

Cold-Formed Steel Framing Seismic Design Optimization Phase 1a: Seismic Equivalency Parameter Evaluation

RESEARCH REPORT RP10-4

2010



American Iron and Steel Institute



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PREFACE

Currently, both wood structural panel and steel sheet sheathed, cold-formed steel framed and wood structural panel sheathed wood framed shear walls qualify for an $R=6.5$ in bearing wall systems and an $R=7.0$ in building frame systems (ASCE 7-10). Recent work done in support of the development of ICC-ES AC322 and an update to AC154 has questioned whether both materials should qualify for the same seismic design parameters.

A multi-phase research project has been undertaken to address the seismic design of cold-formed steel light frame construction from a holistic standpoint by focusing on the optimization of the entire system. The subject of this report, designated Phase 1a, includes an initial evaluation of seismic equivalency parameters for cold-formed steel framed shear walls with wood structural panel and steel sheet sheathing – an effort that parallels the AC322 Advisory Group effort for wood-framed shear walls. Also included is guidance on the next steps of the multi-phase project, which will help the AISI Seismic Code Team as those additional phases are implemented.

The American Iron and Steel Institute and the Steel Framing Alliance acknowledge and are grateful to the following members of the Project Monitoring Task Group who helped guide and monitor this project.

Jeff Ellis, <i>Chair</i>	Simpson Strong-Tie
Don Allen	Steel Stud Manufacturers Association
Bonnie Manley	American Iron and Steel Institute
John Matsen	Matsen Ford Design Associates
Clifton Melcher	Dietrich Industries
Colin Rogers	McGill University
Ben Schafer	Johns Hopkins University
Reynaud Serrette	Santa Clara University
Allan Swartz	Swartz and Kulpa Engineers
Steve Tipping	Tipping Mar + associates

The American Iron and Steel Institute and the Steel Framing Alliance also wish to express their appreciation for the financial support of this project by the Steel Stud Manufacturers Association.



**AISI Cold-Formed Steel Framing Seismic
Design Optimization Project,**

**Phase 1a: Seismic Equivalency
Parameter Evaluation**

30 November 2010
WJE No. 2008.4126

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30 November 2010

AISI Cold-Formed Steel Framing Seismic Design Optimization Project

Phase 1a: Seismic Equivalency Parameter Evaluation

Introduction

AISI has undertaken the Cold-Formed Steel (CFS) Framing Seismic Design Optimization Project as an initiative to take a holistic view of seismic design of cold-formed steel light-frame construction, with the objective of optimizing CFS shear wall systems. This report relates to Phase 1a of AISI's optimization project. Phase 1a includes the evaluation of seismic equivalency parameters for cold-formed steel framed shear walls with wood structural panel and steel sheet sheathing, in an effort parallel to the AC322 Advisory Group effort for wood-framed shear walls. Also included is discussion of several related topics.

1. Seismic Equivalency Parameters

On 31 July 2007, the AC322 Seismic Equivalency Task Group presented to International Code Council Evaluation Services (ICC-ES) Acceptance Criteria Committee a methodology for seismic coefficient equivalency. This methodology was based on evaluation of test data for wood-framed wood structural panel (WSP) sheathed shear walls. The basis of establishing equivalency was the identification of a benchmark system to which critical performance parameters could be compared. Characteristics and parameters considered included indicators of component overstrength, ductility, energy dissipation, and deformation compatibility. The report "Methodology for Seismic Coefficient Equivalency," and the tabulated wood-framed shear wall test data used are included in Appendix A of this report. The parameters recommended by the task group are detailed in Appendix A of the task group report.

This report presents findings from evaluation of seismic equivalency parameters for cold-formed steel shear walls. Existing test data for cold-formed steel shear walls sheathed with either wood structural panel or steel sheet sheathing has been evaluated and parameters identified, consistent with the parameters adopted by the AC322 Advisory Group. Included in the test data are Type I and Type II walls, sheathed with plywood, OSB, or steel sheet. Available testing used either CUREE or SPD types of displacement histories (loading protocols). A total of 116 CSF framed shear walls are included in this data set; a listing of references for the test data is presented in Appendix B. Data has been obtained either directly from electronic testing data or from data plots. Parameter values from the positive and negative quadrants of the load-deflection plots have been averaged. ASD capacities have been determined in accordance with AISI S213-07, using the tabulated values for use in the *United States* and *Mexico*, where available; where not available, ASD capacities per the *Canada* tabulated values have been used.

