8. Wood and Light Metal Framing (Systematic Rehabilitation)

8.1 Scope

This chapter presents the general methods for rehabilitating wood frame buildings and/or wood and light metal framed elements of other types of buildings that are presented in other sections of this document. Refer to Chapter 2 for the general methodology and issues regarding performance goals, and the decisions and steps necessary for the engineer to develop a rehabilitation scheme. The Linear Static Procedure (LSP) presented in Chapter 3 is the method recommended for the systematic analysis of wood frame buildings. However, properties of the idealized elastic and inelastic performance of the 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. A more complete historical perspective is included in the *Commentary*. 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 8.3.2.2B. Properties of shear walls and other lateral-force-resisting systems such as braced frames are described and discussed in Section 8.4, along with various retrofit or strengthening methods. Horizontal floor and roof diaphragms and braced systems are discussed in Section 8.5, which also covers engineering properties and methods of upgrading or strengthening the elements. Wood foundations and pole structures are described in Section 8.6. For additional information regarding foundations, see Chapter 4. Definitions of terms are in Section 8.7; symbols are in Section 8.8. Reference materials for both new and existing materials are provided in Section 8.9.

8.2 Historical Perspective

8.2.1 General

Wood frame construction has evolved over millennia; 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/or floors, in combination with other materials, for other types of buildings.

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. Information regarding the establishment of a building's age and a discussion of the evolution of framing systems can be found in Section C8.2 of the *Commentary*.

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.

8.2.2 Building Age

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 a 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 helpful guide in determining the extent of loss of a structure's capacity to resist loads.

8.2.3 Evolution of Framing Methods

The earliest wood frame buildings built by European immigrants to the United States 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, so that 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 for 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 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 8.4 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

Each structural element in an existing building is composed of a material capable of resisting and transferring applied loads to the foundation systems. One material group historically used in building construction is wood. 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., glued-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 analyze properly the anticipated performance of the building construction.

8.3.2 Properties of In-Place Materials and Components

8.3.2.1 Material Properties

Wood has dramatically different properties in its three orthotropic axes (parallel to grain, transverse to grain, and radial). These properties vary with wood species, grade, and density. The type, grade, and condition of the component(s) must be established in order to compute strength and deformation characteristics. Mechanical properties and configuration of component and connection material dictate the structural behavior of the component under load. The effort required to determine these properties is related to the availability of original and updated construction documents, original quality of construction, accessibility, condition of materials, and the analytical procedure to be used in the rehabilitation (e.g., LSP for wood construction).

The first step in quantifying properties is to establish the species and grade of wood through review of construction documents or direct inspection. If the wood is not easily identified visually or by the presence of a stamped grade mark on the wood surface, then samples can be taken for laboratory testing and identification (see below). The grade of the wood also may be established from grade marks, the size and presence of knots, splits and checks, the slope of the grain, and the spacing of growth rings through the use of appropriate grade rules. The grading shall be performed using a specific grading handbook for the assumed wood species and application (e.g., Department of Commerce American Softwood Lumber Standard PS 20-70 (NIST, 1986), the National Grading Rules for Dimension Lumber of the National Grading Rules Committee), or through the use of the ASTM (1992) D245 grading methodology.

In general, the determination of material species and properties (other than inter-component connection behavior) is best accomplished through removal of samples coupled with laboratory analysis by experts in wood science. Sampling shall take place in regions of reduced stress, such as mid-depth of members. Some local repair may be necessary after sampling.

The properties of adhesives used in fabrication of certain component types (e.g., laminated products) must also be evaluated. Such adhesives may be adversely affected by exposure to moisture and other conditions in-service. Material properties may also be affected by certain chemical treatments (e.g., fire retardant) originally applied to protect the component from environmental conditions.

8.3.2.2 Component Properties

A. 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 following component properties shall be established during a condition assessment in the initial stages of the seismic rehabilitation process, to aid in evaluating component strength and deformation capacities (see Section 8.3.3 for assessment guidelines):

- Original cross sectional shape and physical dimensions (e.g., actual dimensions for 2" x 4" stud) for the primary members of the structure
- Size and thickness of additional connected materials, including plywood, bracing, stiffeners; chord, sills, struts, and hold-down posts
- Modifications to members (e.g., notching, holes, splits, cracks)
- Location and dimension of braced frames and shear walls; type, grade, nail size, and spacing of hold-downs and drag/strut members
- Current physical condition of members, including presence of decay or deformation
- Confirmation of component(s) behavior with overall element behavior

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 perform a thorough inspection of 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.

B. 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.

The following connections shall be established during a condition assessment to aid in the evaluation of the structural behavior:

- Connections between horizontal diaphragms and shear walls and braced frames
- Size and character of all drag ties and struts, including splice connections used to collect loads from the diaphragms to deliver to shear walls or braced frames
- Connections at splices in chord members of horizontal diaphragms
- Connections of horizontal diaphragms to exterior or interior concrete or masonry walls for both in-plane and out-of-plane loads
- Connections of cross-tie members for concrete or masonry buildings
- Connections of shear walls to foundations
- Method of through-floor transfer of wall shears in multistory buildings

8.3.2.3 Test Methods to Quantify 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 glued-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, to establish rapidly the expected strength 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 the examination of wall and diaphragm continuity, and suitability of in-place connectors.

Where it is desired to use an existing assembly and little or no information as to its performance is available, a cyclic load test of a mock-up of the existing structural elements can be utilized to determine the performance of various assemblies, connections, and load transfer conditions. The establishment of the parameters given in Tables 8-1 and 8-2 can be determined from the results of the cyclic load tests. See Section 2.13 for an explanation of the backbone curve and the establishment of parameters.

8.3.2.4 Minimum Number of Tests

In order to accurately quantify expected strength and other in-place properties, it is important that a minimum number of tests 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 utilize the default strength properties, per Section 8.3.2.5 provisions, as opposed to conducting the specified testing. However, these default values should only be used for the LSP. It is strongly encouraged that the expected strengths be derived through testing of assemblies to model behavior more accurately.

In terms of defining expected strength properties, the following guidelines should be followed. Removal of coverings, including stucco, fireproofing and partition materials, is generally required to facilitate the sampling and observations.

- If original construction documents exist that define the grade of wood and mechanical properties, at least one location for each story shall be randomly observed from each component type (e.g., solid sawn lumber, glulam beam, plywood diaphragm) identified as having a different material grade. These shall be verified by sampling and testing or by observing grade stamps and conditions.
- If original construction documents defining properties are limited or do not exist, but the date of construction is known and single material use is confirmed (e.g., all components are Douglas fir solid sawn lumber), at least three observations or samples should be randomly made for each component type (e.g., beam, column, shear wall) for every two floors in the building.
- If no knowledge of the structural system and materials used exists, at least six samples shall be removed or observed from each element (e.g., primary gravity- and lateral-load-resisting components) and component type (e.g., solid sawn lumber, diaphragm) for every two floors of construction. If it is determined from testing and/or observation that more than one material grade exists, additional observations should be made until the extent of use for each grade in component fabrication has been established.
- In the absence of construction records defining connector features present, at least three connectors shall be observed for every floor of the building. 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.
- For an archaic systems test or other full-scale mock-up test of an assembly, 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%.

8.3.2.5 Default Properties

Mechanical properties for wood materials and components are based on available historical data and tests on samples of components, or mock-up tests of typical systems. In the absence of these data, or for comparative purposes, default material strength properties are needed. Unlike other structural materials, default properties for wood are highly variable and dependent on factors including the species, grade, usage, age, and exposure conditions. As a minimum, it is recommended that the type and grade of wood be established. Historically, codes and standards including the National Design Specification (AF&PA, 1991a) have published allowable stresses as opposed to strengths. These values are conservative, representing mean values from previous research. Default strength values, consistent with *NEHRP Recommended Provisions* (BSSC, 1995), may be calculated as the allowable stress multiplied by a 2.16 conversion factor, a capacity reduction factor of 0.8, and time effect factor of 1.6 for seismic loading. This results in an approximate 2.8 factor to translate allowable stress values to yield or limit state values. The expected strength, Q_{CE}, is determined based on these yield or limit state strengths. If significant inherent damage or deterioration is found to be present, default values may not be used. Structural elements with significant damage need to be replaced with new materials, or else a significant reduction in the capacity and stiffness must be incorporated into the analysis.

It is recommended that the results of any material testing performed be compared to the default values for the particular era of building construction; should significantly reduced properties from testing be discovered in this testing, further evaluation as to the cause shall be undertaken. Default values may not be used if they are greater than those obtained from testing.

Default material strength properties may only be used in conjunction with the LSP. For all other analysis procedures, expected strengths from specified testing and/or mock-up testing shall be used to determine anticipated performance.

Default values for connectors shall be established in a manner similar to that for the members. The published values in the National Design Specification (AF&PA,

1994) shall be increased by a factor of 2.8 to convert from allowable stress levels to yield or limit state values, Q_{CE}, for seismic loading.

