the fasteners and shear panels can generally be repaired through addition of a new same-sized fastener for every two overdriven fasteners. To avoid splitting because of closely spaced nails, it may be necessary to predrill to 90% of the nail shank diameter for installation of new nails. For other conditions, such as cases where the addition of new connectors is not possible or where component damage is suspected, further investigation shall be conducted using the guidance of Section 10.3.8.1. (FEMA 178 [BSSC, 1992a], Section 3.5.2.)

10.3.8.4 Condition of Steel

Should visual inspection or testing conducted per Section 10.3.8.1 reveal the presence of steel component or connection deterioration, further evaluation is needed. The source of the damage shall be identified and mitigative action shall be taken to preserve the remaining structure. In areas of significant deterioration, restoration of the material cross section can be performed by the addition of plates or other reinforcing. When sizing reinforcements, the design professional shall consider the effects of existing stresses in the original structure, load transfer, and strain compatibility. The demands on the deteriorated steel elements and components may also be reduced through careful addition of bracing or shear wall panels. (FEMA 178 [BSSC, 1992a], Section 3.5.3.)

10.3.8.5 Condition of Concrete

Should visual inspections or testing conducted per Section 10.3.8.1 reveal the presence of concrete component or reinforcing steel deterioration, further evaluation is needed. The source of the damage shall be identified and mitigative action shall be taken to preserve the remaining structure. Existing deteriorated material, including reinforcing steel, shall be removed to the limits defined by testing; reinforcing steel in good condition shall be cleaned and left in place for splicing purposes as appropriate. Cracks in otherwise sound material shall be evaluated to determine cause, and repaired as necessary using techniques appropriate to the source and activity level. (FEMA 178 [BSSC, 1992a], Sections 3.5.4 through 3.5.8.)

10.3.8.6 Post-Tensioning Anchors

Prestressed concrete systems may be adversely affected by cyclic deformations produced by earthquake motion. One rehabilitation process that may be considered is to add stiffness to the system. Another concern for these systems is the adverse effects of tendon corrosion. A thorough visual inspection of prestressed systems shall be performed to verify absence of concrete cracking or spalling, staining from embedded tendon corrosion, or other signs of damage along the tendon spans and at anchorage zones. If degradation is observed or suspected, more detailed evaluations will be required, as indicated in Chapter 6. Rehabilitation of these systems, except for local anchorage repair, should be in accordance with the Systematic Rehabilitation provisions in the balance of these *Guidelines*. Professionals with special prestressed concrete construction expertise should also be consulted for further interpretation of damage. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

10.3.8.7 Quality of Masonry

Should visual inspections or testing conducted per Section 10.3.8.1 reveal the presence of masonry component or construction deterioration, further evaluation is needed. Certain damage, such as degraded mortar joints or simple cracking, may be rehabilitated through repointing or rebuild. If the wall is repointed, care should be taken to ensure that the new mortar is compatible with the existing masonry units and mortar, and that suitable wetting is performed. The strength of the new mortar is critical to load-carrying capacity and seismic performance. Significant degradation should be treated as specified in Chapter 7 of these *Guidelines*. (FEMA 178 [BSSC, 1992a], Sections A4, 3.5.9, 3.5.10, and 3.5.11.)

10.4 Amendments to FEMA 178

Since the development and publication of FEMA 178 (BSSC, 1992a), significant earthquakes have occurred: the 1989 Loma Prieta earthquake in the San Francisco Bay area, the 1994 Northridge earthquake in the Los Angeles area, and the 1995 Hyogoken-Nanbu earthquake in the Kobe, Japan area. While each one generally validated the fundamental assumptions underlying the procedures, each also offered new insights into the potential weaknesses of certain lateral-force-resisting systems.

In the process of developing the *Guidelines* and *Commentary*, eight new potential deficiencies were identified and are developed below. They are presented in the same style as in FEMA 178 (BSSC, 1992a). Each is presented as a statement to be answered “True” or “False,” which permits rapid screening and identification of potential weak links. Each statement is followed by a paragraph of commentary written to identify the concern clearly. A suggested procedure for evaluating the potential weak link concludes each section, and should be carried out if the statement is found to be false. Completion of the procedure permits each potential deficiency to be properly evaluated and the actual deficiencies identified.

These eight new potential deficiencies should be considered as additions to the general list of building deficiencies (pages A3 to A16 of FEMA 178 [BSSC, 1992a]) and applied to the individual model buildings as indicated in Tables 10-3 through 10-20.

10.4.1 New Potential Deficiencies Related to Building Systems

10.4.1.1 Lateral Load Path at Pile Caps

Evaluation Statement: Pile caps are capable of transferring lateral and overturning forces between the structure and individual piles in the pile group.

Common problems with pile caps include a lack of top reinforcing in the pile cap. A loss of bond of pile and column reinforcing can occur when top cracks form during load reversals.

Procedure: Calculate the moment and shear capacity of the pile cap to transfer uplift and lateral forces based on the forces in FEMA 178 (BSSC, 1992a), from the point of application to each pile.

10.4.1.2 Deflection Compatibility

Evaluation Statement: Column and beam assemblies that are not part of the lateral-force-resisting system (i.e., gravity-load-resisting frames) are capable of accommodating imposed building drifts, including amplified drift caused by diaphragm deflections, without loss of their vertical-load-carrying capacity.

Frame components, especially columns, that are not specifically designed to participate in the lateral system will still undergo displacements associated with the overall seismic story drifts. If the columns are located far from the lateral-force-resisting elements, the added deflections due to semi-rigid floor diaphragms will increase the drifts. Stiff columns, designed for potentially high gravity loads, may develop significant bending moments due to the imposed drifts. The moment-axial force interaction may lead to brittle failures in nonductile columns, which could cause building collapse.

Procedure: Calculate expected drifts on the columns in frames that are not part of the lateral-force-resisting system, using procedures described in FEMA 178 (BSSC, 1992a), Section 2.4.4. Use cracked/transformed sections for all lateral-force-resisting concrete elements. Calculate additional drift from diaphragms by determining the deflection of the diaphragm at forces equal to those prescribed in FEMA 178, Chapter 2, for elements of structures. Evaluate the capacity of the non-lateral-force-resisting column and beam assemblies to undergo the combined drift, considering moment-axial interaction and column shear.

10.4.2 New Potential Deficiencies Related to Moment Frames

10.4.2.1 Moment-Resisting Connections

Evaluation Statement: All moment connections are able to develop the strength of the adjoining members or panel zones.

Connection failure is generally not ductile behavior. It is more desirable to have all inelastic action occur in the members rather than in the connections. The moment-resisting beam-column connection should provide for the development of the lesser of (1) the plastic girder strength in flexure or (2) the moment corresponding to the development of the panel zone shear strength, considering the effects of strain hardening and material overstrength. The deficiency is in the strength of the connections.