Evaluation of AC322 Recommended Parameters

Appendix C provides seismic equivalency parameter values for the entire set of cold-formed steel shear walls listed in Appendix B. This Appendix C set of data will be referred to as the "all walls" data set, and includes data without any data conversion (i.e. for loading protocol). Each of the seismic equivalency parameters recommended for use by the AC322 task group is discussed below.

Ratio of Peak Load Capacity to ASD Load Capacity

The ratio of peak load capacity to ASD load capacity serves as an indicator of the component overstrength. The lower bound is relied on by the building codes to help achieve intended performance in design level and maximum considered earthquakes. The upper bound is used to cap demands made on boundary members and the balance of the seismic load path. Appendix A to the AC322 report requires that the peak load capacity, V_p , be not less than 2.5 times the ASD capacity, V_{ASD} , and not more than 5 times the ASD capacity. Based on the 116 test records in the CFS Appendix C “all walls” set and the 49 test record wood “All Data” set, these requirements are compared in Table 1.

Table 1. Ratio of peak capacity (V_p) to capacity at ASD (V_{ASD})

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Quantity Less Than 2.5x	Quantity Greater Than 5x
CFS all walls	7.06	1.70	3.47	33%	4.63	1.16	10 of 116	18 of 116
Wood	5.4	2.3	3.1	26%	3.9	2.3	3 of 49	3 of 49

The CFS shear walls have a minimum ratio that is smaller than the minimum for wood framed walls, and a greater number falling below the lower bound of 2.5. The primary contributors to ratios less than 2.5 are the steel sheet shear walls. Out of the 116 tests in the all walls data set, 23 are steel sheet sheathed. The overstrength implied by the steel shear walls is consistently less than the wood structural panel shear walls. It is recommended that a separate set of parameters be explored for the steel sheet shear walls. These walls are discussed further in Section 2 of this report.

The CFS shear walls have a maximum ratio that is larger than for wood framed walls, and a larger number of walls falling above the upper limit of 5. For the CFS shear walls, the primary contributors to ratios greater than 5 are the Type II (perforated) shear walls and the shear walls with 4:1 aspect ratios. In both of these wall types, there is overstrength in excess of what is recommended by the AC322 task group parameters. Out of the 116 tests in the all walls data set, 8 are Type II and 24 have 4:1 aspect ratios.

Ratio of Post Peak Displacement to ASD Displacement

The setting of a lower bound on the ratio of displacement at post peak capacity to displacement at ASD capacity serves to ensure that the shear wall is capable of maintaining load resistance when pushed past peak capacity and further serves as one measure of deformation capacity relative to other code-recognized shear wall systems. Ratios higher than the lower bound are viewed as better than required. Appendix A to the AC322 report requires that the ratio of displacement at 0.80 post peak load, $\Delta_{0.8VP}$, to ASD displacement be not less than 11, and the ratio of displacement at 0.65 post peak load, $\Delta_{0.65VP}$, to ASD displacement be not less than 14. The displacement at 0.80 post peak load is measured on the first cycle backbone curve where the load has peaked and then dropped to 80% of peak load. The displacement at 0.65 post peak load is measured similarly.

The 0.80 post peak ratio criterion is compared to the wood-framed shear wall data in Table 2. Here it can be seen that the choice of 11 as the criterion for wood shear walls corresponds approximately to the average minus one standard deviation for the wood-framed shear wall set. The variability of the CFS framed walls appears significantly higher, with the average minus one standard deviation coming out to 1.98. This value is so low as to be inappropriate to specify as an evaluation parameter. Further, if the

value of 11 were used, 31 percent of the shear walls would fail this criterion as opposed to 10 percent of the wood-framed walls. This criterion is explored further using sub sets of the CFS “all walls” data later in this report.

Table 2. Ratio of displacement at $0.80V_p$ ($\Delta_{0.8VP}$) to displacement at ASD (Δ_{ASD})

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 11
CFS all walls	85.86	3.50	17.52	89%	33.07	1.98	31 of 99
Wood	43.4	6.4	20.30	45%	29.4	11.2	5 of 49

The 0.65 post peak ratio criterion is shown in Table 3. Note that this data was not derived for the wood test data set, so only cold-formed steel data is presented. Again, the variability of the CFS framed walls appears high, with the average minus one standard deviation coming out to 3.50. This value is again so low as to be inappropriate to specify as an evaluation parameter, and if the value of 14 were used, more than half of the shear walls would fail this criterion as opposed to 10 percent of the wood-framed walls. Again, this criterion is explored further using sub sets of the “all walls” data later in this report.

