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Seismic Design of Wood Light-Frame Structural Diaphragm Systems

A Guide for Practicing Engineers

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National Institute of Standards and Technology
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(see inside back cover.)

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By
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NIST policy is to use the International System of Units (metric units) in all its publications. In this report, however, information is presented in U.S. Customary Units (e.g., inch and pound), because this is the preferred system of units in the U.S. earthquake engineering industry.

Cover photo. Construction of a five-story, wood-frame apartment building.

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1. Introduction

The seismic force-resisting system (SFRS) of a building consists of a three-dimensional collection of elements that transmit loads and forces from the point of occurrence to the foundation and supporting soils. This system typically consists of horizontal and vertical elements (**Figure 1-1**). When resisting seismic forces, the horizontal elements (i.e., roof and floors) are classified as diaphragms and act to transmit the forces horizontally from the point of origin to the vertical elements. The vertical elements (i.e., walls or frames) transmit the forces down to the next lower level or to the foundation. Together, these elements function as a system to provide a complete load path for seismic forces to flow through the building to the foundation and supporting soils. Diaphragms not only act to distribute the forces horizontally to the vertical elements of the system, they also tie the vertical elements together to act as a system so that they share the load rather than respond individually. For seismic forces, the diaphragms are an integral part of the SFRS and deserve significant attention during the design process.

Seismic design of diaphragms is required for buildings in Seismic Design Categories (SDC) B through F, as defined in the *International Building Code* (IBC) (IBC 2012) and *ASCE/SEI 7 Minimum Design Loads for Buildings and Other Structures* (ASCE 7) (ASCE 2010). In most cases, the diaphragm construction will also serve as the floor or roof surface and resist gravity, wind uplift, and other loads in addition to the loads associated with earthquakes. Where a solid floor or roof surface is not required, elements such as horizontal trusses or space frames can serve the same function as solid diaphragms.

This Guide addresses wood light-frame diaphragms used in buildings of all wood light-frame construction, as well as wood light-frame diaphragms used with other vertical elements of the SFRS, including concrete or masonry walls, steel moment frames, and steel braced frames. "Light-frame" refers to the repetitive, closely spaced wood framing (e.g., joists or rafters) to which the diaphragm sheathing is attached. Of the buildings constructed entirely of wood light-frame construction, many are small buildings, with single-family homes of three or less stories being a majority (**Figure 1-2**). Medium-size buildings constructed entirely of wood light-frame construction include multi-family residential buildings (**Figure 1-3**), hotels, schools, and small commercial buildings (**Figure 1-4**). These buildings are of varying sizes. Buildings of up to three stories have been common for many years. Buildings of up to five or six stories are now being constructed with increasing frequency. A number of

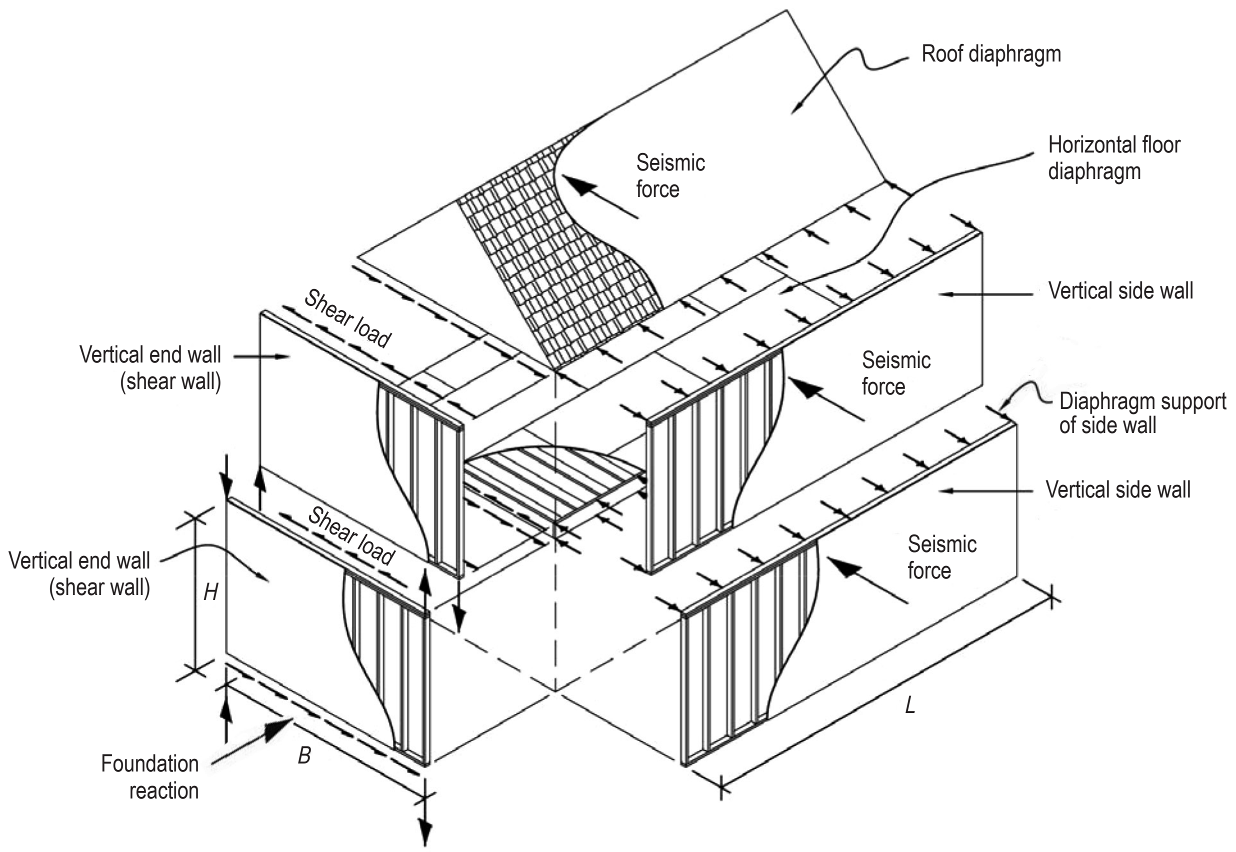


Figure 1-1. Wood light-frame building with load path illustrated (FEMA 2006).

commercial and light-industrial buildings constructed entirely of wood light-frame construction often have a large plan area and are primarily of single-story construction.

Figure 1-2. Single-family residential wood light-frame construction.

Figure 1-3. Multi-family residential wood light-frame construction.

Figure 1-4. Commercial wood light-frame building.

Buildings constructed using wood light-frame diaphragms with concrete and masonry walls, steel frames, or other vertical element types, include commercial, institutional, and light-industrial buildings predominantly of one or two stories and "big-box" retail buildings with a large plan area and predominantly of single-story construction. Concrete tilt-up wall buildings (**Figure 1-5**) represent a significant portion of wood light-frame diaphragm building inventory in the seismically active western states, while steel deck diaphragms are more prevalent in other regions. Additional seismic performance concerns and seismic design requirements are applicable to wood light-frame diaphragms used with concrete or masonry walls; brief discussions of these additional concerns and requirements are provided in this Guide.

Figure 1-5. Concrete tilt-up wall building with wood light-frame roof diaphragm.

Although many of the ideas and analysis methods covered in this Guide are applicable to a wider scope of diaphragm types, this Guide focuses on diaphragms consisting of wood framing (dimension lumber, structural composite lumber, I-joists, metal plate connected wood trusses, or wood nailers attached to steel bar joists) sheathed with wood structural panels (oriented strand board (OSB) or plywood). The sheathing is commonly structurally fastened with nails or staples. Adhesives may be provided between the sheathing and framing to reduce floor squeaking but are not relied on as a structural connection. This Guide addresses platform construction (**Figure 1-6(a)**), where the wall framing extends a single story in height from the top of the foundation or floor below to the bottom of the floor or ceiling/roof above, such that the floor and roof framing are constructed to bear on top of the walls. Balloon framing (i.e., the wall studs are continuous for multiple stories, and the floors are suspended off or let into the inside of the walls) is not a typical framing method for modern construction in the United States and is not addressed in this Guide (**Figure 1-6(b)**).

This Guide is written primarily for practicing structural engineers and should also be useful to architects, building regulators (building officials and plan checkers), and contractors. Students, educators, and others interested in

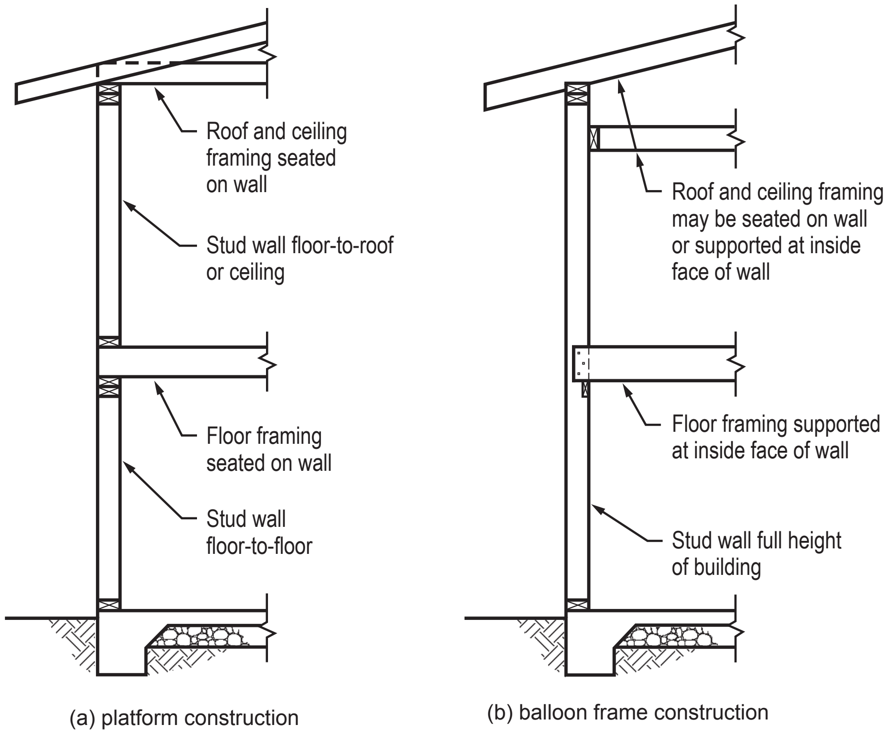


Figure 1-6. Typical wall sections.

understanding the basis for the common design methods used for wood diaphragms will find this document a useful beginning step in expanding that understanding.

Design Requirements for Wood-Frame Diaphragms in IBC, ASCE 7, SDPWS, and NDS

The design requirements for wood-frame diaphragms are in the IBC, ASCE 7, the *Special Design Provisions for Wind and Seismic* (SDPWS) (AWC 2008), and the *National Design Specification for Wood Construction* (NDS) (AWC 2012). The IBC and ASCE 7 primarily define the seismic demand, while the SDPWS and NDS primarily address capacity and related design requirements for diaphragm members, sheathing, and connections. It is recommended that the reader of this Guide have these documents available.

Section 2 of this Guide provides an introduction of the role of the diaphragm in the overall building structure and the way the diaphragm resists both gravity and lateral loads. This is followed in Section 3 with a description of the components that comprise the diaphragm and the way each component functions independently and together to transfer loads. Diaphragm seismic behavior is described in Section 4, along with design philosophy and principles for acceptable seismic performance. Seismic design forces are discussed in Section 5. Detailed guidance for analysis and for design are included in Sections 6 and 7, respectively. Detailing and constructability issues are covered in Section 8. References are provided in Section 9; notations and abbreviations are in Section 10.

This Guide Does Not Address the Following

- Wood structural panel diaphragms with sheathing attached with structural adhesives that are intended to provide capacity
- Diaphragms with sheathing other than wood structural panel sheathing (straight and diagonal board sheathing, gypsum wallboard, structural insulated panels (SIPs) and other stressed skin panel systems)
- Proprietary diaphragm systems
- Proprietary framing (structural composite lumber (SCL), laminated veneer lumber (LVL), or I-joists)
- Proprietary sheathing and/or its fastening
- Proprietary fastening methods for either the connections within the framing or for connections to transfer the forces into the vertical elements of the building
- Horizontal trusses
- Steel-panel-sheathed diaphragms
- Heavy timber construction
- Concrete and timber composite floor systems

2. The Roles of Diaphragms

Diaphragms perform a number of different roles as an element of the building structural system including:

- *Resist vertical loads*—Most diaphragms act as the floor, ceiling, or roof of the building. Therefore, the vertical gravity loads (i.e., dead load, live load, snow load) associated with these elements of the building are supported by the diaphragm. The vertical inertial forces that are induced by earthquake vertical accelerations must also be resisted by the diaphragm.

- *Resist horizontal inertial forces and distribute them to the vertical elements of the SFRS in the story below*—One of the principal functions of the diaphragm is to resist horizontal inertial forces due to the self-weight of the diaphragm and supported components and contents and to distribute these forces to the vertical elements of the SFRS at the story below.

- *Resist out-of-plane forces*—Exterior and interior walls and cladding that are oriented perpendicularly to the direction of seismic accelerations develop out-of-plane seismic forces (as well as out-of-plane wind loads). The wall or cladding connections transfer these forces to the diaphragm, contributing to the forces that the diaphragm transfers to the vertical elements of the SFRS at the story below.

- *Transfer forces through the diaphragm*—The forces associated with the portion of the building above a diaphragm are transferred to the diaphragm through the vertical elements of the SFRS in the story above. These forces may or may not be transferred directly through the diaphragm to aligned (stacked) vertical elements in the story below. When the vertical elements in stories above and below are not aligned, seismic forces are transferred into the diaphragm at discontinued vertical elements above and transferred out of the diaphragm to the vertical elements below. The resulting diaphragm forces, called transfer forces, can be significant contributors to the diaphragm seismic demand. Transfer forces can also occur in diaphragms where the vertical elements are aligned (stacked), but where differences in vertical element stiffness from story to story occur, similarly causing seismic forces from vertical elements above to move through the diaphragm to different vertical elements below. Transfer forces are discussed in ASCE 7 §12.10.1.1.

- *Provide lateral support to vertical elements*—Diaphragms are connected to the vertical elements of the SFRS. Because of this connectivity, diaphragms provide lateral support to, and therefore improve lateral stability of, the vertical elements. In addition, the diaphragm connects all of the vertical elements associated with the story above and the story below and provides the ability for the building to respond to lateral loads as a three-dimensional system rather than as individual elements. This provides alternative load paths (or redundancy) should one or more of the vertical elements become overloaded.

- *Influence dynamic building behavior*—The fundamental period of vibration of buildings with long-span wood light-frame diaphragms is often strongly influenced by the diaphragm, resulting in a longer fundamental building period than is the case with buildings with short-span diaphragms.

- *Redistribute forces due to torsion*—Some building configurations result in torsional response to seismic loading. Although strength is a concern in this situation, diaphragms must also be stiff enough to transfer torsional forces to the vertical elements of the SFRS of the story below. The torsional component of the force is distributed to the lines of resistance both in the direction of loading and the direction perpendicular to loading. Flexible diaphragms may have reduced ability to transfer torsional forces effectively.

- *Transfer forces around openings*—Diaphragms with openings to accommodate stairwells, skylights, elevator shafts, mechanical chases, or large multi-story rooms must transfer the forces around the openings to the vertical elements of the SFRS at the story below. Special detailing is required to accomplish this transfer.

- *Resist soil loads below grade* —Buildings with below-grade levels will have soil pressures applied to the basement walls. Often these walls are designed with the assumption that the top of the wall is supported laterally in the out-of-plane direction of the wall by the floor diaphragm. The resulting diaphragm forces need to be addressed in the diaphragm design and detailing.

3. Diaphragm Components

When resisting in-plane diaphragm forces, the sheathing is generally modeled as resisting only shear, and chord members, which behave analogously to the flanges of a wide-flange beam, are provided to carry tension and compression forces resulting from moments. In addition, collector members transfer diaphragm forces from the sheathing to supporting vertical elements of the SFRS. Collectively the chord and collector elements are referred to as diaphragm boundary elements. Boundary elements are provided at every edge of the diaphragm sheathing, whether at the building perimeter or interior openings, and they are provided at other locations in the interior of the diaphragm where required to transfer forces into or out of the diaphragm. **Figure 3-1(a)** shows a simple diaphragm with chord members provided at the opposite edges (Lines A and B). Collector members will be required at Lines 1 and 2 where wall openings occur. **Figure 3-1(b)** shows a more complex diaphragm with three supports and a collector member internal to the diaphragm at Line 2. The behavior of the diaphragm is more complex than this simplistic modeling approach (because of uneven shear distribution through the diaphragm depth, higher mode effects). This approach, however, has resulted in diaphragm designs that have performed adequately.

3.1 Diaphragm Sheathing

As stated before, it is assumed that the sheathing panels resist the shear forces in the diaphragm, and the chord framing resists the moments. Diaphragm sheathing is made up of panels typically 4 feet by 8 feet. Because of this limited panel size, the mechanism of force transfer between adjacent sheathing panels usually controls diaphragm capacity and stiffness.

Diaphragms may be constructed using either blocked or unblocked construction. **Figure 3-2(a)** illustrates unblocked diaphragm construction, where the sheathing is fastened at only the supporting joists or rafters and boundary elements. **Figure 3-2(b)** illustrates blocked diaphragm construction, where sheathing panel edges not supported on framing members are supported on added wood blocking, allowing sheathing-to-framing fastening to be provided around the entire perimeter of each sheathing panel. Unblocked diaphragms are prevalent in lightly loaded applications where increased strength and stiffness of a blocked diaphragm are not required. Blocking improves diaphragm performance relative to the unblocked diaphragm for a given sheathing fastener size and spacing. Blocking, however, is labor intensive to install and increases construction cost.