Procedure: At the time of this writing, this problem is the subject of the FEMA-funded effort carried out by the SAC Joint Venture, which is composed of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe), and which has produced interim guidance on the evaluation, repair, and rehabilitation of steel moment frames (SAC, 1995).

Using the latest guidelines, demonstrate by test or calculation that the connection meets the expected inelastic rotation demand on the joint, and that inelastic action is not concentrated in the vicinity of welds at the column face.

10.4.2.2 Short Captive Columns

Evaluation Statement: There are no columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level.

Short captive columns (which are usually not designed as part of the primary lateral-load-resisting system) tend to attract shear forces because of their high stiffness relative to other lateral-force-resisting vertical elements at that story level. Significant damage has been observed in parking structure columns adjacent to ramping slabs, even in structures with shear walls. Captive column behavior may also be found in buildings with clerestory windows, in buildings where columns are partially braced by masonry or concrete nonstructural construction, and in buildings with improperly designed mezzanines.

Procedure: Calculate the anticipated story drift, and determine the shear (V_e) demand in the short column caused by the drift ($V_e = 2M/L$). Compare V_e with the member nominal shear capacity (V_n) calculated in accordance with ACI (1989) Chapter 21. The ratio V_e / V_n should be less than or equal to 1.0.

10.4.3 New Potential Deficiencies Related to Shear Walls

10.4.3.1 Narrow Wood Shear Walls

Evaluation Statement: Narrow wood shear walls with an aspect ratio greater than two to one do not resist forces developed in the building.

Most of the deformation of the narrow shear walls occurs at the base, and consists of sliding of the sill plate and stretching of hold-down attachments. Splitting of the end studs at the attachment of hold-downs is also a common failure. Narrow shear walls are relatively flexible and thus tend to take less shear than would be anticipated when compared to wider shear walls. This results in greater loading of the shear walls with lower height-to-width ratios and less load in the narrow walls.

Procedure: Determine the shear capacity of the wall and related overturning demand. Verify that shear and overturning can be transferred to the foundation within allowable stresses calculated in accordance with FEMA 178 (BSSC, 1992a).

10.4.3.2 Stucco (Exterior Plaster) Shear Walls

Evaluation Statement: Multistory buildings do not rely on exterior stucco walls as the primary lateral-force-resisting system.

Exterior stucco plaster walls are often used (intentionally and unintentionally) for resisting lateral earthquake loads. Stucco is relatively stiff and brittle, with low shear resistance value. Differential foundation movement and earthquake shaking cause cracking of the stucco and loss of lateral strength. The cracking can range from minor to severe. Sometimes the stucco delaminates from the framing and the lateral-force-resisting system is lost. Multistory buildings shall not rely on stucco walls as the primary lateral-force-resisting system since there is not enough available strength.

Procedure: Inspect stucco clad buildings to determine if there is a lateral system such as plywood or diagonal sheathing in at least all but the top floor. Where exterior plaster is utilized and there is a supplemental system, verify that the wire reinforcing is attached directly to the wall framing and the wire is completely embedded in the plaster material. Verify that lateral loads do not exceed 100 pounds per linear foot.

10.4.3.3 Gypsum Wallboard or Plaster Shear Walls

Evaluation Statement: Interior plaster or gypsum wallboard is not being used for shear walls on buildings over one story in height.

Gypsum wallboard or gypsum plaster sheathing tends to be easily damaged by differential foundation movement or earthquake shaking. Most residential buildings have numerous walls constructed with plaster or gypsum wallboard. Though the capacity of these walls is low, the amount of wall is often high. As a result, plaster and gypsum wallboard walls may provide adequate resistance to moderate earthquake shaking. The problem that can occur is incompatibility with other lateral-forcing-resisting elements. For example, narrow plywood shear walls are more flexible than long stiff plaster walls; as a result, the plaster or gypsum walls will take all the load until they fail and then the plywood walls will start to resist the lateral loads. Plaster or gypsum wallboard walls should not be used for shear walls except for one-story buildings or on the top story of multistory buildings.

Procedure: Determine the walls with plaster or gypsum sheathing that would be required to resist lateral earthquake forces (i.e., earthquake loads would have to pass through these walls), due to the location of the walls in the building. Verify that all walls have been properly constructed with nailing required by FEMA 222A (BSSC, 1995), and that loads are within allowable limits. Remove gypsum wallboard and plaster as required, and replace with panel shear walls. Cover the new shear walls with gypsum wallboard and plaster.

10.4.4 New Potential Deficiencies Related to Connections

10.4.4.1 Stiffness of Wall Anchors

Evaluation Statement: Anchors of heavy concrete or masonry walls to wood structural elements are installed taut and are stiff enough to prevent movement between the wall and roof. If bolts are used, the bolt holes in both the connector and framing are a maximum of 1/16" larger than the bolt diameter.

The small separation that can occur between the wall and roof sheathing, due to anchors that are not taut, requires movement before taking hold and can result in an out-of-plane failure of the ledger support. Bolts in oversized holes can also cause slippage and separation between the wall and framing.

Procedure: Field check that no anchor has a twist, kink, or offset, and that no anchor is otherwise installed such that some separation must occur prior to its taking hold, and that such movement will lead to a perpendicular-to-grain bending failure in the wood ledger. Remove a representative sample of bolts and verify that the holes are not oversized. For oversized holes, replace bolts and fill gaps with epoxy or other suitable filler.

10.5 FEMA 178 Deficiency Statements

No guidelines are provided for this section. See the *Commentary* for a complete list—augmented with the eight new deficiency statements from Section 10.4, above—presented in a logical, combined order.

10.6 Definitions

Boundary component (boundary member): A member at the perimeter (edge or opening) of a shear wall or horizontal diaphragm that provides tensile and/or compressive strength.

Column (or beam) jacketing: A method in which a concrete column or beam is covered with a steel or concrete “jacket” in order to strengthen and/or repair the member by confining the concrete.

Coupling beam: Flexural member that ties or couples adjacent shear walls acting in the same plane. A coupling beam is designed to yield and dissipate inelastic energy, and, when properly detailed and proportioned, has a significant effect on the overall stiffness of the coupled wall.

Crosstie: A beam or girder that spans across the width of the diaphragm, accumulates the wall loads, and transfers them, over the full depth of the diaphragms, into the next bay and onto the nearest shear wall or frame.

Diaphragm chord: A diaphragm component provided to resist tension or compression at the edges of the diaphragm.

Drag strut: A component parallel to the applied load that collects and transfers diaphragm shear forces to the vertical lateral-force-resisting components or elements, or distributes forces within a diaphragm.

