8. Wood and Light Metal Framing

8.1 Scope

This chapter sets forth requirements for the Systematic Rehabilitation of wood and light metal frame buildings or wood and light metal framed lateral-force-resisting elements within a building. The requirements of this chapter shall apply to existing wood and light metal frame components of a building system, rehabilitated wood and light metal frame components of a building system, and new wood and light metal frame components that are added to an existing building system.

Sections 8.2 and 8.3 specify data collection procedures for obtaining material properties and performing condition assessments. Section 8.4 specifies general assumptions and requirements. Sections 8.5 and 8.6 provide modeling procedures, component strengths, acceptance criteria, and rehabilitation measures for wood and light frame shear walls and wood diaphragms. Section 8.7 specifies requirements for wood foundations. Section 8.8 specifies requirements for other wood components including, but not limited to, knee-braced frames, rod-braced frames, braced horizontal diaphragms, and components supporting discontinuous shear walls.

C8.1 Scope

The Linear Static Procedure (LSP) presented in Chapter 3 is most often used for the systematic analysis of wood frame buildings; however, properties of the idealized inelastic performance of various elements and connections are included so that nonlinear procedures can be used if desired.

A general history of the development of wood framing methods is presented in Section 8.2, along with the features likely to be found in buildings of different ages. The evaluation and assessment of various structural elements of wood frame buildings is found in Section 8.3. For a description and discussion of connections between the various components and elements, see Section 8.3.2.2.2. Properties of shear walls are described in Section 8.5, along with various rehabilitation or strengthening methods. Horizontal floor and roof diaphragms are discussed in Section 8.6, which also covers engineering properties and methods of upgrading or strengthening the elements. Wood foundations and pole structures are addressed in Section 8.7. For additional information regarding foundations, see Chapter 4.

8.2 Historical Information

Available construction documents and as-built information shall be obtained as specified in Section 2.2. Use of material properties based on historical information for use as default values shall be as specified in Section 8.3.2.5. Other approved values of material properties shall be permitted if based on available historical information for a particular type of wood frame construction, prevailing codes, and assessment of existing condition.

C8.2 Historical Information

Wood frame construction has evolved over the years; wood is the primary building material of most residential and small commercial structures in the United States. It has often been used for the framing of roofs and floors, and in combination with other materials.

Establishing the age and recognizing the location of a building can be helpful in determining what types of lateral-force-resisting systems may be present.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic buildings in order to preserve their unique characteristics.

Based on the approximate age of a building, various assumptions can be made about the design and features of construction. Older wood frame structures that predate building codes and standards usually do not have the types of elements considered essential for predictable seismic performance. These elements will generally have to be added, or the existing elements upgraded by the addition of lateral-load-resisting components to the existing structure in order to obtain predictable performance.

If the age of a building is known, the code in effect at the time of construction and the general quality of the construction usual for the time can be helpful in evaluating an existing building. The level of maintenance of a building may be a useful guide in determining the structure's capacity to resist loads.

The earliest wood frame buildings in the U.S. were built with post and beam or frame construction adopted from Europe and the British Isles. This was followed by the development of balloon framing in about 1830 in the Midwest, which spread to the East Coast by the 1860s. This, in turn, was followed by the development of western or platform framing shortly after the turn of the century. Platform framing is the system currently in use for multistory construction.

Drywall or wallboard was first introduced in about 1920; however, its use was not widespread until after World War II, when gypsum lath (button board) also came into extensive use as a replacement for wood lath.

With the exception of public schools in high seismic areas, modern wood frame structures detailed to resist seismic loads were generally not built prior to 1934. For most wood frame structures, either general seismic provisions were not provided—or the codes that included them were not enforced—until the mid-1950s or later, even in the most active seismic areas. This time frame varies somewhat depending on local conditions and practice.

Buildings constructed after 1970 in high seismic areas usually included a well-defined lateral-force-resisting system as a part of the design. However, site inspections and code enforcement varied greatly. Thus, the inclusion of various features and details on the plans does not necessarily mean that they are in place or fully effective. Verification is needed to ensure that good construction practices were followed.

Until about 1950, wood residential buildings were frequently constructed on raised foundations and in some cases included a short stud wall, called a "cripple wall," between the foundation and the first floor framing. This occurs on both balloon-framed and platform-framed buildings. There may be an extra demand on these cripple walls because most interior partition walls do not continue to the foundation. Special attention is required in these situations. Adequate bracing must be provided for cripple walls as well as the attachment of the sill plate to the foundation.

In more recent times, light gage metal studs and joists have been used in lieu of wood framing for some structures. Lateral-load resistance is either provided by metal straps attached to the studs and top and bottom tracks, or by structural panels attached with sheet metal screws to the studs and the top and bottom track in a manner similar to that of wood construction. The metal studs and joists vary in size, gage, and configuration, depending on the manufacturer and the loading conditions.

For systems using structural panels for bracing, see Section 8.5 for analysis and acceptance criteria. For the all-metal systems using steel strap braces, see Chapter 5 for guidance.

8.3 Material Properties and Condition Assessment

8.3.1 General

Mechanical properties for wood and light metal framing materials, components, and assemblies shall be based on available construction documents and as-built conditions for the particular structure. When such information fails to provide adequate information to quantify material properties, capacities of assemblies, or condition of the structure, such information shall be supplemented by materials tests, mock-up tests of assemblies, and assessments of existing conditions as required in Section 2.2.6.

Material properties of existing wood and light metal framing components and assemblies shall be determined in accordance with Section 8.3.2. A condition assessment shall be conducted in accordance with Section 8.3.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor, κ , as specified in Section 8.3.4.

C8.3.1 General

Various grades and species of wood have been used in a cut dimension form, combined with other structural materials (e.g., steel/wood elements), or in multiple layers of construction (e.g., glue-laminated wood components). Wood materials have also been manufactured into hardboard, plywood, and particleboard products, which may have structural or nonstructural functions in construction. The condition of the in-place wood materials will greatly influence the future behavior of wood components in the building system.

Quantification of in-place material properties and verification of existing system configuration and condition are necessary to properly analyze the building. The focus of this effort shall be given to the primary vertical- and lateral-load-resisting elements and components thereof. These primary components may be identified through initial analysis and application of loads to the building model.

The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction documents and as-built records, the quality of materials used and construction performed, and physical condition. A specific problem with wood construction is that structural wood components are often covered with other components, materials, or finishes; in addition, their behavior is influenced by past loading history. Knowledge of the properties and grades of material used in original component/connection fabrication is invaluable, and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction, including design calculations.

Connection configuration also has a very important influence on response to applied loads and motions. A large number of connector types exist, the most prevalent being nails and through bolts. However, more recent construction has included metal straps and hangers, clip angles, and truss plates. An understanding of connector configuration and mechanical properties must be gained to properly analyze the anticipated performance of the building.

8.3.2 Properties of In-Place Materials and Components

8.3.2.1 Material Properties

8.3.2.1.1 General

The species and grade of wood shall be established by one of the following methods:

1. Construction documents shall be reviewed.
2. An inspection shall be conducted.
3. Samples shall be tested in a laboratory.

When materials testing is required by Section 2.2.6, grading shall be performed using the *ASTM D245-00* grading methodology or an approved grading handbook for the assumed wood species and application. Samples shall be taken from regions of reduced stress and tested in accordance with Section 8.3.2.3. The properties of adhesives used in the fabrication of laminated products other than plywood shall be evaluated.

Use of default properties for wood and light metal frame shear walls, wood diaphragms, components, and connectors shall be permitted in accordance with Section 8.3.2.5. For materials comprising individual components, the use of default properties shall be permitted where the species and grade of wood have been determined. Use of default properties for connectors shall be permitted where the species of the connected members has been determined.

8.3.2.1.2 Nominal or Specified Properties

Use of nominal material properties or properties specified in construction documents to compute expected and lower bound material properties shall be permitted in accordance with Section 8.3.2.5.

C8.3.2.1.2 Nominal or Specified Properties

Actions associated with wood and light metal framing components generally are deformation-controlled; thus, expected strength material properties will be used most often. Lower-bound values will be used with components supporting discontinuous shear walls, bodies of connections, and axial compression of individual timber frame components, which are force-controlled. Material properties listed in this chapter are expected strength values. If lower-bound material properties are needed, they should be taken as mean minus one standard deviation values, or adjusted from expected strength values in accordance with Section 8.3.2.5.

8.3.2.2 Component Properties

8.3.2.2.1 Elements

The following component properties shall be determined in accordance with Section 8.3.3:

1. Cross-sectional shape and physical dimensions of the primary components and overall configuration of the structure.
2. Configuration of connections, size and thickness of connected materials, and continuity of load path.
3. Modifications to individual components or overall configuration of the structure.
4. Location and dimension of braced frames and shear walls; type, grade, nail size, and spacing of hold-downs and drag/strut members.
5. Current physical condition of components and extent of any deterioration present.

C8.3.2.2.1 Elements

Structural elements of the lateral-force-resisting system comprise primary and secondary components, which collectively define element strength and resistance to deformation. Behavior of the components—including shear walls, beams, diaphragms, columns, and braces—is dictated by physical properties such as area; material grade; thickness, depth, and slenderness ratios; lateral torsional buckling resistance; and connection details.

The actual physical dimensions should be measured; e.g., 2" x 4" stud dimensions are generally 1.5" x 3.5". Connected members include plywood, bracing, stiffeners, chords, sills, struts, and hold-down posts. Modifications to members include notching, holes, splits, and cracks. The presence of decay or deformation should be noted.

These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity-) and lateral-load-carrying elements and systems, and their critical components and connections. Site inspections should be conducted to verify conditions and to assure that remodeling has not changed the original design concept. In the absence of a complete set of building drawings, the design professional must thoroughly inspect the building to identify these elements, systems, and components as indicated in Section 8.3.3. Where reliable record drawings do not exist, an as-built set of plans for the building must be created.

8.3.2.2.2 Connections

Details of the following connections shall be determined or verified in accordance with Section 8.3.3:

1. Connections between horizontal diaphragms and shear walls and braced frames.
2. Size and character of all drag ties and struts, including splice connections.
3. Connections at splices in chord members of horizontal diaphragms.

4. Connections of horizontal diaphragms to exterior or interior concrete or masonry walls for both in-plane and out-of-plane loads.

5. Connections of cross tie members for concrete or masonry buildings.

6. Connections of shear walls to foundations for transfer of shear and overturning forces.

7. Method of through-floor transfer of wall shear and overturning forces in multistory buildings.

C8.3.2.2.2 Connections

The method of connecting the various elements of the structural system is critical to its performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions.

8.3.2.3 Test Methods to Quantify Material Properties

The stiffness and strength of wood and light metal framing components and assemblies shall be established through in situ testing or observation, or mock-up testing of assemblies in accordance with Section 2.8, unless default values are used in accordance with Section 8.3.2.5. The number of tests required shall be based on Section 8.3.2.4. Expected material properties shall be based on mean values of tests. Lower bound material properties shall be based on mean values of tests minus one standard deviation.

C8.3.2.3 Test Methods to Quantify Material Properties

To obtain the desired in-place mechanical properties of materials and components, including expected strength, it is often necessary to use proven destructive and nondestructive testing methods.

Of greatest interest to wood building system performance are the expected orthotropic strengths of the installed materials for anticipated actions (e.g., flexure). Past research and accumulation of data by industry groups have led to published mechanical properties for most wood types and sizes (e.g., dimensional solid-sawn lumber, and glue-laminated or “glulam” beams). Section 8.3.2.5 addresses these established default strengths and distortion properties. This information may be used, together with tests from recovered samples or observation, to establish the expected properties for use in component strength and deformation analyses. Where possible, the load history for the building shall be assessed for possible influence on component strength and deformation properties.

To quantify material properties and analyze the performance of archaic wood construction, shear walls, and diaphragm action, more extensive sampling and testing may be necessary. This testing should include further evaluation of load history and moisture effects on properties, and an examination of wall and diaphragm continuity and the suitability of in-place connectors.

Where it is desired to use an existing assembly and little or no information about its performance is available, a cyclic load test of a mock-up of the existing structural elements can be used to determine the performance of various assemblies, connections, and load transfer conditions. See Section 2.8 for an explanation of the backbone curve and the establishment of alternative modeling parameters.

8.3.2.4 Minimum Number of Tests

8.3.2.4.1 Usual Testing

The minimum number of tests to quantify expected strength material properties for usual data collection shall be based on the following criteria:

1. If design drawings containing material property information are available, at least one location for each story shall be randomly verified by observing grade stamps or by compliance with grading rules.

2. If design drawings are incomplete or not available, at least two locations for each story shall be randomly verified by observing grade stamps or by compliance with grading rules.

8.3.2.4.2 Comprehensive Testing

The minimum number of tests necessary to quantify expected strength properties for comprehensive data collection shall be defined in accordance with the following requirements:

1. If original construction documents exist that define the grade of wood and mechanical properties, at least one location for each story shall be randomly verified by observing grade stamps, or by compliance with grading rules for each component type identified as having a different material grade.

2. If original construction documents defining properties are not complete or do not exist but the date of construction is known and single material use is confirmed, at least three locations shall be randomly verified—by sampling and testing or by observing grade stamps and conditions—for each component type, for every two floors in the building.

3. If no knowledge of the structural system and materials used exists, at least six locations shall be randomly verified—by sampling and testing or by observing grade stamps and conditions—for each element and component type, for every two floors of construction. If it is determined from testing or observation that more than one material grade exists, additional observations and testing shall be conducted until the extent of use for each grade in component fabrication has been established.

4. In the absence of construction records defining connector features present, at least three connectors of each connector type shall be observed for every floor of the building.

5. A full-scale mock-up test shall be conducted for archaic assemblies; at least two cyclic tests of each assembly shall be conducted. A third test shall be conducted if the results of the two tests vary by more than 20%.

C8.3.2.4 Minimum Number of Tests

In order to quantify expected strength and other in-place properties accurately, a minimum number of tests must be conducted on representative components. The minimum number of tests is dictated by available data from original construction, the type of structural system employed, desired accuracy, and quality/condition of in-place materials. Visual access to the structural system also influences testing program definition. As an alternative, the design professional may elect to use the default strength properties in accordance with Section 8.3.2.5. However, using default values without testing is only permitted with the linear analysis procedures. It is strongly encouraged that the expected strengths be derived through testing of assemblies in order to model behavior accurately.

Removal of coverings, including stucco, fireproofing, and partition materials, is generally required to facilitate sampling and observations.

Component types include solid-sawn lumber, glulam beam, and plywood diaphragm. Element types include primary gravity- and lateral-load-resisting components. The observations shall consist of each connector type present in the building (e.g., nails, bolts, straps), such that the composite strength of the connection can be estimated.

