C10. Simplified Rehabilitation

C10.1 Scope

FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC, 1992a), following the lead of *ATC-14* (ATC, 1987) and *ATC-22* (ATC, 1989), catalogued expected seismic performance by defining Model Building Types in terms of generalized structural systems, loads, load paths, and potential weaknesses. The result of an application of the FEMA 178 evaluation method is the determination that a building either meets its safety criteria, or rehabilitation is needed to correct specific deficiencies. The potential weaknesses used in FEMA 178 were identified using the past behavior of building types, and presented as evaluation statements to be answered "true" or "false," with appropriate procedures suggested for detailed evaluation when necessary. For a particular building, each statement that has a false answer flags a potential area for concern and subsequent analysis. In this manner, the evaluating engineer is led through a consideration and evaluation of the entire structural system to the point of determining whether a building meets the FEMA 178 (BSSC, 1992a) life safety standard, which includes structural and nonstructural criteria based on a specified probability of ground motion. The Simplified Rehabilitation Method is based on the deficiencies defined in FEMA 178 (BSSC, 1992a) and the simple concept that elimination of each deficiency is sufficient for rehabilitation.

The FEMA 178 process and the model buildings presented therein are the basis for the Model Building Types used in this chapter. FEMA 178 (BSSC, 1992a) defined 15 Model Building Types, described in Table 10-2 of the *Guidelines*, that were developed to represent all typical styles of construction throughout the United States. This categorization of buildings has been used throughout the FEMA guideline series and is used here for consistency.

Since these models were first introduced in 1987, however, it has become evident that there were more styles of construction for several classes of buildings. The differences were generally found in the type of diaphragm system, either flexible—in the case of wood or untopped metal deck—or stiff—as in concrete or metal deck with concrete fill. It was decided that, where applicable, each FEMA 178 building type would be separated with respect to its diaphragm system. In addition, the poor behavior of multistory multi-unit wood frame buildings with open fronts in both the 1989 Loma Prieta and 1994 Northridge earthquakes led to the definition of the new W1A Model Building Type. A more complete description of the Model Buildings is given in the companion *Example Applications* volume (ATC, 1997).

Significant damage to certain classes of structures occurred in the 1994 Northridge earthquake. Some of the deficiencies that led to the severe damage and, in some cases, collapse of these structures are not completely identified in the FEMA 178 (BSSC, 1992a) list of potential deficiencies. These have been defined and added to the scope of deficiencies addressed by the Simplified Rehabilitation Method, and are suggested as an amendment to FEMA 178. FEMA 178 (BSSC, 1992a) is currently under revision to include both updated information and the Damage Control Structural Performance Range, as well as Life Safety.

The potential for near-field effects—intense shaking and large, damaging velocity pulses in the earthquake source region—has been a topic of discussion for many decades, but only recently were instances specifically observed and recorded. Recent earthquakes in both California and Japan have provided strong motion records that indicate the need to consider stronger ground motions in the near-field area of large earthquakes. It was also observed in these earthquakes that only mid-rise buildings located very close to the source of the earthquake were affected. As a result, the current trend in seismic design guidelines is to include a near-field factor to essentially increase the design lateral force for mid-rise buildings—those with periods greater that 1.0 second—located within ten kilometers of large active faults. Because the Simplified Rehabilitation Method cannot be used on the classes of buildings affected by these near-fault provisions, they need not be considered. They have been properly considered and included in the appropriate sections of the *Guidelines* as they relate to the Systematic Rehabilitation Method.

The lateral force provisions and analysis procedures used in FEMA 178 (BSSC, 1992a) are based on the 1988 *NEHRP Recommended Provisions* for new buildings (BSSC, 1988). As such, they represent the traditional equivalent lateral force procedure that has been used for decades in most seismic codes and guidelines. This procedure is based on the assumption that buildings designed to resist highly reduced (hence called "equivalent") lateral forces within their elastic limits, and properly detailed for ductility, will behave in an appropriate, life-safe manner when subjected to actual earthquake motions. Based on the historic performance of buildings and the levels of ground motion recorded for various earthquakes, current codes reduce the actual forces by *R* factors and include maximum values for purposes of design. These *R* factors (structural response modification factors) and maximum values are based on the judgment and experience of the code and guideline writers (ATC, 1995). When estimates are needed of the forces or deflections caused by the actual earthquake motion, the procedures use a C_d factor to adjust the values up to an appropriate level.

Because of the unique conditions present in existing buildings, the Systematic Rehabilitation Method of the *Guidelines* takes an entirely different approach to the determination of lateral forces and resulting expected deflections. Since most existing buildings needing rehabilitation do not contain the details of construction needed to validate the large reduction factors, a procedure has been developed that allows the individual evaluation of the various building components' capacity to resist the inelastic deformation and strength demands that are expected. In essence, the *Guidelines* define earthquake motions in terms of the expected maximum displacement based on an acceleration response spectrum, and define the pseudo lateral loads needed to cause those expected displacements for use in the evaluation process. Thus, in the Linear Static Procedure of the *Guidelines*, pseudo lateral loads are much larger than those specified traditionally, since they do not include any reduction factors. The appropriate reduction is considered on a component-by-component basis in terms of the allowable capacities, and the *m* value. The reader is referred to Chapter 3 of both the *Guidelines* and the *Commentary* for a complete discussion of the new procedure.

The example given in Chapter 10 of the *Guidelines* (see Figure 10-1) illustrates this point in terms of two hypothetical reinforced concrete structures with perimeter shear walls. The buildings are 120 feet square, and have nine- inch-thick concrete flat slabs at the floors and roof, and eight-inch-thick exterior walls with approximately 30% openings for windows and doors. They are located on S2 soil (FEMA 178 method) or Class C soil (*Guidelines* method). The six-story building is located in an area of low seismicity and the three-story building is located in a High Seismic Zone. The point of proper comparison in the two methods is the ratio of demand/capacity. While the base shears vary by approximately six times, and the allowable capacities by three times, the values of demand/capacity are similar.

As a matter of comparison, the FEMA 178 (BSSC, 1992a) deflections and shears are also plotted with and without their reduction factors. It could be expected that there would be a rough correlation between the unreduced FEMA 178 values and the *Guidelines* values. However, there is a significant difference in the results, primarily due to the basic definition of the pseudo lateral load, the method used to calculate the building period, and the global reduction (0.85 and 0.67) taken in FEMA 178 spectral ordinates to account for the difference in a mean value response spectrum and a mean plus one standard deviation spectrum. This reduction is not taken in the *Guidelines* procedure.

Traditionally, the spectra used to develop the equivalent lateral forces used in codes and guidelines for new buildings are based on a probable earthquake, defined as one with a probability of exceedance equal to 10% in 50 years (10%/50 year), and a related response spectra that represents the mean plus one standard deviation values. In 1987, the Applied Technology Council's ATC-14 report, *Evaluating the Seismic Resistance of Existing Buildings* (ATC, 1987), which served as the basis for FEMA 178, recommended that the spectra for evaluating existing buildings be modified to represent mean values. They argued that, when evaluating existing buildings where there is a high degree of uncertainty and the cost of strengthening is very high, it is more appropriate to use the values associated with the mean probable earthquake than the probable earthquake. This remains a controversial recommendation.

Integrating FEMA 178 evaluation criteria into a rehabilitation guideline has the advantage of separating building elements and systems into individual units, which can be identified relatively quickly and mitigated somewhat independently. This technique works well for simple, low-rise buildings of uniform construction that match the model buildings. For buildings that exhibit complex interaction between elements, such that mitigating one deficiency may only change the weak link in the overall system—or even make another element worse—a more systematic analysis is necessary, and the Simplified Rehabilitation Method is not appropriate.

Certain building systems are excluded from the Simplified Rehabilitation Methods because of their complexity and the possibly unique behavior among individual buildings. Excluded from Simplified Rehabilitation are tall and irregular structures whose behavior is difficult to predict within the provisions of FEMA 178. Buildings that are of hybrid construction (not one of the common building types), including structures with different structural systems in each direction, are also excluded. In addition, the behavior of concrete frame structures, especially in older buildings and in parking garages, has been shown in recent earthquakes to be highly variable, so these buildings are also excluded except in regions of low seismicity. Buildings with significant plan or vertical irregularities have very different characteristics than those expected in regular buildings. Typical analysis methods may not be appropriate in these cases.

The special procedures for evaluating unreinforced masonry buildings presented in Appendix C of FEMA 178 (BSSC, 1992a) allow their use for buildings up to six stories in height. This is consistent with other guidelines, such as the *Uniform Code for Building Conservation* (*UCBC*) (ICBO, 1994a). This may be somewhat nonconservative in higher seismic zones. The *UCBC* is not regarded widely as a document whose goal is life safety, but rather as a hazard reduction guideline. For this reason, the limitations on height for unreinforced masonry (URM) buildings has been reduced in all regions. Height limitations for other building types were developed from comparisons to the values for URMs and to the typical limitations provided by actual construction practice. These limitations apply only when the rehabilitation goal is to achieve the Life Safety Performance Level.

While an engineer may choose to mitigate all of a building’s identified FEMA 178 deficiencies by using Simplified Rehabilitation, such a technique should not be considered sufficient to achieve the Basic Safety Objective (BSO) or any Enhanced Safety Objective as defined in Chapter 2. Since the method is based on FEMA 178, which evaluates a building only for compliance with life safety criteria based on a level of earthquake shaking estimated to have a 10% probability of being reached or exceeded in a 50-year period of time (10%/50 year), there is no assurance that it will satisfy the Collapse Prevention criteria as described in this document for the BSO, especially in zones of low seismicity.

The BSO defined in Chapter 2 requires meeting both the Life Safety Performance Level for the BSE-1 (typically, the 10%/50 year) level of motion, and the Collapse Prevention Performance Level for the BSE-2 (typically, the 2%/50 year) level of motion. In regions of low to moderate seismicity, the BSE-2 event may be substantially larger than the BSE-1 earthquake. The attainment of the BSO requires the use of the Systematic Rehabilitation Method, described in Chapter 3, to verify performance for the BSE-2. It is highly recommended that consideration be given to the performance of the rehabilitated building under the BSE-2. Such a consideration need not include a complex, nonlinear analysis, nor should it be based on simple increase in the lateral forces used for design. Rather, it requires that the design professional consider the post-elastic behavior of the building, determine its yielding mechanisms and maximum expected displacements, and determine whether the structure will be subject to collapse when the building is subjected to the BSE-2.

The use of the Systematic Rehabilitation Method is also encouraged if the added cost of a more complex analysis can be offset by a substantial reduction in the cost of the mitigation required.

C10.2 Procedural Steps

The FEMA 178 (BSSC, 1992a) evaluation is intended to stand apart from the Systematic Rehabilitation Method described in the *Guidelines*. Existing elements, systems, and mitigation schemes do not have to be checked using the force levels, *m* factors, analysis techniques, and the like, contained in the Systematic Rehabilitation Method.

FEMA 178 lists specific deficiencies both by Model Building Type and as associated with each building system. *Guidelines* Tables 10-3 through 10-21 further group them by general characteristics. For example, the deficiency listing: “Diaphragm Stiffness/Strength,” includes deficiencies related to the type of sheathing used, the diaphragm span, and lack of blocking. Each deficiency group is named and defined in this *Commentary* Section C10.5 and related to all of the FEMA 178 (BSSC, 1992a) deficiencies as amended. *Guidelines* Table 10-22 provides a complete cross-reference.

In addition, within the table for each Model Building Type, each deficiency group is ranked from most critical at the top to least critical at the bottom. For example, in Table 10-14, in a precast/tilt-up concrete shear wall with flexible diaphragm (PC1) building, the lack of positive gravity frame connections (e.g., of girders to posts by sheet metal hardware or bolts) has a greater potential to lower the building's performance (a partial collapse of the roof structure supported by the beam), than a deficiency in lateral forces on foundations (e.g., poor reinforcing in the footings).

