10. Simplified Rehabilitation

10.1 Scope

This chapter sets forth requirements for the rehabilitation of buildings using the Simplified Rehabilitation Method. Section 10.2 outlines the procedure of the Simplified Rehabilitation Method. Section 10.3 specifies actions for correction of deficiencies using the Simplified Rehabilitation Method.

C10.1 Scope

The Simplified Rehabilitation Method is intended primarily for use on a select group of simple buildings.

The Simplified Rehabilitation Method only applies to buildings that fit into one of the Model Building Types and conform to the limitations of Table 10-1, which sets the standard for simple, regularly configured buildings defined in Table 10-2. Building regularity is an important consideration in the application of the method. Regularity is determined by checklist statements addressing building configuration issues. The Simplified Rehabilitation Method may be used if an evaluation shows no deficiencies with regard to regularity. Buildings that have configuration irregularities (as determined by an *FEMA 310* Tier 1 or Tier 2 Evaluation) may use this Simplified Rehabilitation Method to achieve the Life Safety Building Performance Level only if the resulting rehabilitation work eliminates all significant vertical and horizontal irregularities and results in a building with a complete seismic lateral-force-resisting load path.

The technique described in this chapter is one of the two rehabilitation methods defined in Section 2.3. It is to be used only by a design professional, and only in a manner consistent with this standard. Consideration must be given to all aspects of the rehabilitation process, including the development of appropriate as-built information, proper design of rehabilitation techniques, and specification of appropriate levels of quality assurance.

"Simplified Rehabilitation" reflects a level of analysis and design that (1) is appropriate for small, regular buildings and buildings that do not require advanced analytical procedures; and (2) achieves the Life Safety Performance Level for the BSE-1 Earthquake Hazard Level as defined in Chapter 1, but does not necessarily achieve the Basic Safety Objective (BSO).

FEMA 178, the *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, a nationally applicable method for seismic evaluation of buildings, was the basis for the Simplified Rehabilitation Method in *FEMA 273*. *FEMA 178* is based on the historic behavior of buildings in past earthquakes and the success of current code provisions in achieving the Life Safety Building Performance Level. It is organized around a set of common construction styles called model buildings.

Since the preliminary version of *FEMA 178* was completed in the late 1980s, new information has become available and has been incorporated into *FEMA 310*, which is an updated version of *FEMA 178*. This information includes additional Model Building Types, eight new evaluation statements for potential deficiencies, a reorganization of the procedure to clearly state the intended three-tier approach, and new analysis techniques that parallel those of *FEMA 273*. *FEMA 310* is the basis of the Simplified Rehabilitation Method in this standard.

The Simplified Rehabilitation Method may yield a more conservative result than the Systematic Method because of a variety of simplifying assumptions.

10.2 Procedure

Use of Simplified Rehabilitation shall be permitted in accordance with the limitations of Section 2.3.1. The Simplified Rehabilitation Method shall be implemented by completing each of the following steps:

1. The building shall be classified as one of the Model Building Types listed in Table 10-1 and defined in Table 10-2.

2. A Tier 1 and a Tier 2 Seismic Evaluation of the building in its existing state shall be performed for

the Life Safety Building Performance Level in accordance with *FEMA 310*. In the event of differences between this standard and the *FEMA 310* procedures, the *FEMA 310* procedures shall govern.

3. The deficiencies identified by the *FEMA 310* Evaluation conducted in Step 2 shall be ranked from highest to lowest priority.

4. Rehabilitation measures shall be developed in accordance with Section 10.3 to mitigate the deficiencies identified by the *FEMA 310* Evaluation.

5. The proposed rehabilitation scheme shall be designed such that all deficiencies identified by the *FEMA 310* Evaluation of Step 2 are eliminated.

6. A complete Tier 1 and Tier 2 Evaluation of the building in its proposed rehabilitated state shall be performed in accordance with *FEMA 310*. In the event of differences between this standard and the *FEMA 310* procedures, the *FEMA 310* procedures shall govern.

7. Rehabilitation measures for architectural, mechanical, and electrical components shall be developed in accordance with Chapter 11 for the Life Safety Nonstructural Performance Level at the BSE-1 Earthquake Hazard Level.

8. Construction documents, including drawings and specifications and a quality assurance program, shall be developed as defined in Chapter 2.

C10.2 Procedure

The basis of the Simplified Rehabilitation Method is the *FEMA 310* procedure. There are intentional differences between the provisions of this standard and *FEMA 310* with regard to site class amplification factors, seismicity, and design earthquake, among other issues.

For simple buildings with specific deficiencies, it is possible and advisable to prioritize the rehabilitation measures. This is often done when the construction has limited funding or must take place while the building is occupied. In both cases, it is preferable to correct the worst deficiency first.

Potential deficiencies are ranked in Tables C10-1 through C10-19; items in these tables are ordered roughly from highest priority at the top to lowest at the bottom, although this can vary widely in individual cases.

FEMA 310 lists specific deficiencies both by Model Building Type and by association with each building system. Tables C10-1 through C10-19 of this standard further group deficiencies by general characteristics. For example, the deficiency listing “Diaphragm Stiffness/Strength,” includes deficiencies related to the type of sheathing used, diaphragm span, and lack of blocking. Table C10-20 provides a complete cross-reference for sections in this standard, in *FEMA 310*, and in *FEMA 178*.

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 C10-12, 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

   1.1. Building systems: those with a discontinuous load path and little redundancy.

   1.2. Building elements: those with low strength and low ductility.

2. Intermediate

   2.1. Building systems: those with a discontinuous load path but substantial redundancy.

   2.2. 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 intent of Tables C10-1 to C10-19 is to guide the design professional in accomplishing a Partial Rehabilitation Objective. 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.

As a rule, the resulting rehabilitated building must be one of the Model Building Types. For example, adding concrete shear walls to concrete shear wall buildings or adding a complete system of concrete shear walls to a concrete frame building meets this requirement. Steel bracing may be used to strengthen wood or URM construction. For large buildings, it is advisable to explore several rehabilitation strategies and compare alternative ways of eliminating deficiencies.

For a Limited Rehabilitation Objective, the deficiencies identified by the *FEMA 310* Evaluation of Step 2 should be mitigated in order of priority based on the ranking performed in Step 3.

A complete evaluation of the building should confirm that the strengthening of any one element or system has not merely shifted the deficiency to another.

Specific application of the Systematic Rehabilitation Method is needed to achieve the BSO. The total strength of the building should be sufficient, and the ability of the building to experience the predicted maximum displacement without partial or complete collapse must be established.

If only a Partial Rehabilitation or Limited Rehabilitation Objective is intended, deficiencies should be corrected in priority order and in a way that will facilitate fulfillment of the requirements of a higher objective at a later date. Care must be taken to ensure that a Partial Rehabilitation effort does not make the building's overall performance worse by unintentionally channeling failure to a more critical element.

Table 10-1 Limitations on Use of the Simplified Rehabilitation Method

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

n.p. = Use of Simplified Rehabilitation Method shall not be permitted.

1. Seismic Zones shall be as defined in Section 1.6.3.
2. Buildings with different types of flexible diaphragms shall be permitted to be considered as having flexible diaphragms.
Multistory buildings having stiff diaphragms at all levels except the roof shall be permitted to be considered as having stiff diaphragms.
Buildings having both flexible and stiff diaphragms, or having diaphragm systems that are neither flexible nor stiff, in accordance with this chapter, shall be rehabilitated using the Systematic Method.

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

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

n.p. = Use of Simplified Rehabilitation Method shall not be permitted.

1. Seismic Zones shall be as defined in Section 1.6.3.
2. Buildings with different types of flexible diaphragms shall be permitted to be considered as having flexible diaphragms.
Multistory buildings having stiff diaphragms at all levels except the roof shall be permitted to be considered as having stiff diaphragms.
Buildings having both flexible and stiff diaphragms, or having diaphragm systems that are neither flexible nor stiff, in accordance with this chapter, shall be rehabilitated using the Systematic Method.

Table 10-2 Description of Model Building Types

Building Type 1—Wood Light Frame

W1: These buildings are single or multiple family dwellings of one or more stories in height. Building loads are light and the framing spans are short. Floor and roof framing consists of wood joists or rafters on wood studs spaced no more than 24 inches apart. The first floor framing is supported directly on the foundation, or is raised up on cripple studs and post and beam supports. The foundation consists of spread footings constructed on concrete, concrete masonry block, or brick masonry in older construction. Chimneys, when present, consist of solid brick masonry, masonry veneer, or wood frame with internal metal flues. Lateral forces are resisted by wood frame diaphragms and shear walls. Floor and roof diaphragms consist of straight or diagonal lumber sheathing, tongue and groove planks, oriented strand board, or plywood. Shear walls consist of straight or lumber sheathing, plank siding, oriented strand board, plywood, stucco, gypsum board, particle board, or fiberboard. Interior partitions are sheathed with plaster or gypsum board.

W1A: These buildings are multi-story, similar in construction to W1 buildings, but have openings in the exterior walls framed with post-and-beam construction in the lowest level.

Building Type 2—Wood Frames, Commercial and Industrial

W2: These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Lateral forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, oriented strand board, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Wall openings for storefronts and garages, when present, are framed by post-and-beam framing.

Building Type 3—Steel Moment Frames

S1: These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Lateral forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. When all connections are moment-resisting connections, the entire frame participates in lateral force resistance. When only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns may be oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. When the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. When the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. Foundations consist of concrete-spread footings or deep pile foundations.