8.3.2.5 Default Properties

Use of default properties to determine component strengths shall be permitted in conjunction with the linear analysis procedures of Chapter 3.

Default expected strength and stiffness values for wood and light metal frame shear wall assemblies shall be taken from Table 8-1. Default expected strength and stiffness values for wood diaphragm assemblies shall be taken from Table 8-2.

Table 8-1 Default Expected Strength Values for Wood and Light Frame Shear Walls

	Property	
Shear Wall Type[1]	**Shear Stiffness—G_d (lb/in)**	**Yield Capacity (Q_{CE}) (plf)**
Single Layer Horizontal Lumber Sheathing or Siding	2000	80
Single Layer Diagonal Lumber Sheathing	8000	700
Double Layer Diagonal Lumber Sheathing	18000	1300
Vertical Wood Siding	1000	70
Wood Siding over Horizontal Sheathing	4000	500
Wood Siding over Diagonal Sheathing	11000	1100
Wood Structural Panel Sheathing[2]	--	--
Stucco on Studs, Sheathing, or Fiberboard	14000	350
Gypsum Plaster on Wood Lath	8000	400
Gypsum Plaster on Gypsum Lath	10000	80
Gypsum Wallboard	8000	100
Gypsum Sheathing	8000	100
Plaster on Metal Lath	12000	150
Horizontal Lumber Sheathing with Cut-in Braces or Diagonal Blocking	2000	80
Fiberboard or Particleboard Sheathing	6000	100

1. As defined in Section 8.5.

2. See Section 8.5.9 for shear stiffness and yield capacity of wood structural panel shear walls.

Table 8-2 Default Expected Strength Values for Wood Diaphragms

Diaphragm Type[1]		Property: Shear Stiffness—G_d (lb/in)	Property: Yield Capacity (Q_{CE}) (plf)
Single Straight Sheathing[5]		2,000	120
Double Straight Sheathing	Chorded	15,000	600
	Unchorded	7,000	400
Single Diagonally Sheathing	Chorded	8,000	600
	Unchorded	4,000	420
Diagonal Sheathing with Straight Sheathing or Flooring Above	Chorded	18,000	900
	Unchorded	9,000	625
Double Diagonal Sheathing	Chorded	18,000	900
	Unchorded	9,000	625
Wood Structural Panel Sheathing[2]	Unblocked, Chorded	8,000	-
	Unblocked, Unchorded	4,000	-
Wood Structural Panel Overlays on: a. Straight or Diagonal Sheathing[3] or b. Existing Wood Structural Panel Sheathing[4]	Unblocked, Chorded	9,000	450
	Unblocked, Unchorded	5,000	300
	Blocked, Chorded	18,000	-
	Blocked, Unchorded	7,000	-

1. As defined in Section 8.6.
2. See Section 8.6.8 for shear stiffness and yield capacity of wood structural panel diaphragms.
3. See Section 8.6.9 for yield capacity of wood structural panel overlays on straight or diagonal sheathing.
4. See Section 8.6.10 for yield capacity of wood structural panel overlays on existing wood structural panel sheathing.
5. For single straight sheathing, yield capacity shall be multiplied by 1.5 when built-up roofing is present. The value for stiffness shall not be changed.

Default expected strength values for wood materials comprising individual components shall be based on design resistance values associated with the AF&PA/ ASCE 16 *Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction* (ASCE 16) as determined in accordance with *ASTM D5457-93*. All adjustment factors, including the time-effect factor, that are applicable in accordance with *ASCE 16* shall be considered. The resistance factor, ϕ, shall be taken as unity. If components are damaged, reductions in capacity and stiffness shall be applied, considering the position and size of the ineffective cross-section.

Default expected strength values for connectors shall be based on design resistance values associated with *ASCE 16* as determined in accordance with *ASTM D5457-93.* All adjustment factors, including the time-effect factor, that are applicable in accordance with *ASCE 16* shall be considered. The resistance factor, ϕ, shall be taken as unity.

Alternatively, expected strength values shall be permitted to be directly computed from allowable stress values listed in an approved code using the method contained in *ASTM D5457-93*.

Default deformations at yield of connectors shall be taken as:

1. 0.02 inches for wood-to-wood and wood-to-metal nailed connections.

2. 0.05 inches for wood screws.

3. 0.10 inches for lag bolts.

4. 0.2 inches for wood-to-wood bolted connections.

5. 1.05 inches for wood-to-steel bolted connections.

The estimated deformation of any hardware including allowance for poor fit or oversized holes, shall be summed to obtain the total deformation of the connection.

Default expected strength values for connection hardware shall be taken as the average ultimate test values from published reports.

Default lower-bound strength values, when required in this chapter, shall be taken as expected strength values multiplied by 0.85.

C8.3.2.5 Default Properties

The results of any material testing performed should be compared to the default values for the particular era of building construction. If significantly reduced properties from testing are discovered, further evaluation should be undertaken.

Tables 8-1 and 8-2 contain default values for strength and stiffness of shear wall and diaphragm assemblies. The shear stiffness, G_d, for the assemblies should not be confused with the modulus of rigidity, G, for wood structural panels.

The LRFD methodology of *ASCE 16* is based on the concepts of limit state design, similar to the provisions for strength design in steel or concrete. The reference resistance values for wood elements and connections associated with this standard are contained in the *LRFD Manual for Engineered Wood Construction,* including supplements and guidelines, *(AF&PA LRFD).* The resistance values in these documents were developed using *ASTM D5457-93,* which provides methodologies for calculation directly from data or by format conversion from approved allowable stress values. Use of a format conversion (i.e., the LRFD equivalent of allowable stresses) for computing expected strengths of wood materials comprising individual wood components and for wood connectors (nails, screws, lags, bolts, split rings, etc.) is permitted. This methodology is not applicable for wood shear wall and diaphragm assemblies covered in Tables 8-1 and 8-2. For use with this chapter, capacities for shear wall and diaphragm assemblies are to be taken directly from the tables or as indicated by the table footnotes.

The LRFD reference resistance is computed as the allowable stress value multiplied by a format conversion factor. The format conversion factor is defined as $K_E = 2.16/\phi$, where ϕ is the specified LRFD resistance factor: 0.90 for compression, 0.85 for flexure, 0.80 for tension, 0.75 for shear/torsion, and 0.65 for connections. The allowable stress value shall include all applicable adjustment factors, except for the load duration factor. If allowable values already include consideration of duration effects, the load duration adjustment factor must be divided out prior to format conversion. Note that the time-effect factor specified for LRFD is 1.0 for load combinations that include earthquake loads.

The *NEHRP Recommended Provisions* (BSSC, 2000) contain strength-based resistance values for wood structural panel shear walls and diaphragms. Allowable stress values for wood components and connections can be found in the *National Design Specification for Wood Construction* (NDS, 1997) and the *ASD Manual for Engineered Wood Construction,* including supplements and guidelines (*AF&PA ASD*).

AF&PA LRFD contains a guideline for calculating resistance values for connection hardware for which published report values are in allowable stress format. When computing the expected strength of connections, all limit states, including that of the connection hardware, must be considered (e.g., in addition to the published strength of a hold-down device, consider the limit states for the stud bolts, the anchor bolts in the foundation, etc.).

Actions associated with wood and light metal framing components generally are deformation-controlled, and expected strength material properties will be used most often. Lower bound values are needed for actions that are force-controlled. The 0.85 factor included in this standard to convert expected strength to lower bound values is based on the results of shear wall testing. If more precise lower bound material properties are desired, they should be taken as mean minus one standard deviation from test data for the components in question.

8.3.3 Condition Assessment

8.3.3.1 General

A condition assessment of the existing building and site shall be performed as specified in this section.

A condition assessment shall include the following:

1. The physical condition of primary and secondary components shall be examined and the presence of degradation shall be noted.

2. The presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems shall be verified or established.

3. Other conditions, including neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for rehabilitation that may influence building performance, shall be reviewed and documented.

C8.3.3.1 General

The physical condition of existing components and elements and their connections must be examined for degradation. Degradation may include environmental effects (e.g., decay; splitting; fire damage; biological, termite, and chemical attack) or past/current loading effects (e.g., overload, damage from past earthquakes, crushing, and twisting). Natural wood also has inherent discontinuities such as knots, checks, and splits that must be noted. Configuration problems observed in recent earthquakes, including effects of discontinuous components, improper nailing or bolting, poor fit-up, and connection problems at the foundation level, should also be evaluated. Often, unfinished areas such as attic spaces, basements, and crawl spaces provide suitable access to wood components and can give a general indication of the condition of the rest of the structure. Invasive inspection of critical components and connections is typically required. Neighboring party walls and buildings, the presence of nonstructural components, prior remodeling, and limitations for rehabilitation should also be noted.

Connections require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This includes diaphragm-to-component and component-to-component connections. The strength and deformation capacity of connections must be checked where the connection is attached to one or more components that are expected to experience significant inelastic response. Anchorage of exterior walls to roof and floors in concrete and masonry buildings, for which wood diaphragms are used for out-of-plane loading, requires detailed inspection. Bolt holes in relatively narrow straps sometimes preclude the ductile behavior of the steel strap. Twists and kinks in the strap can also have a serious impact on its anticipated behavior. Cross ties, which are part of the wall anchorage system, need to be inspected to confirm their presence, along with the connection of each piece, to ensure that a positive load path exists to tie the building walls together.

The condition assessment also affords an opportunity to review other conditions that may influence wood elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the wood system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space insulation, and other material shall also be defined such that prudent rehabilitation measures can be planned.

8.3.3.2 Scope and Procedures

All primary structural elements and components involved in gravity- and lateral-load resistance shall be included in the condition assessment.

8.3.3.2.1 Visual Condition Assessment

The dimensions and features of all accessible components shall be measured and compared to available design information. Similarly, the configuration and condition of all accessible connections shall be visually verified, with any deformations or anomalies noted.

8.3.3.2.2 Comprehensive Condition Assessment

If coverings or other obstructions exist, indirect visual inspection through the use of drilled holes and a fiberscope shall be used, or covering materials shall be removed locally based on the following requirements:

1. If detailed design drawings exist, at least three different primary connections shall be exposed for each connection type. If no capacity reducing deviations from the drawings exist, the sample shall be considered representative. If deviations are noted, then all coverings from primary connections of that type shall be removed unless the connection strength is ignored in the seismic evaluation.
2. In the absence of accurate drawings, at least 50% of all primary connection types shall be exposed and inspected or inspected fiberscopically. If common detailing is observed, this sample shall be considered representative. If any details or conditions are observed that result in a discontinuous load path, all primary connections shall be exposed.

C8.3.3.2 Scope and Procedures

Accessibility constraints may necessitate the use of instruments such as a fiberscope or video probe to reduce the amount of damage to covering materials and fabrics. The knowledge and insight gained from the condition assessment is invaluable to understanding load paths and the ability of components to resist and transfer loads. The degree of assessment performed also affects the knowledge factor discussed in Section 8.3.4.

Direct visual inspection provides the most valuable information, as it can be used to identify any configuration issues, allows measurement of component dimensions, and identifies the presence of degradation. The continuity of load paths may be established by viewing components and connection condition. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established.

The scope of the removal effort is dictated by the component and element design. For example, in a braced frame, exposure of several key connections may suffice if the physical condition is acceptable and the configuration matches the design drawings. However, for shear walls and diaphragms, it may be necessary to expose more connection points because of varying designs and the critical nature of the connections. For encased walls and frames for which no drawings exist, it is necessary to indirectly view or expose all primary end connections for verification.

Physical condition of components and connectors may also support the need to use certain destructive and nondestructive test methods. Devices normally used for the detection of reinforcing steel in concrete or masonry may be used to verify the metal straps and hardware located beneath finish surfaces.

8.3.3.3 Basis for the Mathematical Building Model

The results of the condition assessment shall be used to quantify the following items needed to create the mathematical building model:

1. Component section properties and dimensions.

2. Component configuration and eccentricities.

3. Interaction of nonstructural components and their involvement in lateral-load resistance.

4. Presence and effects of alterations to the structural system.

All deviations noted between available construction records and as-built conditions shall be accounted for in the structural analysis.

C8.3.3.3 Basis for the Mathematical Building Model

The acceptance criteria for existing components depend on the design professional's knowledge of the condition of the structural system and material properties, as previously noted. Certain damage—such as water staining, evidence of prior leakage, splitting, cracking, checking, warping, and twisting—may be acceptable. The design professional must establish a case-by-case acceptance for such damage on the basis of capacity loss or deformation constraints. Degradation at connection points should be carefully examined; significant capacity reductions may be involved, as well as a loss of ductility.

8.3.4 Knowledge Factor

A knowledge factor, κ, for computation of wood and light metal framing component capacities and permissible deformations shall be selected in accordance with Section 2.2.6.4, with the following additional requirements specific to wood components and assemblies.

If a comprehensive condition assessment is performed in accordance with Section 8.3.3.2.2, a knowledge factor $\kappa = 1.0$ shall be permitted in conjunction with default properties of Section 8.3.2.5, and testing in accordance with Section 8.3.2.4 is not required.

8.4 General Assumptions and Requirements

8.4.1 Stiffness

Component stiffnesses shall be calculated in accordance with Sections 8.5 through 8.8.

Where design actions are determined using the linear procedures of Chapter 3, stiffnesses for wood materials comprising individual components shall be based on material properties determined in accordance with Section 8.3.2.

Where design actions are determined using the nonlinear procedures of Chapter 3, component force-deformation response shall be represented by nonlinear force-deformation relations. Linear relations shall be permitted where nonlinear response will not occur in the component. The nonlinear force-deformation relation shall be either based on experimental evidence or the generalized force-deformation relation shown in Figure 8-1, with parameters *c*, *d*, and *e* as defined in Table 8-4 for wood components and assemblies. Distance *d* is considered the maximum deflection at the point of first loss of strength. Distance *e* is the maximum deflection at a strength or capacity equal to value *c*. Where the yield strength is not determined by testing in accordance with Section 2.8, the strength at point C shall be taken as 1.5 times the yield strength.

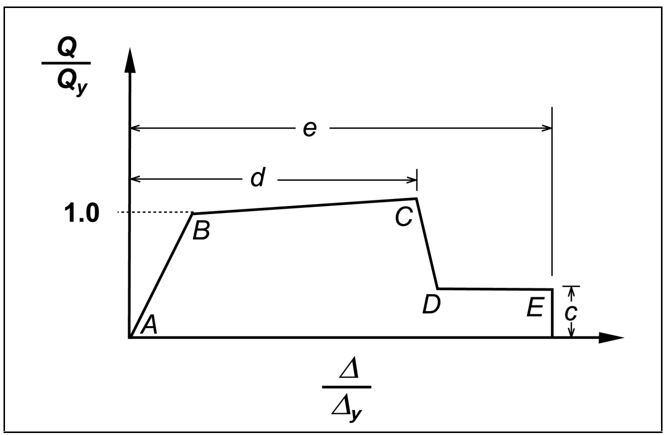

Figure 8-1 Generalized Force-Deformation Relation for Wood Elements or Components

8.4.2 Strength and Acceptance Criteria

8.4.2.1 General

Actions in a structure shall be classified as being either deformation-controlled or force-controlled, as defined in Section 2.4.4. Design strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with Sections 8.4.2.2 and Sections 8.4.2.3, respectively.