Flexible diaphragm: A diaphragm consisting of one of the following systems: plywood sheathing, spaced timber sheathing, straight timber sheathing, diagonal timber sheathing, metal deck without concrete fill, corrugated transit panels, or steel rod bracing or other steel bracing using light members such as angles or split tees.

Inter-story drift: The relative horizontal displacement of two adjacent floors in a building. Inter-story drift can also be expressed as a percentage of the story height separating the two adjacent floors.

Load path: A path that seismic forces pass through to the foundation of the structure and, ultimately, to the soil. Typically, the load travels from the diaphragm through connections to the vertical lateral-force-resisting elements, and then proceeds to the foundation by way of additional connections.

Model Building Type: Fifteen common building types used to categorize expected deficiencies, reasonable rehabilitation methods, and estimated costs. See Table 10-2 for descriptions of Model Building Types.

Narrow wood shear wall: Wood shear walls with an aspect ratio (height to width) greater than two to one. These walls are relatively flexible and thus tend to be incompatible with other building components, thereby taking less shear than would be anticipated when compared to wider walls.

Noncompact member: A steel section in compression whose width-to-thickness ratio does not meet the limiting values for compactness, as shown in Table B5.1 of AISC (1986).

Overturning: Action resulting when the moment produced at the base of vertical lateral-force-resisting elements is larger than the resistance provided by the foundation's uplift resistance and building weight.

Panel zone: Area of a column at the beam-to-column connection delineated by beam and column flanges.

Plan irregularity: Horizontal irregularity in the layout of vertical lateral-force-resisting elements, producing a misalignment between the center of mass and center of rigidity that typically results in significant torsional demands on the structure.

Pounding: Two adjacent buildings coming into contact during earthquake excitation because they are too close together and/or exhibit different dynamic deflection characteristics.

Re-entrant corner: Plan irregularity in a diaphragm, such as an extending wing, plan inset, or E-, T-, X-, or L-shaped configuration, where large tensile and compressive forces can develop.

Redundancy: Quality of having alternative paths in the structure by which the lateral forces are resisted, allowing the structure to remain stable following the failure of any single element.

Rehabilitation Objective: A statement of the desired limits of damage or loss for a given seismic demand, usually selected by the owner, engineer, and/or relevant public agencies. (See Chapter 2.)

Repointing: A method of repairing a cracked or deteriorating mortar joint in masonry. The damaged or deteriorated mortar is removed and the joint is refilled with new mortar.

Short captive column: Columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level. These columns, which may not be designed as part of the primary lateral-load-resisting system, tend to attract shear forces because of their high stiffness relative to adjacent elements.

Simplified Rehabilitation Method: An approach, applicable to some types of buildings and Rehabilitation Objectives, in which analyses of the entire building's response to earthquake hazards are not required.

Stiff diaphragm: A diaphragm consisting of one of the following systems: monolithic reinforced concrete slabs, precast concrete slabs or planks bonded together by a reinforced topping slab or by welded inserts, concrete-filled metal deck, or masonry arches with or without concrete fill or topping.

Strong column-weak beam: A connection required to localize damage and control drift; the capacity of the column in any moment frame joint must be greater than that of the beams, to ensure inelastic action in the beams.

Strong back system: A secondary system, such as a frame, commonly used to provide out-of-plane support for an unreinforced or under-reinforced masonry wall.

Systematic Rehabilitation Method: An approach to rehabilitation in which complete analysis of the building's response to earthquake shaking is performed.

Vertical irregularity: A discontinuity of strength, stiffness, geometry, or mass in one story with respect to adjacent stories.

10.7 Symbols

L Length of the column

M Moment expected in the column at maximum expected drift

V_e Shear demand in the column caused by the drift

V_n Nominal shear capacity of a column

10.8 References

ACI, 1989, *Building Code Requirements for Reinforced Concrete*, ACI 318-89, American Concrete Institute, Detroit, Michigan.

ATC, 1981, *Guidelines for the Design of Horizontal Wood Diaphragms*, Report No. ATC-7, Applied Technology Council, Redwood City, California.

BSSC, 1992a, *NEHRP Handbook for the Seismic Evaluation of Existing Building*s, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 178), Washington, D.C.

BSSC, 1992b, *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 172), Washington, D.C.

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report Nos. FEMA 222A and 223A), Washington, D.C.

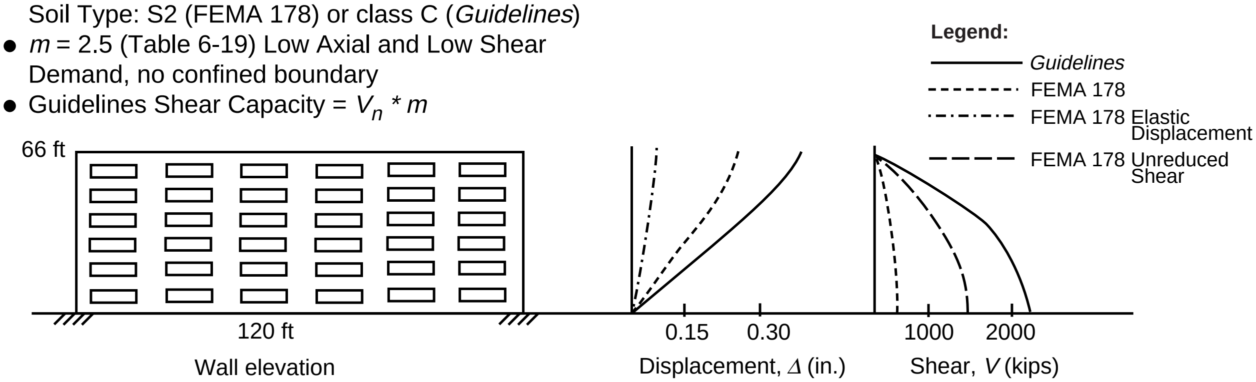
Masonry Standards Joint Committee (MSJC), 1995, *Building Code Requirements for Masonry Structures*, ACI 530-95/ASCE 5-95/TMS 402-95, American Concrete Institute, Detroit, Michigan; American Society of Civil Engineers, New York, New York; and the Masonry Society, Boulder, Colorado.

SAC, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames*, Report No. SAC-95-02, developed by the SAC Joint Venture (the Structural Engineers Association of California [SEAOC], the Applied Technology Council [ATC], and California Universities for Research in Earthquake Engineering [CUREe]) for the Federal Emergency Management Agency (Report No. FEMA 267), Washington, D.C.