Deformation at yield of nailed connectors may be assumed to be 0.02 inches for wood to wood and wood to metal connections. For wood screws the deformation may be assumed to be 0.05 inches; for lag bolts the deformation may be assumed at 0.10 inches. For bolts, the deformation for wood to wood connections can be assumed to be 0.2 inches; for wood to steel connections, 0.15 inches. In addition the estimated deformation of any hardware or allowance, e.g., for poor fit or oversized holes, should be summed to obtain the total connection deformation.

8.3.3 Condition Assessment

8.3.3.1 General

A condition assessment of the existing building and site conditions shall be performed as part of the seismic rehabilitation process. The goal of this assessment is fourfold:

1. To examine the physical condition of primary and secondary components and the presence of any degradation
2. To verify the presence and configuration of components and their connections, and continuity of load paths between components, elements, and systems
3. To review other conditions—such as neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for rehabilitation—that may influence building performance
4. To formulate a basis for selecting a knowledge factor (see Section 8.3.4)

The physical condition of existing components and elements, and their connections, must be examined for presence of 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 accounted for. The condition assessment shall also examine for 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. Often, unfinished areas such as attic spaces, basements, and crawl spaces provide suitable access to wood components used 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.

Connections in wood components, elements, and systems 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 for concrete and masonry buildings, for which wood diaphragms are used for out-of-plane loading, requires detailed inspection. Bolt holes in relatively narrow straps sometime 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 across the building, which are part of the wall anchorage system, need to be inspected to confirm their presence and 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

The scope of a condition assessment shall include all primary structural elements and components involved in gravity- and lateral-load resistance. 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 the understanding of load paths and the ability of components to resist and transfer these loads. The degree of assessment performed also affects the knowledge factor, κ, discussed in Section 8.3.4. General guidelines and procedures are also contained in that section; for additional guidance and references, see the *Commentary.*

Direct visual inspection provides the most valuable information, as it can be used to quickly identify any configuration issues, and allows both measurement of component dimensions, and determination whether degradation is present. The continuity of load paths may be established through viewing of 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 dimensions and features of all accessible components shall be measured and compared to available design information. Similarly, the configuration and condition of all connections shall be verified, with any deformations or other anomalies noted. If design documents for the structure do not exist, this technique shall be followed to develop an as-built drawing set.

If coverings or other obstructions exist, indirect visual inspection through use of drilled holes and a fiberscope shall be utilized (as allowed by access). If this method is not appropriate, then local removal of covering materials will be necessary. The following guidelines shall be used.

- If detailed design drawings exist, exposure of at least three different primary connections shall occur for each connection type (e.g., beam-column, shear wall-diaphragm, shear wall-foundation). If no significant capacity-reducing deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of all coverings from primary connections of that type may be necessary, if reliance is to be placed on the connection.

- In the absence of accurate drawings, either invasive fiberscopic inspections or exposure of at least 50% of all primary connection types for inspection shall occur. If common detailing is observed, this sample may be considered representative. If a multitude of details or conditions are observed, full exposure is required.

The scope of this 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 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 utilized for the detection of reinforcing steel in concrete or masonry can be utilized to verify the extent of metal straps and hardware located beneath the finish surfaces. Further guidelines and procedures for destructive and nondestructive tests that may be used in the condition assessment are contained in the *Commentary.*

8.3.3.3 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models for evaluation of seismic performance. To aid in this effort, the results shall be quantified and reduced, with the following specific topics addressed:

- Component section properties and dimensions

- Component configuration and presence of any eccentricities

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

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

8.3.4 Knowledge (κ) Factor

As described in Section 2.7, computation of component capacities and allowable deformations shall involve the use of a knowledge (κ) factor. For cases where an LSP will be used in the analysis, two categories of κ exist. This section further describes those requirements specific to wood structural elements that must be accomplished in the selection of a κ factor.

If the wood structural system is exposed, significant knowledge regarding configuration and behavior may be gained through condition assessment. In general, a κ factor of 1.0 can be utilized when a thorough assessment is performed on the primary and secondary components and load path, and the requirements of Section 2.7 are met. Similarly, if the wood system is encased, a κ factor of 1.0 can be utilized when three samples of each primary component connection type are exposed and verified as being compliant with construction records, and fiberscopic examinations are performed to confirm the condition and configuration of primary load-resisting components.

If knowledge of as-built component or connection configuration/condition is incomplete or nonexistent, the κ factor used in the final component evaluation shall be reduced to 0.75. Examples of where this value shall be applied are contained in Section 2.7 and Equation 3-18. For encased components where construction documents are limited and knowledge of configuration and condition is incomplete, a factor of 0.75 shall be used.

8.3.5 Rehabilitation Issues

Upon determining that portions of a wood building structure are deficient or inadequate for the Rehabilitation Objective, the next step is to define reinforcement or replacement alternatives. If a reinforcement program is to be followed and attachment to the existing framing system is proposed, it is necessary to closely examine material factors that may influence reinforcement/attachment design, including:

- Degree of any degradation in the component from such mechanisms as biological attack, creep, high static or dynamic loading, moisture, or other effects
- Level of steady state stress in the components to be reinforced (and potential to temporarily remove this stress if appropriate)
- Elastic and plastic properties of existing components, to preserve strain compatibility with any new reinforcement materials
- Ductility, durability, and suitability of existing connectors between components, and access for reinforcement or modification
- Prerequisite efforts necessary to achieve appropriate fit-up for reinforcing components and connections
- Load flow and deformation of the components at end connections (especially at foundation attachments and connections where mixed connectors such as bolts and nails exist)
- 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

8.4 Wood and Light Frame Shear Walls

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, and 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 *Uniform Building Code (UBC)* (ICBO, 1994a) limits the aspect ratio (height-to-width) to 3.5:1. After the 1994 Northridge earthquake, the city of Los Angeles reduced the allowable aspect ratio to 2:1, pending additional tests. 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.

Dissimilar wall sheathing materials on opposite sides of a wall should not be combined when calculating the capacity of the wall. Similarly, different walls sheathed with dissimilar materials along the same line of lateral force resistance should be analyzed based on only one type of sheathing. The wall sheathing with the greatest capacity should be used for determining capacity. The walls should also be analyzed based on the relative rigidity and capacity of the materials to determine if performance of the “nonparticipating” material will be acceptable.

For uplift calculations on shear-wall elements, the overturning moment on the wall should be based on the calculated load on the wall from the base shear and the use of an appropriate *m* factor for the uplift connector. As an alternative, the uplift can be based on a lateral load equal to 1.2 times the yield capacity of the wall. However, no *m* factor is involved in the demand versus capacity equation, and the uplift connector yield capacity should not be exceeded.

Connections between elements, drag ties, struts, and other structured members should be based on the calculated load to the connection under study, and analyzed. As an alternative, the connection can be analyzed for a maximum load, at the connection, of 1.2 times the yield capacity of the weaker element. The yield capacity of the connection should not be exceeded and no *m* factor is used in the analysis.

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 the assembly can resist, regardless of the deflection. See Section 8.5.11 for the effect of openings in diaphragms. The expected capacity, Q_{CE}, is equal to the yield capacity of the shear wall, V_y.

8.4.1 Types of Light Frame Shear Walls

8.4.1.1 Existing Shear Walls

A. 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. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

B. 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) 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.

C. 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 to 10d galvanized nails. The lateral forces are resisted by nail couples, similarly to horizontal siding. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

D. Wood Siding over Horizontal Sheathing

Typically, siding is nailed with 8d to 10d galvanized nails through the sheathing to the studs. Lateral forces are resisted by nail couples for both layers. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

E. 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 truss-shear action.

F. Structural Panel or Plywood 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.

G. Stucco on Studs (over sheathing or wire backed building paper)

Typically, 7/8-inch portland cement plaster is applied on 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. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

H. 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. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

I. Gypsum Plaster on Gypsum Lath

Typically, 1/2-inch plaster is glued or keyed to 16-inch x 48-inch gypsum lath, which is nailed to studs with 13 gage nails. Gypsum plaster on gypsum lath resists lateral loads by panel diaphragm action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

J. 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 utilized in some situations. The assembly resists lateral forces by panel diaphragm-shear action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

K. 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. The strength and stiffness degrades with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

L. 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. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

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

This is installed in the same manner as horizontal sheathing, except the wall is braced at the corners 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. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

N. Fiberboard or Particleboard Sheathing

Typically, 4' x 8' panels are applied directly to the studs with nails. The fiberboard requires nails (typically 8d) with large heads such as roofing nails. Lateral loads are resisted by panel diaphragm action. The strength and stiffness degrade with cyclic loading. (See Section 3.3.1.3 and Table 3-1 for the appropriate C_2 value.)