Table 3. Ratio of displacement at $0.65V_p$ ($\Delta_{0.65VP}$) to displacement at ASD (Δ_{ASD})

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 14
CFS all walls	80.54	3.87	18.51	81%	33.52	3.50	49 of 94

Drift Ratio at Post Peak Load Capacity

The setting of a lower bound on the drift ratio at post peak load capacity serves as another check on the shear wall’s ability to accommodate post peak deformation and to maintain load resistance at post peak deformations. Ratios larger than the lower bound criteria are viewed as better than required. Appendix A to the AC322 report requires that the drift ratio at 0.80 post peak load capacity, $0.80V_p$, be not less than 2.8 percent of the wall height, and at 0.65 post peak load capacity, $0.65V_p$, be not less than 3.6 percent. Based on the 116 test records in the Appendix C “all walls” set, comparisons of these criteria between the cold-formed steel shear walls and the wood-framed shear walls are provided in Tables 4 and 5.

Table 4. Drift ratio at 0.80 post peak load capacity ($0.8V_p$)

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 2.8 percent
CFS all walls	4.38	1.55	2.53	23%	3.10	1.95	64 of 99
Wood	5.5	2.3	3.6	21.3%	4.3	2.8	6 of 49

Table 5. Drift ratio at 0.65 post peak load capacity ($0.65V_p$)

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 3.6 percent
CFS all walls	4.35	1.63	2.74	20%	3.30	2.18	86 of 94

The drift ratios adopted by the AC322 task group are clearly problematic for the CFS shear walls with the “all walls” data set, based on the number of walls not meeting the criteria. It is recommended that alternate ratios be explored for sub-sets of the data. This will be addressed in later sections of this report.

Parameter Conversion Based on Test Loading Protocol

During the CUREE-Caltech Woodframe Project, it was found that the loading protocol used for shear wall loading significantly affected the resulting load and displacement capacity for wood-framed shear walls (Gatto & Uang, 2002). Based on identical wood-framed shear wall assemblies tested with the CUREE and SPD protocols, recommendations were made by Gatto & Uang for conversion between results of CUREE and SPD protocols. Note that these conversions are based on a limited number of test specimens, with one wall configuration and two tests each for each protocol (four tests total). Appendix D provides data sets of the AC322 Task Group parameters for available testing comparing load protocol effects. The bottom portion of the table illustrates the conversion factors that were determined for wood-framed shear wall tests, based on Gatto & Uang Tests 2, 4, 6, and 8. These are summarized in the first row in Table 6. Using the Gatto & Uang tests, moderate differences in peak capacity and significant differences in peak and post peak drift ratios were identified.

For CFS shear walls, testing at McGill University allows comparison of similar parameters for conversion between SPD and CUREE protocol. Again, these ratios are based on a limited number of tests, including one wall configuration with plywood sheathing and three specimens tested with each protocol (six tests total). The derived ratios are given in the second row in Table 6.

Comparing the first and second rows of Table 6, it can be seen that there are significant differences between protocol conversion factors for wood-framed and CFS shear walls. For CFS walls, minor increases are suggested for Δ_{ASD} , V_p and Δ_p . Minor decreases are suggested for $\Delta_{0.80VP}$ and $\Delta_{0.65VP}$, and a significant increase is suggested for the equivalent viscous damping ratio. Conversion factors recommended by Gatto & Uang for evaluating loading protocol effects for CFS shear walls, based on the CUREE testing, are shown in the bottom row of Table 6.

Table 6. Factors to convert SPD parameters to CUREE parameters for CFS walls

	Δ_{ASD}	V_P	Δ_P	$\Delta_{0.80VP}$	$\Delta_{0.65VP}$	Equivalent Viscous Damping Ratio
Wood-framed Multiply SPD data by	0.92	1.25	1.75	1.63	1.65	1.15
CFS-framed Multiply SPD data by	1.07	1.17	1.25	0.89	0.86	2.07
Recommended CFS-framed Multiply SPD data by	1.07	1.17	1.25	1.00	1.00	2.0

A very significant implication of these conversion factors is that the deflections and drift ratios for 0.80 and 0.65 post peak will not be increased due to loading protocol conversions. Note that the data set used by the AC322 task group included only CUREE protocol tests, so conversion for protocol was not required for the wood-framed shear wall parameters.

