C8. Wood and Light Metal Framing (Systematic Rehabilitation)

C8.1 Scope

The scope of Chapter 8 is limited to wood and light metal components and elements that are considered to resist seismic forces as structural members. The chapter includes walls, diaphragms, connections, and other forms of construction. Material is intended to be used with the linear and nonlinear procedures prescribed in Chapter 3. Other wood elements and components are addressed in Chapters 4 and 11.

C8.2 Historical Perspective

C8.2.1 General

The use of wood for building construction is common in most areas of the United States. From colonial times to the present day, many residential structures and smaller commercial, industrial, and institutional buildings have been constructed using wood as the primary building material for the basic structural frame. Generally, the use of wood is limited to small or moderately sized buildings whose superstructures are composed entirely of wood. However, the use of wood as a component or element of virtually every other building type is quite common. Wood floors or roofs in steel frame, masonry, or concrete buildings of extensive size and importance are common in both existing and new construction.

Wood buildings of normal size and shape have performed well in prior moderate earthquakes, with the damage generally limited to nonstructural components. Where large openings, soft stories, and noncontinuous shear walls resulting in offsets of lateral-load-resisting elements exist, as with other materials the performance in significant earthquakes has sometimes been poor. Where torsion of the horizontal diaphragm is utilized to provide seismic resistance, the structures have generally not performed well.

For many years, lateral design of wood buildings typically was based on the assumption that horizontal diaphragms were flexible. Lateral loads were, therefore, distributed to the resisting shear walls based on tributary areas. More recently, it has been recognized that in many cases the relative stiffnesses of the diaphragms and the walls cause the diaphragms to behave more as rigid than as flexible diaphragms. In these cases the loads should be distributed to the walls based on the stiffnesses of the walls rather than the tributary areas to the walls. In addition, it has been general practice to assume that the stiffnesses of the walls were in direct proportion to their length; however, for walls with an aspect ratio greater than one this is generally not true. The effect of bending or overturning can have a greater effect on deflection or wall stiffness than the shear in the wall and distortion of the nails or fasteners.

Due to the relative ease of constructing wood framing, the skill and workmanship of the carpenters and framers should not be assumed. Deviation from codes, accepted plans, and practices is not uncommon. Remodeling and alterations of the structural frame and lateral-force-resisting systems by other trades and building occupants have most likely occurred over the history of the building.

Recently, wood frame construction in urban areas has been extended to three and four stories of apartments or condominiums, often over parking. Many of these buildings, lacking well-conceived designs or good quality construction, have performed poorly in recent earthquakes.

Wood frame residential structures of normal size and shape, even when not specifically engineered to resist seismic loads, generally perform well even in major seismic events. A statistical study of single family detached houses within 10 miles of the epicenter of the Northridge earthquake of January 1994, conducted by the National Home Builders Council, revealed that only a small percentage of the houses performed at levels below the defining characteristics of the Immediate Occupancy Performance Level.

C8.2.2 Building Age

Establishing the age of the building is generally helpful in determining the framing method that may have been used and the materials and structural features that may be found. The age of the building can often be determined from public records and title companies. Local historical societies, preservation groups, city directories, and similar tools—in addition to the architectural features and style of the buildings—can be used to help date the structure.

Buildings constructed prior to 1945 generally will not have plywood sheathing on the floors, roof, or walls. Sheathing of these buildings generally utilized straight or diagonal sheathing boards.

Lumber dimensions have also changed with time. Older structures, built prior to 1940, have members approaching the nominal sizes, while newer buildings have lumber dimensions a half inch to one inch smaller than the nominal size.

Nails have also evolved with time. The early nails were hand wrought. Around 1800, cut nails with a rectangular shank that tapers to a flat point were commonly used. In about 1880 wire nails began replacing the cut nails, but the use of cut nails continued well into the twentieth century. Sampling nails of older buildings can be helpful in establishing approximate ages.

Wood frame walls with wood laths and plaster are commonly found in older wood frame buildings, and were used for interior partition walls of masonry and concrete buildings. Ceilings are often found to be constructed in a similar manner. Wood laths (a quarter inch thick by one and a half inches wide) were nailed to the studs or ceiling joist with one-quarter-inch spaces between the boards. The plaster scratch coat was applied and extruded through the space between the laths, creating physical anchors to the laths. Brown and finish coats were then applied. Over the years, the knobs of plaster have often broken off and the plaster has separated from the wood. Earthquakes often cause sections of the plaster to delaminate from the laths. Plaster ceilings become falling hazards and should be evaluated and corrected as part of the rehabilitation process. For structures constructed prior to 1825, these wood laths are often short, hand split or riven sections of wood that vary in width and thickness. After 1825, the laths were typically manufactured in a mill, resulting in visible circular saw markings. Normally, the laths can be viewed from an attic space by looking at the top side of the ceiling.

Older wood frame buildings were often constructed without plans to show or detail the various conditions and connections of the elements. The standard practice and skill of the carpenter were relied upon to obtain adequate connections. Generally, these older buildings were not systematically designed for the effects of lateral loads, but utilized conventional construction, and were “deemed to comply” with requirements. Many small wood frame buildings in most areas of the country continue to be designed and built on this basis. Load paths tend to be random, with critical ties or connections often completely overlooked.

C8.2.3 Evolution of Framing Methods

Post and beam, half timber, and frame construction are 18th and early 19th century techniques in which posts and beams were used as the general framing method, with the posts at the exterior walls placed three to five feet on center and extending the full height of the building. The spaces between the posts were filled with masonry of various types. This method, although extensively used throughout western Europe and the British Isles, was not generally successful in New England; harsh winters led to the deterioration of the masonry fill materials. Wood siding or brick veneers were found to be more appropriate for the climate. Diagonal bridging or braces between the posts provide the lateral bracing for the walls; these diagonal and vertical members are often exposed on the exterior surface. Many modern frame structures attempt to duplicate the architectural appearance by using exposed boards on or between exterior plaster or brick veneer; however, this is strictly architectural and does not contribute to the lateral strength of the building.

The advent of balloon framing in the early 19th century made the frame building construction techniques essentially obsolete. Appearing around 1830, this new lighter framing method was devised using 2" x 4" and 2" x 6" studs spaced at 16 or 24 inches on centers. The term “balloon framing” arose because the system appeared to be so light when compared with the post and beam or frame system. Balloon framing replaced the post and beam or frame method in the Midwest by 1840; however, it did not spread to the east and west coasts until the 1860 to 1870 time frame. In balloon framing, the studs generally ran the full height of the structure from the first floor to the roof. For multistory structures, the floor framing was supported by a let-in ribbon and the joists were nailed into the sides of the studs. Lateral bracing was achieved by the inclusion of diagonal blocking between the studs, by braces let-in to the studs, or by the finish materials for the interior and exterior walls. Horizontal diaphragms generally consisted of either straight or diagonal sheathing boards.

The balloon framing method creates a poor connection condition for seismic resistance between the floor diaphragm and the exterior wall, since the diaphragm stops at the interior face of the wall and no shear connection is generally present. Around 1910, balloon framing was rapidly replaced by the development of western or platform framing; however, the term balloon framing is still used to indicate full-height studs at a gable or sloped roof condition where no intermediate top plates are present. The major difference between platform framing and balloon framing was that each level of the structure was now constructed separately. The wall framing members are the same as those used for the balloon method, and unless the floor to wall connection is exposed, it is virtually impossible to tell the difference in the building types. Platform framing is the method currently employed for multistory wood frame construction. In the earlier platform framing buildings, bracing was obtained in a fashion similar to that for balloon framing, but in contrast to the balloon frame method, the floor sheathing diaphragm extends out to the exterior wall, resulting in a more positive connection between the exterior walls and floor diaphragm for shear transfer.

For both balloon and platform framed buildings, the finish materials on the stud walls usually provide the lateral resistance for the structure. These materials often perform in a brittle fashion and undergo extensive cracking. Wood lath and plaster wall finish continued to be employed through the 1940s, when they were replaced with gypsum lath or button board and plaster. However, in the mid- to late 1960s gypsum wallboard or drywall—which had been developed some 30 years earlier—became popular, and is now the general finish material in use for interior walls and partitions for both residential and commercial construction.

With the evolution of structural panels, plywood and oriented strand board are typically utilized for both horizontal and vertical lateral bracing systems.