8.4.2.2 Deformation-Controlled Actions

Expected strengths for deformation-controlled actions, Q_{CE}, shall be taken as yield strengths obtained experimentally or calculated using accepted principles of mechanics. Unless other procedures are specified in this chapter, the expected strengths shall be permitted to be based on the LRFD procedures contained in *ASCE 16*, except that the resistance factor, ϕ, shall be taken as unity. Material properties shall be determined in accordance with Section 8.3.2. Acceptance criteria for deformation-controlled actions shall be as specified in Sections 8.5 through 8.8.

C8.4.2.2 Deformation-Controlled Actions

The relative magnitude of the *m*-factors alone should not be interpreted as a direct indicator of performance. The stiffness of a component and its expected strength, Q_{CE}, must be considered when evaluating expected performance, For example, while the *m*-factors for gypsum plaster are higher than those for wood structural panels, the stiffness assigned to gypsum plaster is relatively high and the expected strength values are much lower than those for wood structural panels. As a result, worse performance for a given displacement is predicted.

8.4.2.3 Force-Controlled Actions

Where obtained by testing, lower-bound strengths for force-controlled actions, Q_{CL}, shall be taken as the mean minus one standard deviation of yield strengths. Where calculated using established principles of mechanics or based on LRFD procedures contained in *ASCE 16*, lower-bound strength values shall be determined using the default lower bound material properties defined in Section 8.3.2.5.

Where the force-controlled design actions, Q_{UF}, calculated in accordance with Section 3.4.2.1.2 are based on a limit-state analysis, the expected strength of the components delivering load to the component under consideration shall be taken as 1.5 times the yield strength.

C8.4.2.3 Force-Controlled Actions

The maximum forces developed in yielding shear walls and diaphragms are consistently 1.5 to 2 times the yield force. Other wood components and connectors exhibit similar overstrength.

8.4.3 Connection Requirements

Unless otherwise specified in this standard, connections between wood components in a lateral-force-resisting system shall be considered in accordance with this section. Demands on connectors, including nails, screws, lags, bolts, split rings, and shear plates used to link wood components to other wood or metal components, shall be considered deformation-controlled actions. Demands on bodies of connections, and bodies of connection hardware, shall be considered force-controlled actions.

C8.4.3 Connection Requirements

In considering connections between wood components in this standard, connectors are distinguished from bodies of connections. Connectors, which consist of the nails, screws, lags, bolts, split rings, and shear plates used to link pieces of a connection assembly together, are considered to have the ability to deform in a ductile manner, provided the bodies of the connections do not prematurely fracture. Much of the ductility in a wood shear wall or diaphragm assembly comes from bending in the nails prior to point where nails pull through the sheathing material. In bolted connections, bolt bending or crushing of the wood around the bolt hole are ductile sources of deformation in an assembly, as long as net section fracture or splitting do not occur. For this reason, connectors are considered deformation-controlled and bodies of connections are considered force-controlled. When determining the demand on force-controlled portions of the connection assembly, use of a limit-state analysis to determine the maximum force that can be delivered to the connection is recommended.

When computing the strength of connections, all potential limit states should be considered, including those associated with the bodies of connection hardware or connectors with which the assembly may be composed. For example, in addition to the strength of a hold-down device itself, limit states for the stud bolts, foundation bolts, and net section of the end post should be considered. The controlling condition will determine the expected or lower-bound strength of the connection.

8.4.4 Rehabilitation Measures

If portions of a wood building structure are deficient for the selected Rehabilitation Objective, the structure shall be rehabilitated, reinforced, or replaced. If replacement of the element is selected, the new element shall be designed in accordance with an approved building code. If reinforcement of the existing framing system is selected, the following factors shall be considered:

1. Degree of degradation in the component from such mechanisms as biological attack, creep, high static or dynamic loading, moisture, or other effects.

2. Level of steady state stress in the components to be reinforced and the potential to temporarily remove this stress if appropriate.

3. Elastic and inelastic properties of existing components; strain compatibility with any new reinforcement materials shall be preserved.

4. Ductility, durability, and suitability of existing connectors between components, and access for reinforcement or modification.

5. Efforts necessary to achieve appropriate fit-up for reinforcing components and connections.

6. Load path and deformation of the components at end connections.

7. Presence of components manufactured with archaic materials, which may contain material discontinuities and shall be examined during the rehabilitation design to ensure that the selected reinforcement is feasible.

C8.4.4 Rehabilitation Measures

Special attention is required where connections such as bolts and nails are encountered.

Wood structural panels are used to provide lateral strength and stiffness to most modern wood frame buildings and are generally recommended for the rehabilitation of horizontal diaphragms and shear walls of existing buildings. The system relies on the in-plane strength and stiffness of the panels and their connection to the framing. Panels are connected together by nailing into the same structural member to create, in effect, one continuous panel. The various panels are described in Sections 8.5 and 8.6. The performance of the structural panels is dependent to a great degree on the nailing or attachment to the framing. The nail spacing and effectiveness of the attachment should be investigated if the existing panels are expected to withstand significant loads. If nails are to be added to existing panels, they should be the same size as the existing nails.

8.5 Wood and Light Frame Shear Walls

8.5.1 General

Wood and light frame shear walls shall be categorized as primary or secondary elements. Walls that are considered part of the lateral-force-resisting system shall be considered primary elements. Walls that are not considered part of the lateral-force-resisting system, but must remain stable to support the gravity loads during seismic excitation, shall be considered secondary elements.

Dissimilar wall sheathing materials on opposite sides of a wall shall not be combined when calculating the capacity of the wall. Different walls sheathed with dissimilar materials along the same line of lateral-force resistance shall be analyzed based on only the wall sheathing with the greatest capacity. The walls shall be analyzed based on the relative rigidity and capacity of the materials to determine if performance of the secondary elements is acceptable.

For overturning calculations on shear wall elements, stability shall be evaluated in accordance with Section 3.2.10. Net tension due to overturning shall be resisted by uplift connections.

The effects of openings in wood shear walls shall be considered. Where required, reinforcement consisting of chords and collectors shall be added to provide sufficient load capacity around openings to meet the requirements for shear walls.

Connections between shear walls and other components including drag struts, collectors, diaphragms, posts, and foundations shall be considered in accordance with Section 8.4.3, and designed for forces calculated in accordance with Chapter 3. Components supporting discontinuous shear walls shall be considered in accordance with Section 8.8.2.

The expected strength, Q_{CE}, of wood and light frame shear walls shall be taken as the yield capacity of the shear wall assembly determined in accordance with Sections 8.5.4 through 8.5.18.

C8.5.1 General

The behavior of wood and light frame shear walls is complex and influenced by many factors, the primary factor being the wall sheathing. Wall sheathings can be divided into many categories (e.g., brittle, elastic, strong, weak, good at dissipating energy, poor at dissipating energy). In many existing buildings, the walls were not expected to act as shear walls (e.g., a wall sheathed with wood lath and plaster). Most shear walls are designed based on values from monotonic load tests and historically accepted values. The allowable shear per unit length used for design was assumed to be the same for long walls, narrow walls, walls with stiff tie-downs, and walls with flexible tie-downs. Only recently have shear wall assemblies—framing, covering, anchorage—been tested using cyclic loading.

Another major factor influencing the behavior of shear walls is the aspect ratio of the wall. The *NEHRP Recommended Provisions* (BSSC, 2000) limit the aspect ratio (height-to-width) for structural panel shear walls to 2:1 for full design shear capacity and permit reduced design shear capacities for walls with aspect ratios up to 3.5:1. The interaction of the floor and roof with the wall, the end conditions of the wall, and the redundancy or number of walls along any wall line would affect the wall behavior for walls with the same aspect ratio. In addition, the rigidity of the tie-downs at the wall ends has an important effect in the behavior of narrow walls.

The presence of any but small openings in wood shear walls will cause a reduction in the stiffness and yield capacity due to a reduced length of wall available to resist lateral forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings will reduce the loss in overall stiffness and limit damage in the area of openings. See *BSSC* (2000) for reinforcement requirements around openings in wood shear walls.

For wood and light frame shear walls, the important limit states are sheathing failure, connection failure, tie-down failure, and excessive deflection. Limit states define the point of life safety and, often, of structural stability. To reduce damage or retain usability immediately after an earthquake, deflection must be limited (see Section 2.5). The ultimate capacity is the maximum capacity of the assembly, regardless of the deflection.

8.5.2 Types of Wood Frame Shear Walls

8.5.2.1 Existing Wood Frame Shear Walls

8.5.2.1.1 Single Layer Horizontal Lumber Sheathing or Siding

Single layer horizontal lumber sheathing or siding shall include horizontal sheathing or siding applied directly to studs or horizontal boards nailed to studs 2 inches or greater in width.

C8.5.2.1.1 Single Layer Horizontal Lumber Sheathing or Siding

Typically, 1"x horizontal sheathing or siding is applied directly to studs. Forces are resisted by nail couples. Horizontal boards, from 1" x 4" to 1" x 12", typically are nailed to 2"x or greater width studs with two or more nails (typically 8d or 10d) per stud.

8.5.2.1.2 Diagonal Lumber Sheathing

Diagonal lumber sheathing shall include sheathing applied at a 45-degree angle to the studs in a single or double layer with three or more nails per stud, sill and top plates.

C8.5.2.1.2 Diagonal Lumber Sheathing

Typically, 1" x 6" or 1" x 8" diagonal sheathing, applied directly to the studs, resists lateral forces primarily by triangulation (i.e., direct tension and compression). Sheathing boards are installed at a 45-degree angle to studs, with three or more nails (typically 8d or 10d) per stud, and to sill and top plates. A second layer of diagonal sheathing is sometimes added on top of the first layer, at 90 degrees to the first layer (called Double Diagonal Sheathing), for increased load capacity and stiffness.

8.5.2.1.3 Vertical Wood Siding Only

Vertical wood siding shall include vertical boards nailed directly to studs and blocking 2 inches or greater in width.

C8.5.2.1.3 Vertical Wood Siding Only

Typically, 1" x 8", 1" x 10", or 1" x 12" vertical boards are nailed directly to 2"x or greater width studs and blocking with 8d or 10d galvanized nails. The lateral forces are resisted by nail couples, similarly to horizontal siding.

8.5.2.1.4 Wood Siding over Horizontal Sheathing

Wood siding over horizontal sheathing shall include siding connected to horizontal sheathing with nails that go through the sheathing to the studs.

C8.5.2.1.4 Wood Siding over Horizontal Sheathing

Typically, siding is nailed with 8d or 10d galvanized nails through the sheathing to the studs. Lateral forces are resisted by nail couples for both layers.

8.5.2.1.5 Wood Siding over Diagonal Sheathing

Wood siding over diagonal sheathing shall include siding connected to diagonal sheathing with nails that go through the sheathing to the studs.

C8.5.2.1.5 Wood Siding over Diagonal Sheathing

Typically, siding is nailed with 8d or 10d galvanized nails to and through the sheathing into the studs. Diagonal sheathing provides most of the lateral resistance by triangulation (see Section 8.5.2.1.2).

8.5.2.1.6 Wood Structural Panel Sheathing or Siding

Wood structural panel sheathing or siding shall include wood structural panels, as defined in this standard, oriented vertically or horizontally and nailed to studs 2 inches or greater in width.

C8.5.2.1.6 Wood Structural Panel Sheathing or Siding

Typically, 4′ x 8′ panels are applied vertically or horizontally to 2"x or greater studs and nailed with 6d to 10d nails. These panels resist lateral forces by panel diaphragm action.

8.5.2.1.7 Stucco on Studs

Stucco on studs (over sheathing or wire-backed building paper) shall include portland cement plaster applied to wire lath or expanded metal lath. Wire lath or expanded metal lath shall be nailed to the studs.

C8.5.2.1.7 Stucco on Studs

Typically, 7/8-inch portland cement plaster is applied to wire lath or expanded metal lath. Wire lath or expanded metal lath is nailed to the studs with 11 gage nails or 16 gage staples at 6 inches on center. This assembly resists lateral forces by panel diaphragm action.

8.5.2.1.8 Gypsum Plaster on Wood Lath

Gypsum plaster on wood lath shall include gypsum plaster keyed onto spaced wood lath that is nailed to the studs.

C8.5.2.1.8 Gypsum Plaster on Wood Lath

Typically, 1-inch gypsum plaster is keyed onto spaced 1-1/4-inch wood lath that is nailed to studs with 13 gage nails. Gypsum plaster on wood lath resists lateral forces by panel diaphragm-shear action.

8.5.2.1.9 Gypsum Plaster on Gypsum Lath

Gypsum plaster on gypsum lath shall include plaster that is glued or keyed to gypsum lath nailed to studs.

C8.5.2.1.9 Gypsum Plaster on Gypsum Lath

Typically, 1/2-inch plaster is glued or keyed to 16" x 48" gypsum lath, which is nailed to studs with 13 gage nails. Gypsum plaster on gypsum lath resists lateral loads by panel diaphragm action.

8.5.2.1.10 Gypsum Wallboard or Drywall

Gypsum wallboard or drywall shall include manufactured panels with a paper facing and gypsum core that are oriented horizontally or vertically and nailed to studs or blocking in a single layer or multiple layers.

C8.5.2.1.10 Gypsum Wallboard or Drywall

Typically, 4′x 8′ to 4′ x 12′ panels are laid-up horizontally or vertically and nailed to studs or blocking with 5d to 8d cooler nails at 4 to 7 inches on center. Multiple layers are used in some situations. The assembly resists lateral forces by panel diaphragm action.

8.5.2.1.11 Gypsum Sheathing

Gypsum sheathing shall include manufactured gypsum panels that are oriented horizontally or vertically and nailed to studs or blocking.

C8.5.2.1.11 Gypsum Sheathing

Typically, 4′ x 8′ to 4′ x 12′ panels are laid-up horizontally or vertically and nailed to studs or blocking with galvanized 11 gage 7/16-inch diameter head nails at 4 to 7 inches on center. Gypsum sheathing is usually installed on the exterior of structures with siding over it in order to improve fire resistance. Lateral forces are resisted by panel diaphragm action.

