SDPWS COMMENTARY

C

FOREWORD

The *Special Design Provisions for Wind and Seismic (SDPWS)* document was first issued in 2002. It contains provisions for materials, design, and construction of wood members, fasteners, and assemblies to resist wind and seismic forces. The 2015 edition is the fourth edition of this publication.

The Commentary to the *SDPWS* is provided herein and includes background information for most sections as well as sample calculations for each of the design value tables.

The Commentary follows the same subject matter organization as the *SDPWS*. Discussion of a particular provision in the *SDPWS* is identified in the Commentary by the same section or subsection. When available, references to more detailed information on specific subjects are included.

In developing the provisions of the *SDPWS*, data and experience with structures in-service has been carefully evaluated by the AWC Wood Design Standards Committee for the purpose of providing a standard of practice. It is intended that this document be used in conjunction with competent engineering design, accurate fabrication, and adequate supervision of construction. Therefore AWC does not assume any responsibility for errors or omissions in the *SDPWS* and *SDPWS Commentary*, nor for engineering designs and plans prepared from it.

Inquiries, comments and suggestions from the readers of this document are invited.

American Wood Council

C2 GENERAL DESIGN REQUIREMENTS

C2.1 General

C2.1.1 Scope

Allowable stress design (ASD) and load and resistance factor design (LRFD) provisions are applicable for the design of wood members and systems to resist wind and seismic loads. For other than short-term wind and seismic loads (10-minute basis), adjustment of design capacities for load duration or time effect shall be in accordance with the *National Design Specification® (NDS®) for Wood Construction* (6).

C2.1.2 Design Methods

Both ASD and LRFD (also referred to as strength design) formats are addressed by reference to the *National Design Specification (NDS) for Wood Construction* (6) for design of wood members and connections. The design of elements throughout a structure will generally utilize either the ASD or LRFD format; however, specific requirements to use a single design format for all elements within a structure are not included. The suitability of mixing formats within a structure is the responsibility of the designer in compliance with requirements of the authority having jurisdiction. *ASCE 7 – Minimum Design Loads for Buildings and Other Structures* (5) limits mixing of design formats to cases where there are changes in materials.

C2.1.3 Sizes

Sizes of wood products vary by both product type and end use conditions. Actual wood product dimensions are typically a function of moisture content at the time that measurements are taken. For this reason, dimensions are reported in terms of reference environmental conditions. Products such as lumber, timbers, and wood structural panels have been historically reported in terms of "nominal" dimension nomenclature. These product names are associated with minimum dimensions at reference environmental conditions or a reference moisture content specification for the manufacturing process as described in the product standards. For example, the minimum dressed dry dimensions of a "2x4" are 1.5 inches x 3.5 inches for the moisture content specification that permits the lumber to be marked as "S Dry".

Similarly, wood structural panels are described in terms of "Performance Category" descriptors which are related to the panel thickness range at reference environmental conditions described as "nominal panel thickness" in this standard. Use of the term "nominal panel thickness" is consistent with terminology used in previous versions of this standard and the U.S. model building codes. Common Performance Categories and Span Ratings are provided in Table C4.2.2C.

C2.2 Terminology

ASD Reduction Factor: This term denotes the specific adjustment factor used to convert nominal design values to ASD reference design values.

Nominal Strength: Nominal strength (or nominal capacity) is used to provide a common reference point from which to derive ASD or LRFD reference design values. For wood structural panels, tabulated nominal unit shear capacities for wind, v_w, (nominal strength) were derived using ASD tabulated seismic values from industry design documents and model building codes (2, 18, 19, 20) times a factor of 2.8. The factor of 2.8, based on minimum performance requirements (8), has commonly been considered the target minimum safety factor associated with ASD unit shear capacity for wood structural panel shear walls and diaphragms in seismic applications. The nominal unit shear capacity for seismic, v_s, was derived by dividing the nominal unit shear capacity for wind by 1.4. This was done to be consistent with the ratio of wind and seismic design capacities for wood structural panel shear walls and diaphragms in model building codes (2) and allow for a single ASD Reduction Factor of 2.0 to be used for both wind and seismic applications. For fiberboard and lumber shear walls and lumber diaphragms, similar assumptions were used.

For shear walls utilizing other materials, ASD unit shear capacity values from model building codes (2) and industry design documents (20) were multiplied by 2.0 to develop nominal unit shear capacity values for both wind and seismic.

While varying approaches are used across the variety of products (e.g. wood structural panels, lumber sheathing, structural fiberboard, gypsum wallboard) and applications (e.g. shear walls, diaphragms, wall sheathing, and roof sheathing) covered by *SDPWS*, the most common basis of nominal strength is the factoring up of historic ASD design values such that when nominal strength is divided by the specified ASD reduction factor, the identical design value to that specified in prior codes will result. Under such an approach, conservatisms inherent in the original ASD design value determination and the product standards are maintained. Importantly, nominal strength values and methods of adjusting values for use as part of engineered design using either ASD or LRFD (e.g. ASD reduction factors or LRFD resistance factors) are based upon the premise that the structural products fully comply with the applicable product standards referenced in this Specification. For example, wood structural panels used in shear wall and diaphragm applications or as wall and roof sheathing must conform to the requirements found in DOC PS 1 (58) or PS 2 (8) as applicable. These product standards include minimum performance requirements, applicable test methods, and quality assurance for which design methods in this specification are considered suitable. Examples of such baseline criteria include requirements for strength, stiffness, and fastener performance in product standards for sheathing and panel products used in shear walls and/or diaphragms.

Resistance Factor: For LRFD, resistance factors are assigned to various wood properties with only one factor for each stress mode (i.e. bending, shear, compression, tension, and stability). Theoretically, the magnitude of a resistance factor is considered to, in part, reflect relative variability of wood product properties. However, for wood design provisions, actual differences in product variability are already embedded in the reference design values. This is due to the fact that typical reference design values are based on a statistical estimate of a near-minimum value (5th percentile).

The following resistance factors are used in *SDPWS*: a) sheathing in-plane shear, ϕ_D = 0.80, b) sheathing out-of-plane bending ϕ_b = 0.85, and c) connections, ϕ_z = 0.65. LRFD resistance factors have been determined by an ASTM consensus standard committee (16). Examination of ϕ_D=0.80 is addressed in ASTM D 5457 (16) where calibration is used to reduce differences between ASD and LRFD for in-plane shear. For seismic design of shear walls and diaphragms, the specified ϕ_D = 0.80 is applied to the nominal unit shear capacity for seismic which is a reduced nominal unit shear capacity derived by dividing the nominal unit shear capacity for wind by 1.4 (see C2.2 Nominal Strength). Use of the reduced seismic nominal unit shear capacity times ϕ_D = 0.80 is algebraically equivalent to use of an "effective ϕ_D = 0.57" times the unreduced nominal unit shear capacity (i.e. nominal unit shear capacity for wind, R_{wind}) for calculation of the LRFD design unit shear capacity for seismic:

$$v_{Seismic-LRFD} = 0.8\,R_{Seismic} = \frac{0.8\,R_{Wind}}{1.4} = 0.57\,R_{Wind} \qquad \text{(C2.2-1)}$$

where:

0.80 = ϕ_D; sheathing resistance factor for in-plane shear of shear walls and diaphragms for both wind and seismic

R_{wind} = nominal unit shear capacity for wind

$R_{seismic}$ = nominal unit shear capacity for seismic

0.57 = "effective ϕ_D" where reference strength is associated with the unreduced nominal unit shear capacity (i.e. R_{wind})

C3 MEMBERS AND CONNECTIONS

C3.1 Framing

C3.1.1.1 Wall Stud Bending Strength and Stiffness Design Value Increase: Wall studs sheathed on both sides are stronger and stiffer in flexure (i.e., wind loads applied perpendicular to the wall plane) than those in similar, unsheathed wall assemblies. The enhanced performance of these wood stud wall assemblies is recognized in wood design with the wall stud repetitive member factor, C_r, for bending strength and stiffness, which accounts for effects of partial composite action and load-sharing (1). This is in contrast to the repetitive member factor, C_r, in the *NDS* (6) which applies to a much broader range of repetitive member assembly applications and is limited to bending stress increases of no more than 15%. Increases in the assembly stiffness are directly proportional to partial composite action and load-sharing; the effect can conservatively be approximated with the wall stud repetitive member factor. Wall stud repetitive member factors in *SDPWS* Table 3.1.1.1 are applicable for out-of-plane wind loads and were derived based on wall tests (9). Factors in Table 3.1.1.1 increase the calculated strength and stiffness of pinned-end, bare studs when used in sheathed wall assemblies in accordance with 3.1.1.1. A factor of 1.56 was determined for a wall configured as follows:

Framing	2x4 Stud grade Douglas fir studs at 16" o.c.
Interior Sheathing	1/2" gypsum wallboard attached with 4d cooler nails at 7" o.c. edge and 10" o.c. field (applied vertically).
Exterior Sheathing	3/8" rough sanded 303 siding attached with 6d box nails at 6" o.c. edge and 12" o.c. field (applied vertically).

For design purposes, a slightly more conservative value of 1.5 was chosen to represent a modified 2x4 stud wall system as follows:

Framing	2x4 Stud grade Douglas fir studs at 24" o.c.
Interior Sheathing	1/2" gypsum wallboard attached with 5d cooler nails at 7" o.c. edge and 10" o.c. field (applied vertically).
Exterior Sheathing	3/8" wood structural panels attached with 8d common nails at 6" o.c. edge and 12" o.c. field (blocked).

For other stud depths, the wall stud repetitive member factor is taken as a function of the relative stiffness (EI) of the stud material. A repetitive member factor of 1.15 was assumed for a 2x12 stud in a wall system and Equation C3.1.1-1 was used to interpolate repetitive member factors for 2x6, 2x8, and 2x10 studs:

$$C_r = 1.15\left[\frac{178in^4}{I_{stud}}\right]^{0.076} \quad \text{(C3.1.1-1)}$$

Slight differences between calculated C_r values and those appearing in *SDPWS* Table 3.1.1.1 are due to rounding.

C3.2 Sheathing

Nominal uniform load capacities in *SDPWS* Tables 3.2.1 and 3.2.2 assume a two-span continuous condition. Out-of-plane sheathing capacities are often tabulated in other documents on the basis of a three-span continuous condition. Although the three-span continuous condition results in higher capacity, the more conservative two-span continuous condition was selected because this condition frequently exists at building end zones where the largest wind forces occur.

Table C3.2A provides out-of-plane bending strength capacities for wood structural panels based on bending design values and section properties from AWC's *2012 ASD/LRFD Manual for Engineered Wood Construction* (Section M9: Wood Structural Panels) (60). As noted in footnotes 1 and 2 to Table C3.2A, the bending capacities are based on the lesser of the calculated bending capacities for OSB or plywood for the given span rating.

Table C3.2B provides out-of-plane shear strength capacities for wood structural panels based on shear design values and section properties from AWC's *2012 ASD/LRFD Manual for Engineered Wood Construction* (Section M9: Wood Structural Panels). The shear capacities are based on the lesser of the calculated shear capacities for OSB or plywood for the given span rating.

Examples C3.2.1-1 and C3.2.1-2 illustrate how values in *SDPWS* Table 3.2.1 were generated using wood structural panel out-of-plane bending and shear values given in Tables C3.2A and C3.2B. Although the following two examples are for *SDPWS* Table 3.2.1, the same procedure can be used to generate values shown in *SDPWS* Table 3.2.2.

Table C3.2C provides out-of-plane bending strength capacities for cellulosic fiberboard sheathing based on minimum modulus of rupture criteria in ASTM C 208. Nominal uniform load capacities for cellulosic fiberboard sheathing in *SDPWS* Table 3.2.1 can be derived using the same procedure as described in Example C3.2.1-1.

Table C3.2A Wood Structural Panel Sheathing Dry Design Bending Strength Capacities

Span Rating: *Sheathing*	Bending Strength, F_bS (lb-in./ft width)		
	Strength Axis Perpendicular to Supports	Strength Axis Parallel to Supports	
		12"&16" spacing of framing members	24" spacing of framing members
24/0	250[1]	54[1]	65[2]
24/16	320[1]	64[1]	77[2]
32/16	370[1]	92[1]	110[2]
40/20	625[1]	150[1]	180[2]
48/24	930[2]	270[2]	270[2]

1. Tabulated bending strength capacities are based on the lesser of OSB or plywood with 3 plies for the given span ratings.
2. Tabulated bending strength capacities are based on the lesser of OSB or plywood with 4 plies for the given span ratings.

Table C3.2B Wood Structural Panel Sheathing Dry Shear Capacities in the Plane

Span Rating: *Sheathing*	Shear in the Plane, F_S [lb/Q] (lb/ft width)
	Strength Axis Either Perpendicular or Parallel to Supports
24/0	130
24/16	150
32/16	165
40/20	205
48/24	250

Table C3.2C Cellulosic Fiberboard Sheathing Design Bending Strength Capacities

Span Rating: *Sheathing*	Bending Strength, F_bS (lb-in./ft width)
	Strength Axis Either Perpendicular or Parallel to Supports
Regular 1/2"	55
Structural 1/2"	80
Structural 25/32"	97

EXAMPLE C3.2.1-1 Determine the Nominal Uniform Load Capacity in SDPWS Table 3.2.1

Determine the nominal uniform load capacity in *SDPWS* Table 3.2.1 Nominal Uniform Load Capacities (psf) for Wall Sheathing Resisting Out-of-Plane Wind Loads for the following conditions:

Sheathing type	= wood structural panels
Span rating or grade	= 24/0
Min. nominal thickness	= 3/8 in.
Strength axis	= perpendicular to supports
Actual stud spacing	= 12 in.

ASD (normal load duration, i.e., 10-yr) bending capacity:
F_b S = 250 lb-in./ft width from Table C3.2A

ASD (normal load duration, i.e., 10-yr) shear capacity:
F_s I b/Q = 130 lb/ft width from Table C3.2B

Maximum uniform load based on bending strength for a two-span condition:

$$w_b = \frac{96F_bS}{l^2} = \frac{96\times250}{12^2} = 167 \text{ psf}$$

Maximum uniform load based on shear strength for a two-span condition:

$$w_s = \frac{19.2F_sIb/Q}{l_{clearspan}} = \frac{19.2\times130}{(12-1.5)} = 238 \text{ psf}$$

Maximum uniform load based on bending governs. Converting to the nominal capacity basis of *SDPWS* Table 3.2.1:

$$w_{\text{nominal}} = \left(\frac{2.16}{\phi_b}\right)\times ASD_{10\text{-}yr} \qquad \textit{SDPWS}\text{ Table 3.2.1}$$

$$= \frac{2.16}{0.85}\times167 = 424 \text{ psf}$$

$$\approx 425 \text{ psf}$$

where:

2.16/0.85 = conversion from a normal load duration (i.e., 10-yr ASD basis) to the short-term (10-min) nominal capacity basis of *SDPWS* Table 3.2.1.

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EXAMPLE C3.2.1-2 Determine the Nominal Uniform Load Capacity in SDPWS Table 3.2.1

Determine the nominal uniform load capacity in *SDPWS* Table 3.2.1 Nominal Uniform Load Capacities (psf) for Wall Sheathing Resisting Out-of-Plane Wind Loads for the following conditions:

Sheathing type = wood structural panels
Span rating or grade = 40/20
Min. nominal thickness = 19/32 in.
Strength axis = perpendicular to supports
Actual stud spacing = 12 in.

ASD (normal load duration, i.e., 10-yr) bending capacity:
F_b S = 625 lb-in./ft width from Table C3.2A

ASD (normal load duration, i.e., 10-yr) shear capacity:
F_s I b/Q = 205 lb/ft width from Table C3.2B

Maximum uniform load based on bending strength for a two-span condition:

$$w_b = \frac{96F_bS}{l^2} = \frac{96 \times 625}{12^2} = 417 \text{ psf}$$

Maximum uniform load based on shear strength for a two-span condition:

$$w_s = \frac{19.2F_sIb/Q}{l_{clearspan}} = \frac{19.2 \times 205}{(12-1.5)} = 375 \text{ psf}$$

Maximum uniform load based on shear governs. Converting to the nominal capacity basis of *SDPWS* Table 3.2.1:

$$w_{nominal} = \left(\frac{2.16}{\phi_b}\right) \times ASD_{10-yr} \qquad \textit{SDPWS} \text{ Table 3.2.1}$$

$$= \frac{2.16}{0.85} \times\ 375 = 953 \text{ psf}$$

$$\approx 955 \text{ psf}$$

where:

2.16/0.85 = conversion from a normal load duration (i.e., 10-yr ASD basis) to the short-term (10-min) nominal capacity basis of *SDPWS* Table 3.2.1.

C3.2.1 Wall Sheathing

Where wood structural panel wall sheathing is used to provide the load path for wind uplift loads, the walls must be designed in accordance with 4.4. All other uplift force resisting systems must comply with 3.4.2.

C3.3 Connections

Section 3.3 refers the user to the *NDS* (6) when designing connections to resist wind or seismic forces. In many cases, resistance to out-of-plane forces due to wind may be limited by connection capacity (withdrawal capacity of the connection) rather than out-of-plane bending or shear capacity of the panel.

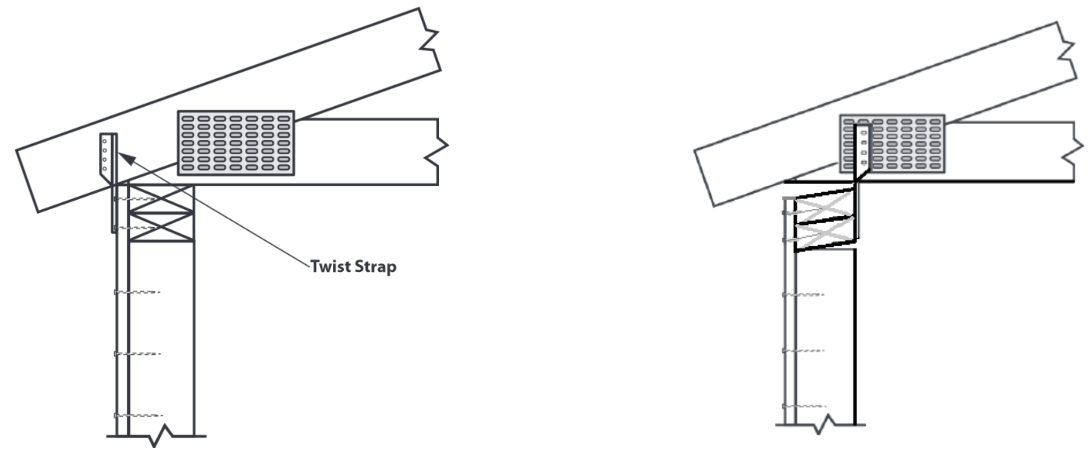
C3.4 Uplift Force Resisting Systems

Section 3.4 applies to all uplift force resisting systems. Where wood structural panels are used to provide load path for wind uplift loads, the provisions of 4.4 apply in lieu of 3.4.2. All other uplift force resisting systems, including systems that utilize metal connectors and/or continuous rod tie-downs consisting of threaded rods or cables, must comply with Section 3.4.2. The design of such systems must be in accordance with accepted engineering practice and, for systems utilizing proprietary components, the manufacturer's literature and code evaluation report, where applicable. The design must account for stresses induced into framing members and all other components and hardware within the system, as well as resulting deflections thereof.