The ranking was based on the following characteristics of each deficiency group:

1. Most critical
    a. Building systems: those with a discontinuous load path and little redundancy
    b. Building elements: those with low strength and low ductility
2. Intermediate
    a. Building systems: those with a discontinuous load path but with substantial redundancy
    b. Building elements: those with substantial strength but low ductility
3. Least critical
    a. Building systems: those with a substantial load path but little redundancy
    b. Building elements: those with low strength but substantial ductility

The intention of Tables 10-3 to 10-21 is to guide the design professional so that partial rehabilitation efforts will be useful. For example, if the foundation is strengthened in a PC1 building but a poor girder/wall connection is left alone, relatively little has been done to improve the expected performance of the building. Considerable professional judgment must be used when evaluating a structure's unique behavior and determining which deficiencies should be strengthened and in what order.

Use of the Systematic Rehabilitation Method is encouraged where the FEMA 178 procedures may be unduly conservative. A thorough, wide-ranging solution is often very cost-effective, making up for the extra time spent in the design process.

C10.3 Suggested Corrective Measures for Deficiencies

The application of the Simplified Rehabilitation Method is essentially the performance of a complete FEMA 178 (BSSC, 1992a) evaluation of a building, correcting any deficiencies that are identified. Although FEMA 178 contains "checklists" of potential deficiencies related to a Model Building Type, it is not intended to be used selectively, but rather applied to a building's entire lateral-force-resisting system. It outlines and describes the procedures to follow to perform a thorough analysis and identification of deficiencies.

This section is organized around the major lateral-force-resisting systems common to the Model Building Types, including the overall building configuration, the different vertical lateral-force-resisting systems, diaphragms, connections, and geological considerations. A section is devoted specifically to the evaluation of URM buildings, corresponding to Appendix C of FEMA 178.

Each of the subsections in this section groups the deficiencies identified in FEMA 178 (BSSC, 1992a) into general categories where appropriate, and provides references to specific FEMA 178 sections relating to each deficiency in the group. An expanded discussion of each group is included, with suggestions for additional evaluation techniques beyond those described in FEMA 178, including those found in Systematic Rehabilitation. Suggestions and references for typical rehabilitation strategies are also provided. Table 10-22 cross-references the FEMA 178 (BSSC, 1992a) and *Guidelines* numbers. Section C10.5 of this *Commentary* provides a complete list of the FEMA 178 deficiency evaluation statements, as well as the eight new potential deficiencies presented in the *Guidelines*, Section 10.4.

C10.3.1 Building Systems

C10.3.1.1 Load Path

A complete load path for the transmission of forces from the point where they are generated to the foundation and supporting soil material is essential for the proper seismic behavior of a structure. If there is a discontinuity in the load path, the building is unable to resist earthquake-induced forces, regardless of the strength of existing elements. (FEMA 178 [BSSC, 1992a], Section 3.1.)

The first step in finding missing links in a load path is to identify the location of loads generated throughout the building. These loads generate forces and moments. The loads are traced through the structure, usually beginning with the diaphragms, proceeding to the vertical lateral-force-resisting systems (walls or frames) through connections, and into the foundation through connections. Certain loads are local, such as bending moments generated in a diaphragm, and are not transferred to the foundation. In cases where there is a structural discontinuity, a load path may exist but it may be a very undesirable one, such as with offset shear walls, which transfer overturning moments through beam or frame elements not intended to be part of the lateral-force-resisting system. Identification of undesirable load paths in a complex structure can be facilitated with the development of appropriate computer modeling.

If the existing load path is complete but potentially undesirable, it may be possible to show that, while not ideal, the existing load path is acceptable. It may also be possible, using the Systematic Rehabilitation Method described in Chapters 2 and 3, to show that alternate load paths can be developed if the primary path is discontinuous or insufficient.

C10.3.1.2 Redundancy

To account for uncertainties in both the expected loads and the analysis methods—and in the inability to know precisely the existing condition of all structural elements—it is essential that buildings contain redundancy in their lateral-force-resisting systems. Redundancy ensures that if a single element—such as a brace, moment connection, or shear wall pier (or entire wall line if it is small)—fails for any reason, the structure has alternative paths by which lateral forces can be resisted. (FEMA 178 [BSSC, 1992a], Section 3.2.)

It is not sufficient to show by analysis that under the design forces (or even a multiple of the design forces) no structural elements yield, because the unknowns associated with the building and the ground motion are potentially large and the consequences of failure significant. Analysis for redundancy should show that if major elements are seriously damaged, a complete load path remains. In this analysis, the engineer does not have to show that the remaining elements are sufficient to resist the design lateral loads. The Nonlinear Static Procedure (Chapter 3) can be used to investigate whether the failure of a single element causes an instability.

C10.3.1.3 Vertical Irregularities

Vertical irregularities in a building may result in a concentration of forces or deflections or in an undesirable load path in the vertical lateral-force-resisting system. In extreme cases, this can result in serious damage to or collapse of a building, since the lateral system is often integral with the gravity-load-resisting system. Vertical irregularities typically occur in a story that is significantly more flexible or weaker than adjacent stories. The irregularity can also occur where there is a significant change in building dimension over its height, such as with setbacks, where there are large concentrations of mass, or where vertical elements are discontinuous in a story.

The use of simplified procedures for determining the significance of vertical irregularities is difficult, especially in tall or complex buildings. The deficiency may be difficult to spot in a visual survey or with simple calculations. The Quick Check procedures in FEMA 178 for calculating story capacities, forces, and drifts can be used to determine the presence of a vertical irregularity, but should be verified through a complete analysis.

While it is possible in some cases to allow the irregularity to remain and to strengthen those structural elements that are insufficient, this may require substantial additional analysis, and does not address the problem directly nor in a manner that is permitted by the Simplified Rehabilitation Method. Because the presence of a vertical irregularity in a single story can affect the force and deflection characteristics of the entire building, dynamic or nonlinear analysis techniques are usually required to evaluate the consequences.

By using one of the procedures in the Systematic Rehabilitation Method, the presence of a vertical irregularity often can be determined to be inconsequential. (FEMA 178 [BSSC, 1992a], Sections 3.3.1 through 3.3.5.)

C10.3.1.4 Plan Irregularities

Horizontal irregularities in the structural system of a building typically result in torsion caused by a differential between the center of mass and the center of rigidity in a story, and may result in undesirable dynamic behavior, including building rotation or excessive deflection at the more flexible building ends. Such plan irregularities, hereafter called “torsional irregularities,” can lead to excessive and concentrated demands on the diaphragms that are often not of adequate strength and are not otherwise identified in the Simplified Rehabilitation Method.

It is often possible to determine the presence of torsional irregularities using simplified procedures such as a relative rigidity analysis. As torsion is the primary horizontal irregularity, the deficiency may not be difficult to spot in a visual survey or with simple calculations. Where adjacent stories affect the stiffness properties of the story in question, or where the irregularity is caused by re-entrant corners, systematic analysis may be warranted.

Using a nonlinear procedure in Systematic Rehabilitation, the presence of a torsional irregularity often can be identified and possibly determined to be insignificant. If the irregularity cannot be eliminated, it may be possible, using these methods, to identify the elements that need to be strengthened as a result of the irregularity.

Other plan irregularities related to the plan configuration of the building require consideration of the interconnection of the building at the re-entrant corners, the strength of diaphragms, and the overall lateral system for each wing. Each of these is addressed later by other potential deficiencies. (FEMA 178 [BSSC, 1992a], Section 3.3.6.)

C10.3.1.5 Adjacent Buildings

Adjacent structures can pound in an earthquake if they are too close and they exhibit different dynamic deflection characteristics. The structures may be part of a single complex of buildings or two buildings separated by a property line. Pounding damage can be especially severe if the floors of adjacent buildings do not line up or one building is significantly taller than the other. In these instances, the floor of one building, which is typically very stiff, pounds into the wall of the other, which is usually very flexible out-of-plane. In severe cases, pounding has led to collapse or partial collapse of one of the two buildings.

The Quick Checks for drift in FEMA 178 (BSSC, 1992a) are used to identify the possibility of pounding, since the actual drifts in a building are much higher than those computed directly from the forces used for designs. Design forces based on reduced accelerations from the elastic earthquake spectrum anticipate some yielding in the elements and therefore will lead to larger expected drifts. Expected drift can be more accurately calculated when based on the actual expected earthquake accelerations using advanced techniques. Chapter 3 provides Analysis Procedures for obtaining more realistic estimates of drift.

The Nonlinear Dynamic Procedure described for use with Systematic Rehabilitation may be used in complex or tall buildings to make a more accurate determination of story drift capacity versus demand. (FEMA 178 [BSSC, 1992a], Section 3.4.)

C10.3.1.6 Lateral Load Path at Pile Caps

This is an amendment to the FEMA 178 (BSSC, 1992a) deficiency lists. Refer to Section 10.4.1.1 of the *Guidelines* for the evaluation statement, comment, and procedure.

C10.3.1.7 Deflection Compatibility

This is an amendment to the FEMA 178 (BSSC, 1992a) deficiency lists. Refer to Section 10.4.1.2 of the *Guidelines* for the evaluation statement, comment, and procedure.

C10.3.2 Moment Frames

C10.3.2.1 Steel Moment Frames

A. Drift

Moment-resisting frames are generally more flexible than shear wall or braced frame structures, and are likely to sustain larger lateral building displacements (total and inter-story drifts). Large inter-story drifts in structures can generally be expected to cause more extensive nonstructural damage to elements such as partitions and cladding; potentially significant P-Δ effects in taller structures; damage to welded beam-column connections; and pounding where there are closely adjacent buildings.

The Quick Check for drift in FEMA 178 (BSSC, 1992a) can be used for short, simple buildings to identify structures that may be susceptible to excessive inter-story drifts. The drifts calculated from the Quick Check will be much smaller than the actual drifts caused by the earthquake, since the calculations are based on the basic equivalent lateral force procedure rather than the expected accelerations and displacements of the earthquake ground motion. The allowable drift values in FEMA 178 (BSSC, 1992a) are intended to take this into account. All buildings failing the Quick Check should be fully analyzed, using the Systematic Rehabilitation Method.

The Systematic Rehabilitation Method should be used in tall and/or irregular buildings to make a more accurate determination of inter-story drifts. (FEMA 178 [BSSC, 1992a], Section 4.2.1.)

B. Frames

Proper performance of steel moment-resisting frames depends on the ability of all of the various elements of the lateral-force-resisting system to develop required member strengths and meet local ductility demands. Without this ability, the frames will be subject to unacceptable damage. As such, the frame elements need to be rehabilitated in a way that will meet both strength and deformation demands.

Structural steel sections are proportioned to maximize their efficiency. This makes them more susceptible to stability concerns (both local and global) than other structural materials. Use of compact sections, which have proper width-to-thickness ratios for the various portions of the cross sections, will delay the onset of local buckling and permit proper inelastic response. Global stability concerns need to be met by providing adequate lateral bracing at locations of expected plastic hinging and other code-specified intervals. Large local member discontinuities, such as web penetrations in beams, may also reduce member strength and deformation capacities.

Evaluation of the impact of noncompact members and members with large web penetrations can be made using procedures provided by the AISC specifications (AISC, 1986, 1989). Stability analyses may be required to determine the effects of lateral bracing that is less than the typically specified requirements. Since inelastic deformation capacities are not explicitly addressed in the calculation procedures recommended by FEMA 178 (BSSC, 1992a), the calculation of member force demands should be done using elevated lateral force levels (by using reduced *R* factors—structural response modification factors) to account partially for the reduced deformation capacities. Evaluation using Systematic Rehabilitation will therefore likely be required, in order to estimate the effects of these considerations on the member deformations that can actually be accommodated unless the deficient members are rehabilitated.

Systematic Rehabilitation should be used in buildings with significant stability concerns to obtain realistic estimates of the member demands. Detailed evaluation of the element deformation capacities and stability will also be required. (FEMA 178 [BSSC, 1992a], Sections 4.2.2, 4.2.3 and 4.2.9.)