Single side wall construction is a unique type of construction generally used only for barns, out-buildings, and cottages in rural and semi-rural areas. The construction utilizes one-inch vertical boards for the exterior walls, with a sill plate at the base and a top plate for connection of the boards. Spaces between the boards (usually 1" x 10"s or 1" x 12"s) are covered by vertical battens, generally 1" x 2"s. These are generally very low-mass structures, and seismic loads are not usually the critical loading criterion for lateral design. In some residential buildings, single side wall construction has been utilized for interior walls in a similar fashion by using one-inch tongue and groove wood boards vertically. This type of construction is no longer permitted by codes.

The development of three- and four-story multifamily structures created a new set of problems relating to the stacking of tall, narrow shear panels, generally at exterior walls. These shear panels are so flexible that they are often ineffective for resisting loads without large associated deflections. These deflections can result in extensive damage to finish materials and the distribution of loads to walls or components not intended in the design to act as part of the lateral-force-resisting system, or to carry the magnitude of load imposed.

The need to provide for parking at the ground level of buildings often creates seismic resistance problems. As is the case for all construction materials, the interruption of the upper level shear walls at the lower levels, where a garage requires large openings, creates soft story effects or, in some cases, torsional effects that may result in deflections in the support frame at the parking level beyond the limited capacity of the frame to maintain lateral stability.

Prior to the common usage of concrete slab-on-grade construction for residential, commercial, and institutional wood framed buildings, the buildings were typically constructed on raised foundations, sometimes incorporating short wood stud walls below the first level, called cripple walls. This results in the lateral loads from interior walls transferring to the exterior walls, placing an extra demand on the wall-to-foundation connection and the cripple walls. These cripple walls have performed poorly in past earthquakes and generally need to be enhanced by the addition of structural panels.

Light gage metal stud walls, floors, and roof joists have been used for the construction of small structures, sometimes in combination with wood members. The members are generally formed into channel or “C” shapes. Each fabricator varies the size and shape somewhat in order to accommodate various features such as nesting or splicing of sections and, in some cases, the ability to apply finish material with nails. Some shapes have webs punched out in various patterns to allow the passage of conduits or the inclusion of bridging between the studs.

C8.3 Material Properties and Condition Assesment

C8.3.1 General

Before an analysis of an existing building can be conducted or an attempt to strengthen or upgrade the structure can be made, the features of the existing structure must first be determined. The lateral-force-resisting system must be identified and the various elements located and evaluated.

The evaluation process can be conducted at several levels of effort, from a simple walk-through to a complete removal of finish surfaces, along with sampling and testing of existing materials or a mock-up test of existing assemblies. For most buildings, the evaluation of existing conditions will involve the removal of some finish materials so that the structural elements and their condition can be inspected and established.

The analysis should reveal those elements that are critical to the performance of the building. Where high load to capacity is indicated, more effort and intensive inspection of the existing condition and elements should be done.

Mechanical properties of wood are affected by moisture, temperature, load history, and presence of decay. Existing in-place properties may vary significantly from those specified on design drawings or those prevalent in the building's era of construction.

Personnel involved in the quantification of material properties shall be highly experienced in testing practices, proper use and application of methods and procedures, and interpretation of results.

In general, existing wood components that have been subjected to a relatively dry environment (e.g., interior or protected exterior location) and normal loading history will likely possess near-original mechanical properties. However, components exposed to the weather or to an unusual loading history, such as heavy static or dynamic loading, may have reduced mechanical properties. The design professional must also consider these factors when establishing properties and the testing/condition survey protocol.

The performance of wood buildings subjected to seismic loading is, to a great extent, dependent on the connections of the various elements in order for the various parts of the building to remain connected under loads or distortions beyond the "elastic" range.

C8.3.2 Properties of In-Place Materials and Components

C8.3.2.1 Material Properties

Generally, the type of wood used in a particular geographic area is dependent on the availability of the various species at the time of construction. Higher grade lumber is often found in older buildings. If the wood is not easily identified visually, core samples can be taken for identification by experts in wood science.

The grade of the existing material will have to be determined by inspection. However, the condition assessment of the various elements and the existence of proper connections are more important to the performance of the structure than the grade of the material used in the structure.

Where existing framing is covered with finish material, attic spaces and underfloor crawl spaces can be used for a preliminary evaluation to view the type and grade of framing without having to damage or remove finishes.

In some cases, inspection may reveal members or elements that have been heavily damaged by insects or decay. These members will have to be replaced regardless of the load or stress level present. Cores can be taken vertically through glue-laminated beams to evaluate the adhesives used and to test the shear capacity between the laminations.

No matter which method of analysis is used in the rehabilitation effort, a continuous load path is required between the foundation and the walls, frames, floors, and roof of the structure. A missing or weak link between elements in the system will have a serious effect of the performance of the building as a whole.

For performance above the Life Safety Performance Level, the traditional method of design and analysis—assuming that wood diaphragms are flexible and that loads are distributed to resisting elements on a tributary area basis, or that the loads in the various walls are in proportion to their length—is not appropriate. The relative stiffness, including bending and overturning effect of walls, must be considered, and the deflections of the various element must be calculated, rather than relying on arbitrary aspect ratios in order to limit anticipated distortions.

For all steel stud systems with diagonal straps or rods for lateral bracing, the provisions of Chapter 5 should be used. For systems using wood panels for bracing, see Section 8.4 for analysis and acceptability criteria.

C8.3.2.2 Component Properties

A. Elements

Refer to Section 8.4 for a description of the various types of shear walls that might be found in an existing building, and Section 8.5 for a description of the horizontal diaphragms. For existing shear walls, it is recommended that some walls be exposed and the nails and conditions examined for proper construction. Nails smaller than specified, overdriven nails, and ineffective nails lacking proper edge distance can significantly reduce the capacity of the walls or horizontal diaphragms.

Components of the lateral-force-resisting system are most likely to be absent or deficient in all but the most recently built existing buildings. These elements are all required for the full development of the load path necessary to deliver the various loads and forces to the resisting elements. Where they are missing, or inadequately designed or constructed, the structure is likely to undergo damage and distortions that could result in local failures or, in some cases, extensive damage to the entire structure. Dramatic catastrophic failures in prior earthquakes have brought about the requirements for some of these components—such as the need for crossties to extend across buildings in order to anchor heavy wall elements and the need to provide ties or collectors at inside corners or wall offsets to carry loads into the walls at those locations. The presence or absence of chords on a diaphragm has a dramatic effect on the magnitude of deflection that the diaphragm will experience when subjected to lateral loads (see Section 8.5).

Nominal and standard dressed size cross-section dimensions are published in the Supplement to the *National Design Specification for Wood Construction* published by the American Forest & Paper Association (AF&PA, 1991), or in publications by the American Institute of Timber Construction (AITC), American Plywood Association (APA), and other organizations. The era of original construction also dictates sectional dimensions (e.g., size of 2" x 4" studs). Variance in these dimensions is also small, and their effect should not affect component strength or deformation calculations unless they are attributed to a degradation process, excessive shrinkage, or creep.

B. Connections

As with all construction materials, and as stated in the *Guidelines*, connection methods are critical to building performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions. Connection capacity limits the magnitude of load that can be delivered to a connected element; connections should be upgraded to the extent that the connected element can resist the load.

The *m* values given in Table 8-1 for evaluating the connections are based on recent research on wood connections at Virginia Polytechnic (Dolan et al., 1994) on cyclic behavior of nails and bolts. Values for screws and lag bolts were estimates based on perceived performance. Past tests of cyclic performance of shear walls—with screws in lieu of nails—have indicated a lack of ductility. The threads on the screws appear to cause a stress concentration that results in a brittle type failure of the screw with a low number of cycles of load.

When evaluating bolted connections, a large amount of the movement that occurs in the connection is due to the oversize condition of the holes for the bolts, in both the wood and the steel, where applicable. Poor workmanship can result in excessive movement in the joints. Removal and inspection of some bolts in deflection-critical joints will give an indication as to the quality of the work and the amount of movement to be anticipated. When adding bolts to existing connections, it is recommended to match the existing bolt sizes. It should also be noted that smaller bolts have been shown to have more ductility than larger bolts.

Connections of heavy concrete or masonry walls to wood roofs or floors have been shown to be a problem in prior earthquakes. Even where positive metal strap ties are present between the wall and wood framing member, failures have occurred at bolt hole locations due to a lack of ductility in the anchor strap.