When detailing uplift force resisting systems, eccentricities in the uplift load path should be minimized or eliminated wherever possible (see Figure C3.4). In locations where this is not possible, the effect of eccentricities in the uplift load path must be considered. Such effects may include, but are not limited to: moments induced into elements of the uplift force resisting system; torsion within, and rotation of, the top plate and/or other elements within the system; cross-grain bending and/or tension perpendicular-to-the-grain of wood members within the system.

Eccentricities resulting in cross-grain bending or tension perpendicular-to-the-grain stresses within wood members should be avoided. Other factors that must be considered include deflection compatibility of the uplift force resisting system with the wall(s) in which it is used and dimensional changes in the wood elements due to moisture content fluctuations.

Figure C3.4 Examples of Roof Uplift Connectors



Appropriate uplift load path with uplift connector aligned with the wood structural panel sheathing providing wind uplift resistance.

Inappropriate uplift load path prone to top plate rotation due to eccentric placement of uplift connector relative to the wood structural panel sheathing providing wind uplift resistance.

C4 LATERAL FORCE-RESISTING SYSTEMS

C4.1 General

C4.1.1 Design Requirements

General design requirements for lateral force-resisting systems are described in this section and are applicable to engineered structures.

C4.1.2 Shear Capacity

Nominal unit shear capacities (see C2.2) for wind and seismic require adjustment in accordance with *SDPWS* 4.2.3 for diaphragms and *SDPWS* 4.3.3 for shear walls to derive an appropriate design value.

C4.1.3 Deformation Requirements

Consideration of deformations (such as deformation of the overall structure, elements, connections, and systems within the structure) that can occur is necessary to maintain load path and ensure proper detailing. Special requirements are provided for wood members resisting forces from concrete and masonry (see C4.1.5) due to potentially large differences in stiffness and deflection limits for wood and concrete systems. Special requirements are also provided for open front buildings (C4.2.5.2) where forces are distributed by diaphragm rotation.

C4.1.4 Boundary Elements

Boundary elements must be sized to transfer design tension and compression forces. Good construction practice and efficient design and detailing for boundary elements utilize framing members in the plane or tangent to the plane of the diaphragm or shear wall.

C4.1.5 Wood Members and Systems Resisting Seismic Forces Contributed by Masonry and Concrete Walls

The use of wood diaphragms with masonry or concrete walls is common practice. Story height and other limitations for wood members and wood systems resisting seismic forces from concrete or masonry walls are given to address deformation compatibility and are largely based on field observations following major seismic events. Wood diaphragms and horizontal trusses are specifically permitted to resist horizontal seismic forces from masonry or concrete walls. For construction over one story in height, wood diaphragms and horizontal trusses are permitted to resist horizontal seismic forces from masonry or concrete walls provided that the design of the diaphragm does not rely on torsional force distribution through the diaphragm. Primary considerations for this limitation are the flexibility of the wood diaphragm relative to masonry or concrete walls and the limited ability of masonry or concrete walls to tolerate out-of-plane wall displacements without failure.

The term "horizontal trusses" refers to trusses that are oriented such that their top and bottom chords and web members are in a horizontal or near horizontal plane. A horizontal truss transmits lateral loads to shear walls in a manner similar to a floor or roof diaphragm. In this context, a horizontal truss is a bracing system capable of resisting horizontal seismic forces contributed by masonry or concrete walls.

Where wood structural panel shear walls are used to provide resistance to seismic forces contributed by masonry and concrete walls, deflections are limited to 0.7% of the story height in accordance with deflection limits (5) for masonry and concrete construction. Strength level forces and appropriate deflection amplification factors, C_d, in accordance with ASCE 7 should be used when calculating design story drift, Δ. The intent of the design story drift limit is to limit failure of the masonry or concrete portions of the structure due to excessive deflection. For example, inadequate diaphragm stiffness may lead to excessive out-of-plane deformation of the attached masonry or concrete wall.

C4.1.5.1 Anchorage of Concrete or Masonry Structural Walls to Diaphragms: For seismic design, the use of subdiaphragms as an analytical tool for transfer of wall anchorage forces was first introduced into the building code in the *1976 Uniform Building Code* (UBC) (50) along with special detailing requirements to address observations of earthquake damage to buildings having plywood dia-

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phragms and walls of structural concrete or masonry (46, 47, 48, and 49). The changes were introduced to prohibit designs that induced cross grain bending in wood members and relied on plywood sheathing in lieu of tension ties. The requirement for continuous ties, the subdiaphragm concept, and special detailing requirements in *SDPWS* are consistent with those found in ASCE 7 and prior editions of the building code where they originally appeared.

Continuous ties spanning the full width of the diaphragm are required to be provided at wall anchorage points to the diaphragm so that the entire diaphragm width is engaged in resisting wall anchorage forces. Because wall anchors 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. This analytical tool enables a designer to detail connections along the continuous tie load paths within the diaphragm, without resorting to complex analysis which would be needed given the high level of redundancy within a wood framed diaphragm. The subdiaphragm is a smaller diaphragm within the main diaphragm (Figure C4.1.5A) designed to ensure the local wall anchorage forces can be safely transferred through the connections and members to the main diaphragm. 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 C4.1.5A illustrates subdiaphragms that anchor the west wall 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 and along the east wall.

Use of subdiaphragms for purposes of distributing concrete or masonry structural wall anchorage forces may entail multiple subdiaphragms with subdiaphragm span taken as the distance between continuous cross-ties of the main diaphragm, or use of a subdiaphragm that spans the full distance between side walls. In either case, subdiaphragms must meet all requirements for diaphragms and additionally are limited to a maximum aspect ratio of 2.5:1 consistent with the ASCE 7 limitation and intended to limit bending deformation and address deformation compatibility with the rest of the diaphragm. The 2.5:1 aspect ratio limit first appeared in the 1997 UBC. Although some designs have successfully used aspect ratios as high as 4, which is the limiting aspect ratio for blocked wood structural panel diaphragms, the aspect ratio limit of 2.5:1 is the maximum permitted and considered more suitable for a broad range of applications.

C4.1.5.1.1 While direct loading of wood framing in cross grain bending or cross grain tension is not associated with an allowable design stress and should therefore be avoided as part of a designed load path, transfer of anchorage forces through wood framing subject to cross grain bending and cross grain tension are specifically prohibited. Details commonly employed for transfer of anchorage forces into the diaphragm use mechanical attachment between the wall anchor and wood framing oriented perpendicular to the wall (Figure C4.1.5B) avoiding direct loading of wood framing in cross grain bending. Figure C4.1.5B illustrates a typical wall anchor attached to a sub-diaphragm roof purlin. Sheathing edge nailing into the purlin is provided as part of the load path between sheets of panel sheathing, but the sheathing is not considered part of the connection between the purlin and the ledger. Detailing of cross ties for tension relies on mechanical attachment of framing members (Figure C4.1.5C) avoiding direct loading of wood framing in cross grain tension.

Figure C4.1.5A Illustration of Subdiaphragm and Continuous Tie Concept (for west wall only)

(Note: For east west loading, subdiaphragm span is 40' between continuous ties and width is 40'. The ratio of L/W = 40'/40' = 1.00 is less than the 2.5:1 aspect ratio limit of 4.1.5.1).

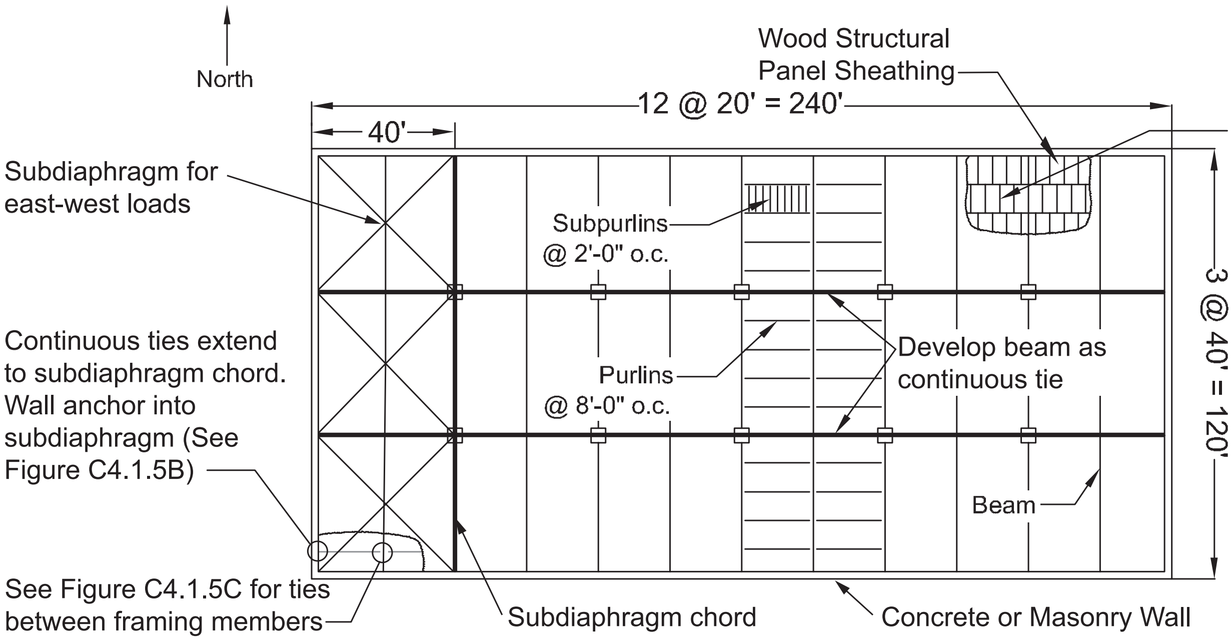
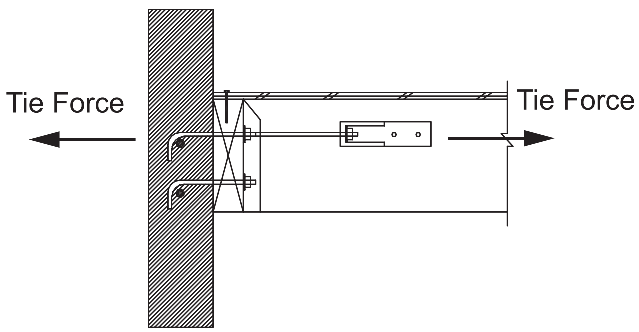
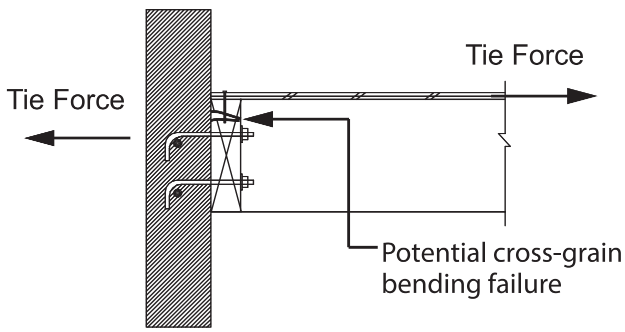


Figure C4.1.5B Example Details of Appropriate and Inappropriate Concrete and Masonry Wall Anchorage to Wood Diaphragms

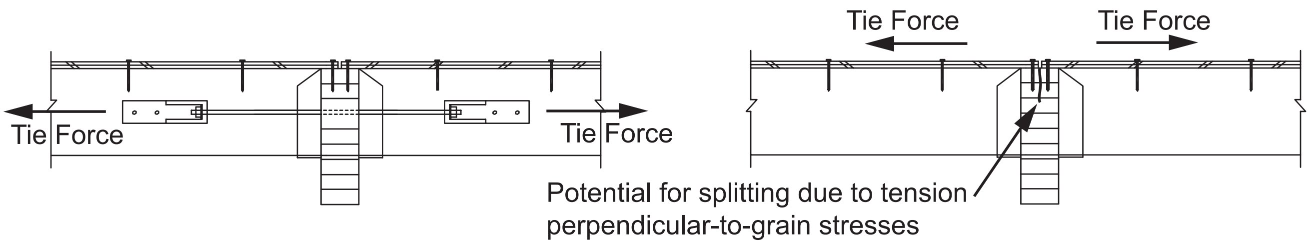


a) **Appropriate** wall anchor detail where anchor forces are transferred directly into diaphragm framing



b) **Inappropriate** wall anchor detail where anchor forces induce cross-grain bending in the wood ledger (not permitted)

Figure C4.1.5C Example Details of Appropriate and Inappropriate Continuity Tie Details



a) **Appropriate** continuity tie detail where continuity of wall anchor forces is maintained by ties between framing members

b) **Inappropriate** continuity tie detail where sheathing is used to maintain continuity of tension where tension perpendicular to grain stresses are induced in the wood beam (not permitted)

C4.1.5.1.2 Reliance on diaphragm sheathing to provide continuity between framing members is specifically prohibited. The intended design approach is to preserve sheathing and sheathing nailing for in-plane shear loading and to reduce potential for concentration of deformation at sheathing joints between chords. The limitation is also intended to prohibit the design of wood sheathing to provide tension continuity for wall anchorage forces (Figure C4.1.5C).

C4.1.6 Wood Members and Systems Resisting Seismic Forces from Other Concrete or Masonry Construction

Seismic forces from other concrete or masonry construction (i.e. other than walls) are permitted and should be accounted for in design. *SDPWS* 4.1.6 is not intended to restrict the use of concrete floors – including wood floors with concrete toppings as well as reinforced concrete slabs – or similar such elements in floor construction. It is intended to clarify that, where such elements are present in combination with a wood system, the wood system shall be designed to account for seismic forces generated by the additional mass of such elements.

Design of wood members to support the additional mass of concrete and masonry elements shall be in accordance with the *NDS* and required deflection limits as specified in concrete or masonry standards or model building codes (2). Masonry is defined as a built-up construction or combination of building units or materials of clay, shale, concrete, glass, gypsum, stone, or other approved units bonded together with or without mortar or grout or other accepted methods of joining.

C4.1.7 Toe-Nailed Connections

Limits on use of toe-nailed connections in seismic design categories D, E, and F for transfer of seismic forces is consistent with building code requirements (2). Test data (12) suggests that the toe-nailed connection limit on a bandjoist to wall plate connection may be too restrictive; however, an appropriate alternative limit requires further study. Where blocking is used to transfer high seismic forces, toe-nailed connections can sometimes split the block or provide a weakened plane for splitting.

C4.2 Wood Diaphragms

C4.2.1 Application Requirements

General requirements for wood diaphragms include consideration of diaphragm strength and deflection.

C4.2.2 Deflection

The total mid-span deflection of a blocked, uniformly nailed (e.g. same panel edge naling) wood structural panel diaphragm can be calculated by summing the effects of four sources of deflection: framing bending deflection, panel shear deflection, deflection from nail slip, and deflection due to chord splice slip:

(bending, chord deformation excluding slip) *(shear, panel deformation)* *(shear, panel nail slip)* *(bending, chord splice slip)*

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{vL}{4G_v t_v} + 0.188Le_n + \frac{\sum(x\Delta_c)}{2W} \quad \text{(C4.2.2-1)}$$

where:

ν = induced unit shear, plf

L = diaphragm dimension perpendicular to the direction of the applied force, ft

E = modulus of elasticity of diaphragm chords, psi

A = area of chord cross-section, in.²

W = width of diaphragm in direction of applied force, ft

$G_v t_v$ = shear stiffness, lb/in. of panel depth. See Table C4.2.2A or C4.2.2B.

x = distance from chord splice to nearest support, ft. For example, a shear wall aligned parallel to the loaded direction of the diaphragm would typically be considered a support.

Δ_c = diaphragm chord splice slip at the induced unit shear, in.

e_n = nail slip, in. See Table C4.2.2D.

Note: the 5/8 constant incorporates background derivations that cancel out the units of feet in the first term of the equation.

SDPWS Equation 4.2-1 is a simplification of Equation C4.2.2-1, using only three terms for calculation of the total mid-span diaphragm deflection:

(bending, chord deformation excluding slip) *(shear, panel shear and nail slip)* *(bending, chord splice slip)*

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum(x\Delta_c)}{2W} \quad \text{(C4.2.2-2)}$$

where:

ν = induced unit shear, plf

L = diaphragm dimension perpendicular to the direction of the applied force, ft

E = modulus of elasticity of diaphragm chords, psi

A = area of chord cross-section, in.²

W = width of diaphragm in direction of applied force, ft

G_a = apparent diaphragm shear stiffness, kips/in.

x = distance from chord splice to nearest support, ft

Δ_c = diaphragm chord splice slip at the induced unit shear, in.

Distribution of shear forces among shear panels in a diaphragm is a function of the layup and nailing pattern of panels to framing. For this reason, shear deflection in a wood diaphragm is related to panel shear, panel layout, nailing pattern, and nail load-slip relationship. In Equation C4.2.2-2, panel shear and nail slip are assumed to be inter-related and have been combined into a single term to account for shear deformations. Equation C4.2.2-3 equates apparent shear stiffness, G_a, to nail slip and panel shear stiffness terms used in the four-term equation:

$$G_a = \frac{1.4v_{s(ASD)}}{\frac{1.4v_{s(ASD)}}{G_v t_v} + 0.75e_n} \quad \text{(C4.2.2-3)}$$

where:

$1.4\ v_{s(ASD)}$ = 1.4 times the ASD unit shear capacity for seismic. The value of 1.4 converts ASD level forces to strength level forces.

Calculated deflection, using either the 4-term (Equation C4.2.2-1) or 3-term equation (*SDPWS* Equation 4.2-1), is identical at the critical strength design level — 1.4 times the allowable shear value for seismic (see Figure C4.3.2).

For unblocked wood structural panel diaphragms, tabulated values of G_a are based on limited test data for blocked and unblocked diaphragms (3, 4, and 11). For diaphragms of Case 1, reduced shear stiffness equal to $0.6G_a$ was used to derive tabulated G_a values. For unblocked diaphragms of Case 2, 3, 4, 5, and 6, reduced shear stiffness equal to $0.4G_a$ was used to derive tabulated G_a values. Examples C4.2.2-1 and C4.2.2-2 show derivations of G_a in *SDPWS* Tables 4.2A and 4.2B, respectively.

A factor of 0.5 is provided in the diaphragm table footnotes to adjust tabulated G_a values (based on fabricated dry condition) to approximate G_a where "green" framing is used. This factor is based on analysis of apparent shear stiffness for wood structural panel shear wall and diaphragm construction where:

1) framing moisture content is greater than 19% at time of fabrication (green), and
2) framing moisture content is less than or equal to 19% in-service (dry).