Six-Story Reinforced Concrete Shear Wall Building in Low-Seismicity Region

- Low-seismicity region (NEHRP Region 1, BSSC [1988])
- 120-foot square in plan with 8-inch-thick reinforced concrete exterior walls and 9-inch-thick reinforced concrete floor slabs
- Life Safety Performance Level
- 10%/50 year ground motion (*Guidelines*)
  Soil Type: S2 (FEMA 178) or class C (*Guidelines*)
- *m* = 2.5 (Table 6-19) Low Axial and Low Shear Demand, no confined boundary
- Guidelines Shear Capacity = $V_n * m$

		Maximum pier shear		
	Base shear	Demand	Capacity	Dema
FEMA 178	300 kips	33 psi	276 psi	0.12
Guidelines	2117 kips	230 psi	690 psi	0.21



Three-Story Reinforced Concrete Shear Wall Building in High-Seismicity Region

- High-seismicity region (NEHRP Region 7, BSSC [1988])
- 120-foot square in plan with 8-inch-thick reinforced concrete exterior walls and 9-inch-thick reinforced concrete floor slabs
- Life Safety Performance Level
- 10%/50 year ground motion (*Guidelines*)
- Soil Type: S2 (FEMA 178) or class C (*Guidelines*)

		Maximum pier shear		
	Base shear	Demand	Capacity	Dema
FEMA 178	1157 kips	126 psi	276 psi	0.46
Guidelines	6091 kips	659 psi	829 psi	0.79

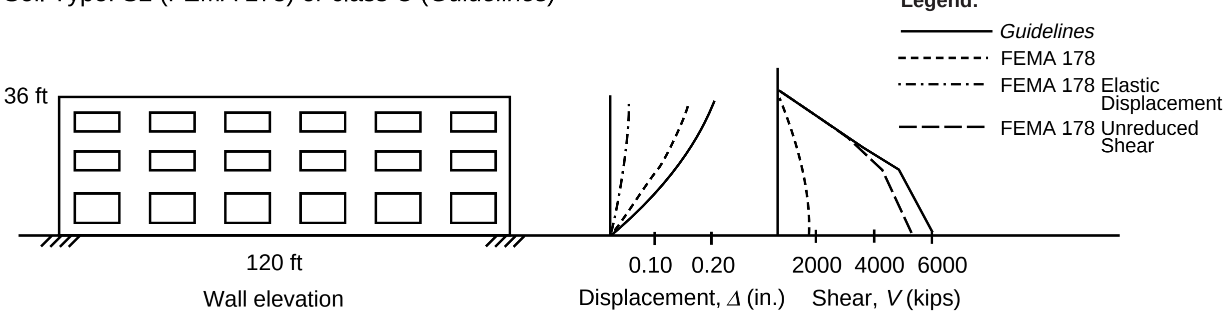


Figure 10-1 Comparison of FEMA 178 (BSSC, 1992a) and Guidelines Acceptance Criteria

Table 10-1 Limitations on Use of Simplified Rehabilitation Method

Model Building Type[2]	Maximum Building Height in Stories by Seismic Zone[1] for Use of Simplified Rehabilitation Method		
	Low	**Moderate**	**High**
Wood Frame			
Light (W1)	3	3	2
Multistory Multi-Unit Residential (W1A)	3	3	2
Commercial and Industrial (W2)	3	3	2
Steel Moment Frame			
Stiff Diaphragm (S1)	6	4	3
Flexible Diaphragm (S1A)	4	4	3
Steel Braced Frame			
Stiff Diaphragm (S2)	6	4	3
Flexible Diaphragm (S2A)	3	3	3
Steel Light Frame (S3)	2	2	2
Steel Frame with Concrete Shear Walls (S4)	6	4	3
Steel Frame with Infill Masonry Shear Walls			
Stiff Diaphragm (S5)	3	3	
Flexible Diaphragm (S5A)	3	3	
Concrete Moment Frame (C1)	3		
Concrete Shear Walls			
Stiff Diaphragm (C2)	6	4	3
Flexible Diaphragm (C2A)	3	3	3
Concrete Frame with Infill Masonry Shear Walls			
Stiff Diaphragm (C3)	3		
Flexible Diaphragm (C3A)	3		
Precast/Tilt-up Concrete Shear Walls			
Flexible Diaphragm (PC1)	3	2	2
Stiff Diaphragm (PC1A)	3	2	2
Precast Concrete Frame			
With Shear Walls (PC2)	3	2	
Without Shear Walls (PC2A)			
Reinforced Masonry Bearing Walls			
Flexible Diaphragm (RM1)	3	3	3
Stiff Diaphragm (RM2)	6	4	3

= Use of Simplified Rehabilitation Method not appropriate.

1. Seismic Zones are defined in Chapter 2 of the *Guidelines*.
2. Buildings with different types of flexible diaphragms may be considered to have flexible diaphragms.
   Multistory buildings having stiff diaphragms at all levels except the roof may be considered as having stiff diaphragms.
   Buildings having both flexible and stiff diaphragms, or having diaphragm systems that are neither flexible nor stiff, in accordance with this chapter, shall be rehabilitated using the Systematic Method.

Table 10-1 Limitations on Use of Simplified Rehabilitation Method (continued)

Model Building Type[2]	Maximum Building Height in Stories by Seismic Zone[1] for Use of Simplified Rehabilitation Method		
	Low	**Moderate**	**High**
Unreinforced Masonry Bearing Walls			
Flexible Diaphragm (URM)	3	3	2
Stiff Diaphragm (URMA)	3	3	2

(shaded) = Use of Simplified Rehabilitation Method not appropriate.

1. Seismic Zones are defined in Chapter 2 of the *Guidelines*.
2. Buildings with different types of flexible diaphragms may be considered to have flexible diaphragms.
   Multistory buildings having stiff diaphragms at all levels except the roof may be considered as having stiff diaphragms.
   Buildings having both flexible and stiff diaphragms, or having diaphragm systems that are neither flexible nor stiff, in accordance with this chapter, shall be rehabilitated using the Systematic Method.

Table 10-2 Description of Model Building Types

Model Building Type designations are similar to those used in FEMA 178 (BSSC, 1992a). Designations ending in the letter A indicate types added to the FEMA 178 list. Refer to FEMA 178 for additional information.