8.4.1.2 Shear Wall Enhancements for Rehabilitation

A. Structural Panel Sheathing Added to Unfinished Stud Walls

Wall shear capacity and stiffness can be increased by adding structural panel sheathing to one side of unfinished stud walls, such as cripple walls or attic end walls.

B. Structural Panel Sheathing Overlay of Existing Shear Walls

For a moderate increase in shear capacity and stiffness that can be applied in most places in most structures, the existing wall covering can be overlaid with structural panel sheathing; for example, plywood sheathing can be applied over an interior wall finish. For exterior applications, the structural panel can be placed over the exterior finish and nailed directly through it to the studs. This rehabilitation procedure typically can be used for the following shear walls, which are described in Section 8.4.1.1:

- Single layer horizontal lumber sheathing or siding
- Single layer diagonal lumber sheathing
- Vertical wood siding only
- Gypsum plaster or wallboard on studs (also on gypsum lath and gypsum wallboard)
- Gypsum sheathing
- Horizontal lumber sheathing with cut-in braces or diagonal blocking
- Fiberboard or particleboard sheathing

The enhanced shear wall is evaluated in accordance with Section 8.4.9, discounting the original sheathing and reducing the yield capacity of the overlay material by 20%.

C. Structural Panel Sheathing Added Under Existing Wall Covering

To obtain a significant increase in shear capacity, the existing wall covering can be removed; structural panel sheathing, connections, and tie-downs added; and the wall covering replaced. 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 structural panels can be cut to fit existing stud spacings.

D. Increased Attachment

For existing structural panel sheathed walls, additional nailing will result in higher capacity and increased stiffness. Other connectors—such as 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.

E. Rehabilitation of Connections

Most shear wall rehabilitation procedures require a check of all existing connections, especially to diaphragms and foundations. Additional blocking between floor or roof joists at shear walls is often needed on existing structures. The blocking must be connected to the shear wall and the diaphragm to provide a load path for lateral loads. 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.

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.

When existing shear walls are overlaid with structural panels, the connections of the structural panels 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 structural panel joints may also be needed.

When framing members or blocking are added to a structure, the wood should be kiln-dried or well-seasoned to prevent it from shrinking away from the existing framing or splitting.

8.4.1.3 New Shear Walls Sheathed with Structural Panels or Plywood Panel Sheathing or Siding

New shear walls using the existing framing or new framing are sheathed with structural panels (i.e., plywood or oriented strand board). 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 can be found in documents from the American Plywood Association (APA), such as APA (1983).

8.4.2 Light Gage Metal Frame Shear Walls

8.4.2.1 Existing Light Gage Metal Frame Shear Walls

A. 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.

B. 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.

C. Plywood or Structural Panels

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

8.4.2.2 Light Gage Metal Frame Enhancements for Rehabilitation

A. Addition of Plywood Structural Panels to Existing Metal Stud Walls

Any existing covering other than plywood is removed and replaced with structural panels. Connections to the diaphragm(s) and the foundation must be checked and may need to be strengthened.

B. Existing Plywood or Structural Panels on Metal Studs

Added screws and possibly additional connections to diaphragms and foundation may be required.

8.4.2.3 New Light Gage Metal Frame Shear Walls

A. Plywood or Structural Panels

Refer to Section 8.4.1.3.

8.4.3 Knee-Braced and Miscellaneous Timber Frames

8.4.3.1 Knee-Braced Frames

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. For a detailed discussion on connections, see Section C8.3.2.2B in the *Commentary*.

8.4.3.2 Rod-Braced Frames

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. See Section 8.3.2.2B.

8.4.4 Single Layer Horizontal Lumber Sheathing or Siding Shear Walls

8.4.4.1 Stiffness for Analysis

Horizontal lumber sheathed shear walls are weak and very flexible and have long periods of vibration. These shear walls are suitable only where earthquake shear loads are low and deflection control is not required. The deflection of these shear walls can be approximated by Equation 8-1:

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

where:

b = Shear wall length, ft
h = Shear wall height, ft
v_y = Shear at yield, lb/ft
G_d = Shear stiffness in lb/in.
Δ_y = Calculated shear wall deflection at yield, in.
d_a = Elongation of anchorage at end of wall determined by anchorage details and load magnitude, in.

For horizontal lumber sheathed shear walls,
G_d = 2,000 lb/in.

8.4.4.2 Strength Acceptance Criteria

Horizontal sheathing or siding has an estimated yield capacity of 80 pounds per linear foot. 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 *Western Woods Use Book* (WWPA, 1983). See also ATC (1981) for a discussion of the nail couple.

8.4.4.3 Deformation Acceptance Criteria

The 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. See Table 8-1 for *m* factors for use in the LSP in performing design analyses.

The coordinates for the normalized force-deflection curve used for modeling in connection with the nonlinear procedures (Figure 8-1) are shown in Table 8-2. The values in this table refer to Figure 8-1 in the following way. 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*. Figure 8-1 also shows the deformation ratios for IO, LS, and CP Performance Levels for primary components. (See Chapter 3 for the use of the force-deflection curve in the NSP.)

Deformation acceptance criteria for use in connection with nonlinear procedures are given in footnotes of Table 8-2 for primary and secondary components, respectively.

Table 8-1 Numerical Acceptance Factors for Linear Procedures—Wood Components

		m Factors for Linear Procedures[2]				
		Primary			Secondary	
		IO	LS	CP	LS	CP
Shear Walls	**Height/Length Ratio (h/L)[1]**					
Horizontal 1" x 6" Sheathing	$h/L < 1.0$	1.8	4.2	5.0	5.0	5.5
Horizontal 1" x 10" Sheathing	$h/L < 1.0$	1.6	3.4	4.0	4.0	5.0
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	$h/L < 1.5$	1.4	2.6	3.0	3.1	4.0
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	$h/L < 1.5$	1.3	2.3	2.6	2.8	3.0
Diagonal 1" x 6" Sheathing	$h/L < 1.5$	1.5	2.9	3.3	3.4	3.8
Diagonal 1" x 8" Sheathing	$h/L < 1.5$	1.4	2.7	3.1	3.1	3.6
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	$h/L < 2.0$	1.3	2.2	2.5	2.5	3.0
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	$h/L < 2.0$	1.3	2.0	2.3	2.5	2.8
Double Diagonal 1" x 6" Sheathing	$h/L < 2.0$	1.2	1.8	2.0	2.3	2.5
Double Diagonal 1" x 8" Sheathing	$h/L < 2.0$	1.2	1.7	1.9	2.0	2.5
Vertical 1" x 10" Sheathing	$h/L < 1.0$	1.5	3.1	3.6	3.6	4.1

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

Table 8-1 Numerical Acceptance Factors for Linear Procedures—Wood Components (continued)

		m Factors for Linear Procedures[2]				
		Primary			Secondary	
		IO	LS	CP	LS	CP
Structural Panel or Plywood Panel Sheathing or Siding	*h*/*L* < 1.0*	1.7	3.8	4.5	4.5	5.5
	h/*L* > 2.0* *h*/*L* < 3.5	1.4	2.6	3.0	3.0	4.0
Stucco on Studs	*h*/*L*<1.0 *	1.5	3.1	3.6	3.6	4.0
	h/*L* = 2.0 *	1.3	2.2	2.5	2.5	3.0
Stucco over 1" x Horizontal Sheathing	*h*/*L* < 2.0	1.5	3.0	3.5	3.5	4.0
Gypsum Plaster on Wood Lath	*h*/*L* < 2.0	1.7	3.9	4.6	4.6	5.1
Gypsum Plaster on Gypsum Lath	*h*/*L* < 2.0	1.8	4.2	5.0	4.2	5.5
Gypsum Plaster on Metal Lath	*h*/*L* < 2.0	1.7	3.7	4.4	3.7	5.0
Gypsum Sheathing	*h*/*L* < 2.0	1.9	4.7	5.7	4.7	6.0
Gypsum Wallboard	*h*/*L* < 1.0 *	1.9	4.7	5.7	4.7	6.0
	h/*L* = 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*/*L* < 1.0	1.7	3.7	4.4	4.2	4.8
Fiberboard or Particleboard Sheathing	*h*/*L* < 1.5	1.6	3.2	3.8	3.8	5.0
Diaphragms	**Length/Width Ratio (*L*/*b*)[1]**					
Single Straight Sheathing, Chorded	*L*/*b* < 2.0	1	2.0	2.5	2.4	3.1
Single Straight Sheathing, Unchorded	*L*/*b* < 2.0	1	1.5	2.0	1.8	2.5
Double Straight Sheathing, Chorded	*L*/*b* < 2.5	1.25	2.0	2.5	2.3	2.8
Double Straight Sheathing, Unchorded	*L*/*b* < 2.5	1	1.5	2.0	1.8	2.3
Single Diagonal Sheathing, Chorded	*L*/*b* < 2.5	1.25	2.0	2.5	2.3	2.9
Single Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	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* < 2.5	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	*L*/*b* < 3.0 * *L*/*b* = 4*	1.5 1.5	3.0 2.5	4.0 3.0	3.5 2.8	4.5 3.5
Wood Structural Panel, Unblocked, Chorded	*L*/*b* < 3* *L*/*b* = 4*	1.5 1.5	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel, Blocked, Unchorded	*L*/*b* < 2.5 *L*/*b* = 3.5	1.25 1.25	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel, Unblocked, Unchorded	*L*/*b* < 2.5 *L*/*b* = 3.5	1.25 1.0	2.0 1.5	2.5 2.0	2.4 2.0	3.0 2.6
Wood Structural Panel Overlay on Sheathing, Chorded	*L*/*b* < 3* *L*/*b* = 4*	1.5 1.5	2.5 2.0	3.0 2.5	2.9 2.6	4.0 3.2
Wood Structural Panel Overlay on Sheathing, Unchorded	*L*/*b* < 2.5 *L*/*b* = 3.5	1.25 1.0	2.0 1.5	2.5 2.0	2.4 1.9	3.0 2.6