8.5.2.1.12 Plaster on Metal Lath

Plaster on metal lath shall include gypsum plaster applied to expanded wire lath that is nailed to the studs.

C8.5.2.1.12 Plaster on Metal Lath

Typically, 1-inch gypsum plaster is applied on expanded wire lath that is nailed to the studs. Lateral forces are resisted by panel diaphragm action.

8.5.2.1.13 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking

Horizontal lumber sheathing with cut-in braces or diagonal blocking shall include 1"x horizontal sheathing or siding applied directly to studs or 1" x 4" to 1" x 12" horizontal boards nailed to studs 2 inches or greater in width. The wall shall be braced with diagonal cut-in braces or blocking extending from corner to corner.

C8.5.2.1.13 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking

Horizontal sheathing with cut-in braces or diagonal blocking is installed in the same manner as horizontal sheathing, except the wall is braced with cut-in (or let-in) braces or blocking. The bracing is usually installed at a 45-degree angle and nailed with 8d or 10d nails at each stud, and at the top and bottom plates. Bracing provides only nominal increase in resistance.

8.5.2.1.14 Fiberboard or Particleboard Sheathing

Fiberboard or particleboard sheathing walls shall include fiberboard or particleboard panels that are applied directly to the studs with nails.

C8.5.2.1.14 Fiberboard or Particleboard Sheathing

Typically, 4′ x 8′ panels are applied directly to the studs with nails. Fiberboard requires nails (typically 8d) with large heads such as roofing nails. Lateral loads are resisted by panel diaphragm action.

8.5.2.2 Enhanced Wood Frame Shear Walls

Enhanced wood frame shear walls shall include existing shear walls rehabilitated in accordance with an approved method. Enhanced wood shear walls shall be evaluated in accordance with Section 8.5.9.

C8.5.2.2 Enhanced Wood Frame Shear Walls

Possible rehabilitation methods for wood shear walls are described in Sections C8.5.2.2.1 through C8.5.2.2.5.

C8.5.2.2.1 Wood Structural Panel Sheathing Added to Unfinished Stud Walls

Wood structural panel sheathing may be added to one side of unfinished stud walls to increase the wall shear capacity and stiffness.

Examples of unfinished stud walls are cripple walls and attic end walls.

C8.5.2.2.2 Wood Structural Panel Sheathing Overlay of Existing Shear Walls

The following types of existing shear walls may be overlaid with wood structural panel sheathing:

1. Single layer horizontal lumber sheathing or siding.
2. Single layer diagonal lumber sheathing.
3. Vertical wood siding only.
4. Gypsum plaster or wallboard on studs (also on gypsum lath and gypsum wallboard).
5. Gypsum sheathing.
6. Horizontal lumber sheathing with cut-in braces or diagonal blocking.
7. Fiberboard or particleboard sheathing.

The original sheathing should not be included in the evaluation conducted in accordance with Section 8.5.9 and the yield capacity of the overlay material should be reduced by 20%.

This method results in a moderate increase in shear capacity and stiffness and can be applied in most places in most structures. For example, plywood sheathing can be applied over an interior wall finish. For exterior applications, the wood structural panel can be nailed directly through the exterior finish to the studs.

When existing shear walls are overlaid with wood structural panels, the connections of the overlay to the existing framing must be considered. Splitting can occur in both the wood sheathing and the framing. The length of nails needed to achieve full capacity attachment in the existing framing must be determined. This length will vary with the thickness of the existing wall covering. Sometimes staples are used instead of nails to prevent splitting. The overlay is stapled to the wood sheathing instead of the framing. Nails are recommended for overlay attachment to the underlying framing. In some cases, new blocking at wood structural panel joints may also be needed.

C8.5.2.2.3 Wood Structural Panel Sheathing Added under Existing Wall Covering

The existing wall covering may be removed; wood structural panel sheathing, connections, and tie-downs may be added; and the wall covering may be replaced.

This method will result in a significant increase in shear capacity. In some cases, where earthquake loads are large, this may be the best method of rehabilitation. This rehabilitation procedure can be used on any of the existing shear wall assemblies. Additional framing members can be added if necessary, and the wood structural panels can be cut to fit existing stud spacings.

C8.5.2.2.4 Increased Attachment

Additional nailing, collector straps, splice straps, tie-downs or other collectors may be added to existing wood structural panel-sheathed walls to increase their rigidity and capacity.

For existing structural panel-sheathed walls, additional nailing will result in higher capacity and increased stiffness. Other connectors—collector straps, splice straps, or tie-downs—are often necessary to increase the rigidity and capacity of existing structural panel shear walls. Increased ductility will not necessarily result from the additional nailing. Access to these shear walls will often require the removal and replacement of existing finishes.

C8.5.2.2.5 Connections

Where absent, new connections between shear walls and diaphragms and foundations may be added. Where needed, blocking between floor and roof joists at shear walls may be added. Blocking should be connected to the shear wall and the diaphragm to provide a load path for lateral loads. Wood for framing members or blocking should be kiln-dried or well-seasoned to prevent it from shrinking away from the existing framing, or splitting.

Most shear wall rehabilitation procedures require a check of all existing connections, especially to diaphragms and foundations. Sheet metal framing clips can be used to provide a verifiable connection between the wall framing, the blocking, and the diaphragm. Framing clips are also often used for connecting blocking or rim joists to sill plates.

Frequently, bolting between sill plates and foundations must be added.

The framing in existing buildings is usually very dry, hard, and easily split. Care must be taken not to split the existing framing when adding connectors. Predrilling holes for nails will reduce splitting, and framing clips that use small nails are less likely to split the existing framing.

8.5.2.3 New Wood Frame Shear Walls

New wood frame shear walls shall include all new wood structural panel shear walls added to an existing lateral-force-resisting system. Design of new walls shall be in accordance with the requirements of an approved building code.

C8.5.2.3 New Wood Frame Shear Walls

New shear walls using the existing framing or new framing generally are sheathed with wood structural panels (i.e., plywood or oriented strand board). According to the *NEHRP Recommended Provisions* (BSSC, 2000), only wood structural panel sheathing is permitted for use in wood frame shear walls in engineered construction. The thickness and grade of these panels can vary. In most cases, the panels are placed vertically and fastened directly to the studs and plates. This reduces the need for blocking at the joints. All edges of panels must be blocked to obtain full capacity. The thickness, size, and number of fasteners, and aspect ratio and connections will determine the capacity of the new walls. Additional information on the various panels available and their application for shear walls can be found in documents from the American Plywood Association (APA) such as APA (1995 and 1997) and Tissell (1993).

8.5.3 Types of Light Gage Metal Frame Shear Walls

8.5.3.1 Existing Light Gage Metal Frame Shear Walls

8.5.3.1.1 Plaster on Metal Lath

Plaster on metal lath shall include gypsum plaster applied to metal lath or expanded metal lath that is connected to the metal framing with wire ties.

C8.5.3.1.1 Plaster on Metal Lath

Typically, 1 inch of gypsum plaster is applied to metal lath or expanded metal that is connected to the metal framing with wire ties.

8.5.3.1.2 Gypsum Wallboard

Gypsum wallboard shear walls shall include gypsum wallboard panels that are attached to the studs.

C8.5.3.1.2 Gypsum Wallboard

Typically, 4′ x 8′ to 4′ x 12′ panels are laid-up horizontally and screwed with No. 6 x 1-inch-long self-tapping screws to studs at 4 to 7 inches on center.

8.5.3.1.3 Wood Structural Panel Sheathing or Siding

Wood structural panel shear walls shall include structural panels that are attached to the studs and tracks.

C8.5.3.1.3 Wood Structural Panel Sheathing or Siding

Typically, the wood structural panels are applied vertically and screwed to the studs and tracks with No. 8 to No. 12 self-tapping screws.

8.5.3.2 Enhanced Light Gage Metal Frame Shear Walls

Enhanced light gage metal frame shear walls shall include existing shear walls rehabilitated in accordance with an approved method.

C8.5.3.2 Enhanced Light Gage Metal Frame Shear Walls

Possible rehabilitation methods for light gage metal frame shear walls are described in Sections C8.5.3.2.1 and C8.5.3.2.2. See Section 8.5.2.2 for additional information concerning enhancement of existing shear walls.

C8.5.3.2.1 Wood Structural Panel Sheathing Added to Existing Metal Stud Walls

Any existing covering other than wood structural panels shall be removed and replaced with wood structural panels. Connections to the diaphragm(s) and the foundation shall be checked and strengthened when not adequate to resist enhanced wall capacity.

C8.5.3.2.2 Increased Attachment

Screws and connections shall be added to connect existing wood structural panels to framing.

8.5.3.3 New Light Gage Metal Frame Shear Walls

New light gage metal frame shear walls shall include all new wood structural panel elements added to an existing lateral-force-resisting system. Design of new walls shall be in accordance with the requirements of an approved building code.

8.5.4 Single Layer Horizontal Lumber Sheathing or Siding Shear Walls

8.5.4.1 Stiffness

The deflection of single layer horizontal lumber sheathing or siding shear walls shall be calculated in accordance with Equation (8-1):

$$\Delta_y = v_y h/G_d + (h/b)d_a \tag{8-1}$$

where:

b	=	Shear wall width
h	=	Shear wall height
v_y	=	Shear at yield in the direction under consideration
G_d	=	Shear stiffness from Table 8-1
Δ_y	=	Calculated deflection of shear wall at yield
d_a	=	Elongation of anchorage at end of wall determined by anchorage details and load magnitude

Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.4.1 Stiffness

Horizontal lumber sheathed shear walls are weak and very flexible and have long periods of vibration. The strength and stiffness degrade with cyclic loading. These shear walls are suitable only where earthquake shear loads are low and deflection control is not required.

8.5.4.2 Strength

The yield capacity of horizontal sheathing or siding shall be determined in accordance with Section 8.3.2.

C8.5.4.2 Strength

This capacity is dependent on the width of the boards, spacing of the studs, and the size, number, and spacing of the nails. Allowable capacities are listed for various configurations, together with a description of the nail couple method, in the *WWPA* (1996). See also *ATC-7* for a discussion of the nail couple method.

8.5.4.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relations, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

C8.5.4.3 Acceptance Criteria

Deformation acceptance criteria are determined by the capacity of lateral- and gravity-load-resisting components and elements to deform with limited damage or without failure. Excessive deflection could result in major damage to the structure and/or its contents.

8.5.4.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

C8.5.4.4 Connections

The capacity and ductility of these connections will often determine the failure mode0well as the capacity of the assembly. Ductile connections with sufficient capacity will give acceptable and expected performance (see Section 8.3.2.2.2).

Table 8-3 Numerical Acceptance Factors for Linear Procedures—Wood Components

		m-factors				
			Primary		Secondary	
		IO	LS	CP	LS	CP
Wood and Light Frame Shear Walls[1, 3]	**Height/Width Ratio (h/b)**					
Horizontal 1" x 6" Sheathing	h/b ≤ 1.0	1.8	4.2	5.0	5.0	5.5
Horizontal 1" x 8" or 1" x 10" Sheathing	h/b ≤ 1.0	1.6	3.4	4.0	4.0	5.0
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	h/b ≤ 1.0	1.4	2.6	3.0	3.1	4.0
Horizontal Wood Siding Over Horizontal 1" x 8" or 1" x 10" Sheathing	h/b ≤ 1.5	1.3	2.3	2.6	2.8	3.0
Diagonal 1" x 6" Sheathing	h/b ≤ 1.5	1.5	2.9	3.3	3.4	3.8
Diagonal 1" x 8" Sheathing	h/b ≤ 1.5	1.4	2.7	3.1	3.1	3.6
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	h/b ≤ 2.0	1.3	2.2	2.5	2.5	3.0
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	h/b ≤ 2.0	1.3	2.0	2.3	2.5	2.8
Double Diagonal 1" x 6" Sheathing	h/b ≤ 2.0	1.2	1.8	2.0	2.3	2.5
Double Diagonal 1" x 8" Sheathing	h/b ≤ 2.0	1.2	1.7	1.9	2.0	2.5
Vertical 1" x 10" Sheathing	h/b ≤ 1.0	1.5	3.1	3.6	3.6	4.1
Wood Structural Panel Sheathing or Siding[2]	h/b < 1.0	1.7	3.8	4.5	4.5	5.5
	2.0 ≤ h/b ≤ 3.5	1.4	2.6	3.0	6.0	7.0
Stucco on Studs[2]	h/b ≤1.0	1.5	3.1	3.6	3.6	4.0
	h/b = 2.0	1.3	2.2	2.5	5.0	6.0
Stucco over 1"-x Horizontal Sheathing	h/b ≤ 2.0	1.5	3.0	3.5	3.5	4.0
Gypsum Plaster on Wood Lath	h/b ≤ 2.0	1.7	3.9	4.6	4.6	5.1
Gypsum Plaster on Gypsum Lath	h/b ≤ 2.0	1.8	4.2	5.0	4.2	5.5
Gypsum Plaster on Metal Lath	h/b ≤ 2.0	1.7	3.7	4.4	3.7	5.0
Gypsum Sheathing	h/b ≤ 2.0	1.9	4.7	5.7	4.7	6.0
Gypsum Wallboard[2]	h/b ≤ 1.0	1.9	4.7	5.7	4.7	6.0
	h/b = 2.0	1.6	3.4	4.0	3.8	4.5
Horizontal 1" x 6" Sheathing with Cut-In Braces or Diagonal Blocking	h/b ≤ 1.0	1.7	3.7	4.4	4.2	4.8
Fiberboard or Particleboard Sheathing	h/b ≤ 1.5	1.6	3.2	3.8	3.8	5.0

1. Shear walls shall be permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 3.2.2.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.
2. Linear interpolation shall be permitted for intermediate values of aspect ratio.
3. Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting lateral loads.
4. Actions on connectors not listed in this table shall be considered force-controlled.
5. For diaphragm components with aspect ratios between maximum listed values and 4.0, *m*-factors shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.