C. Strong Column-Weak Beam

One goal for well-configured moment frame systems is to distribute inelastic action throughout the lateral-force-resisting elements, which requires the capacity of the column at any moment frame joint be greater than the capacity of the beams. In conditions where the beams are stronger than the columns, column hinging can lead to story mechanisms, which can result in an excessively large drift within a single story. The large inelastic rotation demands that result could jeopardize the stability of the frame, due to P-Δ effects. Column hinging is also considered undesirable, since large gravity loads may be supported by a column. A beam, on the other hand, supports a significantly smaller portion of the gravity loads on the structure. Local hinging in the beams will therefore affect a much smaller portion of the building. (FEMA 178 [BSSC, 1992a], Section 4.2.8.)

FEMA 178 prescribes that local joint analyses be performed to evaluate these effects. The effects of gravity forces on the member capacities must be considered. Axial force effects on the columns—due to both gravity forces and frame overturning effects—will reduce the residual capacity for resisting seismically-induced bending moments. The supplemental beam strength provided by the composite action between the concrete floor slabs and the steel beams has generally not been considered, but may be significant in some instances.

The Systematic Rehabilitation Method, including nonlinear procedures and dynamic procedures, should be used in tall and/or irregular buildings to determine whether the potential for the development of story mechanisms exists. Proper consideration of slab effects and column overturning effects is also necessary.

D. Connections

Prior to the 1994 Northridge earthquake, steel moment frame connections consisting of full penetration flange welds and a bolted shear tab were thought to be ductile and capable of developing the full capacity of the beam section. This connection detail, which became almost an industry standard in the period from 1970 to 1995, experienced serious damage in the form of weld and beam or column fractures in over 100 buildings as a result of the Northridge earthquake. Because of this, an emergency code change was made to the 1994 *UBC* (ICBO, 1994b), which removed the "prequalification" of this connection detail. The newly discovered susceptibility of this detail is the focus of a great deal of effort to understand the causes of the damage and to develop methods to design, evaluate, and rehabilitate these structures. Previous laboratory testing on partial penetration column splices has shown little or no ductility. No damage to column splices was noted in the Northridge earthquake, although the 1995 Hyogoken-Nanbu (Kobe), Japan earthquake did produce a number of such failures. Panel zone doubler plates and continuity plates were also damaged in the Northridge earthquake, although to date their design has not been seen as a significant factor.

Because of the Northridge earthquake damage, the use of FEMA 178 procedures related to welded steel moment frame connections needs to be completely revised. The Systematic Rehabilitation Method provided in these *Guidelines* should be followed. Testing of mock-up connection subassemblages may need to be considered for conditions where no previous test results adequately model the conditions being evaluated. The SAC Joint Venture (whose participants are the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering) Program to Reduce the Earthquake Hazards of Steel Moment Frame Structures, and other efforts, are specifically addressing this problem. The most recent publications now recommend that girder flange continuity plates be provided in all cases to reduce the stress concentrations that occur at the web location in the flange welds. See SAC (1995).

At the time of this writing, appropriate systematic solutions are under development by the SAC Steel Program. Such solutions will likely evolve from advanced methods of analysis—such as nonlinear time-history analysis, both on the frame elements and, possibly, on individual joint subassemblages—as well as from extensive additional testing. Because of the variability of construction quality encountered in the post-Northridge inspections, it is likely that the procedure will be explicitly probability-based. At the time of this writing, the latest available guidance is the *SAC Interim Guidelines* (SAC, 1995). (FEMA 178 [BSSC, 1992a], Sections 4.2.4 through 4.2.7.)

C10.3.2.2 Concrete Moment Frames

A. Frame and Nonductile Detail Concerns

Quick Checks. The Quick Checks of FEMA 178 provide generally conservative estimations of shear and drift in the frames, providing the engineer with a "ballpark" estimate of the situation. They are best applied to regular multistory buildings.

Where the initial Quick Check indicates average column shear stress above 60 psi, or if the building is not regular, FEMA 178 refers to the need for a more detailed evaluation. For structures satisfying the limits of Table 10-1, the more detailed evaluation may utilize FEMA 178 forces and procedures. (FEMA 178 [BSSC, 1992a], Sections 4.3.1, 4.3.2.)

Frames. These concerns focus on those elements whose local failure can lead directly to collapse or partial collapse of the building, i.e., precast frames, frames with eccentric joints, and shear-critical columns (shear failure occurs before flexural failure).

In general, prestressed frames should not be justified using Simplified Rehabilitation. It may be possible to show that eccentric joints and shear-critical columns are acceptable by demonstrating that the available shear capacity exceeds the anticipated demand by a significant margin—a factor of approximately 3.0. Reliance on Simplified Rehabilitation to address these concerns should be done with caution and should take into account the structural response as a whole. (FEMA 178 [BSSC, 1992a], Sections 4.3.3, 4.3.4, 4.3.5.)

Strong Column-Weak Beam. Where the sum of the moment capacities of the beams exceeds that of the columns, the failure is likely to occur in the column. This condition is even more critical when the column is shear-critical (see above), because the shear imposed on the column is governed not by the column's flexural capacity but by the capacity of the beams.

Nonductile Detail Concerns. Nonductile frames are elements that do not incorporate the following items addressed in current ductile detailing provisions:

- Anchorage of beam stirrups and column ties into the concrete core with 135-degree hooks
- Close spacing of column ties
- Length and confinement of column bar splices
- Continuity of top and bottom beam bars through the column-beam joint
- Length and location of beam bar splices; close spacing of beam stirrups
- No reliance on bent longitudinal bars for shear reinforcement
- Use of column ties in exterior column/beam joints
- No flat slab/plates working as a beam in frame action

Ductile detailing allows the elements to maintain vertical-load-carrying capacity as the frame displaces beyond the elastic limits of the system and forms plastic hinges.

Current ductile detailing practices have evolved only since the mid-1970s. In general, most frame buildings built before 1973 will likely have nonductile detailing. In some cases, columns were spiral reinforced, which usually provides significant ductility in the columns. However, column bar splices, beam reinforcement, and beam-column joints still need to be evaluated.

Where nonductile components remain essential links in the load path, Systematic Rehabilitation must be used. Careful consideration must be given to the brittle nature of the columns and joints. (FEMA 178 [BSSC, 1992a], Sections 4.3.7 through 4.3.15.)

B. Precast Moment Frames

Precast concrete frames without shear walls may not be addressed under Simplified Rehabilitation (see Table 10-1). Where shear walls are present, the precast connections often govern the performance and need to be carefully evaluated. If the connections are configured such that yielding occurs within the members rather than in the connections, the building should be evaluated as a shear wall system. (FEMA 178 [BSSC, 1992a], Section 4.4.1.)

C10.3.2.3 Frames Not Part of the Lateral-Force-Resisting System

A. Complete Frames

Typically, incomplete frames are essentially bearing wall systems. Damage to the wall may lead to a loss of gravity load resistance. The evaluation should utilize FEMA 178 (BSSC, 1992a) forces and procedures and should include a check of the connection between drag elements (i.e., horizontal reinforcement) and the bearing walls.

Strengthening the wall to reduce the stress under combined gravity and seismic loads may be more appropriate when there is nearly enough existing vertical-load-resisting strength. The addition of columns to complete the gravity load path is the preferred solution because it separates the lateral-force-resisting system and damage it may suffer from the vertical-load-resisting system. Where the wall cannot be strengthened nor columns added, the Systematic Rehabilitation Method should be used, since walls and adjacent columns will probably not have ductile detailing. (FEMA 178 [BSSC, 1992a], Section 4.5.1.)

B. Short Captive Columns

See the *Guidelines* Section 10.4.2.2 for explanation of this addition to the FEMA 178 (BSSC, 1992a) potential deficiency list.

C10.3.3 Shear Walls

C10.3.3.1 Cast-in-Place Concrete Shear Walls

A. Shearing Stress

The shearing stress check provides a quick assessment of the overall level of shearing stress in the building's walls.

Where the average stress exceeds the FEMA 178 (BSSC, 1992a) recommended values, a more detailed evaluation is needed. This detailed evaluation, utilizing FEMA 178 forces and procedures, should account for vertical and horizontal distribution of the seismic forces. Allowable stresses compatible with ACI provisions (ACI, 1989) should be used.

Where the shearing stress limit calculated with the more detailed evaluation is still exceeded, the appropriate

Simplified Rehabilitation solution is to add sufficient shear walls to satisfy the stress check or detailed evaluation criteria. These calculations tend to be very conservative and an appropriate Systematic Rehabilitation should be used if extensive rehabilitation appears to be needed.

Appropriate Systematic Rehabilitation solutions will also address the impact of boundary element configuration on the shear capacity of the walls. (FEMA 178 [BSSC, 1992a], Section 5.1.1.)

B. Overturning

Tall, slender shear walls may have limited overturning resistance. Displacements at the top of the building will be greater than those anticipated by simplifying equations and/or analytical models, if the overturning forces are not properly resisted. Often, sufficient resistance is available in the immediately adjacent columns.

If an extensive amount of work is needed, procedures of the Systematic Rehabilitation Method should be used that include developing analytical models that reflect the load/displacement curves for slender walls, and their interaction throughout the building. (FEMA 178 [BSSC, 1992a], Section 5.1.2.)

C. Coupling Beams

Coupling beams act to tie or couple adjacent walls acting in the same plane. When properly detailed and proportioned, coupling beams have a significant effect on the overall stiffness of the coupled walls and their resistance to overturning.

Appropriate evaluation techniques include first evaluating the walls acting without coupling. This evaluation includes shears, moments, and wall stability. If the walls are stable and satisfy Simplified Rehabilitation wall criteria, the approach would then focus on preventing debris from becoming a falling hazard. (FEMA 178 [BSSC, 1992a], Section 5.1.3.)

D. Boundary Component Detailing

Fully effective shear walls require the following boundary element components to be appropriately detailed: (1) steel column splices, (2) steel column/ concrete wall shear transfer mechanism, and (3) confinement ties at vertical reinforcement. Brittle failure of any one of these components can lead to substantially lower wall capacity.

In the Simplified Rehabilitation evaluation, column splices, shear transfer mechanisms, and confinement should be adequate to develop the amplified FEMA 178 forces.

In Systematic Rehabilitation, reduced capacity of the components can be accounted for. (FEMA 178 [BSSC, 1992a], Sections 5.1.4 - 5.1.6.)

E. Wall Reinforcement

The reinforcement in shear walls controls the ability of the wall to behave appropriately under seismic loads. Openings may significantly interrupt the flow of stresses so that special steel is required around the boundaries.

In the Simplified Rehabilitation evaluation, use forces and procedures outlined in FEMA 178 (BSSC, 1992a).

In Systematic Rehabilitation, the shear walls can be modeled to reflect the anticipated degradation of the wall and, in some cases, allow isolated walls without enough strength to remain without strengthening because there is available strength elsewhere, in other walls. (FEMA 178 [BSSC, 1992a], Sections 5.1.7, 5.1.8.)

C10.3.3.2 Precast Concrete Shear Walls

A. Panel-to-Panel Connections

Welded steel inserts can be brittle and may not be able to transfer the overturning forces between panels. Latent stresses may be present due to shrinkage and temperature effects.

The Simplified Rehabilitation evaluation should follow the procedures outlined in FEMA 178 (BSSC, 1992a). Particular care must be taken to ensure that there is substantial strength available in the as-built connections to resist the actual earthquake forces, since these connections typically have no ductility. It is preferable for the connections to be able to develop the full yield strength of the panel.

B. Wall Openings

In tilt-up construction, walls with large openings require special detailing for collector elements, shear transfer, and overturning. Often, the piers and spandrels were detailed only as walls and not as elements of a lateral-force-resisting concrete frame.

Panel connections should be assessed. If the panel connections are strong enough, the panels will behave like a moment frame, and each element should be evaluated for frame action. It is unlikely that panels with large openings can be shown to be adequate when considered as moment frames.

C. Collectors

Where collectors are needed to transfer lateral forces out of the diaphragm into the shear walls, the collector and its connections should be evaluated using FEMA 178 (BSSC, 1992a), Section 4.4.2, to determine whether they are adequate to develop the design forces. Full consideration should be given to existing continuous slab and beam reinforcing that may naturally serve the collector purpose.