S1A: These buildings are similar to S1 buildings, except that diaphragms consist of wood framing or untopped metal deck, and are flexible relative to the frames.

Building Type 4—Steel Braced Frames

S2: These buildings have a frame of steel columns, beams, and braces. Braced frames develop resistance to lateral forces by the bracing action of the diagonal members. The braces induce forces in the associated beams and columns such that all elements work together in a manner similar to a truss, with all element stresses being primarily axial. When the braces do not completely triangulate the panel, some of the members are subjected to shear and flexural stresses; eccentrically braced frames are one such case. Diaphragms transfer lateral loads to braced frames. The diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames.

S2A: These buildings are similar to S2 buildings, except that diaphragms consist of wood framing or untopped metal deck, and are flexible relative to the frames.

Building Type 5—Steel Light Frames

S3: These buildings are pre-engineered and prefabricated with transverse rigid steel frames. They are one story in height. The roof and walls consist of lightweight metal, fiberglass or cementitious panels. The frames are designed for maximum efficiency and the beams and columns consist of tapered, built-up sections with thin plates. The frames are built in segments and assembled in the field with bolted or welded joints. Lateral forces in the transverse direction are resisted by the rigid frames. Lateral forces in the longitudinal direction are resisted by wall panel shear elements or rod bracing. Diaphragm forces are resisted by untopped metal deck, roof panel shear elements, or a system of tension-only rod bracing.

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

Building Type 6—Steel Frames with Concrete Shear Walls

S4: These buildings consist of a frame assembly of steel beams and steel columns. The floors and roof consist of cast-in-place concrete slabs or metal deck with or without concrete fill. Framing consists of steel beams, open web joists or steel trusses. Lateral forces are resisted by cast-in-place concrete shear walls. These walls are bearing walls when the steel frame does not provide a complete vertical support system. In older construction, the steel frame is designed for vertical loads only. In modern dual systems, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidity. In the case of a dual system, the walls shall be evaluated under this building type and the frames shall be evaluated under S1 or S1A, Steel Moment Frames. Diaphragms consist of concrete or metal deck with or without concrete fill. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections.

Building Type 7—Steel Frame with Infill Masonry Shear Walls

S5: This is an older type of building construction that consists of a frame assembly of steel beams and steel columns. The floors and roof consist of cast-in-place concrete slabs or metal deck with concrete fill. Framing consists of steel beams, open web joists or steel trusses. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. Infill walls may completely encase the frame members, and present a smooth masonry exterior with no indication of the frame. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The diaphragms consist of concrete floors and are stiff relative to the walls.

S5A: These buildings are similar to S5 buildings, except that diaphragms consist of wood sheathing or untopped metal deck, or have large aspect ratios and are flexible relative to the walls.

Building Type 8—Concrete Moment Frames

C1: These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in regions of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in regions of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. Foundations consist of concrete-spread footings or deep pile foundations.

Building Type 9—Concrete Shear Wall Buildings

C2: These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on concrete columns or bearing walls. Lateral forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced, but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with concrete slabs and are stiff relative to the walls. Foundations consist of concrete-spread footings or deep pile foundations.

C2A: These buildings are similar to C2 buildings, except that diaphragms consist of wood sheathing, or have large aspect ratios, and are flexible relative to the walls.

Building Type 10—Concrete Frame with Infill Masonry Shear Walls

C3: This is an older type of building construction that consists of a frame assembly of cast-in-place concrete beams and columns. The floors and roof consist of cast-in-place concrete slabs. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. The seismic performance of this type of construction depends on the interaction between the frame and the infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after racking of the infill, may limit the semiductile behavior of the system. The diaphragms consist of concrete floors and are stiff relative to the walls.

C3A: These buildings are similar to C3 buildings, except that diaphragms consists of wood sheathing or untopped metal deck, or have large aspect ratios and are flexible relative to the walls.

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

Building Type 11—Precast/Tilt-up Concrete Shear Wall Buildings	
PC1:	These buildings are one or more stories in height and have precast concrete perimeter wall panels that are cast on site and tilted into place. Floor and roof framing consists of wood joists, glulam beams, steel beams or open web joists. Framing is supported on interior steel columns and perimeter concrete bearing walls. The floors and roof consist of wood sheathing or untapped metal deck. Lateral forces are resisted by the precast concrete perimeter wall panels. Wall panels may be solid, or have large window and door openings which cause the panels to behave more as frames than as shear walls. In older construction, wood framing is attached to the walls with wood ledgers. Foundations consist of concrete-spread footings or deep pile foundations.
PC1A:	These buildings are similar to PC1 buildings, except that diaphragms consist of precast elements, cast-in-place concrete, or metal deck with concrete fill, and are stiff relative to the walls.
Building Type 12—Precast Concrete Frames	
PC2:	These buildings consist of a frame assembly of precast concrete girders and columns with the presence of shear walls. Floor and roof framing consists of precast concrete planks, tees or double-tees supported on precast concrete girders and columns. Lateral forces are resisted by precast or cast-in-place concrete shear walls. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs.
PC2A:	These buildings are similar to PC2 buildings, except that concrete shear walls are not present. Lateral forces are resisted by precast concrete moment frames that develop their stiffness through beam-column joints rigidly connected by welded inserts or cast-in-place concrete closures. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs.
Building Type 13—Reinforced Masonry Bearing Wall Buildings with Flexible Diaphragms	
RM1:	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. Wood floor and roof framing consists of steel beams or open web joists, steel girders and steel columns. Lateral forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck, and are flexible relative to the walls. Foundations consist of brick or concrete-spread footings.
Building Type 14—Reinforced Masonry Bearing Wall Buildings with Stiff Diaphragms	
RM2:	These building are similar to RM1 buildings, except that the diaphragms consist of metal deck with concrete fill, precast concrete planks, tees, or double-tees, with or without a cast-in-place concrete topping slab, and are stiff relative to the walls. The floor and roof framing is supported on interior steel or concrete frames or interior reinforced masonry walls.
Building Type 15—Unreinforced Masonry Bearing Wall Buildings	
URM:	These buildings have perimeter bearing walls that consist of unreinforced clay brick masonry. Interior bearing walls, when present, also consist of unreinforced clay brick masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. When they exist, ties between the walls and diaphragms consist of bent steel plates or government anchors embedded in the mortar joints and attached to framing. Foundations consist of brick or concrete-spread footings.
URMA:	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In regions of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing.

10.3 Correction of Deficiencies

For Simplified Rehabilitation, deficiencies identified by an *FEMA 310* Evaluation shall be mitigated by implementing approved rehabilitation measures. The resulting building, including strengthening measures, shall comply with the requirements of *FEMA 310* and shall conform to one of the Model Building Types contained in Table 10-1, except that steel bracing in wood or unreinforced masonry buildings shall be permitted.

The Simplified Rehabilitation Method shall only be used to achieve Limited Rehabilitation Objectives. To achieve the Life Safety Building Performance Level (3-C) at the BSE-1 Earthquake Hazard Level, all deficiencies identified by an *FEMA 310* Evaluation shall be corrected to meet the *FEMA 310* criteria. To achieve a Partial Rehabilitation Objective, only selected deficiencies need be corrected.

To achieve the Basic Safety Objective, the Simplified Rehabilitation Method is not permitted, and deficiencies shall be corrected in accordance with the Systematic Rehabilitation Method of Section 2.3.

C10.3 Correction of Deficiencies

Implementing a rehabilitation scheme that mitigates all of a building's *FEMA 310* deficiencies using the Simplified Rehabilitation Method does not, in and of itself, achieve the Basic Safety Objective or any Enhanced Rehabilitation Objective as defined in Chapter 2 since the rehabilitated building may not meet the Collapse Prevention Structural Performance Level for the BSE-2 Earthquake Hazard Level. If the goal is to attain the Basic Safety Objective as described in Chapter 2 or other Enhanced Rehabilitation Objectives, this can be accomplished using the Systematic Rehabilitation Method defined in Chapter 2.

Suggested rehabilitation measures are listed by deficiency in the following sections.

C10.3.1 Building Systems

C10.3.1.1 Load Path

Load path discontinuities can be mitigated by adding elements to complete the load path. This may require adding new well-founded shear walls or frames to fill gaps in existing shear walls or frames that are not carried continuously to the foundation. Alternatively, it may require the addition of elements throughout the building to pick up loads from diaphragms that have no path into existing vertical elements (*FEMA 310*, Section 4.3.1).

C10.3.1.2 Redundancy

The most prudent rehabilitation strategy for a building without redundancy is to add new lateral-force-resisting elements in locations where the failure of a single element will cause an instability in the building. The added lateral-force-resisting elements should be of the same stiffness as the elements they are supplementing. It is not generally satisfactory just to strengthen a non-redundant element (such as by adding cover plates to a slender brace), because its failure would still result in an instability (*FEMA 310*, Sections 4.4.1.1.1, 4.4.2.1.1, 4.4.3.1.1, 4.4.4.1.1).

C10.3.1.3 Vertical Irregularities

New vertical lateral-force-resisting elements can be provided to eliminate the vertical irregularity. For weak stories, soft stories, and vertical discontinuities, new elements of the same type can be added as needed. Mass and geometric discontinuities must be evaluated and strengthened based on the Systematic Rehabilitation Method, if required by Chapter 2 (*FEMA 310*, Sections 4.3.2.4–4.3.2.5).