Table 8-3 Numerical Acceptance Factors for Linear Procedures—Wood Components (continued)

		m-factors				
			Primary		Secondary	
		IO	LS	CP	LS	CP
Diaphragms[5]	**Length/Width Ratio (L/b)**					
Single Straight Sheathing, Chorded	L/b ≤ 3.0	1	2.0	2.5	2.4	3.1
Single Straight Sheathing, Unchorded	L/b ≤ 3.0	1	1.5	2.0	1.8	2.5
Double Straight Sheathing, Chorded	L/b ≤ 3.0	1.25	2.0	2.5	2.3	2.8
Double Straight Sheathing, Unchorded	L/b ≤ 3.0	1	1.5	2.0	1.8	2.3
Single Diagonal Sheathing, Chorded	L/b ≤ 3.0	1.25	2.0	2.5	2.3	2.9
Single Diagonal Sheathing, Unchorded	L/b ≤ 3.0	1	1.5	2.0	1.8	2.5
Straight Sheathing Over Diagonal Sheathing, Chorded	L/b ≤ 3.0	1.5	2.5	3.0	2.8	3.5
Straight Sheathing Over Diagonal Sheathing, Unchorded	L/b ≤ 3.0	1.25	2.0	2.5	2.3	3.0
Double Diagonal Sheathing, Chorded	L/b ≤ 3.5	1.5	2.5	3.0	2.9	3.5
Double Diagonal Sheathing, Unchorded	L/b ≤ 3.5	125	2.0	2.5	2.4	3.1
Wood Structural Panel, Blocked, Chorded[2]	L/b ≤ 3.0	1.5	3.0	4.0	3.0	4.5
	L/b = 4	1.5	2.5	3.0	2.8	3.5
Wood Structural Panel, Unblocked, Chorded[2]	L/b ≤ 3	1.5	2.5	3.0	2.9	4.0
	L/b = 4	1.5	2.0	2.5	2.6	3.2
Wood Structural Panel, Blocked, Unchorded[2]	L/b ≤ 2.5	1.25	2.5	3.0	2.9	4.0
	L/b = 3.5	1.25	2.0	2.5	2.6	3.2
Wood Structural Panel, Unblocked, Unchorded[2]	L/b ≤ 2.5	1.25	2.0	2.5	2.4	3.0
	L/b = 3.5	1.0	1.5	2.0	2.0	2.6
Wood Structural Panel Overlay on Sheathing, Chorded[2]	L/b ≤ 3	1.5	2.5	3.0	2.9	4.0
	L/b = 4	1.5	2.0	2.5	2.6	3.2
Wood Structural Panel Overlay on Sheathing, Unchorded[2]	L/b ≤ 2.5	1.25	2.0	2.5	2.4	3.0
	L/b = 3.5	1.0	1.5	2.0	1.9	2.6

1. Shear walls shall be permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 3.2.2.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.
2. Linear interpolation shall be permitted for intermediate values of aspect ratio.
3. Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting lateral loads.
4. Actions on connectors not listed in this table shall be considered force-controlled.
5. For diaphragm components with aspect ratios between maximum listed values and 4.0, *m*-factors shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.

Table 8-3 Numerical Acceptance Factors for Linear Procedures—Wood Components (continued)

	m-factors				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Components/Elements					
Frame components subject to axial tension and/or bending	1.0	2.5	3.0	2.5	4.0
Frame components subject to axial compression	force-controlled				
Wood piles, bending and axial	1.2	2.5	3.0	—	—
Cantilever pole structures, bending and axial	1.2	3.0	3.5	—	—
Pole structures with diagonal bracing	1.0	2.5	3.0	—	—
Connectors[4]					
Nails—8d and larger—Wood to Wood	2.0	6.0	8.0	8.0	9.0
Nails—8d and larger—Metal to Wood	2.0	4.0	6.0	5.0	7.0
Screws—Wood to Wood	1.2	2.0	2.2	2.0	2.5
Screws—Metal to Wood	1.1	1.8	2.0	1.8	2.3
Lag Bolts—Wood to Wood	1.4	2.5	3.0	2.5	3.3
Lag Bolts—Metal to Wood	1.3	2.3	2.5	2.4	3.0
Machine Bolts—Wood to Wood	1.3	3.0	3.5	3.3	3.9
Machine Bolts—Metal to Wood	1.4	2.8	3.3	3.1	3.7
Split Rings and Shear Plates	1.3	2.2	2.5	2.3	2.7
Bolts—Wood to Concrete or Masonry	1.4	2.7	3.0	2.8	3.5

1. Shear walls shall be permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 3.2.2.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.
2. Linear interpolation shall be permitted for intermediate values of aspect ratio.
3. Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting lateral loads.
4. Actions on connectors not listed in this table shall be considered force-controlled.
5. For diaphragm components with aspect ratios between maximum listed values and 4.0, *m*-factors shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.

Table 8-4 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Wood Components

		Modeling Parameters			Acceptance Criteria				
					Acceptable Deformation Ratio Δ / Δ_y				
					Performance Level				
		$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio		Component Type			
						Primary		Secondary	
		d	e	c	IO	LS	CP	LS	CP
Wood and Light Frame Shear Walls[1]	**Height/Width Ratio (h/b)**								
Horizontal 1" x 6" Sheathing	h/b ≤ 1.0	5.0	6.0	0.3	2.0	4.0	5.0	5.0	6.0
Horizontal 1" x 8" or 1" x 10" Sheathing	h/b ≤ 1.0	4.0	5.0	0.3	1.8	3.3	4.0	4.0	5.0
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	h/b ≤ 1.5	3.0	4.0	0.2	1.5	2.5	3.0	3.0	4.0
Horizontal Wood Siding Over Horizontal 1" x 8" or 1" x 10" Sheathing	h/b ≤ 1.5	2.6	3.6	0.2	1.4	2.2	2.6	2.6	3.6
Diagonal 1" x 6" Sheathing	h/b ≤ 1.5	3.3	4.0	0.2	1.6	2.7	3.3	3.3	4.0
Diagonal 1" x 8" Sheathing	h/b ≤ 1.5	3.1	4.0	0.2	1.5	2.6	3.1	3.1	4.0
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	h/b ≤ 2.0	2.5	3.0	0.2	1.4	2.1	2.5	2.5	3.0
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	h/b ≤ 2.0	2.3	3.0	0.2	1.3	2.0	2.3	2.3	3.0
Double Diagonal 1" x 6" Sheathing	h/b ≤ 2.0	2.0	2.5	0.2	1.3	1.8	2.0	2.0	2.5
Double Diagonal 1" x 8" Sheathing	h/b ≤ 2.0	2.0	2.5	0.2	1.3	1.8	2.0	2.0	2.5
Vertical 1" x 10" Sheathing	h/b ≤ 1.0	3.6	4.0	0.3	1.7	3.0	3.6	3.6	4.0
Wood Structural Panel Sheathing or Siding[2]	h/b ≤ 1.0	4.5	5.5	0.3	1.9	3.6	4.5	4.5	5.5
	2.0 ≤ h/b ≤ 3.5	3.0	4.0	0.2	1.5	2.5	3.0	3.0	4.0
Stucco on Studs[2]	h/b ≤ 1.0	3.6	4.0	0.2	1.7	3.0	3.6	3.6	4.0
	h/b = 2.0	2.5	3.0	0.2	1.4	2.1	2.5	2.5	3.0
Stucco Over 1"x Horizontal Sheathing	h/b ≤ 2.0	3.5	4.0	0.2	1.6	2.9	3.5	3.5	4.0
Gypsum Plaster on Wood Lath	h/b ≤ 2.0	4.6	5.0	0.2	1.9	3.7	4.6	4.6	5.0
Gypsum Plaster on Gypsum Lath	h/b ≤ 2.0	5.0	6.0	0.2	2.0	4.0	5.0	5.0	6.0

1. Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting lateral loads.
2. Linear interpolation shall be permitted for intermediate values of aspect ratio.
3. For diaphragm components with aspect ratios between maximum listed values and 4.0, deformation ratios shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.
4. Actions on connectors not listed in this table shall be considered force-controlled.

Table 8-4 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Wood Components (continued)

		Modeling Parameters			Acceptance Criteria				
					Acceptable Deformation Ratio Δ / Δ_y				
					Performance Level				
		$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio		Component Type			
						Primary		Secondary	
		d	**e**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
Gypsum Plaster on Metal Lath	h/b $\leq$ 2.0	4.4	5.0	0.2	1.9	3.6	4.4	4.4	5.0
Gypsum Sheathing	h/b $\leq$ 2.0	5.7	6.3	0.2	2.2	4.5	5.7	5.7	6.3
Gypsum Wallboard[2]	h/b $\leq$ 1.0	5.7	6.3	0.2	2.2	4.5	5.7	5.7	6.3
	h/b = 2.0	4.0	5.0	0.2	1.8	3.3	4.0	4.0	5.0
Horizontal 1" x 6" Sheathing with Cut-In Braces or Diagonal Blocking	h/b $\leq$ 1.0	4.4	5.0	0.2	1.9	3.6	4.4	4.4	5.0
Fiberboard or Particleboard Sheathing	h/b $\leq$ 1.5	3.8	4.0	0.2	1.7	3.1	3.8	3.8	4.0
Diaphragms[3]	**Length/Width Ratio (L/b)**								
Single Straight Sheathing, Chorded	L/b $\leq$ 2.0	2.5	3.5	0.2	1.4	2.1	2.5	2.5	3.5
Single Straight Sheathing, Unchorded	L/b $\leq$ 2.0	2.0	3.0	0.3	1.3	1.8	2.0	2.0	3.0
Double Straight Sheathing, Chorded	L/b $\leq$ 2.0	2.5	3.5	0.2	1.4	2.1	2.5	2.5	3.5
Double Straight Sheathing, Unchorded	L/b $\leq$ 2.0	2.0	3.0	0.3	1.3	1.8	2.0	2.0	3.0
Single Diagonal Sheathing, Chorded	L/b $\leq$ 2.0	2.5	3.5	0.2	1.4	2.1	2.5	2.5	3.5
Single Diagonal Sheathing, Unchorded	L/b $\leq$ 2.0	2.0	3.0	0.3	1.3	1.8	2.0	2.0	3.0
Straight Sheathing Over Diagonal Sheathing, Chorded	L/b $\leq$ 2.0	3.0	4.0	0.2	1.5	2.5	3.0	3.0	4.0
Straight Sheathing Over Diagonal Sheathing, Unchorded	L/b $\leq$ 2.0	2.5	3.5	0.3	1.4	2.1	2.5	2.5	3.5
Double Diagonal Sheathing, Chorded	L/b $\leq$ 2.0	3.0	4.0	0.2	1.5	2.5	3.0	3.0	4.0
Double Diagonal Sheathing, Unchorded	L/b $\leq$ 2.0	2.5	3.5	0.2	1.4	2.1	2.5	2.5	3.5

1. Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting lateral loads.
2. Linear interpolation shall be permitted for intermediate values of aspect ratio.
3. For diaphragm components with aspect ratios between maximum listed values and 4.0, deformation ratios shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.
4. Actions on connectors not listed in this table shall be considered force-controlled.

Table 8-4 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Wood Components (continued)

		Modeling Parameters			Acceptance Criteria				
					Acceptable Deformation Ratio Δ / Δ_y				
					Performance Level				
		$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio		Component Type			
						Primary		Secondary	
		d	e	c	IO	LS	CP	LS	CP
Wood Structural Panel, Blocked, Chorded[2]	L/b ≤ 3	4.0	5.0	0.3	1.8	3.3	4.0	4.0	5.0
	L/b = 4	3.0	4.0	0.3	1.5	2.5	3.0	3.0	4.0
Wood Structural Panel, Unblocked, Chorded[2]	L/b ≤ 3	3.0	4.0	0.3	1.5	2.5	3.0	3.0	4.0
	L/b = 4	2.5	3.5	0.3	1.4	2.1	2.5	2.5	3.5
Wood Structural Panel, Blocked, Unchorded[2]	L/b ≤ 2.5	3.0	4.0	0.3	1.5	2.5	3.0	3.0	4.0
	L/b = 3.5	2.5	3.5	0.3	1.4	2.1	2.5	2.5	3.5
Wood Structural Panel, Unblocked, Unchorded[2]	L/b ≤ 2.5	2.5	3.5	0.4	1.4	2.1	2.5	2.5	3.5
	L/b = 3.5	2.0	3.0	0.4	1.3	1.8	2.0	2.0	3.0
Wood Structural Panel Overlay On Sheathing, Chorded[2]	L/b ≤ 3	3.0	4.0	0.3	1.5	2.5	3.0	3.0	4.0
	L/b = 4	2.5	3.5	0.3	1.4	2.1	2.5	2.5	3.5
Wood Structural Panel Overlay On Sheathing, Unchorded[2]	L/b ≤ 2.5	2.5	3.5	0.4	1.4	2.1	2.5	2.5	3.5
	L/b = 3.5	2.0	3.0	0.4	1.3	1.8	2.0	2.0	3.0
Connections[4]									
Nails—Wood to Wood		7.0	8.0	0.2	2.5	5.5	7.0	7.0	8.0
Nails—Metal to Wood		5.5	7.0	0.2	2.1	4.4	5.5	5.5	7.0
Screws—Wood to Wood		2.5	3.0	0.2	1.4	2.1	2.5	2.5	3.0
Screws—Wood to Metal		2.3	2.8	0.2	1.3	2.0	2.3	2.3	2.8
Lag Bolts—Wood to Wood		2.8	3.2	0.2	1.5	2.4	2.8	2.8	3.2
Lag Bolts—Metal to Wood		2.5	3.0	0.2	1.4	2.1	2.5	2.5	3.0
Bolts—Wood to Wood		3.0	3.5	0.2	1.5	2.5	3.0	3.0	3.5
Bolts—Metal to Wood		2.8	3.3	0.2	1.5	2.4	2.8	2.8	3.3

1. Shear wall components with aspect ratios exceeding maximum listed values shall not be considered effective in resisting lateral loads.
2. Linear interpolation shall be permitted for intermediate values of aspect ratio.
3. For diaphragm components with aspect ratios between maximum listed values and 4.0, deformation ratios shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.
4. Actions on connectors not listed in this table shall be considered force-controlled.

8.5.5 Diagonal Lumber Sheathing Shear Walls

8.5.5.1 Stiffness

The deflection of diagonal lumber sheathed shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.5.1 Stiffness

Diagonal lumber sheathed shear walls are stiffer and stronger than horizontal sheathed shear walls. They also provide greater stiffness for deflection control, and thereby greater damage control.

8.5.5.2 Strength

The yield capacity of diagonal sheathing shall be determined in accordance with Section 8.3.2.

C8.5.5.2 Strength

The yield capacity of diagonal sheathing is dependent on the width of the boards, the spacing of the studs, the size of nails, the number of nails per board, and the boundary conditions. Allowable capacities are listed for various configurations in *WWPA* (1996).

8.5.5.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.5.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

8.5.6 Vertical Wood Siding Shear Walls

8.5.6.1 Stiffness

The deflection of vertical wood siding shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.6.1 Stiffness

Vertical wood siding has a very low lateral-force-resistance capacity and is very flexible. The strength and stiffness degrade with cyclic loading. These shear walls are suitable only where earthquake shear loads are very low and deflection control is not needed.

8.5.6.2 Strength

The yield capacity of vertical wood siding shear walls shall be determined in accordance with Section 8.3.2.