The average ratio of "green" to "dry" for G_a across shear wall and diaphragm cells ranged from approximately 0.52 to 0.55. A rounded value of 0.5 results in slightly greater values of calculated deflection for "green" framing when compared to the more detailed 4-term deflection equations. Although based on nail slip relationships applicable to wood structural panel shear walls, this reduction can also be extended to lumber sheathed diaphragm construction.

Comparison with Diaphragm Test Data

Tests of blocked and unblocked diaphragms (4) are compared in Table C4.2.2E for diaphragms constructed as follows:

Sheathing material = Sheathing Grade, 3/8" minimum nominal panel thickness
Nail size = 8d common (0.131" diameter, 2½" length)
Diaphragm length, L = 24 ft
Diaphragm width, W = 24 ft
Panel edge nail spacing = 6 in.
Boundary nail spacing = 6 in. o.c. at boundary parallel to load (4 in. o.c. at boundary perpendicular to load for walls A and B)

Calculated deflections at 1.4 x $\nu_{s(ASD)}$ closely match test data for blocked and unblocked diaphragms.

In Table C4.2.2F, calculated deflections using *SDPWS* Equation 4.2-1 are compared to deflections from two tests of 20 ft x 60 ft (W = 20 ft, L = 60 ft) diaphragms (26) at 1.4 times the allowable seismic design value for a horizontally sheathed and single diagonally sheathed lumber diaphragm. Calculated deflections include estimates of deflection due to bending, shear, and chord slip. For both diaphragms, calculated shear deformation accounted for nearly 85% of the total calculated mid-span deflection. Tested deflection for Diaphragm 4 is slightly greater than estimated by calculation and may be attributed to limited effectiveness of the diaphragm chord construction which utilized blocking to transfer forces to the double 2x6 top plate chord. For Diaphragm 2, chord construction utilized 2-2x10 band joists.

Table C4.2.2A Shear Stiffness, $G_v t_v$ (lb/in. of depth), for Wood Structural Panels

Span Rating[4]	Minimum Nominal Panel Thickness (in.)	Structural Sheathing				Structural I			
		Plywood			OSB	Plywood			OSB
		3-ply	4-ply	5-ply[3]		3-ply	4-ply	5-ply[3]	
Sheathing Grades[1]									
24/0	3/8[2]	25,000	32,500	37,500	77,500	32,500	42,500	41,500	77,500
24/16	7/16	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
32/16	15/32	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
40/20	19/32	28,500	37,000	43,000	88,500	37,000	48,000	47,500	88,500
48/24	23/32	31,000	40,500	46,500	96,000	40,500	52,500	51,000	96,000
Single Floor Grades									
16 oc	19/32	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
20 oc	19/32	28,000	36,500	42,000	87,000	36,500	47,500	46,000	87,000
24 oc	23/32	30,000	39,000	45,000	93,000	39,000	50,500	49,500	93,000
32 oc	7/8	36,000	47,000	54,000	110,000	47,000	61,000	59,500	110,000
48 oc	1-1/8	50,500	65,500	76,000	155,000	65,500	85,000	83,500	155,000

1. Sheathing grades used for calculating G_a values for diaphragm and shear wall tables.
2. $G_v t_v$ values for 3/8" panels with span rating of 24/0 used to estimate G_a values for 5/16" panels.
3. 5-ply applies to plywood with five or more layers. For 5-ply plywood with three layers, use $G_v t_v$ values for 4-ply panels.
4. See Table 4.2.2C for relationship between span rating and nominal panel thickness.

Table C4.2.2B Shear Stiffness, $G_v t_v$ (lb/in. of depth), for Other Sheathing Materials

Sheathing Material	Minimum Nominal Panel Thickness (in.)	$G_v t_v$
Plywood Siding	5/16 & 3/8	25,000
Particleboard	3/8	25,000
	1/2	28,000
	5/8	28,500
Structural Fiberboard	1/2 & 25/32	25,000
Gypsum board	1/2 & 5/8	40,000
Lumber	All	25,000

Table C4.2.2C Relationship Between Span Rating and Nominal Thickness

Span Rating	Nominal Thickness (in.)										
	3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	7/8	1	1-1/8
Sheathing											
24/0	**P**	A	A	A							
24/16		**P**	A	A							
32/16			**P**	A	A	A					
40/20					**P**	A	A	A			
48/24							**P**	A	A		
Single Floor Grade											
16 oc					**P**	A					
20 oc					**P**	A					
24 oc							**P**	A			
32 oc									**P**	A	
48 oc											**P**

P = Predominant nominal thickness for each span rating.
A = Alternative nominal thickness that may be available for each span rating. Check with suppliers regarding availability.

Table C4.2.2D Fastener Slip, e_n (in.)

Sheathing	Fastener Size	Maximum Fastener Load (V_n) (lb/fastener)	Fastener Slip, e_n (in.)	
			Fabricated w/green (>19% m.c.) lumber	Fabricated w/dry (≤ 19% m.c.) lumber
Wood Structural Panel (WSP) or Particleboard[1]	6d common	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
	8d common	220	$(V_n/857)^{1.869}$	$(V_n/616)^{3.018}$
	10d common	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$
Structural Fiberboard	All	-	-	0.07
Gypsum Board	All	-	-	0.03
Lumber	All	-	-	0.07

1. Slip values are based on plywood and OSB fastened to lumber with a specific gravity of 0.50 or greater. The slip shall be increased by 20 percent when plywood or OSB is not Structural I. Nail slip for common nails have been extended to galvanized box or galvanized casing nails of equivalent penny weight for purposes of calculating G_a.

Table C4.2.2E Data Summary for Blocked and Unblocked Wood Structural Panel Diaphragms

Wall	Blocked/ Unblocked	$1.4v_{s(ASD)}$ (plf)	Actual Deflection, (in.)	Apparent Stiffness[1], G_a, (kips/in.)	Calculated Deflection, (in.)	Diaphragm Layout
A	Blocked	378	0.22	14.4	0.18	Case 1
D	Unblocked	336	0.26	(0.60 x 14.4) = 8.6	0.26	Case 1
B	Blocked	378	0.15	14.4	0.18	Case 3
E	Unblocked	252	0.23	(0.40 x 14.4) = 5.8	0.29	Case 3

1. Values of G_a for the blocked diaphragm case were taken from *SDPWS* Table 4.2A and multiplied by 1.2 (see footnote 3) because sheathing material was assumed to be comparable to 4/5-ply construction.

Table C4.2.2F Data Summary for Horizontal Lumber and Diagonal Lumber Sheathed Diaphragms

Diaphragm	Description	Calculated			Actual
		$1.4v_{s(ASD)}$ (plf)	G_a (kips/in.)	δ[1] (in.)	δ (in.)
Diaphragm 4	Horizontal Lumber Sheathing – Dry Lumber Sheathing – 2 x 6 chord (double top plates), 5 splices	70	1.5	0.81	0.93
Diaphragm 2	Diagonal Lumber Sheathing – Green Lumber Sheathing – 2 x 10 chord, 3 splices – Exposed outdoors for 1 month	420	6.0	1.23	1.05

1. Calculated deflection equal to 0.81" includes estimates of deflection due to bending, shear, and chord slip (0.036" + 0.7" + 0.07" = 0.81"). Calculated deflection equal to 1.23" includes estimates of deflection due to bending, shear, and chord slip (0.13" + 1.05" + 0.05" = 1.23").

EXAMPLE C4.2.2-1 Derive G_a in SDPWS Table 4.2A

Derive G_a in *SDPWS* Table 4.2A for a blocked wood structural panel diaphragm constructed as follows:

Sheathing grade	= Structural I (OSB)
Sheathing layup	= Case 1
Nail size	= 6d common (0.113" diameter, 2" length)
Minimum nominal panel thickness	= 5/16 in.
Boundary and panel edge nail spacing	= 6 in.
Minimum width of nailed face	= 2x nominal
Nominal unit shear capacity for seismic, ν_s	= 370 plf *SDPWS* Table 4.2A

Allowable unit shear capacity for seismic:

$\nu_{s(ASD)}$ = 370 plf/2 = 185 plf

Panel shear stiffness:

$G_\nu t_\nu$ = 77,500 lb/in. of panel depth Table C4.2.2A

Nail load/slip at 1.4 $\nu_{s(ASD)}$:

V_n = fastener load (lb/nail)
= 1.4 $\nu_{s(ASD)}$ (6 in.)/(12 in.)
= 129.5 lb/nail

$e_n = (V_n/456)^{3.144}$ Table C4.2.2D
$= (129.5/456)^{3.144} = 0.0191$ in.

Calculate G_a:

$$G_a = \frac{1.4v_{s(ASD)}}{\dfrac{1.4v_{s(ASD)}}{G_v t_v} + 0.75e_n} \qquad \text{(C4.2.2-3)}$$

G_a = 14,660 lb/in. ≈ 15 kips/in. *SDPWS* Table 4.2A

EXAMPLE C4.2.2-2 Derive G_a in SDPWS Table 4.2B

Derive G_a in *SDPWS* Table 4.2B for an unblocked wood structural panel diaphragm constructed as follows:

Sheathing grade	= Structural I (OSB)
Nail size	= 6d common (0.113" diameter, 2" length)
Minimum nominal panel thickness	= 5/16 in.
Minimum width of nailed face	= 2x nominal
Boundary and panel edge nail spacing	= 6 in.

G_a = 15 kips/in. *SDPWS* Table 4.2A

Case 1 - unblocked

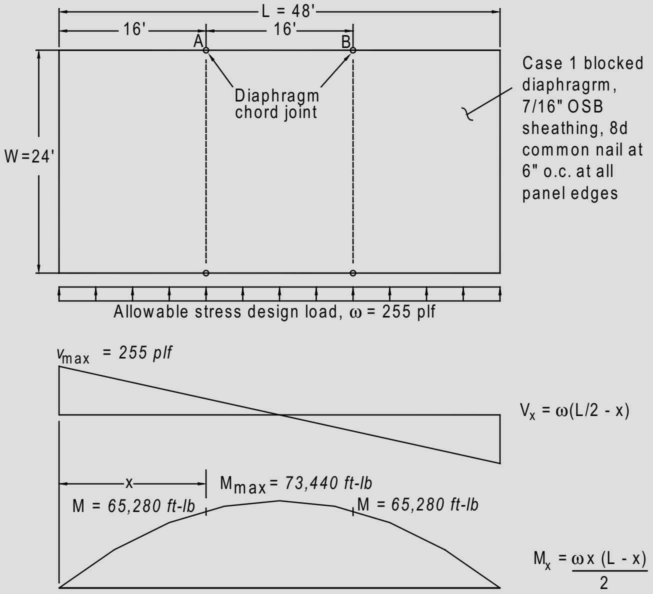
G_a = 0.6 G_a (blocked)
= 0.6 (15.0) = 9.0 kips/in. *SDPWS* Table 4.2B

Cases 2, 3, 4, 5, and 6 - unblocked

G_a = 0.4 G_a (blocked)
= 0.4 (15.0) = 6.0 kips/in. *SDPWS* Table 4.2B

EXAMPLE C4.2.2-3 Calculate Mid-Span Diaphragm Deflection

Figure C4.2.2A Diaphragm Dimensions and Shear and Moment Diagram



Calculate mid-span deflection for the blocked wood structural panel diaphragm shown in Figure C4.2.2A. The diaphragm chord splice is sized using allowable stress design loads from seismic while deflection due to seismic is based on strength design loads in accordance with *ASCE 7*.

Diaphragm apparent shear stiffness, G_a:

G_a = 14 kips/in. (*SDPWS* Table 4.2A)

Diaphragm allowable unit shear capacity for seismic, $v_{s\,(ASD)}$:

$v_{s\,(ASD)}$ = 255 plf (*SDPWS* Table 4.2A)

Diaphragm chord:

Two 2x6 No. 2 Douglas Fir-Larch, E = 1,600,000 psi, and G = 0.50

Part 1 - Calculate the number of 16d common nails in the chord splice

For each chord, one top plate is designed to resist induced axial force (tension or compression) while the second top plate is designed as a splice plate (see Figure C4.2.2B). The connection at the chord splice consists of 16d common nails (0.162" diameter x 3-1/2" length). The allowable design value for a single 16d common nail in a face-nailed connection is: Z'_{ASD} = 226 lb.

The axial force (T or C) at each joint:

$$(T\ or\ C) = \frac{M_x}{W} = \frac{65{,}280\,ft-lb}{24\ ft} = 2{,}720\ lb$$

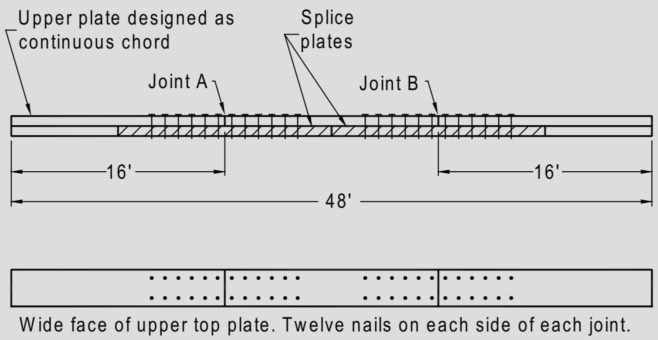
The number of 16d common nails, n, is:

$$n = \frac{2{,}720\ lb}{226\ lb/nail} = 12\ nails$$

Use twelve 16d common nails on each side of joint A and joint B to transfer chord axial forces. Designers should consider whether a single maximum chord force at mid-span of the diaphragm should be used to determine the number of fasteners in each splice joint since the actual location of joints may not be known. The number of 16d common nails based on the maximum chord force at mid-span of the diaphragm is:

$$n = \frac{73{,}440\ ft-lb/24\ ft}{226\ lb/nail} = 14\ nails$$

Figure C4.2.2B Diaphragm Chord, Double Top Plate with Two Joints in Upper Plate



Part 2 - Calculated mid-span deflection

ASCE 7 requires that seismic story drift be determined using strength level design loads; therefore, induced unit shears and chord forces used in terms 1, 2, and 3 of the deflection equation are calculated using strength level

(continued)

EXAMPLE C4.2.2-3 Calculate Mid-Span Diaphragm Deflection (continued)

design loads. Strength level design loads can be estimated by multiplying the allowable stress design seismic loads, shown in Figure C4.2.2A, by 1.4.

A spliced chord member has an "effective" stiffness (EA) due to the splice slip that occurs throughout the chord. In this example, and for typical applications of Equation C4.2.2-2, the effect of the spliced chord on mid-span deflection is addressed by independently considering deflection from: a) chord deformation due to elongation or shortening assuming a continuous chord member per deflection equation Term 1, and b) deformations due to chord splice slip at chord joints per deflection equation Term 3.

Diaphragm deflection is calculated in accordance with the following:

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum(x\Delta_c)}{2W} \qquad (\textit{SDPWS}\ \text{C4.2.2-2})$$

Term 1. Deflection due to bending and chord deformation (excluding chord splice slip):

$$\delta_{dia(bending,\ chords)} = \frac{5vL^3}{8EAW}$$
$$= \frac{5(1.4\ x\ 255\ plf)(48\ ft)^3}{8(1{,}600{,}000\,psi)(8.25\,in.^2)(24\,ft)}$$
$$= 0.078\,in.$$

where:

v = 1.4 x 255 plf, induced unit shear due to strength level seismic load

L = 48 ft, diaphragm length

W = 24 ft, diaphragm width

E = 1,600,000 psi, modulus of elasticity of the 2x6 chord member ignoring effects of chord splice slip. The effect of chord splice slip on chord deformation is addressed in deflection equation Term 3.

A = 8.25 in.^{2}, cross sectional area of one 2x6 top plate designed to resist axial forces.

The second top plate is designed as a splice plate.

Term 2. Deflection due to shear, panel shear, and nail slip:

$$\delta_{dia(panel\ shear+nail\ slip)} = \frac{0.25vL}{1000G_a}$$
$$= \frac{0.25(1.4\ x\ 255\ plf)(48\ ft)}{1000(14\ kips\,/\,in.)}$$
$$= 0.306\ in.$$

where:

G_a = 14 kips/in., apparent shear stiffness (*SDPWS* Table 4.2A)

Term 3. Deflection due to bending and chord splice slip:

$$\delta_{dia(chord\ splice\ slip)} = \frac{\sum(x\Delta_c)}{2W}$$

where:

x = 16 ft, distance from the joint to the nearest support (see Figure C4.2.2A). Each joint is located 16 ft from the nearest support.

Δ_c = Joint deformation (in.) due to chord splice slip in each joint. The chord force, T or C, at each joint is:

$$(T\ or\ C) = \frac{(1.4\ \ x\ \ 65{,}280\ ft-lb)}{24\ ft} = 3{,}808\,lb$$

The slip, Δ, associated with each joint:

$$\Delta_c = \frac{2(T\ or\ C)}{\gamma\ n}$$
$$= \frac{2(3{,}808\,lb)}{11{,}737\,lb\,/\,in.\,/\,nail\ (12\ nails)}$$
$$= 0.054\,in.$$

(continued)

EXAMPLE C4.2.2-3 Calculate Mid-Span Diaphragm Deflection (continued)

where:

γ = 11,737 lb/in./nail, load slip modulus for dowel type fasteners determined in accordance with *National Design Specification for Wood Construction* (NDS) Section 11.3.6, $\gamma = 180{,}000\ D^{1.5}$.

(Note: A constant of 2 is used in the numerator to account for slip in nailed splices on each side of the joint.)

Deflection due to tension chord splice slip is:

$$\delta_{dia(tension\ chord\ splice\ slip)} = \frac{\sum(16\ ft \times 0.054\ in.) + (16\ ft \times 0.054\ in.)}{2(24\ ft)}$$

$$= 0.036\ in.$$

Assuming butt joints in the compression chord are not tight and have a gap that exceeds the splice slip, the tension chord slip calculation is also applicable to the compression chord:

$$\delta_{dia(compression\ chord\ splice\ slip)} = 0.036\ in.$$

Total deflection due to chord splice slip is:

$$\delta_{dia(chord\ splice\ slip)} = 0.036\ in. + 0.036\ in. = 0.072\ in.$$

Total mid-span deflection:

Summing deflection components from deflection equation Term 1, Term 2, and Term 3 results in the following total diaphragm mid-span deflection:

$$\delta_{dia} = 0.078\ in. + 0.306\ in. + 0.072\ in. = 0.456\ in.$$

C4.2.3 Unit Shear Capacities

ASD and LRFD unit shear capacities for wind and seismic are calculated as follows from nominal values for wind, ν_w, and seismic, ν_s.

