September 23, 2013

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Mr. BJ Yeh
Mr. Tom Skaggs
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RE: SBCA/SBCRI Fundamental Unit Shear Capacity Equivalency Benchmarking -- Wood Structural Panel Unit Shear Capacities from SBCRI testing for Special Design Provisions Wind and Seismic (SDPWS)/International Building Code (IBC) and International Residential Code (IRC) Applications

Dear Ed (Elias), BJ, Tom, Ed, Brad, Phil and Buddy:

First we would be remiss if we did not say congratulations to Ed (Elias) on his promotion to the Presidency of APA. Based on our past discussions and your forthrightness, APA's future is in good hands. Again, thank you so much for hosting our meeting on January 3 and your follow-up letter on February 6 (Attachment A); both are very insightful and everything discussed remains even more relevant today.

I am including AWC staff in this letter since they have an interest/stake in this subject area as well.

We apologize in advance for all the information provided here, but we believe that it is very important to be transparent and comprehensive with information and references that will be used to establish a level playing field engineering based approach to shear wall resistance design. The purpose of this letter is to be precise as to what the SBC industry is going to set as its engineering foundation/performance benchmarks for wood structural panel (WSP) unit shear wall capacity values and section 104.11 equivalency evaluation. This evaluation is based on all of the testing and findings to date that we have access to. We believe that it is important to be completely transparent in our approach so that APA/AWC can provide engineering mechanics test data/analysis and installation procedures to provide justification for different benchmarks than those provided here, if so desired. As you said in your February letter, “The technical information required should support or be used to modify existing code supported provisions such as established systems or risk factors related to product equivalency.” This work is clearly in the domain of APA/AWC.

What follows is the technical information that supports “existing systems factors” that have been adopted into the code and advocated by APA/AWC in both the SDPWS and code development environments; essentially being APA/AWC unit shear capacity values. Just to be clear we have no interest in waiting for the code to be changed with respect to using current knowledge, as the “effective date” of unit shear capacity values is when one has good test data that define WSP capacities accurately. This information has been provided by SBCA in several forms beginning in August of 2011. This is our communication with respect to our go forward approach based on the knowledge that we are providing here.

Accurate engineering mechanics based and testing confirmed performance knowledge should always be used right away, particularly if the data causes one to have better information with respect to making resistance engineering decisions. Therefore we have been using this information as we have gained past and now current test knowledge. Our engineering approach has been used in the context of the code-defined (SDPWS/IRC) unit shear capacity values, which we assume APA/AWC stand behind. We assume this since both organizations are the creators of these unit shear capacity values and both have aggressively advocated for them in the code development process that become code-based law.

From this point forward, we will undertake all our testing and engineering analysis based equivalency evaluations using APA/AWC generated benchmarks so that there is a level engineering playing field. Our goal is to provide value for engineering based solutions that have equivalent performance characteristics so that all products performing equivalently have access to the same WSP factors that APA/AWC have placed into law. This will be done with test based and analytical boundary conditions, so that there is reliable and comparable performance to SDPWS and IRC defined performance. Like all engineers, when we believe that we have rock solid design properties we will stand behind them with our seals/certification. We have full confidence in the engineering work of all of our technical staff.

As stated previously many times and in many different venues, our industry believes that engineering mechanics and engineering resistance should be completely understandable with respect to its derivation and consistent no matter what the application is (e.g., braced wall, drag element, lumber studs, LSL joist and rafter connections, etc.) or where the application takes place (e.g., Wisconsin, California, Washington, Florida, etc.). The structure resisting the load does not care what type of load is being applied or where it resides; it merely resists loads based on the combination of materials from which it is made. Applied load resistance should then simply be an accurate allowable design resistance based on testing and subsequent modeling[1] along with a reasonable factor of safety, which APA/AWC have defined through SDPWS and other documents, and which have been confirmed by APA in several venues and written statements.

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[1] This is also generally called the scientific method that all universities use – test, model, compare the model to the test data, evaluate goodness of fit, establish boundary conditions of use, re-check and advance an industry or advance an innovation through good science based engineering. This does not take a standards generating or code bureaucracy to do as if it did innovation and forward progress would be stuck in “status quo” forever. Just ask any university professor if they feel constrained by standards and codes or any group that believes in some way they have attained “god-like” status. By what authority?. We suspect everyone will find that answer is an emphatic no. Good engineers should always be allowed to advance the value of the engineering profession through science based innovation and creative engineering and stand behind their individual work base on their professional expertise.

This information, when placed into the public domain, such as through the building code, should even more so be transparent, understandable, easy to derive or easy to understand the derivation of the base design values and associated factors. In general one also assumes that the public design values will be conservative in approach depending on the precision of information available, given that if adopted by the building code these design values then become law.

To this end, we have had a long-standing industry policy that has recently been updated and approved by the SBCA Board of Directors. Please see Appendix A below as it has become very clear that this policy is central to our long-term success as an engineering-based industry. We sincerely believe that the structural building component industry will increasingly become the "center of the universe" of light frame structural resistance engineering, and with that goes our desire to increase the market value of engineering based design in contrast to prescriptive based design.

As part of a benchmarking process, SBCRI[2] has performed 239 wood structural panel (WSP) based shear wall tests of all types and configurations (e.g., fully sheathed, perforated, 4x8 and 8x8 with anchor bolts (IRC), 4x8 and 8x8 segmented (SDPWS/IBC), etc.). We have done this because we have seen performance that is different than what our expectations were heading into testing WSPs as a lateral load resisting elements.

All this data is proprietary knowledge that is quite valuable. As part of the testing we have performed over the last four years, we have tested 49 WSP segmented shear walls (i.e. hold-downs at each end of the walls 4 or 8 feet in length) following ASTM E564 or ASTM E2126 techniques. We have also tested an equivalent number of IRC based intermittent braced wall panels. ASTM E564 states "Load distribution along the top edge of the wall shall simulate floor or roof members that will be used in the actual building construction." Therefore, actual roof truss elements were used to distribute the load to the shear wall for all of our E564 and E2126 shear wall tests, which we believe accurately simulates product performance as installed in actual building construction. All our work is cross calibrated to our 12x30 foot code compliant building/test structure, so we believe that our data solidly represents true WSP lateral load resistance performance for both the IRC and SDPWS/IBC. Please see Appendix B for a view of our past and present testing approaches.

The WSP walls tested were sheathed with $^{3}/_{8}$", $^{7}/_{16}$", or $^{15}/_{32}$" OSB (sheathing category) fastened with either 8d common (2 ½" x 0.131") or 8d box (2 $^{3}/_{8}$" x 0.113") nails to SPF framing members spaced 16" on center. The nails were spaced 6" o.c. along the panel edges and 12" o.c. in the field. The nails were installed using a minimum of a 3/8" edge distance that was chalk-lined on the product so each nail had SDPWS/code compliant installation, which by definition will provide a test facility unit shear capacity versus a field installed unit shear capacity. Table 1 shows the variation in the shear wall capacity for the two different nails used in our segmented shear wall tests.

[2] This testing has been paid for by Qualtim, Inc. to ensure accuracy of its engineering activities. Qualtim has granted SBCA/SBCRI the exclusive right to use this data to improve the knowledge of the SBC industry regarding shear wall performance to enhance the design and use of walls panels and to provide a foundation for innovation within the engineered wall panel marketplace.

SBCRI Test Data (49 Tests Performed)	Lateral Unit Shear Wall Resistance Capacity from Segmented Shear Wall Testing for 8d Box (2 3/8" x 0.113") Nail (PLF)	Lateral Unit Shear Wall Resistance Capacity from Segmented Shear Wall Testing for 8d Common (2 ½" x 0.131") Nails (PLF)
Minimum	426	538
Maximum	726	818
Range	300	280

Table 1: Range of Shear Wall Capacities

Please review Table 1 in the context of the following documents:

1. Attachment B – APA Report TT-087B Shear Wall Test Results Comparing 8d Common and 8d Box Nails.
2. Attachment C – APA Report T2004-14 Wood Structural Panel Lateral and Shear Wall Connections with Common, Galvanized Box, and Box Nails
3. Attachment D – Special Design Provisions Wind and Seismic (SDPWS) Table 4.3A.
4. Attachment E – SDPWS Appendix Table A1.

Should an evaluation be made using traditional lumber design value thought processes (e.g., 5th percentile statistics) this data would result in a much more conservative design value than is now codified into law via SDPWS and the IRC.

Figures 1 and 2 below provide a histogram showing the distribution of the data for each nail size.

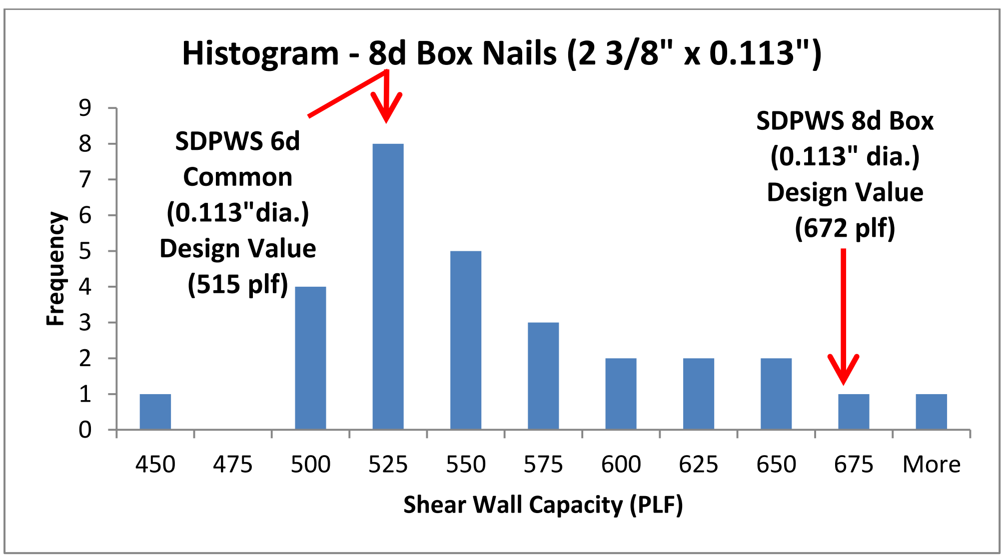

Figure 1: Histogram of Tested Shear Wall Capacity for 8d Box (2 3/8" x 0.113") Nails (Per SDPWS, 3/8" WSP with SPF framing 16" o.c. has a design value of 515 plf for 6d common (2" x 0.113") nails and 672 plf for 8d box (2 ½" x 0.113") nails)