C10.3.3.3 Masonry Shear Walls

A. Reinforcing in Masonry Walls

If there is any possible evidence of reinforcing in masonry walls, or if the standard construction techniques for the region include reinforcing masonry construction, then every effort should be made to identify and take full account of the level of reinforcing. This is especially true for concrete block construction.

Consideration of the building's adequacy as a URM building should precede the addition of new reinforced masonry or shotcrete walls.

B. Shearing Stress

A detailed analysis of the lateral-force-resisting walls should be performed, using the provisions of *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995a). The allowable stresses as specified in MSJC (1995) should be used, multiplied by 2.5 times a capacity reduction factor. (FEMA 178 [BSSC, 1992a], Section 5.3.1.)

In order to utilize MSJC (1995), the prism strength of the masonry and the yield strength of the steel must be established. If the mortar type is lime-sand-mortar or a lime-sand-portland cement mortar, and the approximate strength of the masonry unit can be established, then a reasonable lower bound value, using the tables in MSJC (1995), can be assumed for the prism strength. The yield strength of the reinforcing can be conservatively estimated as 30,000 psi.

C. Reinforcing at Openings

Masonry control joints are sometimes located at openings. The presence of a control joint, large shrinkage cracks, or a steel or precast concrete lintel would indicate that trim reinforcing was not installed.

D. Unreinforced Masonry Shear Walls

The evaluation of URM buildings is based upon the Simplified Rehabilitation Method and consists of using the provisions of FEMA 178 (BSSC, 1992a).

The evaluation is based upon a reduced base shear, building evaluation checklists, and a series of Quick Checks to determine if the strength of the building is satisfactory. In the event that the structure does not pass the Quick Check procedure, it is recommended that the engineer use the Systematic Rehabilitation Method outlined in the *Guidelines*.

An evaluation can also be made using Appendix C of FEMA 178 (BSSC, 1992a). However, the performance objective of Appendix C is significant hazard reduction, which is a lower objective than assumed for the Life Safety Performance Level. In order to comply with the quality control requirements of Appendix C, testing of masonry and anchors is required.

The composition of the wall must be determined in order to compute the shearing stresses in the wall and the thickness that is to be used to resist out-of-plane forces. The lay-up of the walls is deficient if significant voids are left between the wythes. In this case, the walls may not be able to resist out-of-plane forces as expected, due to a lack of composite action between the inner and outer wythes. Appendix C is based upon brick construction. Consequently, there is no procedure established to test concrete masonry units. Appropriate testing is needed. When the net area is required for shearing stress computations, a section of the wall should be removed in order to establish the bedding area. Walls with insufficient thickness should be either strengthened by increasing the thickness, or removed. (FEMA 178 [BSSC, 1992a], Sections 5.4.1, 5.4.2.)

E. Proportions of Solid Walls

The out-of-plane requirements for infill walls also apply to unreinforced masonry bearing walls.

Height-to-thickness ratios are established for areas with ground acceleration greater than 0.2g in Section 5.5 of

FEMA 178 (BSSC, 1992a); areas of acceleration less than 0.2g are covered in Appendix C of FEMA 178.

The procedure to check walls that do not meet the height-to-thickness ratios (Section 2.4.6) for out-of-plane forces in areas with a design acceleration less than 0.2g requires the evaluation of the seismic demand on the wall and calculations to determine the bending stresses.

The MSJC (1995) provisions allow flexural tension in the wall when the building is in moderate seismic areas and the wall is unreinforced or has minimal prescriptive reinforcement. If the construction does not conform to the MSJC (1995) minimum reinforcing requirements, the MSJC allowable stress, multiplied by 2.5, and reduced by the appropriate capacity reduction factor, may be used to determine the flexured capacity.

F. Infill Walls

The shear capacity of the reinforced concrete columns constrained by the infill should be determined using the Quick Check procedures of FEMA 178 (BSSC, 1992a). This check neglects the shear resistance provided by the column ties.

If the column fails the Quick Check, the location and size of the reinforcing and the strength of the concrete should be determined. The column should be analyzed for the capacity to resist the imposed moments and shears, using a more detailed evaluation. If the column is adequate as a “short column,” the partial height infill wall can be connected to the columns and considered to span horizontally. Otherwise, isolation is required.

C10.3.3.4 Shear Walls in Wood Frame Buildings

A. Shear Stress

All walls in wood frame construction participate in the lateral-force-resisting system. The evaluation of these walls is based on the FEMA 178 (BSSC, 1992a) Quick Checks. Where the average stress exceeds the FEMA 178 recommended values, a more detailed evaluation is needed. This detailed evaluation, using FEMA 178 (BSSC, 1992a) forces and procedures, should employ a more accurate estimation of the level and distribution of the lateral loads.

B. Openings

When walls have large openings, little or no resistance is available and they must be specially detailed or braced to other parts of the structure. Such bracing is not a conventional construction procedure. Lack of this bracing can lead to collapse of the wall.

It is necessary to check the ability of the walls and diaphragms to control, through torsional capacity, displacements at walls with large openings. A check should also be made to determine that the diaphragm is a complete system with chords and collectors provided to deliver the lateral loads as required.

C. Wall Detailing

The basic lateral strength and stability of wood walls is limited. Additional strength can be achieved if the wall supports enough dead load to resist overturning and has details adequate to transfer these loads.

D. Cripple Walls

Cripple walls are short stud walls that enclose a crawl space between the first floor and the ground. Often there are no other walls at this level, and these walls have no stiffening elements other than decorative sheathing. If this sheathing fails, the relatively rigid upper part of the building will fall. To be effective, all exterior cripple walls below the first floor level should be checked to ensure that they have adequate shear strength and stiffness, and proper connection to the floor and foundation. Cripple walls that change height along their length, such as in hillside locations, do not distribute shear uniformly to the walls, due to the varying stiffness, and create significant torsion in the building foundation. Simply sheathing all surfaces may not provide adequate strength and stiffness. On extreme slopes, rigid bracing using steel braces, reinforced masonry shearwalls, or concrete shearwalls may need to be added.

E. Narrow Wood Shear Walls

See *Guidelines* Section 10.4.3.1.

F. Stucco Shear Walls

See *Guidelines* Section 10.4.3.2.

G. Gypsum Wallboard or Plaster Shear Walls

See *Guidelines* Section 10.4.3.3.

C10.3.4 Steel Braced Frames

C10.3.4.1 System Concerns

Braced frame structures are inherently stiffer than moment frame structures, since they resist lateral forces through truss action.

The Quick Stress Checks in FEMA 178 (BSSC, 1992a) can be used for simple buildings to assess the strength provided by the braced frames. Consideration of gravity effects on beams and columns in these frames should be combined with the lateral forces in a simplified analysis. Note that this check does not provide any indication of the ductility of these frames, which is also necessary for proper seismic performance. This tool is not appropriate for tall and/or irregular buildings.

Systematic Rehabilitation should be used in tall and/or irregular buildings to determine the expected frame capacity versus demand. (FEMA 178 [BSSC, 1992a], Section 6.1.1.)

C10.3.4.2 Stiffness of Diagonals

Code design requirements have allowed compression diagonal braces to have Kl/r ratios of up to 200 (Kl is the effective length; r minimizes the moment of inertia). Tension-only bracing is also allowed for some buildings. Cyclic tests have demonstrated that elements with high Kl/r ratios subjected to large deformations cannot be expected to provide adequate performance. Tension-only systems may allow the brace to deform with large velocities during cyclic response after tension yielding cycles have occurred. Limited energy dissipation and premature fracture can significantly increase the building displacements and jeopardize the performance of the framing system.

Simple braced frame analysis tools are provided by FEMA 178 (BSSC, 1992a), with a 25% amplification of the seismic forces prescribed where bracing elements have a Kl/r ratio greater than 120. This procedure is intended to require braced frames with relatively flexible diagonals to be capable of resisting larger forces. Differences in the performance of elements of various cross sections (e.g., cold-formed tubes, pipes, double angles or channels, single angles), can also be significant to the cyclic deformation performance and should be considered in the analysis.

Systematic Rehabilitation should be used in tall and/or irregular buildings to determine the expected frame capacity versus demand. Estimation of deformation capacities of bracing elements can be made based on examination of past experimental investigation results. (FEMA 178 [BSSC, 1992a], Sections 6.1.2 and 6.1.3.)

C10.3.4.3 Chevron or K-Bracing

There are many possible configurations for the diagonal elements in a braced frame. Some systems— chevron or V-braced—raise a concern that is not present for other brace configurations. When the compression brace buckles, the ability of the adjacent tension brace to resist additional load is dependent on the capacity of the floor beam to resist the large vertical load—the vertical component of the force in the tension brace. In most cases, the beams have not been designed for these large forces. As a result, the lateral load performance of these systems is considered to be less desirable than that of X-braced or single diagonal systems. K-bracing, where the diagonal members meet within the height of a column, is even less desirable than chevron bracing, since compression brace buckling can result in a large lateral force on the column, which could jeopardize its stability.

FEMA 178 (BSSC, 1992a) prescribes higher force levels for K-braced frames in an attempt to reduce the deformation demands to which the column may be subjected. No specific procedures are provided for chevron or V-braced frames.

Systematic Rehabilitation should be used in tall and/or irregular buildings to determine the expected frame capacity versus demand. (FEMA 178 [BSSC, 1992a], Section 6.1.4.)

C10.3.4.4 Braced Frame Connections

It is generally considered advisable to make the connections between the members of seismically designed frames stronger than the members, since connection failure is generally not ductile and may result in separation of the parts. Member yielding is generally considered to be more desirable than inelastic response of the connections. Especially important connections in braced frames are the column splices, since they may be subject to large tensile forces that could jeopardize stability if the connection were to fail. Proper consideration of any eccentricities between the connected members is necessary to avoid yielding prior to the development of the member strength.

FEMA 178 (BSSC, 1992a) requires that the brace connections be capable of developing the capacity of the diagonals, or else an amplified seismic load must be used. Special requirements for column splices are noted, with increased demands specified for partial penetration splices that have not demonstrated significant ductility in laboratory testing. Any eccentricities in the connections of the braced frames must be properly analyzed to ensure that premature member yielding due to the eccentricity does not occur.

Systematic Rehabilitation Analytical Procedures should be used in tall and/or irregular buildings to determine the expected frame demands. (FEMA 178 [BSSC, 1992a], Sections 6.1.5, 6.1.6, and 6.1.7.)

C10.3.5 Diaphragms

C10.3.5.1 Re-entrant Corners

Diaphragms with plan irregularities such as extending wings, plan insets, or E-, T-, X-, and L-shaped configurations have re-entrant corners where large tensile and compressive forces can develop. The diaphragm may not have sufficient strength at these re-entrant corners to resist these tensile and compressive forces, and locally concentrated damage may occur.

The chord requirements at the re-entrant corners of the diaphragm should be calculated from the required shear force that the diaphragm must resist, the configuration of the diaphragm, and the location of the vertical lateral-force-resisting elements (e.g., moment frames, braced frames, shear walls). Any chords and chord connections that may exist must be evaluated to determine if they have sufficient capacity to resist the required tensile and compressive forces at the re-entrant corner.

C10.3.5.2 Crossties

Continuous crossties between diaphragm chords are needed to resist out-of-plane forces on the walls and transfer these forces through the diaphragm into the supporting walls or frames. It is critical that the crossties have a positive and direct connection to the laterally supported walls that will prevent the walls and the diaphragm from separating. The connection of the crosstie to the wall and connections within the crosstie must be designed so cross-grain bending or cross-grain tension is not present in any wood member. Subdiaphragms may be used to reduce the length of some of the crossties, but full crossties must still be provided between subdiaphragms.

The out-of-plane wall anchorage force that the crossties are required to resist should be calculated. Both the crossties and a positive direct connection between the wall and the crossties should be designed to resist the required force without cross-grain bending or cross-grain tension in any wood members.