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.
2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

Table 8-1 Numerical Acceptance Factors for Linear Procedures—Wood Components (continued)

	m Factors for Linear Procedures[2]				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Component/Element					
Frame elements subject to axial and bending stresses	1.0	2.5	3.0	2.5	4.0
Connections					
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. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

2 Linear interpolation is permitted for intermediate value if h/L has asterisks.

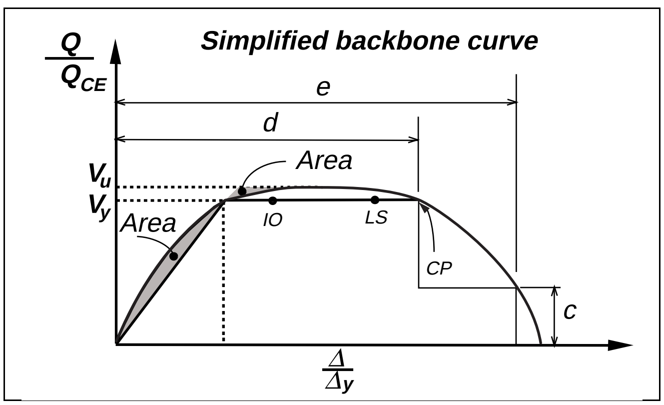

Figure 8-1 Normalized Force versus Deformation Ratio for Wood Elements

8.4.4.4 Connections

The connections between parts of the shear wall assembly and other elements of the lateral-force-resisting system must be investigated and analyzed. 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 and expected performance (see Section 8.3.2.2B).

8.4.5 Diagonal Lumber Sheathing Shear Walls

8.4.5.1 Stiffness for Analysis

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. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in. for single layer diagonal siding and G_d = 18,000 lb/in. for double diagonal siding.

Table 8-2 Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components

		d	e	c
Shear Wall Type - Types of Existing Wood and Light Frame Shear Walls	**Height/Length Ratio *h/L*[1]**			
Horizontal 1" x 6" Sheathing	h/L < 1.0	5.0	6.0	0.3
Horizontal 1" x 10" Sheathing	h/L < 1.0	4.0	5.0	0.3
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	h/L < 1.5	3.0	4.0	0.2
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	h/L < 1.5	2.6	3.6	0.2
Diagonal 1" x 6" Sheathing	h/L < 1.5	3.3	4.0	0.2
Diagonal 1" x 8" Sheathing	h/L < 1.5	3.1	4.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	h/L < 2.0	2.5	3.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	h/L < 2.0	2.3	3.0	0.2
Double Diagonal 1" x 6" Sheathing	h/L < 2.0	2.0	2.5	0.2
Double Diagonal 1" x 8" Sheathing	h/L < 2.0	2.0	2.5	0.2
Vertical 1" x 10" Sheathing	h/L < 1.0	3.6	4.0	0.3
Structural Panel or Plywood Panel Sheathing or Siding	h/L < 1.0*	4.5	5.5	0.3
	h/L > 2.0* h/L < 3.5	3.0	4.0	0.2
Stucco on Studs	h/L < 1.0*	3.6	4.0	0.2
	h/L = 2.0*	2.5	3.0	0.2
Stucco over 1" x Horizontal Sheathing	h/L < 2.0	3.5	4.0	0.2
Gypsum Plaster on Wood Lath	h/L < 2.0	4.6	5.0	0.2
Gypsum Plaster on Gypsum Lath	h/L < 2.0	5.0	6.0	0.2
Gypsum Plaster on Metal Lath	h/L < 2.0	4.4	5.0	0.2
Gypsum Sheathing	h/L < 2.0	5.7	6.3	0.2
Gypsum Wallboard	h/L < 1.0*	5.7	6.3	0.2
	h/L = 2.0*	4.0	5.0	0.2
Horizontal 1" x 6" Sheathing With Cut-In Braces or Diagonal Blocking	h/L < 1.0	4.4	5.0	0.2
Fiberboard or Particleboard Sheathing	h/L < 1.5	3.8	4.0	0.2
Diaphragm Type - Horizontal Wood Diaphragms	**Length/Width Ratio (*L/b*)[1]**			
Single Straight Sheathing, Chorded	L/b < 2.0	2.5	3.5	0.2
Single Straight Sheathing, Unchorded	L/b < 2.0	2.0	3.0	0.3
Double Straight Sheathing, Chorded	L/b < 2.0	2.5	3.5	0.2

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

Notes: (a) Acceptance criteria for primary components

(Δ/Δ_y) IO = 1.0 + 0.2 (d - 1.0)
(Δ/Δ_y) LS = 1.0 + 0.8 (d - 1.0)
(Δ/Δ_y) CP = d

(b) Acceptance criteria for secondary components

(Δ/Δ_y) LS = d
(Δ/Δ_y) CP = e

(c) Linear interpolation is permitted for intermediate values if h/L or L/b has asterisks.

Table 8-2 Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components (continued)

		d	e	c
Double Straight Sheathing, Unchorded	*L*/*b* < 2.0	2.0	3.0	0.3
Single Diagonal Sheathing, Chorded	*L*/*b* < 2.0	2.5	3.5	0.2
Single Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	2.0	3.0	0.3
Straight Sheathing Over Diagonal Sheathing, Chorded	*L*/*b* < 2.0	3.0	4.0	0.2
Straight Sheathing Over Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	2.5	3.5	0.3
Double Diagonal Sheathing, Chorded	*L*/*b* < 2.0	3.0	4.0	0.2
Double Diagonal Sheathing, Unchorded	*L*/*b* < 2.0	2.5	3.5	0.2
Wood Structural Panel, Blocked, Chorded	*L*/*b* < 3* *L*/*b* = 4*	4.0 3.0	5.0 4.0	0.3
Wood Structural Panel, Unblocked, Chorded	*L*/*b* < 3* *L*/*b* = 4*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel, Blocked, Unchorded	*L*/*b* < 2.5* *L*/*b* = 3.5*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel, Unblocked, Unchorded	*L*/*b* < 2.5* *L*/*b* = 3.5*	2.5 2.0	3.5 3.0	0.4
Wood Structural Panel Overlay On Sheathing, Chorded	*L*/*b* < 3* *L*/*b* = 4*	3.0 2.5	4.0 3.5	0.3
Wood Structural Panel Overlay On Sheathing, Unchorded	*L*/*b* < 2.5* *L*/*b* = 3.5*	2.5 2.0	3.5 3.0	0.4
Connection Type				
Nails - Wood to Wood		7.0	8.0	0.2
Nails - Metal to Wood		5.5	7.0	0.2
Screws - Wood to Wood		2.5	3.0	0.2
Screws - Wood to Metal		2.3	2.8	0.2
Lag Bolts - Wood to Wood		2.8	3.2	0.2
Lag Bolts - Metal to Wood		2.5	3.0	0.2
Bolts - Wood to Wood		3.0	3.5	0.2
Bolts - Metal to Wood		2.8	3.3	0.2

1. For ratios greater than the maximum listed values, the component is considered not effective in resisting lateral loads.

Notes: (a) Acceptance criteria for primary components

(Δ/Δ_y) IO = 1.0 + 0.2 (d - 1.0)
(Δ/Δ_y) LS = 1.0 + 0.8 (d - 1.0)
(Δ/Δ_y) CP = d

(b) Acceptance criteria for secondary components

(Δ/Δ_y) LS = d
(Δ/Δ_y) CP = e

(c) Linear interpolation is permitted for intermediate values if h/L or L/b has asterisks.

8.4.5.2 Strength Acceptance Criteria

Diagonal sheathing has an estimated yield capacity of approximately 700 pounds per linear foot for single layer and 1300 pounds per linear foot for double diagonal sheathing. This capacity 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 (1983).