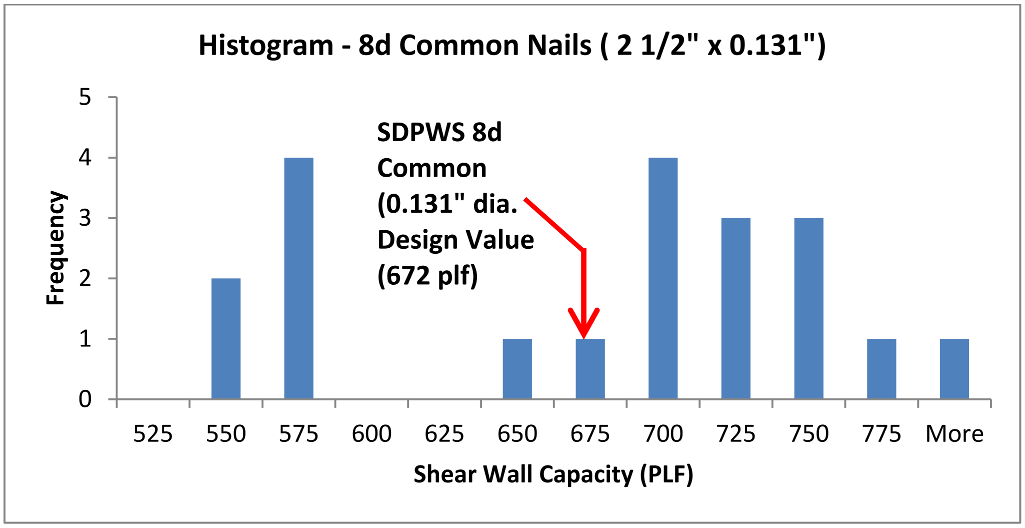

Figure 2: Histogram of Tested Shear Wall Tests for 8d Common (2 1/2" x 0.131") Nails (Per SDPWS, 3/8" to 15/32" WSP with SPF framing 16" o.c. has a design value of 672 plf for 8d common (2 ½" x 0.131") nails)

The load-deflection curves for the SBCRI testing are shown in Figures 3 and 4.

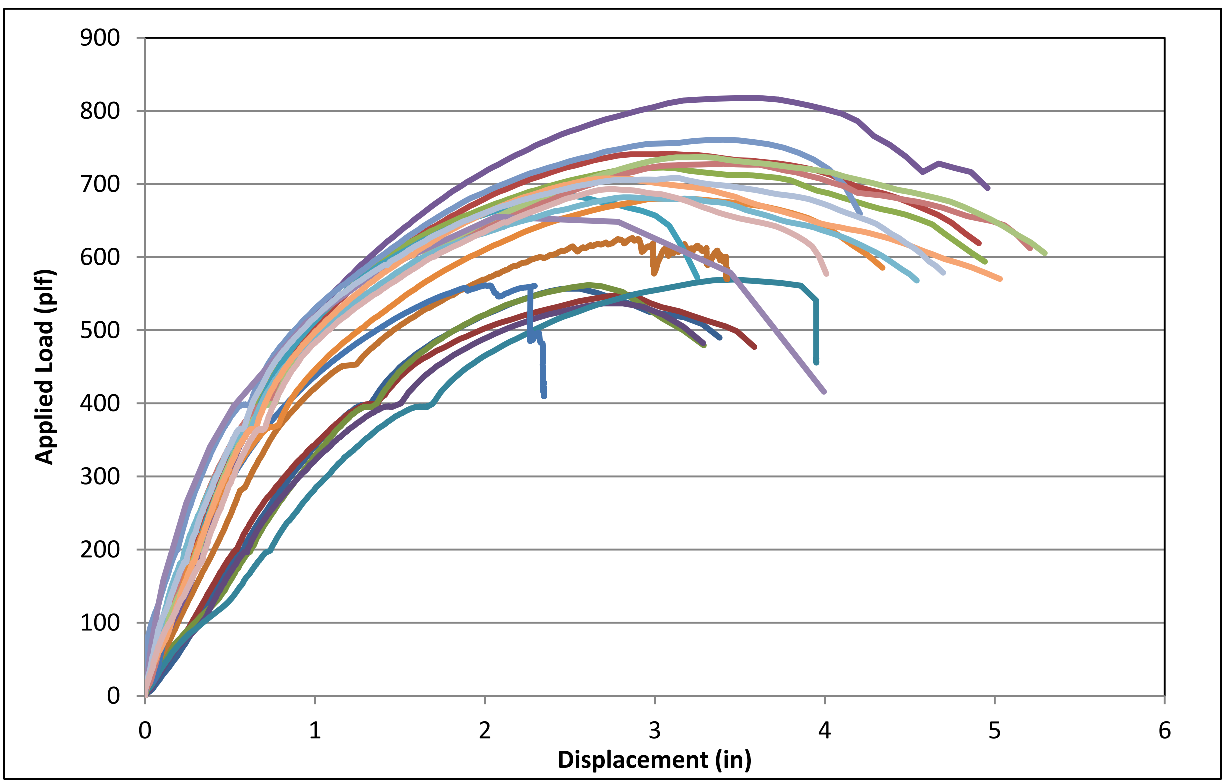

Figure 3: Load-Deflection Plots for 8d Common (2 1/2" x 0.131") Nail Shear Wall Tests

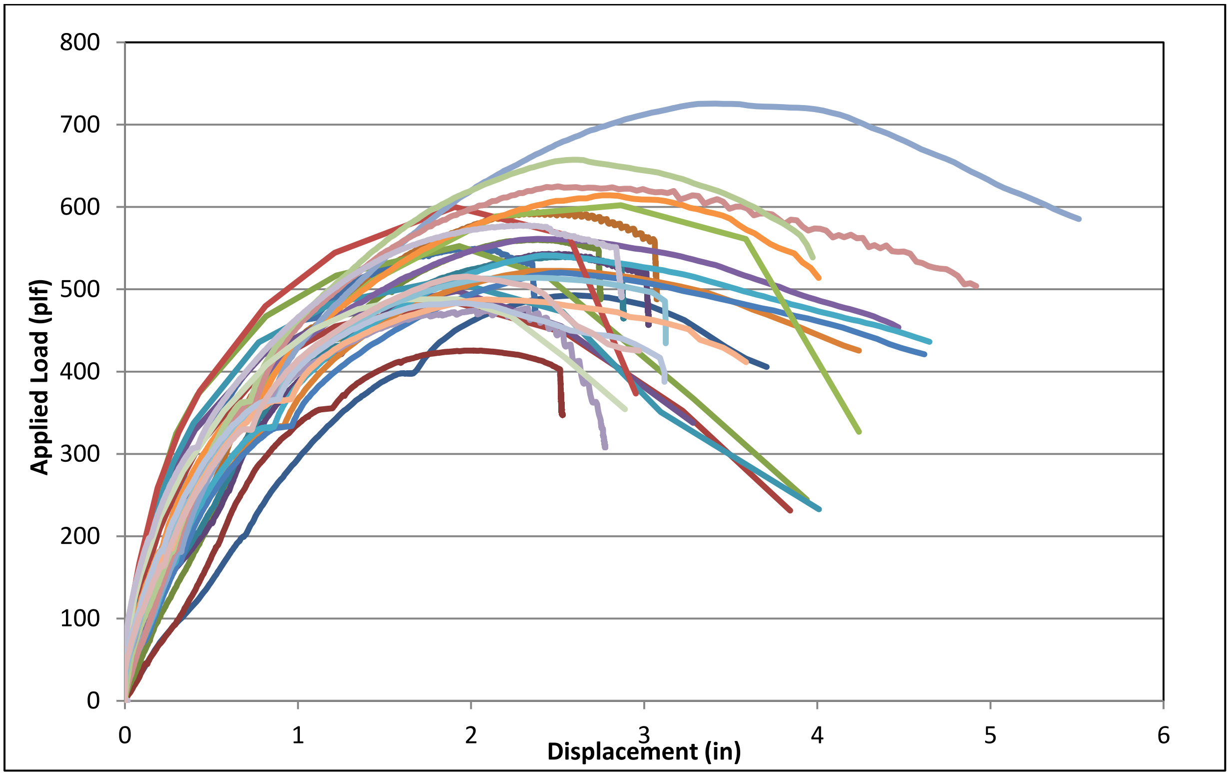

Figure 4: Load-Deflection Plot of 8d Box (2 3/8" x 0.113") Nail Shear Wall Tests

As can be seen from [Table 1] and [Figures 1] through [4,] there is a significant amount of variability in the shear resistance capacity of the tested walls and our recent testing continues to confirm that this is the case for both segmented and intermittent WSP testing.

As you all are very aware, there are a myriad of factors that cause variation in the capacity of shear walls (a.k.a braced wall panels). While some of these factors are quality control issues with the OSB sheathing, wood framing, and nails specifications, more variability than is shown in Table 1 will exist in real world applications. This variability is related to the construction of the shear wall, such as the size and quality of nail used (e.g., what can be readily purchased and its steel yield strength and hardness), nailing edge distance, and nails that miss framing members (shiners) to name just a few of a long list of variables. Each will have an effect on the actual WSP unit shear capacity.

SBCRI testing minimized the variation in the results by carefully controlling the tested materials, construction, and boundary conditions for each shear wall test set-up. The edges of the panels are always chalk-lined to ensure the sheathing fasteners are placed a minimum of 3/8" from the edges of the panel. All shiners (if present) were removed and a new fastener was installed. The studs were always straight. And so forth. In other words, these were pretty ideal walls from a construction practice perspective.

The design values for OSB sheathing in SDPWS allow the use of either common or galvanized box nails as defined in [Table 2.] Please also see [Appendix C]. Suffice it to say that reading through the IRC and IBC to get nailing done correctly is confusing. No reduction is given for the smaller nail diameter of box nails, a position that APA appears to support. In the publication entitled "Shear Wall Test Results Comparing 8d Common and 8d Box Nails" (TT-087B), APA says the following regarding the performance of 8d box and 8d common nails:

> "An 8d common nail has a shank diameter of 0.131 inch but an 8d box (or cooler, or sinker) nail has a diameter of 0.113 inch, which is approximately a 15 percent reduction in shank diameter."