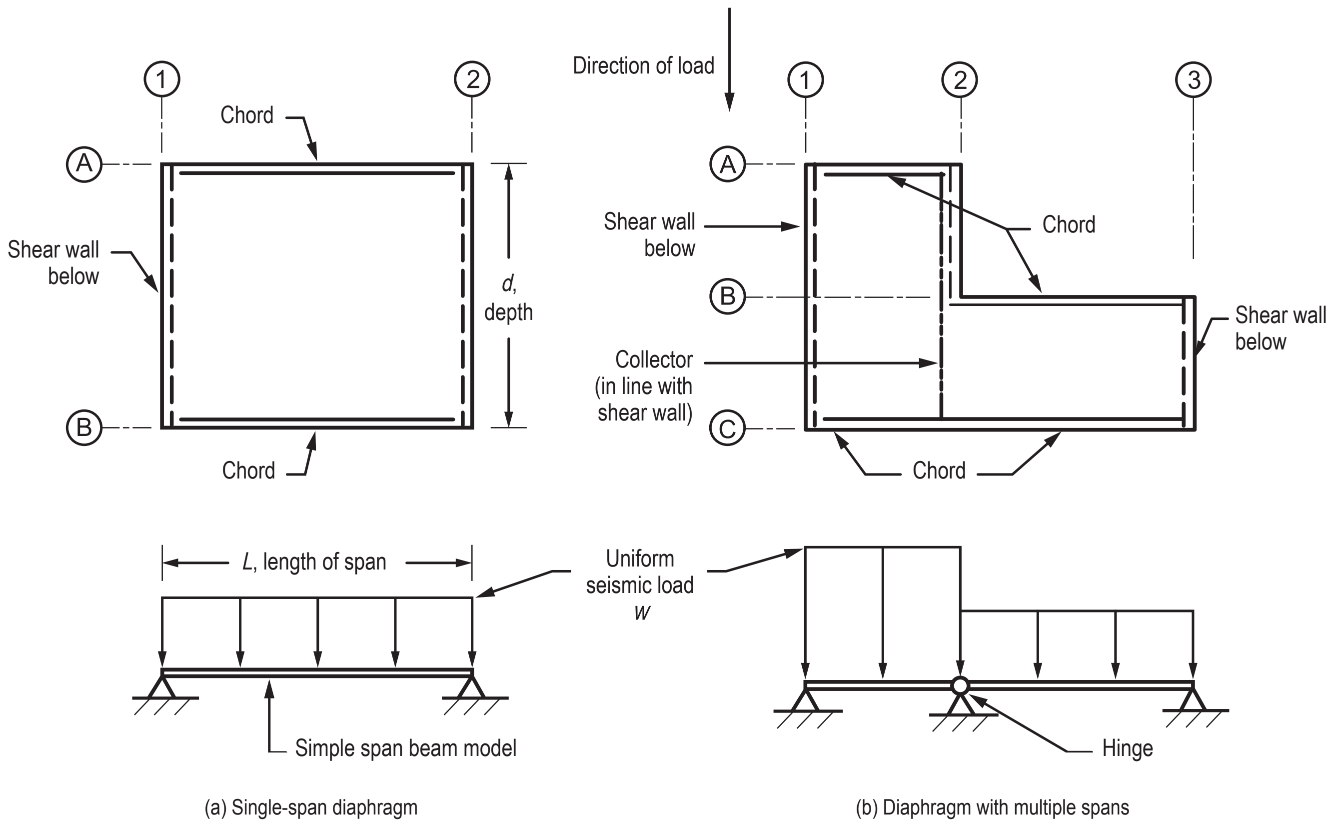
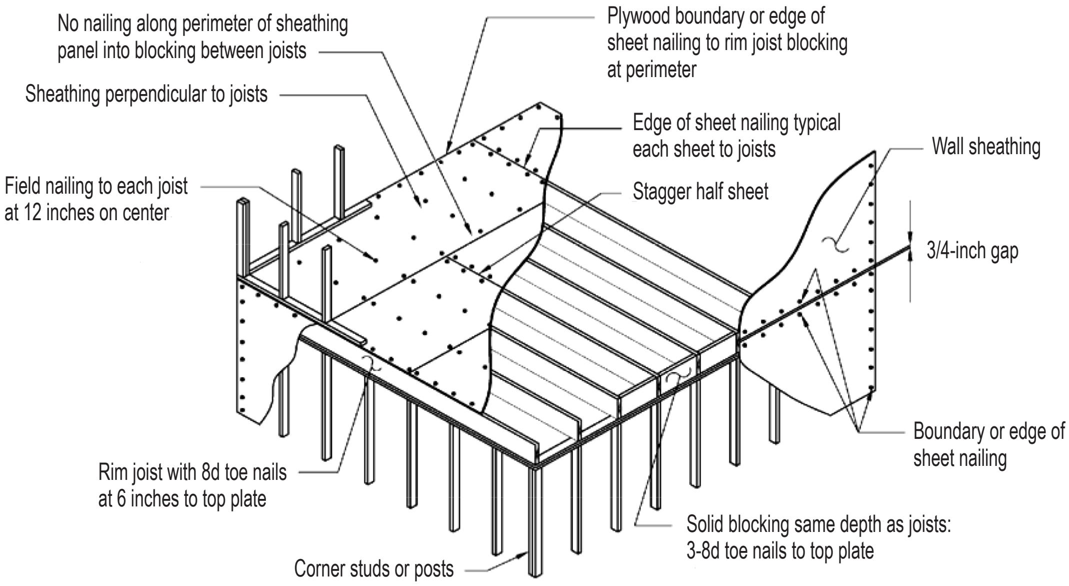
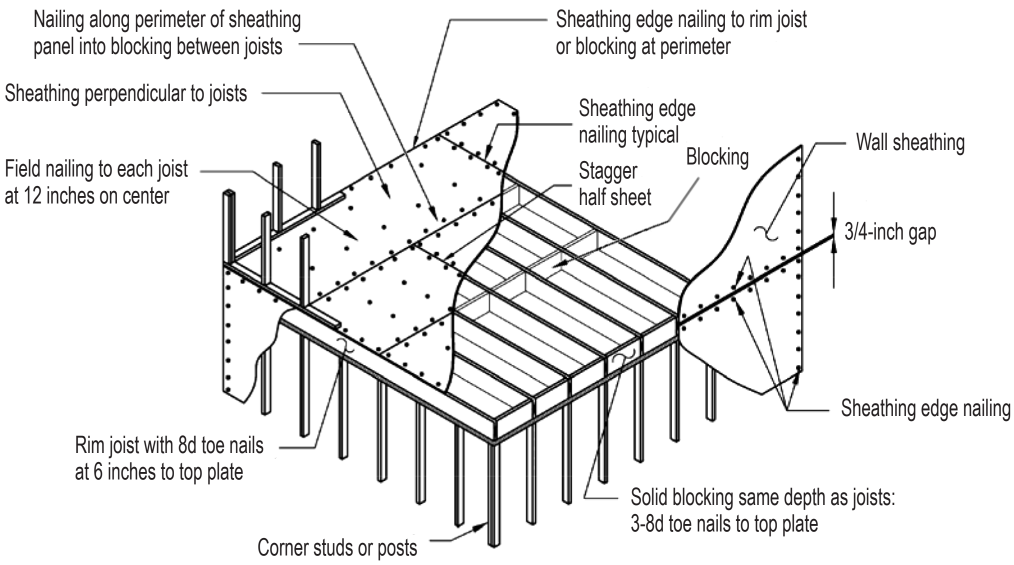


Figure 3-1. Plan view of wood light-frame diaphragms.



(a) Unblocked wood light-frame diaphragm (FEMA 2006).

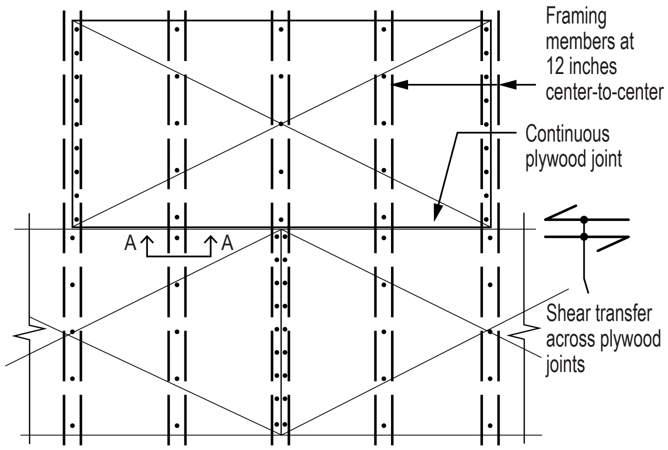


(b) Blocked wood light-frame diaphragm (FEMA 2006).

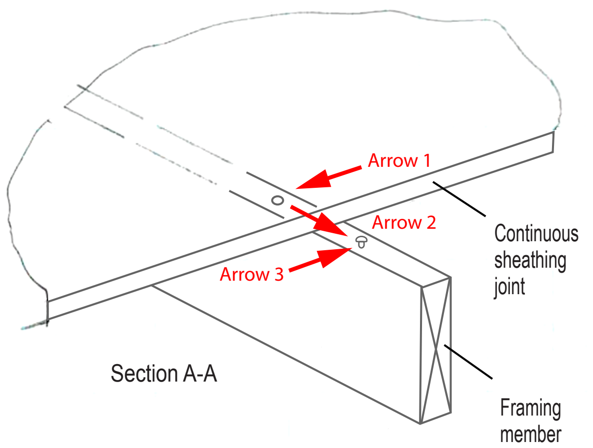
Figure 3-2. Blocked diaphragms provide greater shear capacity than unblocked diaphragms.

Figure 3-3 illustrates the load path for transfer of the shear forces from one sheathing panel to the adjacent panel in an unblocked diaphragm. At 16 or 24 inches on center, the shear force is transferred out of one sheathing panel and into the framing member at one fastener, transferred horizontally through the framing member to the other fastener, and then transferred through the fastener to the adjacent sheathing panel. As a result, the fasteners (nails or staples) attaching the sheathing to the framing are essential to good seismic performance of the diaphragm.

The combination of the sheathing thickness, fastener type and size, fastener spacing, species of framing, and thickness of framing together determine the in-plane strength and stiffness of the diaphragm. Capacities of sheathing and fastener combinations commonly used for design are provided in SDPWS Tables 4.2A to 4.2C. SDPWS Table 4.2A provides capacities for blocked diaphragms, and Table 4.2C provides capacities for unblocked diaphragms. With more closely spaced sheathing fasteners, tabulated capacities for blocked diaphragms can be significantly higher than capacities for



(a) Unblocked wood light-frame diaphragm shear transfer across a continuous sheathing joint.



(b) Close-up of Section A-A. The shear force is transferred out of one sheathing panel and into the framing member at one fastener (Arrow 1), transferred horizontally through the framing member to the other fastener (Arrow 2), and then transferred through the fastener to the adjacent sheathing panel (Arrow 3).

Figure 3-3. A single fastener transfers two feet worth of unit shear across the sheathing joint.

unblocked diaphragms. SDPWS Table 4.2B provides capacities for high-load diaphragms. These diaphragms have multiple rows of fasteners at each edge of each sheathing panel. Because the shear capacity of the diaphragm is primarily driven by the fastener capacity, high-load diaphragms have proportionally higher capacities. High-load diaphragms are most commonly used with concrete or masonry walls. The additional mass of the walls contributes to high seismic forces, requiring the additional strength and stiffness.

Included in the IBC but not in the SDPWS are values for stapled diaphragms. Stapled diaphragms have been tested by APA—the Engineered Wood Association (formerly the American Plywood Association and the Douglas Fir Plywood Association), and stapled shear walls have been tested by a number of researchers (Zacher and Gray 1985, Pardoen et al. 2003, Fonseca et al. 2002). Because the staple leg diameter is much smaller than typical nails used for diaphragm construction, the staples reduce splitting of framing members during installation, which allows the staples to be installed at a very close spacing. Staples are particularly advantageous for high-load diaphragms with close fastener spacing and for retrofit of existing diaphragms where drier lumber framing is more susceptible to splitting. Stapled diaphragm (and shear wall) tables are provided in the IBC, only, because they have not yet been reviewed by the American Wood Council (AWC) standard committee for incorporation into the SDPWS standard. Stapled diaphragms are not considered to be better than nailed diaphragms but rather are a code-accepted alternative. When using stapled diaphragms, attention to staple orientation and edge distance is critical.

A designer can develop design values for combinations of sheathing and fasteners not provided in the SDPWS standard using mechanics-based analysis, but most designers use the tabulated combinations. It is important that the designer understand that the combinations of sheathing, nail type, and nail size that are included in the tabulated values must be precisely maintained for the design values to be accurate.

Sheathing Fasteners Other Than Those Included in SDPWS or IBC

Use of diaphragm fasteners other than those included in the SDPWS and IBC tables should not occur without consideration of the resulting seismic performance of the diaphragm. Alternatives to diaphragm sheathing fasteners prescribed in SDPWS or IBC tables are recognized in product evaluation reports, such as those produced by the International Code Council Evaluation Service, based on comparative testing and analysis between the code-prescribed fastener and the alternative fastener. Performance metrics include results of lateral, withdrawal, and fastener head pull-through testing. Small-scale diaphragm simulation testing employing multiple fasteners is also used as a method to simulate influence of fastener loading perpendicular to the sheathing edge as the sheathing panel rotates.

Construction adhesives have long been used with nails in the construction of light-frame wood floor systems. This common practice is recommended by APA and others as a method to mitigate floor vibration, increase floor stiffness for gravity loading, and reduce the potential for squeaking. While use of adhesives is generally recognized as increasing shear strength and stiffness of the diaphragms, the design strength and stiffness of the diaphragm in accordance with SDPWS or IBC is based on the specified fastening alone. Both the strengthening and stiffening effect of such adhesives are not thought to be detrimental to overall diaphragm performance. Although potential for more sudden loss in strength due to a combination of adhesive and wood failure is increased where adhesives are used, such a mechanism is associated with overstrength levels beyond that provided by nailing alone.

The framing supporting the diaphragm sheathing is also a key part of the diaphragm system, providing a base to which the sheathing is fastened. As previously mentioned the capacity of a wood structural panel diaphragm is affected by the framing species. Capacities tabulated in the SDPWS and IBC are applicable where framing members are Douglas Fir-Larch or Southern Pine species. Adjustments of capacities are required for other species. In addition, special attention is required when using some engineered framing members, as size and spacing of nails should be in accordance with information published by the framing manufacturer. Because of potential for splitting in some framing member types, the framing manufacturer may have limitations on nail size and minimum spacing between nails into the engineered framing.

3.2 Diaphragm Boundary Elements

Boundary elements are made up of boundary members and their splices and connections. They can include members already provided as part of the gravity load system, or additional members may be provided specifically to perform as boundary elements. Wood boundary members are typically provided to resist compression loads. Wood or steel members (including steel flat straps) are typically provided to resist tension loads.

Chord Members

The bending moment induced into a diaphragm is assumed for purposes of design to be resisted by the perimeter framing members, referred to as diaphragm chords. **Figure 3-1** illustrates this concept. The bending moment calculated from simple statics for a simply supported beam is resisted by a force couple at the extremities of the diaphragm (tension at one edge and compression at the opposite edge). **Figure 3-1(a)** illustrates the chord locations for a simple single-span diaphragm.

In wood light-frame buildings, the wall top plates are commonly used as chord members (**Figure 3-4**). The top plates are used because they provide adequate capacity for most diaphragms, staggered joints in the top plates (typically two 2 x 4 or two 2 x 6 members) allow for lap splices of the chord, and the top plates are conveniently located in line with the shear walls, simplifying load path detailing. A load path is required between the diaphragm sheathing and the chord member. With solid sawn framing, shown in **Figure 3-4**, the blocking or rim joist members can be connected to provide this load path. Where trusses are used in place of solid sawn framing, blocking members between the trusses (similar to the solid sawn blocking) are needed to serve the function of the blocking in providing a complete load path.

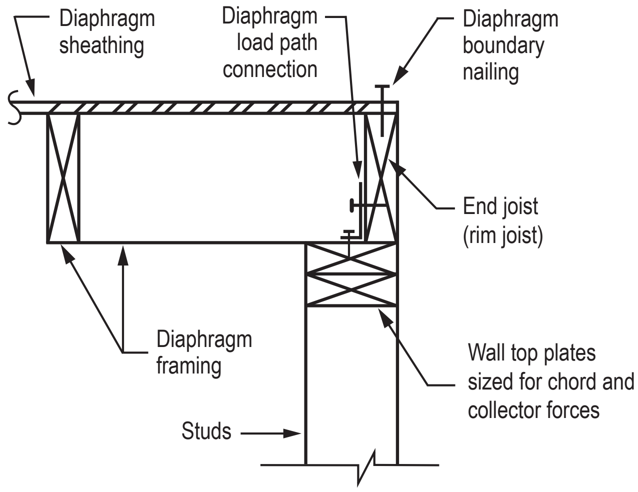


Figure 3-4. Wall top plate acting as chord boundary element. The same top plate can also act as the collector boundary element.

Where roofs have overhangs, it is possible to designate the fascia board as the chord member. However, this increases the complexity of load path detailing because the fascia board is not in line with the shear walls below. Also, required tension and compression connections are less visible at the wall top plate than at the fascia board. Some designers also include the rim joist or the bottom plate member of the walls above as part of the diaphragm chord, but this is not recommended without special detailing to provide splicing of the multiple rim joist or bottom plate members.

Where wood light-frame diaphragms are used in combination with concrete or masonry walls, it is most common for reinforcing steel (rebar) in the wall to serve as the chord member. Where this approach is used, the designer must make sure that the peak diaphragm unit shear is transferred from the diaphragm to the concrete or masonry wall, completing the load path. Where tilt-up concrete wall construction is used, continuous ledger members of wood or steel can be used as the chord member; unit shear transfer to these members must be provided, and the members must be spliced to resist chord forces.