ASD unit shear capacity for wind, $\nu_{w(ASD)}$:

$$v_{w(ASD)} = \frac{v_w}{2.0} \quad \text{(C4.2.3-1)}$$

ASD unit shear capacity for seismic, $\nu_{s(ASD)}$:

$$v_{s(ASD)} = \frac{v_s}{2.0} \quad \text{(C4.2.3-2)}$$

where:

2.0 = ASD reduction factor

LRFD unit shear capacity for wind, $\nu_{w(LRFD)}$:

$$v_{w(LRFD)} = 0.8 v_w \quad \text{(C4.2.3-3)}$$

LRFD unit shear capacity for seismic, $\nu_{s(LRFD)}$:

$$v_{s(LRFD)} = 0.8 v_s \quad \text{(C4.2.3-4)}$$

where:

0.8 = resistance factor, ϕ_D, for shear walls and diaphragms

C4.2.4 Diaphragm Aspect Ratios

Maximum aspect ratios for floor and roof diaphragms (*SDPWS* Table 4.2.4) using wood structural panel or diagonal board sheathing are based on building code requirements (See *SDPWS* 4.2.5.1 for aspect ratio limits for cases where a torsional irregularity exists, for open front buildings, and cantilevered diaphragms).

C4.2.5 Horizontal Distribution of Shear

Seismic design requirements of ASCE 7 (5) classify diaphragms as semi-rigid, idealized as flexible, and idealized as rigid. The significance of "idealized" is to recognize that wood diaphragms always have some rigidity and are neither truly flexible nor truly rigid but can be idealized as such where certain conditions are met. An idealization is employed to simplify structural analysis for distribution of horizontal diaphragm shear loads. For diaphragms idealized as flexible, loads are distributed to vertical resisting elements (e.g. shear walls) according to tributary area, whereas for diaphragms idealized as rigid, loads are distributed according to the relative stiffnesses of the vertical resisting elements. For diaphragms modeled as semi-rigid, the distribution of horizontal diaphragm shear loads to shear wall lines employs a more complex analysis dependent on the relative stiffness of the diaphragm and vertical resisting elements.

The use of semi-rigid diaphragm modeling for purposes of distribution of horizontal force is always permissible under ASCE 7. It is the method considered to most rationally account for actual distribution of horizontal diaphragm shear loads to vertical resisting elements; however, a semi-rigid diaphragm analysis requires significant calculation effort for all but the simplest box structures. An acceptable alternative to semi-rigid diaphragm analysis is the envelope analysis where distribution of horizontal diaphragm shear to each vertical resisting element is the larger of the shear forces resulting from analyses where the diaphragm is idealized as flexible and the diaphragm is idealized as rigid. While two separate analyses must be performed, one for the diaphragm idealized as flexible and one for the diaphragm idealized as rigid, the envelope analysis avoids calculation effort associated with distribution of horizontal diaphragm shear loads based on relative stiffness of the diaphragm and vertical resisting elements.

Consistent with prior editions of *SDPWS*, the idealized as rigid diaphragm condition is permitted only where computed maximum in-plane deflection of the diaphragm itself under lateral load is less than or equal to two times the average deflection of adjoining vertical elements of the lateral force-resisting system of the associated story under equivalent tributary lateral load. This requirement is considered an appropriate corollary to the calculation-based “idealized as flexible diaphragm” condition of ASCE 7. Wood diaphragms used with wood frame shear walls will often calculate as meeting the idealized as rigid condition; however, in most cases wood diaphragms are idealized as flexible in accordance with the ASCE 7 prescriptive conditions allowing the idealized as flexible analysis assumption. Importantly, the ASCE 7 prescriptive condition for light frame construction requires that the allowable story drift be satisfied for each line of vertical resisting elements.

C4.2.5.1 Torsional Irregularity: Excessive torsional response of a structure can be a potential cause of failure. For structures identified as torsionally irregular, special requirements must be met to improve diaphragm rigidity, including limits on diaphragm materials, diaphragm aspect ratio, and drift. The test for torsional irregularity is from ASCE 7 and applicable for diaphragms idealized as rigid or modeled as semi-rigid. For diaphragms idealized as flexible, where distribution is by tributary area, requirements of this section are not applicable because such structures are not considered to be torsionally irregular under ASCE 7.

C4.2.5.2 Open Front Structures: While 4.2.5.2 introduces requirements specific to wood diaphragms in open front structures, these are in addition to and not a replacement of general seismic design criteria of ASCE 7. A defining characteristic of an open front structure is the presence of a cantilevered diaphragm for transfer of forces to vertical elements of the lateral force-resisting system. A structure with shear walls on three sides only (open front) is one simple form of an open front structure. Open front structures rely on diaphragm rigidity for transfer of forces through diaphragm rotation and are considered to be more vulnerable to torsional response than other box type structure configurations due to reliance on the diaphragm for torsional force distribution to elements that are not optimally located at diaphragm edges. As a result, open front structure provisions require limitations on seismic drift and building configuration similar to provisions for torsionally irregular structures that are not open front for the purpose of reducing the likelihood of an unacceptable torsional response of such structures.

For loading parallel to the open side, the requirement to check drift at building edges applies regardless of the analysis method used (e.g. diaphragm idealized as rigid, semi-rigid analysis, or envelope analysis) and regardless of whether the test for torsional irregularity indicates presence of a torsional irregularity. The required use of either idealized as rigid or semi-rigid analysis for loading parallel to the open front is due to the reliance on diaphragm rigidity for transfer of torsional forces. Required use of these analysis methods is also intended to ensure that torsional irregularity and associated requirements for torsion in ASCE 7 are invoked as applicable for this structure type. There are no specific requirements for diaphragm rigidity for loading perpendicular to the open front. For example, it is not required for diaphragms to be modeled as semi-rigid or idealized as rigid for loading perpendicular to the open front. Open front building plans often have regular placement of walls aligned perpendicular to the open front that is conducive to the idealized as flexible analysis method for loading perpendicular to the open side.

Example C4.2.5.2 illustrates components of deflection contributing seismic drift at edges for a simple cantilevered diaphragm structure. The cantilevered diaphragm structure depicted in Figure C4.2.5A consists of a cantilevered diaphragm of length L′, transverse resisting shear walls of length L′, and longitudinal shear wall of length W′. The seismic force is uniformly distributed over cantilever diaphragm length, L′. Center of rigidity, CR, is located at the mid-point of the longitudinal shear wall and center of mass, CM, is located at the geometric center of diaphragm dimensions L′xW′. Components of diaphragm deflection contributing to seismic drift at edges consist of translation, rotation, and diaphragm shear including flexural deformations as depicted in Figure C4.2.5B. An important aspect of the example is that flexural and shear deformations of the diaphragm are to be included in the required check of drift at diaphragm edges. This deformation of the diaphragm is to be included whether the diaphragm is idealized as rigid or modeled as semi-rigid.

The exception in *SDPWS* 4.2.5.2 excludes relatively small diaphragm cantilevers from open front criteria. Small diaphragm cantilevers, with L′ of six feet or less, are often present and the complexity and limitations associated with open front criteria was not judged to be warranted. While such small diaphragm cantilevers are exempt from open front requirements of 4.2.5.2, general seismic design criteria of ASCE 7 remain applicable as well as provisions of 4.2.5.1 for cases where a torsional irregularity is present.

Example C4.2.5.2 Illustrate Components of Deflection Contributing to Seismic Story Drift at Edges for a Simple Cantilever Diaphragm Structure

Figure C4.2.5A Simple Cantilever Diaphragm Structure

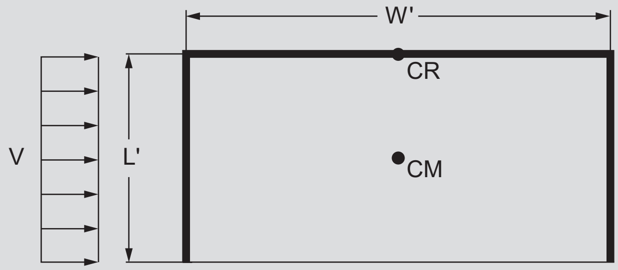
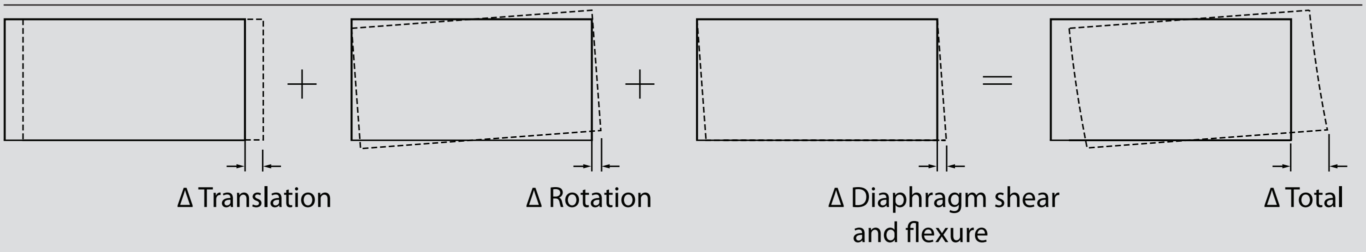


Figure C4.2.5B Components of Diaphragm Deflection Contributing to Seismic Story Drift at Edges



Discussion: Components of diaphragm deflection contributing to story drift consists of translation, rotation, and diaphragm shear and flexural deformations as depicted in Figure C4.2.5B.

- The translation component of deflection is from deflection of the longitudinal shear wall W′ under the applied shear loading.
- The rotation component of deflection is from deflection of transverse shear walls, L′, under torsional moment caused by eccentric loading.
- The diaphragm shear and flexure component of deflection is from the diaphragm itself under the assumed distributed shear loading and includes in-plane shear and flexural deflection of the diaphragm.

Consistent with ASCE 7 requirements, when checking design story drift at edges against ASCE 7 allowable story drift limits, it is necessary to multiply deflection determined using strength level forces by the applicable deflection amplification factor (i.e. Δ Total calculated at strength level forces is multiplied by C_d to estimate design story drift). Where vertical elements of the seismic force resisting system are wood frame wood structural panel shear walls in a bearing wall system, the applicable deflection amplification factor, C_d, is equal to 4.

C4.2.5.2.1 Simplification of open front criteria was judged appropriate for relatively small one story structures where L′ is not more than 25 feet and L′/W′ is less than or equal to 1:1. In such structures, the diaphragm is permitted to be idealized as rigid for distribution of horizontal shear forces regardless of whether the diaphragm meets the calculation based definition of idealized as rigid in *SDPWS* 4.2.5. Other requirements for cantilever diaphragms remain applicable including provisions of 4.2.5.1 for cases where a torsional irregularity is present as well as general seismic design criteria of ASCE 7.

C4.2.6 Construction Requirements

C4.2.6.1 Framing Requirements: The transfer of forces into and out of diaphragms is required for a continuous load path. Boundary elements must be sized and connected to the diaphragm to ensure force transfer. This section provides basic framing requirements for boundary elements in diaphragms. Good construction practice and efficient design and detailing for boundary elements utilize framing members in the plane of the diaphragm or tangent to the plane of the diaphragm (See C4.1.4). Where splices occur in boundary elements, transfer of force between boundary elements should be through the addition of framing members or metal connectors. The use of diaphragm sheathing to splice boundary elements is not permitted.

C4.2.6.2 Sheathing: Sheathing types for diaphragms included in *SDPWS* Table 4.2A and Table 4.2B are categorized in terms of the following structural use panel grades: Structural I, Sheathing, and Single-Floor. Sheathing grades rated for subfloor, roof, and wall use are usually unsanded and are manufactured with exterior glue. The Structural I sheathing grade is used where the greatest available shear and cross-panel strength properties are required. Structural I is made with exterior glue only. The Single-Floor sheathing grade is rated for use as a combination of subfloor and underlayment, usually with tongue and groove edges, and has sanded or touch sanded faces.

SDPWS Table 4.2A and Table 4.2B are applicable to both oriented strand board (OSB) and plywood. While strength properties between equivalent grades and thickness of OSB and plywood are the same, shear stiffness of OSB is greater than that of plywood of equivalent grade and thickness.

Tabulated plywood G_a values are based on 3-ply plywood. Separate values of G_a for 4-ply, 5-ply, and composite panels were calculated and ratios of these values to G_a based on 3-ply were shown to be in the order of 1.09 to 1.22 for shear walls and 1.04 to 1.16 for diaphragms. A single G_a multiplier of 1.2 was chosen for 4-ply, and 5-ply in table footnotes. This option was considered preferable to tabulating G_a values for 3-ply, 4-ply, 5-ply, and composite panels separately.

C4.2.6.3 Fasteners: Adhesive attachment in diaphragms can only be used in combination with fasteners. Details on type, size, and spacing of mechanical fasteners used for typical floor, roof, and ceiling diaphragm assemblies are provided in Tables 4.2A, 4.2B, 4.2C, and 4.2D and in *SDPWS* 4.2.7 Diaphragm Assemblies.

C4.2.7 Diaphragm Assemblies

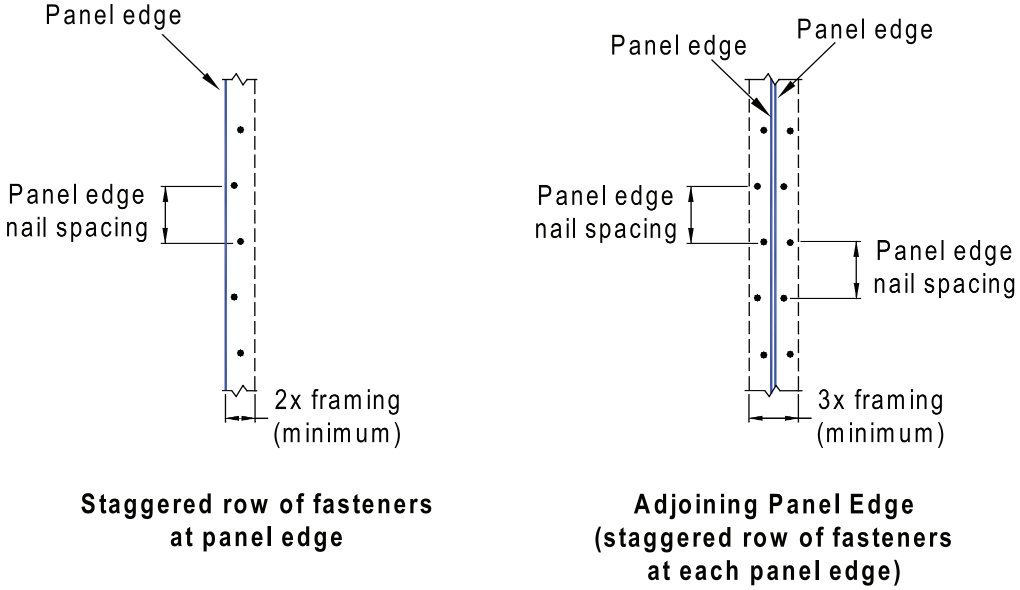
C4.2.7.1 Wood Structural Panel Diaphragms: Where wood structural panel sheathing is applied to solid lumber planking or laminated decking – such as in a retrofit or new construction where wood structural panel diaphragm capacities are desired – additional fastening, aspect ratio limits, and other requirements are prescribed to develop diaphragm capacity and transfer forces to boundary elements.

C4.2.7.1.1 Blocked Diaphragms: Standard construction of wood structural panel diaphragms requires use of full size sheets, not less than 4'x8' except at changes in framing where smaller pieces may be needed to cover the roof or floor. Panel edges must be supported by and fastened to framing members or blocking. The 24" width limit coincides with the minimum width where panel strength capacities for bending and axial tension are applicable (6). For widths less than 24", capacities for bending and axial tension should be reduced in accordance with applicable panel size adjustment factors (panel width adjustment factors are described in the Commentary to the *National Design Specification for Wood Construction* (6)). Apparent shear stiffness values provided in *SDPWS* Table 4.2A are based on standard assumptions for panel shear stiffness for oriented strand board (OSB), plywood, and nail load slip (see C4.2.2).

In accordance with *SDPWS* Table 4.2A, nail spacing requirements for a given unit shear capacity vary by the direction of continuous panel joints with respect to a) the loading direction and b)direction of framing members. The six possible cases are depicted in *SDPWS* Table 4.2A and are grouped as Cases 1 and 3, Cases 2 and 4, and Cases 5 and 6 to show that diaphragm resistance is independent of panel orientation.

C4.2.7.1.1(3) For closely spaced or larger diameter nails, staggered nail placement at each panel edge is intended to prevent splitting in the framing member (see Figure C4.2.7).

Figure C4.2.7 Staggering of Nails at Panel Edges of Blocked Diaphragms



C4.2.7.1.2 High Load Blocked Diaphragms: Provisions for wood structural panel blocked diaphragms with multiple rows of fasteners, also known as "high load diaphragms" are consistent with provisions in the *2006 International Building Code (IBC)* and the *2003 National Earthquake Hazard Reduction Program (NEHRP) Provisions.* Tests of nailed plywood-lumber joints (32) closely match recommended nailing patterns and verify calculations of unit shear associated with multiple rows of 10d common wire nails in Table 4.2B. The high load diaphragm table specifies use of framing with a minimum 3" or 4" nominal width for the nailed face and a minimum 3" nominal depth at adjoining panel edges and boundaries to provide adequate edge distance and penetration depth for multiple rows of 10d common wire nails at these locations (see *SDPWS* Figure 4B). These requirements are important to limit splitting associated with the specified nailing required for high load blocked diaphragms. Fastener spacing per line is listed in Table 4.2B as well as number of lines of fasteners. Nails should not be located closer than 3/8" from panel edges. Where the nominal width of nailed face and nail schedule permits greater panel edge distance, a 1/2" minimum distance from adjoining panel edges is specified. Apparent shear stiffness values are tabulated for each combination of nailing and sheathing thickness consistent with the format of tabulating apparent shear stiffness, G_a, for typical blocked and unblocked diaphragms.

SDPWS Figure 4B depicts a 1/8" minimum gap between adjoining panel edges to allow for dimensional change of the panel. In general, 4′x8′ panels will increase slightly in dimension due to increased moisture content in-use relative to moisture content immediately following manufacture. In some cases, due to exposure conditions following manufacture, the expected increase in panel dimensions is smaller than anticipated by the 1/8" minimum gap and therefore the gap at time of installation may be less than 1/8" minimum. Dimensional change and recommendations for installation can vary by product and manufacturer, therefore recommendations of the manufacturer for the specific product should be followed.

C4.2.7.1.3 Unblocked Diaphragms: Standard construction of unblocked wood structural panel diaphragms requires use of full size sheets, not less than 4′x8′ except at changes in framing where smaller sections may be needed to cover the roof or floor. Unblocked panel widths are limited to 24" or wider. Where smaller widths are used, panel edges must be supported by and fastened to framing members or blocking. The 24" width limit coincides with the minimum width where panel strength capacities for bending and axial tension are applicable (6). For widths less than 24", capacities for bending and axial tension should be reduced in accordance with applicable panel size factors, C_s, in the *National Design Specification (NDS) for Wood Construction* (6). Apparent shear stiffness values provided in *SDPWS* Table 4.2C are based on standard assumptions for panel shear stiffness for oriented strand board (OSB), plywood, and nail load slip (see C4.2.2).