C8.3.2.3 Test Methods to Quantify Properties

Certain field tests—such as determination of wood gradation and moisture, and estimation of stress level—

may be performed, but laboratory testing on clear, straight-grained samples removed from existing construction must be done if confirmation of field tests is desired. Particular laboratory test methods that may be employed include measurement of moisture content and specific gravity, direct tensile and compressive strength, preservative presence, and connector strength and withdrawal resistance, as well as other mechanical property tests. For each test, industry standards published by the American Society for Testing and Materials (e.g., ASTM D143, D196, D1761, D1860, D2555, D2915, F606) shall be followed.

Quantifying material properties for most connection components, including bolts and nails, is relatively simple. The individual components can be visually inspected and removed (without disturbing physical condition) for evaluation in the laboratory. Expected strength properties for these connectors may be derived from standard laboratory tests similar to those provided in Section 5.3 for steel components and connectors. The influence of connector material properties on behavior of pinned and simple shear connections is generally well understood. However, the multitude of possible configurations and orientations of the connectors may complicate connection analysis. When removing connectors, the condition of the installation shall be noted. Oversized holes or splits in the wood at the connection will prevent the element from full participation.

For structures with archaic or nontraditional wall bracing systems, and where the performance is unknown and it is desired to use the existing elements, a mock-up cyclic test can be conducted to determine the envelope of the hysteretic behavior. From this data, the appropriate *m* factors and control points on the idealized nonlinear distortion backbone curve can be determined.

C8.3.2.4 Minimum Number of Tests

For all laboratory test results, the mean yield and ultimate strength may be interpreted as the default strength for component strength calculations if the coefficient of variation in results is less than 20%. For results with higher variation, to 30%, the expected strength shall be taken as the mean value less the average coefficient of variation as derived via simple statistics. If variabilities higher than 30% are witnessed, further testing shall be performed to identify the source. Such testing shall involve increased sampling and testing in all primary components at each floor level. This result may also indicate the presence of differing material grades in the structural system. Use of ultimate strength values in component capacity calculations shall be based on industry-accepted practices.

If a higher degree of confidence in expected strength values is desired, the sample size shall be determined using ASTM Standard E 22 guidelines. Alternatively, the prior knowledge of material grades from Section C8.3.2.5 may be used, in conjunction with Bayesian statistics, to gain greater confidence with the reduced sample sizes and test results noted above.

C8.3.2.5 Default Properties

The traditional method for designing wood frame buildings and the wood members and elements of other types of buildings has been the allowable stress method. All of the code and material reference standards provide information based on the allowable stresses of the members. The in-grade testing program conducted by AF&PA determined that the limit state or ultimate strength of the materials was, on average, 2.16 times the allowable strength. The load duration factor recommended in the more recent codes for seismic loading is 1.6. A yield load of 80% of ultimate gives a combined factor of 2.76. Therefore, use of a factor of 2.8 is recommended, until such time as the codes and standards are revised to provide the limit state values as appropriate. Other capacity reduction factors—such as moisture exposure, and presence or absence of checks or cracks—should be included in the capacity determination.

The deformation values for the various connectors are based on the cyclic tests of nailed and bolted connections of various types, which were conducted by Dolan et al. (1994). Screw and lag bolt values were estimated from the test data.

C8.3.3 Condition Assessment

C8.3.3.1 General

The features of the existing structure must first be determined. This can be based on field measurements of the building or, ideally, from a set of record construction documents. With many existing structures, especially smaller wood frame structures, plans are not available. Searches of current and former owners', architects', engineers', and city or county records, and contractor files can sometimes yield valuable information concerning an existing structure; these resources should be investigated.

An estimate of the mass of the structure is required in order to determine the seismic load demand on the structure, irrespective of the analysis method used (and even if the Simplified Rehabilitation Method is used). Thus, the size and condition of all the various parts of the building must be determined in order to establish the dead load of the building.

A predetermined systematic methodology needs to be established to determine the character of the lateral-force-resisting elements and the specific connections or load transfer elements that are to be investigated. The investigation should include critical locations in the building as well as a general condition survey. A preliminary analysis will determine these critical element locations or "hot spots" so that the expense and inconvenience of removing otherwise serviceable finish surfaces can be controlled and limited.

After the preliminary analysis has been completed, a more detailed investigation of the building can be conducted on those elements and connections that are critical to the building performance with a high load demand to capacity ratio (DCR).

C8.3.3.2 Scope and Procedures

All of the primary lateral-load-resisting elements of the structure need to be assessed as to their features and conditions. This will often involve the removal of finish materials to observe the existing conditions. The availability or absence of record drawings has a great effect on the amount of removal required.

The following paragraphs identify those nondestructive methods having the greatest use and applicability to assessment.

- Surface Nondestructive examination (NDE) methods for wood components include coring, drilling, probing, and sounding. These methods may be used in parallel with visual inspection to find surface degradation such as decay, splitting, service-induced cracks, and other degradation. These methods do not require significant equipment, but depend on suitable access and expertise in application for successful results. Moisture meters may also be used to assess the presence of decay and conditions producing reduced mechanical properties.

- Volumetric NDE methods, including radiography and ultrasonic stress wave testing, may be used to identify the presence of internal discontinuities in base materials, as well as to identify loss of section or strength. Ultrasonics is particularly useful because of the ease of implementation and the ability to estimate elastic properties of the wood (if density is known). Volumetric NDE of wood requires significant expertise because of the number of variables that may influence results.

- Structural condition and performance may be assessed through on-line monitoring using acoustic emissions and strain gauges, and in-place static or dynamic load tests. Monitoring is used to determine if active degradation or deformations are occurring, while nondestructive load testing provides direct insight on component and element strength.

- Reinforcing location devices can be used to verify the presence of metal hardware at various locations. Some of the locations will still need to be exposed to verify the electronic results and to determine the number of nails, bolts, and other hardware.

C8.3.3.3 Quantifying Results

As previously noted, in the absence of degradation, component section properties have been found to be statistically close to nominal published values. Unless splitting or other mechanism is observed in the condition assessment as causing sectional loss, the cross-sectional area and other sectional properties shall be taken as in the design drawings. If some material damage has occurred, the loss of wood or connector capacity shall be quantified via sampling and laboratory testing. The sectional properties shall then be reduced accordingly, using the laws of structural mechanics. If the degradation is significant, rehabilitative measures shall be undertaken on the deficient component(s). The connection of the members and elements warrants special attention, as failures often occur at the connection rather than in the members or elements themselves. Existing condition may result in both a reduction in capacity and a reduction in ductility, which must be evaluated and incorporated into the analysis.

C8.3.4 Knowledge (κ) Factor

The assignment of knowledge (κ) factors is to a large extent dependent on the availability of a reliable set of plans for the original building. Older building plans for wood frame structures often contained very little structural information and are thus of minimal use; in such cases, the structure should be classed along with those for which no original plans are available. Where new elements are being installed, the κ factor is not applicable. New elements should be designed in accordance with the *NEHRP Recommended Provisions for New Buildings* (BSSC, 1995), incorporating the appropriate phi (ϕ) factor where applicable.

Using the defined κ factor and allowable stresses derived from testing or other source (e.g., *National Design Specifications*, AF&PA, 1991), the stress capacity of the component(s) for different limit states may be established. Adjustment of the stress capacity may be applied on a composite basis to the building or on the basis of individual components. For components with strengths derived from testing, the capacity and deformation limits shall be adjusted by multiplying the strength or deformation limit by a κ factor of 1.0. For wood components not tested and found in fair or better condition, with limited amounts of warping, splitting, or other minor degradation, the capacity shall incorporate a κ factor of 0.75. For wood found in poor condition, rehabilitative measures shall be undertaken, with attention paid to mitigating the cause for existing degradation. In all capacity calculations, the adjustment factors for size, environmental conditions, and load history shall be considered. For connections where plans do not exist, the condition must be exposed to establish the number and size of bolts, nails, and other connections. These exposed connections should utilize a κ factor of 1.0. If all connections are not exposed, but assumed to be similar to those exposed, a κ factor of 0.75 should be used.

C8.3.5 Rehabilitation Issues

Structural panels are used to provide lateral strength and stiffness to most modern wood frame buildings, and are generally recommended for the retrofit or rehabilitation of horizontal diaphragms and shear walls of existing buildings. The system relies on the in-plane strength and stiffness of the panels and their connection to the framing. Panels are connected together by nailing into the same structural member to, in effect, create one continuous panel. The various panels listed have different strengths and stiffnesses; they are discussed and described in Sections 8.4 and 8.5. The performance of the structural panels is dependent to a great extent on the nailing or attachment to the framing. The nail spacing and effectiveness of the attachment should be investigated if the existing panels are to be relied upon to withstand significant loads. If nails are to be added to existing panels they should be of the same size as the existing nails.