Furthermore, the APA document says:

> "Using the NDS equations, a 15 percent reduction in shank diameter leads to approximately a 25 percent reduction in the lateral load resistance (assuming other variables remain equal) for typical wood-structural-panel-to-framing connections."

This document finally states:

> "Since the mid-1990's, several studies have been completed comparing the cyclic performance of wood structural panels built with 8d box nails to those built with 8d common nails (Ficcadenti et. al, 1995; 1997, Pardeon, et. al. 2003; APA 2004). Published results from 32 full-scale cycle tests show that the racking resistance of shear walls built with 8d box nails is comparable to those built with 8d common nails. A decrease in initial shear wall stiffness was noted to be in the range of 0-10 percent, which is on the order of the typical differences between "like" shear wall assemblies. Other response characteristics based on these full scale cyclic tests, such as peak displacement, energy dissipation, and ductility, are also similar for shear walls constructed with 8d box and 8d common nails. The differences between the full-scale shear wall test results and the NDS analytical calculations may be attributable to less wood splitting due to smaller-diameter nail shank and/or to an assembly/group effect that overshadows the small difference in nail shank diameter (neither the splitting not the system/group effect is accounted for in the NDS single-fastener yield equations)."
>
> …… "Available test results suggest similar shear wall performance between walls constructed with 8d common and 8d box nails."

SBCRI testing shows that, on average, there is about a 20% decrease in the lateral load resistance when 8d box nails (0.113) are used instead of 8d common nails (0.131). This difference is very similar to the 23% decrease (560/730 = 77% using Douglas Fir studs for wind conditions) between the design values for 6d common nails (0.113) vs. 8d common nails (0.131) in $^3/_8$" OSB given in SDPWS. As currently written Table 4.3A in SDPWS (Table 2) suggests that a $^3/_8$" WSP shear wall with 6d common nails (0.113" diameter) has a nominal unit shear capacity of 515 plf (560 plf times 0.92 for DF to SPF reduction), while a $^3/_8$" WSP shear wall with 8d box nails (also having a 0.113" diameter) has a nominal unit shear capacity of 672 plf (730 plf for times 0.92 for DF to SPF reduction for wind conditions).

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

Wood-based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC – Panel Edge Fastener Spacing (in.) 6: v_s (plf)	6: G_a (kips/in.) OSB	6: G_a (kips/in.) PLY	4: v_s (plf)	4: G_a OSB	4: G_a PLY	3: v_s (plf)	3: G_a OSB	3: G_a PLY	2: v_s (plf)	2: G_a OSB	2: G_a PLY	B WIND – Panel Edge Fastener Spacing (in.) 6: v_w (plf)	4: v_w (plf)	3: v_w (plf)	2: v_w (plf)
Wood Structural Panels - Structural I[4,5]			Nail (common or galvanized box)																
	5/16	1-1/4	6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16[2]			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435
Wood Structural Panels – Sheathing[4,5]	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8[2]	1-3/8	8d	440	17	12	640	25	15	820	31	17	1060	45	20	615	895	1150	1485
	7/16[2]			480	15	11	700	22	14	900	28	17	1170	42	21	670	980	1260	1640
	15/32			520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
	15/32	1-1/2	10d	620	22	14	920	30	17	1200	37	19	1540	52	23	870	1290	1680	2155
	19/32			680	19	13	1020	26	16	1330	33	18	1740	48	22	950	1430	1860	2435
Plywood Siding			Nail (galvanized casing)																
	5/16	1-1/4	6d	280	13		420	16		550	17		720	21		390	590	770	1010
	3/8	1-3/8	8d	320	16		480	18		620	20		820	22		450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")			Nail (common or galvanized box)																
	3/8		6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8		8d	260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	1/2		10d	370	21		550	23		720	24		920	25		520	770	1010	1290
	5/8			400	21		610	23		790	24		1040	26		560	855	1105	1455
Fiberboard Sheathing - Structural	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5			475	645	730
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5			475	645	730

1. Nominal unit shear values shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard shear walls, and 4.3.7.3 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
2. Shears are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
3. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 11.3.2A). The Specific Gravity Adjustment Factor shall not be greater than 1.
4. Apparent shear stiffness values G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be increased by 1.2.
5. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
7. Galvanized nails shall be hot-dipped or tumbled.

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Table 2: Table 4.3 of SDPWS defining the fact that nails can be common or galvanized box and associated wind and seismic nominal unit shear capacities or design properties for the lateral resistance (Shear Wall) performance of Wood Structural Panels

The only difference being the 1/8" greater fastener penetration per SDPWS or the ½" greater fastener penetration per the IRC/IBC codes (see Appendix C for code fastener schedules) into the framing members. If box nails per the codes or galvanized box nails per SDPWS are allowed to replace common nails in WSP shear walls, the current design values are not conservative and should be adjusted to correspond with the worst case nailing scenario. Of the 29 SBCRI shear wall tests, only one of the tests with the 8d box nails (0.113) met the published SDPWS design value (see Figures 1 and 2).

This is clearly confusing to anyone at its very best characterization.

Currently, the primary nail being used in the field is the 8d box (2 $^3/_8$" x 0.113") nail. In our experience, gun-driven 8d common (2 ½" x 0.131") nails can only be obtained by special order. Also, we are unaware of any installation instructions being printed on the WSP panels or otherwise provided in individual job site packages that alert the end user to the key factors that must be considered and the expected influence these factors will have on WSP shear wall performance. Clearly in the normal field environment, there is little or no quality control on the required minimum edge distance and no guidance are available on the number of allowable shiners. This means that the majority of the WSP shear walls constructed have a shear capacity potentially significantly below the nominal unit shear capacities given in SDPWS and the IRC. These inaccuracies reduce the expected factor of safety for structures using WSPs. Without jobsite package installation information and labeling on each WSP, accurate load resistance performance has little chance of occurring in IRC or IBC compliant structures. Yet as BJ Yeh said in our January meeting which can be found in our January 24th letter that we sent as a meeting re-cap (Attachment F):

> BJ Yeh articulated, in a very elegant and forthright manner, our industry's primary concern using the following words to reflect the point of view he expressed, which is a concept we have all heard many times; the prescriptive code is based on historical performance and essentially fundamental engineering does not really "apply" or "work" because structures built using traditional and conventional methods have a good resistance track record.

Our approach to design value equivalency is to use realistic tested capacity values and to be conservative in defining WSP benchmark performance for "IBC/IRC Section 104.11 equivalent to code" purposes. The extensive ASTM E564 and ASTM E2126 testing and analysis we have undertaken shows that a reasonable (some would say generous) estimate of the lower bound for segmented shear wall unit capacity values for WSPs are as follows:

Wood Structural Panel Lateral Unit Shear Wall Resistance Capacity from 4 foot and 8 foot in Length Segmented Shear Wall Testing (i.e. with hold down connectors) -- ASTM E564 Boundary Condition Testing	**Fastener**	**Fastener Spacing**	**Shear Resistance Capacity**	**Shear Resistance Capacity with GWB**
			SPF Framing	**SPF Framing**
			Benchmark Unit Shear Capacity (PLF)	**Benchmark Unit Shear Capacity with GWB (PLF)**
3/8", 7/16", or 15/32" WSP (sheathing) with Studs Spaced 16" o.c.	8d box nails (2 3/8" x 0.113") or 8d common nails (2 1/2" x 0.131")	6:12	500	600

Notes:
1. ASTM E564 states, "Load distribution along the top edge of the wall shall simulate floor or roof members that will be used in the actual building construction. When required to minimize distortion, reinforcement, such as a strong-back attached along the length of the top plate or a steel bearing plate attached to the end of the top plate shall be installed. The wall test assembly shall be laterally supported along its top with rollers or equivalent means so as to restrict assembly displacement outside the plane of loading. Lateral support rigidity shall not exceed that provided in the actual building construction." The same would be true for ASTM E2126 for cyclic testing when test data is desired to ascertain actual real-world performance characteristics of shear walls.
2. Unit shear resistance capacity values are based on SBCRI testing (proprietary data of Qualtim, Inc.) The shear resistance capacity represents the typical low end of the distribution of the test data. The variability in the test data is high.
3. SBCRI testing is generally confirmed by the testing undertaken by others, such as Dolan, Sedars, Toothman, Gruber, etc.
4. SBCRI has performed 49 E564/E2126 tests using the boundary conditions defined by ASTM E564 to arrive at the above tabulations.
5. Values with GWB have 1/2" gypsum wallboard applied horizontally with screws spaced 16" o.c. along the edges and 16" o.c. along intermediate framing members. This installation is typical of field construction.

Table 3: Wood Structural Panel Unit Shear Capacity from Segmented Shear Wall Testing (i.e. with hold down connectors) -- ASTM E564 Boundary Condition Testing as Our Shear Capacity Benchmarks

We are providing APA/AWC this information because it is our desire to be transparent regarding our testing, analysis and findings. The values in each of the cells in Table 3 are the WSP unit shear capacities generated from our segmented shear wall testing and related research. These are the values that the SBC industry will be using when it undertakes comparative equivalency testing and analysis as we develop engineering mechanics based resistance models. We will also be using these for all code compliance and professional engineering related shear wall capacity value equivalency work that our industry undertakes. Obviously, should APA/AWC provide us with test data and modeling information that shows different results for any of the cells in Table 1 or Table 3 or for the IRC as described in Figure 5 below; where the test data follows requirements of ASTM E564 purely (or the techniques of E564 using E2126 cyclic testing methods) and is performed in a similar manner to the SBCRI testing, we will assess this testing and analysis to determine if our Table 3 or Figure 5 benchmarks need to change.