C4.2.7.2 Diaphragms Diagonally Sheathed with Single Layer of Lumber: Single diagonally sheathed lumber diaphragms have comparable strength and stiffness to many wood structural panel diaphragm systems. Apparent shear stiffness in *SDPWS* Table 4.2D is based on assumptions of relative stiffness and nail slip (see C4.2.2).

C4.2.7.3 Diaphragms Diagonally Sheathed with Double-Layer of Lumber: Double diagonally sheathed lumber diaphragms have comparable strength and stiffness to many wood structural panel diaphragm systems. Apparent shear stiffness in *SDPWS* Table 4.2D is based on assumptions of relative stiffness and nail slip (see C4.2.2).

C4.2.7.4 Diaphragms Horizontally Sheathed with Single-Layer of Lumber: Horizontally sheathed lumber diaphragms have low strength and stiffness when compared to those provided by wood structural panel diaphragms and diagonally sheathed lumber diaphragms of the same overall dimensions. In new and existing construction, added strength and stiffness can be developed through attachment of wood structural panels over horizontally sheathed lumber diaphragms (see *SDPWS* 4.2.7.1). Apparent shear stiffness in *SDPWS* Table 4.2D is based on assumptions of relative stiffness and nail slip (see C4.2.2).

C4.3 Wood Shear Walls

C4.3.1 Application Requirements

General requirements for wood shear walls include consideration of shear wall deflection (discussed in 4.3.2) and strength (discussed in 4.3.3), and are predicated on using products that are in compliance with the product standards referenced in this Specification.

Shear wall performance has been evaluated by monotonic and cyclic testing and references to test reports are provided throughout the Commentary. Cyclic testing in accordance with ASTM E 2126 (34) Method C is commonly used to study seismic performance of wood frame shear wall behavior (14, 22, 25, 29, and 33). The cyclic loading protocol associated with ASTM E 2126 Method C is also known as the "CUREE" protocol (37). Reports containing results (15, 28, and 36) from other cyclic protocol, such as ASTM E 2126 Method A and Method B, commonly referred to as the "SEAoSC" and "ISO" protocols respectively, are also included as references for seismic design provisions of the *SDPWS*.

C4.3.2 Deflection

The deflection of a shear wall can be calculated by summing the effects of four sources of deflection: framing bending deflection, panel shear deflection, deflection from nail slip, and deflection due to wall anchorage slip:

$$\delta_{SW} = \underbrace{\frac{8vh^3}{EAb}}_{(bending)} + \underbrace{\frac{vh}{G_v t_v}}_{(shear)} + \underbrace{0.75he_n}_{(nail\ slip)} + \underbrace{\frac{h}{b}\Delta_a}_{(wall\ anchorage\ slip)} \quad \text{(C4.3.2-1)}$$

where:

ν = induced unit shear, plf

h = shear wall height, ft

E = modulus of elasticity of end posts, psi

A = area of end posts cross-section, in.^{2}

b = shear wall length, ft

$G_v t_v$ = shear stiffness, lb/in. of panel depth. See Table C4.2.2A or C4.2.2B.

Δ_a = total vertical elongation of wall anchorage system (including fastener slip, device elongation, rod elongation, etc.) at the induced unit shear in the shear wall, in.

e_n = nail slip, in. See Table C4.2.2D.

Note: the constant 8 in the first term and the constant 0.75 in the third term incorporate background derivations that cancel out the units of feet in each term.

SDPWS Equation 4.3-1 is a simplification of Equation C4.3.2-1, using only three terms for calculation of shear wall deflection:

$$\delta_{sw} = \underbrace{\frac{8vh^3}{EAb}}_{(bending)} + \underbrace{\frac{vh}{1000G_a}}_{(shear)} + \underbrace{\frac{h}{b}\Delta_a}_{(wall\ anchorage\ slip)} \quad \text{(C4.3.2-2)}$$

where:

ν = induced unit shear, plf

h = shear wall height, ft

E = modulus of elasticity of end posts, psi

A = area of end post cross-section, in.^{2}

b = shear wall length, ft

G_a = apparent shear wall shear stiffness, kips/in.

Δ_a = total vertical elongation of wall anchorage system (including fastener slip, device elongation, rod elongation, etc.) at the induced unit shear in the shear wall, in.

In *SDPWS* Equation 4.3-1, deflection due to panel shear and nail slip are accounted for by a single apparent shear stiffness term, G_a. Calculated deflection, using either the 4-term (Equation C4.3.2-1) or 3-term equation (*SDPWS* Equation 4.3-1), are identical at 1.4 times the allowable shear value for seismic (see Figure C4.3.2). Below 1.4 times the allowable shear value for seismic, calculation using the 3-term equation overestimates deflection relative to the 4-term equation but are generally negligible for design purposes. These small differences, however, can influence load distribution assumptions based on relative stiffness if both deflection calculation methods are used in a design. For consistency and to minimize calculation-based differences, either the 4-term equation or 3-term equation should be used.

Each term of the 3-term deflection equation accounts for independent deflection components that contribute to overall shear wall deflection. For example, apparent shear stiffness is intended to represent only the shear component of deflection and does not also attempt to account for bending or wall anchorage slip. In many cases, such as for gypsum wallboard shear walls and fiberboard shear walls, results from prior testing (17 and 23) used to verify apparent shear stiffness estimates were based on ASTM E 72 (41) where effect of bending and wall anchorage slip are minimized due to the presence of metal tie-down rods in the standard test set-up. The relative contribution of each of the deflection components will vary by aspect ratio of the shear wall. For other than narrow shear walls, deformation due to shear deformation (combined effect of nail slip and panel shear deformation) is the largest component of overall shear wall deflection.

Effect of wall anchorage slip becomes more significant as the aspect ratio increases. The *SDPWS* requires an anchoring device (see *SDPWS* 4.3.6.4.2) at each end of the shear wall where dead load stabilizing moment is not sufficient to prevent uplift due to overturning. For standard anchoring devices (tie-downs), manufacturers' literature typically includes ASD capacity (based on short-term load duration for wind and seismic), and corresponding deflection of the device at ASD levels. Deflection of the device at strength level forces may also be obtained from manufacturers' literature. Reported deflection may or may not include total deflection of the device relative to a wood post and elongation of the tie-down bolt or strap in tension. All sources of vertical elongation of the anchoring device, such as slip in the connection of the device to the wood post, elongation of the tie-down rod, and slack in the anchorage strap, should be considered when estimating the Δ_a term in *SDPWS* Equation 4.3-1. Estimates of Δ_a at strength level forces are needed when evaluating drift in accordance with ASCE 7 is required.

In shear wall table footnotes (*SDPWS* Table 4.3A), a factor of 0.5 is provided to adjust tabulated G_a values (based on fabricated dry condition) to approximate G_a where "green" framing is used. This factor is based on analysis of apparent shear stiffness for wood structural panel shear wall and diaphragm construction where:

1) framing moisture content is greater than 19% at time of fabrication (green), and
2) framing moisture content is less than or equal to 19% in-service (dry).

The average ratio of "green" to "dry" for G_a across shear wall and diaphragm cells ranged from approximately 0.52 to 0.55. A rounded value of 0.5 results in slightly greater values of calculated deflection for "green" framing when compared to the more detailed 4-term deflection equations. Although based on nail slip relationships applicable to wood structural panel shear walls, this reduction is also extended to other shear wall types.

In Table C4.3.2A, calculated deflections using *SDPWS* Equation 4.3-1 are compared to deflections from tests at 1.4 times the allowable design value of the assembly for shear walls with fiberboard, gypsum sheathing, and lumber sheathing. For lumber sheathing, calculated stiffness is underestimated when compared to test-based stiffness val-

Figure C4.3.2 Comparison of 4-Term and 3-Term Deflection Equations

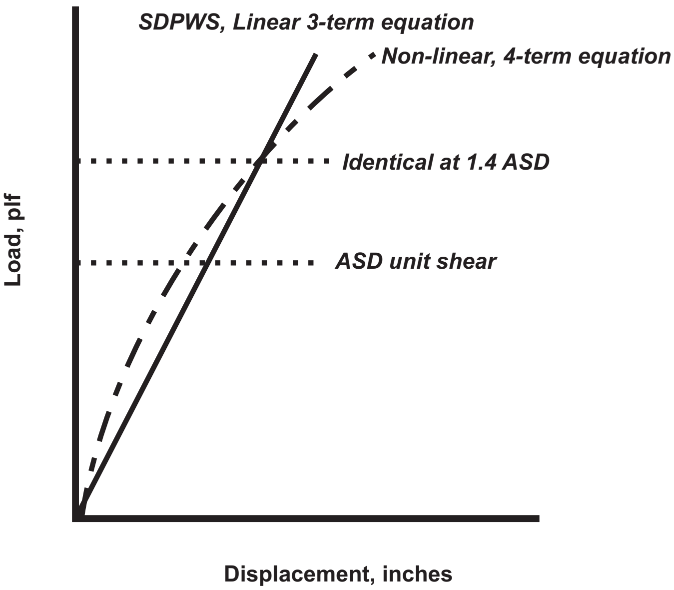


Table C4.3.2A Data Summary for Structural Fiberboard, Gypsum Wallboard, Gypsum Sheathing, and Lumber Sheathed Shear Walls

Reference	Description	Calculated[1]			Actual	
		$1.4v_{s(ASD)}$ (plf)	G_a (kips/in.)	d (in.)	d (in.)	G_a (kips/in.)
Structural Fiberboard Sheathing						
Ref. 17	1/2" structural fiberboard, roofing nail (11 gage x 1-3/4"), 2" edge spacing, 6" field spacing, 16" stud spacing. 8' x 8' wall. (3 tests).	364	5.5	0.53	0.46	6.3
	25/32" structural fiberboard, roofing nail (11 gage x 1-3/4"), 2" edge spacing, 6" field spacing, 16" stud spacing. 8' x 8' wall. (3 tests).	378	5.5	0.55	0.53	5.7
Gypsum Wallboard (GWB) and Gypsum Sheathing						
Ref. 57[2]	1/2" GWB interior applied vertically and joint finished. GWB Nail (1-1/4") at 8" o.c. at all framing members. 24" stud spacing. 8' x 8' wall. (3 tests – controls).	92	3.5	0.21	0.16	4.6
Ref. 23[2,3]	1/2" GWB interior applied horizontally and joint finished and 1/2" gypsum sheathing exterior applied vertically. GWB Nail (1-1/4") at 8" o.c. at all framing members. Gypsum sheathing nail (1-1/2" galvanized roofing) at 4" o.c. at panel edges and 8" o.c. in field. 24" stud spacing. 8' x 8' wall. (3 tests – configuration 8).	315	12.0	0.21	0.23	11.0
	1/2" GWB interior applied horizontally and joint finished and 1/2" gypsum sheathing exterior applied vertically.GWB Nail (1-1/4") at 8" o.c. at all framing members. Gypsum sheathing nail (1-1/2" galvanized roofing) at 4" o.c. at panel edges and 8" o.c. in field. 16" stud spacing. 8' x 8' wall. (3 tests – configuration 10).	338	13.3	0.20	0.22	12.3
Lumber Sheathing						
Ref. 21	Horizontal lumber sheathing, 8' x 12' wall. 1x8 boards. (2) 8d nails at each stud crossing. Stud spacing 16" o.c. (13 tests - controls).	70	1.5	0.37	0.34	1.7
Ref. 24	Horizontal lumber sheathing. 9' x 14' wall. 1x6 and 1x8 boards. (2) 8d nails at each stud crossing. Stud spacing 16" o.c. (3 tests - panel 2A, 33, 27).	70	1.5	0.42	0.25	2.5
	Diagonal lumber sheathing (in tension), 9' x 14' wall. 1x8 boards. (2) 8d nails at each stud crossing. Stud spacing 16" o.c. (test panel 5).	420	6.0	0.63	0.50	7.6
	Diagonal lumber sheathing (in tension), 7.33' x 12.1' wall. 1x8 boards. (2) 8d nails at each stud crossing. Stud spacing 16" o.c. (test panel 26).	420	6.0	0.51	0.36	8.6

1. Calculated deflection based on shear component only. For walls tested, small aspect ratio and use of tie-down rods (ASTM E 72) or use of hold-downs (ASTM E 546) in testing of gypsum wall board shear walls (57) is assumed to keep bending and tie-down slip components of deflection small relative to the shear component of deflection.
2. Unit shear and apparent shear stiffness in *SDPWS* Table 4.3B for 1/2″ GWB with 7″ fastener spacing multiplied by 7/8 to approximate unit shear and stiffness for GWB tested assemblies using 8″ fastener spacing.
3. Calculated unit shear and G_a for the two-sided test assembly based on 4.3.3.3.1.

EXAMPLE C4.3.2-1 Calculate the Apparent Shear Stiffness, G_a, in SDPWS Table 4.3A

Calculate the apparent shear stiffness, G_a, in *SDPWS* Table 4.3A for a wood structural panel shear wall constructed as follows:

Sheathing grade	= Structural I (OSB)
Nail size	= 6d common (0.113" diameter, 2" length)
Minimum nominal panel thickness	= 5/16 in.
Panel edge fastener spacing	= 6 in.
Nominal unit shear capacity for seismic, ν_s	= 400 plf *SDPWS* Table 4.3A

Allowable unit shear capacity for seismic:

$\nu_{s(ASD)}$ = 400 plf/2 = 200 plf

Panel shear stiffness:

$G_v t_v$ = 77,500 lb/in. of panel depth Table C4.2.2A

Nail load/slip at 1.4 $\nu_{s(ASD)}$:

V_n = fastener load (lb/nail)
= 1.4 $\nu_{s(ASD)}$ (6 in.)/(12 in.)
= 140 lb/nail

$e_n = (V_n/456)^{3.144}$ Table C4.2.2D
$= (140/456)^{3.144} = 0.0244$ in.

Calculate G_a:

$$G_a = \frac{1.4 v_{s(ASD)}}{\dfrac{1.4 v_{s(ASD)}}{G_v t_v} + 0.75 e_n} \quad \text{Equation C4.2.2-3}$$

G_a = 12,772 lb/in. ≈ 13 kips/in. *SDPWS* Table 4.3A

ues. However, the lower stated stiffness for horizontal and diagonal lumber sheathing is considered to better reflect stiffness after lumber sheathing dries in service. Early studies (24) suggest that stiffness after drying in service may be 1/2 of that during tests where friction between boards in lumber sheathed assemblies is a significant factor.

C4.3.2.1 Deflection of Perforated Shear Walls: The deflection of a perforated shear wall can be calculated using *SDPWS* Equation 4.3-1 using substitution rules as follows to account for reduced stiffness of full-height perforated shear wall segments:

ν = maximum induced unit shear force (plf) in a perforated shear wall per *SDPWS* Equation 4.3-9

b = sum of perforated shear wall segment lengths (full-height), ft

C4.3.2.2 Deflection of Unblocked Wood Structural Panel Shear Walls: Unblocked shear walls exhibit load deflection behavior similar to that of a blocked shear wall but with reduced values of strength and stiffness. The unblocked shear wall adjustment factor, C_{ub}, accounts for the effect of unblocked joints on strength and stiffness. Nominal unit shear capacity of a blocked wood structural panel shear wall with stud spacing of 24" o.c. and panel edge nail spacing of 6" o.c. is the reference condition for determination of unblocked shear wall nominal unit shear capacity (e.g. $\nu_{ub} = \nu_b C_{ub}$). Blocked shear wall nominal unit shear capacity is not to be adjusted by Table 4.3A footnote 2 even if unblocked shear wall construction consists of studs spaced a maximum of 16" o.c. or panels applied with the long dimension across studs.

To account for the reduction in unblocked shear wall stiffness, which is proportional to reduction in strength, *SDPWS* 4.3.2.2 specifies that deflection of unblocked shear walls is to be calculated from standard deflection equations using an amplified value of induced unit shear equal to ν / C_{ub}. Substituting ν / C_{ub} for ν in Equation 4.3-1 results in the following equation for unblocked shear wall deflection:

(bending) *(shear)* *(wall anchorage slip)*

$$\delta_{sw} = \frac{8\left(\dfrac{v}{C_{ub}}\right)h^3}{EAb} + \frac{\left(\dfrac{v}{C_{ub}}\right)h}{1000G_a} + \frac{h}{b}\Delta_a \quad \text{(C4.3.2.2-1)}$$

Where values of C_{ub} are less than 1.0, induced unit shear is amplified by $1/C_{ub}$ resulting in larger deflection for the less stiff unblocked shear wall relative to the blocked shear wall reference condition. The C_{ub} factor can also be viewed as a stiffness reduction factor. For example, simplification of the shear term in Eq. C4.3.2.2-1 yields:

$$\frac{vh}{1000\left(C_{ub}\ G_a\right)} \qquad \text{(C4.3.2.2-2)}$$

where:

$(C_{ub}\ G_a)$ = Apparent shear stiffness of an unblocked shear wall, G_a unblocked

C4.3.2.3 Deflection of Structural Fiberboard Shear Walls: The calculated deflection of shear walls sheathed with structural fiberboard having an aspect ratio greater than 1.0 is underestimated when compared to results from cyclic testing (29). An adjustment factor equal to $(h/b_s)^{1/2}$ is therefore used to account for the increased deflection of structural fiberboard shear walls having aspect ratios greater than 1.0.

C4.3.3 Unit Shear Capacities

See C4.2.3 for calculation of ASD unit shear capacity and LRFD factored unit shear resistance. Shear capacity of perforated shear walls is discussed further in section C4.3.3.5.

C4.3.3.1 Tabulated Nominal Unit Shear Capacities: *SDPWS* Table 4.3A provides nominal unit shear capacities for seismic, v_s, and for wind, v_w, (see C2.2) for OSB, plywood, plywood siding, particleboard, and structural fiberboard sheathing. *SDPWS* Table 4.3B provides nominal unit shear capacities for wood structural panels applied over 1/2" or 5/8" gypsum wallboard or gypsum sheathing board. *SDPWS* Table 4.3C provides nominal unit shear capacities for gypsum wallboard, gypsum sheathing, plaster, gypsum lath and plaster, and portland cement plaster (stucco). Nominal unit strength capacities are based on adjustment of allowable values in building codes and industry reference documents (See C2.2).

Table 4.3A lists nominal unit shear capacities for Wood Structural Panels in three distinct groupings: Structural I, Sheathing, and Plywood Siding. Structural I and Sheathing are applicable for Sheathing and Single-Floor wood structural panel grades in conformance with requirements of PS1 and PS2 (58, 8). The Structural I designation is associated with panels that meet additional requirements including those for cross-panel strength and stiffness and for racking load performance. For Plywood Siding, tabulated nominal unit shear capacities are associated with reduced thickness at ornamental grooves and panel shiplap joints. The combination of a small head galvanized casing nail (see Table C.4.3.3.1) and nailing through the reduced thickness portion of the siding panel reduces the shear resistance, as compared with other panel types and fastener options. Except for Plywood Siding of Group 5 species (e.g. Basswood and Balsam Poplar) as defined in PS 1, nominal unit shear capacities associated with the "Sheathing" designation are applicable for Plywood Siding when nailed with the larger galvanized box or common nails and the nominal panel thickness is determined at the point of nailing along panel edges.