C8.4 Wood and Light Frame Shear Walls

The systematic analysis and design of existing wood and light frame shear walls, presented in the *Guidelines*, is a significant change from present design methodology. Shear walls with the same wall coverings, but of different lengths, are no longer considered to have equal capacity per unit length. Aspect ratio is taken into account, as is tie-down connection efficiency. Stiffnesses and deflections can be calculated. Walls of different construction can be compared on the basis of stiffness for distribution of loads. Wall deflections can be compared to diaphragm deflections for determination of diaphragm flexibility. Moreover, a larger wall assembly can be tested or modeled and used in place of the typical isolated, rectangular shear wall for design. A better, more accurate understanding and analysis of shear walls in buildings will result. Shear walls should be designed on the basis of performance; the *Guidelines* will provide the engineer with a more realistic understanding of shear wall performance.

Existing wood frame shear wall types addressed in this section include wood or metal stud walls with various kinds of sheathing. The sheathing generally defines the shear wall. The common existing sheathings are horizontal or diagonal lumber, horizontal or vertical wood siding, structural panels including plywood, stucco, gypsum plaster on various kinds of lath, various gypsum and wood panels, and combinations of various sheathings. Also included in this section are stud walls with various kinds of braces, and braced frames.

Standard test procedures need to be developed to replicate existing conditions as much as possible. These tests should provide the data needed to determine the strength capacities, stiffnesses, and governing of critical components of wood frame assemblies with various aspect ratios. See SEAOSC (1995) for a draft of a proposed testing standard.

C8.4.1 Types of Light Frame Shear Walls

No commentary is provided for this section.

C8.4.2 Light Gage Metal Frame Shear Walls

No commentary is provided for this section.

C8.4.3 Knee-Braced and Miscellaneous Timber Frames

C8.4.3.1 Knee-Braced Frames

No commentary is provided for this section.

C8.4.3.2 Rod-Braced Frames

These frames act as vertical trusses to resist lateral loads. Typically, the rods act only in tension. Once the capacity of the connection is determined, the elongation of the rods, as well as the movement in the connection of the rod to the wood frame, need to be investigated along with the other joints to establish the strength and stiffness of the frame.

C8.4.4 Single Layer Horizontal Lumber Sheathing or Siding Shear Walls

C8.4.4.1 Stiffness for Analysis

Very little is known about the stiffness of single layer horizontal lumber sheathing or siding shear walls. No cyclic test data for this assembly were found. Some indications of stiffness were derived from one dynamic diaphragm test that was studied. The shear wall stiffness presented in the *Guidelines* is surmised from the limited information available and is probably conservative. Single layer horizontal lumber sheathing or siding is very flexible and will experience degradation of stiffness and shear strength capacity when stressed beyond its yield capacity. The aspect ratio (height-to-length) of the shear wall may be the greatest determining factor of the wall's flexibility. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will probably not affect performance at yield or ultimate strength. More research is needed to more accurately determine the behavior of these shear walls. Where the height-to-width ratio exceeds 1.0, the wall should be disregarded as part of the lateral-force-resisting system.

C8.4.4.2 Strength Acceptance Criteria

For vertical diaphragms, the moment capacity—formed by the nail couple where each board crosses a stud—is obtained by multiplying the lateral strength for the size of nail used by the distance between nails in the same board. The resisting moment furnished by the nail couple is the moment per board per stud spacing. Multiplying the moment due to the nail couple by the number of boards in the height of the diaphragm gives the total moment capacity per stud spacing. Dividing the moment capacity of the nail couples by the wall height gives the lateral load capacity in pounds per stud spacing. This can be converted to pounds per linear foot by dividing by the stud spacing in feet. The allowable shear load per foot can then be multiplied by a factor of 2.8 to obtain the yield capacity of the shear wall. Details such as nailing and width of the individual sheathing boards will determine the capacity of the element. Connections to elements above and below will also determine the performance and force-displacement characteristics. The size of studs, plates, and boundary members will affect performance. Additional information on nail couple analysis can be found in the *Western Woods Use Book* published by the Western Wood Products Association (WWPA, 1983).

This analysis has not been compared to cyclic test results and may not be applicable. The indications from the one dynamic diaphragm test performed were used to provide the estimated yield strength presented in the *Guidelines*. Additional research is needed for greater accuracy.

C8.4.4.3 Deformation Acceptance Criteria

Accurate shear values and the associated deformations for single layer horizontal lumber sheathing or siding have not been developed. However, single layer horizontal lumber sheathing or siding will most likely be too flexible to limit displacements and associated damage. It is not recommended that these shear walls be used to resist lateral loads at higher Performance Levels such as Immediate Occupancy. Where lateral loads on these walls are low, attaining a Life Safety Performance Level is possible. This should be reviewed on a case-by-case basis, because the magnitude of deformation acceptable at Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations of other structural and nonstructural elements.

C8.4.4.4 Connections

No commentary is provided for this section.

C8.4.5 Diagonal Lumber Sheathing Shear Walls

C8.4.5.1 Stiffness for Analysis

The stiffness of diagonal lumber sheathed shear walls has not been determined. As of this writing, no cyclic test data have been found. However, there is some cyclic test data available for horizontal diagonally sheathed diaphragms. These few tests indicate a significant increase in stiffness over single layer horizontal sheathed shear walls. Also, displacements should be significantly less for diagonally sheathed shear walls. Deflections are still large when compared to plywood shear walls. The stiffness values presented in the *Guidelines* are estimated. More research is needed to determine the behavior of these shear walls.

C8.4.5.2 Strength Acceptance Criteria

Cyclic tests of diagonally sheathed shear walls are not available. The yield capacity presented in the *Guidelines* is estimated, based on general information and the limited relationships that can be inferred from the few cyclic diaphragm tests that have been conducted involving diagonal sheathing.

In general, diagonally sheathed shear walls have greater yield capacity than single layer horizontal sheathed shear walls because of the triangulated structural system. The lateral forces are resisted by tension and compression in the sheathing boards, and, because the sheathing boards are laid on a 45-degree angle, forces at the end members are also on a 45-degree angle to the end members. Nailing at the ends of the sheathing boards must be sufficient to transfer the desired force from the sheathing to the end members. The outward and inward thrust from the sheathing boards in compression or in tension introduces bending stresses in the perimeter members. Where shear stresses are high, special consideration must be given to the design of perimeter members for bending forces. The attachments of the perimeter members at the corners of the shear wall are also important. Sufficient attachment must be provided to prevent the perimeter members from separating at the corners due to the bending forces. Details such as the nailing and width of the individual sheathing boards will determine the capacity of the component or element. The sizes of studs, plates, and boundary members will also affect performance.

C8.4.5.3 Deformation Acceptance Criteria

Allowable shear values and the associated deformations for diagonally sheathed shear walls have not been fully developed, due to the lack of cyclic test data. Diagonally sheathed shear walls are suitable where lower Performance Levels are desired. Where a higher Performance Level such as Immediate Occupancy is desired, diagonally sheathed shear walls may or may not provide suitable shear strength, and stiffness depending on load levels. Great care is recommended if these shear walls are used to resist lateral loads at higher Performance Levels such as Immediate Occupancy. The magnitude of deformation acceptable at the Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations of other structural and nonstructural elements.

C8.4.5.4 Connections

No commentary is provided for this section.

C8.4.6 Vertical Wood Siding Shear Walls

C8.4.6.1 Stiffness for Analysis

The stiffness of vertical wood siding shear walls has not been determined. As of this writing, no cyclic test data have been found. Vertical wood siding is very flexible and will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity. The stiffness value presented in the *Guidelines* is a best estimate. More research is needed to determine the behavior of these shear walls.

C8.4.6.2 Strength Acceptance Criteria

Cyclic tests of vertical wood siding shear walls are not available. The yield capacity presented in the *Guidelines* is estimated.

Vertical wood siding develops lateral capacity by nail couples in much the same manner as single layer horizontal wood siding. Since vertical boards are nailed to blocking between the studs, the spacing of the blocking will determine the capacity. Otherwise, the discussion of strength acceptance for horizontal wood sheathing and siding applies equally to vertical siding.