C10.3.5.3 Diaphragm Openings

Openings in diaphragms cause an increased shear demand in the segments of the diaphragm adjacent to the opening. Tension and compression forces caused by bending moments are at the edges of these segments of the diaphragm. Openings that are small relative to the diaphragm depth will cause only a slight increase in the shear demand. Openings that are large relative to the diaphragm depth can result in excessive shear demand and large moments and forces in the diaphragm. The stiffness of a diaphragm with openings of significant size is less than that of a comparable diaphragm without openings.

The shear capacity of the segments of the diaphragm adjacent to the opening should be checked to see if they have sufficient capacity to resist the required shear force, and, if the opening is adjacent to a vertical lateral-force-resisting element, a check should be made to confirm that there is a complete load path with sufficient strength to deliver the diaphragm shear to it. The moments and forces in the segments of the diaphragm adjacent to the opening, and the adequacy of any chords or drag struts, should also be checked.

C10.3.5.4 Diaphragm Stiffness/Strength

A. Board Sheathing

Straight-sheathed diaphragms are very flexible and have low shear capacity when compared to other types of wood diaphragms. Individual boards in the straight-sheathed diaphragm must have at least two nails into each of the supporting members to develop the nail couple, which provides the limited shear capacity of these diaphragms. Because of the limited strength and stiffness of these diaphragms, they are most suitable in areas of low seismicity. In areas of moderate to high seismicity, the span between vertical elements and the span-to-depth ratio of straight-sheathed diaphragms should be limited or the diaphragm should be strengthened. Other considerations include the type of vertical elements—because wood-frame walls tolerate much greater diaphragm deformations than do masonry walls—and the size of the loads, which may be small for many roof diaphragms even in areas of high seismicity.

The shear force that the diaphragm is required to resist should be calculated, and an analysis made to determine if the diaphragm has sufficient strength and stiffness to resist this force.

B. Unblocked Diaphragms

Wood structural panel diaphragms may or may not have blocking at the panel edges that are perpendicular to the framing and not supported by the framing. The shear capacity of unblocked wood structural panel diaphragms is quite limited, due to the reduced shear transfer capacity between panels at the unblocked panel edges. Unblocked diaphragms are also more flexible than comparable blocked diaphragms and will experience increased lateral deflections.

C. Spans

Diaphragms with long spans between vertical elements will often experience large lateral deflections and excessive diaphragm shears. Large deflection in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shears will cause damage and reduced stiffness in the diaphragm.

D. Span-to-Depth Ratio

Diaphragms with a high span-to-depth ratio will experience higher flexibility and diaphragm shear than comparable diaphragms with a low span-to-depth ratio. This is especially true for span-to-depth ratios greater than three to one. Large deflection in the diaphragm can result in increased damage or collapse of elements laterally supported by the diaphragm. Excessive diaphragm shears will cause damage and reduced stiffness in the diaphragm.

E. Diaphragm Continuity

Split level floors and roofs or diaphragms interrupted by expansion joints create discontinuities, unless special details are used or lateral-force-resisting elements are provided at the vertical offset of the diaphragm or on both sides of the expansion joint. Such a discontinuity may cause the diaphragm to function as a cantilever element or three-sided diaphragm. If the diaphragm is not supported on at least three sides by lateral-force-resisting elements, torsional forces in the diaphragm may cause it to become unstable. In both the cantilever and three-sided cases, increased lateral deflection in the discontinuous diaphragm may cause increased damage to, or collapse of, the supported elements.

F. Chord Continuity

Diaphragms with discontinuous chords or without chords will be more flexible and will experience more damage at perimeter areas than diaphragms with chords that are continuous and have sufficient connection capacity. Vertical offsets or elevation changes in a diaphragm often cause a chord discontinuity. This is especially critical in wood diaphragms that lack any natural tensile capacity.

C10.3.6 Connections

C10.3.6.1 Diaphragm/Wall Shear Transfer

The diaphragm shear at each floor or roof must be connected to the shear wall in order to provide a complete load path for the shear to transfer. Where the wall does not extend the full depth of the diaphragm, collectors or drag/strut components are required to deliver the shear to the wall.

After calculating the shear force at the shear wall, this force should be divided by the length of the wall to determine the shear transfer connection required per foot of wall. Where the wall does not extend the full depth of the diaphragm, the wall shear should be divided by the diaphragm width to determine the load per foot to the collector. The collector forces and connection requirements can then be determined by multiplying the load per foot to the collector by the collector length from its end to the location being analyzed.

C10.3.6.2 Diaphragm/Frame Shear Transfer

The floor and roof diaphragm must be adequately connected to the steel frames to provide a load path for the shears in the diaphragm to be delivered to the frames.

After calculating the shear force at the frame being analyzed, this force should be divided by the depth of the diaphragm to determine the shear per foot transfer requirement to the collector and frames. Collector forces can be determined by multiplying the shear per foot by the length from the end of the collector to the location being analyzed.

C10.3.6.3 Anchorage for Normal Forces

Walls that are not well anchored to the diaphragms may separate from the remainder of the structure and collapse during an earthquake. If these walls are bearing walls, partial floor collapse may result. The hazard amplifies with the height above the building base, and is affected by the soil type and the type and configuration of the walls and/or diaphragms.

Several guidelines for the evaluation of wall anchorage are provided in FEMA 178 (BSSC, 1992a). First, cross-grain tension can lead to abrupt brittle failures in wood ledgers; this condition should be eliminated. Second, wood diaphragms should be directly anchored to the walls for out-of-plane loading. Third, steel anchors should be utilized, and well developed into the diaphragm to achieve adequate capacity and ductility. Finally, anchorage from the floors or roof into the walls should have sufficient spacing, strength, and stability. For further explanation of these statements, refer to FEMA 178.

C10.3.6.4 Girder-Wall Connections

Where girder-wall connections are a primary part of the out-of-plane load path, the anchorage into the wall should be ductile. If the girder rests on a corbel, the bearing length should be adequate to accommodate expected motions. Where precast girders are welded to column corbels, unintended frame action may attract high seismic forces.

C10.3.6.5 Precast Connections

Precast concrete frames without shear walls must not be addressed under Simplified Rehabilitation (see *Guidelines* Table 10-1). For precast frames that are braced by concrete shear walls, the interconnections of elements that serve as the chords, ties, and collectors must be similar. These connections should be evaluated to determine whether they are adequate. Special consideration must be given to their as-built condition, since they are susceptible to failures induced by thermal stresses and corrosion.

C10.3.6.6 Wall Panels and Cladding

The connections between wall panels or cladding and the structural framing are important for preventing damage to both elements. Typically, cladding is not constructed integrally with the framing but is added afterward, so the connection often forms a potential weak link. The cladding, which is not designed as part of the lateral-force-resisting system, should be isolated so as not to be damaged by building drifts, yet anchored to prevent falling out under strong shaking. Precast concrete wall panels can themselves be much stiffer than the lateral-force-resisting system in a moment frame building; thus, if rigidly attached to the frame they can actually attract forces and route them through unintended load paths.

Systematic Rehabilitation Analysis Procedures may be beneficial for determining the actual expected building drifts. (FEMA 178 [BSSC, 1992a], Section 8.6.2.)

C10.3.6.7 Light Gage Metal, Plastic or Cementitious Roof Panels

The connections between flexible roof diaphragms and the structural framing are important for developing a building's load path. Typically, these types of roofs are not constructed integrally with the framing (as opposed to a concrete slab or deck and fill), so the connection often forms a potential weak link. (FEMA 178 [BSSC, 1992a], Section 8.6.1.)

The forces in the diaphragm can typically be determined by noncomputerized analysis using tributary areas. The existing connections should be checked for the forces developed.

C10.3.6.8 Mezzanine Connections

It is very common for mezzanines to lack a lateral-force-resisting system. If the mezzanine lacks bracing elements or is not adequately connected to walls or framing capable of adequately bracing the mezzanine, the mezzanine can be fully isolated and investigated as a separate structure. Lateral-force-resisting elements must be present in both directions to provide bracing.

C10.3.7 Foundations and Geologic Hazards

C10.3.7.1 Anchorage to Foundations

For FEMA 178 evaluation statements to be true, steel columns and wood posts must be positively attached to the foundation. Concrete columns are required to have longitudinal steel doweled into the foundation. Similarly, doweled reinforcing for masonry and concrete walls is required. It is also required that wood walls be anchored with bolts or drilled anchors. The ends of shear walls must be substantially anchored into the building foundation to resist overturning.

Where the bases of steel and wood columns are exposed, it is relatively easy to identify whether they are anchored to the foundation. In the case of concrete columns or walls it may be very difficult to determine, in the absence of drawings, whether there are foundation dowels. Generally, it is relatively more important that columns—particularly wood and steel columns—be anchored to the foundation to prevent movement during seismic shaking and potential loss of vertical support, than that walls be so anchored. It is improbable that concrete columns or walls would be displaced to the point of causing a vertical load-carrying deficiency during an earthquake due to lack of dowels into a footing. It also seems unreasonable to require URM walls to be anchored to the foundation, whereas reinforced masonry or concrete walls would be required to be doweled. With respect to wood frame walls and foundation anchorage, it is not generally considered to be a life safety hazard if a wood frame building is not anchored to its foundation. Judgment should be exercised in determining the need for the type of anchorage implied by the FEMA 178 provision. If lateral loads are resisted by a relatively few, highly stressed elements, such anchorage may be important. However, in buildings where there are a substantial number of walls resisting loads at relatively low stress, anchorage to the footings may not be necessary for the Life Safety Performance Level.

When anchorage requirements for vertical elements are determined to be necessary because of high stresses or relatively few elements, and the repairs required to do so are costly and/or intrusive, Systematic Rehabilitation measures are recommended. This is due to the fact that the more detailed Analysis Procedures may allow reduction in forces and, in some cases, justification that anchorage is not required, especially in the case of anchorage at ends of shear walls where some rocking due to lack of tension restraint at the ends of walls may be analytically justified. (FEMA 178 [BSSC, 1992a], Sections 8.4.1 - 8.4.7.)

C10.3.7.2 Condition of Foundations

The FEMA 178 evaluation statements relate to signs of excessive foundation movement or of deterioration due to corrosion or other material conditions. The intention is to verify that the foundation has performed adequately under prior loading, which normally includes dead loads, live loads, wind, and, in some cases, previous earthquakes. If this performance has been satisfactory there is less reason to be concerned over future performance during earthquakes. Similarly, with respect to deterioration of foundation elements and materials, if no signs of degradation are present it is reasonable to assume that the foundations will remain in serviceable condition.

The procedure for investigating the condition of existing foundations in FEMA 178 is essentially one of visual inspection. The difficulty is that both the deterioration of existing elements and materials problems are not always readily observable. In some cases excavation can be used to expose existing piles or pier footings for investigation. Some conditions can be easily identified, including spalling of concrete due to corrosion of rebar, or discoloration due to sulfate attack. With respect to settlement or distress due to loads in existing foundations, some measurements may be helpful. It is expected that building foundations, particularly shallow spread footings, will undergo some movement during the life of a structure; however, excessive differential settlements can cause distress to structural elements that are needed to resist seismic loading. For example, differential settlement in steel frames can actually cause yielding of moment connections. Angular distortions that exceed 0.25% to 0.50%, depending on the type of construction, should be investigated using more detailed field investigation and, probably, Systematic Rehabilitation. In addition to measuring changes in relative elevations, observations can be made of brittle concrete or masonry elements to identify cracking.

For foundations with signs of excessive distress—due to either service loading or material conditions—detailed investigations, including Systematic Rehabilitation, are warranted. These cases, however, will be unusual because building foundations generally perform well and should not be subject to intense scrutiny, unless there are signs of significant deterioration or distress. (FEMA 178 [BSSC, 1992a], Sections 9.1.1 - 9.1.2.)

C10.3.7.3 Overturning

If a building is sufficiently short compared to its base dimension, overturning effects may be neglected. The criteria in FEMA 178 (BSSC, 1992a) are related to anticipated seismicity of the area by the velocity-rated acceleration factor. Buildings in areas of relatively low seismicity may be more slender and still not require consideration of overturning effects.