Table C.4.3.3.1 Dimension of Galvanized Casing Nails in Accordance With ASTM F1667 (59)

Penny-weight	Length (in)	Shank Diameter (in)	Head Diameter (in)
6d	2	0.099	0.142
8d	2-1/2	0.113	0.155

C4.3.3.2 Unblocked Wood Structural Panel Shear Walls: Monotonic and cyclic tests of unblocked wood structural panel shear walls (18, 27, and 28) are the basis of the unblocked shear wall factor, C_{ub}, which accounts for reduced strength and stiffness of unblocked shear walls when compared to similarly constructed blocked shear walls. Test results show comparable displacement capacity characteristics to similarly constructed blocked wood structural panel shear walls over a range of unblocked panel configurations. Tests included a range of panel edge and field nail spacing, stud spacing, wall height, gap distance at adjacent unblocked panel edges, and simultaneous application of gravity load. The maximum unblocked shear wall height tested was 16′ and the maximum gap distance between adjacent unblocked panel edges was 1/2". Maximum unit shear capacities are limited to values applicable for 6" o.c. panel edge nail spacing regardless of actual panel edge nail spacing used. The limit on maximum unit shear is to address limited observations of stud splitting in walls tested to higher shear capacities associated with panel edge nail spacing of 4" o.c.

C4.3.3.3 Summing Shear Capacities: A wall sheathed on two-sides (e.g., a two-sided wall) has twice the capacity of a wall sheathed on one-side (e.g., a one-sided wall) where sheathing material and fastener attachment schedules on each side are identical. Where sheathing materials are the same on both sides, but different fastening schedules are used, provisions of *SDPWS* 4.3.3.3.1 are applicable. Although not common for new construction, use of different fastening schedules is more likely to occur in retrofit of existing construction.

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C4.3.3.3.1 For two-sided walls with the same sheathing material on each side (e.g., wood structural panel) and same fastener type, *SDPWS* Equation 4.3-3 and *SDPWS* Equation 4.3-4 provide for determination of combined stiffness and unit shear capacity based on relative stiffness of each side.

C4.3.3.3.2 For seismic design of two-sided walls with different materials on each side (e.g., gypsum on side one and wood structural panels on side two), the combined unit shear capacity is taken as twice the smaller nominal unit shear capacity or the larger nominal unit shear capacity, whichever is greater. Due to lateral system combination rules for seismic design (5), the two-sided unit shear capacity based on different materials on each side of the wall will require use of the least seismic response modification coefficient, R, for calculation of seismic loads. For a two-sided shear wall consisting of wood-structural panel exterior and gypsum wallboard interior, R = 2 is applicable where shear wall design is based on two times the capacity of the gypsum wallboard because R = 2 (associated with gypsum wallboard shear walls in a bearing wall system) is the least R contributing to the two-sided shear wall design capacity. For the same wall condition, when design is based on wood structural panel shear wall capacity alone, R = 6.5 (associated with wood structural panel shear walls in a bearing wall system) is applicable and commonly employed for design of WSP shear walls as the seismic force resisting system.

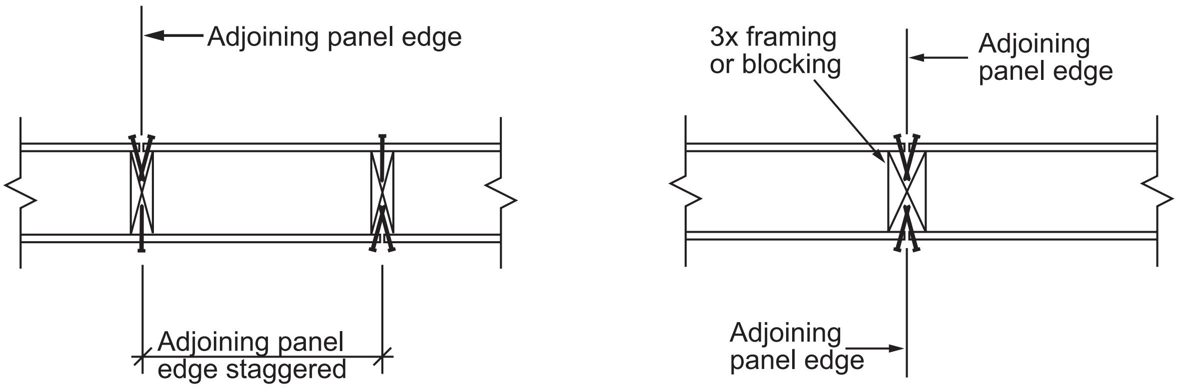
For wind design, direct summing of the contribution of gypsum wallboard with the unit shear capacity of wood structural panel, structural fiberboard, or hardboard panel siding is permitted based on tests (10 and 15).

Figure C4.3.3 illustrates the provisions in Footnote 6 of Table 4.3A and Footnote 5 of Table 4.3B requiring panel joints to be offset to fall on different framing members when panels are applied on both faces of a shear wall, nail spacing is less than 6" on center on either side, and the framing member nailed face width is less than 3x framing.

C4.3.3.4 Shear Walls in a Line: The provisions for distribution of shear force to shear walls in a line are limited to shear walls of similar materials and construction; however, materials and construction are not required to be identical. The intended purpose of the requirement is to limit applicability of provisions to various assemblies of the same material (e.g. nailed wood structural panels, or nailed structural fiberboard) that also exhibit similar load/deformation behavior up to failure. Nailed wood structural panel shear walls, regardless of sheathing grade or thickness and nailing schedule are considered to exhibit compatible behavior. For example, combination of a shear wall with 2" nail spacing in line with a shear wall with 6" nail spacing does not violate the similar materials and construction type requirement. Provisions for distribution force apply whether force is from wind or seismic and for shear walls of any length.

C4.3.3.4.1 The distribution of shear force to shear walls in a line is in proportion to the stiffness of each shear wall. In design, at a given deflection the force in each wall is determined by multiplying the wall stiffness times the deflection (e.g. commonly referred to as distribution based on relative stiffness or the equal deflection approach). For all but the case where a wall line is comprised entirely of equal stiffness shear walls, this approach results in a design capacity of the shear wall line that is less than would result from the sum of shear wall lengths times the full design unit shear capacity, because for a given deflection, full unit shear forces are not developed simultaneously in all walls (see Example C4.3.3.4.1-1). The design capacity of the shear wall line will be the sum of the forces in each shear wall at a given deflection. The limiting value of deflection

Figure C4.3.3 Detail for Adjoining Panel Edges where Structural Panels are Applied to Both Faces of the Wall



a. Adjoining panel edges staggered b. Adjoining panel edges not staggered may be associated with the shear wall in the line whose design strength occurs at the smallest deflection of any shear wall in the wall line or may be associated with drift or deflection limits.

Drift limits for resistance to seismic forces are provided in ASCE 7 and vary by building risk category, story height, and construction material. For wood frame shear wall structures, allowable story drift limits for seismic design range from 1% to 2.5% of the story height. While there is no prescribed deflection limit for wind design, consideration should be given to limiting deflections to avoid serviceability problems associated with finish materials and operability of doors and windows. Compliance with construction and materials requirements and associated design unit shear capacities in *SDPWS* for wind design have been considered to provide acceptable serviceability performance for resistance to wind loads.

A simplified approach is also permitted. In lieu of distribution of shear based on the equal deflection calculation method, it is permitted to distribute shear in proportion to the strength of the shear wall provided that certain requirements are met. For wood structural panels, distribution of shear in proportion to shear strength of each shear wall is permitted provided that shear walls with aspect ratio greater than 2:1 have strength adjusted by the $2b_s/h$ factor. For structural fiberboard, distribution in proportion to shear strength of each shear wall is permitted provided that shear walls with aspect ratio greater than 1:1 have strength multiplied by the $0.1+0.9b_s/h$ factor. Both factors are based on reduced stiffness observed from testing (29, 35, and 36) and provide roughly similar results to equal deflection for a reference wall line configuration comprised of 1:1 to 3.5:1 aspect ratio walls (see Example C4.3.3.4.1-2). It was judged suitable for design purposes for any combination of shear wall lengths.

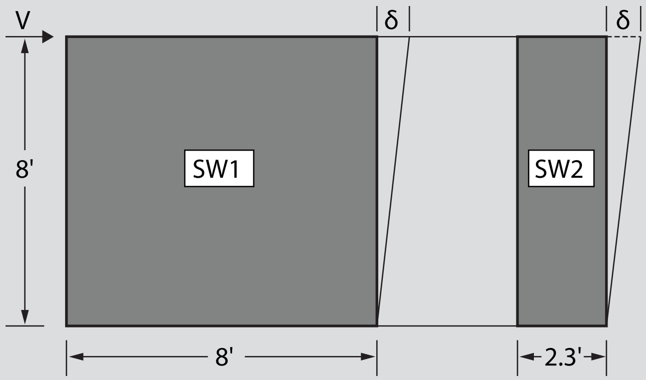
Where distribution of shear is based on the simplified alternative adjustment factor methods (e.g. $2b_s/h$ for wood structural panels and $0.1+0.9b_s/h$ for structural fiberboard) further reduction of shear strength by the aspect ratio factors in 4.3.4.2 is not required because the strength reductions to provide for deflection compatibility represent the more conservative of the adjustments to account for reduced stiffness and reduced strength of high aspect ratio shear walls.

Example C4.3.3.4.1-1. Calculate the ASD Shear Capacity of Shear Walls in a Line Using the Equal Deflection Calculation Approach

Calculate the ASD shear capacity for the shear wall line as shown in Figure C4.3.3A using the equal deflection calculation approach. The individual blocked wood structural panel shear walls are constructed with 15/32 in. thick wood structural panel sheathing attached to No.2 Douglas fir 2x4 framing with 8d nails (common or galvanized box) spaced at 6 in. and have a reference nominal unit shear value for seismic of 520 plf and apparent shear stiffness, G_a, of 13 kips/in. The end posts for both walls are double 2x4's. Vertical elongation of wall anchorage is 1/8" at 3500 lb load.

The solution approach is to determine the proportion of load in each shear wall at a given deflection by use of the *SDPWS* shear wall deflection equations and then summing the shears to arrive at an ASD shear capacity for the shear wall line (Figure C4.3.3B).

Figure C4.3.3A Shear Wall Line



Shear Wall 1 (SW1):

Nominal unit shear capacity for seismic = 520 plf (*SDPWS* Table 4.3A)

SW1 Aspect ratio (h/b_s) = 1.0
Aspect Ratio Factor (WSP) for strength = 1.0 (*SDPWS* 4.3.4.2)
ASD unit shear capacity for seismic, v_{SW1} = 520 plf/2 x 1.0 = 260 plf

G_a = 13.0 kips/in
EA = 16,800,000 lb

Shear Wall 2 (SW2):
Nominal unit shear capacity for seismic = 520 plf (*SDPWS* Table 4.3A)

SW2 Aspect ratio (h/b_s) = 3.5
Aspect Ratio Factor (WSP) for strength = $1.25 - 0.125h/b_s$ = 0.81 (*SDPWS* 4.3.4.2)
ASD unit shear capacity for seismic, v_{SW2} = 520 plf/2 x 0.81 = 210 plf

G_a = 13.0 kips/in
EA = 16,800,000 lb

Recognizing that the ASD unit shear capacity of SW1 is associated with the smaller deflection, the problem can be simplified to finding the reduced design unit shear in the less stiff SW2 that produces the same deflection as SW1.

Part 1 – Determine the deflection of SW1 at its ASD unit shear capacity

Deflection associated with the ASD unit shear capacity is calculated in accordance with the following equation

$$\delta_{SW1} = \frac{8v_{SW1}h^3}{EAb_{SW1}} + \frac{v_{SW1}h}{1000G_a} + \frac{h\Delta_a}{b_{SW1}}$$

$$\delta_{SW1} = 0.008 + 0.16 + 0.074 = 0.242\,in$$

where:

v_{SW1} = 260 plf
h = 8 ft
EA = 16,800,000 lb
b_{SW1} = 8 ft
G_a = 13 kips/in
$\Delta_{a,SW1}$ = 0.074 in.

Note: Vertical elongation of wall anchorage is assumed to vary linearly with overturning anchorage force calculated as 2080 lb (i.e. 260 plf x 8 ft = 2080 lb). Under this linear elastic assumption, the stiffness of the anchorage is: k = 3500 lb/0.125 in = 28,000 lb/in. Vertical elongation of the anchorage in SW1 is: 2080 lb/3500 lb x 0.125in. = 0.074 in. For purposes of this example, overturning resistance contributed by dead load is ignored.

(continued)

Example C4.3.3.4.1-1. Calculate the ASD Shear Capacity of Shear Walls in a Line Using the Equal Deflection Calculation Approach (continued)

Part 2 – Determine the unit shear in SW2 that produces the same deflection as SW1

Using equation 4.3-1, solve for unit shear that produces equal deflection to SW1:

$$v_{SW2} = \frac{\delta}{\dfrac{8h^3}{EAb_{SW2}} + \dfrac{h}{1000G_a} + \dfrac{h^2}{kb_{SW2}}} = 141\,plf$$

where:

δ = 0.242 in.

h = 8 ft

EA = 16,800,000 lb

= 2.3 ft

G_a = 13 kips/in.

k = 28,000 lb/in.

Note: For SW2, the overturning anchorage force is 1128 lb (i.e. 141 plf x 8 ft = 1128 lb). The corresponding vertical elongation of the anchorage is: $\Delta_{a,SW2}$ = 1128 lb/28,000 lb/in = 0.040 in. For purposes of this example, overturning resistance contributed by dead load is ignored.

v_{SW2} = 141 plf ≤ 210 plf ; OK - 141plf controls

Note: The induced unit shear of 141 plf does not exceed the unit shear capacity of 210 plf determined from aspect ratio strength reductions in accordance with 4.3.4.2.

Deflection associated with this unit shear is confirmed to equal 0.242 in. calculated in accordance with the following equation:

$$\delta_{SW2} = \frac{8v_{SW2}h^3}{EAb_{SW2}} + \frac{v_{SW2}h}{1000G_a} + \frac{h\Delta_{a,SW2}}{b_{SW2}}$$

$$\delta_{SW2} = 0.015 + 0.087 + 0.140 = 0.242\,in$$

Part 3 – Sum design strengths associated with distribution of shear based on the equal deflection calculation approach

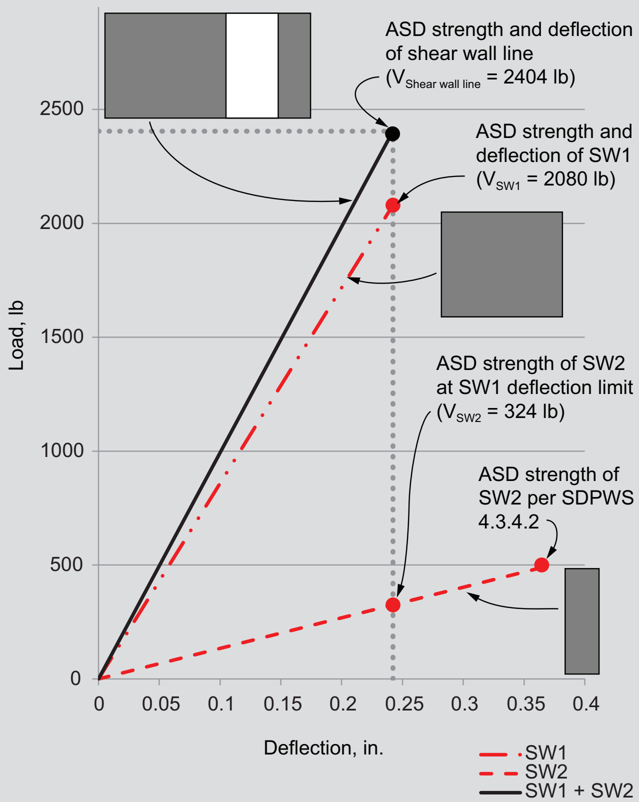
V_{SW1} = 260 plf x 8 ft = 2080 lb

V_{SW2} = 141 plf x 2.3 ft = 324 lb

$V_{\text{Shear wall line}}$ = 2080 lb + 324 lb = 2404 lb

This example illustrates calculation of the ASD shear capacity for the shear wall line (i.e. $V_{\text{Shear wall line}}$ = 2404 lb). Distribution of shear by the equal deflection calculation method is equally applicable to a more typical design problem where a design load is associated with a given shear wall line and the shear wall line is designed to provide resistance in excess of the design load. This calculation example includes a check of the induced unit shear in each shear wall (i.e. SW1 and SW2) to ensure that unit shear does not exceed the unit shear capacity determined from aspect ratio strength reductions in accordance with 4.3.4.2.

Figure C4.3.3B Illustration of Equal Deflection Calculation Method



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Example C4.3.3.4.1-2. Calculate the ASD Shear Capacity of Shear Walls in a Line Using the Exception to 4.3.3.4.1

Calculate the ASD shear capacity for the shear wall line from Example C4.3.3.4.1-1. In lieu of the equal deflection calculation approach of 4.3.3.4.1, the solution approach is based on the Exception to 4.3.3.4.1 which permits distribution of shear in proportion to strength where strength of wood structural panel shear walls is adjusted by the $2b_s/h$ factor.

Shear wall 1 (SW1):

Nominal unit shear capacity for seismic = 520 plf

(*SDPWS* Table 4.3A)

SW1 Aspect ratio $(h/b_s) = 1.0$

Aspect ratio adjustment = $2b_s/h = 1.0$

(*SDPWS* 4.3.3.4.1 Exception 1)

Aspect Ratio Factor (WSP) for strength =1.0

(*SDPWS* 4.3.4.2)

ASD unit shear capacity for seismic, v_{SW1} = 520 plf/2 x 1.0 = 260 plf

Unit shear capacity from 4.3.3.4.1 Exception 1 does not exceed the unit shear capacity determined from aspect ratio strength reductions in accordance with 4.3.4.2.

Shear wall 2 (SW2):

Nominal unit shear capacity for seismic = 520 plf

(*SDPWS* Table 4.3A)

SW2 Aspect ratio $(h/b_s) = 3.5$

Aspect ratio adjustment = $2b_s/h = 0.57$

(*SDPWS* 4.3.3.4.1 Exception 1)

Aspect Ratio Factor (WSP) for strength = 0.81

(*SDPWS* 4.3.4.2)

ASD unit shear capacity for seismic, v_{SW2} = 520 plf/2 x 0.57 = 148 plf

Unit shear capacity from 4.3.3.4.1 Exception 1 does not exceed the unit shear capacity determined from aspect ratio strength reductions in accordance with 4.3.4.2.