C8.4.6.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for vertical wood siding have not been fully developed due to the lack of cyclic test data. As of this writing, it is not recommended that these walls be used to resist lateral loads at higher Performance Levels such as Immediate Occupancy, even at low load levels.

C8.4.6.4 Connections

No commentary is provided for this section.

C8.4.7 Wood Siding over Horizontal Sheathing Shear Walls

C8.4.7.1 Stiffness for Analysis

Very little is known about the stiffness of wood siding over horizontal sheathing; no cyclic test data were found. Some indications of stiffness can be derived from one dynamic horizontal diaphragm test (ABK, 1981). The shear wall stiffness presented in the *Guidelines* is estimated.

This is a very common type of construction for older existing buildings. Compared to single layer horizontal sheathed shear walls, some additional stiffness—due to the wood siding—is expected for these shear walls. Greater stiffness occurs where the siding layers are at right angles to each other. More research is needed to determine the behavior of these shear walls.

C8.4.7.2 Strength Acceptance Criteria

Cyclic tests of these shear walls are not available. The yield capacity presented in the *Guidelines* is estimated, based on the general information noted and the limited relationships that can be inferred from the few available cyclic diaphragm tests involving two layers of transverse sheathing.

Typically, the horizontal sheathing will take most of the load, as it is the stiffer element. Some additional strength from lamination of siding and sheathing may occur, especially with vertical siding over horizontal sheathing. Details such as nailing and width of the individual sheathing boards will determine the capacity of the component or element. Connections to components or elements above and below will also determine the performance and force-displacement characteristics. The size of studs, plates, and boundary members will affect performance. More research is needed to determine the behavior of these shear walls.

C8.4.7.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for wood siding over horizontal sheathing have not been fully developed, due to the lack of cyclic test data. Great care is recommended if these shear walls are used to resist lateral loads at higher Performance Levels such as Life Safety or Immediate Occupancy. Wood siding over horizontal sheathing will probably be too flexible to limit displacements and associated damage to an acceptable level, except in areas of low seismicity. The magnitude of deformation acceptable at Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations for other structural and nonstructural elements.

C8.4.7.4 Connections

No commentary is provided for this section.

C8.4.8 Wood Siding over Diagonal Sheathing Shear Walls

C8.4.8.1 Stiffness for Analysis

Very little is known about the stiffness of wood siding over diagonal sheathing. As of this writing, no cyclic test data were found. Some indications of stiffness could be derived from one dynamic horizontal diaphragm test (ABK, 1981). The shear wall stiffness presented in the *Guidelines* is an estimate.

The cyclic test data available for horizontal diaphragms indicate that a significant increase in stiffness could be expected over single layer diagonally sheathed shear walls. The outside layer of wood siding has a stiffening effect on the diagonal sheathing and counteracts the bending effects in the edge members. As previously stated, these bending effects are present in single layer diagonally sheathed shear walls and can cause decreased stiffness in the shear wall. More research is needed to determine the behavior of these shear walls.

C8.4.8.2 Strength Acceptance Criteria

Cyclic tests of wood siding over diagonally sheathed shear walls are not available. The yield capacity presented in the *Guidelines* is estimated, based on the general information noted below and the limited relationships that can be inferred from the one dynamic horizontal diaphragm test involving straight sheathing over diagonal sheathing.

Typically, the diagonal sheathing would take the load as the stiffer element until failure. Some additional strength from lamination of siding and sheathing certainly will occur. Tests from horizontal diaphragms with straight sheathing over diagonal sheathing suggest that this type of shear wall may be suitable for moderate to fairly high shear loads. For shear walls with wood siding over diagonal sheathing, the forces in the diagonal sheathing will produce bending in the perimeter members that is counteracted by the wood siding. This counteracting of force within the shear wall assembly may relieve the perimeter members of bending stresses. Because of this reduction of bending in the perimeter members, both the yield capacity and stiffness of the shear wall are increased over those of a diagonally sheathed shear wall. Detailing, such as nailing and width of the individual sheathing boards, will also determine the capacity of the component or element. The size of studs, plates, and boundary members will also affect performance.

C8.4.8.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for wood siding over diagonally sheathed shear walls have not been fully developed, due to the lack of cyclic test data. Wood siding over diagonally sheathed shear walls may be used for higher Performance Levels such as Life Safety and Immediate Occupancy, due to the increase in yield capacity and stiffness. Full-scale mock-up cyclic load tests are recommended if these shear walls are used to resist lateral loads at these higher Performance Levels.

C8.4.8.4 Connections

No commentary is provided for this section.

C8.4.9 Structural Panel or Plywood Panel Sheathing Shear Walls

C8.4.9.1 Stiffness for Analysis

Deflections for structural panel or plywood panel sheathed shear walls can be calculated according to the methods shown in Section 8.4.9 of the *Guidelines*. These methods are based on the *Uniform Building Code* (UBC) (ICBO, 1994a), and various APA publications. A significant amount of monotonic shear wall testing has been performed by the APA. In addition, some cyclic loading test data are available for plywood panel sheathing and structural panel shear walls. However, because there is no standard testing procedure or data recording protocol for cyclic loading tests, much of the information supplied in the tests is incomplete. The stiffness of wood structural shear walls is affected by the thickness, the height-to-length ratio, the nailing pattern, the blocking, and the tie-downs of panels, as well as other factors. The stiffness cannot be determined with great accuracy. More cyclic testing is needed to determine the behavior of these shear walls.
Equation 8-2 is taken from Section 23.223 of the UBC (ICBO, 1994a) with (h/b) modifier added to the deflection component d_a. The accuracy of this equation needs confirmation by additional research. Of particular concern is deflection due to anchorage details; the effect on wall performance can be significant and may overshadow all other factors. At present there is very limited information on d_a values.

C8.4.9.2 Strength Acceptance Criteria

Tables with allowable shear values for various types of wood structural panel shear walls have been published by a number of building code agencies, and industry organizations such as the APA. These tables contain allowable shear values that are derived from monotonic tests.

A standard cyclic test would be valuable to determine allowable cyclic shear values for these shear walls. Presently, the ultimate cyclic capacity can be estimated as 80% of the static ASTM-E72 ultimate as determined by APA tests. This estimate is only applicable to walls with aspect ratios of 1.0 or less. There are some tests from Japan (Yasumura, 1992) that support this estimate. Detailing, such as nailing and thickness of panels, will determine the capacity of the component or element. Connections to components or elements above and below the wall will also determine performance and force-displacement characteristics. The size of studs, plates, and boundary members will also affect performance. Components and elements with openings will be more flexible. Equation 8-4 is taken from Yasumura (1992).

Wood structural panel shear walls have a broad range of shear capacities and stiffnesses; therefore, these shear walls are suitable for a wide range of Performance Levels. Shear wall capacity and stiffness must be compatible with the desired Performance Level and the level of acceptable damage. At higher Performance Levels such as Immediate Occupancy, wood structural panel shear walls are capable of higher yield capacities with decreased displacements, due to higher stiffness as compared to other types of shear walls. Figure 8-1 was constructed using: (1) adjusted available values, (2) equations for deflection (from Section 23.223 of the 1994 UBC), (3) Yasumura (1992), and (4) a comparison of the backbone curves from test results with constructed backbone curves. Future research should provide a more accurate method for constructing a backbone curve. Future research should also provide more information on larger wall assemblies, with various size openings.

C8.4.9.3 Deformation Acceptance Criteria

No commentary is provided for this section.

C8.4.9.4 Connections

No commentary is provided for this section.

C8.4.10 Stucco on Studs, Sheathing, or Fiberboard Shear Walls

C8.4.10.1 Stiffness for Analysis

The stiffness of stucco shear walls has not been determined. As of this writing, no cyclic test data were found, and therefore no shear wall stiffness has been determined. The stiffness given in the *Guidelines* is estimated based on the following information. Stucco on studs is brittle and will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity. The aspect ratio of the shear wall may control the wall's flexibility. More research is needed to determine the behavior of these shear walls.

C8.4.10.2 Strength Acceptance Criteria

The performance of stucco shear walls may have two stages. In the first stage, before yielding, the stucco shear wall will be stiff, similar to a concrete wall. For the second stage, the stucco shear wall will be flexible from yielding and wire deformation. The capacity given in the *Guidelines* is estimated for the first stage of performance. Detailing, such as nailing or stapling of the stucco nettings, will effect the capacity of the component or element. The size of studs, plates, and boundary members will also affect performance. Components or elements with openings will be more flexible. Connections to elements above and below will also determine performance and force-displacement characteristics.