Additionally we will need to know the measures that will be taken to assure that these shear resistance values are maintained in field applications through a reasonable "duty to warn and inform" product labeling or job site package process to get key information into the field. This information should include any needed reductions in unit shear capacities due to the quality of field construction, nail specification(s) to be used, etc. Should APA/AWC provide shear wall unit capacity reductions, which we believe should be considered, these unit shear capacities will then become the key equivalency benchmarks replacing Table 3 and Figure 5 along with any installed assembly factors. Obviously then the testing specifications to achieve these benchmark capacity values will need to be defined (e.g., the size and quality of nail used (i.e., what can be readily purchased and its steel yield strength and hardness), nailing edge distance, and nails that miss framing members (shiners) to name a few of a long list of variables for which we believe APA/AWC are surely the experts at defining.) Finally a quantification and justification for the assembly factors and what they specifically are (e.g. windows, perpendicular walls, boundary conditions, etc.) should be defined so that everyone using these shear wall capacity values fully understands all the elements that go into WSP lateral load resistance. This would help with all downstream engineering judgments.

As you are also aware, and as discussed at our January 3 meeting, our isolated 4x8 panel; our 6:1, 4:1, and 2:1 aspect ratio, our perforated, and our fully sheathed testing, based on the minimum IRC installation methods, provides the justification behind our IRC code change proposal that follows. As also discussed in January, we believe that transparent IRC factors should be provided to the engineering and building community so that everyone knows how the braced wall lengths provided in IRC tables are derived. Much better engineering decisions will be made with complete transparency.

RB308-13

R602.10.4.4 (New), Table R602.10.4.4 (New)

Individual Consideration Agenda

This item is on the agenda for individual consideration because a public comment was submitted.

Public Comment:

Name: Larry Wainright, representing the Structural Building Components Association

Modify the proposal as follows:

R602.10.4.4 Design Values. For the purpose of braced wall design, the capacity of wood structural panels to resist lateral loads, as found in Table R 602.10.3(1) are found in Table R602.10.4.4.

TABLE R602.10.4.4
SIMPLIFIED SHEAR VALUES FOR WIND LOADING OF BRACED WALL LINES

Sheathing Material	Bottom plate connection to foundation	Fastener	Fastener Spacing	Any Species Stud Framing		
				Tested capacity	System Effects Factor	IRC Lateral Design Capacity
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing ~~-- Wind.~~	Anchor bolts per code requirements	6d (2" x 0.113" nails) or 8d (2 1/2 x 0.131"	6:12	~~335~~350	1.8~~0~~	600
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing (with 1/2" gypsum on interior face of wall~~. -- Wind~~	Anchor bolts per code requirements	6d (2" x 0.113") or 8d (2 1/2 x 0.131"nails and Types S or W drywall screws.	6:12 WSP & 16:16 for GWB	~~465~~450	1.8~~0~~	840

1. The lateral design capacity of braced wall panels is based on full scale wall assembly tests using the minimum restraint provisions of the IRC, further adjusted by the partial restraint/systems effect factor.

Commenter's Reason:

In addition to the original reason statements provided in RB308 and RB309 the following should be considered:

SBCRI has completed additional testing and as a result, proposes the modifications shown above. The proposed 350 plf for wood structural panels (WSP's) installed without gypsum is the tested capacity of WSP's in full scale tests as well as in 23' wall assemblies when built to the minimum requirements of the IRC. The stated System Effects factor is simply a factor used to convert the tested capacities to the capacity currently in use in the IRC. It is recognized that the systems effect factor does not exactly result in the stated IRC capacity. The calculated value is rounded to the capacity currently in use. This proposal does not seek to modify what is currently in use. (I.e., the tested capacity, 350 plf times the systems effect factor of 1.8 equals 630 plf. This was rounded down to the 600 plf currently in use.)

When the Ad-Hoc Wall Bracing Committee (AHWBC) first developed these provisions, they did the best that they could, given the testing that was available at the time. Most of the testing that was available came from testing of fully restrained walls. This testing formed the basis of the committees work and judgments were made with regard to the partial restraint of buildings constructed to the IRC as well as the systems effects of completed construction. The table able does not change any of that work, but simply restates the basis of the design capacities using the capacities from tests of buildings constructed in accordance with the minimum IRC and then applying the factor necessary to get back to the current IRC design values.

With regard to the addition of gypsum to braced wall panels: The Ad-Hoc Wall bracing committee used 200 plf as the capacity of the gypsum added to the back side of the braced wall panel. The 200 plf capacity is predicated on the use of nailing at 7" o.c. at the edges of the panel and in the field. **Additionally, the gypsum must be installed vertically (See Table R602.3 (1), Line 37 and footnote "d").** This orientation and fastening pattern is rarely accomplished in the field. The more common fastening is in accordance with the interior coverings section (R702.3.5) which allows both horizontal and vertical applications and screw spacing at 16" o.c. SBCRI tested both of these conditions. The 200 plf capacity of the gypsum is confirmed when installed per the AHWBC assumptions, but only achieves 100 plf when installed with 16:16 screws.

The IRC-Building Committee's stated two reasons for disapproving RB309 follow. First, the proposal was not limited to wind as stated in testimony. While the limitation was stated in the table, the revision above moves the wind limitation to the title of the table to be clearer as to the application. Second, they stated that design values do not belong in a prescriptive code. However, there are often parts of a building that do not comply with the IRC and that must be designed. Currently, the only direction a building designer has to obtain design values to use engineering based reference documents such as SDPWS which provide design capacities based on fully restrained conditions. This proposal simply gives the building designer an accurate assessment of the design capacities currently provided for in the IRC using the minimum IRC construction as the basis of the capacity.

Figure 5: SBCA IRC Code Change Proposal for Transparent Shear Capacity Values for Wind Loading of Braced Wall Lines

Unless APA/AWC provide test data and corresponding analysis that justifies a different testing and engineering assessment than is provided in the foregoing (i.e. Table 3 and Figure 5), the SBC industry will use our ASTM E564/ASTM E2126 real truss boundary condition test results as SBC industry lateral shear wall resistance capacity benchmarks (SDPWS segmented and IRC conventional light frame installed) for equivalency to the law purposes. We will undertake all our equivalency calibrations and IBC/IRC Section 104.11 engineering assessments based on the above provided benchmarks in the context of the SDPWS/IBC code adopted and legally approved lateral segmented shear wall capacity values found in Table 4 below and the IRC code adopted and approved braced wall panel design values as defined in our code change proposal in Figure 5 above.

IBC Nominal Unit Shear Capacity Values for the Primary WSP Products Used in the Market – Taken from SDPWS.	Fastener	Fastener Spacing	Wind	Seismic	Wind	Seismic
			SPF Framing	SPF Framing	SPF Framing	SPF Framing
			IBC Nominal Unit Shear Capacity (PLF)	*IBC* Nominal Unit Shear Capacity (PLF)	*IBC* Nominal Unit Shear Capacity w/GWB (PLF)	*IBC* Nominal Unit Shear Capacity w/GWB (PLF)
3/8" WSP (sheathing) with Studs Spaced 16" o.c.	6d (2" x 0.113" nails)	6:12	515	368	715	368
3/8, 7/16", 15/32 " WSP (sheathing) with Studs Spaced 16" o.c.	8d common (2 1/2" x 0.131") or 8d box (2 3/8" x 0.113") nails	6:12	672	478	872	478
Gypsum wall board is fastened with 5d cooler nails spaced 7" o.c. – 200 plf for studs 16" o.c. in accordance with SDPWS Section 4.3.3.3.2 and Table 4.3C.						

Table 4: *SDPWS* Nominal Unit Shear Capacity Values for the Primary WSP Products Used in the Market
SDPWS Serves as Our Shear Capacity Benchmark

In other words 500 plf equals 672 plf per SPDWS/IBC for WSPs without gypsum wallboard (GWB) applied and 350 plf equals 600 plf per the IRC without GWB applied. As defined in Figure 5, we also have found the application and performance of GWB to be defined as follows:

> With regard to the addition of gypsum to braced wall panels: The Ad-Hoc Wall bracing committee used 200 plf as the capacity of the gypsum added to the back side of the braced wall panel. The 200 plf capacity is predicated on the use of nailing at 7" o.c. at the edges of the panel and in the field. Additionally, the gypsum must be installed vertically (See Table R602.3 (1), Line 37 and footnote "d"). This orientation and fastening pattern is rarely accomplished in the field. The more common fastening is in accordance with the interior coverings section (R702.3.5) which allows both horizontal and vertical applications and screw spacing at 16" o.c. SBCRI tested both of these conditions. The 200 plf capacity of the gypsum is confirmed when installed per the AHWBC assumptions, but only achieves 100 plf when installed with 16:16 screws.

As stated above, if APA/AWC thinks that our approach is incorrect, we would encourage providing us with test data and corresponding analysis that justifies a different testing and engineering assessment than is provided in the foregoing that can be relied upon for benchmark and equivalency purposes. This should include details regarding each of the following items that affect shear resistance capacities and therefore law adopted WSP unit shear capacities and thus allowable lateral resistance design values:

1. Test method and test boundary conditions and how they affect real building lateral resistance and what calibration factors to use to match test data to actual in building performance.
2. Effect of variability in WSP sheathing and framing material properties (specific gravity, etc.).
3. Effect of the variation from the SDPWS required 3/8" edge distance.
4. Number of shiners allowed.
5. Transparent nail specifications. Nail properties such as size (common, box, sinker, etc.), finish (smooth, ring shank, twisted, etc.), coatings (glue, vinyl, etc.), head shape (offset, clipped, etc.), fastener bending yield strength, etc. should be specified along with how the installers and building officials can determine that the proper nail has been used.
6. Any other factors that may only be known to APA/AWC that should be used for IRC applications and justification of those factors.

7. Justification of the use of ASTM E72 for the purposes of generating nominal unit shear capacities and related design values.
    a. As we are seeing in the ASTM E72 process there is a strong desire by the WSP industry to use ASTM E72 for design values when the standard, through its history, has specifically said that this should not be done.
    b. Please see Attachment G for SBCA comments on changes to E72.
8. Justification of the use of ASTM E2126 and AC322 appendix A for establishing seismic design coefficients when roughly 30% of the AC130 WSP database provides an R factor of 2.0.
    a. SBCA and SBCRI have a good deal of in-depth testing and analysis here as well.
    b. This is another area where testing and conservative analysis of the data would show that the use of an R factor of 2 for WSPs is certainly reasonable and conservative. Yet this R factor of 2 has been granted an R factor of 6.5 in a manner where no closed form engineering analytical techniques can be used to justify this result. Even when analytical techniques exist to calculate an accurate R factor, for components like WSP shear walls, they were not used.
    c. Again APA/AWC are encouraged to provide data and engineering justification to show where our assessment of this area of testing and equal energy/AC 322 appendix A analysis is incorrect.