C10.3.1.4 Plan Irregularities

The effects of plan irregularities that create torsion can be eliminated with the addition of lateral-force-resisting bracing elements that will support all major diaphragm segments in a balanced manner. While it is possible in some cases to allow the irregularity to remain and instead strengthen those structural elements that are overstressed by its existence, this does not directly address the problem and will require the use of the Systematic Rehabilitation Method (*FEMA 310*, Section 4.3.2.6).

C10.3.1.5 Adjacent Buildings

Stiffening elements (typically braced frames or shear walls) can be added to one or both buildings to reduce the expected drifts to acceptable levels. With separate structures in a single building complex, it may be possible to tie them together structurally to force them to respond as a single structure. The relative stiffnesses of each and the resulting force interactions must be determined to ensure that additional deficiencies are not created. Pounding can also be eliminated by demolishing a portion of one building to increase the separation (*FEMA 310*, Section 4.3.1.2).

C10.3.1.6 Lateral Load Path at Pile Caps

Typically, deficiencies in the load path at the pile caps are not a life safety concern. However, if the design professional has determined that there is a strong possibility of a life safety hazard due to this deficiency, piles and pile caps may be modified, supplemented, repaired, or in the most severe condition, replaced in their entirety. Alternatively, the building system may be rehabilitated such that the pile caps are protected (*FEMA 310*, Section 4.6.3.10).

C10.3.1.7 Deflection Compatibility

Vertical lateral-force-resisting elements can be added to decrease the drift demands on the columns, or the ductility of the columns can be increased. Jacketing the columns with steel or concrete is one approach to increase their ductility (*FEMA 310*, Section 4.4.1.6.2).

C10.3.2 Moment Frames

C10.3.2.1 Steel Moment Frames

C10.3.2.1.1 Drift

The most direct mitigation approach is to add properly placed and distributed stiffening elements—new moment frames, braced frames, or shear walls—that can reduce the inter-story drifts to acceptable levels. Alternatively, the addition of energy dissipation devices to the system may reduce the drift, though these are outside the scope of the Simplified Rehabilitation Method (*FEMA 310*, Section 4.4.1.3.1).

C10.3.2.1.2 Frames

Noncompact members can be eliminated by adding appropriate steel plates. Eliminating or properly reinforcing large member penetrations will develop the demanded strength and deformations. Lateral bracing in the form of new steel elements can be added to reduce member unbraced lengths to within the limits prescribed. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands (*FEMA 310*, Sections 4.4.1.3.7, 4.4.1.3.8, and 4.4.1.3.10).

C10.3.2.1.3 Strong Column-Weak Beam

Steel plates can be added to increase the strength of the steel columns to beyond that of the beams to eliminate this issue. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands (*FEMA 310*, Section 4.4.1.3.6).

C10.3.2.1.4 Connections

Adding a stiffer lateral-force-resisting system (e.g., braced frames or shear walls) can reduce the expected rotation demands. Connections can be modified by adding flange cover plates, vertical ribs, haunches, or brackets, or removing beam flange material to initiate yielding away from the connection location (e.g., via a pattern of drilled holes or the cutting out of flange material). Partial penetration splices, which may become more vulnerable for conditions where the beam-column connections are modified to be more ductile, can be modified by adding plates and/or welds. Adding continuity plates alone is not likely to enhance the connection performance significantly (*FEMA 310*, Sections 4.4.1.3.3 – 4.4.1.3.5, and 4.4.1.3.9).

Moment-resisting connection capacity can be increased by adding cover plates or haunches, or using other techniques as stipulated in *FEMA 351*.

C10.3.2.2 Concrete Moment Frames

C10.3.2.2.1 Frame and Nonductile Detail Concerns

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

It must be verified that this new system sufficiently reduces the frame shears and inter-story drifts to acceptable levels (*FEMA 310*, Section 4.4.1.4).

C10.3.2.2.2 Precast Moment Frames

Precast concrete frames without shear walls may not be addressed under the Simplified Rehabilitation Method (see Table 10-1). Where shear walls are present, the precast connections must be strengthened sufficiently to meet the *FEMA 310* requirements.

The development of a competent load path is extremely critical in these buildings. If the connections have sufficient strength so that yielding will first occur in the members rather than in the connections, the building should be evaluated as a shear wall system Type C2 (*FEMA 310*, Section 4.4.1.5).

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

C10.3.2.3.1 Complete Frames

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

Incomplete frames are essentially bearing wall systems. The wall must be strengthened to resist the combined gravity/seismic loads or new columns added to complete the gravity load path (*FEMA 310*, Section 4.4.1.6.1).

C10.3.2.3.2 Short Captive Columns

Columns may be jacketed with steel or concrete such that they can resist the expected forces and drifts. Alternatively, the expected story drifts can be reduced throughout the building by infilling openings or adding shear walls (*FEMA 310*, Section 4.4.1.4.5).

C10.3.3 Shear Walls

C10.3.3.1 Cast-in-Place Concrete Shear Walls

C10.3.3.1.1 Shearing Stress

New shear walls can be provided and/or the existing walls can be strengthened to satisfy seismic demand criteria. New and strengthened walls must form a complete, balanced, and properly detailed lateral-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength such that yielding will first occur in the wall. All shear walls must have sufficient shear and overturning resistance to meet the *FEMA 310* load criteria (*FEMA 310*, Section 4.4.2.2.1).

C10.3.3.1.2 Overturning

Lengthening or adding shear walls can reduce overturning demands; increasing the length of footings will capture additional building dead load (*FEMA 310*, Section 4.4.2.2.4).

C10.3.3.1.3 Coupling Beams

To eliminate the need to rely on the coupling beam, the walls may be strengthened as required. The beam should be jacketed only as a means of controlling debris. If possible, the opening that defines the coupling beam should be infilled (*FEMA 310*, Section 4.4.2.2.3).

C10.3.3.1.4 Boundary Component Detailing

Splices may be improved by welding bars together after exposing them. The shear transfer mechanism can be improved by adding steel studs and jacketing the boundary components. (*FEMA 310*, Sections 4.4.2.2.5, 4.4.2.2.8, and 4.4.2.2.9).

C10.3.3.1.5 Wall Reinforcement

Shear walls can be strengthened by infilling openings, or by thickening the walls (see *FEMA 172*, Section 3.2.1.2) (*FEMA 310*, Sections 4.4.2.2.2 and 4.4.2.2.6).

C10.3.3.2 Precast Concrete Shear Walls

C10.3.3.2.1 Panel-to-Panel Connections

Appropriate Simplified Rehabilitation solutions are outlined in *FEMA 172*, Section 3.2.2.3 (*FEMA 310*, Section 4.4.2.3.5).

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

C10.3.3.2.2 Wall Openings

Infilling openings or adding shear walls in the plane of the open bays can reduce demand on the connections and eliminate frame action (*FEMA 310*, Section 4.4.2.3.3).

C10.3.3.2.3 Collectors

Upgrading the concrete section and/or the connections (e.g., exposing the existing connection, adding confinement ties, increasing embedment) can increase strength and/or ductility. Alternative load paths for lateral forces can be provided, and shear walls can be added to reduce demand on the existing collectors (*FEMA 310*, Section 4.4.2.3.4).

C10.3.3.3 Masonry Shear Walls

C10.3.3.3.1 Reinforcing in Masonry Walls

Nondestructive methods should be used to locate reinforcement, and selective demolition used if necessary to determine the size and spacing of the reinforcing. If it cannot be verified that the wall is reinforced in accordance with the minimum requirements, then the wall should be assumed to be unreinforced, and therefore must be supplemented with new walls, or the procedures for unreinforced masonry should be followed (*FEMA 310*, Section 4.4.2.4.2).

C10.3.3.3.2 Shearing Stress

To meet the lateral force requirements of *FEMA 310*, new walls can be provided or the existing walls can be strengthened as needed. New and strengthened walls must form a complete, balanced, and properly detailed lateral-force-resisting system for the building. Special care is needed to ensure that the connection of the new walls to the existing diaphragm is appropriate and of sufficient strength to deliver the actual lateral loads or force yielding in the wall. All shear walls must have sufficient shear and overturning resistance (*FEMA 310*, Section 4.4.2.4.1).

C10.3.3.3.3 Reinforcing at Openings

The presence and location of reinforcing steel at openings may be established using nondestructive or destructive methods at selected locations to verify the size and location of the reinforcing, or using both methods. Reinforcing must be provided at all openings as required to meet the *FEMA 310* criteria. Steel plates may be bolted to the surface of the section as long as the bolts are sufficient to yield the steel plate (*FEMA 310*, Section 4.4.2.4.3).

C10.3.3.3.4 Unreinforced Masonry Shear Walls

Openings in the lateral-force-resisting walls should be infilled as needed to meet the *FEMA 310* stress check. If supplemental strengthening is required, it should be designed using the Systematic Rehabilitation Method as defined in Chapter 2. Walls that do not meet the masonry lay-up requirements should not be considered as lateral force-resisting elements and shall be specially supported for out-of-plane loads (*FEMA 310*, Sections 4.4.2.5.1 and 4.4.2.5.3).

C10.3.3.3.5 Proportions of Solid Walls

Walls with insufficient thickness should be strengthened either by increasing the thickness of the wall or by adding a well-detailed strong back system. The thickened wall must be detailed in a manner that fully interconnects the wall over its full height. The strong back system must be designed for strength, connected to the structure in a manner that: (1) develops the full yield strength of the strong back, and (2) connects to the diaphragm in a manner that distributes the load into the diaphragm and has sufficient stiffness to ensure that the elements will perform in a compatible and acceptable manner. The stiffness of the bracing should limit the out-of-plane deflections to acceptable levels such as $L/600$ to $L/900$ (*FEMA 310*, Sections 4.4.2.4.4 and 4.4.2.5.2).