C8.4.10.3 Deformation Acceptance Criteria

A stucco shear wall is expected to have a higher yield capacity than a gypsum plaster wall and, due to the brittle nature of stucco, a smaller elastic range than a plywood wall. Allowable shear values and associated deformations for stucco have not been developed, due to the lack of cyclic test data. Stucco shear walls should be considered to be brittle.

C8.4.10.4 Connections

No commentary is provided for this section.

C8.4.11 Gypsum Plaster on Wood Lath Shear Walls

C8.4.11.1 Stiffness for Analysis

Very little is known about the stiffness of gypsum plaster on wood lath. As of this writing, no cyclic test data were found, and therefore no shear wall stiffness could be determined. The stiffness given in the *Guidelines* is an estimate. Gypsum plaster on wood lath is relatively stiff until the plaster cracks; after that the wall becomes more flexible. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will not affect performance at yield or ultimate strength. More research is needed to determine the behavior of these shear walls.

C8.4.11.2 Strength Acceptance Criteria

Cyclic tests of gypsum plaster on wood lath are not available. The yield capacity presented in the *Guidelines* is an estimate. The strength of the plaster probably governs the capacity. Detailing, such as nailing, may have some influence in determining the capacity of the component or element. After the plaster cracks, strength is reduced and flexibility will increase.

C8.4.11.3 Deformation Acceptance Criteria

Due to the lack of cyclic test data, allowable shear values and associated deformations for gypsum plaster on wood lath have not been fully developed. These shear walls are not recommended to resist lateral loads at higher Performance Levels such as Immediate Occupancy. Gypsum plaster on wood lath will most likely be too flexible after the plaster cracks to limit displacements and associated damage to an acceptable level.

C8.4.11.4 Connections

No commentary is provided for this section.

C8.4.12 Gypsum Plaster on Gypsum Lath Shear Walls

C8.4.12.1 Stiffness for Analysis

The stiffness of gypsum plaster on gypsum lath has not been fully determined. As of this writing, no shear wall stiffness could be determined, because no cyclic test data were found. The stiffness given in the *Guidelines* is an estimate. Gypsum plaster on gypsum lath should be relatively stiff until the plaster cracks; after that the wall becomes more flexible. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will not affect performance at yield or ultimate strength. More research is needed to determine the behavior of these shear walls.

C8.4.12.2 Strength Acceptance Criteria

Cyclic tests of gypsum plaster on gypsum lath are not available. The yield capacity presented in the *Guidelines* is an estimate. The strength of the combined plaster and lath will probably govern the capacity. Detailing such as nailing should have some influence in determining the capacity of the component or element. After the plaster and lath crack, strength is reduced and flexibility will increase.

C8.4.12.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for gypsum plaster on gypsum lath have not been fully developed, due to the lack of cyclic test data. These walls are not recommended for resisting lateral loads at higher Performance Levels such as Immediate Occupancy.

C8.4.12.4 Connections

No commentary is provided for this section.

C8.4.13 Gypsum Wallboard Shear Walls

C8.4.13.1 Stiffness for Analysis

Cyclic testing for gypsum wallboard is available from various sources. However, the testing methods differed and results were reported differently. One of the sources is Report No. UCB/EERC-85/06 (Oliva, 1986). The walls in this test were one-sided, without either tie-downs at the end of the walls or dead load applied to the top of the wall to simulate usual conditions. As in the test, most gypsum wallboard shear walls do not have tie-downs at the ends of the walls. If an actual wall frames into a corner at each end of the wall and the aspect ratio is low, a higher ultimate capacity should be expected. Both additional research on the available data and new testing are needed. The effect of the aspect ratio has not been addressed, but may determine the wall's flexibility and mode of failure. The report cited above showed that glued gypsum wallboard panels were much stiffer and stronger, but less ductile. Gypsum wallboard will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity. Cut-in braces and diagonal blocking will provide some additional stiffness at lower force levels, but will probably not affect performance at yield or ultimate strength. As with other wall assemblies, more research is needed to determine the behavior of these shear walls. In the interim, an estimated stiffness is included in the *Guidelines*.

C8.4.13.2 Strength Acceptance Criteria

The strength of the gypsum wallboard, and detailing such as nailing, should have some influence in determining the capacity of the element. The capacity given in the *Guidelines* is an estimate; a more accurate capacity should be available once a standard test method is developed. After the wallboard cracks, or the nails enlarge the holes in the boards, strength is reduced and flexibility increases.

C8.4.13.3 Deformation Acceptance Criteria

The tests available indicate very little deflection can be tolerated without enlargement of nail holes. These shear walls are not recommended for resisting lateral loads at higher Performance Levels such as Immediate Occupancy.

C8.4.13.4 Connections

No commentary is provided for this section.

C8.4.14 Gypsum Sheathing Shear Walls

C8.4.14.1 Stiffness for Analysis

See Section C8.4.13.1.

C8.4.14.2 Strength Acceptance Criteria

See Section C8.4.13.2.

C8.4.14.3 Deformation Acceptance Criteria

See Section C8.4.13.3.

C8.4.14.4 Connections

No commentary is provided for this section.

C8.4.15 Plaster on Metal Lath Shear Walls

C8.4.15.1 Stiffness for Analysis

The stiffness of plaster on metal lath has not been fully determined. At this time, no cyclic test data were found, and therefore no shear wall stiffness could be determined. The stiffness given in the *Guidelines* is an estimate. Plaster on metal lath is relatively brittle and will experience degradation of stiffness and shear capacity when stressed beyond its yield capacity.

Braces and diagonal straps will provide some additional stiffness at lower force levels, but will probably not affect performance at yield or ultimate strength. More research is needed to determine the behavior of these shear walls.

C8.4.15.2 Strength Acceptance Criteria

As with stucco on studs, the performance of plaster on metal lath may have two stages. In the first stage, before yielding, the plaster on metal lath shear wall will be stiff, similar to a concrete wall. For the second stage, the plaster on metal lath shear wall will be flexible from yielding and wire deformation. The capacity given in the *Guidelines* is a best estimate for the first stage of performance. Detailing, such as nailing of the metal lath, will affect the capacity of the component or element. The size of studs, plates, and boundary members will affect performance. Components or elements with openings will be more flexible. Connections to elements above and below will also determine performance and force-displacement characteristics.

C8.4.15.3 Deformation Acceptance Criteria

A plaster on metal lath shear wall is expected to have a higher yield capacity than plaster by itself and, due to the brittle nature of plaster, a smaller elastic range than plywood panel sheathed shear walls. Allowable shear values and associated deformations for plaster on metal lath have not been developed, due to the lack of cyclic test data. Plaster on metal lath shear walls will be too brittle to provide for higher Performance Levels except in areas of low seismicity.

C8.4.15.4 Connections

No commentary is provided for this section.

C8.4.16 Horizontal Lumber Sheathing with Cut-In Braces or Diagonal Blocking Shear Walls

C8.4.16.1 Stiffness for Analysis

See Section C8.4.4.1.

C8.4.16.2 Strength Acceptance Criteria

See Section C8.4.4.2.

C8.4.16.3 Deformation Acceptance Criteria

See Section C8.4.4.3.

C8.4.16.4 Connections

No commentary is provided for this section.

C8.4.17 Fiberboard or Particleboard Sheathing Shear Walls

C8.4.17.1 Stiffness for Analysis

See Section C8.4.9.1.

C8.4.17.2 Strength Acceptance Criteria

See Section C8.4.9.2.

C8.4.17.3 Deformation Acceptance Criteria

See Section C8.4.9.3.

C8.4.17.4 Connections

No commentary is provided for this section.

C8.4.18 Light Gage Metal Frame Shear Walls

C8.4.18.1 Plaster on Metal Lath

See Section C8.4.15.1.

C8.4.18.2 Gypsum Wallboard

See Section C8.4.13.

C8.4.18.3 Plywood or Structural Panels

No commentary is provided for this section.

C8.5 Wood Diaphragms

There are a number of resource documents pertaining to wood diaphragms. Various APA publications and research reports contain more detailed information on analysis methods and testing data for wood diaphragms. *Guidelines for the Design of Horizontal Wood Diaphragms* (ATC, 1981) also contains valuable information on the design and detailing of wood diaphragms. The National Science Foundation (NSF) has sponsored static and dynamic tests of wood diaphragms, performed by the joint venture ABK. This document is entitled *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing* (ABK, 1981).