Building Type 1—Wood, Light Frame	
Type W1:	These building are typically single- or multiple-family dwellings of one or more stories. The essential structural character of this type is repetitive framing by wood joists on wood studs. Loads are light and spans are small. These buildings may have relatively heavy chimneys and may be partially or fully covered with veneer. Most of these buildings are not engineered; however, they usually have the components of a lateral-force-resisting system even though it may be incomplete. Lateral loads are transferred by diaphragms to shear walls. The diaphragms are roof panels and floors. Shear walls are exterior walls sheathed with plank siding, stucco, plywood, gypsum board, particleboard, or fiberboard. Interior partitions are sheathed with plaster or gypsum board.
Type W1A:	Similar to W1 buildings, but are typically multistory multi-unit residential structures, often with open front garages at the first story.
Building Type 2—Wood, Commercial and Industrial	
Type W2:	These buildings usually are commercial or industrial buildings with a floor area of 5,000 square feet or more and few, if any, interior walls. The essential structural character is framing by beams on columns. The beams may be glulam beams, steel beams, or trusses. Lateral forces usually are resisted by wood diaphragms and exterior walls sheathed with plywood, stucco, plaster, or other paneling. The walls may have rod bracing. Large openings for stores and garages often require post-and-beam framing. Lateral force resistance on those lines can be achieved with steel rigid frames or diagonal bracing.
Building Type 3—Steel Moment Frame	
Type S1:	These buildings have a frame of steel columns and beams. In some cases, the beam-column connections have very small moment-resisting capacity, but in other cases some of the beams and columns are fully developed as moment frames to resist lateral forces. Usually the structure is concealed on the outside by exterior walls, which can be of almost any material (curtain walls, brick masonry, or precast concrete panels), and on the inside by ceilings and column furring. Lateral loads are transferred by diaphragms to moment-resisting frames. The diaphragms are typically concrete or metal deck with concrete fill, and are considered stiff with respect to the frames. The frames develop their stiffness by full or partial moment connections. The frames can be located almost anywhere in the building. Usually the columns have their strong directions oriented so that some columns act primarily in one direction while the others act in the other direction, and the frames consist of lines of strong columns and their intervening beams. Steel moment frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large inter-story drifts that may lead to extensive nonstructural damage.
Type S1A:	Similar to Type S1, except that diaphragms are typically wood, tile arch, or bare metal deck and are considered flexible with respect to the frames. Concrete or metal deck with concrete fill diaphragms may be considered flexible if the span-to-depth ratio between lines of moment frames is high. Steel frame with wood or tile floors is more common in older styles of construction.
Building Type 4—Steel Braced Frame	
Type S2:	These buildings are similar to Type 3 (S1) buildings, except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.
Type S2A:	These buildings are similar to Type 3 (S1A) buildings, except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.
Building Type 5—Steel Light Frame	
Type S3:	These buildings are pre-engineered and prefabricated with transverse rigid frames. The roof and walls consist of lightweight panels. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of tension-only rod bracing, or a combination of panels and bracing.

Table 10-2 Description of Model Building Types (continued)

Model Building Type designations are similar to those used in FEMA 178 (BSSC, 1992a). Designations ending in the letter A indicate types added to the FEMA 178 list. Refer to FEMA 178 for additional information.

Building Type 6—Steel Frame with Concrete Shear Walls	
Type S4:	The shear walls in these buildings are cast-in-place concrete and may be bearing walls. The steel frame is designed for vertical loads only. Lateral loads are transferred by diaphragms—typically of cast-in-place concrete—to the shear walls. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections. In modern “dual” systems, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidities. In this case, the walls would be evaluated under this building type and the frames would be evaluated under Building Type 3, Steel Moment Frame.
Building Type 7—Steel Frame with Infill Masonry Shear Walls	
Type S5:	This is one of the older types of building. The infill walls usually are offset from the exterior frame members, wrap around them, and present a smooth masonry exterior with no indication of the frame. Solidly infilled masonry panels act as a diagonal compression strut between the intersections of the moment frame. If the walls do not fully engage the frame members (i.e., lie in the same plane), the diagonal compression struts will not develop. The peak strength of the diagonal strut is determined by the diagonal tensile stress capacity of the masonry panel. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The analysis should be based on published research and should treat the system as a composite of a frame and the infill. An analysis that attempts to treat the system as a frame and shear wall is not capable of assuring compatibility. Diaphragms are typically concrete or tile arch with short spans between infill walls, and are considered stiff with respect to the walls.
Type S5A:	Similar to Type S5, except that diaphragms either are wood, or contain concrete or tile floors with a high span-to-depth ratio between infill walls, and are considered flexible with respect to the walls.
Building Type 8—Concrete Moment Frame	
Type C1:	These buildings are similar to Type 3 (S1) buildings, except that the frames are of concrete. Some older concrete frames may be proportioned and detailed such that brittle failure can occur. There is a large variety of frame systems. Buildings in zones of low seismicity or older buildings in zones of high seismicity can have frame beams that have broad shallow cross sections or are simply the column strips of flat slabs. Modern frames in zones of high seismicity are detailed for ductile behavior and the beams and columns have definitely regulated proportions. Diaphragms are typically concrete.
Building Type 9—Concrete Shear Walls	
Type C2:	The vertical components of the lateral-force-resisting system in these buildings are concrete shear walls that are usually bearing walls. In older buildings, the walls often are quite extensive and the wall stresses are low, but reinforcing is light. When remodeling calls for enlarging the windows, the strength of the modified walls becomes a critical concern. In newer buildings, the shear walls often are limited in extent, thus generating concerns about boundary members and overturning forces. Diaphragms are typically cast-in-place concrete slabs with or without beams and are considered stiff with respect to the walls.
Type C2A:	Similar to Type C2, except that diaphragms are typically wood and are considered flexible with respect to the frames. This is typically evident in older styles of construction. Concrete diaphragms may be considered flexible if the span-to-depth ratio between shear walls is high. This is common in parking structures and in buildings with narrow aspect ratios.
Building Type 10—Concrete Frame with Infill Masonry Shear Walls	
Type C3:	These buildings are similar to Type 7 (S5) buildings except that the frame is of reinforced concrete. The analysis of this building is similar to that recommended for Type 7 (S5), except that the shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. Research that is specific to confinement of the infill by reinforced concrete frames should be used for the analysis.
Type C3A:	Similar to Type C3, except that diaphragms either are wood, or contain concrete or tile floors with a high span-to-depth ratios between infill walls, and are considered flexible with respect to the walls.

Table 10-2 Description of Model Building Types (continued)

Model Building Type designations are similar to those used in FEMA 178 (BSSC, 1992a). Designations ending in the letter A indicate types added to the FEMA 178 list. Refer to FEMA 178 for additional information.