If the geometric requirement (base-to-height ratio) of FEMA 178 (BSSC, 1992a) is exceeded, simplified calculations are required. For shallow foundations, if bearing pressures under total gravity loads plus earthquake loads do not exceed two times the allowable static bearing pressures, the foundation is considered adequate for overturning. For deep foundations, the total load may not exceed the ultimate vertical capacity of the pile or piers.

If the simplified calculations are required, FEMA 178 does not provide guidance on the determination of the allowable capacity of shallow foundations nor the ultimate capacity of deep foundations. In some cases this information may be available from previous soils reports or from consultation with a qualified geotechnical engineer. Building failures from excessive foundation loading have very seldom been observed in past earthquakes. Additionally, some amount of foundation yielding and movement tends to reduce the forces transmitted to the superstructure. In this sense, inelastic behavior in the foundation is considered desirable.

The type of mitigative action required to correct overturning problems of foundations is generally very expensive. For this reason, it is strongly recommended that Systematic Rehabilitation be used for evaluation, design, and construction of mitigation measures for overturning. Chapter 4 of the *Guidelines* provides procedures for estimating foundation stiffnesses and capacities, for use in analyses to evaluate foundation performance more realistically. More realistic evaluation and design methods slightly increase engineering cost, but in cases such as this are likely to reduce construction costs considerably. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

C10.3.7.4 Lateral Loads

Lateral loads at the foundation level are transferred to the supporting soil by friction or passive pressure on the sides and bottoms of foundation elements. FEMA 178 evaluation statements require that these elements be capable of transferring lateral loads. Specific guidance on allowable horizontal loads or pressures is not provided. Ties between foundation elements are also required to be “adequate.” Also, building sites where significant difference in grade exists across a building site must account for lateral earthquake forces due to soil pressures on foundation walls.

FEMA 178 provides only a very qualitative assessment of lateral load transfer. Judgment should be used. For buildings in which the lateral load is transferred to the supporting soil in relatively few locations that are not generally tied to the rest of the structure, some conservatism is warranted. Concrete slabs on grade are most often adequate to tie foundations together as a unit. Experience in past earthquakes does not indicate that sliding, or lateral bearing, failure causes life safety problems in the absence of some differential vertical or horizontal permanent ground displacement—due to liquefaction, lateral spreading, or some other geologic site hazard.

C10.3.7.5 Geologic Site Hazards

FEMA 178 includes evaluation statements for liquefaction, slope failure, and surface fault rupture, which identify cases requiring detailed investigation.

C10.3.8 Evaluation of Materials and Conditions

C10.3.8.1 General

Techniques used in this evaluation step may range from simple visual inspection through sample removal and destructive testing in a laboratory. Visual inspection includes direct viewing techniques, noninvasive techniques (e.g., temporary removal of coverings, use of a fiberscope), or invasive exploration, which requires repairs to finishes after access and completion of inspection. Nondestructive and destructive testing techniques used are specific to the material type (e.g., wood, steel). Typical methods and their application are addressed in Chapters 5 through 8. Extension of visual inspection techniques includes the grading of wood lumber type and quality of construction, and evaluation of seismic deficiencies using FEMA 178 (BSSC, 1992a).

Recovery of original design and construction documentation is also necessary, as this information generally defines original component sizes, material strengths, connection configuration, and overall dimensions. The design professional shall conduct research to accumulate available construction documents, including interviews with the original architect-engineer and contractor. If the data do not exist and the original design and construction team is not known, it is necessary to prepare as-built layouts of the existing structural system and to determine material properties for the affected components.

Default material properties that may be used for guidance are included in Chapters 5 through 8 of the

Guidelines; these values would be verified as representative through a limited amount of testing of samples from existing components. Sampling and test methods for determining materials strength and other properties are similarly contained in Chapters 5 through 8. In general, the following minimum numbers of tests should be performed (the amount of data already known about the structure and quality of construction may reduce that number).

- When drawings and data on original construction exist, material variability is low (less than 25%), building height is two stories or less, and plan area is less than 2,000 square feet, three tests may be performed on random samples from each primary component type affected.

- When only limited drawings or information exist, the deficiency or damage is comprehensive, material properties have significant variance, or the building height and plan area exceed two stories and 2,000 square feet, six tests should be performed on random samples removed from each primary component type affected.

It is expected that additional tests will be planned by the design professional to address any abnormal conditions or deficiencies.

The extent of the deficiency or damage shall be determined through a combination of visual inspection and testing. The design professional shall establish the condition of in-place materials and affected structural systems as part of the evaluation process. Similarly, any constraints associated with the rehabilitation process—such as reinforcing material fit-up, access for strengthening, temporary abandonment of the building, and removal of coverings with historical value—shall also occur at this stage. Information gained in the evaluation phase shall be used in the analysis and design of rehabilitation measures. If possible, the scope of rehabilitation shall be reviewed with the client, owner, code official, and other involved parties (e.g., contractor) at the building site to ensure that all rehabilitation goals are met.

C10.3.8.2 Condition of Wood

The condition of the wood in a structure has a direct relationship to its performance in a seismic event. Wood that is split, rotten, or has insect damage may have a very low capacity to resist loads imposed by earthquakes. Structures with wood elements depend to a large extent on the connections between members. If the wood at a bolted connection is split, the connection will possess only a fraction of the capacity of a similar connection in sound wood.

A preliminary analysis of the structure will generally lead to an indication of the critical connections and members that are part of the lateral-load-resisting system for the structure. These members and connections are the logical areas to inspect for possible deterioration problems. The wood members should be examined by exposing a representative sample of locations and visually examining and probing the wood with an awl or small drill to determine the condition and extent of any rot or decay. (FEMA 178 [BSSC, 1992a], Section 3.5.1.)

C10.3.8.3 Overdriven Fasteners

Fasteners connecting structural panels to the framing are supposed to be driven flush with—but should not penetrate—the surface of the sheathing.

For structures built prior to the wide use of nailing guns (pre-1970), the problem is generally not present. More recent projects are often constructed with alternative fasteners, such as staples, T-nails, clipped nail heads, or cooler nails, installed with pneumatic nail guns and often overdriven, completely penetrating one or more panel plys. This effectively reduces the shear capacity of the fastener. Nail shank diameter should also be checked for conformance with the common nail value, which is the basis for the shear values established in most reference documents.

The overdriven fasteners can be evaluated by comparing the length of the fastener in the panel to the thickness of the panel and reducing the capacity of the panel by the same ratio. (FEMA 178 [BSSC, 1992a], Section 3.5.2.)

C10.3.8.4 Condition of Steel

Environmental effects over prolonged periods of time may lead to deterioration of elements of steel lateral-force-resisting frames. Deterioration, in the form of rusting or corrosion, can significantly reduce the member cross sections, with a corresponding reduction in capacity. Such deterioration must be considered in the seismic evaluation.

Appropriate estimates of the capacity reduction that has occurred must be based on the extent of field investigation performed. If significant deterioration is observed, more extensive field work may be justified. Estimates of the deterioration in other elements that were not specifically evaluated may be required.

In addition to repair of damage, the causes of deterioration must be determined through investigation, and eliminated to protect the steel in the future. The demands on the existing elements can be reduced by the addition of braced bays, shear wall panels, or base isolation.

Systematic Rehabilitation Analytical Procedures should be used in tall and/or irregular buildings to determine the expected frame demands. (FEMA 178 [BSSC, 1992a], Section 3.5.3.)

C10.3.8.5 Condition of Concrete

Damaged or deteriorated material may not be readily observable. Visual inspection should be conducted.

Visual inspection of the material may be adequate if the damage is not severe and the intent is to patch and repair the distressed region. Where the existing material will remain without modification, appropriate tests should be conducted to determine the usable strength.

In general, the most straightforward Simplified Rehabilitation Method solution would be to identify the causes of the condition and define corrective methods to prevent the deterioration from continuing, and to remove and replace the deteriorated material using appropriate repair techniques (see ACI publications). (FEMA 178 [BSSC, 1992a], Sections 3.5.4, 3.5.5, 3.5.8.)

C10.3.8.6 Post-Tensioning Anchors

Corrosion in post-tensioning anchors can lead to failure of gravity systems if ground shaking causes a release or slip of prestressing strands. Coil anchors (with or without corrosion) have performed poorly under cyclic loads.

The material around the anchors should be sound and capable of providing adequate encasement of the anchor. Inspection of the anchors should be visual, and may involve chipping away surface material if there is evidence of internal corrosion or deterioration. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

C10.3.8.7 Quality of Masonry

If the masonry walls do not pass the FEMA 178 evaluation statements, one alternative is to discount the strength of sections of walls that do not pass the calculations.

The ASTM standards on mortar, sponsored by ASTM Committee C-12, provide information on repointing mortar. In order to restore the strength of the wall to its initial condition, all of the eroded mortar must be replaced by repointing. In the event that this is not practical, the wall should be tested in accordance with Appendix C of FEMA 178 to determine the allowable stresses that may be used. (FEMA 178 [BSSC, 1992a], Sections A4, Sections 3.5.9, 3.5.10, and 3.5.11.)

C10.4 Amendments to FEMA 178

Several amendments to FEMA 178 (BSSC, 1992a) have been developed in Section 10.4 of the *Guidelines*. They are based on deficiencies observed as a result of significant earthquakes that have occurred since the publication of FEMA 178. The eight new deficiencies are presented in the same style as in FEMA 178; the format includes a true/false evaluation statement, a paragraph of commentary to identify the concern, and a suggested procedure to follow if the evaluation statement is found to be false. The new amendments are covered in *Guidelines* Section 10.4 and are included in the complete list of FEMA 178 (BSSC, 1992a) deficiencies, including the amendments, in Section C10.5 of this *Commentary.*

C10.5 FEMA 178 Deficiency Statements

This *Commentary* section provides a complete list of all FEMA 178 (BSSC, 1992a) deficiency evaluation statements, as well as the eight new potential deficiencies listed in Section 10.4 of the *Guidelines*, presented in a logical, combined order.

C10.5.1 Building Systems

C10.5.1.1 Load Path

The structure contains a complete load path, for seismic force effects from any horizontal direction, that serves to transfer the inertial forces from the mass to the foundation. (FEMA 178 [BSSC, 1992a], Section 3.1.)

C10.5.1.2 Redundancy

The structure will remain laterally stable after the failure of any single element. (FEMA 178 [BSSC, 1992a], Section 3.2.)

C10.5.1.3 Vertical Irregularities

A. Weak Story

Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80% of the strength of the story above. (FEMA 178 [BSSC, 1992a], Section 3.3.1.)

B. Soft Story

Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. (FEMA 178 [BSSC, 1992a], Section 3.3.2.)

C. Geometry

There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to the adjacent stories). (FEMA 178 [BSSC, 1992a], Section 3.3.3.)

D. Mass

There are no significant mass irregularities; there is no change of effective mass of more than 50% from one story to the next, excluding light roofs. (FEMA 178 [BSSC, 1992a], Section 3.3.4.)

E. Vertical Discontinuities

All shear walls, infilled walls, and frames are continuous to the foundation. (FEMA 178 [BSSC, 1992a], Section 3.3.5.)

C10.5.1.4 Plan Irregularities Creating Torsion

The lateral-force-resisting elements form a well-balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20% of the width of the structure in either major plan dimension. (FEMA 178 [BSSC, 1992a], Section 3.3.6.)

C10.5.1.5 Adjacent Buildings

There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be “immediately adjacent” if it is within two inches times the number of stories away from the building being evaluated. (FEMA 178 [BSSC, 1992a], Section 3.4.)

C10.5.1.6 Lateral Load Path at Pile Caps

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

C10.5.1.7 Deflection Compatibility

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.

C10.5.2 Moment Frames

C10.5.2.1 Steel Moment Frames

A. Drift Check

The building satisfies the Quick Check of the frame drift. (FEMA 178 [BSSC, 1992a], Section 4.2.1.)

