2015 Special Design Provisions for Wind and Seismic

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The 2015 Edition of *Special Design Provisions for Wind and Seismic* (SDPWS) was approved as an American National Standard on September 8, 2014, with the designation ANSI/AWC SDPWS-2015 (*Figure 1*). The 2015 SDPWS was developed by AWC's Wood Design Standards Committee (WDSC) and contains provisions for design of wood members, fasteners, and assemblies to resist wind and seismic forces. Some of the more notable revisions include the following, which are explained in more detail below (see *Table 1,* for a summary of changes by Chapter):

- Expanded applicability of the wall stud repetitive member factor to stiffness values (EI) for calculating stud out-of-plane deflection under wind load for stud spacing up to 24 inches on center
- Added a new section on wind uplift force resisting systems
- Added a new section on seismic anchorage of concrete/masonry structural walls to wood diaphragms consistent with ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
- Updated diaphragm flexibility terminology to coordinate with ASCE 7-10
- Added "envelope" analysis as an alternative to "semi-rigid" diaphragm analysis for horizontal distribution of shear to vertical resisting elements (i.e., shear walls)
- Consolidated formerly separate provisions for design of open front structures and design of cantilevered diaphragms under Section 4.2.5.2 for design of open front structures with cantilevered diaphragms
- Added minimum 3-inch nominal depth requirement for framing and blocking used in high load blocked diaphragms consistent with the *International Building Code* (IBC)
- Updated provisions for distribution of shear to shear walls in a line to clearly address stiffness compatibility and to clarify basis of the familiar $2b_s/h$ factor
- Added a new shear wall strength reduction factor for high aspect ratio wood structural panel shear walls applicable for wind and seismic design
- Added a new method to account for strength of high aspect ratio perforated shear wall segments
- Added a new table for anchor bolt spacing along the bottom plate of wood structural panel shear walls designed to resist combined shear and wind uplift

Repetitive Member Factor Applied to Stiffness

Section 3.1.1.1 was revised to permit application of the wall stud repetitive member factor for stiffness, EI, in calculation of out-of-plane deflection of wall studs for stud spacing up to 24 inches on center. The values of the repetitive member factor remain unchanged, ranging from 1.5 for 2x4 studs to 1.15 for 2x12 studs, and are applicable only where certain specific conditions are met including use of blocked wood structural panel sheathing.

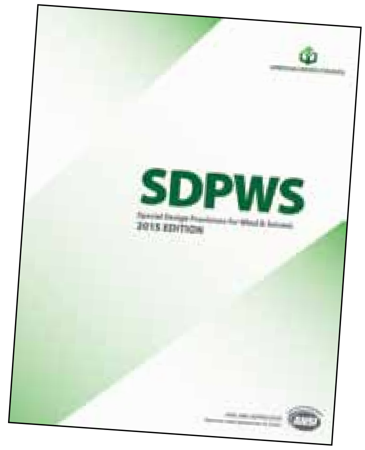

Figure 1. 2015 SDPWS is referenced in the 2015 International Building Code.

Wind Uplift Force Resisting Systems

New Section 3.4 addresses wind uplift resistance and is based on general requirements of structural design to provide load path for structural loads. It describes general design considerations for proportioning, designing, and detailing members and connections resisting wind uplift. For example, load path elements must have adequate strength and stiffness, and their design must also account for additional forces and deflections resulting from eccentricities in the uplift load path.

Anchorage of Concrete or Masonry Structural Walls to Wood Diaphragms

New Section 4.1.5.1 addresses the anchorage of concrete or masonry structural walls to wood diaphragms for seismic forces consistent with provisions of ASCE 7-10 Section 12.11 and prior editions of the building code where they originally appeared. The requirements in SDPWS include those for continuous ties, use of the subdiaphragm concept, and prohibition of anchorage force transfer through wood framing that could induce cross grain bending and cross grain tension (*Figure 2*).

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Table 1. Summary of Changes in 2015 SDPWS

Chapter 1 – No Changes

Chapter 2 – General Design Requirements

1) Add section 2.1.3 Sizes for how WSP nominal thickness relates to Performance Category in PS1 and PS2.
2) Definition for "subdiaphragm" is added to coordinate with new Section 4.1.5.1.
3) Remove definitions for Flexible and Rigid Diaphragms to avoid conflicts with varying definitions in ASCE 7-10.
4) Add definition for "open front structure" to coordinate with section 4.2.5.2.
5) Add notation for L′ and W′ for cantilevered diaphragms. Coordinating changes are made in notation for L and W and include deletion of L_c.
6) Revise definition for collectors to clarify use for transfer of diaphragm shear forces to shear walls.