Until we obtain from APA/AWC a detailed, transparent, and in-depth technical foundation for a given set of "benchmarkable" shear resistance capacity values and related allowable design values that we can verify through SBCRI testing and this verification shows consistency of performance per our testing, the SBC industry will be following this code equivalency and engineering roadmap:
1. IRC nominal unit shear capacity equivalency benchmarks will be used as defined in the IRC code change proposal found in Figure 5 above.
2. SDPWS nominal unit shear capacity equivalency benchmarks will be used as defined in Table 3 above.
3. Seismic design coefficients will be generated using equivalent energy engineering concepts and be compared to the AC130 WSP performance database, which was used to establish ASCE 7-10 Table 12.2-1 seismic design coefficients (SDC). This database establishes a set of SDC equivalency benchmarks when one uses equal energy concepts to analytically assess the data. The one area of contention that exists with the AC130 database is that it uses E2126 testing with steel fixture boundary conditions that will result in lateral shear resistance capacity err. We know this through the ASTM E72 work that is taking place, and this is also clear given what we know to be true about E564 boundary condition testing as shown again in Appendix B.

There is a good deal of public domain literature that supports our equivalency benchmarking approach. In addition to the listed appendices, we have provided a minor set of references (attached or provided links where they are readily available) to the following documents which are supportive:
1. Appendix A -- Structural Building Components Industry Truss and Component Raw Material and Construction Products Design Properties Policy
2. Appendix D -- ASTM E72/E564 Testing and Boundary Condition Effects on BWP Capacity
3. Appendix E -- Ed Keith created a really nice graphical depiction of the issue at hand for our January 3rd meeting.
4. Appendix F – Ed Keith article and email exchange on Thursday, November 15, 2012 10:17 AM
5. "Reliability and Effect of Partially Restrained Wood Shear Walls." Gruber, John Joseph, (2012).Wayne State University Dissertations. Paper 442.

6. "Light-Frame Shear Wall Length and Opening Effects." Patton-Mallory, Marcia, etc. al. ASCE Journal of Structural Engineering, Vol. 111, No. 10, October, 1985.
7. Attachment H attached -- "SBCRI Background and Testing of Braced Wall Performance." Presentation by Kirk Grundahl dated May, 2013.
8. A REVIEW OF LARGE SCALE WOOD STRUCTURAL PANEL BRACING TESTS by Zeno Martin, P.E., Tom Skaggs, Ph.D.., P.E., Ed Keith, P.E., Borjen Yeh, Ph.D., P.E.
9. Wood Design Focus, Spring 2009, "The Story Behind the 2009 IRC Wall Bracing Provisions (Part 2: New Wind Bracing Requirements)", Jay H. Crandell, P.E. and Zeno Martin, P.E.
10. APA – The Engineered Wood Association (Report to BSSC Bracing Committee May 2007).
11. *Establishing seismic equivalency to code-listed light-frame wood wall systems*; Ned Waltz, Tom Skaggs, Philip Line, and David Gromala; Proceeding of World Conference on Timber Engineering (WCTE), WCTE, Miyazaki, Japan; 2008.
12. *Establishing seismic equivalency to code-listed light-frame wood wall systems*; Ned Waltz, Tom Skaggs, Philip Line, and David Gromala; Proceeding of World Conference on Timber Engineering (WCTE), WCTE, Miyazaki, Japan; 2008.
13. *Minimum Design Loads for Buildings and other Structures* (ASCE 7-10); American Society of Civil Engineers; 2010.
14. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, 2003 Edition, Part 2: Commentary (FEMA 450-2); Building Seismic Safety Council, Washington, D.C.
15. "Seismic Force-Resisting Systems Part 1: Seismic Design Factors"; SEAOC Seismology Committee; *Structure Magazine*, January 2009.
16. "Seismic Force-Resisting Systems Part 2: Codified Systems"; SEAOC Seismology Committee; *Structure Magazine*, February 2009.
17. *Reinforced Concrete Structures*; Robert Park and T. Paulay; John Wiley and Sons; 1975.
18. *Design of Multistory Reinforced Concrete Buildings for Earthquake Motions*; John Blume, Nathan Newmark, and Leo Corning, Portland Cement Association, Skokie, Illinois; 1961.
19. Finally we believe that relevant reading includes the recent "Public Letter to ALSC, AWC, NAHB, SFPA and ICC" by Kent Pagel as published in an SBCA Special Industry News item and associated footnotes.

We again apologize for all the information provided here, but we know of no other way to be precise and transparent in what will be our approach to code equivalency and related engineering analysis so that all our work ties directly to and is directly comparable to what APA/AWC have advocated inside the code and standards arena which then becomes law. This information will be shared publically and transparently on our SBC Industry News website, to those that we know have an interest in this subject area and anyone asking us why and how we are doing what we are doing from an engineering equivalency perspective. Until we have "like-kind" test data and a well-defined installation approach that we find is consistently repeatable in our test facility, we sincerely believe that the foregoing information is a very reasonable engineering assessment of shear wall lateral resistance performance. We also know that the IRC/IBC and ASCE 7 Table 12.2-1 will not change very soon given the bureaucratic nature of those processes, so we have to assume that our current knowledge and equivalency factors have to be used to provide reliable shear wall level playing field engineering, at least until APA/AWC provide transparent justification for more accurate and reliable shear wall capacity values that are repeatable and set up using the techniques as defined in ASTM E564 (again please see Appendix B for SBCRI approach to E564 testing) for all its shear wall capacity and seismic design coefficient assessments:

5.1 *General*—A wall assembly consists of frame elements including any diagonal bracing members or other reinforcements, sheathing elements, and connections. The wall assembly tested in accordance with this practice shall represent the minimum acceptable stiffness using the targeted frame and sheathing materials.
5.4 *Test Setup*—Provisions shall be made to resist rigid body rotation in the plane of the wall where this reflects the use of the assembly in actual building constructions. This shall be done by application of relevant gravity or other loadings simultaneously with the racking loads. The bottom of the assembly shall be attached to the test base with anchorage connections simulating those that will be used in service. Load distribution along the top edge of the wall shall simulate floor or roof members that will be used in the actual building construction. When required to minimize distortion, reinforcement, such as a strong-back attached along the length of the top plate or a steel bearing plate attached to the end of the top plate shall be installed. The wall test assembly shall be laterally supported along its top with rollers or equivalent means so as to restrict assembly displacement outside the plane of loading. Lateral support rigidity shall not exceed that provided in the actual building construction.

As we have said repeatedly over the last few years in a wide variety of communications, the SBC industry believes that non-transparent "prescriptive code or ICC-ES AC130/AC322 Appendix A like" approaches to engineering, where design values can easily be overstated or unknown, devalues the work of all professional engineers. We also sincerely believe that reliable and safe building performance is predicated upon having accurate and fully transparent raw material design properties including considerations that are need for successful application or installation. Suppliers of raw material to the SBC industry (and the engineering community overall) are responsible to ensure that there is easy access to this information along with any relevant factors that should be considered during the design process. These factors may include good engineering based reasons to apply them. However the transparent and easy to understand methodology behind why 500 plf should be allowed to equal 672 plf or 350 plf equal to 600 plf or an R factor of 2 be allowed to equal an R factor of 6.5, should be easy to understand. Any clear, concise and easy to follow justification backed up by test data that we can use to do the same testing and analysis ourselves and arrive at the same end result, is always greatly appreciated. This builds engineering confidence in raw material performance. This is certain to improve construction performance and foster future engineering innovation by the SBC industry.

Respectfully Yours,

Kirk Grundahl, P.E.
Executive Director

Cc: Hardy Wentzel
Kevin Blau
SBCA

Appendix A

Structural Building Components Industry Truss and Component Raw Material and Construction Products Design Properties Policy

Raw Material and Construction Product Purchasers, Resellers and Users Depend on Design Properties in the Raw Materials and Construction Products to be Accurate and Reliable.

Resistance of load by the structural framework of any building and its assumed factor of safety are predicated on accurate and reliable raw material and construction products fundamental design values, application factors and related engineering properties ("Design Properties"). These Design Properties are in turn used in span tables, lateral resistance tables, connection resistances, and engineering equations utilized through engineering software and otherwise.

Truss and component manufacturers currently purchase billions of board feet of lumber and wood construction products each year. When a manufacturer purchases lumber for use in the manufacture of trusses and components, it is effectively purchasing and relying upon the published lumber Design Properties. This means a purchase of Southern Pine No. 2 grade 2x4 is essentially a purchase of 1050 psi of fiber in bending in addition to other published lumber strength properties (see *Supplement No. 9 to the Southern Pine Inspection Bureau 2002 Grading Rules Effective June 1, 2012*). The same concept holds true for framers, carpenters, builders and other users of lumber who purchase and use lumber to resist loads through traditional building code adopted span tables, which span tables are based on published lumber Design Properties. Therefore, all lumber purchasers and users are purchasing and using lumber for its load resisting Design Properties and depend on the Design Properties to be accurate.

These concepts likewise apply to the purchase and use of other wood construction products that are regularly re-sold by truss and component manufacturers (such as OSB, plywood, LVL, PSL, glulam, and I-joists) as well as with the metal connector plates that are used in the manufacture of trusses and hardware and fasteners that are re-sold by truss and component manufacturers. The Design Properties for such wood and other construction products, through the utilization of engineering software or otherwise, must be accurate and the users of such products are relying on the published Design Properties.

Regular Testing and Analysis of Construction Raw Materials and Construction Products is a Necessity.

Truss and component designs are supported by historical testing and analysis. Likewise, testing of all types of lumber species and grades regularly occurs and ensures that the published Design Properties in the lumber being utilized in the manufacture of trusses and components (and upon which truss and component design software is based) and otherwise in all construction, continue to be accurate. Similarly, the design properties published for OSB, plywood, LVL, PSL, glulam, hardware and other wood and miscellaneous construction products must be accurate as they are input into engineering software programs where the output is expected to represent the safe resistance of all applied loads. Therefore, these types of construction products should be tested regularly as well.