B. Frame Concerns

Compact Members. All moment frame elements meet the compact section requirements of the basic AISC documents (AISC, 1986 and 1989). (FEMA 178 [BSSC, 1992a], Section 4.2.2.)

Beam Penetrations. All openings in beam webs have a depth less than one-quarter of the beam depth and are located in the center half of the beams. (FEMA 178 [BSSC, 1992a], Section 4.2.3.)

Out-of-Plane Bracing. Beam-column joints are braced out-of-plane. (FEMA 178 [BSSC, 1992a], Section 4.2.9.)

C. Strong Column-Weak Beam

In areas of high seismicity (A_v greater than or equal to 0.2), at least one-half of the joints are strong column-weak beam (33% on every line of moment frame). Roof frame joints need not be considered. (FEMA 178 [BSSC, 1992a], Section 4.2.8.)

D. Connections

Moment Connections. All beam-column connections in the lateral-force-resisting moment frame have full-penetration flange welds and a bolted or welded web connection. (FEMA 178 [BSSC, 1992a], Section 4.2.4.)

Column Splices. In areas of high seismicity (A_v greater than or equal to 0.2), all column splice details of the moment-resisting frames include connection of both flanges and the web. (FEMA 178 [BSSC, 1992a], Section 4.2.5.)

Joint Webs. All web thicknesses within joints of moment-resisting frames meet the AISC criteria for web shear (AISC, 1986 and 1989). (FEMA 178 [BSSC, 1992a], Section 4.2.6.)

Girder Flange Continuity Plates. There are girder flange continuity plates at joints. (FEMA 178 [BSSC, 1992a], Section 4.2.7.)

Moment-Resisting Connections. All moment connections are able to develop the strength of the adjoining members or panel zones.

C10.5.2.2 Concrete Moment Frames

A. Quick Checks, Frame, and Nonductile Detail Concerns

Shearing Stress Check. The building satisfies the Quick Check of the average shearing stress in the columns. (FEMA 178 [BSSC, 1992a], Section 4.3.1.)

Drift Check. The building satisfies the Quick Check of story drift. (FEMA 178 [BSSC, 1992a], Section 4.3.2.)

Prestressed Frame Elements. The lateral-load-resisting frames do not include any prestressed or post-tensioned elements. (FEMA 178 [BSSC, 1992a], Section 4.3.3.)

Joint Eccentricity. There are no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines. (FEMA 178 [BSSC, 1992a], Section 4.3.4.)

No Shear Failures. The shear capacity of frame members is greater than the moment capacity. (FEMA 178 [BSSC, 1992a], Section 4.3.5.)

Strong Column-Weak Beam. The moment capacity of the columns is greater than that of the beams. (FEMA 178 [BSSC, 1992a], Section 4.3.6.)

Stirrup and Tie Hooks. The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more. (FEMA 178 [BSSC, 1992a], Section 4.3.7.)

Column-Tie Spacing. Frame columns have ties spaced at $d/4$ or less throughout their length and at $8d_b$ or less at all potential plastic hinge regions. (FEMA 178 [BSSC, 1992a], Section 4.3.8.)

Column-Bar Splices. All column-bar lap splice lengths are greater than $35d_b$ long and are enclosed by ties spaced at $8d_b$ or less. (FEMA 178 [BSSC, 1992a], Section 4.3.9.)

Beam Bars. At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the steel provided at the joints for either positive or negative moment is continuous throughout the members. (FEMA 178 [BSSC, 1992a], Section 4.3.10.)

Beam-Bar Splices. The lap splices for the longitudinal beam reinforcing are located within the center half of the member lengths and not in the vicinity of potential plastic hinges. (FEMA 178 [BSSC, 1992a], Section 4.3.11.)

Stirrup Spacing. All beams have stirrups spaced $d/2$ or less throughout their length and at $8d_b$ or less at potential hinge locations. (FEMA 178 [BSSC, 1992a], Section 4.3.12.)

Beam Truss Bars. Bent-up longitudinal steel is not used for shear reinforcement. (FEMA 178 [BSSC, 1992a], Section 4.3.13.)

Joint Reinforcing. Column ties extend at their typical spacing through all beam-column joints at exterior columns. (FEMA 178 [BSSC, 1992a], Section 4.3.14.)

Flat Slab Frames. The system is not a frame consisting of a flat slab/plate without beams. (FEMA 178 [BSSC, 1992a], Section 4.3.15.)

B. Precast Moment Frames

The lateral loads are not resisted by precast concrete frame elements. (FEMA 178 [BSSC, 1992a], Section 4.4.1.)

C10.5.2.3 Frames Not Part of the Lateral-Force-Resisting System

A. Short Captive Columns

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.

C10.5.3 Shear Walls

C10.5.3.1 Cast-in-Place Concrete Shear Walls

A. Shearing Stress Check

The building satisfies the Quick Check of the shearing stress in the shear walls. (FEMA 178 [BSSC, 1992a], Section 5.1.1.)

B. Overturning

All shear walls have h_w/l_w ratios less than four to one. (FEMA 178 [BSSC, 1992a], Section 5.1.2.)

C. Coupling Beams

The stirrups in all coupling beams over means of egress are spaced at $d/2$ or less and are anchored into the core with hooks of 135 degrees or more. (FEMA 178 [BSSC, 1992a], Section 5.1.3.)

D. Boundary Element Detailing

Column Splices. Steel column splice details in shear wall boundary elements can develop the tensile strength of the column. (FEMA 178 [BSSC, 1992a], Section 5.1.4.)

Wall Connections. There is positive connection between the shear walls and the steel beams and columns. (FEMA, 178, Section 5.1.5.)

Confinement Reinforcing. For shear walls with h_w/l_w greater than 2.0, the boundary elements are confined with spirals or ties with spacing less than $8d_b$. (FEMA 178 [BSSC, 1992a], Section 5.1.6.)

E. Wall Reinforcement

Reinforcing Steel. The area of reinforcing steel for concrete walls is greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes, and the maximum spacing of reinforcing steel is 18 inches. (FEMA 178 [BSSC, 1992a], Section 5.1.7.)

Reinforcing at Openings. There is special wall reinforcement around all openings. (FEMA 178 [BSSC, 1992a], Section 5.1.8.)

Shear Stress Check. The building satisfies the Quick Check of the shearing stress in wood shear walls. (FEMA 178 [BSSC, 1992a], Section 5.6.1.)

Openings. Walls with garage doors or other large openings are braced with plywood shear walls, or supported by adjacent construction through substantial positive ties. (FEMA 178 [BSSC, 1992a], Section 5.6.2.)

Wall Requirements. All walls supporting tributary areas of 24 to 100 square feet per foot of wall are plywood-sheathed with proper nailing, or rod-braced, and have a height-to-depth ratio of one to one or less, or have properly detailed and constructed hold-downs. (FEMA 178 [BSSC, 1992a], Section 5.6.3.)

Cripple Walls. All exterior cripple walls below the first floor level are braced to the foundation with shear elements. (FEMA 178 [BSSC, 1992a], Section 5.6.4.)

C10.5.3.2 Precast Concrete Shear Walls

A. Panel-to-Panel Connections

Adjacent wall panels are not connected by welded steel inserts. (FEMA 178 [BSSC, 1992a], Section 5.2.1.)

B. Wall Openings

Openings constitute less than 75% of the length of any perimeter wall, with the wall piers having h_w/l_w ratios of less than 2.0. (FEMA 178 [BSSC, 1992a], Section 5.2.2.)

C. Collectors

Wall elements with openings larger than a typical panel at a building corner are connected to the remainder of the wall with collector reinforcing. (FEMA 178 [BSSC, 1992a], Section 5.2.3.)

C10.5.3.3 Masonry Shear Walls

A. Reinforcing in Masonry Walls

In areas of high seismicity (A_v greater than or equal to 0.2): (1) the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall, with a minimum of 0.0007 in either of the two directions; (2) the spacing of reinforcing steel is less than 48 inches; and (3) all vertical bars extend to the top of the walls. (FEMA 178 [BSSC, 1992a], Section 5.3.2.)

B. Shearing Stress Check

The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (FEMA 178 [BSSC, 1992a], Section 5.3.1.)

C. Reinforcing at Openings

All wall openings that interrupt rebar have trim reinforcing on all sides. (FEMA 178 [BSSC, 1992a], Section 5.3.3.)

D. Unreinforced Masonry Shear Walls

Shearing Stress Check. The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (FEMA 178 [BSSC, 1992a], Section 5.4.1.)

Masonry Lay-up. Filled collar joints of multiwythe masonry walls have negligible voids. (FEMA 178 [BSSC, 1992a], Section 5.4.2.)

E. Proportions, Solid Walls

Proportions. In areas of high seismicity (A_v greater than or equal to 0.2), the height-to-thickness ratio of the unreinforced masonry wall panels is as follows:

One-story building $h_w/t < 14$

Multistory building

Top story $h_w/t < 9$

Other stories $h_w/t < 20$

(FEMA 178 [BSSC, 1992a], Section 5.5.1.)

Solid Walls. The unreinforced masonry infill walls are not of cavity construction. (FEMA 178 [BSSC, 1992a], Section 5.5.2.)

F. Infill Walls

The unreinforced masonry infill walls are continuous to the soffits of the frame beams. (FEMA 178 [BSSC, 1992a], Section 5.5.3.)

C10.5.3.4 Shear Walls in Wood Frame Buildings

A. Shear Stress Check

The building satisfies the Quick Check of the shearing stress in wood shear walls. (FEMA 178 [BSSC, 1992a], Section 5.6.1.)

B. Openings

Walls with garage doors or other large openings are braced with plywood shear walls or supported by adjacent construction through substantial positive ties. (FEMA 178 [BSSC, 1992a], Section 5.6.2.)

C. Wall Requirements

All walls supporting tributary areas of 24 to 100 square feet per foot of wall are plywood sheathed with proper nailing, or rod-braced, and have a height-to-depth ratio of one to one or less, or have properly detailed and constructed hold-downs. (FEMA 178 [BSSC, 1992a], Section 5.6.3.)

D. Cripple Walls

All exterior cripple walls below the first floor level are braced to the foundation with shear elements. (FEMA 178 [BSSC, 1992a], Section 5.6.4.)

E. Narrow Wood Shear Walls

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

F. Stucco Shear Walls

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

G. Gypsum Wallboard or Plaster Shear Walls

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

C10.5.4 Steel Braced Frames

C10.5.4.1 Stress Check

The building satisfies the Quick Check of the stress in the diagonals. (FEMA 178 [BSSC, 1992a], Section 6.1.1.)

C10.5.4.2 Stiffness of Diagonals

A. Stiffness of Diagonals

All diagonal elements required to carry compression have Kl/r ratios less than 120. (FEMA 178 [BSSC, 1992a], Section 6.1.2.)

B. Tension-only Braces

Tension-only braces are not used as the primary diagonal bracing elements in structures over two stories in height. (FEMA 178 [BSSC, 1992a], Section 6.1.3.)

C10.5.4.3 Chevron or K-Bracing

The bracing system does not include chevron, V-, or K-braced bays. (FEMA 178 [BSSC, 1992a], Section 6.1.4.)

C10.5.5 Diaphragms

C10.5.5.1 Plan Irregularities: Re-entrant Corners

There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (FEMA 178 [BSSC, 1992a], Section 7.1.1.)

C10.5.5.2 Crossties

There are continuous crossties between diaphragm chords. (FEMA 178 [BSSC, 1992a], Section 7.1.2.)

C10.5.5.3 Diaphragm Openings

A. Reinforcing at Openings

There is reinforcing around all diaphragm openings that are larger than 50% of the building width in either major plan dimension. (FEMA 178 [BSSC, 1992a], Section 7.1.3.)

B. Openings at Shear Walls

Diaphragm openings immediately adjacent to the shear walls constitute less than 25% of the wall length, and the available length appears sufficient. (FEMA 178 [BSSC, 1992a], Section 7.1.4.)