Chapter 3 – Members and Connections

1) Revise 3.1.1.1 Wall Stud Bending Design Value Increase to permit wall stud repetitive member factor for stiffness and for studs spaced up to 24 inches o.c.
2) Revise values and footnotes in Tables 3.2.1 and 3.2.2 to reflect that 3-ply plywood is not commercially available for thicker panels.
3) Revise Table 3.2.2 to add a case for roof sheathing strength axis parallel to supports to address a common technique in panelized roof construction.
4) Add new section 3.4 and modify section 3.2.1 to address wind uplift force resisting systems.

Chapter 4 – Lateral Force-Resisting Systems

1) Add new section 4.1.5.1 to address seismic anchorage of concrete or masonry structural walls to wood diaphragms consistent with ASCE 7-10.
2) Revise section 4.2.5 Horizontal Distribution of Shear to use terms "idealized as flexible", "idealized as rigid", and "semi-rigid" consistent with ASCE 7-10.
3) Revise section 4.2.5.1 Torsional Irregularity to clarify requirements and improve consistency with ASCE 7-10.
4) Consolidate Open Front Structures and Cantilevered Diaphragms into Section 4.2.5.2 Open Front Structures to clarify requirements and improve consistency.
5) Revise section 4.2.7.1.2 High Load Blocked Diaphragms to add minimum 3" nominal depth of framing/blocking.
6) Replace diaphragm configuration figures in Tables 4.2A, 4.2B 4.2C to illustrate that diaphragm resistance is dependent on the direction of continuous panel joints with respect to loading direction as well as direction of framing members, but is independent of the panel orientation.
7) Revise column headings in Table 4.2C to clarify 6" nail spacing is for supported panel edges.
8) Add new section 4.3.2.3 Deflection of Structural Fiberboard Shear Walls for such walls with $h/b_s > 1.0$.
9) Revise section 4.3.3.4 and add new section 4.3.3.4.1 to establish equal deflection as the general requirement for distribution of shear to shear walls in a line. An Exception permits distribution of shear in proportion to shear capacity when certain conditions are met.
10) Revise section heading 4.3.4 to add "and Capacity Adjustments" to reflect section content.
11) In section 4.3.4.3, adjust the length of each perforated shear wall segment with h/b_s exceeding 2:1 by $2b_s/h$.
12) Add a new section 4.3.4.2 for strength adjustment for high aspect ratio walls.
13) Revise 4.3.5.1 Individual Full-Height Wall Segments to remove reference to shear wall line.
14) Revise 4.3.5.3 to clarify materials requirements for the perforated shear wall design method.
15) Add new section 4.3.6.1.1 Common Framing Member permitting (2) 2x framing members to replace a 3x framing member and reference from 4.3.7.1(4) WSP Shear Walls and 4.3.7.3(4) Particleboard Shear Walls.
16) Revise 4.4.1 Application to clarify that the walls are designed to resist wind uplift and not the sheathing only.
17) Revise section 4.4.1.2 Panels to address panels with the strength axis parallel or perpendicular to studs.
18) Revise 4.4.1.6(2) by permitting increased anchor bolt spacing in accordance with new Table 4.4.1.6 for wood structural panel shear walls designed to resist combined shear and wind uplift.
19) Add new 4.4.1.6(3) to provide a minimum end distance for anchor bolts used for wood structural panel shear walls designed to resist combined shear and wind uplift.

Appendix A – None

References – Update References

Diaphragm Flexibility Terminology

In Section 4.2.5, diaphragm flexibility terminology was revised to utilize the terms "idealized as flexible," "idealized as rigid," and "semi-rigid" consistent with ASCE 7-10. The condition for which a wood diaphragm is permitted to be idealized as rigid (e.g. in-plane deflection of the diaphragm is less than or equal to two times the average deflection of adjoining vertical elements) remains unchanged from prior editions of SDPWS. 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. These idealizations are employed to simplify structural analysis for distribution of horizontal diaphragm shear loads.