Where any design is not supported through the use of accurate Design Properties or by engineering mechanics based Design Properties development testing, but rather is deemed to comply based on an industry or committee "judgment" or because the design is prescribed by the building code through tradition and the code consensus process, the load resistance analysis that is provided in the end-use application is neither accurate nor reliable. This view, that because the historical or code based performance has been acceptable and there is therefore no need to otherwise verify through testing and analysis, is simply flawed.

This in fact is the case with certain building code adopted wood product prescriptive applications. If prescriptive designs for these wood and other products are only supported by historical reference and cannot be supported by clear and understandable engineered design or testing, they must be replaced with designs that are in fact supported by transparent and recurring testing and analysis. This fact not only has life safety ramifications, but furthermore potentially places the trusses and components manufactured by SBCA member companies in a non-competitive technical and marketplace position, as their product designs are based on current Design Properties and are otherwise supported by testing and analysis. It is therefore in the best interests of both the construction industry at large, as well as the truss and component manufacturing industry in particular, that engineering and thus construction, be entirely based on tested and accurate raw material load resistance data. This will not only improve construction performance that is based on engineering and is therefore safe, but will further allow for future engineering innovation.

As the use of engineering software becomes more sophisticated and accounts for flow of loads from one structural element to the next and full structure systems effects, the engineering reliability demands on the raw materials and wood or other products that are utilized will certainly increase. By way of example the International Building Code ("IBC"), which becomes law when adopted by a jurisdiction, states the following:

> "**IBC Chapter 16, Section 1604.4** Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.
>
> Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.
>
> Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements."

Reliable and safe building performance is predicated upon accurate Design Properties, engineering precision and a complete understanding of raw material engineering considerations needed for successful application or installation. The suppliers of these products are responsible to ensure that there is easy access to this understanding along with any relevant factors that should be considered in that design process. It is furthermore the responsibility of building officials to review and ensure all designs comprehensively comply with the latest published Design Properties that are based on testing and generally accepted engineering practice.

Utilization of Published Design Properties.

When new Design Properties for lumber are published, they become the current standard or "state of the art" and must be adopted and utilized upon the published effective date by all manufacturers, sellers, specifiers, purchasers and users of such lumber. When Design Properties for wood and other construction products (such as OSB, plywood, LVL, PSL, glulam, metal connector plates, hardware, and fasteners) are likewise published, they also become the current standard or state of the art and must be adopted and utilized upon the published date by all manufacturers, sellers, specifiers, purchasers and users of such products. Building officials should furthermore monitor and require such utilization.

For example when SPIB issued its *Supplement No. 9* setting forth new design values effective June 1, 2012 for visually graded Southern Pine and Mixed Southern Pine (sized 2" to 4" wide and 2" to 4" thick in No.2 Dense and lower grades), all designs (truss or otherwise) that utilized such Southern Pine grades after June 1, 2012 must have used the new lower Design Properties to be compliant with current standard or the state of the art. Any truss design that utilized the previously published lumber Design Properties prior to June 1, 2012 was compliant and conforming to the then current standard or state of the art. The only exception to the use of the published new lumber Design Properties after June 1, 2012 would be with the consent of the building engineer of record and assurances of no responsibility on the part of the person or entity undertaking such design, as the engineer of record is otherwise intimately aware of the design of the structure of the building and the margins of safety that exist with respect to such building design.

Irrespective of whether a building official chooses to enforce the June 1, 2012 published lumber Design Properties in a particular jurisdiction, if a lumber purchaser or user relies on an outdated lumber span table that was based on lumber Design Properties that existed prior to the June 1, 2012 new published Design Properties, subjects that purchaser of the lumber, the Contractor and the Owner to potential legal responsibility as each are not utilizing the current standard or state of the art.

It would furthermore be an error for a lumber purchaser or user to rely on a specific Building Code reference that studs be of a "minimum No. 3, standard or stud grade lumber" irrespective of the change in lumber Design Properties for No. 3 Southern Pine that resulted in a decrease in compression and bending strength of ____% as of June 1, 2012 because of the SPIB published *Supplement No. 9*. It is difficult to understand how a prudent lumber purchaser or user could rely upon the Building Code reference to a grade mark and ignore the same lumber's new Design Properties without resulting legal responsibility.

For any person or entity to ignore the use of newly published lumber Design Properties or the Design Properties of any other construction product, wood or otherwise, subjects that person or entity and perhaps others in the chain of distribution, as well as building owners, to legal responsibility as the current standard and state of the art is not being followed.

SBCA Design Property Policy Summary:

1. Resistance of load by the structural framework of any building and its assumed factor of safety are predicated on accurate and reliable raw material and construction products fundamental design values, application factors and related engineering properties ("Design Properties").

2. All purchasers and users of products developed to resist applied loads are purchasing and using those products for their load resisting Design Properties and depend on the Design Properties to be accurate.

3. Design properties published for lumber, OSB, plywood, LVL, PSL, glulam, hardware and other wood and miscellaneous construction products must be accurate as they are input into engineering software programs where the output is expected to represent the safe resistance of all applied loads. Therefore, these types of construction products should be tested regularly to assure accurate Design Properties.

4. Accurate Design Properties should be assured by the manufacturer of the product or by Design Properties developed by testing and calibrated to an engineering mechanics based model. Design Values should not be deemed to comply based on an industry or committee "judgment" or because the design is prescribed by the building code through tradition and/or through the code consensus process. More often than not these activities are political in nature and the load resistance outcomes provided in the end-use application is neither accurate nor reliable. This view, that because the historical or code based performance has been acceptable and there is therefore no need to otherwise verify through testing and analysis, is simply flawed.

5. Irrespective of whether a building official chooses to enforce effective date published raw material Design Properties in a particular jurisdiction, if a purchaser or user relies on an outdated span table that was based on raw material Design Properties that existed prior to a new effective date published Design Properties, reliance upon the building official choice subjects that purchaser of raw material Design Properties, the Contractor and the Owner to potential legal responsibility as each are not utilizing the current standard or state of the art.

6. Furthermore, it is difficult to understand how a prudent raw material Design Properties purchaser or user could rely upon the Building Code reference to a prior raw material Design Property, such as a grade mark, and ignore the new Design Properties, such as the same grade mark would have given the new properties, without resulting legal responsibility.

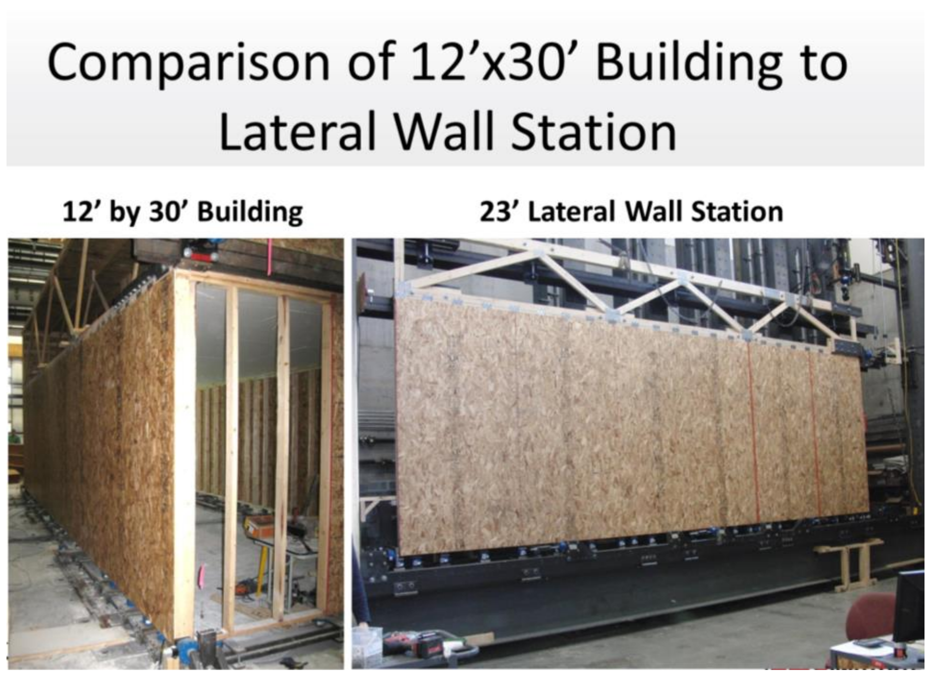
7. It is the responsibility of building officials to review and ensure all designs comprehensively comply with the latest published Design Properties that are based on testing and generally accepted engineering practice.

8. Reliable and safe building performance is predicated upon accurate Design Properties, engineering precision and a complete understanding of raw material engineering considerations needed for successful application or installation. The suppliers of these products are responsible to ensure that there is easy access to this understanding along with any relevant factors that should be considered in that design process.

9. It is in the best interests of the construction industry at large, as well as the truss and component manufacturing industry in particular, that engineering and thus construction, be entirely based on tested and accurate raw material load resistance data. This will not only improve construction performance that is based on engineering and is therefore safe, but will further allow for future engineering innovation.

SBCA Mission Statement:
SBCA supports research, development and testing of structural building components – trusses, wall panels, and related structural components – to root the industry in sound engineering and improve the quality, efficiency and cost-effectiveness of our products, for the purpose of achieving greater product acceptance. Therefore, SBCA promotes the consistent, safe, economic, and structurally sound design, construction and use of all structural building components, thereby increasing engineering innovation.