8.4.5.3 Deformation Acceptance Criteria

The deformation acceptance criteria will be determined by the capacity of lateral- and gravity-load-resisting elements to deform without failure. See Table 8-1 for *m* factors for use in the LSP.

The coordinates for the normalized force-deflection curve used in the nonlinear procedures are shown in Table 8-2. The values in this table refer to Figure 8-1 in the following way. Distance *d* is considered the maximum deflection at the point of loss of strength. Distance *e* is the maximum deflection at a strength or capacity equal to value *c*. (See Chapter 3 for the use of the force deflection curve in the NSP.)

8.4.5.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.6 Vertical Wood Siding Shear Walls

8.4.6.1 Stiffness for Analysis

Vertical wood siding has a very low lateral-force-resistance capacity and is very flexible. These shear walls are suitable only where earthquake shear loads are very low and deflection control is not needed. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 1,000 lb/in.

8.4.6.2 Strength Acceptance Criteria

Vertical siding has a yield capacity of approximately 70 pounds per linear foot. This capacity 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 can be used to calculate the capacity of vertical wood siding, in a manner similar to the method used for horizontal siding.

8.4.6.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.6.4 Connections

The load capacity of the vertical siding is low; this makes the capacity of connections between the shear wall and the other elements of secondary concern (see Section 8.3.2.2B).

8.4.7 Wood Siding over Horizontal Sheathing Shear Walls

8.4.7.1 Stiffness for Analysis

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. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 4,000 lb/in.

8.4.7.2 Strength Acceptance Criteria

Wood siding over horizontal sheathing has a yield capacity of approximately 500 pounds per linear foot. 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.

8.4.7.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.7.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.8 Wood Siding over Diagonal Sheathing Shear Walls

8.4.8.1 Stiffness for Analysis

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. The deflection of these shear walls can be approximated by using Equation 8-1, with G_d = 11,000 lb/in.

8.4.8.2 Strength Acceptance Criteria

Wood siding over diagonal sheathing has an estimated yield capacity of approximately 1,100 pounds per linear foot. This capacity 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.

8.4.8.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.8.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.9 Structural Panel or Plywood Panel Sheathing Shear Walls

8.4.9.1 Stiffness for Analysis

The response of wood structural shear walls is dependent on the thickness of the wood structural panels, the height-to-length (h/L) ratio, the nailing pattern, and other factors. The approximate deflection of wood structural shear walls at yield can be determined using Equation 8-2:

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

where:

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

h = Wall height, ft

E = Modulus of wood end boundary member, psi

A = Area of boundary member cross section, in.^{2}

b = Wall width, ft

G = Modulus of rigidity of plywood, psi

t = Effective thickness of structural panel, in.

d_a = Deflection at yield of tie-down anchorage or deflection at load level to anchorage at end of wall, anchorage details, and dead load, in.

e_n = Nail deformation, in.

For 6d nails at yield: $e_n = .10$

For 8d nails at yield: $e_n = .06$

For 10d nails at yield: $e_n = .04$

8.4.9.2 Strength Acceptance Criteria

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 yield shear capacity, V_y, of wood structural shear walls can be calculated as follows:

$$V_y = .8V_u \qquad (8\text{-}3)$$

Values of ultimate capacity , V_u, of structural panel shear walls are provided in Table 8-3.

If there is no ultimate load for the assembly, use:

$$Q_{CE} = V_u = 6.3Zs/a \qquad (8\text{-}4)$$

where:

Z = Nail value from NDS (1991)

s = Minimum $[m\text{-}1 \text{ or } (n\text{-}1)(a/h)]$

m = Number of nails along the bottom of one panel

n = Number of nails along one side of one panel

a = Length of one panel

h = Height of one panel

8.4.9.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.9.4 Connections

See Sections 8.3.2.2B and 8.4.4.4.

8.4.10 Stucco on Studs, Sheathing, or Fiberboard Shear Walls

8.4.10.1 Stiffness for Analysis

Stucco is brittle and the lateral-force-resistance capacity of stucco shear walls is low. However, the walls are stiff until cracking occurs. These shear walls are suitable only where earthquake shear loads are low. The deflection of these shear walls can be determined using Equation 8-1 with G_d = 14,000 lb/in.

8.4.10.2 Strength Acceptance Criteria

Stucco has a yield capacity of approximately 350 pounds per linear foot. This capacity is dependent on the attachment of the stucco netting to the studs and the embedment of the netting in the stucco.

8.4.10.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.10.4 Connections

The connection between the stucco netting and the framing is of primary concern. Of secondary concern is the connection of the stucco to the netting. Unlike plywood, the tensile capacity of the stucco material

Table 8-3 Ultimate Capacities of Structural Panel Shear Walls[2, 3, 5, 6]

Panel Grade	Minimum Nominal Panel Thickness (inches)	Minimum Nail Penetration in Framing[4] (inches)	Nail Size[4] (Common or Galvanized Box)	Nail Spacing at Panel Edges (in.) Ultimate Capacities (lb/ft) 6"	4"	3"[1]	2"[1]
Structural 1	5/16	1 1/4	6d	700	1010	1130	1200
	3/8	1 1/2	8d	750	1080	1220	1540
	7/16			815	1220	1340	1590
	15/32			880	1380	1550	1620
	15/32	1 5/8	10d[1]	1130	1500	1700	2000
C-D, C-C	5/16	1 1/4	6d	650	700	900	1200
Sheathing, plywood panel siding (and other grades covered in UBC Standard 23-2 or 23-3), structural particleboard	3/8			680	800	1000	1350
	3/8	1 1/2	8d	700	880	1200	1500
	7/16			720	900	1300	1560
	15/32			820	1040	1420	1600
	15/32	1 5/8	10d[1]	900	1400	1500	1900
	19/32			1000	1500	1620	1950

1. 3x or greater framing at plywood joints.
2. Panels applied directly to framing, blocked at all edges.
3. Value extrapolated from cyclic testing.
4. For other nail sizes or nail penetration less than indicated, adjust values based on calculated nail strength (see AF&PA, 1991).
5. Values are for panels on one side. Values may be doubled for panels on both sides.
6. Use 80% of values listed for yield capacity.

(portland cement) rather than the connections, will often govern failure. The connections between the shear wall and foundation and between the shear wall and diaphragm must be investigated. See Section 8.3.2.2B.

8.4.11 Gypsum Plaster on Wood Lath Shear Walls

8.4.11.1 Stiffness for Analysis

Gypsum plaster shear walls are similar to stucco, except their strength is lower. Again, the walls are stiff until failure. These shear walls are suitable only where earthquake shear loads are very low. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in.

8.4.11.2 Strength Acceptance Criteria

Gypsum plaster has a yield capacity of approximately 400 pounds per linear foot.

8.4.11.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.11.4 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 secondary concern.

8.4.12 Gypsum Plaster on Gypsum Lath Shear Walls

8.4.12.1 Stiffness for Analysis

Gypsum plaster on gypsum lath is similar to gypsum wallboard (see Section 8.4.13 for a discussion of gypsum wallboard). The deflection of these shear walls can be determined using Equation 8-1, with G_d = 10,000 lb/in.

8.4.12.2 Strength Acceptance Criteria

These are similar to those for gypsum wallboard, with an approximate yield capacity of 80 pounds per linear foot. See Section 8.4.13.

8.4.12.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.12.4 Connections

See Section 8.4.11.4.

8.4.13 Gypsum Wallboard Shear Walls

8.4.13.1 Stiffness for Analysis

Gypsum wallboard has a very low lateral-force-resistance capacity, but is relatively stiff until cracking occurs. These shear walls are suitable only where earthquake shear loads are very low. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in.

8.4.13.2 Strength Acceptance Criteria

Gypsum wallboard has a yield capacity of approximately 100 pounds per linear foot. This capacity 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.

8.4.13.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.13.4 Connections

See Section 8.4.11.4.

8.4.14 Gypsum Sheathing Shear Walls

8.4.14.1 Stiffness for Analysis

Gypsum sheathing is similar to gypsum wallboard (see Section 8.4.13 for a detailed discussion). The deflection of these shear walls can be determined using Equation 8-1, with G_d = 8,000 lb/in.

8.4.14.2 Strength Acceptance Criteria

These are similar to those for gypsum wallboard (see Section 8.4.13).

8.4.14.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.14.4 Connections

See Section 8.4.11.4.

8.4.15 Plaster on Metal Lath Shear Walls

8.4.15.1 Stiffness for Analysis

Plaster on metal lath is similar to stucco but with less strength. Metal lath and plaster walls are stiff until cracking occurs. These shear walls are suitable only where earthquake shear loads are low. The deflection of these shear walls can be determined using Equation 8-1, with G_d = 12,000 lb/in.

8.4.15.2 Strength Acceptance Criteria

Plaster on metal lath has a yield capacity of approximately 150 pounds per linear foot.

8.4.15.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.15.4 Connections

See Section 8.3.2.2B.