Building Type 11—Precast/Tilt-up Concrete Shear Walls	
Type PC1:	These buildings have a wood or metal deck roof diaphragm, which often is very large, that distributes lateral forces to precast concrete shear walls and is considered flexible with respect to the walls. They may also have precast concrete diaphragms if the span-to-depth ratio between walls is very high or there is no topping slab. The walls are thin but relatively heavy, while the roofs are relatively light. Older buildings often have inadequate connections for anchorage of the walls to the roof for out-of-plane forces, and the panel connections often are brittle. Tilt-up buildings often have more than one story. Walls may have numerous openings for doors and windows of such size that the wall looks more like a frame than a shear wall.
Type PC1A:	Similar to Type PC1, except that diaphragms are precast or cast-in-place concrete with small span-to-depth ratios, and are considered stiff with respect to the walls.
Building Type 12—Precast Concrete Frame	
Type PC2:	These buildings contain floor and roof diaphragms typically composed of precast concrete elements with or without cast-in-place concrete topping slabs. The diaphragms are supported by precast concrete girders and columns. The girders often bear on column corbels. Closure strips between precast floor elements and beam-column joints usually are cast-in-place concrete. Welded steel inserts often are used to interconnect precast elements. Lateral loads are resisted by precast or cast-in-place concrete shear walls. Buildings with precast frames and concrete shear walls should perform well if the details used to connect the structural elements have sufficient strength and displacement capacity; however, in some cases the connection details between the precast elements have negligible ductility.
Type PC2A:	Similar to Type PC2, except that lateral loads are resisted by the concrete frames directly without the presence of shear walls. This type of construction is not permitted in regions of high seismicity.
Building Type 13—Reinforced Masonry Bearing Walls with Flexible Diaphragms	
Type RM1:	These buildings have bearing and shear walls of reinforced brick or concrete-block masonry, which are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing, or with steel beams with metal deck with or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns; steel beams are supported by steel columns.
Building Type 14—Reinforced Masonry Bearing Walls with Stiff Diaphragms	
Type RM2:	These buildings have walls similar to those of Type 13 (RM1) buildings, but the roof and floors are composed of precast concrete elements such as planks or T-beams, and the precast roof and floor elements are supported on interior beams and columns of steel or concrete (cast-in-place or precast). The precast horizontal elements often have a cast-in-place topping.
Building Type 15—Unreinforced Masonry Bearing Walls	
Type URM:	These buildings include structural elements that vary depending on the building's age and, to a lesser extent, its geographic location. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In buildings built after 1950, unreinforced masonry building with wood floors usually have plywood rather than board sheathing. The diaphragms are considered flexible with respect to the walls. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can have the effect of reducing diaphragm displacements.
Type URMA:	Similar to Type URM, except that the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. This is more common in older, large, multistory buildings. In regions of lower seismicity, buildings of this type constructed more recently can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The diaphragms are considered stiff with respect to the walls.

Table 10-3 W1: Wood Light Frame

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities

Shear Walls in Wood Frame Buildings
- Shear Stress
- Openings
- Wall Detailing
- Cripple Walls
- Narrow Wood Shear Walls
- Stucco Shear Walls
- Gypsum Wallboard or Plaster Shear Walls

Diaphragm Openings
Diaphragm Stiffness/Strength
- Spans
- Diaphragm Continuity

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Condition of Wood

Table 10-4 W1A: Multistory, Multi-Unit, Wood Frame Construction

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities

Shear Walls in Wood Frame Buildings
- Shear Stress
- Openings
- Wall Detailing
- Cripple Walls
- Narrow Wood Shear Walls
- Stucco Shear Walls
- Gypsum Wallboard or Plaster Shear Walls

Diaphragm Openings
Diaphragm Stiffness/Strength
- Spans
- Diaphragm Continuity

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Condition of Wood

Table 10-5 W2: Wood, Commercial, and Industrial

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities

Shear Walls in Wood Frame Buildings
- Shear Stress
- Openings
- Wall Detailing
- Cripple Walls
- Narrow Wood Shear Walls
- Stucco Shear Walls
- Gypsum Wallboard or Plaster Shear Walls

Diaphragm Openings
Diaphragm Stiffness/Strength
- Sheathing
- Unblocked Diaphragms
- Spans
- Span-to-Depth Ratio
- Diaphragm Continuity
- Chord Continuity

Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards

Condition of Wood

Table 10-6 S1 and S1A: Steel Moment Frames with Stiff or Flexible Diaphragms

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Adjacent Buildings
Lateral Load Path at Pile Caps

Steel Moment Frames
- Drift Check
- Frame Concerns
- Strong Column-Weak Beam
- Connections

Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength

Diaphragm/Frame Shear Transfer

Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards

Condition of Steel

Table 10-7 S2 and S2A: Steel Braced Frames with Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps
Stress Level Stiffness of Diagonals Chevron or K-Bracing Braced Frame Connections
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Steel

Table 10-8 S3: Steel Light Frames

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities
Steel Moment Frames   Frame Concerns
Masonry Shear Walls   Infill Walls

Steel Braced Frames
  Stress Level
  Braced Frame Connections

Re-entrant Corners Diaphragm Openings
Diaphragm/Frame Shear Transfer Wall Panels and Cladding Light Gage Metal, Plastic, or Cementitious Roof Panels
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Condition of Steel

Table 10-9 S4: Steel Frames with Concrete Shear Walls

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps

Cast-in-Place Concrete Shear Walls
  Shear Stress
  Overturning
  Coupling Beams
  Boundary Component Detailing
  Wall Reinforcement

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Steel Condition of Concrete

Table 10-10 S5, S5A: Steel Frames with Infill Masonry Shear Walls and Stiff or Flexible Diaphragms

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Lateral Load Path at Pile Caps

Frames Not Part of the Lateral Force Resisting System
  Complete Frames

Masonry Shear Walls
  Reinforcing in Masonry Walls
  Shear Stress
  Reinforcing at Openings
  Unreinforced Masonry Shear Walls
  Proportions, Solid Walls
  Infill Walls

Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength
  Span/Depth Ratio

Diaphragm/Wall Shear Transfer
Anchorage for Normal Forces

Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards

Condition of Steel
Quality of Masonry

Table 10-11 C1: Concrete Moment Frames

Typical Deficiencies

Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Adjacent Buildings
Lateral Load Path at Pile Caps
Deflection Compatibility

Concrete Moment Frames
  Quick Checks, Frame and Nonductile Detail Concerns
  Precast Moment Frame Concerns
Frames Not Part of the Lateral Force Resisting System
  Short “Captive” Columns

Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength

Diaphragm/Frame Shear Transfer
Precast Connections

Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards

Condition of Concrete

Table 10-12 C2, C2A: Concrete Shear Walls with Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps Deflection Compatibility
Frames Not Part of the Lateral Force Resisting System Short "Captive" Columns
Cast-in-Place Concrete Shear Walls Shear Stress Overturning Coupling Beams Boundary Component Detailing Wall Reinforcement
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength Sheathing
Diaphragm/Wall Shear Transfer
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-13 C3, C3A: Concrete Frames with Infill Masonry Shear Walls and Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps Deflection Compatibility
Frames Not Part of the Lateral Force Resisting System Complete Frames
Masonry Shear Walls Reinforcing in Masonry Walls Shear Stress Reinforcing at Openings Unreinforced Masonry Shear Walls Proportions, Solid Walls Infill Walls
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength Span/Depth Ratio
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete Quality of Masonry