Collector Members

As the name implies, a diaphragm collector collects the diaphragm unit shear along the diaphragm boundary (line of the diaphragm reaction) and transmits this force to the vertical elements of the SFRS at the story below. At the diaphragm perimeter, the collector is often the same member as is used for the chord under orthogonal loading. At the diaphragm interior, supplemental members are added to act as collectors. As illustrated in **Figure 3-1(b)**, the diaphragm shear is collected on Line 2 between Lines B and C and transmitted to the shear wall between Lines A and B. This collected load includes the unit shear from the reaction at Line 2 of the diaphragm that spans between Lines 1 and 2 and in addition, the unit shear from the diaphragm that spans between Lines 2 and 3. A complete load path for a collector includes the following: (1) boundary nailing of the diaphragm sheathing to the collector; (2) splices between individual members that make up the collector; and (3) connection of the collector to the vertical element framing that is capable of transmitting the collector force to the vertical element.

Where there is an opening in a shear wall, as illustrated in **Figure 3-5**, the collector must transfer the unit shear from the diaphragm above the opening to the walls on either side of the opening through tension and compression. The axial forces and the connections required to carry these design forces can easily be determined by using a shear flow diagram.

Diaphragm Openings

Special detailing is required around diaphragm openings to provide a load path for the shear forces in the diaphragm sheathing surrounding the opening. Boundary elements are required at every edge of the diaphragm opening. In addition, the boundary elements need to be continued beyond the opening to transfer boundary element forces into the overall diaphragm (**Figure 3-6**). Methods of analyzing diaphragms with openings are provided in APA Research Report 138 (APA 2000) and a design guide developed by the National Council of Structural Engineering Associations (Prasad et al. 2009).

When diaphragms are designed for load transfer around an opening, chords and collectors at the edges of the opening can accumulate significant forces. If sufficient distance is not provided to transfer the seismic loads back into the diaphragm beyond the opening, the local area around the termination of the chord or collector can be overloaded and damaged. The designer needs to consider that the load is being transferred into the sheathing, and the sheathing fasteners may already be highly loaded by the unit shear in the diaphragm in that location. When the chord or collector is oriented perpendicularly to the diaphragm framing direction, blocking and straps need to be added, with fasteners between the straps, blocking, and sheathing transferring the loads to the sheathing. When the chord or collector is oriented parallel to the framing direction, the chord or collector should be fastened to a framing member of sufficient length to avoid stress concentrations. Once the loads are in the sheathing, the added load from the chord or collector is resisted by the regular sheathing nailing at the location, and the chord or collector loads must be added to the regular unit shear. As a result, the chord or collector needs to extend well beyond the edge of the opening, such that the reserve nail capacity (difference in the design capacity of the diaphragm and the actual unit shear in that region of the diaphragm) is adequate to accommodate the additional unit shear resulting from the chord or collector. The chord or collector should be extended

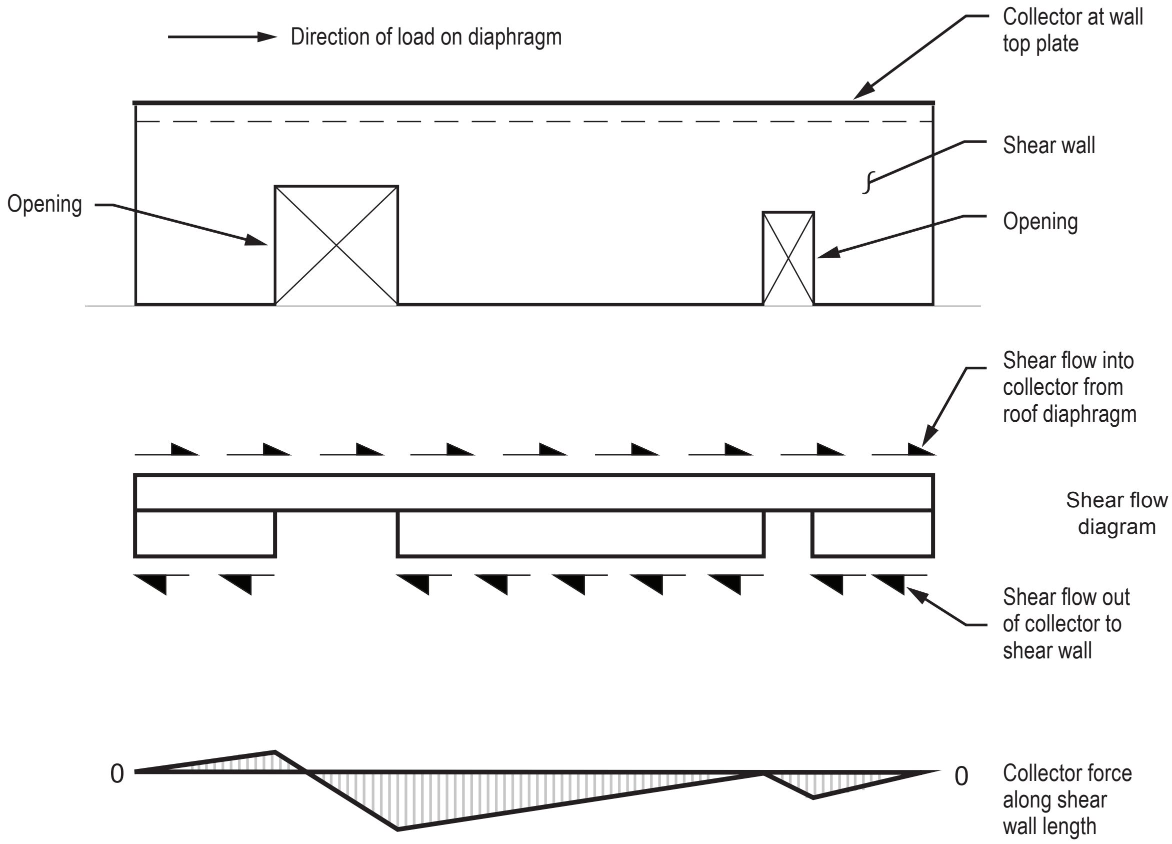


Figure 3-5. Wall top plate acting as collector boundary element. The diagram below the wall shows the variation in collector force along the collector length.

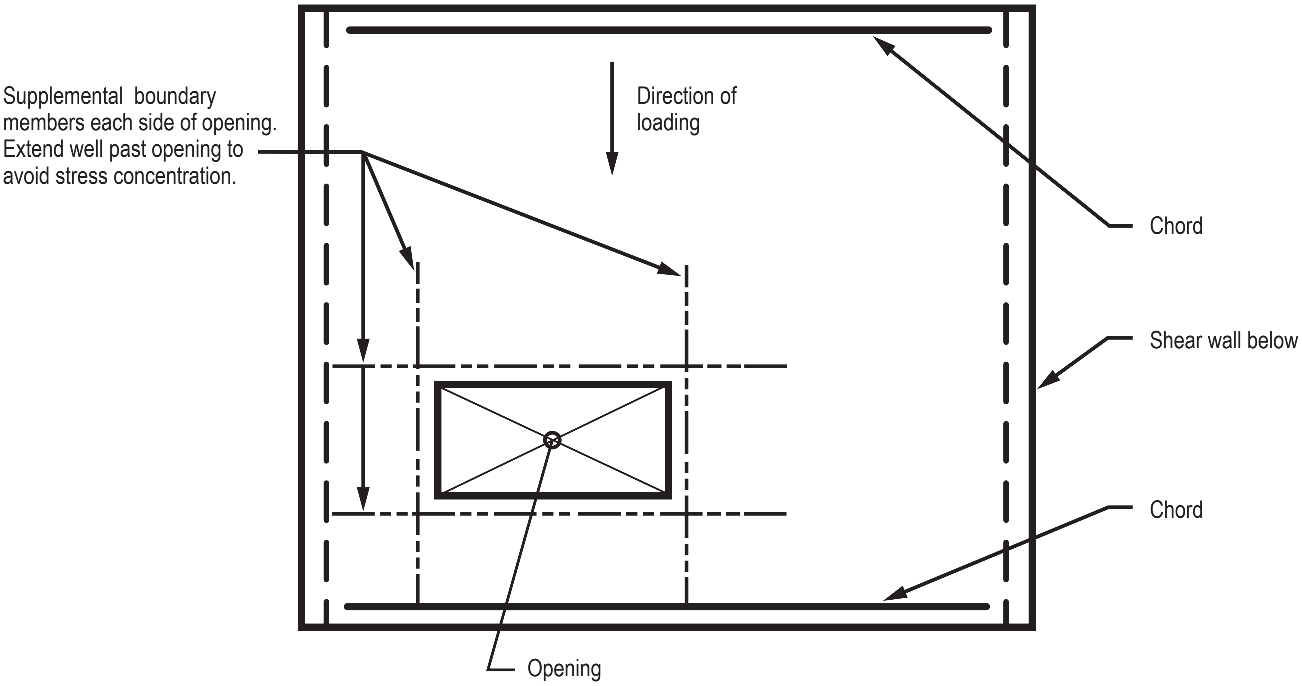


Figure 3-6. Wood light-frame diaphragm with opening. Supplemental boundary members are added around the opening.

past the opening as far as is feasible. Where the dimension of the opening is significant relative to the diaphragm span, it is recommended that extension of the chord or collector across the full diaphragm span be considered. Determining the length that the boundary member must extend past the opening is complex and should be done in accordance with one of the design methods referenced above.

3.3 Concrete and Masonry Wall Anchorage and Subdiaphragms

Where wood structural panel diaphragms provide out-of-plane support for concrete or masonry walls, ASCE 7 §12.11.2.2 imposes additional requirements related to the wall anchorage to the diaphragm. (Note that at the time of writing this Guide, an erratum to ASCE 7 modifying the title of ASCE 7 §12.11.2.2 to indicate that it is applicable only for concrete or masonry structural walls is being prepared). Continuous ties are required to be provided from the wall-to-diaphragm anchorage across the full width of the diaphragm to the far side so that the entire diaphragm width is engaged in resisting wall anchorage forces. This requirement is based on past earthquake experience in which concrete tilt-up walls separated from the wood diaphragm, in some cases resulting in local collapse of the roof. This seismic vulnerability is discussed further in Section 4.1. Because wall anchorages are often spaced as close as 4 feet on center, and because it is inefficient to provide continuous ties across the full diaphragm width at this close spacing, the subdiaphragm concept has been developed as an analytical tool. The subdiaphragm is a smaller diaphragm within the main diaphragm (**Figure 3-7**). Wall anchor forces are developed into the subdiaphragm, and continuous ties across the diaphragm are provided at each end of each subdiaphragm rather than at each wall anchor. **Figure 3-7** illustrates subdiaphragms that anchor the east and west walls for seismic loading in the east-west direction. Similar subdiaphragms would be provided along the north and south walls for loading in the north-south direction. **Figure 3-8** illustrates a typical wall anchor anchored to a sub-diaphragm roof purlin. Sheathing edge nailing is provided into the purlin as part of the load path, but the sheathing is not considered part of the connection between the purlin and the ledger. Complete failure of the wall-diaphragm connection and subsequent roof collapse has been observed in several earthquakes where the connection was made via the ledger only, without the steel tie connecting the wall to the diaphragm framing (**Figure 3-9**). The wall anchorage force of ASCE 7 §12.11.2.1 is used for design of the subdiaphragm and continuous diaphragm ties.

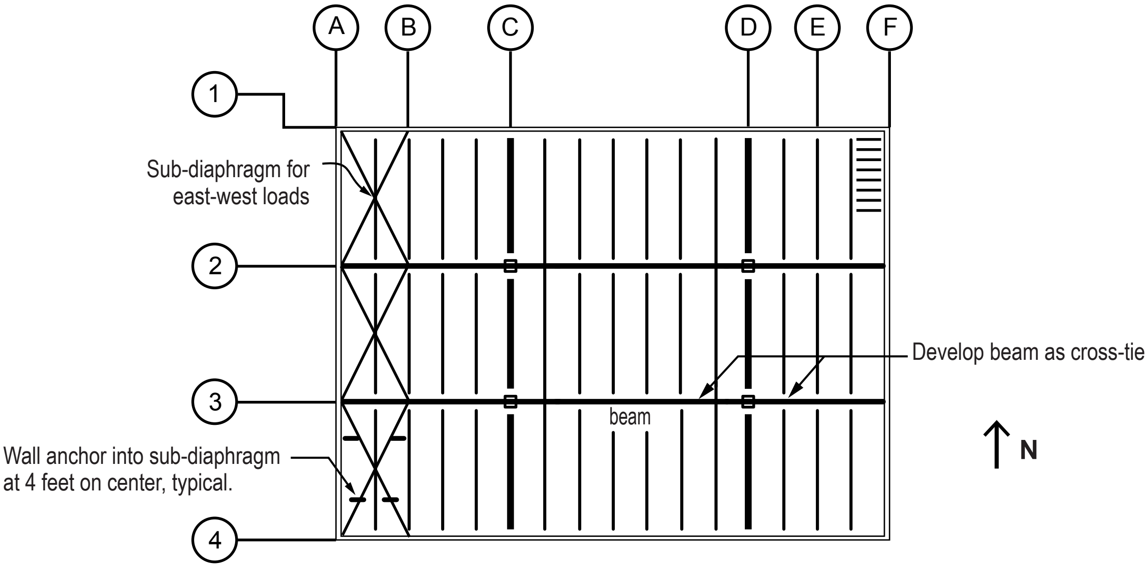


Figure 3-7. Plan of roof diaphragm with subdiaphragms for concrete or masonry wall seismic anchorage for loading in the east-west direction.

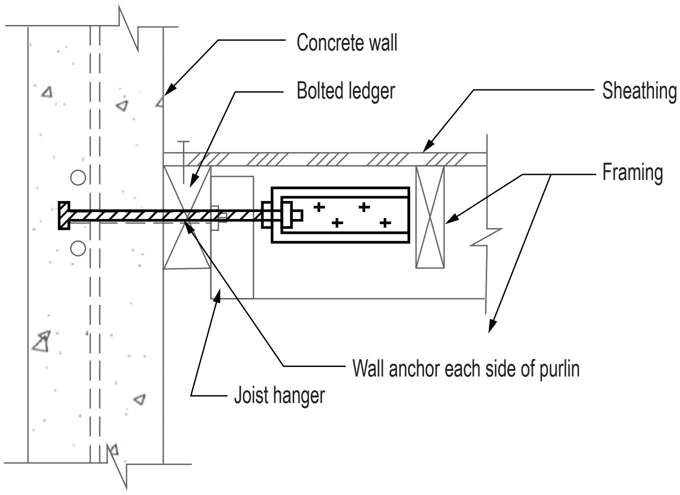


Figure 3-8. Concrete wall anchor to wood light-frame diaphragm or sub-diaphragm.

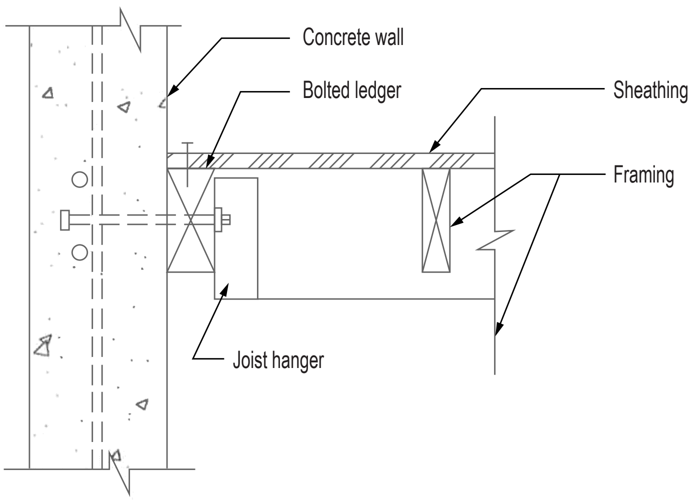


Figure 3-9. Inadequate concrete wall to wood light-frame diaphragm connection, lacking wall anchor capable of resisting direct tension.

4. Diaphragm Behavior and Design Principles

4.1 Diaphragm Design Philosophy

Intended Diaphragm Performance

A wood light-frame diaphragm behaves similarly to a shear wall sheathed with plywood or OSB in that the primary mechanism of energy dissipation in the diaphragm is intended to occur in the fasteners connecting the sheathing to supporting framing. Significant deflection can occur in a diaphragm loaded to peak capacity. The primary sources of this deflection are yielding of the fastener, fastener withdrawal, and local crushing of the wood under the fastener head and around the fastener shank. These behaviors serve as energy dissipation mechanisms, along with friction between sheathing and framing. Like plywood and OSB shear walls, wood diaphragms are found to have consistent and predictable hysteretic behavior, significant overstrength, and significant deformation capacity.

Observations of past earthquake performance and testing, are evidence that for the majority of wood light-frame buildings, less displacement demand will be imposed on diaphragms than is imposed on the shear walls. This behavior is anticipated despite current design procedures that incorporate demands and capacities for diaphragms that are the same as, or very similar to, demands and capacities for the shear walls supporting the diaphragms. The reasons for this observed behavior are not fully understood, but a combination of design overstrength and element overstrength are thought to contribute. Diaphragms in buildings with concrete or masonry shear walls might experience more inelastic behavior because of the higher seismic mass, although diaphragm performance has not been identified as a topic of concern in these buildings. The typical failure type observed in post-event investigations has been the failure of the out-of-plane connections between the concrete or masonry walls and the diaphragm, as discussed below.

Observed Earthquake Performance

Over the last five decades, post-earthquake field investigations of wood light-frame building damage have not reported diaphragm performance as an issue. Study of the damage to modern wood light-frame construction caused by earthquakes in the United States has not revealed diaphragm-initiated building damage nor inelastic behavior or degradation of diaphragms (ATC 1976, EERI 1989, 1994, 1996; LATF 1994a, 1994b, 1994c; Hamburger 1994, SEAOC 1991, and SSC 1995). In contrast, significant degradation of vertical elements of the SFRS and finish materials have been identified. While these results should not be taken as an indication that there was no diaphragm damage, diaphragm damage has not shown up as a significant item of concern, meriting modifications to design practice.