8.4.16 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls

8.4.16.1 Stiffness for Analysis

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. See Section 8.4.4 for more information about horizontal sheathing.

8.4.16.2 Strength Acceptance Criteria

See Section 8.4.4.

8.4.16.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.16.4 Connections

See Section 8.3.2.2B.

8.4.17 Fiberboard or Particleboard Sheathing Shear Walls

8.4.17.1 Stiffness for Analysis

Fiberboard sheathing is very weak, lacks stiffness, and is not able 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. Fiberboard sheathing is not suitable for resisting lateral loads, and nonstructural particleboard should only be used to resist very low earthquake loads. For structural particleboard sheathing, see Section 8.4.9. The deflection of shear walls sheathed in nonstructural particleboard can be determined using Equation 8-1, with G_d = 6,000 lb/in.

8.4.17.2 Strength Acceptance Criteria

Fiberboard has very low strength. For structural particleboard, see the structural panel section (Section 8.4.9). Nonstructural particleboard has a yield capacity of approximately 100 pounds per linear foot.

8.4.17.3 Deformation Acceptance Criteria

See Section 8.4.5.3.

8.4.17.4 Connections

See Section 8.4.11.4.

8.4.18 Light Gage Metal Frame Shear Walls

8.4.18.1 Plaster on Metal Lath

See Section 8.4.15.

8.4.18.2 Gypsum Wallboard

See Section 8.4.13.

8.4.18.3 Plywood or Structural Panels

See Section 8.4.9. Refer to fastener manufacturer's data for allowable loads on fasteners. Yield capacity can be estimated by multiplying normal allowable load values for 2.8, or for allowable load values that are listed for wind or seismic loads, multiply by 2.1 to obtain estimated yield values.

8.5 Wood Diaphragms

The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size, and amount of fasteners, presence of perimeter chord or flange members, and the ratio of span to depth of the diaphragm. Openings or penetrations through the diaphragm also effect the behavior and capacity of the diaphragm (see Section 8.5.11).

The expected capacity of the diaphragm, Q_{CE}, is determined from the yield shear capacity of the existing or enhanced diaphragm as described in Sections 8.5.2 through 8.5.9. For braced or horizontal truss type systems, the expected capacity, Q_{CE}, is determined from the member or connection yield capacity and conventional static truss analysis, as described in Section 8.5.10.

8.5.1 Types of Wood Diaphragms

8.5.1.1 Existing Wood Diaphragms

A. Single Straight Sheathed Diaphragms

Typically, these 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 at each sheathing board. 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.

B. Double Straight Sheathed Diaphragms

Construction 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.

C. Single Diagonally Sheathed Wood Diaphragms

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. The recommended nailing for diagonally sheathed diaphragms is published in the *Western Woods Use Book* (WWPA, 1983) and UBC (ICBO, 1994a). The shear capacity of the diaphragm is dependent on the size and quantity of the nails at each sheathing board.

D. 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 nailed with 8d nails, with two or more nails per board.

E. Double Diagonally Sheathed Wood Diaphragms

Typically, these 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. The recommended nailing for double diagonally sheathed diaphragms is published in the WWPA (1983).

F. Wood Structural Panel Sheathed Diaphragms

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.

G. 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.

8.5.1.2 Wood Diaphragms Enhanced for Rehabilitation

A. Wood Structural Panel Overlays on Straight or Diagonally Sheathed Diaphragms

Diaphragm shear capacity and stiffness can be increased by overlaying new wood structural panels over existing sheathed diaphragms. 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 only to the existing framing members—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. Nails and staples should be of sufficient length to provide the required embedment into framing members below the sheathing.

If a stronger and stiffer diaphragm is desired, the joints of the new wood structural panel overlay can 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 through the sheathing into the framing below. New structural panel overlays shall be connected to the shear wall 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.

B. Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

New wood structural panel overlays may be placed over existing wood structural panel diaphragms to strengthen and stiffen existing diaphragms. The placement of a new overlay over an existing diaphragm should follow the same construction methods and procedures as for straight and diagonally sheathed diaphragms (see Section 8.5.1.2A). Panel joints should be offset, or else the overlay should be placed at a 45-degree angle to the existing wood structural panels.

C. Increased Attachment

In some cases, existing diaphragms may be enhanced by increasing the nailing or attachment of the existing sheathing to the supporting framing. 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.

Plywood diaphragms can also be enhanced by increased nailing or attachment to the supporting framing and by adding blocking to the diaphragm at the plywood joints. In some cases, increased nailing at the plywood panel infield may also be required. If the required shear capacity and/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.

8.5.1.3 New Wood Diaphragms

A. Wood Structural Panel Sheathed Diaphragms

Typically, these consist of wood structural panels—such as plywood or oriented strand board—placed, nailed, or stapled in place on 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 APA publications (APA, 1983) and various codes (e.g., ICBO, 1994a).

B. Single Diagonally Sheathed Wood Diaphragms

See Section 8.5.1.1C.

C. Double Diagonally Sheathed Wood Diaphragms

See Section 8.5.1.1E.

D. Braced Horizontal Diaphragms

See Section 8.5.1.1G. Because the special horizontal framing in the truss is an added structural feature, it is usually more economical to design floor or roof sheathing as a diaphragm in new construction, which eliminates the need for the "X" bracing and stronger 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.

8.5.2 Single Straight Sheathed Diaphragms

8.5.2.1 Stiffness for Analysis

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. The deflection of straight sheathed diaphragms can be approximated using Equation 8-5:

$$\Delta = v\, L^4 / (G_d\, b^3) \qquad (8\text{-}5)$$

where:

b	=	Diaphragm width, ft
G_d	=	Diaphragm shear stiffness, lb/in.
L	=	Diaphragm span, ft between shear walls or collectors
v	=	Maximum shear in the direction under consideration, lb/ft
Δ	=	Calculated diaphragm deflection, in.

For straight sheathed diaphragms with or without chords, G_d = approximately 200,000 lb/in.

8.5.2.2 Strength Acceptance Criteria

Straight sheathed diaphragms have a low yield capacity of approximately 120 pounds per foot for chorded and unchorded diaphragms. The yield capacity for 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 (1981) for a discussion of calculating the shear capacity of straight sheathed diaphragms.

8.5.2.3 Deformation 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. See Table 8-1 for m factors for use in Equation 3-18 for the LSP.

The coordinates for the normalized force-deflection curve for use in nonlinear procedures are shown in Table 8-2. The values in this table refer to Figure 8-1 in the following way. Distance d (see Figure 8-1) is considered the maximum deflection the diaphragm can undergo and still maintain its yield strength. Distance e is the maximum deflection at a reduced strength c.

8.5.2.4 Connections

The load capacity of connections between diaphragms and shear walls or other vertical elements, as well as diaphragm chords and shear collectors, is very important. These connections should have sufficient load capacity and ductility to deliver the required force to the vertical elements without sudden brittle failure in a connection or series of connections.

8.5.3 Double Straight Sheathed Wood Diaphragms

8.5.3.1 Stiffness for Analysis

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. It is important that both layers of straight sheathing have sufficient nailing, and that the joints of the top layer are either offset or perpendicular to the bottom layer. The approximate deflection of double straight sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Double straight sheathing, chorded:	G_d = 1,500,000 lb/in.
Double straight sheathing, unchorded:	G_d = 700,000 lb/in.

8.5.3.2 Strength Acceptance Criteria

Typical yield shear capacity of double straight sheathed diaphragms is approximately 600 pounds per foot for chorded diaphragms. For unchorded diaphragms, the typical yield capacity is approximately 400 pounds per foot. 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.

8.5.3.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.3.4 Connections

See Section 8.5.2.4.

8.5.4 Single Diagonally Sheathed Wood Diaphragms

8.5.4.1 Stiffness for Analysis

Single diagonally sheathed diaphragms are significantly stiffer than straight sheathed diaphragms, but are still quite flexible. The deflection of single diagonally sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Single diagonal sheathing, chorded:	G_d = 500,000 lb/in.
Single diagonal sheathing, unchorded:	G_d = 400,000 lb/in.

8.5.4.2 Strength Acceptance Criteria

Diagonally sheathed diaphragms are usually capable of resisting moderate shear loads. Typical yield shear capacity for diagonally sheathed wood diaphragms with chords is approximately 600 pounds per foot. Typical yield capacity for unchorded diaphragms is approximately 70% of the value for chorded diaphragms, or 420 pounds per foot. The shear capacity of diagonally sheathed diaphragms can be calculated based on the shear capacity of the nails in each of the sheathing boards. 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 is 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 (1981) for methods of calculating the shear capacity of diagonally sheathed diaphragms.

8.5.4.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.4.4 Connections

See Section 8.5.2.4.