Table 10-14 PC1: Precast/Tilt-up Concrete Shear Walls with Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Deflection Compatibility
Precast Concrete Shear Walls   Panel-to-Panel Connections
Wall Openings   Collectors

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength
   Sheathing
   Unblocked Diaphragms
   Span/Depth Ratio
   Chord Continuity

Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Girder/Wall Connections Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundation Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-15 PC1A: Precast/Tilt-up Concrete Shear Walls with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities

Precast Concrete Shear Walls
   Panel-to-Panel Connections
   Wall Openings
   Collectors

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Girder/Wall Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-16	*PC2: Precast Concrete Frames with Shear Walls*

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps Deflection Compatibility
Concrete Moment Frames   Precast Moment Frame Concerns

Cast-in-Place Concrete Shear Walls
   Shear Stress
   Overturning
   Coupling Beams
   Boundary Component Detailing
   Wall Reinforcement

Re-entrant Corners Crossties Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Girder/Wall Connections Precast Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-17	*PC2A: Precast Concrete Frames Without Shear Walls*

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings Lateral Load Path at Pile Caps Deflection Compatibility

Concrete Moment Frames
   Precast Moment Frame Concerns
Frames Not Part of the Lateral Force Resisting System
   Short Captive Columns

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer Precast Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table 10-18	*RM1: Reinforced Masonry Bearing Wall Buildings with Flexible Diaphragms*

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities

Masonry Shear Walls
   Reinforcing in Masonry Walls
   Shear Stress
   Reinforcing at Openings

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength
   Sheathing
   Unblocked Diaphragms
   Span/Depth Ratio

Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-19 RM2: Reinforced Masonry Bearing Wall Buildings with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities

Masonry Shear Walls
  Reinforcing in Masonry Walls
  Shear Stress
  Reinforcing at Openings

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-20 URM: Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings

Masonry Shear Walls
  Unreinforced Masonry Shear Walls
  Properties, Solid Walls

Re-entrant Corners
Crossties
Diaphragm Openings
Diaphragm Stiffness/Strength
  Sheathing
  Unblocked Diaphragms
  Span/Depth Ratio

Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-21 URMA: Unreinforced Masonry Bearing Walls Buildings with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings

Masonry Shear Walls
  Unreinforced Masonry Shear Walls
  Properties, Solid Walls

Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
3.1	**Load Path**	10.3.1.1	Load Path
3.2	**Redundancy**	10.3.1.2	Redundancy
3.3	**Configuration**		
3.3.1	Weak Story	10.3.1.3	Vertical Irregularities
3.3.2	Soft Story	10.3.1.3	Vertical Irregularities
3.3.3	Geometry	10.3.1.3	Vertical Irregularities
3.3.4	Mass	10.3.1.3	Vertical Irregularities
3.3.5	Vertical Discontinuities	10.3.1.3	Vertical Irregularities
3.3.6	Torsion	10.3.1.4	Plan Irregularities
3.4	**Adjacent Buildings**	10.3.1.5	Adjacent Buildings
3.5	**Evaluation of Materials and Conditions**		
3.5.1	Deterioration of Wood	10.3.8.2	Condition of Wood
3.5.2	Overdriven Nails	10.3.8.3	Overdriven Fasteners
3.5.3	Deterioration of Steel	10.3.8.4	Condition of Steel
3.5.4	Deterioration of Concrete	10.3.8.5	Condition of Concrete
3.5.5	Post-Tensioning Anchors	10.3.8.6	Post-Tensioning Anchors
3.5.6	Concrete Wall Cracks	10.3.8.5	Condition of Concrete
3.5.7	Cracks in Boundary Columns	10.3.8.5	Condition of Concrete
3.5.8	Precast Concrete Walls	10.3.8.5	Condition of Concrete
3.5.9	Masonry Joints	10.3.8.7	Quality of Masonry
3.5.10	Masonry Units	10.3.8.7	Quality of Masonry
3.5.11	Cracks in Infill Walls	10.3.8.7	Quality of Masonry
4.1	**Frames with Infill Walls**		
4.1.4	Interfering Walls	10.3.3.3F	Infill Walls
4.2	**Steel Moment Frames**		
4.2.1	Drift Check	10.3.2.1A	Drift
4.2.2	Compact Members	10.3.2.1B	Frames
4.2.3	Beam Penetration	10.3.2.1B	Frames
4.2.4	Moment Connections	10.3.2.1D	Connections
4.2.5	Column Splices	10.3.2.1B	Frames
4.2.6	Joint Webs	10.3.2.1D	Connections
4.2.7	Girder Flange Continuity Plates	10.3.2.1D	Connections
4.2.8	Strong Column-Weak Beam	10.3.2.1C	Strong Column-Weak Beam
4.2.9	Out-of-Plane Bracing	10.3.2.1B	Frames
4.3	**Concrete Moment Frames**		
4.3.1	Shearing Stress Check	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.2	Drift Check	10.3.2.2A	Frame and Nonductile Detail Concerns

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
4.3.3	Prestressed Frame Elements	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.4	Joint Eccentricity	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.5	No Shear Failures	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.6	Strong Column-Weak Beam	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.7	Stirrup and Tie Hooks	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.8	Column-Tie Spacing	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.9	Column-Bar Splices	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.10	Beam Bars	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.11	Beam-Bar Splices	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.12	Stirrup Spacing	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.13	Beam Truss Bars	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.14	Joint Reinforcing	10.3.2.2A	Frame and Nonductile Detail Concerns
4.3.15	Flat Slab Frames	10.3.2.2A	Frame and Nonductile Detail Concerns
4.4	**Precast Moment Frames**		
4.4.1	Precast Frames	10.3.2.2B	Precast Moment Frames
4.4.2	Precast Connections	10.3.6.5	Precast Connections
4.5	**Frames Not Part of the Lateral-Force-Resisting System**		
4.5.1	Complete Frames	10.3.2.3A	Complete Frames
5.1	**Concrete Shear Walls**		
5.1.1	Shearing Stress Check	10.3.3.1A	Shearing Stress
5.1.2	Overturning	10.3.3.1B	Overturning
5.1.3	Coupling Beams	10.3.3.1C	Coupling Beams
5.1.4	Column Splices	10.3.3.1D	Boundary Component Detailing
5.1.5	Wall Connection	10.3.3.1D	Boundary Component Detailing
5.1.6	Confinement Reinforcing	10.3.3.1D	Boundary Component Detailing
5.1.7	Reinforcing Steel	10.3.3.1E	Wall Reinforcement
5.1.8	Reinforcing at Openings	10.3.3.1E	Wall Reinforcement
5.2	**Precast Concrete Shear Walls**		
5.2.1	Panel-to-Panel Connections	10.3.3.2A	Panel-to-Panel Connections
5.2.2	Wall Openings	10.3.3.2B	Wall Openings
5.2.3	Collectors	10.3.3.2C	Collectors
5.3	**Reinforced Masonry Shear Walls**		
5.3.1	Shearing Stress Check	10.3.3.3B	Shearing Stress
5.3.2	Reinforcing	10.3.3.3A	Reinforcing in Masonry Walls
5.3.3	Reinforcing at Openings	10.3.3.3C	Reinforcing at Openings