Where wood light-frame diaphragms occur in concrete tilt-up wall buildings, however, damage to the anchorage of the wall to the diaphragm has been seen in a number of earthquakes. An example is shown in **Figure 4-1**. The roof system can lose its gravity load path, which has caused localized collapse of the roof as well as wall panel collapse. A comprehensive reference on the topic of past earthquake performance of concrete tilt-up buildings is *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures* (SEAONC 2001).

Figure 4-1. Concrete tilt-up building with wall connection to wood light-frame diaphragm damaged in the 1994 Northridge earthquake. External temporary braces have been added to stabilize the tilt-up wall.

Observed Testing Performance

Full-scale building tests incorporating typical wood light-frame diaphragms have been performed around the world approximately 10 to 20 times, with much of the body of work originating in Japan over the last decade and a half. Full-scale testing of wood light-frame buildings subjected to seismic loading sufficient to cause collapse has been conducted only a few times worldwide. A series of tests were conducted at the E-Defense laboratory in Miki, Japan on a full-scale two-story town house. In 2004, a two-story Japanese conventional wood light-frame house was tested to investigate the collapse mechanism and predict the collapse margin for these types of buildings (Miyake et al. 2004, Koshihara et al. 2004). These studies were conducted to improve numerical simulation models. Good performance of diaphragms has been observed; failures, where they occur, initiate in the vertical seismic force-resisting elements. The number of tests of full-scale wood light-frame building specimens conducted in the United States has increased since approximately the year 2000. This has been motivated by the need to better understand the interaction of the subassemblies (such as wood shear walls and floor diaphragms) within full-scale buildings. A summary of full-scale building tests is provided in a 2009 report prepared by the National Association of Home Builders Research Center (NAHB 2009), which provides the objectives and selected results of each test program.

Fischer et al. (2001) tested a rectangular-plan two-story house with an integrated one-car garage; no diaphragm damage was observed in this testing. In addition, Filiatrault et al. (2002a) reported on a parametric study of the in-plane stiffness of wood diaphragms. As part of the NEESWood Project, researchers tested a significantly larger two-story three-bedroom, 1,800-square-foot townhouse with an integrated two-car garage (Christovasilis et al. 2009). In that study, no direct damage to the floor diaphragms was observed, but shear in the ceiling diaphragm during a Maximum Considered Earthquake shake table test resulted in severe damage to ceiling finish panels, as shown in **Figure 4-2**. The building had a re-entrant corner that may have contributed to this higher shear in the second floor ceiling. A 14,000-square-foot six-story wood light-frame condominium building was tested within the NEESWood Project (Pei et al. 2010, van de Lindt et al. 2010) at Japan's E-Defense facility in Miki, Japan. Although most shear walls were stacked vertically, there were several locations where diaphragm transfer forces occurred, resulting in boundary element strapping and adjusted diaphragm nailing schedules. Again, as with other tests, when designed using basic principles of mechanics in combination with current design codes, there was no observable damage to the diaphragms. The most recent large-scale test in the United States conducted at the University of California, San Diego,

Figure 4-2. Ceiling diaphragm damage in two-story wood light-frame townhouse building that might be attributed to diaphragm deformation.

was on an open-front soft-story building depicted in **Figure 4-3**. Retrofits were designed that included both vertical and horizontal shear elements. The diaphragms were retrofitted with plywood, and collectors were required in a number of locations because of offsets in the vertical shear elements on the second level compared to the ground floor (see **Figure 4-3**). Details are available in papers by Bahmani et al. (2014) and van de Lindt et al. (2014).

Figure 4-3. Soft ground story building, representing older San Francisco apartment building conditions, tested on the University of California, San Diego shake table.

Dolan et. al. (2003) conducted cyclic diaphragm component testing to determine the stiffness of diaphragms in the small-deformation range in order to provide load-deflection relationships for use by designers. The test program included diaphragms with and without blocking, with and without chords, with and without adhesive, and with several different opening locations in the diaphragms. This study found that the strength increased by 15 percent and stiffness by 35 percent when blocking was added. The SDPWS provides for a 12 percent increase in design strength and a 61 percent increase in stiffness for the same configurations that Dolan tested. The large difference in stiffness is because of the use of a different definition for the variable. Additional diaphragm-related testing includes Ficcadenti et al. (2003), who investigated load path connections between shear walls and diaphragms.

Much of the component testing that has been conducted on wood diaphragms by APA—The Engineered Wood Association. For wood structural panel diaphragms, the primary available resources for test data are four APA test reports (APA 1966, APA 2000, DFPA 1954, DFPA 1963). The tests included diaphragm spans (loaded as simple-span beams) ranging from 24 to 48 feet, aspect ratios ranging between 1 and 3.3, and diaphragm construction covering a range of construction types including blocked and unblocked construction, and regular and high-load diaphragms. The loading was applied with a series of point loads at varying spacing, representing a loading condition reasonably close to uniform. Tests were conducted using a monotonic displacement-based protocol, sometimes with limited load cycling. Shear wall loading protocol studies (Gatto and Uang 2002) support the conclusion that the monotonic load-deflection behavior is reasonably representative of the cyclic load-deflection envelope, suggesting that it is appropriate to use monotonic load-deflection behavior in the estimation of diaphragm load deformation response for seismic design. This diaphragm testing served as the basis for diaphragm capacities provided in building codes and standards, as well as derivation of deflection equations. Full descriptions of the behavior of tested diaphragms are available in the referenced APA reports.

Proposed Modifications to Diaphragm Design in the 2015 NEHRP Provisions

The forthcoming 2015 edition of the *NEHRP Provisions* (FEMA 2015) recommends modifications to the diaphragm design provisions of ASCE 7. During development of these modifications, the ductility, drift capacity, and overstrength of diaphragms of varying construction types were considered in detail. For the wood structural panel diaphragms that were studied, ductility ratios, defined as the displacement at peak recorded capacity divided by the displacement at load and resistance factor design (LRFD) strength, exhibited a wide range of values, with a ratio of approximately 5 for many common types of diaphragm construction. Among evaluated diaphragms, the ratio of peak recorded strength to LRFD design strength ranged from 1.7 to 4, and the ratio of peak recorded strength to ASD ranged from 2.4 to 5.7. Collectively, the wood structural panel diaphragms demonstrated significant overstrength, ductility, and deformation capacity. A summary of the test data will be provided in a forthcoming 2015 *NEHRP Provisions* resource paper.

Retrofit

Although the design of diaphragms is typically based on demand calculated using ASCE 7 and capacity-based on SDPWS and NDS, the design of a seismic retrofit for an existing wood light-frame building may present less common challenges for an engineer. Specifically, it may not be possible to stack vertical shear elements due to architectural constraints in the building. One example of such a case is the seismic retrofit of a soft-story wood light-frame building (**Figure 4-3**). The vertical shear elements in the ground floor (the soft and weak story) are often not aligned with vertical elements above because of functional requirements at the ground story. As a result, diaphragm stiffening or strengthening may be required to ensure that diaphragm seismic forces can be transferred to the ground story vertical shear elements. Plywood or OSB sheathing applied overhead to the underside of floor framing is a common diaphragm strengthening method. An example of a plywood sheathing retrofit on the ground floor ceiling of a soft-story wood frame building is shown in **Figure 4-4**, with steel strapping provided at the diaphragm collector.

Figure 4-4. Detail of first floor diaphragm from Figure 4-3 test building, strengthened with plywood diaphragm sheathing, with collector strap applied over the face of the plywood sheathing.

4.2 Diaphragm Classification

Wood light-frame diaphragms are classified as flexible, rigid, or semi-rigid in order to model distribution of seismic forces to the vertical elements of the SFRS and in order to determine forces for diaphragm design. From a theoretical standpoint, no diaphragm is either completely flexible or completely rigid; rather all diaphragms are semi-rigid. For purposes of design, however, it is necessary to classify diaphragms into one of these three types.

ASCE 7 §12.3.1 provides rules by which diaphragms are permitted to be classified as flexible through either prescriptive or calculated criteria:

> ***Prescriptive***: Wood structural panel diaphragms are permitted to be prescriptively idealized as flexible for any of the following:
>
> - Structures where vertical elements are steel braced frames or steel and concrete composite braced frames, or are concrete, masonry, steel, or composite shear walls
> - One- and two-family dwellings
> - In light-frame construction, wood structural panel diaphragms that are untopped or have up to 1 1/2 inch nonstructural topping and meet code-required drift limits at each line of vertical elements of the SFRS
>
> ***Calculated***: Diaphragms are permitted to be classified as flexible when the calculated mid-span deflection of the diaphragm under seismic loading is greater than twice the calculated average story drift of supporting shear walls in the story below. Note that wood structural panel diaphragms often cannot meet this criterion.

Similarly, ASCE 7 §12.3.1 provides criteria by which diaphragms are permitted to be prescriptively idealized as rigid. However, those criteria apply only to composite steel deck and concrete diaphragms. Diaphragms not classified as flexible or rigid are required to be classified and analyzed as semi-rigid, with the relative stiffness of both the diaphragm and supporting walls explicitly considered.

The third bullet item above for diaphragms prescriptively idealized as flexible is new in the 2010 edition of ASCE 7. It can be interpreted to permit use of flexible diaphragm modeling for almost every new wood diaphragm in a light-frame structure. This is consistent with historical modeling assumptions. Designers have commonly assumed flexible diaphragm behavior for almost all wood diaphragms, with a distribution of loads using tributary areas and simple-span beam models. The modeling assumption of flexible diaphragm behavior is the simplest to use because seismic force distributions and diaphragm seismic forces do not need to consider the stiffnesses of either the vertical elements of the SFRS or of the diaphragm. As a result, the variability that occurs in these stiffnesses does not impact the analysis results.

When diaphragms are classified as rigid, the flexibility of each vertical element of the SFRS is included in the analysis model. This results in higher seismic forces being attracted to more rigid vertical elements. Where rigid diaphragm analysis is performed, both actual and accidental torsion are required to be included in the analysis. All of the vertical elements of the SFRS are included in the analysis model, including the elements that are perpendicular to the loading direction because all vertical elements contribute to and resist the torsional response of the diaphragm.

Semi-rigid diaphragm analysis requires that the flexibility of the diaphragm is considered in analysis in addition to the flexibility of the vertical elements of the SFRS. Both actual torsion calculated from an eccentricity and accidental torsion prescribed by code are again required to be included in the analysis. This modeling approach will reflect behavior somewhere between that seen for flexible diaphragms and rigid diaphragms. Because of the complexity of this analysis method and lack of available analysis tools, this modeling approach is seldom used in design.

The common assumption of flexible diaphragm modeling received considerable discussion following poor performance of soft-story tuck-under parking buildings in the 1994 Northridge, California earthquake (SEAOC 1999). Since that time, appropriate assumptions for the distribution of forces have remained an item of ongoing discussion within the structural engineering profession. The 2005 edition of ASCE 7 provides more limited conditions under which diaphragms could be idealized as flexible, resulting in many diaphragms being classified as semi-rigid. This was broadened somewhat by an exception in the 2006 and 2009 editions of the IBC, which permitted flexible diaphragm modeling for buildings braced entirely by wood structural panel shear walls, but many light-frame structures that incorporated moment frames or braced frames were still required to be classified as having semi-rigid diaphragms. In response, some engineers performed analysis using both flexible and rigid diaphragm models and they designed the diaphragm and vertical elements for the most critical demand from both models (referred to as an envelope method). The envelope method is a conservative approach but it results in less efficient designs.

The language incorporated in the 2010 edition of ASCE 7 has clarified its minimum requirements. However, the American Wood Council Wood Standards Design Committee has proposed further restrictions to the ASCE 7-10 provisions for diaphragm classification for the 2015 edition of SDPWS. The following sidebars provide some details of the ongoing discussion regarding diaphragm classification, as well as proposed SDPWS changes.

Diaphragm Classification and Modeling for Purposes of Understanding Structure Behavior

While the classification of diaphragms is clarified by ASCE 7 for purposes of minimum code requirements, for purposes of understanding building behavior and performance the question of appropriate modeling remains open. The modeling assumptions for rigid and semi-rigid behavior are dependent on the load-deflection behavior of the diaphragm and shear walls. There are many influences on this load-deflection behavior, many of which are difficult to quantify for analysis. Among these:

- The load-deflection behavior and resulting response of light-frame structures is significantly affected by floor, ceiling, and wall finish materials, partitions, and other items not typically included in structural models.
- The load-deflection behavior of both structure and finishes is highly nonlinear.
- The load-deflection behavior of finishes shows large variability in available test results.

As a result there is no single correct answer regarding the distribution of forces, but rather varying distributions, because the range of behaviors is inherently highly variable. This makes the selection of semi-rigid or rigid diaphragm analysis a matter of engineering judgment. Current design practice tends to ignore influences of finish materials as well as geometry: the bare wood structure comprises the model. To date there has been no indication that this approach results in unacceptable seismic behavior. On the other hand, it must be recognized that this approach does not provide useful predictions of anticipated seismic performance, which would require consideration of the nonstructural influences just discussed.

For practicing engineers, the CUREE W-30 report (Cobeen et al. 2004) made the following two interim recommendations for classification of diaphragms for purposes of code design of wood light-frame structures:

- Based on available data it appears that better building performance results when seismic forces are resisted locally rather than redistributed to other portions of the structure. For this reason, tributary area analysis is recommended for the great majority of buildings. Where tributary analysis is used, code drift limits should be applied at each shear wall line, rather than to the story as a whole.
- For buildings where code drift limits cannot be met at each wall line and for buildings where the distribution of shear walls suggests a significant torsional irregularity, analysis using rigid diaphragms will be necessary, and special attention should be given to the loading condition of the perpendicular walls. The designer should consider superimposing the torsional loading with the in-plane loading for the perpendicular walls. If torsion is to be resisted, the diaphragm needs to be designed to be rigid, with extra attention given to the sheathing-to-framing connections and boundary elements.

The 1999 SEAOC Blue Book (SEAOC 1999) makes similar design recommendations, and design provisions related to these issues are proposed for inclusion in the 2015 SDPWS. The CUREE W-30 report provides the following discussion of additional study needed: “In order to move beyond interim recommendations and rationally evaluate the appropriate threshold for rigid diaphragm distribution, the next needed step is an analytical study evaluating a range of shear wall and diaphragm stiffness and varying configurations reflecting actual buildings. Because of the wide range of shear wall and diaphragm behavior, the study should focus on determining acceptable building behavior rather than simply developing methods to more accurately model behavior. Those designers interested in understanding and predicting building behavior will need to consider these issues.

Proposed 2015 SDPWS Treatment of Torsional Irregularity and Cantilevered Diaphragms

The 2015 edition of the SDPWS standard prohibits use of flexible diaphragm (tributary area) analysis for diaphragms that cantilever more than 6 feet and for diaphragms in buildings that are torsionally irregular according to ASCE 7. This limit is imposed because more attention must be paid to analysis and distribution of seismic forces in these structures. In addition, where diaphragms cantilever more than 6 feet, compliance with ASCE 7 drift limits at all diaphragm edges is required regardless of the presence of a torsional irregularity, in recognition of the likely torsional response of structures with cantilevered diaphragms. Finally, diaphragms are now permitted to cantilever up to 35 feet, provided that: (1) the diaphragm is modeled as rigid or semi-rigid; (2) drift at edges complies with ASCE 7 allowable story drift; (3) forces include those from torsion and accidental torsion in accordance with ASCE 7; and (4) the diaphragm aspect ratio does not exceed 1.5:1 for wood structural panel diaphragms. These changes were made in response to additional scrutiny of the design of cantilevered diaphragm structures in recent years because of increased use, as well as to address changes in ASCE 7 with respect to diaphragm flexibility assumptions, torsional force provisions, and identification of torsional irregularities. A limited analytical study funded in part by a SEAOC research grant is currently underway to evaluate buildings seismically braced in one direction using only central corridor walls, with diaphragms cantilevered to both sides. The objective of the study is better understanding of the anticipated seismic performance of these buildings relative to similar buildings that provide seismic bracing in the exterior walls in addition to the central corridors.

4.3 *Dynamic Response of Structures and Diaphragms*

General Dynamic Response

Dynamic response acceleration of a single-degree-of-freedom oscillator subjected to earthquake ground motion varies with time, and that the peak response will be a function of the period of vibration (Chopra 2005). The smoothed design response spectrum of ASCE 7 (**Figure 4-5**) represents this period dependency.

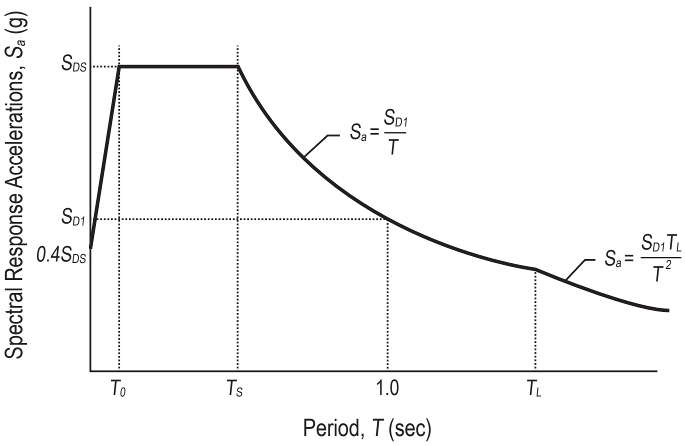


Figure 4-5. ASCE 7 design response spectrum showing spectral response acceleration as a function of vibration period.

In **Figure 4-5**, the term S_{DS} represents the design spectral acceleration for short-period structures. The peak ground acceleration, which is the spectral acceleration at a period of zero, has a value of $0.4S_{DS}$. The ratio of the peak response acceleration to the peak ground acceleration is called the response acceleration magnification. Its value for short period structures is 2.5 in this design spectrum. Many low-rise wood light-frame buildings fall into this short-period range.