Appendix B



Appendix C

TABLE R602.3(1)—continued
FASTENER SCHEDULE FOR STRUCTURAL MEMBERS

ITEM	DESCRIPTION OF BUILDING MATERIALS	DESCRIPTION OF FASTENER[b, c, e]	SPACING OF FASTENERS	
			Edges (inches)[i]	Intermediate supports[c, e] (inches)
	Wood structural panels, subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing			
32	$^3/_8$" - $^1/_2$"	6d common (2" × 0.113") nail (subfloor wall)[j] 8d common ($2^1/_2$" × 0.131") nail (roof)[f]	6	12[g]
33	$^{19}/_{32}$" - 1"	8d common nail ($2^1/_2$" × 0.131")	6	12[g]
34	$1^1/_8$" - $1^1/_4$"	10d common (3" × 0.148") nail or 8d ($2^1/_2$" × 0.131") deformed nail	6	12
	Other wall sheathing[h]			
35	$^1/_2$" structural cellulosic fiberboard sheathing	$1^1/_2$" galvanized roofing nail, $^7/_{16}$" crown or 1" crown staple 16 ga., $1^1/_4$" long	3	6
36	$^{25}/_{32}$" structural cellulosic fiberboard sheathing	$1^3/_4$" galvanized roofing nail, $^7/_{16}$" crown or 1" crown staple 16 ga., $1^1/_2$" long	3	6
37	$^1/_2$" gypsum sheathing[d]	$1^1/_2$" galvanized roofing nail; staple galvanized, $1^1/_2$" long; $1^1/_4$ screws, Type W or S	7	7
38	$^5/_8$" gypsum sheathing[d]	$1^3/_4$" galvanized roofing nail; staple galvanized, $1^5/_8$" long; $1^5/_8$" screws, Type W or S	7	7
	Wood structural panels, combination subfloor underlayment to framing			
39	$^3/_4$" and less	6d deformed (2" × 0.120") nail or 8d common ($2^1/_2$" × 0.131") nail	6	12
40	$^7/_8$" - 1"	8d common ($2^1/_2$" × 0.131") nail or 8d deformed ($2^1/_2$" × 0.120") nail	6	12
41	$1^1/_8$" - $1^1/_4$"	10d common (3" × 0.148") nail or 8d deformed ($2^1/_2$" × 0.120") nail	6	12

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 mile per hour = 0.447 m/s; 1 Ksi = 6.895 MPa.

a. All nails are smooth-common, box or deformed shanks except where otherwise stated. Nails used for framing and sheathing connections shall have minimum average bending yield strengths as shown: 80 ksi for shank diameter of 0.192 inch (20d common nail), 90 ksi for shank diameters larger than 0.142 inch but not larger than 0.177 inch, and 100 ksi for shank diameters of 0.142 inch or less.

b. Staples are 16 gage wire and have a minimum $^7/_{16}$-inch on diameter crown width.

c. Nails shall be spaced at not more than 6 inches on center at all supports where spans are 48 inches or greater.

d. Four-foot by 8-foot or 4-foot by 9-foot panels shall be applied vertically.

e. Spacing of fasteners not included in this table shall be based on Table R602.3(2).

f. For regions having basic wind speed of 110 mph or greater, 8d deformed ($2^1/_2$" × 0.120) nails shall be used for attaching plywood and wood structural panel roof sheathing to framing within minimum 48-inch distance from gable end walls, if mean roof height is more than 25 feet, up to 35 feet maximum.

g. For regions having basic wind speed of 100 mph or less, nails for attaching wood structural panel roof sheathing to gable end wall framing shall be spaced 6 inches on center. When basic wind speed is greater than 100 mph, nails for attaching panel roof sheathing to intermediate supports shall be spaced 6 inches on center for minimum 48-inch distance from ridges, eaves and gable end walls; and 4 inches on center to gable end wall framing.

h. Gypsum sheathing shall conform to ASTM C 1396 and shall be installed in accordance with GA 253. Fiberboard sheathing shall conform to ASTM C 208.

i. Spacing of fasteners on floor sheathing panel edges applies to panel edges supported by framing members and required blocking and at all floor perimeters only. Spacing of fasteners on roof sheathing panel edges applies to panel edges supported by framing members and required blocking. Blocking of roof or floor sheathing panel edges perpendicular to the framing members need not be provided except as required by other provisions of this code. Floor perimeter shall be supported by framing members or solid blocking.

j. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule, provide two toe nails on one side of the rafter and toe nails from the ceiling joist to top plate in accordance with this schedule. The toe nail on the opposite side of the rafter shall not be required.

❖ The fastener schedule provides minimum nailing requirements (i.e., size, spacing) for connecting building elements used in wood framed construction. For wood structural panels, both edge nailing and intermediate (field) nailing are specified. In addition to the nailing for wood structural panels, fasteners are specified for gypsum wall sheathing, cellulosic fiberboard wall sheathing and combination subfloor underlayment.

TABLE 2304.9.1—continued
FASTENING SCHEDULE

CONNECTION	FASTENING[a, m]		LOCATION
30. Ledger strip	3 - 16d common ($3^1/_2$" × 0.162") 4 - 3" × 0.131" nails 4 - 3" 14 gage staples		face nail at each joist
31. Wood structural panels and particleboard[b] Subfloor, roof and wall sheathing (to framing)	$^1/_2$" and less	6d[c, l] $2^3/_8$" × 0.113" nail[n] $1^3/_4$" 16 gage[o]	
	$^{19}/_{32}$" to $^3/_4$"	8d[d] or 6d[e] $2^3/_8$" × 0.113" nail[p] 2" 16 gage[p] 8d[c]	
	$^7/_8$" to 1"	10d[d] or 8d[e]	
	$1^1/_8$" to $1^1/_4$"		
Single floor (combination subfloor-underlayment to framing)	$^3/_4$" and less	6d[e]	
	$^7/_8$" to 1"	8d[e]	
	$1^1/_8$" to $1^1/_4$"	10d[d] or 8d[e]	
32. Panel siding (to framing)	$^1/_2$" or less	6d[f]	
	$^5/_8$"	8d[f]	
33. Fiberboard sheathing[g]	$^1/_2$"	No. 11 gage roofing nail[h] 6d common nail (2" × 0.113") No. 16 gage staple[i]	
	$^{25}/_{32}$"	No. 11 gage roofing nail[h] 8d common nail ($2^1/_2$" × 0.131") No. 16 gage staple[i]	
34. Interior paneling	$^1/_4$"	4d[j]	
	$^3/_8$"	6d[k]	

For SI: 1 inch = 25.4 mm.

a. Common or box nails are permitted to be used except where otherwise stated.

b. Nails spaced at 6 inches on center at edges, 12 inches at intermediate supports except 6 inches at supports where spans are 48 inches or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Section 2305. Nails for wall sheathing are permitted to be common, box or casing.

c. Common or deformed shank (6d - 2" × 0.113"; 8d - $2^1/_2$" × 0.131"; 10d - 3" × 0.148").

d. Common (6d - 2" × 0.113"; 8d - $2^1/_2$" × 0.131"; 10d - 3" × 0.148").

e. Deformed shank (6d - 2" × 0.113"; 8d - $2^1/_2$" × 0.131"; 10d - 3" × 0.148").

f. Corrosion-resistant siding (6d - $1^7/_8$" × 0.106"; 8d - $2^3/_8$" × 0.128") or casing (6d - 2" × 0.099"; 8d - $2^1/_2$" × 0.113") nail.

g. Fasteners spaced 3 inches on center at exterior edges and 6 inches on center at intermediate supports, when used as structural sheathing. Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications.

h. Corrosion-resistant roofing nails with $^7/_{16}$-inch-diameter head and $1^1/_2$-inch length for $^1/_2$-inch sheathing and $1^3/_4$-inch length for $^{25}/_{32}$-inch sheathing.

i. Corrosion-resistant staples with nominal $^7/_{16}$-inch crown or 1-inch crown and $1^1/_4$-inch length for $^1/_2$-inch sheathing and $1^1/_2$-inch length for $^{25}/_{32}$-inch sheathing. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).

j. Casing ($1^1/_2$" × 0.080") or finish ($1^1/_2$" × 0.072") nails spaced 6 inches on panel edges, 12 inches at intermediate supports.

k. Panel supports at 24 inches. Casing or finish nails spaced 6 inches on panel edges, 12 inches at intermediate supports.

l. For roof sheathing applications, 8d nails ($2^1/_2$" × 0.113") are the minimum required for wood structural panels.

m. Staples shall have a minimum crown width of $^7/_{16}$ inch.

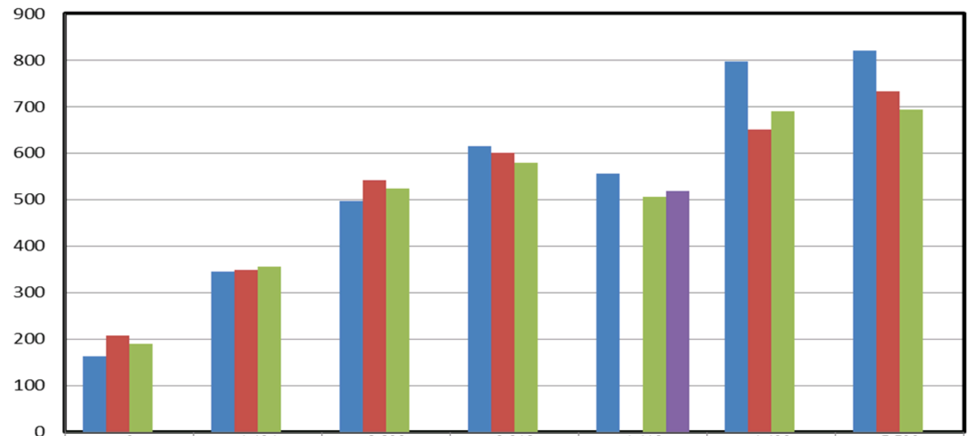
n. For roof sheathing applications, fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports.

o. Fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports for subfloor and wall sheathing and 3 inches on center at edges, 6 inches at intermediate supports for roof sheathing.

p. Fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports.

Appendix D

Effect of Applied Vertical Load and Hold Down Connector in an ASTM E72/E564 Single Element Test on Nominal Unit Shear Capacity (plf)



Nominal Unit Shear Capacity from Test in PLF

What the graph means:

Anchor bolts only provide shear wall capacities of 162, 207 and 190 plf.

A 3,300 lb. gravity load provides shear wall capacities of 615, 600 and 578 plf.

A HDU8 provides shear wall capacities of 555 and 506 plf.

A Quick Tie provides a shear wall capacity of 519 plf.