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
5.4	**Unreinforced Masonry Shear Walls**		
5.4.1	Shearing Stress Check	10.3.3.3D	Unreinforced Masonry Shear Walls
5.4.2	Masonry Lay-up	10.3.3.3D	Unreinforced Masonry Shear Walls
5.5	**Unreinforced Masonry Infill Walls in Frames**		
5.5.1	Proportions	10.3.3.3E	Proportions of Solid Walls
5.5.2	Solid Walls	10.3.3.3E	Proportions of Solid Walls
5.5.3	Cavity Walls	10.3.3.3F	Infill Walls
5.5.4	Wall Connections	10.3.3.3F	Infill Walls
5.6	**Walls in Wood Frame Buildings**		
5.6.1	Shearing Stress Check	10.3.3.4A	Shear Stress
5.6.2	Openings	10.3.3.4B	Openings
5.6.3	Wall Requirements	10.3.3.4C	Wall Detailing
5.6.4	Cripple Walls	10.3.3.4D	Cripple Walls
6.1	**Concentrically Braced Frames**		
6.1.1	Stress Check	10.3.4.1	System Concerns
6.1.2	Stiffness of Diagonals	10.3.4.2	Stiffness of Diagonals
6.1.3	Tension-Only Braces	10.3.4.2	Stiffness of Diagonals
6.1.4	Chevron Bracing	10.3.4.3	Chevron or K-Bracing
6.1.5	Concentric Joints	10.3.4.4	Braced Frame Connections
6.1.6	Connection Strength	10.3.4.4	Braced Frame Connections
6.1.7	Column Splices	10.3.4.4	Braced Frame Connections
7.1	**Diaphragms**		
7.1.1	Plan Irregularities	10.3.5.1	Re-entrant Corners
7.1.2	Cross Ties	10.3.5.2	Crossties
7.1.3	Reinforcing at Openings	10.3.5.3	Diaphragm Openings
7.1.4	Openings at Shear Walls	10.3.5.3	Diaphragm Openings
7.1.5	Openings at Braced Frames	10.3.5.3	Diaphragm Openings
7.1.6	Openings at Exterior Masonry Shear Walls	10.3.5.3	Diaphragm Openings
7.2	**Wood Diaphragms**		
7.2.1	Sheathing	10.3.5.4A	Board Sheathing
7.2.2	Spans	10.3.5.4C	Spans
7.2.3	Unblocked Diaphragms	10.3.5.4B	Unblocked Diaphragms
7.2.4	Span/Depth Ratio	10.3.5.4D	Span-to-Depth Ratio
7.2.5	Diaphragm Continuity	10.3.5.4E	Diaphragm Continuity
7.2.6	Chord Continuity	10.3.5.4F	Chord Continuity
8.2	**Anchorage for Normal Forces**		
8.2.1	Wood Ledgers	10.3.6.3	Anchorage for Normal Forces

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
8.2.2	Wall Anchorage	10.3.6.3	Anchorage for Normal Forces
8.2.3	Masonry Wall Anchors	10.3.6.3	Anchorage for Normal Forces
8.2.4	Anchor Spacing	10.3.6.3	Anchorage for Normal Forces
8.2.5	Tilt-up Walls	10.3.6.3	Anchorage for Normal Forces
8.2.6	Panel-Roof Connection	10.3.6.3	Anchorage for Normal Forces
8.3	**Shear Transfer**		
8.3.1	Transfer to Shear Walls	10.3.6.1	Diaphragm/Wall Shear Transfer
8.3.2	Transfer to Steel Frames	10.3.6.2	Diaphragm/Frame Shear Transfer
8.3.3	Topping Slab to Walls and Frames	10.3.6.1	Diaphragm/Wall Shear Transfer
		10.3.6.2	Diaphragm/Frame Shear Transfer
8.4	**Vertical Components to Foundations**		
8.4.1	Steel Columns	10.3.7.1	Anchorage to Foundations
8.4.2	Concrete Columns	10.3.7.1	Anchorage to Foundations
8.4.3	Wood Posts	10.3.7.1	Anchorage to Foundations
8.4.4	Wall Reinforcing	10.3.7.1	Anchorage to Foundations
8.4.5	Shear-Wall-Boundary Columns	10.3.7.1	Anchorage to Foundations
8.4.6	Wall Panels	10.3.7.1	Anchorage to Foundations
8.4.7	Wood Sills	10.3.7.1	Anchorage to Foundations
8.5	**Interconnection of Elements**		
8.5.1	Girders	10.3.6.4	Girder-Wall Connections
8.5.2	Corbel Bearing	10.3.6.4	Girder-Wall Connections
8.5.3	Corbel Connections	10.3.6.4	Girder-Wall Connections
8.6	**Roof Decking**		
8.6.1	Light-Gage Metal Roof Panels	10.3.6.7	Light Gage Metal, Plastic, or Cementitious Roof Panels
8.6.2	Wall Panels	10.3.6.6	Wall Panels and Cladding
9.1	**Condition of Foundations**		
9.1.1	Foundation Performance	10.3.7.2	Condition of Foundations
9.1.2	Deterioration	10.3.7.2	Condition of Foundations
9.2	**Capacity of Foundations**		
9.2.1	Overturning	10.3.7.3	Overturning
9.2.2	Ties Between Foundation Elements	10.3.7.4	Lateral Loads
9.2.3	Lateral Force on Deep Foundations	10.3.7.4	Lateral Loads
9.2.4	Pole Buildings	10.3.7.4	Lateral Loads
9.2.5	Sloping Sites	10.3.7.4	Lateral Loads

Table 10-22 Cross Reference Between the Guidelines and FEMA 178 (BSSC, 1992a) Deficiency Reference Numbers (continued)

FEMA 178		*Guidelines*	
Section	**Section Heading**	**Section**	**Section Heading**
9.3	**Geologic Site Hazards**		
9.3.1	Liquefaction	10.3.7.5	Geologic Site Hazards
9.3.2	Slope Failure	10.3.7.5	Geologic Site Hazards
9.3.3	Surface Fault Rupture	10.3.7.5	Geologic Site Hazards