The behavior of multi-story buildings is similar. Studies of building responses (Shakal et al. 1995, Rodriguez et al. 2007) show that the response acceleration magnification also is approximately 2.5 for buildings responding essentially elastically. For buildings responding inelastically, a lower response acceleration magnification is generally obtained. In general, wood light-frame buildings respond inelastically at even small displacements, and thus it is not uncommon for a wood light-frame building to have an averaged response acceleration magnification below 2.5.

One important observation regarding multi-story buildings is that the different floors will trace out different acceleration histories. Each floor should be designed to resist an inertial force proportional to the peak response acceleration of that floor. This is the concept behind the ASCE 7 F_{px} diaphragm design forces discussed in Section 5.1 of this Guide. It would be overly conservative to design the vertical elements of the SFRS for the sum of the individual peaks, because each floor reaches its peak response at a different time during the dynamic response. ASCE 7 F_x vertical element design forces are proportional to the maximum forces the vertical elements might experience. Thus, two different sets of design forces commonly are specified for design, one for the design of vertical elements of the SFRS and another for diaphragm inertial forces (**Figure 4-6**).

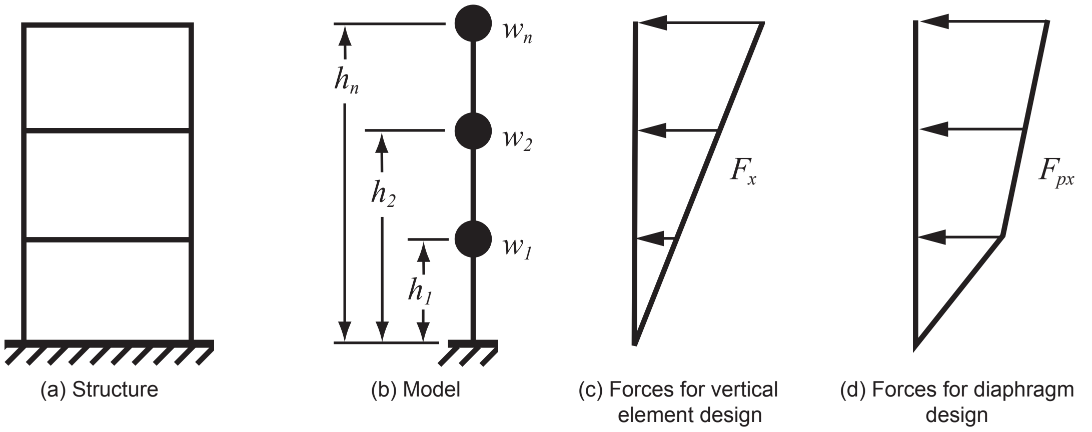


Figure 4-6. Vertical distribution of seismic forces based on ASCE 7 equivalent static force design procedures.

In addition to resisting inertial forces (tributary mass times floor acceleration), diaphragms also must be able to transfer forces between different vertical elements of the SFRS. Transfer forces were discussed in Section 2 of the Guide. Buildings that have transfer forces of this type must be designed for combined inertial and transfer forces as discussed in Section 5.1.

Response of Low-Rise Wood Light-Frame Buildings

Low-rise wood light-frame buildings have been observed to deform primarily in shear with some potential to slide and rock. Overturning forces are present but are not as dominant in building response as taller light-frame wood buildings. Low-rise wood light-frame buildings typically have periods of between 0.1 and 0.3 seconds and exhibit total effective damping (equivalent viscous plus hysteretic) of between 15 percent and 20 percent (Filiatrault et al. 2002b). Their response to an earthquake is generally a first mode response, but some contribution of the second mode has been observed in three-story wood light-frame buildings. The vertical shear elements are most often wood shear walls. Horizontal diaphragms are typically of wood light-frame construction with nailed wood structural panels. In typical low-rise construction, in which there are closely spaced supporting walls, this results in short-to-medium diaphragm spans. These diaphragms are often observed to behave with high stiffness without exhibiting distress associated with significant inelastic response. Buildings with long-span wood diaphragms, particularly in combination with concrete or masonry walls, can undergo significant diaphragm deflection under seismic loading. These buildings have longer fundamental periods associated with the diaphragm flexibility; this lengthening of the building period is not currently considered in diaphragm design.

Response of Mid-Rise Wood Light-Frame Buildings

Although low-rise wood light-frame buildings deform primarily in shear, the same cannot be said for mid-rise wood light-frame buildings. Buildings three or more stories tall have significant shear wall overturning forces that result in uplift of the shear wall stacks. Shear wall overturning restraint to control flexural response is important to seismic performance of mid-rise buildings. If the shear wall stacks are not continuous over the full height of the building, then large transfer forces can develop at vertical discontinuities. Diaphragm seismic design forces based on ASCE 7 vertical distribution of base shear are explained in Section 5.1. **Figure 4-7** shows a typical wall stack in a mid-rise wood light-frame building. In mid-rise wood light-frame construction, diaphragm spans tend to be short to medium, and they do not have a significant effect on the building dynamic response. With the short- to medium-span diaphragms, boundary element forces tend to be moderate. The designer must take into account the transfer of collector forces into shear wall stacks, as well as the continuity of load path over the height of the shear wall stacks.

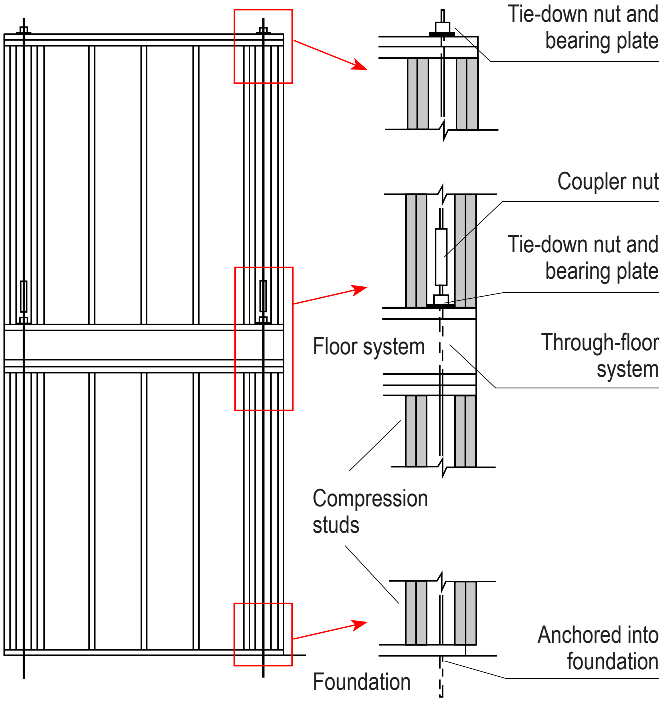


Figure 4-7. Elevation view of two-stories of a typical mid-rise wood light-frame shear wall stack.

An issue concerning mid-rise wood light-frame building diaphragms that is less prevalent in low-rise wood light-frame buildings is the shear wall boundary element compressive forces caused by overturning moments, forces that are imposed vertically on the diaphragm at the base of each shear wall as the shear wall stack deforms under horizontal loading. This requires attention to detailing: the condition is often handled by building up the rim joist with one or more additional joists or blocking. A similar shear wall boundary element uplift force occurs at each story of the wall stack. A four-story stack with continuous rod tie-downs that uplift 1/4 inch per story, could have a full inch of cumulative uplift through rod elongation (stretching) before the wall fully engages its uplift restraint.

Long-Span Flexible Diaphragm Studies underway

A Building Seismic Safety Council (BSSC) study of rigid-wall, flexible-diaphragm buildings is underway. The objective of the study is to recommend new procedures for design of this building type. Key to the behavior of these buildings is that the seismic response is dominated by the dynamic response of the flexible diaphragm rather than the more rigid shear walls. Recommendations from this study will be available in a white paper in the 2015 NEHRP Provisions. This research has also evaluated seismic forces imposed by heavy masonry and concrete walls, and it may lead to future revisions to wall anchorage forces (Koliou et al. 2014). A comprehensive report will also be available from BSSC following completion of the study.

5. Diaphragm Seismic Design Forces

This section addresses diaphragm seismic design forces with a focus on ASCE 7 requirements. Section 6 of this Guide continues with diaphragm modeling and analysis guidance. The forces developed in a diaphragm under seismic loading are dependent on the overall response of the building to earthquake ground motions. The building period; the type, stiffness, and placement of the vertical elements of the SFRS; discontinuities; and torsional response all play a role in diaphragm response and resulting diaphragm seismic forces. ASCE 7 analysis procedures take these aspects into consideration. In most circumstances the diaphragm cannot be designed until there is a preliminary analysis of the overall SFRS.

To calculate diaphragm seismic design forces, the building base shear, V, and the building story forces, F_x, must first be determined. For buildings with wood light-frame diaphragms, it is most common for the base shear and story forces to be determined using the ASCE 7 §12.8 Equivalent Lateral Force analysis procedure. However, the base shear and story forces may also be determined in accordance with the Modal Response Spectrum analysis procedure of ASCE 7 §12.9 or the Seismic Response History procedures of Chapter 16 ASCE 7. Regardless of what method is used for determining base shear and story forces, the seismic forces for design of diaphragms, including their chords and collectors, are specified in ASCE 7 §12.10.

Two types of seismic forces need to be considered in diaphragm design: inertial forces and transfer forces. Diaphragm inertial forces, F_{px}, introduced in Section 4.3, are caused by the effect of diaphragm accelerations on the weight (mass) of the diaphragm and other tributary weight that the diaphragm supports under lateral loading (e.g., walls, partitions, storage loads). The inertial forces used for diaphragm design, F_{px}, are different and distinct from the seismic story forces used for design of the vertical elements, F_x, as discussed in Sections 2 and 4.3. The F_{px} forces are determined in accordance with ASCE Equation 12.10-1:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

where:

F_{px} = the diaphragm (inertial) design force

F_i = the (vertical element) design force, F_x, applied at level i

w_i = the weight tributary to level i

w_{px} = the weight tributary to the diaphragm at level x

Figure 4-6 illustrates the F_x forces determined from ASCE 7 §12.8 and F_{px} forces determined from ASCE 7 Equation 12.10-1 for a building with uniform story weight and height. F_{px} forces are always equal to or greater than F_x forces. In addition to Equation 12.10-1, upper and lower bounds on seismic forces are provided. I_e is the seismic importance factor defined in ASCE 7 Table 1.5-2. F_{px} is not permitted to be taken as less than $0.2S_{DS}I_e w_{px}$ and need not be taken as greater than $0.4S_{DS}I_e w_{px}$. For buildings with an R factor greater than 5, the lower bound F_{px} forces will control at all but the highest stories; this is true for most wood light-frame buildings.

Diaphragm transfer forces occur under two common conditions: (1) shear walls or other vertical elements at upper stories are discontinued (or offset out-of-plane) at lower stories; or (2) the stiffness of shear walls changes significantly between the stories above and below a given diaphragm. In either case, forces from the vertical elements above will be transferred into the diaphragm and then back out to the vertical elements below. When this occurs, the diaphragm acts as an extension of the vertical elements. This transfer of forces is typically identified in the building analysis, and the transfer force contribution to diaphragm seismic forces is taken from the analysis results using the F_x vertical element forces. Design for transfer forces is described in ASCE 7 §12.10.1.1. Although the redundancy factor, ρ (included in ASCE 7 §12.3.4 to penalize structures with low redundancy in vertical elements of the SFRS) is permitted to be taken as 1.0 for diaphragm inertial forces, the calculated redundancy factor is used for diaphragm transfer forces. Where buildings have plan or vertical structural irregularities, the additional requirements of ASCE 7 §12.3.3.4 may apply, as is discussed in Section 5.3.

Minor Transfer Forces

For transfer force condition (2) as described above, the design of the diaphragm for transfer forces is intended to be triggered where a significant redistribution of forces occurs because of a significant change in vertical element stiffness from above to below the diaphragm. A minor amount of redistribution will often be seen when any rigid or semi-rigid diaphragm analysis is performed, and the diaphragm design is not intended to capture this minor force redistribution. The designer must determine when redistribution is minor enough to be neglected in diaphragm design.

The seismic forces used in design of the diaphragm and its boundary elements need to be consistent with the diaphragm classification used for distribution of seismic forces to the vertical elements of the SFRS and with the modeling and analysis methods. Diaphragm classification is introduced in Section 4.2. Modeling and analysis are discussed in Section 6. The diaphragm needs to be designed to deliver the seismic forces to the vertical elements. Where flexible diaphragm design is used, the resulting shear distribution consistent with simple-span beams may be used. Where rigid or semi-rigid diaphragm classification is assumed, the diaphragm must be designed based on the shear distribution resulting from that assumption. This generally means higher unit shears and boundary element forces in the vicinity of stiffer vertical elements that attract higher seismic forces.

5.1 Sheathing Design Forces

Where diaphragms are subject only to inertial forces, F_{px} forces are used for sheathing design. Where transfer forces occur, sheathing design forces include the sum of transfer forces based on F_x and inertial forces. Sheathing forces are generally shear forces, for which the sheathing, its fastening to framing members, and its fastening to boundary elements, need to be designed. The capacity of sheathing, including consideration of the fastener type, size, and spacing, the framing member species, and the panel layout are commonly determined in accordance with the provisions of the SDPWS. Capacities can be determined at ASD or LRFD levels and compared to ASD or LRFD seismic demands, respectively. The seismic design forces for attachment of the sheathing to boundary elements are at the same level as the force for the sheathing design, even where the boundary elements themselves are required to be designed for a higher force level. As an example, the sheathing boundary nailing to a collector uses the same force level as used for the sheathing design, even when the collector is required to be designed for an overstrength force level.

5.2 Boundary Element Design Forces

Boundary elements (boundary members and their connections) are provided to carry axial tension and compression forces at all sheathing edges and internally as required to transfer loads into shear walls. They also provide continuity for diaphragm openings and offsets. To provide adequate seismic performance, the boundary elements and load path connections for diaphragms must be capable of developing the diaphragm peak demand. Post-earthquake observations and testing to date suggest that current design and detailing practice accomplishes adequate performance without specific capacity design requirements. A number of factors are thought to contribute to this performance, including the high inherent overstrength present in tension and compression members and in connections designed in accordance with the NDS. In instances where development of the diaphragm capacity is critical to seismic performance, however, the designer may want to consider use of a capacity design methodology.

Chord Design Forces

Diaphragm chords, including members and their connections, are required by ASCE 7 to be designed for the same force level as the diaphragm sheathing. Where diaphragms are subject only to inertial forces, F_{px} forces are used for chord design. Where transfer forces occur, chord member forces, including the sum of inertial and transfer forces, are used in design. Chord forces are generally tension and compression forces. Capacities for chords and their connections are commonly determined in accordance with the provisions of the NDS. Capacities can be determined at ASD or LRFD levels and compared to ASD or LRFD seismic demands, respectively.

Occasionally, chord members are offset from the diaphragm sheathing such that the sheathing is not in direct contact with the chord member. In a small number of cases the resulting eccentricity creates moments in the chord member, which makes it neccessary to design the chord for combined axial and flexural forces. The eccentricities inherent in common details, such as those shown in **Figure 3-4** (the eccentricity is the height of blocking or rim joist) are not thought to be of concern and not explicitly considered in design. The designer is encouraged to evaluate eccentricities and identify configurations where additional consideration of member or connection demand is required due to eccentricities.

Boundary Member Capacity

Wood member strength is highly variable because of naturally occurring growth characteristics. As a result, within a species and grade, member allowable stresses or capacities are commonly assigned so that 95 percent of the members will be stronger, in addition to the inclusion of safety factors. This 95 percent rule is the basis of both ASD and LRFD adjusted capacities. As a result, the designer cannot expect that the capacity of the member will serve to limit the seismic force that can be developed in a wood system; any limiting strength would have to come from the fasteners or connections rather than from the member itself. This aspect of wood capacity design is significantly different than in steel design, where a predictable upper bound strength of members can be defined and considered to limit the seismic force developed in the system.

> **Distributed Chord Systems**
>
> Some engineers have recently proposed that rather than considering only the contribution of a single chord member at the diaphragm edge, multiple parallel framing members distributed over a larger dimension contribute to the chord capacity. This has been proposed based on design theory only and not studied with testing to date. Caution should be exercised when using this approach, as this will: (1) reduce the effective depth of the diaphragm, increasing chord forces and (2) increase the unit shear in the diaphragm in the central portion. In addition, appropriate tension and compression detailing of each of the additional chord members is required, as well as sheathing boundary nailing to all members acting as chords.

Collector Design Forces

Seismic design forces for collectors are addressed in ASCE 7 §12.10.2. For most diaphragm systems in SDC C to F, collector elements (collector members and collector connections, including connections to vertical elements of the SFRS) are required to be designed for seismic forces amplified by the applicable overstrength factor, Ω_0, in accordance with ASCE 7 §12.4.3. The resulting overstrength forces are required to be used in ASD or LRFD load combinations defined in ASCE 7 §12.4.3.2. The requirement for force amplification by the overstrength factor was first introduced in the 1997 Uniform Building Code (ICBO 1997) as a measure to help ensure that the collector would not serve as a weak link in the SFRS. It is not required in SDC B.

Systems braced only by wood light-frame walls, however, are exempt from overstrength forces and need only be designed for the force level used for design of the sheathing and chords. This exemption was included in the 1997 UBC at the time that the overstrength factor was introduced, because wood light-frame bracing systems tend to be more distributed and have lower forces than systems for other construction types, and because of observed good performance of the collector systems in past earthquakes. The exemption permitted continuation of past collector design practice for buildings braced by wood light-frame walls. Since the publication of the 1997 UBC, no additional information has come to light suggesting that current practice is inadequate, so the exception still exists in ASCE 7 §12.10.2. However, a designer might consider use of an overstrength factor or capacity design approach where a collector is critical to the seismic performance of a building, although this is not a code requirement.