8.5.5 Diagonal Sheathing with Straight Sheathing or Flooring Above Wood Diaphragms

8.5.5.1 Stiffness for Analysis

Straight sheathing or flooring over diagonal sheathing will provide a significant increase in stiffness over single sheathed diaphragms. The approximate deflection of diagonally sheathed diaphragms with straight sheathing or flooring above can be calculated using Equation 8-5, with G_d as follows:

Diagonal sheathing with straight sheathing, chorded:	G_d = 1,800,000 lb/in.
Diagonal sheathing with straight sheathing, unchorded:	G_d = 900,000 lb/in.

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

8.5.5.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. Typical yield capacity for these diaphragms is approximately 900 pounds per foot for chorded diaphragms and approximately 625 pounds per foot for unchorded diaphragms. 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 at each support.

8.5.5.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.5.4 Connections

See Section 8.5.2.4.

8.5.6 Double Diagonally Sheathed Wood Diaphragms

8.5.6.1 Stiffness for Analysis

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 approximate deflection of double diagonally sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Double diagonal sheathing, chorded:	G_d = 1,800,000 lb/in.
Double diagonal sheathing, unchorded:	G_d = 900,000 lb/in.

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

8.5.6.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm, but these diaphragms are usually suitable for moderate to high shear loads, and have a typical yield capacity of approximately 900 pounds per foot for chorded diaphragms and 625 pounds per foot for unchorded diaphragms. 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. 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.

8.5.6.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.6.4 Connections

See Section 8.5.2.4.

8.5.7 Wood Structural Panel Sheathed Diaphragms

8.5.7.1 Stiffness for Analysis

The response of wood structural panel sheathed diaphragms is dependent on the thickness of the wood structural panels, nailing pattern, and presence of chords in the diaphragm, as well as other factors. The deflection of blocked and chorded wood structural panel diaphragms with constant nailing across the diaphragm length can be determined using Equation 8-6:

$$\Delta_y = 5\, v_y\, L^3 / (8EAb) + v_y\, L / (4Gt) + 0.188\, L\, e_n + \Sigma(\Delta_c X) / (2b) \qquad (8\text{-}6)$$

where:

A = Area of chord 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, based on maximum shear per foot v_y divided by the number of nails per foot
For 8d nails, e_n = .06
For 10d nails, e_n = .04

G = Modulus of rigidity of wood structural panel, psi

L = Diaphragm span between shear walls or collectors, ft

t = Effective thickness of plywood for shear, in.

v_y = Yield shear in the direction under consideration, lb/ft

Δ_y = Calculated diaphragm deflection 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 can be determined using Equation 8-7:

$$\Delta_y = 5\, v_y\, L^3 / (8EAb) + v_y\, L / (4Gt) + 0.376\, L\, e_n + \Sigma(\Delta_c X) / (2b) \qquad (8\text{-}7)$$

The deflection for unblocked diaphragms may be calculated using Equation 8-5, with diaphragm shear stiffness G_d as follows:

Unblocked, chorded diaphragms:	G_d = 800,000 lb/in.
Unblocked, unchorded diaphragms:	G_d = 400,000 lb/in.

8.5.7.2 Strength Acceptance Criteria

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. The yield shear capacity, V_y, of wood structural panel diaphragms with chords can be calculated as follows:

If test data are available: V_y = 0.8 x ultimate diaphragm shear value V_u

If test data are not available: V_y = 2.1 x allowable diaphragm shear value (see ICBO, [1994a] and APA [1983])

For unchorded diaphragms, multiply the yield shear capacity for chorded diaphragms calculated above by 70%. Unchorded diaphragms with $L/b > 3.5$ are not considered to be effective for resisting lateral forces.

8.5.7.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.7.4 Connections

See Section 8.5.2.4.

8.5.8 Wood Structural Panel Overlays on Straight or Diagonally Sheathed Diaphragms

8.5.8.1 Stiffness for Analysis

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. Placement of the new wood structural panel overlay should be consistent with Section 8.5.1.2A. 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 approximate deflection of wood structural panel overlays on straight or diagonally sheathed diaphragms can be calculated using Equation 8-5, with G_d as follows:

Unblocked, chorded diaphragm: G_d = 900,000 lb/in.

Unblocked, unchorded diaphragm: G_d = 500,000 lb/in.

Blocked, chorded diaphragm: G_d = 1,800,000 lb/in.

Blocked, unchorded diaphragm: G_d = 700,000 lb/in.

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

8.5.8.2 Strength Acceptance Criteria

Typical yield capacity for diaphragms with a plywood overlay over existing straight and diagonal sheathing is approximately 450 pounds per foot for unblocked chorded diaphragms and approximately 300 pounds per foot for unblocked unchorded diaphragms. The yield capacity of blocked and chorded wood structural panel overlays over existing sheathing is approximately 65% of the ultimate shear capacity, or 2 times the allowable shear capacity of a comparable wood structural panel diaphragm without the existing sheathing below. The yield capacity of blocked and unchorded wood structural panel overlays over existing sheathing is approximately 50% of the ultimate shear capacity, or 1.5 times the allowable shear capacity of a comparable wood structural panel diaphragm without the existing sheathing below.

8.5.8.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.8.4 Connections

See Section 8.5.2.4.

8.5.9 Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

8.5.9.1 Stiffness for Analysis

Diaphragm deflection shall be calculated according to Equation 8-6. Since two layers of plywood are present, the effective thickness of plywood t will be based on two layers of plywood. The nail slip e_n portion of the equation shall be adjusted for the increased nailing with two layers of plywood. Nail slip in the outer layer of plywood shall be increased 25% to account for the increased slip. It is important that nails in the upper layer of plywood have sufficient embedment in the framing to resist the required force and limit slip to the required level.

8.5.9.2 Strength Acceptance Criteria

Yield shear capacity for chorded diaphragms shall be calculated based on the combined two layers of plywood, using the methodology in Section 8.5.7.2. The yield shear capacity of the overlay should be limited to 75% of the values calculated using these procedures.

8.5.9.3 Deformation Acceptance Criteria

See Section 8.5.2.3.

8.5.9.4 Connections

See Section 8.5.2.4.

8.5.10 Braced Horizontal Diaphragms

8.5.10.1 Stiffness for Analysis

The stiffness and deflection of braced horizontal diaphragms can be determined using typical analysis techniques for trusses.

8.5.10.2 Strength Acceptance Criteria

The strength of the horizontal truss system can be determined using typical analysis techniques for trusses, and is dependent on the strength of the individual components and connections in the truss system. In many cases the capacity of the connections between truss components will be the limiting factor in the strength of the horizontal truss system.

8.5.10.3 Deformation 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 desired damage state of the diaphragm. The individual *m* factors for the components and connections in the horizontal truss system listed in Table 8-1 will need to be applied in the truss analysis for the desired Performance Level.

8.5.10.4 Connections

The load capacity of connections between the members of the horizontal truss and shear walls or other vertical elements is very important. These connections must have sufficient load capacity and ductility to deliver the required force to the vertical elements without sudden brittle failure in a connection or series of connections. See Section 8.3.2.2B.

8.5.11 Effects of Chords and Openings in Wood Diaphragms

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 (1981) and APA (1983) 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 Wood Foundations

8.6.1 Wood Piling

Wood piles are generally used with a concrete pile cap, and simply key into the base of the concrete cap. The piles are usually treated with preservatives; they should be checked to determine whether deterioration has occurred and to verify the type of treatment. 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. Maximum stress in the piles should be based on 2.8 time the values given in the *National Design Specification for Wood Construction, Part VI* (AF&PA, 1991a). Deflection of piles under seismic load can be calculated based on the assumed point of fixity. 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.

For Immediate Occupancy, an *m* factor of 1.25 should be used in the analysis of the pile bending or axial force; for Life Safety, a factor of 2.5 can be used; and a factor of 3.0 can be used for Collapse Prevention.

8.6.2 Wood Footings

Wood grillage footings, sleepers, skids, and pressure-treated all-wood foundations are sometime encountered in existing structures. These footings should be thoroughly inspected for indications of deterioration, and replaced with reinforced concrete footings where possible. The seismic resistance for these types of footings is generally very low; they are essentially dependent on friction between the wood and soil for their performance.

8.6.3 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. Added horizontal and diagonal braces can be used to reduce the flexibility of tall poles or reduce the torsional eccentricity of the structure.

8.6.3.1 Materials and Component Properties

The strength of the components, elements, and connections of a pole structure are the same as for a conventional structure. See Section 8.3 for recommendations.

8.6.3.2 Deformation Acceptance Criteria

Deformation characteristics are the same as for a member or frame that is subject to combined flexural and axial loads; pole structures are analyzed using conventional procedures.

8.6.3.3 Factors for the Linear Static Procedure

It is recommended that an *m* factor of 1.2 be used for a cantilevered pole structure for the Immediate Occupancy Performance Level, a value of 3.0 for Life Safety, and 3.5 for Collapse Prevention. Where concentrically braced diagonals are used or added to enhance the capacity of the structure, an *m* factor of 1.0 should be used for Immediate Occupancy, a value of 2.5 for Life Safety, and 3.0 for Collapse Prevention.