C10.3.3.3.6 Infill Walls

The partial infill wall should be isolated from the boundary columns to avoid a "short column" effect, except when it can be shown that the column is adequate. In sizing the gap between the wall and the columns, the anticipated inter-story drift must be considered. The wall must be positively restrained against out-of-plane failure by either bracing the top of the wall or installing vertical girts. These bracing elements must not violate the isolation of the frame from the infill (*FEMA 310*, Section 4.4.2.6).

C10.3.3.4 Shear Walls in Wood Frame Buildings

C10.3.3.4.1 Shear Stress

Walls may be added or existing openings filled. Alternatively, the existing walls and connections can be strengthened. The walls should be distributed across the building in a balanced manner to reduce the shear stress for each wall. Replacing heavy materials such as tile roofing with lighter materials will also reduce shear stress. (*FEMA 310*, Section 4.4.2.7.1).

C10.3.3.4.2 Openings

Local shear transfer stresses can be reduced by distributing the forces from the diaphragm. Chords and/or collector members can be provided to collect and distribute shear from the diaphragm to the shear wall or bracing (see *FEMA 172*, Figure 3.7.1.3). Alternatively, the opening can be closed off by adding a new wall with plywood sheathing. (*FEMA 310*, Section 4.4.2.7.8).

C10.3.3.4.3 Wall Detailing

If the walls are not bolted to the foundation or if the bolting is inadequate, bolts can be installed through the sill plates at regular intervals (see *FEMA 172*, Figure 3.8.1.2a). If the crawl space is not deep enough for vertical holes to be drilled through the sill plate, the installation of connection plates or angles may be a practical alternative (see *FEMA 172*, Figure 3.8.1.2b). Sheathing and additional nailing can be added where walls lack proper nailing or connections. Where the existing connections are inadequate, adding clips or straps will deliver lateral loads to the walls and to the foundation sill plate (*FEMA 310*, Section 4.4.2.7.9).

C10.3.3.4.4 Cripple Walls

Where bracing is inadequate, new plywood sheathing can be added to the cripple wall studs. The top edge of the plywood is nailed to the floor framing and the bottom edge is nailed into the sill plate (see *FEMA 172*, Figure 3.8.1.3). Verify that the cripple wall does not change height along its length (stepped top of foundation). If it does, the shorter portion of the cripple wall will carry the majority of the shear and significant torsion will occur in the foundation. Added plywood sheathing must have adequate strength and stiffness to reduce torsion to an acceptable level. Also, it should be verified that the sill plate is properly anchored to the foundation. If anchor bolts are lacking or insufficient, additional anchor bolts should be installed. Blocking and/or framing clips may be needed to connect the cripple wall bracing to the floor diaphragm or the sill plate. (*FEMA 310*, Section 4.4.2.7.7).

C10.3.3.4.5 Narrow Wood Shear Walls

Where narrow shear walls lack capacity, they should be replaced with shear walls with a height-to-width aspect ratio of two-to-one or less. These replacement walls must have sufficient strength, including being adequately connected to the diaphragm and sufficiently anchored to the foundation for shear and overturning forces (*FEMA 310*, Section 4.4.2.7.4).

C10.3.3.4.6 Stucco Shear Walls

For strengthening or repair, the stucco should be removed, a plywood shear wall added, and new stucco applied. The plywood should be the manufacturer’s recommended thickness for the installation of stucco. The new stucco should be installed in accordance with building code requirements for waterproofing. Walls should be sufficiently anchored to the diaphragm and foundation (*FEMA 310*, Section 4.4.2.7.2).

C10.3.3.4.7 Gypsum Wallboard or Plaster Shear Walls

Plaster and gypsum wallboard can be removed and replaced with structural panel shear wall as required, and the new shear walls covered with gypsum wallboard (*FEMA 310*, Section 4.4.2.7.3).

C10.3.4 Steel Braced Frames

C10.3.4.1 System Concerns

If the strength of the braced frames is inadequate, more braced bays or shear wall panels can be added. The resulting lateral-force-resisting system must form a well-balanced system of braced frames that do not fail at their joints, are properly connected to the floor diaphragms, and whose failure mode is yielding of braces rather than overturning (*FEMA 310*, Sections 4.4.3.1.1 and 4.4.3.1.2).

C10.3.4.2 Stiffness of Diagonals

Diagonals with inadequate stiffness should be strengthened using supplemental steel plates, or replaced with a larger and/or different type of section. Global stiffness can be increased by the addition of braced bays or shear wall panels (*FEMA 310*, Sections 4.4.3.1.3 and 4.4.3.2.2).

C10.3.4.3 Chevron or K-Bracing

Columns or horizontal girts can be added as needed to support the tension brace when the compression brace buckles, or the bracing can be revised to another system throughout the building. The beam elements can be strengthened with cover plates to provide them with the capacity to fully develop the unbalanced forces created by tension brace yielding (*FEMA 310*, Sections 4.4.3.2.1 and 4.4.3.2.3).

C10.3.4.4 Braced Frame Connections

Column splices or other braced frame connections can be strengthened by adding plates and welds to ensure that they are strong enough to develop the connected members. Connection eccentricities that reduce member capacities can be eliminated, or the members can be strengthened to the required level by the addition of properly placed plates. Demands on the existing elements can be reduced by adding braced bays or shear wall panels (*FEMA 310*, Sections 4.4.3.1.4 and 4.4.3.1.5).

C10.3.5 Diaphragms

C10.3.5.1 Re-entrant Corners

New chords with sufficient strength to resist the required force can be added at the re-entrant corner. If a vertical lateral-force-resisting element exists at the re-entrant corner, a new collector element should be installed in the diaphragm to reduce tensile and compressive forces at the re-entrant corner. The same basic materials used in the diaphragm should be used for the chord (*FEMA 310*, Section 4.5.1.7).

C10.3.5.2 Cross Ties

New cross ties and wall connections can be added to resist the required out-of-plane wall forces and distribute these forces through the diaphragm. New strap plates and/or rod connections can be used to connect existing framing members together so they function as a crosstie in the diaphragm (*FEMA 310*, Section 4.5.1.2).

C10.3.5.3 Diaphragm Openings

New drag struts or diaphragm chords can be added around the perimeter of existing openings to distribute tension and compression forces along the diaphragm. The existing sheathing should be nailed to the new drag struts or diaphragm chords. In some cases it may also be necessary to: (1) increase the shear capacity of the diaphragm adjacent to the opening by overlaying the existing diaphragm with a wood structural panel, or (2) decrease the demand on the diaphragm by adding new vertical elements near the opening (*FEMA 310*, Sections 4.5.1.4 through 4.5.1.6 and 4.5.1.8).

C10.3.5.4 Diaphragm Stiffness/Strength

C10.3.5.4.1 Board Sheathing

When the diaphragm does not have at least two nails through each board into each of the supporting members, and the lateral drift and/or shear demands on the diaphragm are not excessive, the shear capacity and stiffness of the diaphragm can be increased by adding nails at the sheathing boards. This method of upgrade is most often suitable in areas of low seismicity. In other cases, a new wood structural panel should be placed over the existing straight sheathing, and the joints of the wood structural panels placed so they are near the center of the sheathing boards or at a 45-degree angle to the joints between sheathing boards (see *FEMA 172*, Section 3.5.1.2; *ATC-7*, and *FEMA 310*, Section 4.5.2.1).

C10.3.5.4.2 Unblocked Diaphragm

The shear capacity of unblocked diaphragms can be improved by adding new wood blocking and nailing at the unsupported panel edges. Placing a new wood structural panel over the existing diaphragm will increase the shear capacity. Both of these methods will require the partial or total removal of existing flooring or roofing to place and nail the new overlay or nail the existing panels to the new blocking. Strengthening of the diaphragm is usually not necessary at the central area of the diaphragm where shear is low. In certain cases when the design loads are low, it may be possible to increase the shear capacity of unblocked diaphragms with sheet metal plates stapled on the underside of the existing wood panels. These plates and staples must be designed for all related shear and torsion caused by the details related to their installation (*FEMA 310*, Section 4.5.2.3).

C10.3.5.4.3 Spans

New vertical elements can be added to reduce the diaphragm span. The reduction of the diaphragm span will also reduce the lateral deflection and shear demand in the diaphragm. However, adding new vertical elements will result in a different distribution of shear demands. Additional blocking, nailing, or other rehabilitation measures may need to be provided at these areas (*FEMA 172*, Section 3.4 and *FEMA 310*, Section 4.5.2.2).

C10.3.5.4.4 Span-to-Depth Ratio

New vertical elements can be added to reduce the diaphragm span-to-depth ratio. The reduction of the diaphragm span-to-depth ratio will also reduce the lateral deflection and shear demand in the diaphragm. Typical construction details and methods are discussed in *FEMA 172*, Section 3.4.

C10.3.5.4.5 Diaphragm Continuity

The diaphragm discontinuity should in all cases be eliminated by adding new vertical elements at the diaphragm offset or the expansion joint (see *FEMA 172,* Section 3.4). In some cases, special details may be used to transfer shear across an expansion joint—while still allowing the expansion joint to function—thus eliminating a diaphragm discontinuity (*FEMA 310*, Section 4.5.1.1).