Where diaphragms are subject only to inertial forces, F_{px} forces are used for collector design, amplified by Ω_0 where required by ASCE 7 §12.10.2. Where transfer forces occur, the combined inertial and transfer forces are used in collector design, with both inertial and transfer forces amplified by Ω_0 where required by ASCE 7 §12.10.2. Like chord forces, collector forces are generally tension and compression. **Figure 5-1** shows a diagram of the diaphragm collector force. The force in the collector on Line 2 can be calculated as the unit shear in the diaphragm (summed from the areas of the diaphragm at both sides of the collector in this case) under the load direction in question, multiplied by the length of the collector, and then multiplied by Ω_0 factors where applicable. As shown in **Figure 5-1**, it is useful to draw the diaphragm shear diagram and then develop the collector diagram based on the shear diagram. Where collectors occur at openings within shear walls, the collector force can go from tension at one end of the opening to compression at the other end, as shown in **Figure 3-5**. The collector needs to extend across the supporting shear wall a distance that is adequate to transfer collector tension and compression forces into the shear wall. The overlap length also needs to be long enough to avoid the creation of a stress concentration at the overlap. Collectors will ideally extend the full length of the supporting shear walls to ensure uniform force transfer into the wall over the length of the wall.

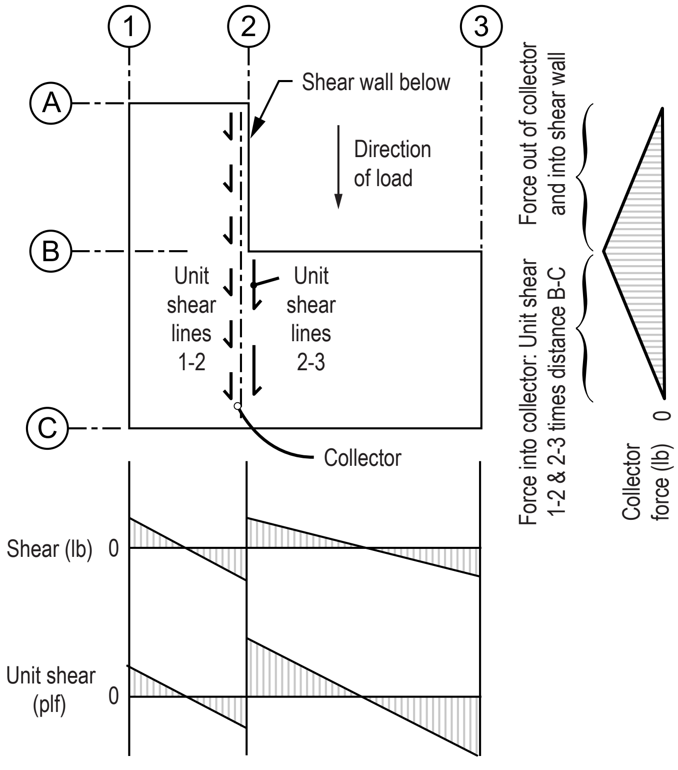


Figure 5-1. Plan view of diaphragm with its collector force diagrams.

5.3 Design Forces in Irregular Structural Systems

Irregular structural systems cause concentrations of force and deformation demand in portions of the SFRS. In diaphragm design, these force and deformation concentrations create increased demands on the diaphragm and boundary elements. Vertical and plan irregularities, applicable to all SFRS types, are identified in ASCE 7 Tables 12.3-1 and 12.3-2. This section discusses the effect of some of the irregularities on the design of wood light-frame diaphragms.

Structures will generally have a torsional response where the vertical elements of the SFRS are not distributed in proportion to the seismic mass. As a result, the center of rigidity of the vertical elements and the center of mass are separated, and this separation causes the structure to twist about its vertical axis. Torsionally irregular buildings create increased demand on the diaphragms, and attention to distribution of this demand within the diaphragms is necessary. The diaphragm must be designed to ensure that diaphragm forces can be delivered to the vertical elements of the SFRS. In addition, where torsional irregularities as defined by ASCE 7 Table 12.3-1 occur, design forces for connection of the diaphragm to the vertical elements are required to be increased by 25 percent in SDC D to F. Also, forces for design of collectors and their connections to vertical elements of the SFRS are required to be increased by 25 percent unless they are already being designed using overstrength factors. This 25 percent increase will affect collectors and their connections to vertical elements in systems braced entirely by light-frame walls because these are exempt from using overstrength design forces.

Reentrant corners within diaphragms are also identified by ASCE 7 Table 12.3-1 as a plan irregularity, and the 25 percent increase design forces is also applicable for diaphragm connections to vertical elements, collectors, and collector connections to vertical elements in SDC D to F. The behavior of concern involves both "wings" of the diaphragm moving apart, creating a concentration of tension at the corner (**Figure 5-2**) or the wings moving towards each other, creating a compression force concentration. Where wood light-frame diaphragms are modeled as simple spans between supporting vertical elements, reentrant corners are considered to occur only within the diaphragm span. Offsets in the diaphragm can occur at diaphragm supports (see **Figure 3-1(b)**) without being classified as a reentrant corner; detailing of nominal continuity at the support, however, is a good practice. For the diaphragm and load direction illustrated in **Figure 5-2**, the boundary elements along Lines A and B would best be identified as chords, and the boundary elements along Lines 2 and 3 as collectors. This classification reverses when the direction of load changes.

Diaphragm discontinuity irregularities include large openings in diaphragms. These openings can result in significant changes in diaphragm stiffness and strength. Similar to previously described irregularities, seismic design forces for boundary members at diaphragm openings are required to be increased by 25 percent unless overstrength forces are used. Where diaphragm openings are large enough to be considered a diaphragm discontinuity irregularity per ASCE 7 Table 12.1-1, the 25 percent increase will apply to all boundary elements in SDC D to F. Use of overstrength forces is not commonly considered to be triggered for boundary elements at diaphragm openings.

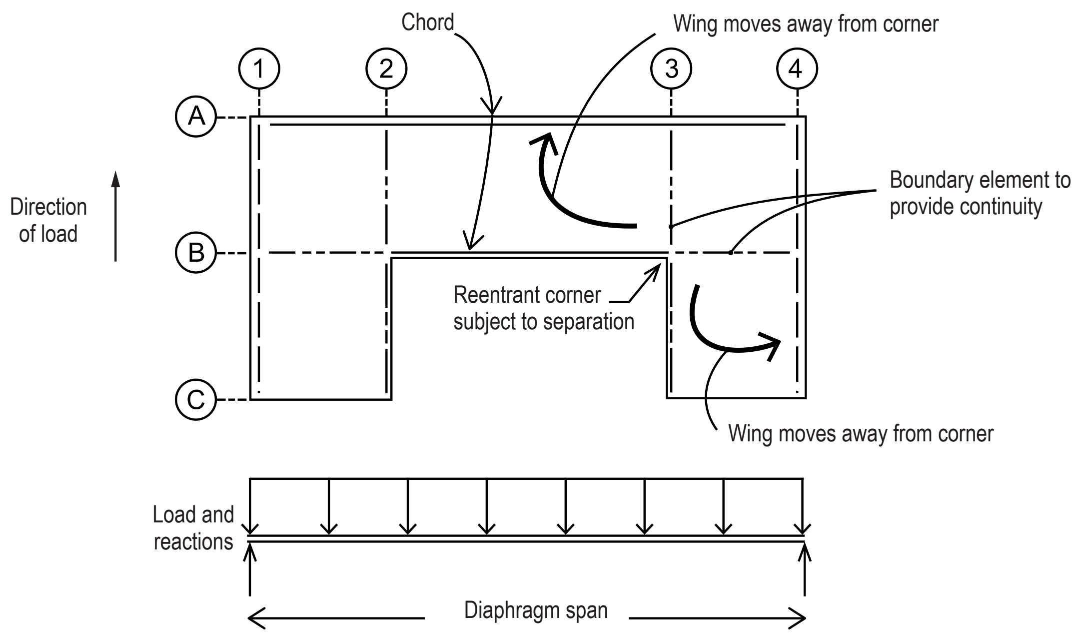


Figure 5-2. Plan view of diaphragm showing reentrant corner and potential seismic response.

Out-of-plane offsets in vertical elements of the SFRS cause transfer forces to occur. These transfer forces can significantly increase diaphragm design forces and need to be added to the inertial forces for diaphragm design. The 25 percent increase in diaphragm design forces required by ASCE 7 must also be applied for this irregularity unless overstrength forces are used.

Proposed Modifications to ASCE 7 Diaphragm Design Force Levels in the 2015 NEHRP Provisions

Recommended modifications to the diaphragm design forces of ASCE 7 §12.10 are included in the 2015 *NEHRP Provisions*. For design of wood light-frame diaphragms these modifications provide an alternate method for determination of design forces. The modifications were developed based on extensive testing and analysis conducted by the precast concrete industry and evaluation of seismic demand from a number of recent testing and analysis projects. The proposed modifications specifically recognize the effect that diaphragm system ductility and deformation capacity have on the force level required to be resisted by the diaphragm. For diaphragm systems of low ductility and overstrength, diaphragm design forces are recommended to increase significantly above current ASCE 7 design forces. For systems of high ductility and overstrength, diaphragm design forces are recommended to remain approximately equivalent to current design practice. Precast concrete diaphragms fall in the former category, while wood light-frame diaphragms fall in the later category.

6. Modeling and Analysis Guidance

The seismic forces for design of diaphragm sheathing and boundary elements must be determined from the building analysis. This section discusses the modeling and analysis methods used to determine the distribution of seismic forces to vertical elements and to determine the seismic demands in the diaphragm sheathing and boundary elements. Included are discussions of equivalent lateral force analysis and dynamic analysis and guidance on suitability of analysis methods. Also provided in this section is guidance on the determination of diaphragm stiffness and calculation of diaphragm deflections; although not common, there are some instances where these calculations are necessary.

6.1 Equivalent Lateral Force Analysis

The vast majority of buildings with wood light-frame diaphragms are designed for seismic forces using the Equivalent Lateral Force procedure of ASCE 7 §12.8. When using this procedure, the diaphragm is classified as flexible, rigid, or semi-rigid, as discussed in Section 4.2. This classification controls both the distribution of forces to vertical elements of the SFRS and the design of the diaphragm.

Flexible Diaphragm Analysis (Beam Analogy)

In its most rudimentary form, flexible diaphragm analysis can be visualized as tributary area analysis, where a line is drawn midway between bracing walls on a floor or roof plan and where the seismic load generated on each side of the line is considered tributary to the corresponding bracing wall. Flexible diaphragm analysis is usually implemented using analysis models based on a simply spanning beam, spanning between each supporting wall line or other vertical element of the SFRS in the story below. This concept is illustrated in **Figure 3-1(a)**. The diaphragm components are then sized to resist the maximum shear and moment determined from the simple span beam, with the sheathing and associate nailing resisting the shear and the chords resisting the moment. Where inertial forces generate a uniform design seismic force, *w*, the maximum shear, unit shear, moment, and chord forces are determined as

$$V = \frac{wL}{2} \qquad v = \frac{V}{d}$$

$$M = \frac{wL^2}{8} \qquad C = T = \frac{M}{d}$$

where, *w*, *d*, and *L* are shown in **Figure 3-1a**, *V* is the maximum total shear on the diaphragm (i.e., the reaction), *v* is the unit shear used for design and tabulated resistances, *M* is the maximum moment resisted by the diaphragm (located at mid-span), and *C* and *T* are the tension and compression forces to be resisted by the chords. The distributed seismic design force, *w*, along the length of the diaphragm, *L*, represents the inertial force associated with all of the tributary seismic force for the diaphragm divided by the length of the diaphragm. In this analysis method, the shear is assumed to have a uniform distribution over the depth of the diaphragm, *d*, rather than the parabolic shear stress distribution typically associated with solid rectangular beams.

When wood diaphragms extend over lines of vertical elements, it is typically assumed that the diaphragm can be broken into multiple simple span beams, as shown in **Figure 3-1(b)**. It is more realistic, however, to consider that the diaphragm will have some moment continuity over interior supports. Common design practice has been to ignore the continuity in diaphragm design but to provide chord member detailing that will allow for nominal moment continuity. To date this modeling assumption has not been identified as the cause of poor seismic performance observed in actual earthquakes. The designer may choose to consider full diaphragm continuity at interior supports, provided that diaphragm shear and moment demands used for diaphragm design correspond to this assumption. This choice will result in higher unit shears and boundary element forces in the vicinity of the diaphragm interior supports.

Rigid or Semi-Rigid Diaphragm Analysis

When diaphragms are classified as rigid, the forces to be transferred in and out of the diaphragm are determined from analysis of the whole building and then imposed on the diaphragm. A rigid diaphragm analysis is illustrated in **Figure 6-1**, where the supporting shear walls in the direction of the loading have significantly different stiffness because of the difference in wall length. Similarly, this condition might be caused by the two supporting lines of resistance being constructed of different materials, such as one masonry wall and one light-frame wall. Although the diaphragm loading is in equilibrium, in this analysis the simple span beam moment diagram described in the previous section will not close (i.e., will not return to zero at Line 2) because of the torsional response that is resisted by the perpendicular walls. When this condition occurs, the analysis can be adjusted to account for the moment resisted by the perpendicular walls, as illustrated in **Figure 6-1**. The shear diagram for the single span beam analogy for the diaphragm is shown, with the unequal reactions because of the differences in stiffness for each wall line. The correction for relatively stiff elements resisting the torsional effects is illustrated in the difference between the unadjusted and adjusted moment diagrams. The non-zero moment at Line 2 is then adjusted to zero. **Figure 6-1** illustrates adjustment of the moment diagram for a uniform seismic design force, *w*. The adjustment process would be similar for a concentrated load, as might occur with transfer forces. The moment at Line 2 will be equivalent to the seismic force in Lines A and

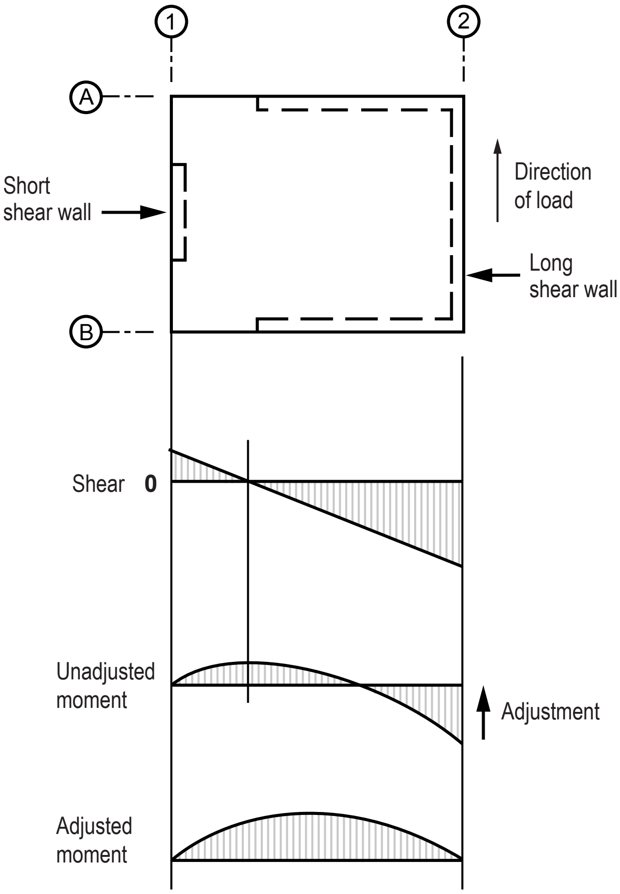


Figure 6-1. Plan view of a rigid diaphragm supported on shear walls of differing stiffness.

B, multiplied by the distance between Lines A and B. This diaphragm moment results in forces in the Line A and B walls, as well as forces in the diaphragm to transfer forces in the Line A and B walls. The unit shears that are developed in response to the torsional response of the diaphragm must be combined with the unit shears determined from the simple beam analysis and they should be included in the analysis of the shears in the orthogonal direction.

For the behavior illustrated in **Figure 6-1** to occur, the Line A and B walls need to be adequately stiff to contribute to torsional resistance; if the walls are not adequately stiff, the diaphragm would exhibit flexible diaphragm behavior. Sometimes the supporting elements that are perpendicular to the loading direction and that act to resist the torsional response of the diaphragm are not located along the chords of the diaphragm but rather are located at some interior position as illustrated in **Figure 6-2**. In this case, resistance to the torsion will cause shear forces in the diaphragm between the lines of resistance and between the elements and the lines associated with the chords as shown.

Semi-rigid diaphragm analysis requires that the flexibility of the diaphragm be considered in analysis, in addition to the flexibility of the shear walls. The modeling assumptions for rigid and semi-rigid behavior are dependent on the load-deflection behavior of the diaphragm and shear walls. As previously noted in the Section 4.2 discussion of diaphragm flexibility, there is a wide range of influences on the load-deflection behavior, many of which range from difficult to impossible to quantify for purposes of analysis. Use of semi-rigid or rigid diaphragm analysis is highly subject to the judgment of the designer.