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 diaphragm idealized flexible and one for diaphragm idealized as rigid, the envelope analysis provides a conservative alternative means of shear distribution and avoids calculation effort associated with semi-rigid diaphragm modeling. Specific recognition of the envelope analysis method is new in SDPWS Section 4.2.5.

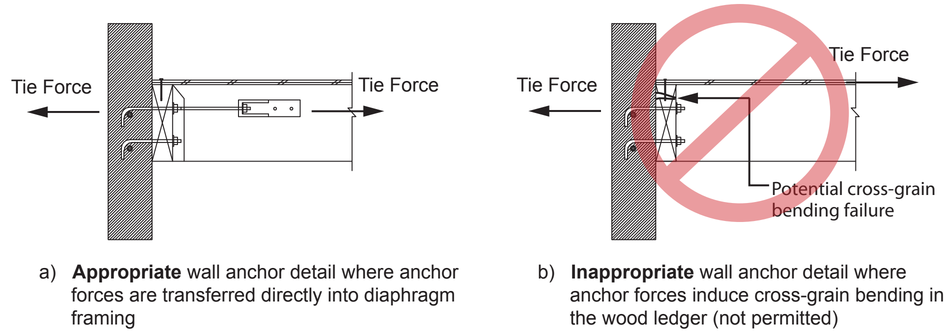

Figure 2.

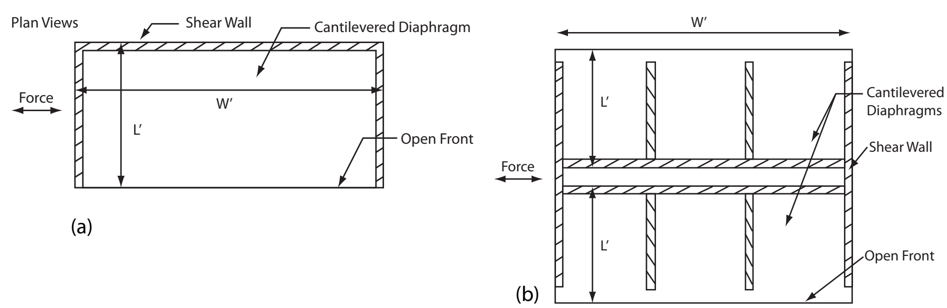

Figure 3. Excerpt from 2015 SDPWS new Figure 4A showing examples of open front structures. In this case, an example of a) a simple open front structure with walls on three sides only and b) a simple corridor wall structure.

Torsional Irregularity and Open Front Structures

Revised provisions of 4.2.5.1 (Torsional Irregularity) and 4.2.5.2 (Open Front Structures) reflect efforts to clarify requirements that include use of terminology that is more consistent with ASCE 7. A coordinated hierarchy of requirements has been established for seismic design whereby open front structures with cantilevered diaphragms are subject to increased limitations on story drift and building configuration when compared to provisions for torsionally irregular structures that are not open front. Open front structures must rely on diaphragm rigidity for distribution of forces to vertical elements of the seismic force resisting system by diaphragm rotation. Such structures 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. A structure with shear walls on three sides only (open front) is one simple form of an open front structure; however, open front structure requirements are applied to alternative forms employing cantilevered diaphragms (*Figure 3*).

Revised provisions of 4.2.5.2 (Open Front Structures) remove ambiguity from prior editions of SDPWS, primarily by consolidation of separate sets of provisions previously applicable to structure types described as either "open front" or "cantilevered diaphragm." Under new provisions of 4.2.5.2, open front structures with cantilevered diaphragms are subject to increased limitations relative to torsionally irregular structures that are not open front. For example, open front structures with cantilevered diaphragms are subject to the following design limitations:

- for loading parallel to the open side, the maximum story drift at each edge of the structure shall not exceed the ASCE 7 allowable story drift regardless of whether a torsional irregularity is present
- where a torsional irregularity is present, the L′/W′ ratio shall not exceed 0.67:1 for structures over one-story in height, and 1:1 for structures one-story in height
- the cantilevered diaphragm length, L′, (normal to the open side) shall not exceed 35 feet.