Sum design strengths associated with distribution of shear based on the aspect ratio factor adjustment approach (e.g. 4.3.3.4.1 Exception)

V_{SW1}	= 260 plf x 8 ft	= 2080 lb
V_{SW2}	= 148 plf x 2.3 ft	= 340 lb
$V_{\text{Shear wall line}}$	= 2080 lb + 340 lb	= 2420 lb

Note: This example illustrates the calculation approach in accordance with 4.3.3.4.1 Exception 1. The aspect ratio adjustment, $2b_s/h$ is not applied cumulatively with the Aspect Ratio Factor (WSP) for strength reduction of 4.3.4.2. Both are evaluated as separate checks on design shear strength.

C4.3.3.5 Shear Capacity of Perforated Shear Walls: The shear capacity adjustment factor, C_o, for perforated shear walls accounts for reduced shear wall capacity due to presence of openings and is derived from empirical Equations C4.3.3.5-1 and C4.3.3.5-2 (13):

$$F = r/(3 - 2r) \tag{C4.3.3.5-1}$$

$$r = 1/(1+A_o/(h\sum L_i)) \tag{C4.3.3.5-2}$$

The opening adjustment factor, C_o, and the shear capacity ratio, F, are related as follows:

$$C_o(\Sigma L_i) = F(L_{tot}) \tag{C4.3.3.5-3}$$

SDPWS Equation 4.3-5 can be obtained by simplification of C4.3.3.5-1, C4.3.3.5-2, and C4.3.3.5-3. Values of the shear capacity adjustment factors in Table 4.3.3.5 can be determined by assuming a constant maximum opening height, $h_{o\text{-}max}$, such that $A_o = h_{o\text{-}max}(L_{tot}\text{-}\Sigma L_i)$. Substituting this value of A_o into Equation C4.3.3.5-2 and simplifying:

$$C_o = \left(\frac{L_{tot}}{\sum L_i}\right)\left(\frac{\left(1+\frac{h_{o-\max}}{h}\left(\frac{L_{tot}}{\sum L_i}-1\right)\right)^{-1}}{3-2\left(1+\frac{h_{o-\max}}{h}\left(\frac{L_{tot}}{\sum L_i}-1\right)\right)^{-1}}\right) \tag{C4.3.3.5-4}$$

where:

C_o = shear capacity adjustment factor (for perforated shear wall segments)

F = shear capacity ratio based on total length of the perforated shear wall

r = sheathing area ratio

A_o = total area of openings, ft^2

h = wall height, ft

$h_{o\text{-}max}$ = maximum opening height, ft

L_{tot} = total length of a perforated shear wall including lengths of perforated shear wall segments and segments containing openings, ft

$\sum L_i$ = sum of perforated shear wall segment lengths, L_i, ft. Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with 4.3.4.3.

Full-scale perforated shear wall tests include monotonic and cyclic loads, long perforated shear walls with asymmetrically placed openings, perforated shear walls sheathed on two sides, and perforated shear walls with high aspect ratio shear wall segments (15, 42, 43, and 44).

C4.3.4 Shear Wall Aspect Ratios and Capacity Adjustments

C4.3.4.2 The aspect ratio factor, 1.25 - 0.125h/b_s, is applicable to blocked wood structural panel shear walls designed to resist either wind or seismic forces. The factor ranges in value from 1.0 for 2:1 aspect ratio shear walls to 0.81 for 3.5:1 aspect ratio shear walls and accounts for reduced unit shear capacity of high aspect ratio shear walls relative to lower aspect ratio shear walls observed from monotonic and cyclic tests (35, 36). The aspect ratio factor, 1.09 - 0.09 h/b_s, is applicable to structural fiberboard shear walls designed to resist either wind or seismic forces. The factor accounts for observed reduction in peak unit shear capacity from testing (29) and varies from 1.0 for 1:1 aspect ratio shear walls to 0.78 for 3.5:1 aspect ratio shear walls.

Reductions in shear wall unit shear capacity are addressed with aspect ratio factors in 4.3.4.2; however, the loss of stiffness as aspect ratio increases can be large and affect usable design unit shear capacity. For example, due to reduced stiffness, a high aspect ratio shear wall may reach a deflection limit or drift limit prior to developing even the reduced unit shear capacity associated with the high aspect ratio shear wall based on unit shear capacity reductions of 4.3.4.2. In addition, where a high aspect ratio shear wall is in line with a low aspect ratio shear wall, development of the full design unit shear capacity of the low aspect ratio shear wall is likely to occur at relatively small deflections where only a fraction of the full design unit shear capacity of the high aspect ratio shear wall is developed.

Requirements of 4.3.4.2 address the shear wall strength limit state while those in 4.3.3.4 address deflection compatibility between shear walls in a line. Both strength and deflection compatibility criteria must be addressed as part of a design. For example, unit shear capacity associated with each shear wall must satisfy deflection compatibility criteria as well as criteria of 4.3.4.2 establishing unit shear capacity based on shear wall aspect ratio. The smaller of the design unit shear capacities associated with requirements of 4.3.4.2 and 4.3.3.4 is used as the controlling design unit shear capacity for each shear wall (see Example C4.3.3.4.1-1 and C4.3.3.4.1-2).

C4.3.5 Shear Wall Types

SDPWS identifies shear walls as one of the following "types":

1. Individual Full-Height Wall Segment Shear Walls (i.e., no openings within an individual full-height segment);

2. Force-transfer Shear Walls (i.e., with openings, but framing members, blocking, and connections around openings are designed for force-transfer);
3. Perforated Shear Walls (i.e., with openings, but rather than design for force-transfer around openings, reduced shear strength is used based on size of openings).

C4.3.5.1 Individual Full-Height Wall Segments: Shear wall design provisions for individual full-height wall segments, designed as shear walls without openings, are applicable to walls with wood structural panel sheathing, designed and constructed in accordance with provisions as outlined in *SDPWS* 4.3.5.1.

C4.3.5.2 Force-transfer Shear Walls: Force-transfer shear wall design provisions are applicable to walls with wood structural panel sheathing, designed and constructed in accordance with provisions as outlined in *SDPWS* 4.3.5.2. Design of shear walls with openings as force-transfer shear walls, also known as design for force transfer around openings (FTAO), is required to be in accordance with a rational analysis as described in *SDPWS* 4.3.5.2. Under this approach, it is the responsibility of the designer to establish detailing to ensure appropriateness of the assumptions in the analysis. Limited testing has been conducted to evaluate several rational analysis methods commonly referred to as the drag strut, cantilever beam and Diekmann methods (53). The mathematical development of the drag strut method and cantilever beam method (54) are based on assumptions for shear transfer above and below the openings and development of the Diekmann method (55) is based on resolving internal forces assuming rigid body behavior. Another rational analysis method, referred to as the SEAOC-Thompson method (56) is commonly employed for the design of shear walls with openings for FTAO. Limited evaluation of the SEAOC-Thompson method has shown that the results for strap force predictions are similar to those of the Diekman method for several tested wall configurations (46).

C4.3.5.3 Perforated Shear Walls: Perforated shear wall design provisions are applicable to walls with wood structural panel sheathing, designed and constructed in accordance with provisions as outlined in *SDPWS* 4.3.5.3. The single side limits for seismic and wind, 1,740 plf nominal and 2,435 plf nominal respectively, are based on tests utilizing 10d common nails at 2" o.c. at panel edges on one side (45). The single side limits on maximum nominal unit shear capacity are also applicable for double-sided walls (walls sheathed on two sides) because the tested walls represent the maximum unit shear strength for which tests are available. The maximum nominal unit shear capacity for seismic design of a double-sided wall, is 1,740 plf for consistency with the judgment to limit maximum nominal unit shear capacity for wind based on tests.

C4.3.6 Construction Requirements

C4.3.6.1 Framing Requirements: Framing requirements are intended to ensure that boundary members and other framing are adequately sized to resist induced loads.

C4.3.6.1.1 General framing requirements for shear walls permits the use of two 2x members in lieu of a single member provided they are adequately connected for transfer of induced shear forces. Cyclic tests of shear walls confirms that use of two 2x members nailed (22, 25, and 30) or screwed (33) together results in shear wall performance that is comparable to that obtained by use of a single 3x member at the adjoining panel edge. While introduced as a substitute for a 3x member at adjoining panel edges in shear wall construction, it is also permissible to use two 2x members to substitute for a single 2x member (e.g. for blocking, and top plates). Attachment of the two 2x members to each other is required to equal or exceed design unit shear forces in the shear wall. As an alternative, a capacity-based design approach can be used where the connection between the two 2x members equals or exceeds the capacity of the sheathing to framing attachment. Where fastener spacing used in the interconnection of the two 2x stud members is closer than 4" on center, staggered placement is required to limit potential for wood splitting.

C4.3.6.1.2 Tension and Compression Chords: *SDPWS* Equation 4.3-7 provides for calculation of tension and compression chord force due to induced unit shear acting at the top of the wall (e.g., tension and compression due to wall overturning moment). To provide an adequate load path per *SDPWS* 4.3.6.4.4, design of elements and connections must consider forces contributed by each story (i.e., shear and overturning moment must be accumulated and accounted for in the design).

C4.3.6.1.3 Tension and Compression Chords of Perforated Shear Walls: *SDPWS* Equation 4.3-8 provides for calculation of tension force and compression force at each end of a perforated shear wall, due to shear in the wall, and includes the term $1/C_o$ to account for the non-uniform distribution of shear in a perforated shear wall. For example, a perforated shear wall segment with tension end restraint at the end of the perforated shear wall can develop the same shear capacity as an individual full-height wall segment (7).

C4.3.6.3 Fasteners: Details on type, size, and spacing of mechanical fasteners used for typical shear wall assemblies in Table 4.3A, 4.3B, 4.3C, and 4.3D are provided in *SDPWS* 4.3.7 Shear Wall Systems.

C4.3.6.3.1 Adhesives: Adhesive attachment of shear wall sheathing is generally prohibited unless approved by the authority having jurisdiction. Because of limited ductility and brittle failure modes of rigid adhesive shear wall systems (38) such systems are limited to seismic design categories A, B, and C and the values of R and Ω_0 are limited (R =1.5 and Ω_0 = 2.5 unless other values are approved). If adhesives are used to attach shear wall sheathing, the effects of increased stiffness (see C4.1.3 and C4.2.5), increased strength, and potential for brittle failure modes corresponding to adhesive or wood failure, should be addressed.

Tabulated values of apparent shear stiffness, G_a, are based on assumed nail slip behavior (see Table C4.2.2D) and are therefore not applicable for adhesive shear wall systems where shear wall sheathing is rigidly bonded to shear wall boundary members.

C4.3.6.4.1.1 In-plane Shear Anchorage for Perforated Shear Walls: *SDPWS* Equation 4.3-9 for in-plane shear anchorage includes the term $1/C_o$ to account for non-uniform distribution of shear in a perforated shear wall. For example, a perforated shear wall segment with tension end restraint at the end of the perforated shear wall can develop the same shear capacity as an individual full-height wall segment (7).

C4.3.6.4.2.1 Uplift Anchorage for Perforated Shear Walls: Attachment of the perforated shear wall bottom plate to elements below is intended to ensure that the wall capacity is governed by sheathing to framing attachment (shear wall nailing) and not bottom plate attachment for shear (see C4.3.6.4.1.1) and uplift. An example design (7) provides typical details for transfer of uplift forces.

C4.3.6.4.3 Anchor Bolts: Plate washer size and location are specified for anchoring of wall bottom plates to minimize potential for cross-grain bending failure in the bottom plate (see Figure C4.3.6A). For a 3" x 3" plate washer centered on the wide face of a 2x4 bottom plate, edges of the plate washer are always within 1/2" of the sheathed side of the bottom plate. For wider bottom plates, such as 2x6, a larger plate washer may be used so that the edge of the plate washer extends to within 1/2" of the sheathed side, or alternatively, the anchor bolt can be located such that the 3" x 3" plate washer extends to within 1/2" of the sheathed side of the wall.

The washer need not extend to within 1/2" of the sheathed edge where sheathing material nominal unit shear capacity for seismic is less than or equal to 400 plf nominal. This allowance is based on observations from tests and field performance of gypsum products where sheathing fastener tear-out or sheathing slotting at fastener locations were the dominant failure modes. Other sheathing materials with nominal unit shear capacity for seismic less than 400 plf nominal are included in this provision based on the judgment that the magnitude of unit uplift force versus sheathing type is the significant factor leading to potential for bottom plate splitting.

Cyclic testing of wood structural panel shear walls (25 and 30) forms the basis of the exception to the 1/2" distance requirement. In these tests, edge distance was not a significant factor for shear walls having full-overturning restraint provided at end posts. Overturning restraint of wall segments coupled with the nominal capacity of walls tested were viewed as primary factors in determining wall performance and failure limit states.

Bottom plate anchor straps can also be effective in mitigating cross-grain bending failure in the bottom plate provided they have been properly tested, load rated, and installed on the sheathed side of the bottom plate.

The extension of plate washer requirements to foundation sill plate applications is depicted in Figure C4.3.6B. Locating the washer's edge within 1/2" of the sheathed edge is accomplished by placement of cuts, notches or holes in the rim board or blocking. The extent of such alterations to the rim board or blocking for placement of the washer should be kept to a practical minimum and be in accordance with the manufacturer's recommendations or be specifically addressed in the design of the member.

Anchor bolt connections designed per *SDPWS* 4.3.6.4.3 are designed for the shear load in the sill plate. If the shear capacity of a double-sided shear wall is twice that of a single-sided shear wall, the anchor bolt spacing derived based on the anchor bolt shear capacity for a double-sided shear wall would be half the spacing of a single-sided shear wall. Staggering the anchor bolts 1/2" from the plate edge for a double-sided shear wall provides uplift resistance on each edge of the sill plate equivalent to a single row of anchor bolts located 1/2" from the plate edge on a single-sided shear wall.

Figure C4.3.6A Distance for Plate Washer Edge to Sheathed Edge

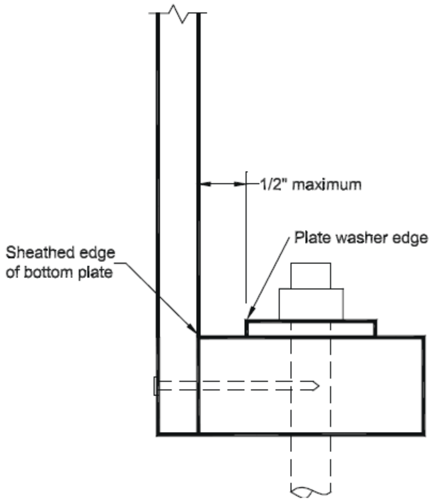
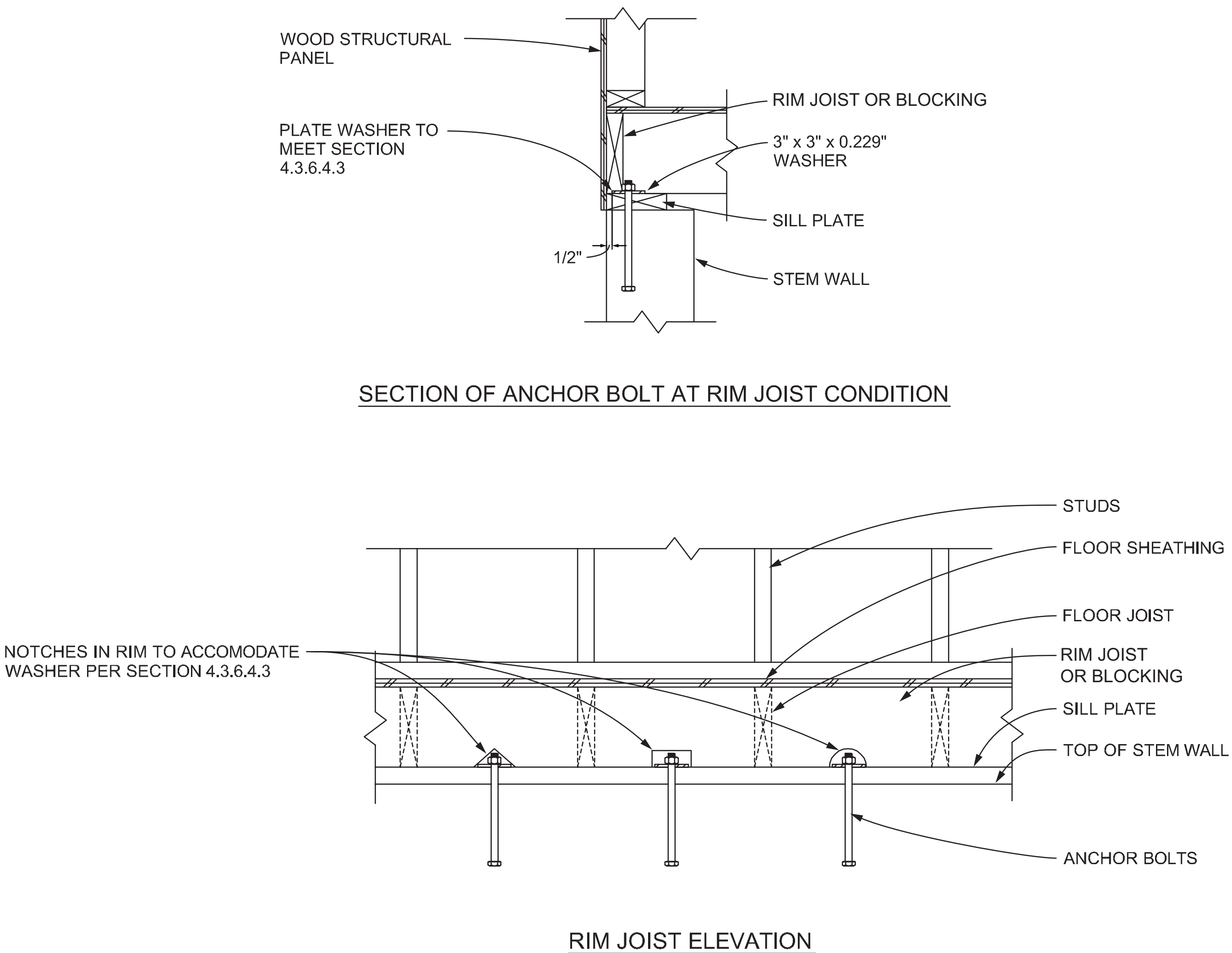


Figure C4.3.6B Section and Elevation View of Plate Washer in Foundation Sill Plate (Raised Floor) Application



C4.3.6.4.4 Load Path: Specified requirements for shear, tension, and compression *in SDPWS* 4.3.6 are to address the effect of induced unit shear on individual wall elements. Overall design of an element must consider forces contributed from multiple stories (i.e., shear and moment must be accumulated and accounted for in the design). In some cases, the presence of load from stories above may increase forces (e.g., effect of gravity loads on compression end posts) while in other cases it may reduce forces (e.g., effect of gravity loads reduces net tension on end posts).