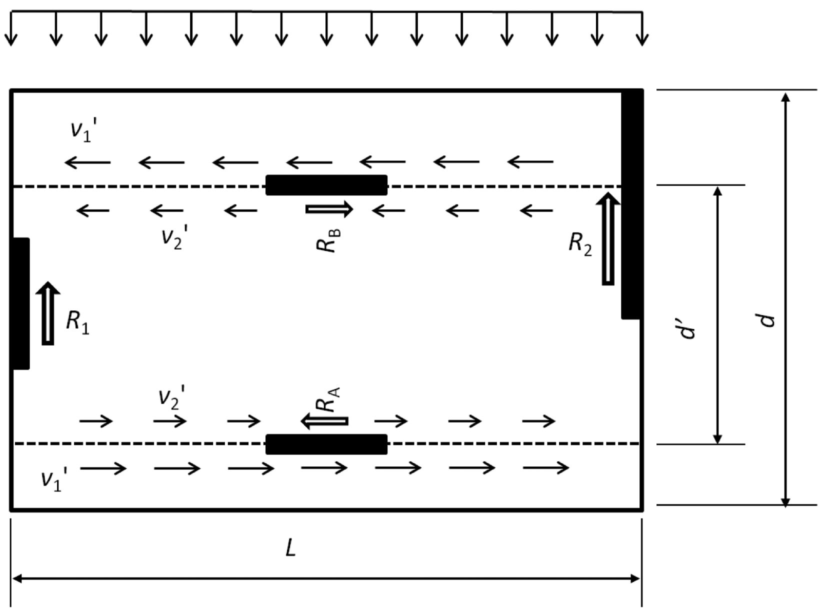


Figure 6-2. Plan view of diaphragm with shear walls inset from the diaphragm perimeter. Dimension *d* is the full dimension of the diaphragm, whereas dimension *d'* is the dimension betweeb shear walls resisting torsion

Internal Load Paths

In wood light-frame diaphragms, the mechanism typically assumed for shear is that of the deep beam as discussed above. This mechanism assumes that the fasteners attaching the sheathing to the framing are providing ductility and redistribution of forces through the diaphragm, and that this results in a linear accumulation of axial force in collectors. Furthermore, this assumes that the relative local stiffness of diaphragm sheathing and associated attachment is lower than the axial stiffness of the collector. This mechanism has been confirmed in diaphragm tests and field observations. The uniform distribution of unit shear is illustrated in **Figure 3-5**. However, if a more rigorous analysis is needed, the analysis should include the nonlinear behavior of the fasteners, as well as a full range of ground motions.

Alternative Design Approach: Performance-Based Seismic Design

Although almost all wood light-frame building design is conducted using forces determined based on ASCE 7 or the IBC equivalent lateral force approaches, there is also a design philosophy known as performance-based seismic design (PBSD). A PBSD method known as direct displacement design (DDD) was originally developed by Priestley (1998) for reinforced concrete bridges and has since been applied to the design of steel and concrete buildings. DDD enables the engineer to explicitly consider the seismic performance of the building during the design process, and it may provide better performance for the structure, particularly for strong earthquake ground motions. A number of researchers have investigated DDD solutions for wood light-frame buildings (Folz and Filiatrault 2002, Pang et al. 2010, van de Lindt et al. 2014). DDD focuses on understanding seismic deformation demand, followed by determination of forces consistent with the deformation demand. This results in identification of real anticipated seismic forces, rather than the reduced seismic forces used in current seismic design methods. The designer is reminded that the higher DDD seismic forces must be transferred through the diaphragm to enable the vertical elements of the SFRS to perform as intended. It is recommended that the designer use NDS LRFD design capacities for demand-to-capacity comparison with this increased DDD diaphragm force level. This was the approach used by van de Lindt et al. (2010) on a six-story wood light-frame building tested at the Maximum Considered Earthquake level, and no diaphragm damage was observed.

6.2 Dynamic Analysis

For wood light-frame buildings, use of linear response history analysis (response spectrum analysis) or nonlinear response history analysis must be undertaken with caution, because any dynamic analysis requires careful incorporation of the effect of nonstructural finishes on the dynamic response of the building.

Response spectrum analysis may be a reasonable tool for studying the dynamic response of unusual building configurations when material properties are appropriately incorporated. However, response spectrum analysis is not appropriate for deriving a reduced base shear for wood light-frame buildings based on the lengthened period of the bare structural system acting alone. Any suggested increase in period and decrease in seismic demand is most often artificial and cannot be relied upon without detailed incorporation of the effect of finishes, including the upper and lower bounds of possible influence.

Recently, tools have become available for nonlinear response history analysis of wood light-frame buildings. These tools are used mostly in research studies. Judgment is required in applying them to building design and in capturing the influence of the range of finish materials. Any application of the tools to building design should be undertaken with caution. One such generic tool is OpenSees (2013), which permits broad description of nonlinear behavior. Where such nonlinear description is used, hysteretic behavior developed in the CUREE project SAWS Program (Folz and Filiatrault 2002) and NEESWood SAPWood Program (Pei and van de Lindt 2010) provide the best available identification of wood structural panel diaphragm behavior. Commercially available analysis software are generally not easily adapted to description of hysteretic behavior for wood light-frame systems.

6.3 Diaphragm Stiffness Modeling, Deflection Calculations

Common reasons for calculating diaphragm stiffness and deflection include classification of diaphragms, cantilevered diaphragm deflection calculations, building separations, evaluation of torsional irregularity, and concern about deflection of supported elements and components. Most analysis of buildings using light-frame diaphragms and equivalent lateral forces will tend toward use of a flexible or rigid diaphragm assumption. Flexible diaphragm analyses most often use hand calculations or simple spreadsheets. Rigid diaphragm analysis is often conducted using analysis spreadsheets. Some analysis programs are available that automate analysis using these simplified approaches.

Deflection equations for wood structural panel diaphragms are provided in the SDPWS standard, accounting for four primary sources of deflection of the overall diaphragm: (1)

chord bending, (2) panel shear deformation, (3) panel nail slip, and (4) chord splice slip. These sources are provided in three terms, with the second and third sources combined into one term. Material properties for these components of deflection are provided in SDPWS and its commentary. Equations in SDPWS are applicable for the calculation of the mid-span deflection of a single span uniformly loaded diaphragm. Adjustment of the design equations is necessary for other diaphragm support and loading conditions, such as cantilevered diaphragms, or where loading departs from a uniform loading condition. Similar four-part equations are provided in the IBC for calculation of deflections of diaphragms fastened with staples rather than nails.

Although not common design practice, where detailed computer analysis models using semi-rigid diaphragms are developed for seismic design, approaches for linear elastic design include modeling the diaphragm using finite element shell or panel elements or using equivalent tension and compression struts. In both of these cases, the material properties need to be derived from available load-deflection descriptions. Diaphragm deflection equations from the AWC SDPWS Standard (AWC 2008) can be used for identifying linear stiffness in the range of design capacities, from which shell or diagonal strut properties can be derived. Built into the derivation of the diaphragm equations is the modeling of a simple span beam in which the shear varies from zero up to the design capacity. In the three-part SDPWS deflection equation, the effective stiffness portion will often dominate. Where this is the case, an average effective area and shear modulus can be derived for the finite element shell element, with flexural section properties set to near zero to avoid influence. Diagonal strut properties can be similarly assigned. Where this approach is used, variability from the modeled solution is likely to occur from a number of sources, including varying load level, hysteretic behavior significantly more complex than described, and influence of finish materials including flooring, roofing, and gypsum board or plaster ceilings. Where diaphragm stresses exceed design levels, use of testing data published by APA and other sources is recommended for development of model load-deflection relationships. Where semi-rigid descriptions of diaphragms are used for analysis, the user is cautioned to carefully consider the stiffness and potentially significant variation in stiffness of the vertical elements of the SFRS.

7. Design Guidance

As noted in Section 1, design requirements for wood light-frame diaphragms are found in the IBC, ASCE 7, SDPWS, and the NDS. The IBC and ASCE 7 primarily define the seismic demand, while the SDPWS and NDS primarily address capacity and related design requirements for diaphragm members, sheathing, and connections.

7.1 Design Limitations

The primary provisions for design of wood light-frame diaphragms are found in SDPWS §4.1 and §4.2. These sections impose design limitations in addition to addressing design methods and capacities. This section discusses these design limitations.

SDPWS §4.1.1 requires that a continuous load path be provided to transfer all forces from the point of application to the point of resistance. SDPWS §4.1.2 requires that deformation compatibility of members and connections be considered in design, and in particular it should be determined that the anticipated deflection of the SFRS will not cause failure of any structural element or connection. These are important concepts that need to be considered in design and detailing.

SDPWS §4.1.7 limits seismic forces in toe-nailed connections for buildings in SDC D, E, and F. The connections are limited to low unit shears (150 plf ASD, 205 plf LRFD) based on concerns regarding their performance because of the difficulty of adequate toe-nail installation.

Where wood light-frame diaphragms provide seismic support to concrete and masonry walls, SDPWS §4.1.5 and Exception 1 limit support by wood diaphragms to configurations where torsional force distribution through the diaphragm does not occur. This means that the diaphragm would need to be classified as a flexible diaphragm in accordance with ASCE 7 §12.3.1. This is generally not an issue, because ASCE 7 permits untopped wood structural panel diaphragms to be idealized as flexible where vertical elements are concrete or masonry walls. It does create the limitation that a building cannot have walls on three sides and be completely unbraced on the fourth side, because this configuration would rely on diaphragm rotation for seismic stability.

Blocked and unblocked wood structural panel diaphragms have aspect ratio limits beyond which their use is not recognized in SDPWS provisions. For blocked construction, the maximum ratio of length to width, L/W, is 4, where W is measured parallel to the load direction under consideration, and L is measured perpendicularly. For unblocked construction, the maximum aspect ratio, L/W, is 3. More restrictive limitations on diaphragm aspect ratio are applied for cases involving open front and cantilevered diaphragms where distribution of story shears occurs through diaphragm rotation (**Figure 7-1**).

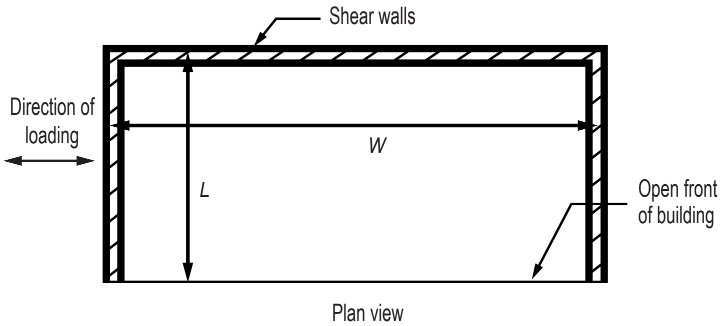


Figure 7-1. Diaphragm in open front structure.

7.2 Diaphragm Design

The processes of diaphragm design and detailing are significantly interconnected for all material types and particularly for wood light-frame diaphragms. For this reason, the discussions in this section and in Section 8 are interconnected and should be considered together.

Design for Diaphragm Shear

Design for shear entails taking shear demands determined as discussed in Sections 5 and 6 of this Guide and determining the combination of sheathing, fastening, and framing required to provide a capacity that meets or exceeds the demand. In making this determination, a number of specific choices that combine design and detailing are made, such as:

- Blocked or unblocked construction, regular or high-load blocked diaphragm
- Framing species, framing member width
- Sheathing grade (Structural I or not), thickness, span direction
- Fastener type, size, spacing

With these items decided, a nominal shear capacity can be selected from SDPWS Table 4.2A, 4.2B, or 4.2C for nailed diaphragms or IBC Chapter 23 for stapled diaphragms. These tables incorporate default choices for the specified choices noted above, identified either in the table headings or in the footnotes. Adjustments to tabulated capacities are required where the default choices are not used.

To select diaphragm sheathing construction of adequate capacity, the designer must carefully determine the maximum unit shear occurring in the diaphragm. The designer should include analysis of the diaphragm on both axes and the increased diaphragm unit shears that occur at diaphragm openings and diaphragm offsets (changes in plan dimension). On larger diaphragms with higher seismic demands, the sheathing fastener spacing is sometimes varied in different areas (zones) of the diaphragm. In this case, shear design for peak shear demand in each zone and consideration of seismic loading in each orthogonal direction are necessary.

Design of Boundary Elements

Design for boundary elements entails design of the boundary members themselves, splices in the boundary members, and force transfer from the boundary members into the supporting shear walls. When designing boundary elements, the following should be kept in mind:

- Members will generally resist tension forces for seismic load in one direction and compression forces when loads are reversed. Design of members and connections needs to consider both loading directions.

- Every edge of wood structural panel sheathing along the diaphragm boundaries, whether at the diaphragm perimeter or internal to the diaphragm, needs to be provided with a boundary element or shear transfer directly to a supporting shear wall.

- Forces in boundary elements need to be transferred to the diaphragm sheathing or to supporting members in a manner that does not cause stress concentrations that are beyond member capacity. For this reason, it is recommended that collectors be carried for the full length of supporting shear walls, and that boundary members at significant diaphragm openings or offsets extend for the full available width of the diaphragm sheathing.

- Collectors interior to the diaphragm will typically be loaded by unit shears coming from two separate diaphragm areas, as defined by the structural modeling, one on each side of the collector line. It is particularly important that sheathing nailing to the collector be designed to transfer shear from both diaphragm areas into the collector. This transfer will often take more than a single line of diaphragm boundary nailing. The nail spacing required should be calculated or two lines of boundary nailing installed.

With boundary member demands determined as discussed in Sections 5 and 6, capacities of members and connections are determined in accordance with the NDS. The capacity of fasteners in wood light-frame members is dependent on the density of the framing member, which for solid-sawn framing is primarily a function of the wood species. Where engineered wood framing members are used, the manufacturer will typically publish a specific gravity that can be assumed for fastener design. The designer must consider this in both sheathing and boundary element design.

Most wood light-frame diaphragm systems have the framing member length primarily oriented in one direction, making it easy to introduce interior boundary elements parallel to the framing direction, using either the framing members themselves or members installed between and parallel to typical framing members. It is often very difficult, however, to install boundary elements perpendicular to the framing direction. This difficultly particularly applies to wood light-frame buildings where the story clear height is often limited to eight feet. In this direction, perpendicular to the framing direction, boundary members are often constructed of straps acting in tension and blocking below the strap and sheathing acting in compression. See **Figure 7-2**. This type of construction requires particular attention to detailing. The blocking and strapping need to be installed tight enough that significant deformation of the strap in tension or the blocking in compression will not occur under seismic loading.

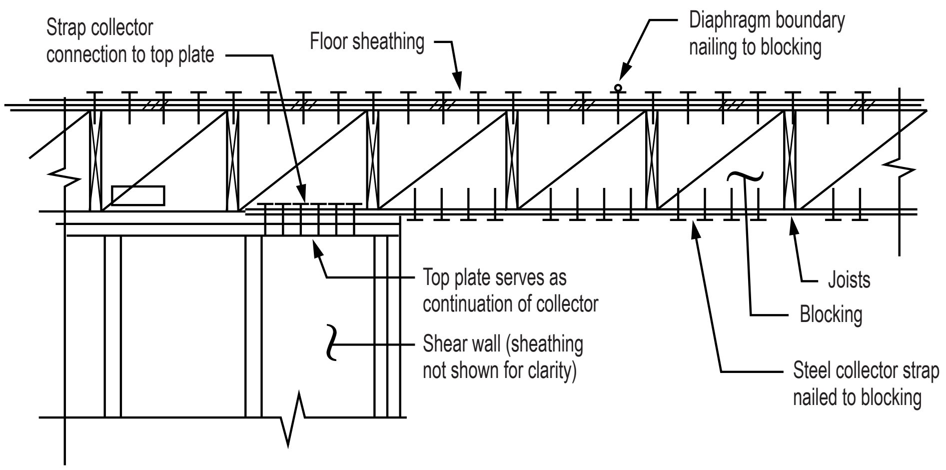


Figure 7-2. Steel strap and blocking providing a boundary member perpendicular to framing direction.

Shear Transfer to Boundary Elements and Shear Walls

The designer must provide a load path design for diaphragm unit shears at all boundaries of a diaphragm. Where the diaphragm shear is simply transferred to supporting shear walls, with no boundary elements required, the design must ensure that boundary nailing is provided at the diaphragm perimeter to the supporting framing and that the load path from supporting framing to the shear wall below is complete. This is primarily a detailing issue, which is addressed in Section 8.

Shear walls sometimes occur inboard of the edge of the diaphragm. This condition is common where diaphragm overhangs occur. The designer must communicate to the builder that diaphragm boundary nailing is required at a location away from the diaphragm edge.

7.3 Local Effect at Discontinuities

Many buildings have discontinuities associated with non-rectangular shapes, vertical offsets, and openings. These irregularities require additional considerations.

Diaphragm openings are almost always present to accommodate stair wells, elevators, mechanical chases, or skylights. If possible, the openings should be located so that the shear force is at a minimum (i.e., in the central portion of the diaphragm) and where narrow sections of the diaphragm are not required to resist large unit shears. Locating the openings at the edges of the diaphragm will require detailing to resist higher forces and could cause a torsional response in the diaphragm because of unbalanced support conditions. It is usually better to set diaphragm openings back from the edge of the diaphragm (by at least the dimension of several sheathing panels, if not more) and to separate openings from each other. This setback will reduce the distance that the collectors are required to transfer the forces and reduce the magnitude of the forces being resisted. A simple analogy that can be used to visualize the effect is to consider the openings as being similar to cutting holes in the web of an I-joist or wide flange beam. Locating the holes near a support can cause the web to fail because of the high shear stresses induced in the remaining web material. Similarly, openings in the diaphragm cause the unit shear to increase in the remaining section of the diaphragm, and if sufficiently increased, they can cause localized failure of the diaphragm. If necessary, additional vertical elements of the SFRS may be required in the vicinity of openings to allow the shear forces to be transferred to the story below and to remove them from the section of the diaphragm with openings.

Another common configuration that causes problems for light-frame wood diaphragms is the reentrant corner, as illustrated in **Figure 5-2**. The chord forces at the reentrant corner must be transferred into the body of the diaphragm. The chord forces at Lines B and 3 need to be transferred into the diaphragm along Line B between Lines 3 and 4 in this example. Similarly, the collector force along Line 3 at Lines B and 3 must to be transferred into the diaphragm along Line 3 between Lines A and B. If the diaphragm framing is oriented such that continuous members can be used for this, the transfer becomes easy. However, if the framing for the diaphragm is oriented perpendicularly to the line of action for the chord forces, then blocking and strapping must be used to transfer the compression and tension forces respectively. The length of this transfer depends on the magnitude of the force and the number of fasteners required to transfer the force from the framing into the sheathing. If internal vertical elements of the SFRS are in-line with the chord or forces at the reentrant corner, then the collectors only have to transfer the forces to the elements. Locating vertical SFRS elements in the reentrant corner can help reduce the loads imposed on the collectors by transferring some of the force out of the chord or strut to the story below.