8.7 Definitions

Assembly: A collection of structural members and/or components connected in such a manner that load applied to any one component will affect the stress conditions of adjacent parallel components.

Aspect ratio: Ratio of height to width for vertical diaphragms, and width to depth for horizontal diaphragms.

Balloon framing: Continuous stud framing from sill to roof, with intervening floor joists nailed to studs and supported by a let-in ribbon. (See platform framing.)

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

Composite panel: A structural panel comprising thin wood strands or wafers bonded together with exterior adhesive.

Chord: See diaphragm chord.

Collector: See drag strut.

Condition of service: The environment to which the structure will be subjected. Moisture conditions are the most significant issue; however, temperature can have a significant effect on some assemblies.

Connection: A link between components or elements that transmits actions from one component or element to another component or element. Categorized by type of action (moment, shear, or axial), connection links are frequently nonductile.

Cripple wall: Short wall between foundation and first floor framing.

Cripple studs: Short studs between header and top plate at opening in wall framing or studs between base sill and sill of opening.

Decay: Decomposition of wood caused by action of wood-destroying fungi. The term “dry rot” is used interchangeably with decay.

Decking: Solid sawn lumber or glued laminated decking, nominally two to four inches thick and four inches and wider. Decking may be tongue-and-groove or connected at longitudinal joints with nails or metal clips.

Design resistance: Resistance (force or moment as appropriate) provided by member or connection; the product of adjusted resistance, the resistance factor, confidence factor, and time effect factor.

Diaphragm: A horizontal (or nearly horizontal) structural element used to distribute inertial lateral forces to vertical elements of the lateral-force-resisting system.

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

Diaphragm ratio: See aspect ratio.

Diaphragm strut: See drag strut.

Dimensioned lumber: Lumber from nominal two through four inches thick and nominal two or more inches wide.

Dowel bearing strength: The maximum compression strength of wood or wood-based products when subjected to bearing by a steel dowel or bolt of specific diameter.

Dowel type fasteners: Includes bolts, lag screws, wood screws, nails, and spikes.

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

Dressed size: The dimensions of lumber after surfacing with a planing machine. Usually 1/2 to 3/4 inch less than nominal size.

Dry service: Structures wherein the maximum equilibrium moisture content does not exceed 19%.

Edge distance: The distance from the edge of the member to the center of the nearest fastener. When a member is loaded perpendicular to the grain, the loaded edge shall be defined as the edge in the direction toward which the fastener is acting.

Gauge or row spacing: The center-to-center distance between fastener rows or gauge lines.

Glulam beam: Shortened term for glued-laminated beam.

Grade: The classification of lumber in regard to strength and utility, in accordance with the grading rules of an approved agency.

Grading rules: Systematic and standardized criteria for rating the quality of wood products.

Gypsum wallboard or drywall: An interior wall surface sheathing material sometimes considered for resisting lateral forces.

Hold-down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

King stud: Full height stud or studs adjacent to openings that provide out-of-plane stability to cripple studs at openings.

Light framing: Repetitive framing with small uniformly spaced members.

Load duration: The period of continuous application of a given load, or the cumulative period of intermittent applications of load. (See time effect factor.)

Load/slip constant: The ratio of the applied load to a connection and the resulting lateral deformation of the connection in the direction of the applied load.

Load sharing: The load redistribution mechanism among parallel components constrained to deflect together.

LRFD (Load and Resistance Factor Design): A method of proportioning structural components (members, connectors, connecting elements, and assemblages) using load and resistance factors such that no applicable limit state is exceeded when the structure is subjected to all design load and resistance factor combinations.

Lumber: The product of the sawmill and planing mill, usually not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing machine, crosscutting to length, and matching.

Lumber size: Lumber is typically referred to by size classifications. Additionally, lumber is specified by manufacturing classification. Rough lumber and dressed lumber are two of the routinely used manufacturing classifications.

Mat-formed panel: A structural panel designation representing panels manufactured in a mat-formed process, such as oriented strand board and waferboard.

Moisture content: The weight of the water in wood expressed as a percentage of the weight of the oven-dried wood.

Nominal size: The approximate rough-sawn commercial size by which lumber products are known and sold in the market. Actual rough-sawn sizes vary from the nominal. Reference to standards or grade rules is required to determine nominal to actual finished size relationships, which have changed over time.

Oriented strandboard: A structural panel comprising thin elongated wood strands with surface layers arranged in the long panel direction and core layers arranged in the cross panel direction.

Panel: A sheet-type wood product.

Panel rigidity or stiffness: The in-plane shear rigidity of a panel, the product of panel thickness and modulus of rigidity.

Panel shear: Shear stress acting through the panel thickness.

Particleboard: A panel manufactured from small pieces of wood, hemp, and flax, bonded with synthetic or organic binders, and pressed into flat sheets.

Pile: A deep structural component transferring the weight of a building to the foundation soils and resisting vertical and lateral loads; constructed of concrete, steel, or wood; usually driven into soft or loose soils.

Pitch or spacing: The longitudinal center-to-center distance between any two consecutive holes or fasteners in a row.

Planar shear: The shear that occurs in a plane parallel to the surface of a panel, which has the ability to cause the panel to fail along the plies in a plywood panel or in a random layer in a nonveneer or composite panel.

Platform framing: Construction method in which stud walls are constructed one floor at a time, with a floor or roof joist bearing on top of the wall framing at each level.

Ply: A single sheet of veneer, or several strips laid with adjoining edges that form one veneer lamina in a glued plywood panel.

Plywood: A structural panel comprising plies of wood veneer arranged in cross-aligned layers. The plies are bonded with an adhesive that cures upon application of heat and pressure.

Pole: A round timber of any size or length, usually used with the larger end in the ground.

Pole structure: A structure framed with generally round continuous poles that provide the primary vertical frame and lateral-load-resisting system.

Preservative: A chemical that, when suitably applied to wood, makes the wood resistant to attack by fungi, insects, marine borers, or weather conditions.

Pressure-preservative treated wood: Wood products pressure-treated by an approved process and preservative.

Primary (strong) panel axis: The direction that coincides with the length of the panel.

Punched metal plate: A light steel plate fastening having punched teeth of various shapes and configurations that are pressed into wood members to effect transfer shear. Used with structural lumber assemblies.

Required member resistance: Load effect (force, moment, stress, action as appropriate) acting on an element or connection, determined by structural analysis from the factored loads and the critical load combinations.

Resistance: The capacity of a structure, component, or connection to resist the effects of loads. It is determined by computations using specified material strengths, dimensions, and formulas derived from accepted principles of structural mechanics, or by field or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Resistance factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value, and the manner and consequences of failure.

Row of fasteners: Two or more fasteners aligned with the direction of load.

Rough lumber: Lumber as it comes from the saw prior to any dressing operation.

Subdiaphragm: A portion of a larger diaphragm used to distribute loads between members.

Seasoned lumber: Lumber that has been dried. Seasoning takes place by open-air drying within the limits of moisture contents attainable by this method, or by controlled air drying (i.e., kiln drying).

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Shrinkage: Reduction in the dimensions of wood due to a decrease of moisture content.

Structural-use panel: A wood-based panel product bonded with an exterior adhesive, generally 4' x 8' or larger in size. Included under this designation are plywood, oriented strand board, waferboard, and composite panels. These panel products meet the requirements of PS 1-95 (NIST, 1995) or PS 2-92 (NIST, 1992) and are intended for structural use in residential, commercial, and industrial applications.

Stud: Wood member used as vertical framing member in interior or exterior walls of a building, usually 2" x 4" or 2" x 6" sizes, and precision end-trimmed.

Tie: See drag strut.

Tie-down: Hardware used to anchor the vertical chord forces to the foundation or framing of the structure in order to resist overturning of the wall.

Timbers: Lumber of nominal five or more inches in smaller cross-section dimension.

Time effect factor: A factor applied to adjusted resistance to account for effects of duration of load. (See load duration.)

Waferboard: A nonveneered structural panel manufactured from two- to three-inch flakes or wafers bonded together with a phenolic resin and pressed into sheet panels.

8.8 Symbols

This list may exclude symbols appearing once only, when defined at that appearance.

E	Young's modulus of elasticity of chord members
G	Modulus of rigidity of wood structural panel
G_d	Modulus of rigidity of diaphragm
h	Height of wall
h/L	Aspect ratio
L	Length of wall or floor/roof diaphragm
L/b	Diaphragm ratio
V	Shear to element or component
V_y	Shear to element at yield
b	Depth of floor/roof horizontal diaphragm
b	Diaphragm width, ft
e_n	Nail deformation at yield load level
v	Shear per foot
v_y	Shear per foot at yield
Δ	Deflection of diaphragm or bracing element, in.
Δ_y	Deflection of diaphragm or bracing element at yield

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