C8.5.1 Types of Wood Diaphragms

No commentary is provided for this section.

C8.5.2 Single Straight Sheathed Diaphragms

C8.5.2.1 Stiffness for Analysis

Deflection of straight sheathed diaphragms cannot be calculated by rational methods of analysis. The diaphragm shear stiffness has been determined from testing results of typical straight sheathed diaphragms, which are very flexible and experience degradation of stiffness and shear capacity when stressed beyond their yield capacity and at high deflections. More research is needed to determine diaphragm behavior where forces act parallel to the sheathing. Shear capacity parallel to the sheathing boards is dependent on shear transfer between sheathing boards by nails into the framing members.

C8.5.2.2 Strength Acceptance Criteria

For horizontal diaphragms, the moment capacity, formed by the nail couple where each board crosses a joist, is obtained by multiplying the lateral strength for the size of nail used, by the distance between nails in the same board. Dividing this moment by the joist spacing gives the end reaction or shear load per board width. This in turn is multiplied by the ratio of the net width of the board to one foot, which results in the allowable end reaction or shear load in pounds per linear foot for the diaphragm. The allowable shear load per foot can be multiplied by a factor of 2.8 to obtain the yield capacity of the diaphragm. See ATC (1981) for a discussion on calculating the allowable shear capacity of straight sheathed diaphragms.

C8.5.2.3 Deformation Acceptance Criteria

Allowable shear values and associated deformations for straight sheathed diaphragms have been developed for seismic rehabilitation to the Collapse Prevention Performance Level. Great care should be exercised if these diaphragms are used to resist lateral loads at higher Performance Levels such as Immediate Occupancy. Straight sheathed diaphragms will most likely be too flexible to limit displacements and associated damage to an acceptable level, except in areas of low seismicity. The magnitude of deformation acceptable at Life Safety and Immediate Occupancy Performance Levels is dependent on acceptable deformations for other structural and nonstructural components and elements.

C8.5.2.4 Connections

No commentary is provided for this section.

C8.5.3 Double Straight Sheathed Wood Diaphragms

C8.5.3.1 Stiffness for Analysis

Information on force versus displacement curves for double straight sheathed diaphragms has not been located. Further research on the response of double straight sheathed diaphragms would be valuable.

C8.5.3.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. This type of diaphragm is suitable for moderate to high shear loads. Placement of the second layer of straight sheathing will provide a significant increase in both the yield capacity and stiffness of the diaphragm over that of a single sheathed diaphragm. Further research needs to be done on this type of diaphragm to obtain more information on the yield shear capacity.

C8.5.3.3 Deformation Acceptance Criteria

Because of the increased yield capacity and stiffness over many other types of wood diaphragms, double sheathed diaphragms may be compatible with higher Performance Levels such as Life Safety and Immediate Occupancy, where shear demands are low. Diaphragm displacements will need to be compatible with other building materials and the desired Performance Level.

C8.5.3.4 Connections

No commentary is provided for this section.

C8.5.4 Single Diagonally Sheathed Wood Diaphragms

C8.5.4.1 Stiffness for Analysis

Force-versus-displacement curves for these diaphragms have been developed as part of various testing programs. These testing programs indicated a significant increase in stiffness over straight sheathed diaphragms. While displacements will be significantly less than for straight sheathed diaphragms, displacements will still be large. Diaphragm deflections cannot be calculated by rational analysis, and will need to be predicted using the procedures of Section 8.5.4 of the *Guidelines*.

C8.5.4.2 Strength Acceptance Criteria

Diagonally sheathed diaphragms have greater yield shear capacity than straight sheathed diaphragms because of the triangulated structural system. The lateral forces are resisted by tension and compression in the sheathing boards, and, because the sheathing boards are laid on a 45-degree angle, forces at the end members are also on a 45-degree angle to the end members. Nailing at the ends of the sheathing boards must be sufficient to transfer the desired force from the sheathing to the end members. The shear capacity of the diaphragm is the component of the force that is parallel to the end members, which is transferred by the end nailing at each board. The outward and inward thrust from sheathing boards in compression or in tension introduces bending stresses in the perimeter members, in addition to the axial stresses accruing from their position as flange or chord members in the diaphragm. Special consideration must be taken to design the perimeter members for bending forces. The attachment of the perimeter members at the corners of the diaphragm is also important. Sufficient attachment must be provided to prevent the perimeter members from separating at the corners due to the bending forces.

C8.5.4.3 Deformation Acceptance Criteria

Because displacements will be significant for diagonally sheathed diaphragms, they are best suited where lower Performance Levels such as Collapse Prevention are desired. Where higher Performance Levels such as Immediate Occupancy are desired, diagonally sheathed diaphragms may not provide suitable shear strength and stiffness.

C8.5.4.4 Connections

No commentary is provided for this section.

C8.5.5 Diagonal Sheathing with Straight Sheathing or Flooring Above Wood Diaphragms

C8.5.5.1 Stiffness for Analysis

Diaphragm testing programs by ABK (1981) and others indicate a significant increase in stiffness for these diaphragms over single sheathed diaphragms. The upper layer of straight sheathing or flooring has a significant stiffening effect in the diaphragm and counteracts the bending effects in the diaphragm edge members that are present in single diagonally sheathed diaphragms.

C8.5.5.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. This type of diaphragm is suitable for moderate to high shear loads. For diaphragms with diagonal sheathing and straight sheathing or flooring above, the forces in the diagonal sheathing that produce bending in the perimeter members are resisted by the straight sheathing. This cornerstone relieves the perimeter members of bending stresses, leaving only the axial stresses from chord action. Because of this reduction of stress in the perimeter members, both the yield capacity and stiffness of the diaphragm are greatly increased over those of a single sheathed diaphragm.

C8.5.5.3 Deformation Acceptance Criteria

Because of the increased yield capacity and stiffness over many other types of wood diaphragms, diagonally sheathed diaphragms with straight sheathing or flooring above may be more compatible with higher Performance Levels such as Life Safety and Immediate Occupancy. Diaphragm displacements will need to be compatible with other building materials and the desired Performance Level.

C8.5.5.4 Connections

No commentary is provided for this section.

C8.5.6 Double Diagonally Sheathed Wood Diaphragms

C8.5.6.1 Stiffness for Analysis

Testing and related force-versus-displacement information for double diagonally sheathed diaphragms is limited, but the diaphragm will respond similarly to diagonally sheathed diaphragms with straight sheathing or flooring above. Double sheathed diaphragms will be significantly stiffer than single sheathed diaphragms. Further research on the response of double diagonally sheathed diaphragms would be valuable.

C8.5.6.2 Strength Acceptance Criteria

Shear capacity is dependent on the nailing of the diaphragm. When double diagonal sheathing is used, the outward forces on the perimeter members from that portion of the sheathing in compression, are counteracted by the inward forces from that portion of the sheathing in tension. This counteracting of forces within the sheathing assembly relieves the perimeter members of bending stresses, leaving only the axial stresses from their chord action. Because of this reduction of bending in the perimeter members, both the yield capacity and stiffness of the diaphragm are increased over those of a single sheathed diaphragm.

C8.5.6.3 Deformation Acceptance Criteria

Because of the increased yield capacity and stiffness over many other types of wood diaphragms, double diagonally sheathed diaphragms are more compatible with higher Performance Levels such as Life Safety and Immediate Occupancy. Diaphragm displacements will need to be compatible with other building materials and the desired Performance Level.

C8.5.6.4 Connections

No commentary is provided for this section.

C8.5.7 Wood Structural Panel Sheathed Diaphragms

C8.5.7.1 Stiffness for Analysis

Deflections for wood structural panel diaphragms can be calculated according to the accepted methods shown in Section 8.5.7 of the *Guidelines*, which are based on ATC (1981), UBC (ICBO, 1994a), and various APA publications. A significant amount of monotonic diaphragm testing has been performed by APA and other agencies. Some dynamic testing was performed during the ABK (1981) testing program. Testing programs have indicated that wood structural panel diaphragms that are blocked and chorded are stiffer and have a higher shear capacity than unblocked or unchorded wood structural panel diaphragms and many other types of wood diaphragms. Even with this increase in stiffness, wood structural panel diaphragms are still considered to be flexible diaphragms in most cases. In cases with low diaphragm length-to-width ratios and fairly flexible vertical lateral-force-resisting elements, wood structural panel diaphragms may need to be considered as rigid or semi-rigid diaphragms.