C. Openings at Braced Frames

Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the length of the bracing. (FEMA 178 [BSSC, 1992a], Section 7.1.5.)

D. Openings at Exterior Masonry Shear Walls

Diaphragm openings immediately adjacent to exterior masonry walls are no more than eight feet long. (FEMA 178 [BSSC, 1992a], Section 7.1.6.)

C10.5.5.4 Sheathing

None of the diaphragms consist of straight sheathing or have span-to-depth ratios greater than two to one. (FEMA 178 [BSSC, 1992a], Section 7.2.1.)

C10.5.5.5 Unblocked Diaphragms

Unblocked wood panel diaphragms consist of horizontal spans less than 40 feet and have span-to-depth ratios less than or equal to three to one. (FEMA 178 [BSSC, 1992a], Section 7.2.3.)

C10.5.5.6 Spans

All diaphragms with spans greater than 24 feet have plywood or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (FEMA 178 [BSSC, 1992a], Section 7.2.2.)

C10.5.5.7 Span-to-Depth Ratio

If the span-to-depth ratios of wood diaphragms are greater than three to one, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (FEMA 178 [BSSC, 1992a], Section 7.2.4.)

C10.5.5.8 Diaphragm Continuity

None of the diaphragms are composed of split-level floors or, in wood commercial or industrial buildings, have expansion joints. (FEMA 178 [BSSC, 1992a], Section 7.2.5.)

C10.5.5.9 Chord Continuity

All chord elements are continuous, regardless of changes in roof elevation. (FEMA 178 [BSSC, 1992a], Section 7.2.6.)

C10.5.6 Connections

C10.5.6.1 Diaphragm/Wall Shear Transfer

A. Transfer to Shear Walls

Diaphragms are reinforced for transfer of loads to the shear walls. (FEMA 178 [BSSC, 1992a], Section 8.3.1.)

B. Topping Slab to Walls and Frames

Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled into the shear wall or frame elements. (FEMA 178 [BSSC, 1992a], Section 8.3.3.)

C10.5.6.2 Diaphragm/Frame Shear Transfer

A. Transfer to Steel Frames

The method used to transfer diaphragm shears to the steel frames is approved for use under lateral loads. (FEMA 178 [BSSC, 1992a], Section 8.3.2.)

B. Topping Slab to Walls and Frames

Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled into the shear wall or frame elements. (FEMA 178 [BSSC, 1992a], Section 8.3.3.)

C10.5.6.3 Anchorage for Normal Forces

A. Wood Ledgers

The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (FEMA 178 [BSSC, 1992a], Section 8.2.1.)

B. Wall Anchorage

The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads. (FEMA 178 [BSSC, 1992a], Section 8.2.2.)

C. Masonry Wall Anchors

Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (FEMA 178 [BSSC, 1992a], Section 8.2.3.)

D. Anchor Spacing

The anchors from the floor and roof systems into exterior masonry walls are spaced at four feet or less. (FEMA 178 [BSSC, 1992a], Section 8.2.4.)

E. Tilt-up Walls

Precast bearing walls are connected to the diaphragms for out-of-plane loads; steel anchors or straps are embedded in the walls and developed into the diaphragm. (FEMA 178 [BSSC, 1992a], Section 8.2.5.)

F. Panel-Roof Connection

There are at least two anchors from each precast wall panel into the diaphragm elements. (FEMA 178 [BSSC, 1992a], Section 8.2.6.)

G. Stiffness of Wall Anchors

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 inch larger than the bolt diameter.

C10.5.6.4 Girder-Wall Connections

A. Girders

Girders that are supported by walls or pilasters have special ties to secure the anchor bolts. (FEMA 178 [BSSC, 1992a], Section 8.5.1.)

B. Corbel Bearing

If the frame girders bear on column corbels, the length of bearing is greater than three inches. (FEMA 178 [BSSC, 1992a], Section 8.5.2.)

C. Corbel Connections

The frame girders are not supported on corbels with welded elements. (FEMA 178 [BSSC, 1992a], Section 8.5.3.)

C10.5.6.5 Braced Frame Connections

A. Concentric Joints

All the diagonal braces frame into the beam-column joints concentrically. (FEMA 178 [BSSC, 1992a], Section 6.1.5.)

B. Connection Strength

All the brace connections are able to develop the yield capacity of the diagonals. (FEMA 178 [BSSC, 1992a], Section 6.1.6.)

C. Column Splices

All column splice details of the braced frames can develop the column yield capacity. (FEMA 178 [BSSC, 1992a], Section 6.1.7.)

C10.5.6.6 Precast Connections

For buildings with concrete shear walls, the connection between precast frame elements—such as chords, ties, and collectors—in the lateral-force-resisting system can develop the capacity of the connected members. (FEMA 178 [BSSC, 1992a], Section 4.4.2.)

C10.5.6.7 Wall Panels

All wall panels (metal, fiberglass, or cementitious) are properly connected to the wall framing. (FEMA 178 [BSSC, 1992a], Section 8.6.2.)

C10.5.6.8 Light Gage Metal, Plastic, or Cementitious Roof Panels

All light gage metal, plastic, or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center. (FEMA 178 [BSSC, 1992a], Section 8.6.1.)

C10.5.7 Foundations and Geologic Hazards

C10.5.7.1 Anchorage of Vertical Components to Foundations

A. Steel Columns

The columns in the lateral-force-resisting frames are substantially anchored to the building foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.1.)

B. Concrete Columns

All longitudinal column steel is doweled in the foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.2.)

C. Wood Posts

There is positive connection of wood posts to the foundation and the elements being supported. (FEMA 178 [BSSC, 1992a], Section 8.4.3.)

D. Wall Reinforcing

All vertical wall reinforcing is doweled into the foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.4.)

E. Shear-Wall-Boundary Columns

The shear wall columns are substantially anchored to the building foundation. (FEMA 178 [BSSC, 1992a], Section 8.4.5.)

F. Wall Panels

The wall panels are connected to the foundation and/or ground floor slab with dowels equal to the vertical panel reinforcing. (FEMA 178 [BSSC, 1992a], Section 8.4.6.)

G. Wood Sills

All wall elements are bolted to the foundation sill at six-foot spacing or less, with proper edge distance for concrete and wood. (FEMA 178 [BSSC, 1992a], Section 8.4.7.)

C10.5.7.2 Condition of Existing Foundations

A. Foundation Performance

The structure does not show evidence of excessive foundation movement, such as settlement or heave, that would affect its integrity or strength. (FEMA 178 [BSSC, 1992a], Section 9.1.1.)

B. Deterioration

There is no evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown, or other reasons, in a manner that would affect the integrity or strength of the structure. (FEMA 178 [BSSC, 1992a], Section 9.1.2.)

C10.5.7.3 Overturning

The ratio of the effective horizontal dimension, at the foundation level of the seismic-force-resisting system, to the building height (base-to-height) exceeds $1.44A_v$. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

C10.5.7.4 Lateral Loads

A. Overturning

The ratio of the effective horizontal dimension, at the foundation level of the seismic-force-resisting system, to the building height (base-to-height) exceeds $1.44A_v$. (FEMA 178 [BSSC, 1992a], Section 9.2.1.)

B. Ties Between Foundation Elements

Foundation ties adequate for seismic forces exist where footings, piles, and piers are not restrained by beams, slabs, or competent soils or rock. (FEMA 178 [BSSC, 1992a], Section 9.2.2.)

C. Lateral Force on Deep Foundations

Piles and piers are capable of transferring the lateral forces between the structure and the soil. (FEMA 178 [BSSC, 1992a], Section 9.2.3.)

D. Pole Buildings

Pole foundations have adequate embedment. (FEMA 178 [BSSC, 1992a], Section 9.2.4.)

E. Sloping Sites

The grade difference from one side of the building to another does not exceed one-half story. (FEMA 178 [BSSC, 1992a], Section 9.2.5.)

C10.5.7.5 Geologic Site Hazards

A. Liquefaction

Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building. (FEMA 178 [BSSC, 1992a], Section 9.3.1.)

B. Slope Failure

The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures, or is capable of accommodating small predicted movements without failure. (FEMA 178 [BSSC, 1992a], Section 9.3.2.)

C. Surface Fault Rupture

Surface fault rupture and surface displacement at the building site are not anticipated. (FEMA 178 [BSSC, 1992a], Section 9.3.3.)

C10.5.8 Evaluation of Materials and Conditions

C10.5.8.1 Condition of Wood

None of the wood members shows signs of decay, shrinkage, splitting, fire damage, or sagging, and none of the metal accessories is deteriorated, broken, or loose. (FEMA 178 [BSSC, 1992a], Section 3.5.1.)

C10.5.8.2 Overdriven Nails

There is no evidence of overdriven nails in the shear walls or diaphragms. (FEMA 178 [BSSC, 1992a], Section 3.5.2.)

C10.5.8.3 Condition of Steel

There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical- or lateral-force-resisting systems. (FEMA 178 [BSSC, 1992a], Section 3.5.3.)

C10.5.8.4 Condition of Concrete

A. Deterioration of Concrete

There is no visible deterioration of concrete or reinforcing steel in any of the frame elements. (FEMA 178 [BSSC, 1992a], Section 3.5.4.)

B. Post-Tensioning Anchors

There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors have not been used. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

C. Concrete Wall Cracks

All diagonal cracks in the wall elements are 1.0 mm or less in width, are in isolated locations, and do not form an X pattern. (FEMA 178 [BSSC, 1992a], Section 3.5.6.)

D. Cracks in Boundary Columns

There are no diagonal cracks wider than 1.0 mm in concrete columns that encase the masonry infills. (FEMA 178 [BSSC, 1992a], Section 3.5.7.)

E. Precast Concrete Walls

There is no significant visible deterioration of concrete or reinforcing steel nor evidence of distress, especially at the connections. (FEMA 178 [BSSC, 1992a], Section 3.5.8.)

C10.5.8.5 Post-Tensioning Anchors

There is no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors have not been used. (FEMA 178 [BSSC, 1992a], Section 3.5.5.)

C10.5.8.6 Quality of Masonry

A. Masonry Joints

The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (FEMA 178 [BSSC, 1992a], Section 3.5.9.)

B. Masonry Units

There is no visible deterioration of large areas of masonry units. (FEMA 178 [BSSC, 1992a], Section 3.5.10.)

C. Cracks in Infill Walls

There are no diagonal cracks in the infilled walls that extend throughout a panel or are greater than 1.0 mm wide. (FEMA 178 [BSSC, 1992a], Section 3.5.11.)

C10.6 Definitions

No commentary is provided for this section.

C10.7 Symbols

No commentary is provided for this section.

C10.8 References

ACI, 1989, *Building Code Requirements for Reinforced Concrete*, Report No. ACI 318-89, American Concrete Institute, Detroit, Michigan.

AISC, 1986, *Manual of Steel Construction—Allowable Stress Design*, Eighth Edition, Second Revision, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1989, *Manual of Steel Construction—Allowable Stress Design*, Ninth Edition, Second Revision, American Institute of Steel Construction, Chicago, Illinois.

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

ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings*, Report No. ATC-14, Applied Technology Council, Redwood City, California.

ATC, 1989, *A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)*, Report No. ATC-22, Applied Technology Council, Redwood City, California.

ATC, 1995, *Structural Response Modification Factors*, Report No. ATC-19, Applied Technology Council, Redwood City, California.

ATC, 1997, *Guidelines for the Seismic Rehabilitation of Buildings: Example Application*, prepared by the Applied Technology Council for the Building Seismic Safety Council and the Federal Emergency Management Agency (Report No. FEMA 276), Washington, D.C.

BSSC, 1988, *NEHRP Recommended Provisions for the Department of Seismic Regulations for New Buildings*, 1988 Edition, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washinton, D.C.

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.

ICBO, 1994a, *Uniform Code for Building Conservation*, International Conference of Building Officials, Whittier, California.

ICBO, 1994b, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

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, the Applied Technology Council, and California Universities for Research in Earthquake Engineering for the Federal Emergency Management Agency (Report No. FEMA 267), Washington, D.C.