C10.3.5.4.6 Chord Continuity

If members such as edge joists, blocking, or wall top plates have the capacity to function as chords but lack connection, adding nailed or bolted continuity splices will provide a continuous diaphragm chord. New continuous steel or wood chord members can be added to the existing diaphragm where existing members lack sufficient capacity or no chord exists. New chord members can be placed at either the underside or topside of the diaphragm. In some cases, new vertical elements can be added to reduce the diaphragm span and stresses on any existing chord members (see *FEMA 172*, Section 3.5.1.3, and *ATC-7*). New chord connections should not be detailed such that they are the weakest element in the chord (*FEMA 310*, Section 4.5.1.3).

C10.3.6 Connections

C10.3.6.1 Diaphragm/Wall Shear Transfer

Collector members, splice plates, and shear transfer devices can be added as required to deliver collector forces to the shear wall. Adding shear connectors from the diaphragm to the wall and/or to the collectors will transfer shear. See *FEMA 172*, Section 3.7 for Wood Diaphragms, 3.7.2 for concrete diaphragms, 3.7.3 for poured gypsum, and 3.7.4 for metal deck diaphragms (*FEMA 310*, Sections 4.6.2.1 and 4.6.2.3).

C10.3.6.2 Diaphragm/Frame Shear Transfer

Adding collectors and connecting the framing will transfer loads to the collectors. Connections can be provided along the collector length and at the collector-to-frame connection to withstand the calculated forces. See *FEMA 172*, Sections 3.7.5 and 3.7.6 (*FEMA 310*, Sections 4.6.2.2 and 4.6.2.3).

C10.3.6.3 Anchorage for Normal Forces

To account for inadequacies identified by *FEMA 310*, wall anchors can be added. Complications that may result from inadequate anchorage include cross-grain tension in wood ledgers or failure of the diaphragm-to-wall connection due to: (1) insufficient strength, number, or stability of anchors; (2) inadequate embedment of anchors; (3) inadequate development of anchors and straps into the diaphragm; and (4) deformation of anchors and their fasteners that permit diaphragm boundary connection pullout, or cross-grain tension in wood ledgers.

Existing anchors should be tested to determine load capacity and deformation potential, including fastener slip, according to the requirements in *FEMA 310*. Special attention should be given to the testing procedure to maintain a high level of quality control. Additional anchors should be provided as needed to supplement those that fail the test, as well as those needed to meet the *FEMA 310* criteria. The quality of the rehabilitation depends greatly on the quality of the performed tests (*FEMA 310*, Sections 4.6.1.1 through 4.6.1.5).

C10.3.6.4 Girder-Wall Connections

The existing reinforcing must be exposed, and the connection modified as necessary. For out-of-plane loads, the number of column ties can be increased by jacketing the pilaster or, alternatively, by developing a second load path for the out-of-plane forces. Bearing length conditions can be addressed by adding bearing extensions. Frame action in welded connections can be mitigated by adding shear walls (*FEMA 310*, Section 4.6.4.1).

C10.3.6.5 Precast Connections

The connections of chords, ties, and collectors can be upgraded to increase strength and/or ductility, providing alternative load paths for lateral forces. Upgrading can be achieved by such methods as adding confinement ties or increasing embedment. Shear walls can be added to reduce the demand on connections (*FEMA 310*, Section 4.4.1.5.3).

C10.3.6.6 Wall Panels and Cladding

It may be possible to improve the connection between the panels and the framing. If architectural or occupancy conditions warrant, the cladding can be replaced with a new system. The building can be stiffened with the addition of shear walls or braced frames to reduce the drifts in the cladding elements (*FEMA 310*, Section 4.8.4.6).

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

It may be possible to improve the connection between the roof and the framing. If architectural or occupancy conditions warrant, the roof diaphragm can be replaced with a new one. Alternatively, a new diaphragm may be added using rod braces or plywood above or below the existing roof, which remains in place (*FEMA 310*, Section 4.6.5.1).

C10.3.6.8 Mezzanine Connections

Diagonal braces, moment frames, or shear walls can be added at or near the perimeter of the mezzanine where bracing elements are missing, so that a complete and balanced lateral-force-resisting system is provided that meets the requirements of *FEMA 310* (*FEMA 310*, Section 4.3.1.3).

C10.3.7 Foundations and Geologic Hazards

C10.3.7.1 Anchorage to Foundations

For wood walls, expansion anchors or epoxy anchors can be installed by drilling through the wood sill to the concrete foundation at an appropriate spacing of four to six feet on center. Similarly, steel columns and wood posts can be anchored to concrete slabs or footings, using expansion anchors and clip angles. If the concrete or masonry walls and columns lack dowels, a concrete curb can be installed adjacent to the wall or column by drilling dowels and installing anchors into the wall that lap with dowels installed in the slab or footing. However, this curb can cause significant architectural problems. Alternatively, steel angles may be used with drilled anchors. The anchorage of shear wall boundary components can be challenging due to very high concentrated forces (*FEMA 310*, Sections 4.6.3.2 through 4.6.3.9).

C10.3.7.2 Condition of Foundations

All deteriorated and otherwise damaged foundations should be strengthened and repaired using the same materials and style of construction. Some conditions of material deterioration can be mitigated in the field, including patching of spalled concrete. Pest infestation or dry rot of wood piles can be very difficult to correct, and often require full replacement. The deterioration of these elements may have implications that extend beyond seismic safety and must be considered in the rehabilitation (*FEMA 310*, Sections 4.7.2.1 and 4.7.2.2).

C10.3.7.3 Overturning

Existing foundations can be strengthened as needed to resist overturning forces. Spread footings may be enlarged, or additional piles, rock anchors, or piers may be added to deep foundations. It may also be possible to use grade beams or new wall elements to spread out overturning loads over a greater distance. Adding new lateral-load-resisting elements will reduce overturning effects of existing elements (*FEMA 310*, Section 4.7.3.2).

C10.3.7.4 Lateral Loads

As with overturning effects, the correction of lateral load deficiencies in the foundations of existing buildings is expensive and may not be justified by more realistic analysis procedures. For this reason, the Systematic Rehabilitation Method is recommended for these cases. (*FEMA 310*, Sections 4.7.3.1, 4.7.3.3 through 4.7.3.5).

C10.3.7.5 Geologic Site Hazards

Site hazards other than ground shaking should be considered. Rehabilitation of structures subject to life safety hazards from ground failures is impractical unless site hazards can be mitigated to the point where acceptable performance can be achieved. Not all ground failures need necessarily be considered as life safety hazards. For example, in many cases liquefaction beneath a building does not pose a life safety hazard; however, related lateral spreading can result in collapse of buildings with inadequate foundation strength. For this reason, the liquefaction potential and the related consequences should be thoroughly investigated for sites that do not satisfy the *FEMA 310* requirements. Further information on the evaluation of site hazards is provided in Chapter 4 of this standard (*FEMA 310*, Sections 4.7.1.1 through 4.7.1.3).

C10.3.8 Evaluation of Materials and Conditions

C10.3.8.1 General

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

1. Verify existing data (e.g., accuracy of drawings).
2. Develop other needed data (e.g., measure and sketch building if necessary).
3. Verify the vertical and lateral systems.
4. Check the condition of the building.
5. Look for special conditions and anomalies.
6. Address the evaluation statements and goals during the inspection.
7. Perform material tests that are justified through a weighing of the cost of destructive testing and the cost of corrective work.

C10.3.8.2 Condition of Wood

An inspection should be conducted to grade the existing wood and verify physical condition, using techniques from Section C10.3.8.1. Any damage or deterioration and its source must be identified. Wood that is significantly damaged due to splitting, decay, aging, or other phenomena must be removed and replaced. Localized problems can be eliminated by adding new appropriately sized reinforcing components extending beyond the damaged area and connecting to undamaged portions. Additional connectors between components should be provided to correct any discontinuous load paths. It is necessary to verify that any new reinforcing components or connectors will not be exposed to similar deterioration or damage (*FEMA 310*, Section 4.3.3.1).

C10.3.8.3 Overdriven Fasteners

Where visual inspection determines that extensive overdriving of fasteners exists in greater than 20% of the installed connectors, the fasteners and shear panels can generally be repaired through addition of a new same-sized fastener for every two overdriven fasteners. To avoid splitting because of closely spaced nails, it may be necessary to predrill to 90% of the nail shank diameter for installation of new nails. For other conditions, such as cases where the addition of new connectors is not possible or where component damage is suspected, further investigation shall be conducted using the guidance of Section C10.3.8.1 (*FEMA 310*, Section 4.3.3.2).

C10.3.8.4 Condition of Steel

Should visual inspection or testing conducted in accordance with Section C10.3.8.1 reveal the presence of steel component or connection deterioration, further evaluation is needed. The source of the damage shall be identified and mitigated to preserve the remaining structure. In areas of significant deterioration, restoration of the material cross-section can be performed by the addition of plates or other reinforcing techniques. When sizing reinforcements, the design professional shall consider the effects of existing stresses in the original structure, load transfer, and strain compatibility. The demands on the deteriorated steel elements and components may also be reduced through careful addition of bracing or shear wall panels (*FEMA 310*, Section 4.3.3.3).

C10.3.8.5 Condition of Concrete

Should visual inspections or testing conducted in accordance with Section C10.3.8.1 reveal the presence of concrete component or reinforcing steel deterioration, further evaluation is needed. The source of the damage shall be identified and mitigated to preserve the remaining structure. Existing deteriorated material, including reinforcing steel, shall be removed to the limits defined by testing; reinforcing steel in good condition shall be cleaned and left in place for splicing purposes as appropriate. Cracks in otherwise sound material shall be evaluated to determine cause, and repaired as necessary using techniques appropriate to the source and activity level (*FEMA 310*, Section 4.3.3.4).