An exception to Section 4.2.5.2 exempts small cantilevers having L′ of 6 feet or less as a practical approach to avoid unnecessarily triggering special open front provisions where cantilevers are small. Similarly, provisions of new Section 4.2.5.2.1 permit simplification of analysis by allowing the use of the idealized as rigid diaphragm assumption for relatively small one-story structures with diaphragm span not more than 25 feet and the L′/W′ ratio not more than 1. While 4.2.5.2 provides requirements specific to wood diaphragms in open front structures, these are in addition to and not a replacement of seismic design criteria of ASCE 7 (*Figure 4*).

High Load Blocked Diaphragms

Section 4.2.7.1.2 on high load blocked diaphragms now clarifies requirements for

Figure 4. 2015 SDPWS includes provisions to coordinate with ASCE 7-10.

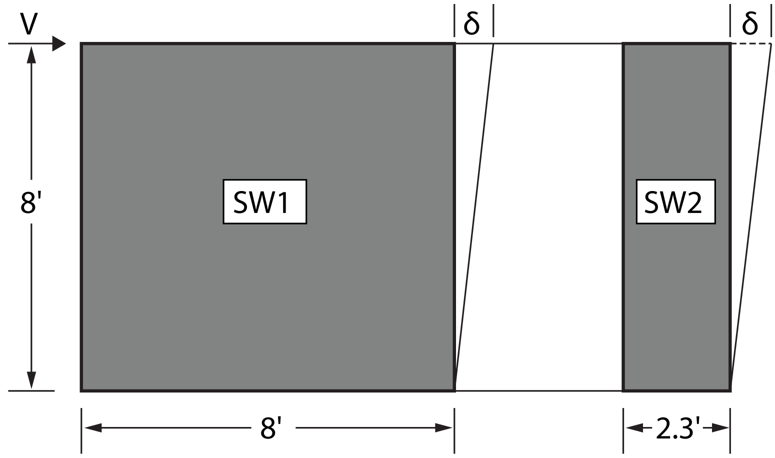

Figure 5. Distribution of shear to shear walls in a line to produce equal deflection in each shear wall is the underlying basis of the familiar $2b_s/h$ factor.

minimum depth of framing members and blocking consistent with similar provisions for stapled high load diaphragms in 2015 IBC Table 2306.2(2) footnote (e). Section 4.2.7.1.2 item 4 states: "The depth of framing members and blocking into which the nail penetrates shall be 3 inches nominal or greater."

Distribution of Shear to Shear Walls in a Line

Provisions of Section 4.3.3.4.1 contain the equal deflection requirement for distribution of shear to shear walls in a line. While this concept is not new to SDPWS, the organization of requirements pertaining to distribution of shear to shear walls in a line is new.

New Section 4.3.3.4.1 states that "Shear distribution to individual shear walls in a shear wall line shall provide the same calculated deflection, δ_{sw}, in each shear wall." At a given deflection, the force in each wall is determined by multiplying the wall stiffness times the deflection (commonly referred to as distribution based on relative stiffness or the equal deflection approach). A simplified approach permits distribution of shear in proportion to the nominal shear capacities of the individual full-height wall segments, provided that certain requirements are met. For wood structural panel shear walls, distribution of shear in proportion to the nominal shear capacity of each shear wall segment is permitted provided that the nominal shear capacity is adjusted by a factor of $2b_s/h$ for wall segments with aspect ratios greater than 2:1. This factor is based on reduced stiffness observed from testing and provides roughly similar results to the equal deflection calculation method for a reference wall line configuration comprised of a 1:1 aspect ratio shear wall and a 3.5:1 aspect ratio shear wall, as depicted in *Figure 5*. Whether there is a strength benefit in one method over the other depends on the specific wall configuration under consideration.

A common misunderstanding of the $2b_s/h$ factor was that it represented an actual reduction in unit shear capacity for high aspect ratio shear walls. The actual strength reduction associated with high aspect ratio shear walls is less severe and addressed by new Section 4.3.4.2. The $2b_s/h$ factor accounts primarily for stiffness compatibility of the high aspect ratio segment. Where $2b_s/h$ is used to comply with load distribution requirements of Section 4.3.3.4.1, the strength reduction adjustments of 4.3.4.2 for high aspect ratio shear wall segments need not be applied.