A common building configuration that was popular in the 1960s and 1970s and is still used in some cases is the split-level layout, where there are vertical offsets to the diaphragm in a given story (usually a few stair risers in height). If a wall or frame is located at the offset position, it must be designed for the forces from each diaphragm acting in opposite directions because the two levels may try to deflect toward or away from each other. These forces can be quite large in some cases. If a wall or frame is not located at the discontinuity for the diaphragms, the designer must ensure that there is an adequate load path to transfer both the chord forces and the overturning forces generated over the height of the diaphragm step. Depending on the size of the vertical offset, knee bracing or some other truss action configurations can be used to transfer the forces in the planes of the two diaphragms.

8. Detailing and Constructability Issues

With each earthquake, our understanding of seismic performance of wood light-frame structures has improved. A common finding documented in post-earthquake reconnaissance of poorly performing wood structures is the lack of adequately detailed load paths. Providing adequate load paths is critical in design and is required by ASCE 7 §12.1.3. The detailing of an adequate load path involves the detailing of a chain of elements, connections, and fasteners that are adequate to transfer seismic forces from the point of origin through the foundation to the supporting soils. These elements can be panels (plywood, OSB), nails, blocking, straps, framing members, and connecting hardware (including anchor bolts.) Any break in the chain of elements, connections, and fasteners will act as a weak link, potentially resulting in poor performance of the structure.

Past earthquakes have shown that the best way to improve performance is through improved quality of construction drawings, including detailing, plan review, and verification through construction inspection and observation. This section discusses the important issues of detailing and constructability that are essential to this improved quality. In addition, *Seismic Detailing Examples for Engineered Light-Frame Timber Construction* (SEAOC 1997) is a recommended reference for load path detailing.

8.1 Panels

Specifications and Grades

Sheathing for wood structural panel diaphragms is required to comply with the U.S. Department of Commerce voluntary product standards PS 1-09 (NIST 2010) or PS 2-10 (NIST 2011). PS 1-09 provides details of fabrication for all-veneer plywood, while PS 2-10 provides performance requirements for plywood and OSB. Three wood structural panel grades are commonly used: Structural I, Sheathing, and Single Floor. Structural I grade sheathing is used where the horizontal shear forces require additional strength for seismic or wind design, or is used where additional cross-panel strength is necessary for gravity loads (such as in a panelized layout where the strength axis of the panel is parallel with the framing members). The Single Floor sheathing grade is intended to be used as a combination of subfloor and underlayment and usually comes with tongue and groove panel edges. Span ratings indicate the maximum center-to-center distance for framing members. The span rating is given in two numbers e.g., 32/16, where the first number is the maximum span in inches for the panel used as a roof and the second number is the maximum span for the panel used as a floor, for average residential occupancy loads.

The panel thickness and span rating are most often selected based on the intended usage for vertical (gravity) loads. Prior to selecting the required thickness and span rating, the orientation of the panels needs to be considered. The common panel orientation is to lay the sheets with the long dimension of the sheet perpendicular to the framing supports. The exception to this common panel orientation is the panelized layout where the long dimension of the sheet is parallel to the framing members. The panel thickness can be selected from IBC Tables 2304.7(3), 2304.7(4), or 2304.7(5), NDS Table C9.2.3, or SDPWS Table C4.2.2C where the sheathing is continuous over two or more spans. Proper specification of floor or roof sheathing panels needs to include the panel thickness, grade, type, and span rating. Orientation of the panel to the framing, if staggered layout is required or not as shown in SDPWS Tables 4.2A and 4.2B, must also be specified.

8.2 Blocked and Unblocked Diaphragms

Nominal Shear Capacities

Determining the nominal unit shear capacities for diaphragms is dependent on two conditions: (1) continuous panel edges and (2) blocking at the panel edges. The direction of lateral force to the framing can be seen in SDPWS Tables 4.2A and 4.2B. The most common nailing pattern for smaller wood structures would be unblocked diaphragms. See **Figure 8-1**, where an unblocked diaphragm has two unsupported panel edges that are always the long sides of the panels. The unsupported edges may be free to deflect, have a panel clip, or have tongue-and-groove edges. The standard nail spacing for unblocked diaphragms is 6 inches at the edges (edge nailing) and 12 inches along the intermediate framing members of the panel or field (field nailing). The nominal shear capacities are 33 percent higher when the direction of the lateral force is perpendicular to the continuous edge. When the diaphragm panel edges are supported with blocking or framing members in a panelized system, the minimum nail spacing is the same as for unblocked diaphragms. However the nominal unit shear capacities for blocked diaphragms can be up to 50 percent higher than for unblocked diaphragms with similar fastening. The nominal unit shear capacities are considerably higher for blocked diaphragms, and for an edge spacing of 2 inches can be as much as 250 percent higher than for unblocked diaphragms. The reason for the higher capacities is that the fasteners are not only providing a stronger shear transfer mechanism but also are restraining the panel edges from buckling. See **Figure 3-2(b)** and **Figure 8-1**.

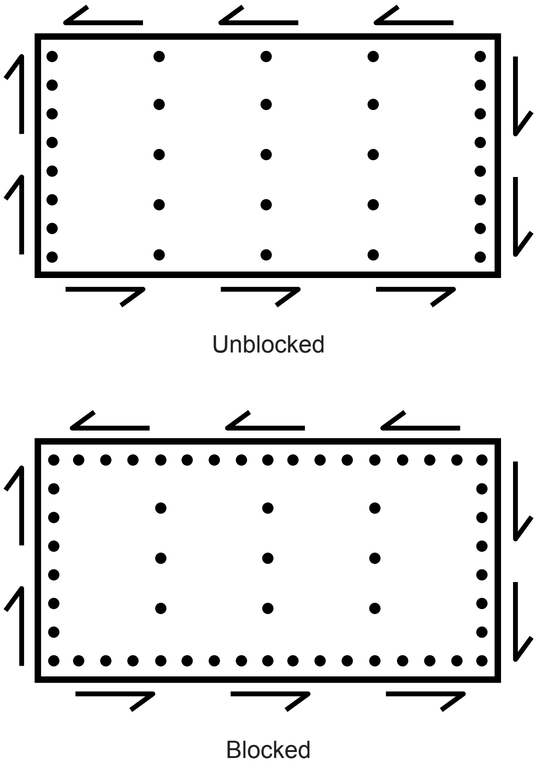


Figure 8-1. Shear transfer in wood structural panels.

Although the strength properties of OSB and plywood are essentially the same, the shear stiffness of OSB can be significantly higher than for plywood, with the difference being about 20 percent higher for the thinner panels with minimum nailing to about 60 percent higher for the thicker panels with a close nail spacing. The listed shear stiffness values in the tables are for 3-ply plywood panels and can be increased by 20 percent when 5-ply plywood panels are used. Footnote 3 of SDPWS Tables 4.2A and 4.2B intends that the apparent shear stiffness values G_a be multiplied by 1.2.

Number of Sheathing Plys

It can be difficult to know when there are 5 plys to the plywood panels because some panels that are 5/8 inch thick can be manufactured with only 3 plys. Refer to APA documents (APA 2012 a, b, c; APA 2013) for number of layers and plys based upon Panel Grade and Span Rating.

Installation

SDPWS Figure 4C shows a 1/8-inch gap between panel edges. The actual panel dimensions for a 4-foot by 8-foot panel are 47 7/8 inches and 95 7/8 inches. These dimensions allow for framing members to be laid out at 16 or 24 inches center-to-center without the spacing having to be adjusted to accommodate the 1/8-inch gap. APA requires use of this 1/8-inch gap for all panel installations. This gap (commonly set by the installer with a nail) is necessary for dimensional changes in the panels when their moisture content increases. The amount of dimensional change varies between plywood and OSB and between manufacturers. In general, OSB panel dimensions can change more than plywood panels for the same change in moisture content because of such factors as differences in wood fiber orientation and method of manufacturing. When the 1/8-inch gap is not installed, the panels have been observed to buckle up or down because of the increased dimensions. These buckled panels can create a problem with the bonding of the built-up roofing systems. OSB panels generally have a mat (non-skid) finish intended for the panel side up and a smooth surface intended for the panel side down. Unblocked diaphragms should use full-size sheets and where cut should be 24 inches minimum in width. For unblocked diaphragms where the panel dimension is less than 24 inches, blocking should be added for panel edge support. This 24-inch minimum is based upon bending strength properties. There is no minimum panel width for a blocked diaphragm because the panel edges already have edge support.

High-Load Diaphragms

Diaphragms carrying large amounts of in-plane shear force have two or three rows of fasteners at the panel edges. This requires framing members to be nominal 3x or 4x members with two rows of nails and 4x members with three rows of nails. Just as for the single row of fasteners, there is a minimum dimension of 3/8 inch required from the sheathing panel edge to the center of the first line of sheathing fasteners, as well as 1/2 inch to 3/4 inch between the rows depending on the framing member width. Nail placements including the 1/8-inch gap at panel edges are depicted in SDPWS Figure 4C. These same nailing figures must be shown on the drawings to ensure proper placement and inspection of placement. The nominal unit shears are listed in SDPWS Table 4.2B and are listed only for 10d nails. SDPWS-08 §4.3.6.3 requires the nail head to be driven flush with the surface of the sheathing.

8.3 Fastening

The fastening of the diaphragm sheathing panels is determined by the in-plane shear. Fastener size and spacing are determined based upon required capacity and the requirements listed in SDPWS Tables 4.2A, 4.2B, and 4.2C for nailed diaphragms and IBC Table 2306.2(1) and 2306.2(2) for stapled diaphragms. Proper specification of fastening is by type, size (pennyweight), length, and spacing for nails; and gage, crown width, length and spacing for staples. The traditional specification called for the nail pennyweight. With the industry standard of using power-actuated devices for fastener installation (power nailers) and the manufacturers producing varying diameters and lengths, simply specifying pennyweight is not sufficient. Proper specification of nail size should also include a diameter (or wire gauge size) and minimum nail length, where the length is not less than the sheathing thickness plus the minimum fastener penetration into framing member specified in SDPWS Tables 4.2A, 4.2B, or 4.2C. Nails should not be located closer than 3/8 inch from the panel edges. Tests have shown that when this dimension is less than 3/8 inch, it can cause premature failure of the panels by fracture. A spacing of 1/2 inch adds considerable toughness to the diaphragm by reducing likelihood of fastener tear-out at the panel edge. This distance can be provided at edge nailing where panels do not abut or where the framing is wider than 2x.

IBC §2304.9.5 requires fasteners used in fire-retardant-treated wood in interior locations to be in accordance with the manufacturer's recommendations and, in the absence of such recommendations, to be in accordance with IBC §23.04.9.5.3.

Proprietary fasteners are available that have embossed heads that are coded to the nail diameter and length as well as nail coating. This enables anyone to quickly know if the correct nail size has been installed without having to witness the installation or having to pull out a nail.

Overdriving of sheathing fasteners, making the fastener head break the face ply, is a common construction problem. This improper installation reduces the capacity of the diaphragm. APA Report TT-012 (APA 2007) lists conditions for when a percentage of nails are overdriven and the magnitude of reduction for shear resistance. Power-actuated overdriving can often be reduced or eliminated with proper depth-of-drive adjustment of the work contact element on the nose of the tool. Warning regarding overdriving and remedies for overdriven nails can be discussed in typical details or general notes in construction documents in order to provide guidance to the builder and inspector.

8.4 Minimum Framing Width

Splitting of Framing

Splitting of framing members supporting diaphragm sheathing can occur when insufficient attention is paid to detailing in construction. There are two considerations for avoiding splitting of framing members: nail size and spacing, with 10d common nails at a close spacing needing the most attention. SDPWS Tables 4.2A, 4.2B, and 4.2C list nominal unit shear capacities for three nail sizes: 6d, 8d, and 10d. These tables also list minimum penetration in the framing member and minimum panel thicknesses for the respective nail sizes and are listed for Douglas-fir larch or southern pine species framing members. These tables also specify that the listed values are for common nails. Minimum width of framing member and edge distances must be maintained. In most cases, splitting of framing members cannot be seen during construction because the splits can be hidden by the sheathing panels. The ends of the framing members need to be inspected to see if splitting is occurring, however, these ends are usually hidden inside of metal hangers. NDS and SDPWS require that nailing be installed so that framing does not split. Where splitting of framing is likely (i.e., closely spaced nails, nailing into older, dryer framing) pre-drilling of nail holes should be specified. For closely spaced large size nails, SDPWS Figure C4.2.7.1.1(3) shows the required staggering of fasteners at panel edges for 2x framing members and required staggering of fasteners at adjoining panel edges where 3x framing is required. SDPWS also permits a pair of 2x framing members to be used in lieu of a 3x.

Nails in Diaphragm Tables

Hot dip galvanized box nails have long been considered equivalent to common nails and indicated as acceptable in the building code capacity tables. SDPWS permits use of galvanized box nails in the shear wall tables (hot dip galvanized is not specifically noted but should be used because electro-galvanized nails are not recognized to have equivalent capacity). SDPWS does not currently recognize hot dip galvanized box nails in the diaphragm capacity tables; however, their use based on past practice may be justifiable. Caution is required because in general hot dip galvanized box nails are not available for use in power nailers; specifying of hot dip galvanized nails may result in hand nailing being required.

Splitting of Engineered Lumber Members

Splitting of engineered lumber framing members is highly dependent upon the type of engineered lumber (laminated veneer lumber or LVL, laminated strand lumber or LSL, and parallel strand lumber or PSL) and also dependent upon the nails being driven into the face of the member or the edge of the member. The designer should follow the manufacturer's recommendations for nail spacing.

Moisture Content

The APA conducted tests (APA 2002) with shear walls framed with green lumber that was allowed to dry after construction and compared results to tests of shear walls framed with lumber that was dry during the time of construction and remained dry. Green lumber has a moisture content, MC, greater than 19 percent. The tests showed that shear wall stiffness is greatly affected by use of green lumber, but strength is not: a strength loss of approximately 10 percent was observed in the study. Therefore, diaphragm deflections are modified when green lumber is used, but diaphragm capacities need not be modified. Footnote 4 to SDPWS Tables 4.2A to 4.2C provides the required adjustment to the shear stiffness term. An engineer can also require that sheathing not be installed until the MC has dropped to below 19 percent. This can take as little as a week, depending on location and time of year.

8.5 Chords, Collectors, and Boundaries

Many times the actual diaphragm chord is not located directly under the diaphragm sheathing. **Figure 3-4** shows the diaphragm chords located below the framing members, and in this case, the blocking or rim board transfers the forces from the sheathing to the chord. SDPWS §4.1.4 states that the diaphragms, chords, and collectors are to be placed in or in contact with the diaphragm framing unless it can be demonstrated the eccentricities can be tolerated. **Figure 3-4** shows the diaphragm chord and collector in contact with the underside of the framing members. Demonstrating that eccentricities can be accommodated is extremely difficult and should be avoided where possible. SDPWS §4.2.2 requires the designer to consider chord splices and slippage as well as fastener deformations when calculating the diaphragm deflections.

When diaphragm boundaries and boundary elements are located interior to the diaphragm rather than at the diaphragm edges, shear transfer to both shear walls and boundary elements are required at interior locations that might not be obvious to the builder when standing on top of the diaphragm while installing sheathing fasteners. It is important that required sheathing boundary nailing locations be made clear to the builder and inspector in the design documents and observed during construction. This communication can be improved by indicating boundary fastener locations on the diaphragm plans as well as in the diaphragm details.

8.6 Continuity in Boundary Member Detailing

The design drawings should clearly show the necessary boundary member splices and state that they should be considered part of the SFRS of the diaphragm. ASCE 7 requires a 25 percent increase in design forces for diaphragms with irregularities for structures in SDC D through F, as discussed in Section 5.3.

8.7 Inspections

IBC §1705.11 requires special inspection of the SFRS in SDC C to F, including the diaphragms and boundary members. There is an exception: if the diaphragm nailing spacing is more than 4 inches center-to-center, the special inspections can be omitted.

Special inspections of high-load diaphragms are required in IBC §1705, in which case the special inspector is required to inspect the sheathing thickness, grade, nominal width of framing member, nail or staple size, and the size and spacing of the nails. In addition, the conditions need to be shown on the approved plans.

Structural observations are required by IBC §1704.5 for structures assigned to SDC D, E, or F when one of four conditions are met per IBC §1704.5.1.

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10. Notations and Abbreviations

B width, ft

C compressive force, lb

d depth, ft

d pennyweight of nail (e.g., 10d nail)

F_i the (vertical element) design force, lb

F_{px} diaphragm (inertial) design force, lb

F_x vertical element design force, lb

H story height, ft

h_x height above the base to level x, ft

I_e seismic importance factor

L length, ft

M moment, ft-lb

MC moisture content

plf pounds per linear foot

R response modification factor

S_a spectral response acceleration

S_{DS} design spectral acceleration for short-period structures

S_{D1} design spectral response acceleration parameter at 1 sec. period

T_L long period transition

V base shear; also maximum total shear, lb

w weight, pounds per foot

w_i weight tributary to level i, lb

w_{px} weight tributary to the diaphragm at level x, lb

W total load, lb; also width, ft

Ω_0 amplification factor to account for overstrength of the seismic force-resisting system defined in ASCE 7

Abbreviations

ANSI	American National Standards Institute
APA	formerly the American Plywood Association, currently APA—The Engineered Wood Association
ASD	allowable stress design
AWC	American Wood Council
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
BSSC	Building Seismic Safety Council
CUREE	Consortium of Universities for Research in Earthquake Engineering
DDD	direct displacement design
DFPA	Douglas Fir Plywood Association
IBC	International Building Code
LRFD	load and resistance factor design
NDS	National Design Specification for Wood Construction
NEHRP	National Earthquake Hazards Reduction Program
NIST	National Institute of Standards and Technology
OSB	oriented strand board
SDC	seismic design category
SDPWS	Special Design Provisions for Wind and Seismic
SEAOC	Structural Engineers Association of California
SEAONC	Structural Engineers Association of Northern California
SFRS	seismic force-resisting system
UBC	Uniform Building Code

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