C10.3.8.6 Post-Tensioning Anchors

Prestressed concrete systems may be adversely affected by cyclic deformations produced by earthquake motion. One rehabilitation process that may be considered is to add stiffness to the system. Another concern for these systems is the adverse effects of tendon corrosion. A thorough visual inspection of prestressed systems shall be performed to verify absence of concrete cracking or spalling, staining from embedded tendon corrosion, or other signs of damage along the tendon spans and at anchorage zones. If degradation is observed or suspected, more detailed evaluations will be required as indicated in Chapter 6. Rehabilitation of these systems, except for local anchorage repair, should be in accordance with the Systematic Rehabilitation provisions in this standard. Professionals with special prestressed concrete construction expertise should also be consulted for further interpretation of damage (*FEMA 310*, Section 4.3.3.5).

C10.3.8.7 Quality of Masonry

Should visual inspections or testing conducted in accordance with Section C10.3.8.1 reveal the presence of masonry components or construction deterioration, further evaluation is needed. Certain damage such as degraded mortar joints or simple cracking may be rehabilitated through repointing or rebuilding. If the wall is repointed, care should be taken to ensure that the new mortar is compatible with the existing masonry units and mortar, and that suitable wetting is performed. The strength of the new mortar is critical to load-carrying capacity and seismic performance. Significant degradation should be treated as specified in Chapter 7 of this standard (*FEMA 310*, Sections 4.3.3.7, 4.3.3.8, 4.3.3.10, 4.3.3.11, and 4.3.3.12).

Table C10-1 W1: Wood Light Frame

Typical Deficiencies
Load Path Redundancy Vertical Irregularities
Shear Walls in Wood Frame Buildings Shear Stress Openings Wall Detailing Cripple Walls Narrow Wood Shear Walls Stucco Shear Walls Gypsum Wallboard or Plaster Shear Walls
Diaphragm Openings Diaphragm Stiffness/Strength Spans Diaphragm Continuity
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Condition of Wood

Table C10-2 W1A: Multistory, Multi-Unit, Wood Frame Construction

Typical Deficiencies
Load Path Redundancy Vertical Irregularities
Shear Walls in Wood Frame Buildings Shear Stress Openings Wall Detailing Cripple Walls Narrow Wood Shear Walls Stucco Shear Walls Gypsum Wallboard or Plaster Shear Walls
Diaphragm Openings Diaphragm Stiffness/Strength Spans Diaphragm Continuity
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Condition of Wood

Table C10-3 W2: Wood, Commercial, and Industrial

Typical Deficiencies
Load Path Redundancy Vertical Irregularities
Shear Walls in Wood Frame Buildings Shear Stress Openings Wall Detailing Cripple Walls Narrow Wood Shear Walls Stucco Shear Walls Gypsum Wallboard or Plaster Shear Walls
Diaphragm Openings Diaphragm Stiffness/Strength Sheathing Unblocked Diaphragms Spans Span-to-Depth Ratio Diaphragm Continuity Chord Continuity
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Condition of Wood

Table C10-4 S1 and S1A: Steel Moment Frames with Stiff or Flexible Diaphragms

Typical Deficiencies
Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Adjacent Buildings
Lateral Load Path at Pile Caps
Steel Moment Frames
Drift Check
Frame Concerns
Strong Column-Weak Beam
Connections
Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer
Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards
Condition of Steel

Table C10-5 S2 and S2A: Steel Braced Frames with Stiff or Flexible Diaphragms

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

Table C10-6 S3: Steel Light Frames

Typical Deficiencies
Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Steel Moment Frames
Frame Concerns
Masonry Shear Walls
Infill Walls
Steel Braced Frames
Stress Level
Braced Frame Connections
Re-entrant Corners
Diaphragm Openings
Diaphragm/Frame Shear Transfer
Wall Panels and Cladding
Light Gage Metal, Plastic, or Cementitious Roof Panels
Anchorage to Foundations
Condition of Foundations
Geologic Site Hazards
Condition of Steel

Table C10-7 S4: Steel Frames with Concrete Shear Walls

Typical Deficiencies
Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Lateral Load Path at Pile Caps
Cast-in-Place Concrete Shear Walls
Shear Stress
Overturning
Coupling Beams
Boundary Component Detailing
Wall Reinforcement
Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer
Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards
Condition of Steel
Condition of Concrete

Table C10-8 S5, S5A: Steel Frames with Infill Masonry Shear Walls and Stiff or Flexible Diaphragms

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

Table C10-9 C1: Concrete Moment Frames

Typical Deficiencies
Load Path
Redundancy
Vertical Irregularities
Plan Irregularities
Adjacent Buildings
Lateral Load Path at Pile Caps
Deflection Compatibility
Concrete Moment Frames
Quick Checks, Frame and Nonductile Detail Concerns
Precast Moment Frame Concerns
Frames Not Part of the Lateral Force Resisting System
Short “Captive” Columns
Re-entrant Corners
Diaphragm Openings
Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer
Precast Connections
Anchorage to Foundations
Condition of Foundations
Overturning
Lateral Loads
Geologic Site Hazards
Condition of Concrete

Table C10-10 C2, C2A: Concrete Shear Walls with Stiff or Flexible Diaphragms

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

Table C10-11 C3, C3A: Concrete Frames with Infill Masonry Shear Walls and Stiff or Flexible Diaphragms

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

Table C10-12 PC1: Precast/Tilt-up Concrete Shear Walls with Flexible Diaphragms

Typical Deficiencies

Load Path
  Redundancy
  Vertical Irregularities
  Plan Irregularities
  Deflection Compatibility

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

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

Diaphragm/Wall Shear Transfer
  Anchorage for Normal Forces
  Girder/Wall Connections
  Stiffness of Wall Anchors

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

Condition of Concrete

Table C10-13 PC1A: Precast/Tilt-up Concrete Shear Walls with Stiff Diaphragms

Typical Deficiencies

Load Path
  Redundancy
  Vertical Irregularities
  Plan Irregularities

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

Re-entrant Corners
  Diaphragm Openings
  Diaphragm Stiffness/Strength

Diaphragm/Wall Shear Transfer
  Anchorage for Normal Forces
  Girder/Wall Connections

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

Condition of Concrete

Table C10-14 PC2: Precast Concrete Frames with Shear Walls

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Lateral Load Path at Pile Caps Deflection Compatibility
Concrete Moment Frames Precast Moment Frame Concerns
Cast-in-Place Concrete Shear Walls Shear Stress Overturning Coupling Beams Boundary Component Detailing Wall Reinforcement
Re-entrant Corners Cross Ties Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Girder/Wall Connections Precast Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table C10-15 PC2A: Precast Concrete Frames Without Shear Walls

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings Lateral Load Path at Pile Caps Deflection Compatibility
Concrete Moment Frames Precast Moment Frame Concerns Frames Not Part of the Lateral-Force-Resisting System Short Captive Columns
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Frame Shear Transfer Precast Connections
Anchorage to Foundations Condition of Foundations Overturning Lateral Loads Geologic Site Hazards
Condition of Concrete

Table C10-16 RM1: Reinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities
Masonry Shear Walls Reinforcing in Masonry Walls Shear Stress Reinforcing at Openings
Re-entrant Corners Cross Ties Diaphragm Openings Diaphragm Stiffness/Strength Sheathing Unblocked Diaphragms Span/Depth Ratio
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table C10-17 RM2: Reinforced Masonry Bearing Wall Buildings with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities
Masonry Shear Walls Reinforcing in Masonry Walls Shear Stress Reinforcing at Openings
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table C10-18 URM: Unreinforced Masonry Bearing Wall Buildings with Flexible Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings
Masonry Shear Walls Unreinforced Masonry Shear Walls Properties, Solid Walls
Re-entrant Corners Cross Ties Diaphragm Openings Diaphragm Stiffness/Strength Sheathing Unblocked Diaphragms Span/Depth Ratio
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces Stiffness of Wall Anchors
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table C10-19 URMA: Unreinforced Masonry Bearing Walls Buildings with Stiff Diaphragms

Typical Deficiencies
Load Path Redundancy Vertical Irregularities Plan Irregularities Adjacent Buildings
Masonry Shear Walls Unreinforced Masonry Shear Walls Properties, Solid Walls
Re-entrant Corners Diaphragm Openings Diaphragm Stiffness/Strength
Diaphragm/Wall Shear Transfer Anchorage for Normal Forces
Anchorage to Foundations Condition of Foundations Geologic Site Hazards
Quality of Masonry

Table C10-20 Cross-Reference Between this Standard, FEMA 310 and FEMA 178 Deficiency Reference Numbers