C8.5.7.2 Strength Acceptance Criteria

Tables with allowable shear values for various types of wood structural panel diaphragms have been published by a number of building code agencies and industry organizations such as APA. These diaphragms have a fairly broad range of allowable shear capacities. Yield capacities for wood structural panel diaphragms can be estimated by multiplying the allowable shear values by a factor of 2.1 for chorded diaphragms and 1.75 for unchorded diaphragms. The factor 2.1 is used in lieu of a 2.8 factor because a load duration factor of 1.33 is included in the National Design Specification (AF&PA, 1991) value.

C8.5.7.3 Deformation Acceptance Criteria

Wood structural panel diaphragms have a broad range of shear capacity and stiffness, so the diaphragms may be suitable for a broad range of Performance Levels. Diaphragm shear capacity and stiffness must be compatible with the desired Performance Level and the level of allowable damage. At higher Performance Levels such as Life Safety and Immediate Occupancy, wood structural panel diaphragms are capable of higher yield capacities with decreased displacements, due to higher stiffness.

C8.5.7.4 Connections

No commentary is provided for this section.

C8.5.8 Wood Structural Panel Overlays On Straight or Diagonally Sheathed Diaphragms

C8.5.8.1 Stiffness for Analysis

Testing of these diaphragms has been performed by APA as well as ABK (1981). The wood structural panel overlay creates a very significant increase in diaphragm strength and stiffness when placed over a straight sheathed diaphragm. When a new wood structural panel overlay is placed over a diagonally sheathed diaphragm, the increase in strength and stiffness will not be proportional to that achieved for a straight sheathed diaphragm, but will still be significant. This is due to the initial stiffness of the diagonally sheathed diaphragm being higher than that of the straight sheathed diaphragm.

C8.5.8.2 Strength Acceptance Criteria

The allowable shear capacity for wood structural panel overlays has been limited by the *Uniform Code for Building Conservation* (UCBC) (ICBO, 1994b) to 225 pounds per foot for unblocked diaphragms, regardless of the nailing used to attach the plywood to the supporting framing members. For blocked wood structural panel diaphragms, the UCBC limits the allowable shear capacity of the overlay to 75% of the value specified for the horizontal diaphragm shear table of the UBC (ICBO, 1994a). The reason for the lower values is that the nail sizes commonly used for nailing of wood structural panels have required embedment lengths that exceed the board thickness of the existing sheathing. Splitting of the existing sheathing boards is also common, especially at the closely spaced edge nailing at the perimeter of the wood structural panels.

The values given for wood structural panels applied over existing sheathing boards are for that assembly only. If the existing boards are removed and the wood structural panels are applied directly to the existing framing members, shear values discussed in Section 8.5.7 should be used. The increase in allowable shear capacity will provide a moderate to high increase in diaphragm yield capacity over that provided by the existing sheathing. Diaphragm yield capacity and displacement requirements at various Performance Levels will need to be coordinated with the force-versus-displacement curves to ensure compatibility with the type of construction of the existing components and elements in the building.

C8.5.8.3 Deformation Acceptance Criteria

Wood structural panel overlays on existing sheathed diaphragms have a broad range of shear capacities and stiffnesses, so the diaphragms may be suitable for a broad range of Performance Levels. Diaphragm shear capacity and stiffness must be compatible with the desired Performance Level and allowable damage. At higher Performance Levels such as Life Safety and Immediate Occupancy, wood structural panel overlays over existing sheathed diaphragms may be capable of higher yield capacities with decreased displacements, due to higher stiffnesses.

C8.5.8.4 Connections

No commentary is provided for this section.

C8.5.9 Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

C8.5.9.1 Stiffness for Analysis

Some monotonic testing of these diaphragms has been performed by APA. Test results indicate that shear capacity and stiffness of an existing wood structural panel can be increased significantly by adding a new wood structural panel overlay over an existing diaphragm.

C8.5.9.2 Strength Acceptance Criteria

See Section C8.5.8.2.

C8.5.9.3 Deformation Acceptance Criteria

See Section C8.5.8.3.

C8.5.9.4 Connections

No commentary is provided for this section.

C8.5.10 Braced Horizontal Diaphragms

C8.5.10.1 Stiffness for Analysis

The stiffness of braced horizontal diaphragms can vary with different systems, but is most often flexible, with a long period of vibration. Classical deflection analysis procedures can be used to determine the stiffness of the horizontal truss. Length-to-width ratios of the truss system can have a significant effect on the stiffness of the horizontal truss. Lower length-to-width ratios will result in increased stiffness of the horizontal truss; higher length-to-width ratios will result in decreased stiffness of the horizontal truss. Distortion in the rod brace connection shall be incorporated into the truss deflection analysis.

C8.5.10.2 Strength Acceptance Criteria

The size and mechanical properties of the tension rods, compression struts, and connection detailing are all important to the yield capacity of the braced horizontal diaphragm. Standard truss analysis techniques can be used to determine the yield capacity of the braced horizontal diaphragm. Special attention is required at connections between different members of the truss system. Yield capacity of the connections will in many cases limit the yield capacity of the truss system. Connections that will develop the yield capacity of the truss members and reduce the potential for brittle failure are desired. If enhancement of existing braced horizontal diaphragms is required, classical truss analysis methods can be used to determine which members or connections require enhancement. Analysis of existing connections, and enhancement of connections with insufficient yield capacity, should be performed in a manner that will encourage yielding in the truss members rather than brittle failure in the truss connections.

C8.5.10.3 Deformation Acceptance Criteria

More flexible, lower-strength braced horizontal diaphragm systems may perform well for rehabilitation to the Collapse Prevention Performance Level. Upgrades to Life Safety or Immediate Occupancy Performance Levels will require proportional increases in yield capacity and stiffness to control lateral displacements. Displacements must be compatible with the type of construction supported by the horizontal truss system.

C8.5.10.4 Connections

No commentary is provided for this section.

C8.5.11 Effects of Chords and Openings in Wood Diaphragms

Static and dynamic diaphragm testing programs have indicated that wood diaphragms with chords are stiffer than comparable diaphragms without chords. Chords may not be required in the diaphragm for a lower Performance Level such as Collapse Prevention. Documents such as the UCBC and the ABK methodology do not require chords in the diaphragm in most cases. These documents are geared toward Collapse Prevention Performance Levels. Where higher Performance Levels such as Life Safety or Immediate Occupancy are desired, chords will usually be required to limit deflections, except in areas of low seismicity.

Care should be exercised in stiffening diaphragms by overlaying with new materials, adding new chords, or other methods. Increased stiffness in the diaphragm will result in a shorter period of vibration and an associated increase in lateral force on the diaphragm. Under some conditions this decreased period and increased force may not be desirable. If displacements are not critical to the performance of the diaphragm or supported wall elements, the diaphragm may actually perform better at the longer period with a lower dynamic force.

C8.6 Wood Foundations

C8.6.1 Wood Piling

The method of analyzing wood piles is based on past performance and is empirical in application. Environmental conditions, such as changes in the water table, can result in deterioration of piles with a resultant loss in capacity. The assumption of point of restraint or fixity of the pile for lateral load analysis is very subjective and will have a significant effect on the results of the analysis both for stress level and anticipated deflection. Battered piles can be analyzed for static resistance to base shear.

C8.6.2 Wood Footings

Wood is generally not used as a foundation material for permanent structures, although there are code-approved pressure-treated wood systems for the foundations of small residential structures, which have been used in recent years in some areas.

C8.6.3 Pole Structures

Pole type structures, as well as structures constructed above grade on post or pole supports, are used in some areas of the country to reduce flood or storm damage, or accommodate sloping or irregular terrain. If not properly designed and detailed, pole-supported structures can be at high risk under seismic loading.

The pole structure is generally analyzed as a braced frame; it resists lateral loads by both the cantilever action of the poles embedded into the ground and by braced or sheathed frames in the superstructure. Like the wood piles, the stiffness of the structure is dependent on the character of the ground, fixity, and distortion of the soil into which the pole is founded.

C8.7 Definitions

In addition to the *Guidelines* listings, additional terms and descriptions can be found in standard construction dictionaries or encyclopedias. See Section C8.9.

C8.8 Symbols

The symbols used are generally in the form used in the reference material.

C8.9 References

In addition to the following references, many Canadian standards could be used to good advantage in the evaluation of existing buildings and possible upgrading methodologies.

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