Strength Adjustment Factor for High Aspect Ratio Walls

As noted previously, Section 4.3.4.2 contains a new strength adjustment factor to account for the decreased unit shear capacity of high aspect ratio wood structural panel shear walls. The new factor, $1.25 - 0.125\ h/b_s$, is applicable to shear walls with an aspect ratio greater than 2:1. As previously noted, where distribution of shear is based on the simplified alternative adjustment factor methods (e.g. $2b_s/h$ for wood structural panels), further reduction of shear strength by the aspect ratio factors in 4.3.4.2 is not required.

Requirements of 4.3.4.2 are in addition to those in 4.3.3.4.1 to ensure deflection compatibility between shear walls in a line and, therefore, the smaller of the design capacities associated with requirements of 4.3.4.2 and 4.3.3.4 is to be used as the controlling design capacity for each individual shear wall.

High Aspect Ratio Perforated Shear Wall Adjustments

Provisions for accounting for the strength contribution of high aspect ratio shear wall segments within a perforated shear wall have been revised. In prior editions of SDPWS, where a high aspect ratio perforated shear wall segment (e.g. h/b_s>2:1) was considered in the calculated strength of the perforated shear wall, the shear capacity of the overall perforated shear wall required adjustment by the $2b_s/h$ factor. The revised provisions of Section 4.3.4.3 allow the adjustment to apply only to the high aspect ratio perforated shear wall segments, based on stiffness compatibility considerations, as opposed to the calculated strength of the overall perforated shear wall. Because the more severe $2b_s/h$ factor is used, unit shear values of high aspect ratio shear wall segments within a perforated shear wall are not required to be adjusted by the aspect ratio factors of Section 4.3.4.2. While this revised method will generally permit increases in design strength of perforated shear walls incorporating high aspect ratio segments, there are cases where there is little change such as perforated shear walls comprised entirely of identical high aspect ratio perforated shear wall segments.

Anchor Bolt Spacing for Combined Shear & Wind Uplift

Section 4.4.1.6(2) regarding anchorage of bottom plates and sill plates to resist combined uplift and shear was revised to permit determination of anchor bolt spacing in accordance with a new Table 4.4.1.6 (*Figure 6*) developed based on testing and analysis. Previously, an anchor bolt spacing of 16 inches on center, associated with the maximum wind uplift

Table 4.4.1.6 Maximum Anchor Bolt Spacing (inches) for Combined Shear and Wind Uplift[1,2]

Nail Size	Nominal Unit Shear Capacity (plf)		Nominal Uplift Capacity (plf)										
	G=0.50		0	216	432	648	864	1080	1296	1458	1728	1944	2160
		G=0.42	0	200	400	600	800	1000	1200	1350	1600	1800	2000
8d common (0.131" x 2-1/2")	0	0	48[3]	42	36	36	32	24	24	19.2	16	-	-
	400	368	48	42	36	36	32	24	24	19.2	16	-	-
	670	616	36	32	24	24	24	24	19.2	19.2	16	-	-
	980	902	24	24	19.2	19.2	19.2	16	16	-	-	-	-
10d common (0.148" x 3")	0	0	48[3]	42	36	36	32	24	24	19.2	16	16	16
	400	368	48	42	36	36	32	24	24	19.2	16	-	-
	870	800	24	24	24	19.2	19.2	19.2	16	16	-	-	-

Figure 6. Excerpt of 2015 SDPWS new Table 4.4.1.6.

capacity, was prescribed. Using new Table 4.4.1.6, anchor bolt spacing varies from 16 to 48 inches on center, based on the nominal uplift capacity of the wood structural panel sheathing or siding.

Conclusion

The 2015 SDPWS is currently available as a free download in electronic format (PDF) as a non-printable read-only document, and a printable electronic version is available for purchase (**www.awc.org**). Additional information on SDPWS provisions is available in the SDPWS Commentary. The 2015 SDPWS Commentary is scheduled to be available in June 2015. The 2015 SDPWS represents the state-of-the-art for design of wood members and connections to resist wind and seismic loads. Reference to the 2015 SDPWS in the 2015 IBC will make it a required design standard in those jurisdictions adopting the latest building code.▪

A note from the NCSEA Code Advisory Committee: NCSEA, through its member organizations (MOs), often contributes to the basis of code changes. SEAOC (the California MO of NCSEA) publishes articles that comprise the "Blue Book" a significant source of engineering consideration of code provisions and important earthquake engineering issues. For more information, please explore **www.seaoc.org/bookstore**.