FEMA 310		FEMA 178	FEMA 356 Standard	
Section	**Section Heading**	**Section**	**Section**	**Section Heading**
4.3.1.1	**Load Path**	**3.1**	C10.3.1.1	Load Path
4.3.1.2	**Adjacent Buildings**	**3.4**	C10.3.1.5	Adjacent Buildings
4.3.1.3	Mezzanines	—	C10.3.6.8	Mezzanine Connections
4.3.2	**Configuration**	**3.3**	C10.3.1	Building Systems
4.3.2.1	Weak Story	3.3.1	C10.3.1.3	Vertical Irregularities
4.3.2.2	Soft Story	3.3.2	C10.3.1.3	Vertical Irregularities
4.3.2.3	Geometry	3.3.3	C10.3.1.3	Vertical Irregularities
4.3.2.4	Vertical Discontinuities	3.3.5	C10.3.1.3	Vertical Irregularities
4.3.2.5	Mass	3.3.4	C10.3.1.3	Vertical Irregularities
4.3.2.6	Torsion	3.3.6	C10.3.1.4	Plan Irregularities
4.3.3	**Condition of Materials**	**3.5**	C10.3.8	Evaluation of Materials and Conditions
4.3.3.1	Deterioration of Wood	3.5.1	C10.3.8.2	Condition of Wood
4.3.3.2	Wood Structural Panel Shear Wall Fasteners	3.5.2	C10.3.8.3	Overdriven Fasteners
4.3.3.3	Deterioration of Steel	3.5.3	C10.3.8.4	Condition of Steel
4.3.3.4	Deterioration of Concrete	3.5.4	C10.3.8.5	Condition of Concrete
4.3.3.5	Post-Tensioning Anchors	3.5.5	C10.3.8.6	Post-Tensioning Anchors
4.3.3.6	Precast Concrete Walls	3.5.8	C10.3.8.5	Condition of Concrete
4.3.3.7	Masonry Units	3.5.10	C10.3.8.7	Quality of Masonry
4.3.3.8	Masonry Joints	3.5.9	C10.3.8.7	Quality of Masonry
4.3.3.9	Concrete Wall Cracks	3.5.6	C10.3.8.5	Condition of Concrete
4.3.3.10	Reinforced Masonry Wall Cracks	—	C10.3.8.7	Quality of Masonry
4.3.3.11	Unreinforced Masonry Wall Cracks	—	C10.3.8.7	Quality of Masonry
4.3.3.12	Cracks in Infill Walls	3.5.11	C10.3.8.7	Quality of Masonry
4.3.3.13	Cracks in Boundary Columns	3.5.7	C10.3.8.5	Condition of Concrete
4.4.1.1.1	Redundancy	**3.2**	C10.3.1.2	Redundancy
4.4.1.2	**Frames with Infill Walls**	**4.1**	C10.3.3	Shear Walls
4.4.1.2.1	Interfering Walls	4.1.4	C10.3.3.3.6	Infill Walls
4.4.1.3	**Steel Moment Frames**	**4.2**	C10.3.2.1	Steel Moment Frames
4.4.1.3.1	Drift Check	4.2.1	C10.3.2.1.1	Drift
4.4.1.3.2	Axial Stress Check	—	C10.3.2.1.2	Frames
4.4.1.3.3	Moment-Resisting Connections	4.2.4	C10.3.2.1.4	Connections
4.4.1.3.4	Panel Zones	4.2.6	C10.3.2.1.4	Connections
4.4.1.3.5	Column Splices	4.2.5	C10.3.2.1.2	Frames
4.4.1.3.6	Strong Column-Weak Beam	4.2.8	C10.3.2.1.3	Strong Column-Weak Beam
4.4.1.3.7	Compact Members	4.2.2	C10.3.2.1.2	Frames
4.4.1.3.8	Beam Penetration	4.2.3	C10.3.2.1.2	Frames

Table C10-20 Cross-Reference Between this Standard, FEMA 310 and FEMA 178 Deficiency Reference Numbers (continued)

FEMA 310		FEMA 178	FEMA 356 Standard	
4.4.1.3.9	Girder Flange Continuity	4.2.7	C10.3.2.1.4	Connections
4.4.1.3.10	Out-of-Plane Bracing	4.2.9	C10.3.2.1.2	Frames
4.4.1.3.11	Bottom Flange Bracing	—	C10.3.2.1.2	Frames
4.4.1.4	**Concrete Moment Frames**	**4.3**	C10.3.2.2	Concrete Moment Frames
4.4.1.4.1	Shear Stress Check	4.3.1	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.2	Axial Stress Check	—	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.3	Flat Slab Frames	4.3.15	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.4	Prestressed Frame Elements	4.3.3	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.5	Short Captive Columns	—	C10.3.2.3.2	Short Captive Columns
4.4.1.4.6	No Shear Failures	4.3.5	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.7	Strong Column-Weak Beam	4.3.6	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.8	Beam Bars	4.3.10	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.9	Column-Bar Splices	4.3.9	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.10	Beam-Bar Splices	4.3.11	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.11	Column-Tie Spacing	4.3.8	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.12	Stirrup Spacing	4.3.12	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.13	Joint Reinforcing	4.3.14	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.14	Joint Eccentricity	4.3.4	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.4.15	Stirrup and Tie Hooks	4.3.7	C10.3.2.2.1	Frame and Nonductile Detail Concerns
		4.3.2	C10.3.2.2.1	Frame and Nonductile Detail Concerns
		4.3.13	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.1.5	**Precast Moment Frames**	**4.4**	C10.3.2.2.2	Precast Moment Frames
4.4.1.5.1	Precast Connection Check	—	C10.3.2.2.2	Precast Moment Frames
4.4.1.5.2	Precast Frames	4.4.1	C10.3.2.2.2	Precast Moment Frames
4.4.1.5.3	Precast Connections	4.4.2	C10.3.6.5	Precast Connections
4.4.1.6	**Frames Not Part of the Lateral-Force-Resisting System**	**4.5**	C10.3.2.3	Frames Not Part of the Lateral-Force-Resisting System
4.4.1.6.1	Complete Frames	4.5.1	C10.3.2.3.1	Complete Frames
4.4.1.6.2	Deflection Compatibility	—	C10.3.1.7	Deflection Compatibility
4.4.1.6.3	Flat Slabs	—	C10.3.2.2.1	Frame and Nonductile Detail Concerns
4.4.2.2	**Concrete Shear Walls**	**5.1**	C10.3.3	Shear Walls
4.4.2.1.1	Redundancy	**3.2**	C10.3.1.2	Redundancy
4.4.2.2.1	Shear Stress Check	5.1.1	C10.3.3.1.1	Shearing Stress
4.4.2.2.2	Reinforcing Steel	5.1.7	C10.3.3.1.5	Wall Reinforcement
4.4.2.2.3	Coupling Beams	5.1.3	C10.3.3.1.3	Coupling Beams
4.4.2.2.4	Overturning	5.1.2	C10.3.3.1.2	Overturning
4.4.2.2.5	Confinement Reinforcing	5.1.6	C10.3.3.1.4	Boundary Component Detailing
4.4.2.2.6	Reinforcing at Openings	5.1.8	C10.3.3.1.5	Wall Reinforcement

Table C10-20 Cross-Reference Between this Standard, FEMA 310 and FEMA 178 Deficiency Reference Numbers (continued)

FEMA 310		FEMA 178	FEMA 356 Standard	
4.4.2.2.7	Wall Thickness	—	C10.3.3.1.1	Shearing Stress
4.4.2.2.8	Wall Connections	5.1.5	C10.3.3.1.4	Boundary Component Detailing
4.4.2.2.9	Column Splices	5.1.4	C10.3.3.1.4	Boundary Component Detailing
4.4.2.3	**Precast Concrete Shear Walls**	**5.2**	C10.3.3.2	Precast Concrete Shear Walls
4.4.2.3.1	Shear Stress Check	—	C10.3.3.1.1	Shearing Stress
4.4.2.3.2	Reinforcing Steel	—	C10.3.3.1.5	Wall Reinforcement
4.4.2.3.3	Wall Openings	5.2.2	C10.3.3.2.2	Wall Openings
4.4.2.3.4	Corner Openings	5.2.3	C10.3.3.2.3	Collectors
4.4.2.3.5	Panel-to-Panel Connections	5.2.1	C10.3.3.2.1	Panel-to-Panel Connections
4.4.2.3.6	Wall Thickness	—	C10.3.3.1.1	Shearing Stress
4.4.2.4	**Reinforced Masonry Shear Walls**	**5.3**	C10.3.3.3	Masonry Shear Walls
4.4.2.4.1	Shear Stress Check	5.3.1	C10.3.3.3.2	Shearing Stress
4.4.2.4.2	Reinforcing Steel	5.3.2	C10.3.3.3.1	Reinforcing in Masonry Walls
4.4.2.4.3	Reinforcing at Openings	5.3.3	C10.3.3.3.3	Reinforcing at Openings
4.4.2.4.4	Proportions	—	C10.3.3.3.5	Proportions of Solid Walls
4.4.2.5	**Unreinforced Masonry Shear Walls**	**5.4**	C10.3.3.3.4	Unreinforced Masonry Shear Walls
4.4.2.5.1	Shear Stress Check	5.4.1	C10.3.3.3.4	Unreinforced Masonry Shear Walls
4.4.2.5.2	Proportions	—	C10.3.3.3.5	Proportions of Solid Walls
4.4.2.5.3	Masonry Lay-up	5.4.2	C10.3.3.3.4	Unreinforced Masonry Shear Walls
4.4.2.6	**Infill Walls in Frames**	**5.5**	C10.3.3.3.6	Infill Walls
4.4.2.6.1	Wall Connections	5.5.4	C10.3.3.3.6	Infill Walls
4.4.2.6.2	Proportions	5.5.1	C10.3.3.3.5	Proportions of Solid Walls
4.4.2.6.3	Solid Walls	5.5.2	C10.3.3.3.5	Proportions of Solid Walls
4.4.2.6.4	Infill Walls	5.5.3	C10.3.3.3.6	Infill Walls
4.4.2.7	**Walls in Wood Frame Buildings**	**5.6**	C10.3.3.4	Shear Walls in Wood Frame Buildings
4.4.2.7.1	Shear Stress Check	5.6.1	C10.3.3.4.1	Shear Stress
4.4.2.7.2	Stucco (Exterior Plaster) Shear Walls	—	C10.3.3.4.6	Stucco Shear Walls
4.4.2.7.3	Gypsum Wallboard or Plaster Shear Walls	—	C10.3.3.4.7	Gypsum Wallboard or Plaster Shear Walls
4.4.2.7.4	Narrow Wood Shear Walls	—	C10.3.3.4.5	Narrow Wood Shear Walls
4.4.2.7.5	Walls Connected Through Floors	—	C10.3.3.4.3	Wall Detailing
4.4.2.7.6	Hillside Site	—	C10.3.3.4.4	Cripple Walls
4.4.2.7.7	Cripple Walls	5.6.4	C10.3.3.4.4	Cripple Walls
4.4.2.7.8	Openings	5.6.2	C10.3.3.4.2	Openings
4.4.2.7.9	Hold-Down Anchors	5.6.3	C10.3.3.4.3	Wall Detailing