C8.5.6.2 Strength

The yield capacity of vertical wood siding is dependent on the width of the boards, the spacing of the studs, the spacing of blocking, and the size, number, and spacing of the nails. The nail couple method *WWPA* (1996) can be used to calculate the capacity of vertical wood siding in a manner similar to the method used for horizontal siding.

8.5.6.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.6.4 Connections

The presence of connections between parts of the vertical wood siding shear wall assembly and other elements of the lateral-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 3. In the absence of connections, connections shall be provided in accordance with Section 8.5.1.

C8.5.6.4 Connections

The load capacity of the vertical siding is low, which makes the capacity of connections between the shear wall and the other elements of less concern (see Section 8.3.2.2.2).

8.5.7 Wood Siding over Horizontal Sheathing Shear Walls

8.5.7.1 Stiffness

The deflection of wood siding over horizontal sheathing shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.7.1 Stiffness

Double-layer horizontal sheathed shear walls are stiffer and stronger than single-layer horizontal sheathed shear walls. These shear walls are often suitable for resisting earthquake shear loads that are low to moderate in magnitude. They also provide greater stiffness for deflection control and, thereby, greater damage control.

8.5.7.2 Strength

The yield capacity of wood siding over horizontal sheathing shall be determined in accordance with Section 8.3.2.

C8.5.7.2 Strength

This capacity is dependent on the width of the boards, the spacing of the studs, the size, number, and spacing of the nails, and the location of joints.

8.5.7.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.7.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

8.5.8 Wood Siding over Diagonal Sheathing

8.5.8.1 Stiffness

The deflection of these shear walls shall be calculated in accordance with Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.8.1 Stiffness

Horizontal wood siding over diagonal sheathing will provide stiff, strong shear walls. These shear walls are often suitable for resisting earthquake shear loads that are moderate in magnitude. They also provide good stiffness for deflection control and damage control.

8.5.8.2 Strength

The yield capacity of wood siding over diagonal sheathing shall be determined in accordance with Section 8.3.2.

C8.5.8.2 Strength

The capacity of wood siding over diagonal sheathing is dependent on the width of the boards, the spacing of the studs, the size, number, and spacing of the nails, the location of joints, and the boundary conditions.

8.5.8.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.8.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

8.5.9 Wood Structural Panel Sheathing

8.5.9.1 Stiffness

The deflection of wood structural shear walls at yield shall be determined using Equation (8-2):

$$\Delta_y = 8\, v_y\, h^3/(E\, A\, b) + v_y\, h/(G\, t) + 0.75h\, e_n + (h/b)d_a \tag{8-2}$$

where:

v_y	=	Shear at yield in the direction under consideration in lb/ft.
h	=	Shear wall height, ft.
E	=	Modulus of elasticity of boundary member, psi
A	=	Area of boundary member cross section, in.^{2}
b	=	Shear wall width, ft.
G	=	Modulus of rigidity of wood structural panel, psi
t	=	Effective thickness of wood structural panel, in.
d_a	=	Deflection at yield of tie-down anchorage or deflection at load level to anchorage at end of wall, determined by anchorage details and dead load, in.
e_n	=	Nail deformation at yield load per nail, in. Values listed are for Structural I panels; multiply by 1.2 for all other panel grades. For 6d nails at yield, e_n = 0.13 For 8d nails at yield, e_n = 0.08 For 10d nails at yield, e_n = 0.08

Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.9.1 Stiffness

The response of wood structural panel shear walls is dependent on the thickness of the wood structural panels, the height-to-width (*h/b*) ratio, the nailing pattern, and other factors. Values for modulus of rigidity, *G*, and effective thickness, *t*, for various sheathing materials are contained in *APA* (1995 and 1997).

8.5.9.2 Strength

Where the strength of wood structural panel shear walls is derived from testing or based on principles of mechanics, the requirements in Section 8.4.2.2 shall apply. Where the strength is calculated using LRFD-based values in an approved code, the expected strength shall be taken as the nominal strength (ϕ = 1) times 0.8 for plywood, or 0.65 for wood structural panels other than plywood.

Conversion from tabulated allowable stress values in accordance with Section 8.3.2.5 shall not be permitted for wood structural panel shear walls, but approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 8.3.2.5 where the strength of a shear wall is computed using principles of mechanics.

C8.5.9.2 Strength

Shear capacities of wood structural panel shear walls are primarily dependent on the nailing at the plywood panel edges, and the thickness and grade of the plywood.

The ratio between yield strength and nominal design strength is based on preliminary results of cyclic testing performed at the University of California at Irvine in the year 2000. This testing has shown that the ratio between yield strength and design strength is lower for sheathing materials other than plywood (e.g., oriented strand board).

LRFD-based design values for various configurations are listed in *AF&PA LRFD* and *BSSC* (2000). A method for calulating the capacity of wood structural shear walls based on accepted nail values is provided in Tissell (1993). For this method, use LRFD-based fastener strengths. Due to the differences in load duration / time-effect factors between the allowable stress and LRFD formats, direct conversion of shear wall tables using the method outlined in Section 8.3.2.5 is not permitted. However, the tabulated LRFD design values, with $\phi = 1$, are intended to be 2.0 times the associated allowable stress design values.

8.5.9.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described in Equation (8-1), and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.9.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

8.5.10 Stucco on Studs, Sheathing, or Fiberboard

8.5.10.1 Stiffness

The deflection of stucco on studs, sheathing, or fiberboard shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.10.1 Stiffness

Stucco is brittle, and the lateral-force-resisting capacity of stucco shear walls is low. The walls are stiff until cracking occurs, but he strength and stiffness degrade under cyclic loading. These shear walls are suitable only where earthquake shear loads are low.

8.5.10.2 Strength

The yield capacity of stucco on studs, sheathing, or fiberboard shall be determined in accordance with Section 8.3.2.

C8.5.10.2 Strength

This capacity is dependent on the attachment of the stucco netting to the studs and the embedment of the netting in the stucco.

8.5.10.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.10.4 Connections

The connection between the stucco netting and the framing shall be investigated. The connections between the shear wall and foundation, and between the shear wall and other elements of the lateral-force-resisting system, shall be considered in accordance with Section 8.5.1.

C8.5.10.4 Connections

Of less concern is the connection of the stucco to the netting. Unlike plywood, the tensile capacity of the stucco material (portland cement) rather than the connections, will often govern failure. See Section 8.3.2.2.2.

8.5.11 Gypsum Plaster on Wood Lath

8.5.11.1 Stiffness

The deflection of gypsum plaster on wood lath shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.11.1 Stiffness

Gypsum plaster shear walls are similar to stucco, except their strength is lower. As is the case for stucco, the walls are stiff until failure, but the strength and stiffness degrade under cyclic loading. These shear walls are suitable only where earthquake shear loads are very low.

8.5.11.2 Strength

The yield capacity of gypsum plaster shall be determined in accordance with Section 8.3.2.

8.5.11.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.11.4 Connections

The presence of connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 3. If connections are absent, they shall be provided in accordance with Section 8.5.1.

C8.5.11.4 Connections

The tensile and bearing capacity of the plaster, rather than the connections, will often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the lateral-force-resisting system of less concern.

8.5.12 Gypsum Plaster on Gypsum Lath

8.5.12.1 Stiffness

The deflection of gypsum plaster on gypsum lath shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.12.1 Stiffness

Gypsum plaster on gypsum lath is similar to gypsum wallboard (see Section 8.5.13).

8.5.12.2 Strength

The yield capacity of gypsum plaster on gypsum lath shear walls shall be determined in accordance with Section 8.3.2.

8.5.12.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.12.4 Connections

The presence of connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 3. If connections are absent, they shall be provided in accordance with Section 8.5.1.

C8.5.12.4 Connections

The tensile and bearing capacity of the plaster, rather than the connections, will often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the lateral-force-resisting system of less concern.

8.5.13 Gypsum Wallboard

8.5.13.1 Stiffness

The deflection of gypsum wallboard shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.13.1 Stiffness

Gypsum wallboard has a very low lateral-force-resisting capacity, but is relatively stiff until cracking occurs. The strength and stiffness degrade under cyclic loading. These shear walls are suitable only where earthquake shear loads are very low.

8.5.13.2 Strength

The yield capacity of gypsum wallboard shear walls shall be determined in accordance with Section 8.3.2.

C8.5.13.2 Strength

The default capacity listed in Section Table 8-1 is for typical 7-inch nail spacing of 1/2-inch or 5/8-inch-thick panels with 4d or 5d nails. Higher capacities can be used if closer nail spacing, multilayers of gypsum board, and/or the presence of blocking at all panel edges is verified.

8.5.13.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.13.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

8.5.14 Gypsum Sheathing

8.5.14.1 Stiffness

The deflection of gypsum sheathed shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.14.1 Stiffness

Gypsum sheathing is similar to gypsum wallboard (see Section 8.5.13.1).

8.5.14.2 Strength

The yield capacity of gypsum wallboard shear walls shall be determined in accordance with Section 8.3.2.

C8.5.14.2 Strength

The default capacity listed in Table 8-1 is based on typical 7-inch nail spacing of 1/2-inch or 5/8-inch-thick panels with 4d or 5d nails. Higher capacities can be used if closer nail spacing, multilayers of gypsum board, and/or the presence of blocking at all panel edges is verified.

8.5.14.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.14.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

8.5.15 Plaster on Metal Lath

8.5.15.1 Stiffness

The deflection of plaster on metal lath shear walls shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.15.1 Stiffness

Plaster on metal lath is similar to plaster on wood lath, and the lateral-force-resisting capacity of these shear walls is low. The walls are stiff until cracking occurs, but the strength and stiffness degrade under cyclic loading. These shear walls are suitable only where earthquake shear loads are low.

8.5.15.2 Strength

The yield capacity of plaster on metal lath shear walls shall be determined in accordance with Section 8.3.2.

8.5.15.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.15.4 Connections

The presence of connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 3. If connections are absent, they shall be provided in accordance with Section 8.5.1.

C8.5.15.4 Connections

The tensile and bearing capacity of the plaster, rather than the connections, will often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the lateral-force-resisting system of less concern.

8.5.16 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking

8.5.16.1 Stiffness

The deflection of horizontal lumber sheathing with cut-in braces or diagonal blocking shear walls shall be calculated using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2.

C8.5.16.1 Stiffness

This assembly is similar to horizontal sheathing without braces, except that the cut-in braces or diagonal blocking provide higher stiffness at initial loads. After the braces or blocking fail (at low loads), the behavior of the wall is the same as with horizontal sheathing without braces. The strength and stiffness degrade under cyclic loading.

8.5.16.2 Strength

The yield capacity of horizontal sheathing or siding shall be determined in accordance with Section 8.3.2.

8.5.16.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.16.4 Connections

The connections between the parts of the shear wall assembly and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

C8.5.16.4 Connections

The capacity and ductility of these connections will often determine the failure mode as well as the capacity of the assembly. Ductile connections with sufficient capacity will give acceptable performance (see Section 8.3.2.2.2).

8.5.17 Fiberboard or Particleboard Sheathing

8.5.17.1 Stiffness

For structural particleboard sheathing, see Section 8.5.9. The deflection of shear walls sheathed in nonstructural particleboard shall be determined using Equation (8-1). Properties used to compute shear wall deflection and stiffness shall be based on Section 8.3.2. Fiberboard sheathing shall not be used to resist lateral loads.

C8.5.17.1 Stiffness

Fiberboard sheathing is very weak, lacks stiffness, and is unable to resist lateral loads. Particleboard comes in two varieties: one is similar to structural panels, the other (nonstructural) is slightly stronger than gypsum board but more brittle. Nonstructural particleboard should only be used where earthquake loads are very low.

8.5.17.2 Strength

The strength of structural particleboard shall be based on Section 8.5.9. The strength of nonstructural fiberboard or particleboard sheathed walls shall be determined in accordance with Section 8.3.2.

C8.5.17.2 Strength

Fiberboard has very low strength.

8.5.17.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria for primary and secondary components shall be taken from Table 8-4.

8.5.17.4 Connections

The connections between parts of structural particleboard shear wall assemblies and other elements of the lateral-force-resisting system shall be considered in accordance with Section 8.5.1.

The presence of connections between parts of nonstructural particleboard shear wall assemblies and other elements of the lateral-force-resisting system shall be verified. If connections are present, they need not be considered in the analysis conducted in accordance with Chapter 3. If connections are absent, they shall be provided in accordance with Section 8.5.1.

C8.5.17.4 Connections

The capacity and ductility of the connections in structural particleboard shear walls will often determine the failure mode as well as the capacity of the assembly. Ductile connections with sufficient capacity will give acceptable performance. The tensile and bearing capacity of the nonstructural particleboard, rather than the connections, will often govern failure. The relatively low strength of this material makes connections between parts of the shear wall assembly and the other elements of the lateral-force-resisting system of less concern.

8.5.18 Light Gage Metal Frame Shear Walls

8.5.18.1 Plaster on Metal Lath

The criteria for plaster on metal lath shall be based on Section 8.5.15.

8.5.18.2 Gypsum Wallboard

The criteria for gypsum wallboard shall be based on Section 8.5.13.

8.5.18.3 Wood Structural Panels

The criteria wood structural panels shall be based on Section 8.5.9. The expected strength values of fasteners shall be calculated in accordance with Section 8.3.2.5, based on approved data.

8.6 Wood Diaphragms

8.6.1 General

The expected strength of wood diaphragms, Q_{CE}, shall be taken as the yield capacity of the diaphragm assembly determined in accordance with Sections 8.6.3 through 8.6.10. The expected strength, Q_{CE}, of braced horizontal diaphragm systems shall be determined in accordance with Section 8.8.1.

The effects of openings in wood diaphragms shall be considered. Chords and collectors shall be added to provide sufficient load capacity around openings to meet the requirements for diaphragms.

Connections between diaphragms and other components including shear walls, drag struts, collectors, cross ties, and out-of-plane anchors shall be considered in accordance with Section 8.4.3, and designed for forces calculated in accordance with Chapter 3.

C8.6.1 General

The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size and amount of fasteners, existence of perimeter chord or flange members, and the ratio of span length to width of the diaphragm.

The presence of any but small openings in wood diaphragms will cause a reduction in the stiffness and yield capacity of the diaphragm due to a reduced length of diaphragm available to resist lateral forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings will reduce the loss in stiffness of the diaphragm and limit damage in the area of the openings. See *ATC-7* and Tissell and Elliott (1997) for a discussion of the effects of openings in wood diaphragms.

The presence of chords at the perimeter of a diaphragm will significantly reduce the diaphragm deflection due to bending, and increase the stiffness of the diaphragm over that of an unchorded diaphragm. However, the increase in stiffness due to chords in a single straight-sheathed diaphragm is minimal due to the flexible nature of these diaphragms.