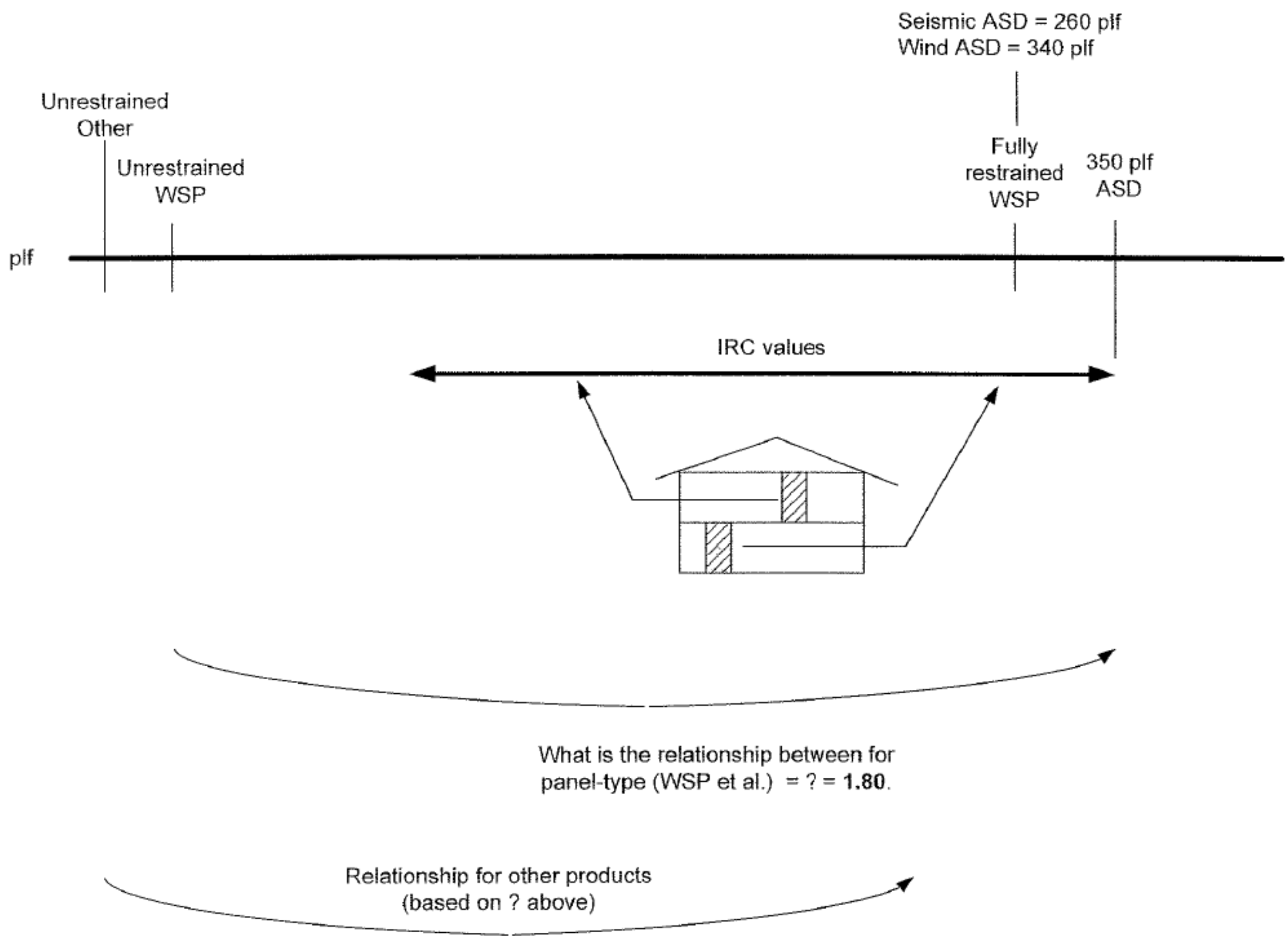
	0 Anchor Bolts (Abs) Only -- Test 2	1,104 ABs with 1,104 lbs. Vertical Load --Test 3.1	2,208 ABs with 2,208 lbs. Vertical Load --Test 3.2	3,312 ABs with 3,312 lbs. Vertical Load --Test 3.3	4,416 HDU8 ASD Design Value of 4,305 lbs Compare to Quick Tie	4,400 ABs with 4,400 lbs. Vertical Load --Test 3.4	5,500 ABs with 5,500 lbs. Vertical Load --Test 3.5
■ Actual Nominal Unit Shear Capacity in PLF (15/32" OSB) Based on Ph.D Single Element Testing with Applied Vertical Load in lbs.	162	345	496	615	555	798	821
■ Actual Nominal Unit Shear Capacity in PLF Based on Proprietary Product Testing with Apllied Vertical Load in lbs.	207	348	542	600		650	733
■ Actual Nominal Unit Shear Capacity in PLF (3/8" OSB) SBCRI Single Element Testing with Applied Vertical Load in lbs.	190	356.5	524	578.5	506	689.5	693.5
■ Actual Nominal Unit Shear Capacity in PLF (3/8" OSB) SBCRI Single Element Testing with 2378 lbs. Tensioned Quick Tie generating a 3935 lb. Vertical Load at Top of Wall					519		

Graph 1: ASTM E72/E564 Testing and Boundary Condition Effects on BWP Capacity in plf

Appendix E

From Kirk's February letter

Ed Keith created a really nice depiction of the issue at hand. This is found below in Appendix C. He also provided a great quote in an article that he and I exchanged emails with respect to that is provided in Appendix D.



Appendix F

From: Ed Keith [mailto:ed.keith@apawood.org]
Sent: Thursday, November 15, 2012 10:17 AM
To: Kirk Grundahl
Subject: RE: Ed Great Article *** See Highlighted Items ***

Kirk:

I received your e-mails and will respond soon. I have made a promise to get my IRC code change proposals to BJ by Thanksgiving so he can review over the holidays. That has me pretty much wrapped up as I have over 40 changes to complete. Be communicating soon.

Ed

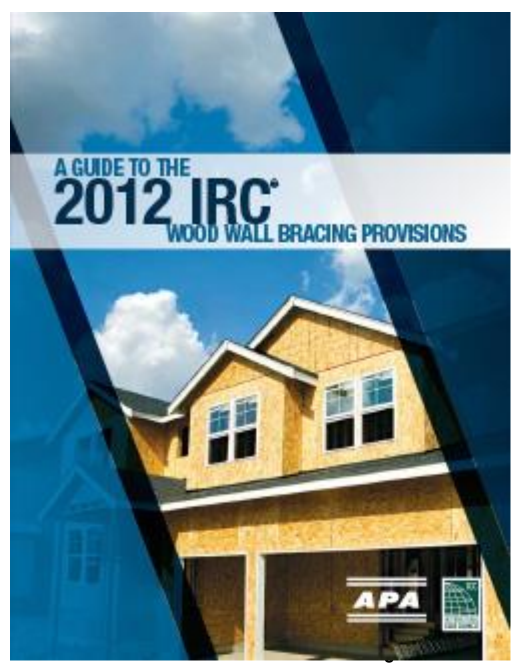
From: Kirk Grundahl [mailto:kgrundahl@qualtim.com]
Sent: Monday, November 12, 2012 3:52 AM
To: Ed Keith
Subject: Ed Great Article *** See Highlighted Items ***

Ed, assuming that you mean what you say, I am going to send you several emails as your expertise applied to the perspective that we have would be valuable. It is clear that I should have copied you on all of this before. Hope all is going well with you.
Kirk
608-217-3713

APA and ICC Team to Publish IRC Lateral Bracing Guide

Third edition of the book aims to improve understanding and application of the 2012 IRC lateral bracing requirements.

A new illustrated book, co-published by the International Code Council (ICC) and *APA—The Engineered Wood Association*, provides an explanation of the lateral bracing provisions of the *2012 International Residential Code* (IRC). The *Guide to the 2012 IRC Wood Wall Bracing Provisions*, the third edition in the series, details the correct application of the code-bracing requirements, explores the history and theory behind wall bracing, and provides real-world bracing examples. The book is now available in hardcopy and digital format.



"Bracing is one of the most critical, yet most misunderstood, safety elements in one- and two-family dwellings and townhouses constructed under the IRC," says Mark A. Johnson, ICC Executive Vice President and Director of Business Development. "The *Guide* is an important and helpful resource for inspectors, plan checkers, builders, designers and others involved in residential construction. The ongoing collaboration between APA and ICC benefits public safety and the industry. We are pleased to build on a long-standing relationship with APA."

The IRC contains numerous prescriptive lateral bracing provisions intended to help residential structures resist lateral loads that can result from wind and seismic events. The type and amount of bracing required for a given structure depends on many factors, including location and size of the structure, and the location of bracing segments within the structure. Bracing must be applied correctly and consistently to sufficiently protect the building from lateral loads, according to ICC and APA.
“Most of the buildings in the U.S. are residential, and most of them are built to the IRC. Wall bracing is what makes those buildings perform well against wind and seismic loads,” says co-author Ed Keith, Senior Engineer for the APA Technical Services Division. “So I would say that the bracing provisions are very important."

"These provisions are complex, given the great number of aesthetic, cultural, economic and energy-related variables that factor in,” says Keith. “This guide makes these provisions easy to understand.”

A Guide to the 2012 IRC Wood Wall Bracing Provisions addresses bracing options available to the builders and designers, the amount of bracing required with adjustments and variations, rules for the use of bracing, the new simplified wall bracing provisions, whole house bracing considerations and many other related topics. The full-color book features numerous specific examples and more than 200 figures, tables and photos.

While a portion of the book’s content was adopted from the previous edition, *A Guide to the 2009 IRC Wood Wall Bracing*, Keith says that the 2012 version reflects several refinements to the 2009 provisions. He also notes that the book was reformatted extensively to better accommodate the user in search of specific code references. “In the book, the bracing provisions are explained in the same order as they appear in the IRC, and the top of each page is annotated with the page content, so looking up a specific provision of the code is much simpler."

"The book is written to help the more causal user understand the bracing provisions,” Keith adds, “but we have also provided plenty of background information and theory to clarify the principles of bracing to engineers, architects and building officials."
The International Code Council is a member-focused association dedicated to helping the building safety community and construction industry provide safe and sustainable construction through the development of codes and standards used in the design, build and compliance process. Most U.S. communities and many global markets choose the International Codes.

Based in Tacoma, Washington, APA is a nonprofit trade association representing North American manufacturers of plywood, oriented strand board, glued laminated timber, wood I-joists, structural composite lumber, and other structural engineered wood products. Its primary functions are product certification and testing, applied research, and market support and development.
A Guide to the 2012 IRC Wood Wall Bracing Provisions is available for purchase in hardcopy for $42.00 ($33.50 for ICC Members, Product ID #7102S12) or digital PDF form for $39.95 (Product ID #8799P12) directly from the ICC. The 2009 edition of the guide is also available. Visitwww.iccsafe.org for more information.