Table C10-20 Cross-Reference Between this Standard, FEMA 310 and FEMA 178 Deficiency Reference Numbers (continued)

FEMA 310		**FEMA 178**	**FEMA 356 Standard**	
4.4.3.1	**Braced Frames**	**6.1**	C10.3.4	Steel Braced Frames
4.4.3.1.1	Redundancy	**3.2**	C10.3.1.2	Redundancy
4.4.3.1.2	Axial Stress Check	6.1.1	C10.3.4.1	System Concerns
4.4.3.1.3	Slenderness of Diagonals	6.1.2	C10.3.4.2	Stiffness of Diagonals
4.4.3.1.4	Connection Strength	6.1.6	C10.3.4.4	Braced Frame Connections
4.4.3.1.5	Column Splices	6.1.7	C10.3.4.4	Braced Frame Connections
4.4.3.1.6	Out-of-Plane Bracing	—	C10.3.4.1	System Concerns
4.4.3.2	**Concentrically Braced Frames**	—	C10.3.4	Steel Braced Frames
4.4.3.2.1	K-Bracing	—	C10.3.4.3	Chevron or K-Bracing
4.4.3.2.2	Tension-Only Braces	6.1.3	C10.3.4.2	Stiffness of Diagonals
4.4.3.2.3	Chevron Bracing	6.1.4	C10.3.4.3	Chevron or K-Bracing
4.4.3.2.4	Concentric Joints	6.1.5	C10.3.4.4	Braced Frame Connections
4.4.3.3	**Eccentrically Braced Frames**	—	C10.3.4.1	System Concerns
4.5	**Diaphragms**	**7.1**	C10.3.5	Diaphragms
4.5.1.1	Diaphragm Continuity	7.2.5	C10.3.5.4.5	Diaphragm Continuity
4.5.1.2	Cross Ties	7.1.2	C10.3.5.2	Cross Ties
4.5.1.3	Roof Chord Continuity	7.2.6	C10.3.5.4.6	Chord Continuity
4.5.1.4	Openings at Shear Walls	7.1.4	C10.3.5.3	Diaphragm Openings
4.5.1.5	Openings at Braced Frames	7.1.5	C10.3.5.3	Diaphragm Openings
4.5.1.6	Openings at Exterior Masonry Shear Walls		7.1.6	C10.3.5.3
4.5.1.7	Plan Irregularities	7.1.1	C10.3.5.1	Re-entrant Corners
4.5.1.8	Reinforcing at Openings	7.1.3	C10.3.5.3	Diaphragm Openings
4.5.2	**Wood Diaphragms**	**7.2**	C10.3.5	Diaphragms
4.5.2.1	Straight Sheathing	7.2.1	C10.3.5.4.1	Board Sheathing
4.5.2.2	Spans	7.2.2	C10.3.5.4.3	Spans
4.5.2.3	Unblocked Diaphragms	7.2.3	C10.3.5.4.2	Unblocked Diaphragms
		7.2.4	C10.3.5.4.4	Span-to-Depth Ratio
4.5.3	Metal Deck Diaphragms	—	C10.3.5	Diaphragms
4.5.3.1	Non-Concrete Diaphragms	—	C10.3.5	Diaphragms
4.5.4	**Concrete Diaphragms**	—	C10.3.5	Diaphragms
4.5.5	**Precast Concrete Diaphragms**	—	C10.3.5	Diaphragms
4.5.5.1	Topping Slab	—	C10.3.5	Diaphragms
4.5.6	**Horizontal Bracing**	—	C10.3.5	Diaphragms
4.5.7.1	**Other Diaphragms**	—	C10.3.5	Diaphragms

Table C10-20 Cross-Reference Between this Standard, FEMA 310 and FEMA 178 Deficiency Reference Numbers (continued)

FEMA 310		FEMA 178	FEMA 356 Standard	
4.6.1	**Anchorage for Normal Forces**	**8.2**	C10.3.6.3	Anchorage for Normal Forces
4.6.1.1	Wall Anchorage	8.2.2	C10.3.6.3	Anchorage for Normal Forces
4.6.1.2	Wood Ledgers	8.2.1	C10.3.6.3	Anchorage for Normal Forces
4.6.1.3	Anchor Spacing	8.2.4	C10.3.6.3	Anchorage for Normal Forces
4.6.1.4	Precast Panel Connections	8.2.5	C10.3.6.3	Anchorage for Normal Forces
4.6.1.5	Stiffness of Wall Anchors	8.2.3	C10.3.6.3	Anchorage for Normal Forces
4.6.2	**Shear Transfer**	**8.3**	C10.3.6	Connections
4.6.2.1	Transfer to Shear Walls	8.3.1	C10.3.6.1	Diaphragm/Wall Shear Transfer
4.6.2.2	Transfer to Steel Frames	8.3.2	C10.3.6.2	Diaphragm/Frame Shear Transfer
4.6.2.3	Topping Slab to Walls and Frames	8.3.3	C10.3.6.1	Diaphragm/Wall Shear Transfer
4.6.3	**Vertical Components**	**8.4**	C10.3.7.1	Anchorage to Foundations
4.6.3.1	Steel Columns	8.4.1	C10.3.7.1	Anchorage to Foundations
4.6.3.2	Concrete Columns	8.4.2	C10.3.7.1	Anchorage to Foundations
4.6.3.3	Wood Posts	8.4.3	C10.3.7.1	Anchorage to Foundations
4.6.3.4	Wood Sills	8.4.7	C10.3.7.1	Anchorage to Foundations
4.6.3.5	Foundation Dowels	8.4.4	C10.3.7.1	Anchorage to Foundations
4.6.3.6	Shear-Wall-Boundary Columns	8.4.5	C10.3.7.1	Anchorage to Foundations
4.6.3.7	Precast Wall Panels	—	C10.3.7.1	Anchorage to Foundations
4.6.3.8	Wall Panels	8.4.6	C10.3.7.1	Anchorage to Foundations
4.6.3.9	Wood Sill Bolts	8.4.7	C10.3.7.1	Anchorage to Foundations
4.6.3.10	Lateral Load Path at Pile Caps	—	C10.3.7.4	Lateral Loads
4.6.4	**Interconnection of Elements**	**8.5**	C10.3.6	Connections
4.6.4.1	Girder/Column Connection		C10.3.6.4	Girder-Wall Connections
4.6.4.2	Girders	8.5.1	C10.3.6.4	Girder-Wall Connections
4.6.4.3	Corbel Bearing	8.5.2	C10.3.6.4	Girder-Wall Connections
4.6.4.4	Corbel Connections	8.5.3	C10.3.6.4	Girder-Wall Connections
4.6.5	**Panel Connections**	—	C10.3.6	Connections
4.6.5.1	Roof Panels	8.6.1	C10.3.6.7	Light Gage Metal, Plastic, or Cementitious Roof Panels
4.6.5.2	Wall Panels	8.6.2	C10.3.6.6	Wall Panels and Cladding
4.6.5.3	Roof Panel Connections	—	C10.3.6.7	Light Gage Metal, Plastic, or Cementitious Roof Panels
4.7.1	**Geologic Site Hazards**	**9.3**	C10.3.7	Foundations and Geologic Hazards
4.7.1.1	Liquefaction	9.3.1	C10.3.7.5	Geologic Site Hazards
4.7.1.2	Slope Failure	9.3.2	C10.3.7.5	Geologic Site Hazards
4.7.1.3	Surface Fault Rupture	9.3.3	C10.3.7.5	Geologic Site Hazards

Table C10-20 Cross-Reference Between this Standard, FEMA 310 and FEMA 178 Deficiency Reference Numbers (continued)

FEMA 310		FEMA 178	FEMA 356 Standard	
4.7.2	**Condition of Foundations**	**9.1**	C10.3.7.2	Condition of Foundations
4.7.2.1	Foundation Performance	9.1.1	C10.3.7.2	Condition of Foundations
4.7.2.2	Deterioration	9.1.2	C10.3.7.2	Condition of Foundations
4.7.3	**Capacity of Foundations**	**9.2**	C10.3.7	Foundations and Geologic Hazards
4.7.3.1	Pole Foundations	9.2.4	C10.3.7.4	Lateral Loads
4.7.3.2	Overturning	9.2.1	C10.3.7.3	Overturning
4.7.3.3	Ties Between Foundation Elements	9.2.2	C10.3.7.4	Lateral Loads
4.7.3.4	Deep Foundations	9.2.3	C10.3.7.4	Lateral Loads
4.7.3.5	Sloping Sites	9.2.5	C10.3.7.4	Lateral Loads