8.6.2 Types of Wood Diaphragms

8.6.2.1 Existing Wood Diaphragms

8.6.2.1.1 Single Straight Sheathing

Single straight-sheathed diaphragms shall include diaphragms with sheathing laid perpendicular to the framing members.

C8.6.2.1.1 Single Straight Sheathing

Typically, single straight-sheathed diaphragms consist of 1"x sheathing laid perpendicular to the framing members; 2"x or 3"x sheathing may also be present. The sheathing serves the dual purpose of supporting gravity loads and resisting shear forces in the diaphragm. Most often, 1"x sheathing is nailed with 8d or 10d nails, with two or more nails per sheathing board at each support. Shear forces perpendicular to the direction of the sheathing are resisted by the nail couple. Shear forces parallel to the direction of the sheathing are transferred through the nails in the supporting joists or framing members below the sheathing joints.

8.6.2.1.2 Double Straight Sheathing

Double straight-sheathed diaphragms shall include diaphragms with one layer of sheathing laid perpendicular to the framing members and a second layer of sheathing laid either perpendicular or parallel to the first layer.

C8.6.2.1.2 Double Straight Sheathing

Construction of double straight-sheathed diaphragms is the same as that for single straight-sheathed diaphragms, except that an upper layer of straight sheathing is laid over the lower layer of sheathing. The upper sheathing can be placed either perpendicular or parallel to the lower layer of sheathing. If the upper layer of sheathing is parallel to the lower layer, the board joints are usually offset sufficiently that nails at joints in the upper layer of sheathing are driven into a common sheathing board below, with sufficient edge distance. The upper layer of sheathing is nailed to the framing members through the lower layer of sheathing.

8.6.2.1.3 Single Diagonal Sheathing

Single diagonally sheathed diaphragms shall include diaphragms with sheathing laid at a 45-degree angle to the framing members.

C8.6.2.1.3 Single Diagonal Sheathing

Typically, 1"x sheathing is laid at an approximate 45-degree angle to the framing members. In some cases 2"x sheathing may also be used. The sheathing supports gravity loads and resists shear forces in the diaphragm. Commonly, 1"x sheathing is nailed with 8d nails, with two or more nails per board at each support. The recommended nailing for diagonally sheathed diaphragms is published in *WWPA* (1996) and *BSSC* (2000). The shear capacity of the diaphragm is dependent on the size and quantity of the nails at each sheathing board.

8.6.2.1.4 Diagonal Sheathing with Straight Sheathing or Flooring Above

Diagonal sheathing with straight sheathing or flooring above shall include diaphragms with sheathing laid at a 45-degree angle to the framing members, with a second layer of straight sheathing or wood flooring laid on top of the diagonal sheathing at a 90-degree angle to the framing members.

C8.6.2.1.4 Diagonal Sheathing with Straight Sheathing or Flooring Above

Typically, these consist of a lower layer of 1"x diagonal sheathing laid at a 45-degree angle to the framing members, with a second layer of straight sheathing or wood flooring laid on top of the diagonal sheathing at a 90-degree angle to the framing members. Both layers of sheathing support gravity loads, and resist shear forces in the diaphragm. Sheathing boards are commonly connected with two or more 8d nails per board at each support.

8.6.2.1.5 Double Diagonal Sheathing

Double diagonally sheathed diaphragms shall include diaphragms with one layer of sheathing laid at a 45-degree angle to the framing members and a second layer of sheathing laid at a 90-degree angle to the first layer.

C8.6.2.1.5 Double Diagonal Sheathing

Typically, double diagonally sheathed diaphragms consist of a lower layer of 1"x diagonal sheathing with a second layer of 1"x diagonal sheathing laid at a 90-degree angle to the lower layer. The sheathing supports gravity loads and resists shear forces in the diaphragm. The sheathing is commonly nailed with 8d nails, with two or more nails per board at each support. The recommended nailing for double diagonally sheathed diaphragms is published in *WWPA* (1996).

8.6.2.1.6 Wood Structural Panel Sheathing

Wood structural panel-sheathed diaphragms shall include diaphragms with plywood, or other wood structural panels as defined in this standard, fastened to the framing members.

C8.6.2.1.6 Wood Structural Panel Sheathing

Typically, these consist of wood structural panels, such as plywood or oriented strand board, placed on framing members and nailed in place. Different grades and thicknesses of wood structural panels are commonly used, depending on requirements for gravity load support and shear capacity. Edges at the ends of the wood structural panels are usually supported by the framing members. Edges at the sides of the panels can be blocked or unblocked. In some cases, tongue and groove wood structural panels are used. Nailing patterns and nail size can vary greatly. Nail spacing is commonly in the range of 3 to 6 inches on center at the supported and blocked edges of the panels, and 10 to 12 inches on center at the panel infield. Staples are sometimes used to attach the wood structural panels.

8.6.2.1.7 Braced Horizontal Diaphragms

Braced horizontal diaphragms shall include diaphragms with a horizontal truss system at the floor or roof level of the building.

C8.6.2.1.7 Braced Horizontal Diaphragms

Typically, these consist of “X” rod bracing and wood struts forming a horizontal truss system at the floor or roof levels of the building. The “X” bracing usually consists of steel rods drawn taut by turnbuckles or nuts. The struts usually consist of wood members, which may or may not be part of the gravity-load-bearing system of the floor or roof. The steel rods function as tension members in the horizontal truss, while the struts function as compression members. Truss chords (similar to diaphragm chords) are needed to resist bending in the horizontal truss system.

8.6.2.2 Enhanced Wood Diaphragms

Enhanced wood diaphragms shall include existing diaphragms rehabilitated by an approved method.

C8.6.2.2 Enhanced Wood Diaphragms

Possible rehabilitation methods for wood diaphragms are described in Sections C8.6.2.2.1 through C8.6.2.2.3.

C8.6.2.2.1 Wood Structural Panel Overlays on Straight or Diagonal Sheathing

Existing sheathed diaphragms may be overlaid with new wood structural panels. Nails or staples may be used to connect the new structural panels to the existing diaphragms. Nails should be of sufficient length to provide the required embedment into framing members below the sheathing.

These diaphragms typically consist of new wood structural panels placed over existing straight or diagonal sheathing and nailed or stapled to the existing framing members through the existing sheathing. If the new overlay is nailed to the existing framing members only—without nailing at the panel edges perpendicular to the framing—the response of the new overlay will be similar to that of an unblocked wood structural panel diaphragm.

If a stronger and stiffer diaphragm is desired, the joints of the new wood structural panel overlay should be placed parallel to the joints of the existing sheathing, with the overlay nailed or stapled to the existing sheathing. The edges of the new wood structural panels should be offset from the joints in the existing sheathing below by a sufficient distance that the new nails may be driven into the existing sheathing without splitting the sheathing. If the new panels are nailed at all edges as described above, the response of the new overlay will be similar to that of a blocked wood structural panel diaphragm. As an alternative, new blocking may be installed below all panel joints perpendicular to the existing framing members.

Because the joints of the overlay and the joints of the existing sheathing may not be offset consistently without cutting the panels, it may be advantageous to place the wood structural panel overlay at a 45-degree angle to the existing sheathing. If the existing diaphragm is straight-sheathed, the new overlay should be placed at a 45-degree angle to the existing sheathing and joists. If the existing diaphragm is diagonally sheathed, the new wood structural panel overlay should be placed perpendicular to the existing joists at a 45-degree angle to the diagonal sheathing. Nails should be driven into the existing sheathing with sufficient edge distance to prevent splitting of the existing sheathing. At boundaries, nails should be of sufficient length to penetrate the sheathing into the framing below. New structural panel overlays shall be connected to shear walls or vertical bracing elements to ensure the effectiveness of the added panel.

Care should be exercised when placing new wood structural panel overlays on existing diaphragms. The changes in stiffness and dynamic characteristics of the diaphragm may have negative effects by causing increased forces in other components or elements. The increased stiffness and the associated increase in dynamic forces may not be desirable in some diaphragms for certain performance levels.

C8.6.2.2.2 Wood Structural Panel Overlays on Existing Wood Structural Panels

Existing wood structural panel diaphragms may be overlaid with new wood structural panels. Panel joints should be offset, or the overlay should be placed at a 45-degree angle to the existing wood structural panels.

The placement of a new overlay over an existing diaphragm should follow the same construction methods and procedures as those used for straight-sheathed and diagonally sheathed diaphragms (see Section C8.6.2.2.1).

C8.6.2.2.3 Increased Attachment

The nailing or attachment of the existing sheathing to the supporting framing may be increased. Nailing or attachment to the supporting framing should be increased and blocking for the diaphragm at the plywood joints should be added.

For straight-sheathed diaphragms, the increase in shear capacity will be minimal. Double straight-sheathed diaphragms with minimal nailing in the upper or both layers of sheathing may be enhanced significantly by adding new nails or staples to the existing diaphragm. The same is true for diaphragms that are single diagonally sheathed, double diagonally sheathed, or single diagonally sheathed with straight sheathing or flooring.

In some cases, increased nailing at the plywood panel infield may also be required. If the required shear capacity or stiffness is greater than that which can be provided by increased attachment, a new overlay on the existing diaphragm may be required to provide the desired enhancement.

8.6.2.3 New Wood Diaphragms

8.6.2.3.1 New Wood Structural Panel Sheathing

New wood structural panel sheathed diaphragms shall include new wood structural panels connected to new framing members, or connected to existing framing members after existing sheathing has been removed.

C8.6.2.3.1 New Wood Structural Panel Sheathing

Typically, these consist of wood structural panels—such as plywood or oriented strand board—nailed or stapled to existing framing members after existing sheathing has been removed. Different grades and thicknesses of wood structural panels can be used, depending on the requirements for gravity load support and diaphragm shear capacity. In most cases, the panels are placed with the long dimension perpendicular to the framing members, and panel edges at the ends of the panels are supported by, and nailed to, the framing members. Edges at the sides of the panels can be blocked or unblocked, depending on the shear capacity and stiffness required in the new diaphragm. Wood structural panels can be placed in various patterns as shown in *AF&PA LRFD* and *BSSC* (2000).

8.6.2.3.2 New Single Diagonal Sheathing

New single diagonally sheathed wood diaphragms shall include new sheathing laid at a 45-degree angle and connected to the existing framing members.

8.6.2.3.3 New Double Diagonal Sheathing

New double diagonally sheathed wood diaphragms shall include diaphragms with new sheathing laid at a 45-degree angle to the existing framing members with a second layer of sheathing laid at a 90-degree angle to the first layer.

8.6.2.3.4 New Braced Horizontal Diaphragms

New braced horizontal diaphragms shall include a new horizontal truss system attached to the existing framing at the floor or roof level of the building.

C8.6.2.3.4 New Braced Horizontal Diaphragms

Because new horizontal truss systems will induce new forces on existing framing members, it may be more economical to design floor or roof sheathing as a diaphragm. This eliminates the potential need to strengthen wood members at the compression struts. Braced horizontal diaphragms are more feasible where sheathing cannot provide sufficient shear capacity, or where diaphragm openings reduce the shear capacity of the diaphragm and additional shear capacity is needed.

8.6.3 Single Straight Sheathing

8.6.3.1 Stiffness

The deflection of straight-sheathed diaphragms shall be calculated using Equation (8-3):

$$\Delta_y = v_y L / (2G_d) \qquad (8\text{-}3)$$

where:

G_d = Diaphragm shear stiffness from Table 8-2

L = Diaphragm span, distance between shear walls or collectors

v_y = Shear at yield in the direction under consideration

Δy = Calculated diaphragm deflection at yield

Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.3.1 Stiffness

Straight sheathed diaphragms are characterized by high flexibility with a long period of vibration. These diaphragms are suitable for low shear conditions where control of diaphragm deflections is not needed to attain the desired performance levels.

8.6.3.2 Strength

The yield capacity of straight-sheathed diaphragms shall be determined in accordance with Section 8.3.2.

C8.6.3.2 Strength

The yield capacity of straight-sheathed diaphragms is dependent on the size, number, and spacing between the nails at each sheathing board, and the spacing of the supporting framing members. The shear capacity of straight-sheathed diaphragms can be calculated using the nail-couple method. See *ATC-7* for a discussion of calculating the shear capacity of straight-sheathed diaphragms.

8.6.3.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

C8.6.3.3 Acceptance Criteria

Deformation acceptance criteria will largely depend on the allowable deformations for other structural and nonstructural components and elements that are laterally supported by the diaphragm. Allowable deformations must also be consistent with the permissible damage state of the diaphragm.

8.6.3.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

C8.6.3.4 Connections

The load capacity of connections between diaphragms and shear walls or other vertical elements, as well as diaphragm chords and shear collectors, is critical.

8.6.4 Double Straight Sheathing

8.6.4.1 Stiffness

The deflection of double straight-sheathed diaphragms shall be calculated using Equation (8-3). Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.4.1 Stiffness

The double-sheathed system will provide a significant increase in stiffness over a single straight-sheathed diaphragm, but very little test data is available on the stiffness and strength of these diaphragms. Both layers of straight sheathing must have sufficient nailing, and the joints of the top layer must be either offset or perpendicular to the bottom layer.

8.6.4.2 Strength

The yield capacity of double straight-sheathed diaphragms shall be determined in accordance with Section 8.3.2.

C8.6.4.2 Strength

The strength and stiffness of double straight-sheathed diaphragms is highly dependent on the nailing of the upper layer of sheathing. If the upper layer has minimal nailing, the increase in strength and stiffness over a single straight-sheathed diaphragm may be slight. If the upper layer of sheathing has nailing similar to that of the lower layer of sheathing, the increase in strength and stiffness will be significant.

8.6.4.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.4.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.5 Single Diagonal Sheathing

8.6.5.1 Stiffness

The deflection of single diagonally sheathed diaphragms shall be calculated using Equation (8-3). Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.5.1 Stiffness

Single diagonally sheathed diaphragms are significantly stiffer than straight-sheathed diaphragms, but are still quite flexible.

8.6.5.2 Strength

The yield capacity for diagonally sheathed wood diaphragms with chords shall be determined in accordance with Section 8.3.2.

C8.6.5.2 Strength

Diagonally sheathed diaphragms are usually capable of resisting moderate shear loads.

Because the diagonal sheathing boards function in tension and compression to resist shear forces in the diaphragm, and the boards are placed at a 45-degree angle to the chords at the ends of the diaphragm, the component of the force in the sheathing boards that is perpendicular to the axis of the end chords will create a bending force in the end chords. If the shear in diagonally sheathed diaphragms is limited to approximately 300 pounds per foot or less, bending forces in the end chords are usually neglected. If shear forces exceed 300 pounds per foot, the end chords should be designed or reinforced to resist bending forces from the sheathing. See *ATC-7* for methods of calculating the shear capacity of diagonally sheathed diaphragms.