Consistent with a continuous load path for individual full-height wall segments and force transfer shear walls, a continuous load path to the foundation must also be provided for perforated shear walls. Consideration of accumulated forces (for example, from stories above) is required. Accumulation of forces will affect tie-downs at each end of the perforated shear wall, compression resistance at each end of each perforated shear wall segment, and distributed forces, ν and t, at each perforated shear wall segment. Where ends of perforated shear wall segments occur over beams or headers, the beam or header will need to be checked for vertical tension and compression forces in addition to gravity forces. Where adequate collectors are provided to distribute shear, the average shear in the perforated shear wall above (e.g., equivalent to design shear loads), and not the increased shear for anchorage of upper story wall bottom plates to elements below (7), needs to be considered.

C4.3.7 Shear Wall Systems

Requirements for shear wall sheathing materials, framing, and nailing are consistent with industry recommendations and building code requirements. The minimum width of the nailed face of framing members and blocking for all shear wall types is 2" nominal with maximum spacing between framing of 24". Edges of wood-based panels (wood structural panel, particleboard, and structural fiberboard) are required to be backed by blocking or framing except as specified in 4.3.3.2. In addition, fasteners are to be placed at least 3/8" from edges and ends of panels but not less than distances specified by the manufacturer in the manufacturers' literature or code evaluation report.

C4.3.7.1 Wood Structural Panel Shear Walls: For wood structural panel shear walls, framing members or blocking is required at edges of all panels except as specified in 4.3.3.2 and a minimum panel dimension of 4′ x 8′ is specified except at boundaries and changes in framing. Shear wall construction is intended to consist primarily of full-size sheets except where wall dimensions require use of smaller sheathing pieces (e.g. where shear wall height or length is not in increments of 4′, shear wall height is less than a full 8′, or shear wall length is less than 4′). Racking tests conducted on 4.5′ x 8.5′ blocked shear walls showed similar performance whether sheathed length and height consisted of: one 4′x8′ panel and two 6" wide sheathing pieces to make up the height and length, or one 2.5′ x 6.5′ panel and two 2′ wide sheathing pieces to make up the height and length (14).

C4.3.7.1(5): A single 3x framing member is specified at adjoining panel edges for cases prone to splitting and where nominal unit shear capacity exceeds 700 plf in seismic design categories (SDC) D, E, and F. An alternative to single 3x framing, included in *SDPWS*, and based on principles of mechanics, is the use of 2-2x "stitched" or interconnected two 2x members adequately fastened together (See C4.3.6.1.1 for additional information). For sheathing attachment to framing with closely spaced or larger diameter nails, staggered nail placement at each panel edge is intended to prevent splitting in the framing member (Figure C4.2.7).

C4.3.7.2 Shear Walls using Wood Structural Panels over Gypsum Wallboard or Gypsum Sheathing Board: Shear walls using wood structural panels applied over gypsum wallboard or gypsum sheathing are commonly used for exterior walls of buildings that are fire-resistance-rated for both interior and exterior fire exposure. For example, a one hour fire resistance rating can be achieved with 5/8" Type X gypsum wallboard. Nominal unit shear capacities and apparent shear stiffness values in Table 4.3B for 8d and 10d nails (common or galvanized box) are based on nominal unit shear capacities and apparent shear stiffness values in Table 4.3A for 6d and 8d nails (common or galvanized box), respectively, to account for the effect of gypsum wallboard or gypsum sheathing between wood framing and wood structural panel sheathing. Tests of 3/8" wood structural panels over 1/2" and 5/8" gypsum wallboard support using lower nominal unit shear capacities associated with smaller nails (18).

C4.3.7.3 Particleboard Shear Walls: Panel size requirements are consistent with those for wood structural panels (see C4.3.7.1). Apparent shear stiffness in *SDPWS* Table 4.3A is based on assumptions of relative stiffness and nail slip (see C4.2.2 and C4.3.2). For closely spaced or larger diameter nails, staggered nail placement at each panel edge is intended to prevent splitting in the framing member (Figure C4.2.7).

C4.3.7.4 Structural Fiberboard Shear Walls: Panel size requirements are consistent with those for wood structural panels (see C4.3.7.1). Apparent shear stiffness in *SDPWS* Table 4.3A is based on assumptions of relative stiffness and nail slip (see C4.2.2 and C4.3.2). Minimum panel edge distance for nailing at top and bottom plates is 3/4" to

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match edge distances present in cyclic tests of high aspect ratio structural fiberboard shear walls (29).

C4.3.7.5 Gypsum Wallboard, Gypsum Veneer Base, Water-Resistant Backing Board, Gypsum Sheathing, Gypsum Lath and Plaster, or Portland Cement Plaster Shear Walls: The variety of gypsum-based sheathing materials reflects systems addressed in the model building code (2). Appropriate use of these systems requires adherence to referenced standards for proper materials and installation. Where gypsum wallboard is used as a shear wall, edge fastening (e.g. nails or screws) in accordance with *SDPWS* Table 4.3C requirements should be specified and overturning restraint provided where applicable (see *SDPWS* 4.3.6.4.2). Apparent shear stiffness in *SDPWS* Table 4.3C is based on assumptions of relative stiffness and nail slip (see C4.2.2 and C4.3.2). The nominal unit shear capacity and apparent shear stiffness values for plain or perforated gypsum lath with staggered vertical joints are based on results from cyclic tests (31). Unit shear capacity and apparent shear stiffness values are larger than those for plain or perforated gypsum lath where vertical joints are not staggered.

C4.3.7.6 Shear Walls Diagonally Sheathed with Single-Layer of Lumber: Diagonally sheathed lumber shear walls have comparable strength and stiffness to many wood structural panel shear wall systems. Apparent shear stiffness in *SDPWS* Table 4.3D is based on assumptions of relative stiffness and nail slip (see C4.2.2 and C4.3.2). Early reports (24) indicated that diagonally sheathed lumber shear walls averaged four times the rigidity of horizontally sheathed lumber walls when boards were loaded primarily in tension. Where load was primarily in compression, a single test showed about seven times the rigidity of a horizontally sheathed lumber wall.

C4.3.7.7 Shear Walls Diagonally Sheathed with Double-Layer of Lumber: Double diagonally sheathed lumber shear walls have comparable strength and stiffness to many wood structural panel shear wall systems. Apparent shear stiffness in *SDPWS* Table 4.3D is based on assumptions of relative stiffness and nail slip (see C4.2.2 and C4.3.2).

C4.3.7.8 Shear Walls Horizontally Sheathed with Single-Layer of Lumber: Horizontally sheathed lumber shear walls have limited unit shear capacity and stiffness when compared to those provided by wood structural panel shear walls of the same overall dimensions. Early reports (21 and 24) attributed strength and stiffness of lumber sheathed walls to nail couples at stud crossings and verified low unit shear capacity and stiffness when compared to other bracing methods.

C4.4 Wood Structural Panels Designed to Resist Combined Shear and Uplift from Wind

C4.4.1 Application

Panels with a minimum thickness of 7/16" and strength axis oriented parallel to studs are permitted to be used in combined uplift and shear applications for resistance to wind forces. Tabulated values of nominal uplift capacity (see *SDPWS* Table 4.4.1) for various combinations of nailing schedules and panel type and thickness are based on calculations in accordance with the *National Design Specification (NDS) for Wood Construction* and verified by full scale testing (39 and 40).

ASD and LRFD unit uplift and shear capacities are calculated as follows from nominal unit uplift and shear values.

ASD unit uplift capacity for wind, $u_{w(ASD)}$:

$$u_{w(ASD)} = \frac{u_w}{2.0} \quad \text{(C4.4.1-1)}$$

ASD unit shear capacity for wind, $v_{w(ASD)}$:

$$v_{w(ASD)} = \frac{v_w}{2.0} \quad \text{(C4.4.1-2)}$$

where:

2.0 = ASD reduction factor

LRFD unit uplift capacity for wind, $u_{w(LRFD)}$:

$$u_{w(LRFD)} = 0.65u_w \quad \text{(C4.4.1-3)}$$

LRFD unit shear capacity for wind, $v_{w(LRFD)}$:

$$v_{w(LRFD)} = 0.8v_w \quad \text{(C4.4.1-4)}$$

where:

0.65 = resistance factor, ϕ_z, for connections

0.8 = resistance factor, ϕ_D, for shear walls and diaphragms

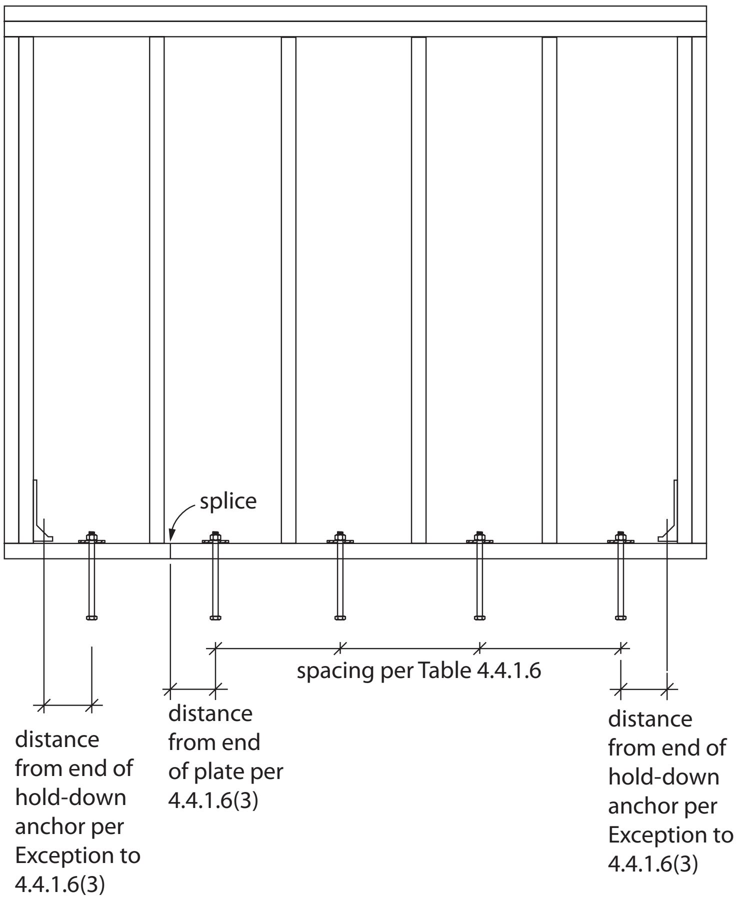
Examples C4.4.1-1 and C4.4.2-1 illustrate how the values in *SDPWS* Tables 4.4.1 and 4.4.2, respectively, were generated. Tabulated values of nominal uplift capacity in Table 4.4.1 and Table 4.4.2 are based on assumed use of framing with specific gravity, G, equal to 0.42. An increase factor is provided in table footnotes to adjust values for effect of higher specific gravity framing on the strength of the nailed connection between sheathing and framing. Where lower specific gravity framing is used, reduced values of nominal uplift capacity are applicable based on the effect of lower specific gravity framing on the strength of the nailed connection between sheathing and framing – for example, the reduction factor is 0.92 for framing with G = 0.35. Adjustment factors over a range of framing specific gravity can be determined as follows: Specific Gravity Adjustment Factor = [1-(0.5-G)]/0.92 for $0.35 \leq G \leq 0.49$.

C4.4.1.2 Panels: Full-scale testing (see C4.4.1) utilized panels with strength axis oriented parallel and perpendicular to studs. *NDS* nail connection capacities are independent of panel strength axis orientation.

C4.4.1.6. Sheathing Extending to Bottom Plate or Sill Plate: Construction requirements for use of wood structural panels to resist uplift and shear closely match construction present in verification tests. For example, testing of shear walls resisting uplift and combined uplift and shear used 16" o.c. and up to 48" anchor bolt spacing, 3"x3" plate washers, and nails with minimum 1/2" to 3/4" panel edge distance depending on the number of rows of nails. Anchor bolt spacing and size and location of plate washers were found to be important factors enabling strength of the sheathing to bottom plate connection to develop prior to onset of bottom plate failure. Required anchor bolt spacings as shown in Table 4.4.1.6 are a function of the combined uplift and shear forces on the bottom plate and are based on a combination of full-scale tests and numerical analysis (51, 52). Where other anchoring devices are used, it is intended that spacing not exceed the values provided in Table 4.4.1.6 and in addition that such devices enable performance of walls to be comparable to those tested with required anchor bolts and plate washers.

Figure C4.4.1A illustrates the anchor bolt spacing provisions in 4.4.1.6 and depicts the requirements of 4.4.6(3) regarding placement of anchor bolts at ends of plates.

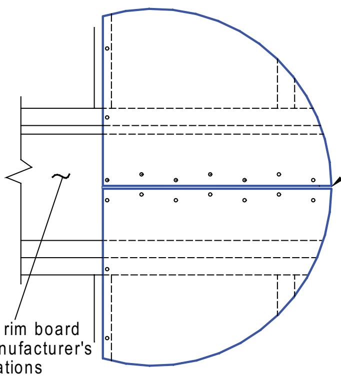
Figure C4.4.1A Anchor Bolt Placement Requirements For Wood Structural Panels Designed to Resist Combined Shear and Wind Uplift



C4.4.1.7 Sheathing Splices: In multi-story applications where the upper story and lower story sheathing adjoin over a common horizontal framing member, the connection of the sheathing to the framing member can be designed to maintain a load path for tension and shear. It is recognized that wood is directly stressed in tension perpendicular to grain in some details; however, those cases are prescriptively permitted and also limited to nail size and spacing verified by testing. Splice panel orientation does not affect capacity of the sheathed tension splice joint and therefore panel orientation can be either parallel or perpendicular to studs.

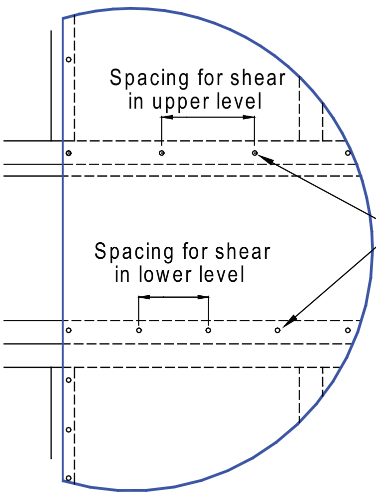
C4.4.1.7(1) Where sheathing edges from the upper and lower story meet over a common horizontal framing member, wood stressed in tension perpendicular to grain is relied upon directly to maintain load path for tension (Figure C4.4.1B). The location of sheathing splices need not occur at mid-height of the horizontal framing. Wall height, floor depth, available panel lengths, and maintaining minimum edge distances between sheathing nails and framing will influence the practical location of the sheathing splice in the horizontal framing member. Wood member stresses in this application are limited to that which can be developed with nail spacing to 3" o.c. (minimum) for a single-row and 6" o.c. (minimum) for a double-row at each panel edge based on results from testing. Limiting tension stresses perpendicular to grain in horizontal framing members is accomplished by limiting nail spacing to 3" o.c. (minimum) for a single-row and 6" o.c. (minimum) for a double-row. This limitation does not preclude use of more closely spaced nails where the horizontal framing member is an engineered rim board or similar product that can resist higher induced tension stresses perpendicular to grain. Follow manufacturers' recommendations for minimum nail spacing permitted for this application.

Figure C4.4.1B Panel Splice Over Common Horizontal Framing Member



C4.4.1.7(2) The panel splice across studs detail in Figure 4I relies on increased nailing between vertical framing (e.g. studs) and sheathing to transfer tension forces while shear is transferred through nailed connections to horizontal framing such as horizontal blocking. This detail assumes no direct loading of framing members in tension perpendicular to grain for development of the tension load path. Additional nailing between sheathing and vertical framing on each side of the panel splice maintains load path for tension. Where the panel is continuous between stories, as shown in Figure 4I, one option to maintain load path for shear utilizes attachment of sheathing to wall plate framing as shown in Figure C4.4.1C .

Figure C4.4.1C Detail for Continuous Panel Between Levels (Load Path for Shear Transfer Into and Out of the Diaphragm Not Shown)



Continous panel between levels

C4.4.2 Wood Structural Panels Designed to Resist Only Uplift from Wind

Panels with a minimum thickness of 3/8" are permitted to be used in this application to resist uplift from wind only when panels are installed with the strength axis parallel to studs (see *SDPWS* 4.4.1 for provisions on resistance to combined shear and uplift from wind). Tabulated unit uplift capacities are applicable for wood structural panels with 3/8" and greater thickness. For applications where panel strength axis is oriented perpendicular to studs, manufacturer recommendations should be followed.

EXAMPLE C4.4.1-1 Calculate Nominal Uplift Capacity for Combined Uplift and Shear Case

Calculate the nominal uplift capacity in *SDPWS* Table 4.4.1 for a wood structural panel shear wall constructed as follows:

Sheathing grade	=	Structural I (OSB)
Nail size	=	10d common (0.148" diameter, 3" length)
Minimum nominal panel thickness	=	15/32"
Nailing for shear	=	6" panel edge spacing (2 nails per foot), 12" field spacing
Alternate nail spacing at top and bottom plate edges	=	3" (single row, 4 nails per foot)
Nails available for uplift	=	Nails from alternate nail spacing − Nails available for shear only
	=	4 nails per foot − 2 nails per foot = 2 nails per foot

Z = 82 lb *NDS* Table 12Q (Main member: G = 0.42 (SPF), Side member: 15/32" OSB)

C_D = 1.6 (*NDS* Table 2.3.2)

Z' = 82 lb x 1.6 = 131 lb

Allowable uplift capacity = 131 lb x 2 nails/foot = 262 plf

Nominal uplift capacity = 262 plf x ASD reduction factor

Nominal uplift capacity = 262 plf x 2 = 524 plf (*SDPWS* Table 4.4.1)

When subjected to combined shear and wind uplift forces, the calculation for nominal uplift capacity is based on the assumption that nails resist either shear or wind uplift forces.

EXAMPLE C4.4.2-1 Calculate Nominal Uplift Capacity for Wind Uplift Only Case

Calculate nominal uplift capacity, in *SDPWS* Table 4.4.2 for wood structural panel sheathing over framing constructed as follows:

Sheathing grade	=	Structural I (OSB)
Nail size	=	10d common (0.148" diameter, 3" length)
Minimum nominal panel thickness	=	3/8"
Alternate nail spacing at top and bottom plate edges	=	3" (single row, 4 nails per foot)
Nails available for uplift	=	Nails from alternate nail spacing
	=	4 nails per foot

Z = 78 lb *NDS* Table 12Q (Main member: G = 0.42 (SPF), Side member: 3/8" OSB)

C_D = 1.6 (*NDS* Table 2.3.2)

Z' = 78 x 1.6 = 125 lb

Allowable uplift capacity = 125 lb x 4 nails/ft = 500 plf

Nominal uplift capacity = 500 plf x ASD reduction factor

Nominal uplift capacity = 500 plf x 2 = 1,000 plf (*SDPWS* Table 4.4.2)

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American Wood Council

AWC Mission Statement

To increase the use of wood by assuring the broad regulatory acceptance of wood products, developing design tools and guidelines for wood construction, and influencing the development of public policies affecting the use and manufacture of wood products.

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