8.6.5.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.5.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.6 Diagonal Sheathing with Straight Sheathing or Flooring Above

8.6.6.1 Stiffness

The deflection of diagonally sheathed diaphragms with straight sheathing or flooring above shall be calculated using Equation (8-3). Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.6.1 Stiffness

Straight sheathing or flooring over diagonal sheathing provides a significant increase in stiffness over single-sheathed diaphragms. The increased stiffness of these diaphragms may make them suitable where Life Safety or Immediate Occupancy Structural Performance Levels are desired.

8.6.6.2 Strength

The yield capacity of diagonally sheathed diaphragms with straight sheathing or flooring above shall be determined in accordance with Section 8.3.2.

C8.6.6.2 Strength

Shear capacity is dependent on the nailing of the diaphragm. The strength and stiffness of diagonally sheathed diaphragms with straight sheathing above is highly dependent on the nailing of both layers of sheathing. Both layers of sheathing should have at least two 8d common nails per board at each support.

8.6.6.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.6.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.7 Double Diagonal Sheathing

8.6.7.1 Stiffness

The deflection of double diagonally sheathed diaphragms shall be calculated using Equation (8-3). Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.7.1 Stiffness

Double diagonally sheathed diaphragms have greater stiffness than diaphragms with single diagonal sheathing. The response of these diaphragms is similar to the response of diagonally sheathed diaphragms with straight sheathing overlays.

The increased stiffness of these diaphragms may make them suitable where Life Safety or Immediate Occupancy Structural Performance Levels are desired.

8.6.7.2 Strength

The yield capacity of double diagonally sheathed wood diaphragms shall be determined in accordance with Section 8.3.2.

C8.6.7.2 Strength

Shear capacity is dependent on the nailing of the diaphragm, but these diaphragms are usually suitable for moderate to high shear loads.

Yield shear capacities are similar to those of diagonally sheathed diaphragms with straight sheathing overlays. The sheathing boards in both layers of sheathing should be nailed with at least two 8d common nails at each support. The presence of a double layer of diagonal sheathing will eliminate the bending forces that single diagonally sheathed diaphragms impose on the chords at the ends of the diaphragm. As a result, the bending capacity of the end chords does not have an effect on the shear capacity and stiffness of the diaphragm.

8.6.7.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.7.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.8 Wood Structural Panel Sheathing

8.6.8.1 Stiffness

The deflection of blocked and chorded wood structural panel diaphragms with constant nailing across the diaphragm length shall be determined using Equation (8-4):

$$\Delta_y = 5\, v_y\, L^3 / (8EAb) + v_y\, L / (4Gt) + 0.188\, L\, e_n + \Sigma(\Delta_c X) / (2b) \qquad (8\text{-}4)$$

where:

A = Area of diaphragm chords cross-section, in.^{2}

b = Diaphragm width, ft.

E = Modulus of elasticity of diaphragm chords, psi

e_n = Nail deformation at yield load per nail, in. Values listed are for Structural I panels; multiply by 1.2 for all other panel grades.
For 6d nails at yield, e_n = 0.13
For 8d nails at yield, e_n = 0.08
For 10d nails at yield, e_n = 0.08

G = Modulus of rigidity of wood structural panels, psi.

L = Diaphragm span, distance between shear walls or collectors, ft.

t = Effective thickness of wood structural panel for shear, in.

v_y = Shear at yield in the direction under consideration, lb/ft.

Δ_y = Calculated deflection of diaphragm at yield, in.

$\Sigma\,(\Delta_c X)$ = Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support

The deflection of blocked and chorded wood structural panel diaphragms with variable nailing across the diaphragm length shall be determined using Equation (8-5):

$$\Delta_y = 5\, v_y\, L^3 / (8EAb) + v_y\, L / (4Gt) + 0.376\, L\, e_n + \Sigma(\Delta_c X) / (2b) \qquad (8\text{-}5)$$

The deflection of unblocked diaphragms shall be calculated using Equation (8-3). Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.8.1 Stiffness

The response of wood structural panel diaphragms is dependent on the thickness of the wood structural panels, the length-to-width (L/b) ratio, nailing pattern, and presence of chords in the diaphragm, as well as other factors. Values for modulus rigidity, G, and effective thickness, t, for various sheathing materials are contained in *APA* (1995 and 1997).

In most cases the area of the diaphragm chord equals the area of the continuous wood (or steel) member to which the sheathing is attached. For buildings with wood diaphragms and concrete or masonry walls, however, the area of the diaphragm chord is more difficult to identify and engineering judgment is required. The tension area of the diaphragm chord on both edges of the diaphragm should be used for deflection calculations. Generally, this is conservative as it results in a larger calculated deflection. Use of the tension area of the diaphragm chord may not yield conservative results, however, when calculating the period of the building using Equation (3-8).

8.6.8.2 Strength

The strength of wood structural panel diaphragms shall be determined in accordance with Section 8.4.2.2.

Conversion for tabulated allowable stress values in accordance with Section 8.3.2.5 shall not be permitted for wood structural panel diaphragms, but approved allowable stress values for fasteners shall be permitted to be converted in accordance with Section 8.3.2.5 where the strength of a shear wall is computed using principles of mechanics.

The yield shear capacity of unchorded diaphragms shall be calculated by multiplying the values given for chorded diaphragms by 0.70.

C8.6.8.2 Strength

Shear capacities of wood structural panel diaphragms are primarily dependent on the nailing at the plywood panel edges, and the thickness and grade of the plywood in the diaphragm.

LRFD-based design values for various configurations are listed in *AF&PA LRFD* and *BSSC* (2000). A method for calculating the capacity of wood structural panel diaphragms based on accepted nail values and panel shear strength is provided in Tissell and Elliott (1997). For this method, use LRFD-based fastener strengths. Due to the differences in load duration/time effect factors between the allowable stress and LRFD formats, direct conversion of diaphragm tables using the method outlined in Section 8.3.2.5 is not permitted. However, the tabulated LRFD design values, with $\phi = 1$, are intended to be 2.0 times the associated allowable stress design values.

8.6.8.3 Acceptance Criteria

For linear procedures, m-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.8.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.9 Wood Structural Panel Overlays on Straight or Diagonal Sheathing

8.6.9.1 Stiffness

Placement of the new wood structural panel overlay shall be consistent with Section 8.6.2.2.

The deflection of wood structural panel overlays on straight or diagonally sheathed diaphragms shall be calculated using Equation (8-3). Properties used to compute diaphragm deflection and stiffness shall be based on Section 8.3.2.

C8.6.9.1 Stiffness

The stiffness of existing straight-sheathed diaphragms can be increased significantly by placing a new plywood overlay over the existing diaphragm. The stiffness of existing diagonally sheathed diaphragms and plywood diaphragms will be increased, but not in proportion to the stiffness increase for straight-sheathed diaphragms.

Depending on the nailing of the new overlay, the response of the diaphragm may be similar to that of a blocked or an unblocked diaphragm.

The increased stiffness of these diaphragms may make them suitable where Life Safety or Immediate Occupancy Structural Performance Levels are desired.

8.6.9.2 Strength

Strength of wood structural panel overlays shall be determined in accordance with Section 8.4.2.2. It shall be permitted to take the yield capacity as the value for the corresponding wood structural panel diaphragm without the existing sheathing below, computed in accordance with Section 8.6.8.2.

8.6.9.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.9.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.10 Wood Structural Panel Overlays on Existing Wood Structural Panel Sheathing

8.6.10.1 Stiffness

Diaphragm deflection shall be calculated in accordance with Equation (8-3) or using accepted principles of mechanics. Nails in the upper layer of plywood shall have sufficient embedment in the framing to meet the requirements of *ASCE 16*.

C8.6.10.1 Stiffness

According to Tissell and Elliott (1997), Equation (8-4) is not applicable to two-layer diaphragms, presumably due to the difficulty in estimating the combined nail slip. Diaphragm deflection may be estimated using principles of mechanics that include consideration of nail slip, blocking, and the embedment of nails into the framing.

8.6.10.2 Strength

Yield capacity shall be calculated based on the combined two layers of plywood. The yield shear capacity of the overlay shall be limited to 75% of the values calculated in accordance with Section 8.6.8.2.

8.6.10.3 Acceptance Criteria

For linear procedures, *m*-factors for use with deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, the coordinates of the generalized force-deformation relation, described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

8.6.10.4 Connections

Connections between diaphragms and shear walls and other vertical elements shall be considered in accordance with Section 8.6.1.

8.6.11 Braced Horizontal Diaphragms

Braced horizontal diaphragms shall be considered in accordance with Section 8.8.1.

Connections between members of the horizontal bracing system and shear walls or other vertical elements shall be considered in accordance with Section 8.6.1.

8.7 Wood Foundations

8.7.1 Types of Wood Foundations

Types of wood foundations include wood piling, wood footings, and pole structures. Wood piling shall include friction or end-bearing piles that resist only vertical loads.

C8.7.1 Types of Wood Foundations

1. Wood Piling. Wood piles are generally used with a concrete pile cap and are usually keyed into the base of the concrete cap. The piles are usually treated with preservatives.

Piles are either friction- or end-bearing piles resisting only vertical loads. Piles are generally not able to resist uplift loads because of the manner in which they are attached to the pile cap. The piles may be subjected to lateral loads from seismic loading, which are resisted by bending of the piles. The analysis of pile bending is generally based on a pinned connection at the top of the pile, and fixity of the pile at some depth established by the geotechnical engineer. However, it should be evaluated with consideration for the approximate nature of the original assumption of the depth to point of fixity. Where battered piles are present, the lateral loads can be resisted by the horizontal component of the axial load.

2. Wood Footings. Wood grillage footings, sleepers, skids, and pressure-treated all-wood foundations can be encountered in existing structures. These foundations are highly susceptible to deterioration. The seismic resistance of wood footings is generally very low; they are essentially dependent on friction between the wood and soil for their performance.

3. Pole Structures. Pole structures resist lateral loads by acting as cantilevers fixed in the ground, with the lateral load considered to be applied perpendicular to the pole axis. It is possible to design pole structures to have moment-resisting capacity at floor and roof levels by the use of knee braces or trusses. Pole structures are frequently found on sloping sites. The varying unbraced lengths of the poles generally affect the stiffness and performance of the structure, and can result in unbalanced loads to the various poles along with significant torsional distortion, which must be investigated and evaluated. Additional horizontal and diagonal braces can be used to reduce the flexibility of tall poles or reduce the torsional eccentricity of the structure.

8.7.2 Analysis, Strength, and Acceptance Criteria of Wood Foundations

The expected strength of wood piles shall be computed in accordance with Section 8.4.2.2. Deflection of piles under seismic loads shall be calculated based on an assumed point of fixity. Unless rigidly connected to the pile cap, wood piles shall be taken as pinned at the top.

Flexure and axial loads in wood piles shall be considered deformation-controlled. *m*-factors shall be taken from Table 8-3.

Wood footings shall be thoroughly investigated for the presence of deterioration. Acceptability of soils below wood footings shall be determined in accordance with Chapter 4.

Component and connection strength of pole structures shall be based on Section 8.3. Pole structures shall be modeled as cantilever elements and analyzed in accordance with Chapter 3.

Flexure and axial loads in pole structures shall be considered deformation controlled. *m*-factors shall be taken from Table 8-3. Where concentrically braced diagonals are added to enhance the capacity of the pole structure, reduced *m*-factors taken from Table 8-3 shall be used.

C8.7.2 Analysis, Strength, and Acceptance Criteria of Wood Foundations

The strength of the components, elements, and connections of a pole structure are the same as for a conventional structure.

8.7.3 Rehabilitation Measures

Wood foundations not meeting the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated by approved methods. Wood foundations exhibiting signs of deterioration shall be rehabilitated or replaced.

C8.7.3 Rehabilitation Measures

Wood footings showing signs of deterioration may be replaced with reinforced concrete footings. Wood pole structures can be rehabilitated with the installation of diagonal braces or other supplemental lateral-force-resisting elements. Wood piles may be rehabilitated by the installation of additional piles.

8.8 Other Wood Elements and Components

8.8.1 General

Wood elements and components, other than shear walls, diaphragms, and foundations, shall be considered in accordance with this section. Where an assembly includes wood components and steel rods, the rods shall be considered in accordance with applicable provisions of Chapter 5.

C8.8.1 General

Other wood elements include knee-braced frames, rod-braced frames, and braced horizontal diaphragms, among other systems.

Knee-braced frames produce moment-resisting joints by the addition of diagonal members between columns and beams. The resulting "semi-rigid" frame resists lateral loads. The moment-resisting capacity of knee-braced frames varies widely. The controlling part of the assembly is usually the connection; however, bending of members can be the controlling feature of some frames. Once the capacity of the connection is determined, members can be checked and the capacity of the frame can be determined by statics. Particular attention should be given to the beam/column connection. Additional tensile forces may be developed in this connection due to knee-brace action under vertical loads.

Similarly to knee-braced frames, the connections of rods to timber framing will usually govern the capacity of the rod-braced frame. Typically, the rods act only in tension. Once the capacity of the connection is determined, the capacity of the frame can be determined by statics.

Braced horizontal diaphragms are described in Section 8.6.2.1.7.

8.8.1.1 Stiffness

The stiffness and deflection of wood elements shall be determined based on a mathematical model of the assembly considering the configuration, stiffness, and interconnection of the individual components.

8.8.1.2 Strength

The capacities of individual components, including connections, shall be determined in accordance with Section 8.4.2.

C8.8.1.2 Strength

The strength of wood elements is dependent on the strength of the individual components that comprise the assembly. In many cases the capacity of the connections between components will be the limiting factor in the strength of the assembly.

8.8.1.3 Acceptance Criteria

Design actions shall be compared with design capacities in accordance with Section 3.4.2.2. Connections shall be considered in accordance with Section 8.4.3. Axial tension, axial tension with bending shall be considered deformation-controlled. Axial compression, and connections between steel rods and wood components shall be considered force-controlled. *m*-factors for deformation-controlled actions shall be taken from Table 8-3. For nonlinear procedures, coordinates of the generalized force-deformation relation described by Figure 8-1, and deformation acceptance criteria shall be taken from Table 8-4.

C8.8.1.3 Acceptance Criteria

Deformation acceptance criteria will largely depend on the allowable deformations for other structural and nonstructural components and elements that are supported by the element. Allowable deformations must also be consistent with the desired performance level. Actions on connection types that do not appear in Table 8-3 (e.g., truss plates) are force-controlled.

8.8.2 Components Supporting Discontinuous Shear Walls

Axial compression on wood posts and flexure and shear on wood beams that support discontinuous shear walls shall be considered force-controlled actions. Lower-bound strengths shall be determined in accordance with Section 8.4.2.3.