Converted Parameter Comparison to the Wood Shear Wall Data Base

The following Tables 7 through 11 repeat Tables 1 through 5 using the recommended protocol conversions from the bottom row of Table 6 for all SPD protocol data. The converted CFS shear wall seismic equivalency parameters are taken from the “all walls” data set with conversion applied, included in Appendix E.

Table 7. Ratio of peak capacity (V_P) to capacity at ASD (V_{ASD}) - Converted

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Quantity Less Than 2.5x	Quantity Greater Than 5x
CFS converted	8.26	1.74	3.70	35%	4.98	1.28	8 of 116	22 of 116
Wood	5.4	2.3	3.1	26%	3.9	2.3	3 of 49	3 of 49

Table 7 values are changed from Table 1 based on the 1.17 protocol adjustment to V_P . The factor of 1.17, used to increase V_P for SPD tests, slightly increases the ratio of V_P to V_{ASD} , but the number of walls failing the criteria of 2.5 and 5 is still significantly higher for CFS walls than wood-framed. This will be explored further with sub-sets of the data.

Table 8. Ratio of displacement at $0.80V_P$ ($\Delta_{0.8VP}$) to displacement at ASD (Δ_{ASD}) - Converted

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 11
CFS converted	85.86	3.58	17.17	91%	32.77	1.57	36 of 99
Wood	43.4	6.4	20.30	45%	29.4	11.2	5 of 49

Table 8 values are changed from Table 2 based on the 1.07 protocol adjustment to Δ_{ASD} . This is a minor change, which results in a few additional cold-formed steel walls failing the criterion of 11.

Table 9. Ratio of displacement at $0.65V_P$ ($\Delta_{0.65VP}$) to displacement at ASD (Δ_{ASD}) - Converted

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 14
CFS converted	80.54	3.75	17.84	84%	32.85	2.90	50 of 94

Table 9 values are changed from Table 3 due to the 1.07 protocol adjustment to Δ_{ASD} . This is a minor change, which results in one additional cold-formed steel wall failing the criterion of 14.

Table 10. Drift ratio at 0.80 post peak load capacity ($0.8V_P$) - Converted

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 2.8 percent
CFS	4.38	1.55	2.53	23%	3.10	1.95	64 of 99
Wood	5.5	2.3	3.6	21%	4.3	2.8	6 of 49

Table 10 values are unchanged from Table 4 because values were not affected by conversions.

Table 11. Drift ratio at 0.65 post peak load capacity ($0.65V_P$) - Converted

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 3.6 percent
CFS	4.53	1.63	2.74	20%	3.30	2.18	86 of 94

Table 11 values are unchanged from Table 5 because values were not affected by conversions.

Based on the very minor influence of the protocol conversions and the limited conversion data available, the data included in the balance of this report will not include protocol conversions.

2. Other Parameter Observations

Because the “all walls” data set in Appendix C includes wide-ranging shear wall types, several groups within the overall data set have been identified for further evaluation of their effect on parameters.

The first group identified as having notably different behavior is the Type II (perforated shear wall methodology) walls. The testing used an overall 40 foot long framed wall, with full height sheathing ranging from 30 to 70 percent of the 40 feet. A data set for Type II walls is provided in Appendix F, and information on parameters is provided in Table 12.

Table 12. Type II Wall Parameters

	V_P / V_{ASD}		$\Delta_{0.80P} / \Delta_{ASD}$		$\Delta_{0.65P} / \Delta_{ASD}$		Drift Ratio 0.80P		Drift Ratio 0.65P	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV
All Walls	3.47	33%	17.52	89%	18.51	81%	2.53	23%	2.74	20%
Type II Only	4.64	13%	16.55	19%	21.01	25%	2.18	17%	2.63	22%

One noticeable difference in Table 12 values is the large ratio of peak capacity to ASD capacity, indicating a large overstrength for these walls. The other noticeable aspect is the much smaller coefficient of variation for the Type II walls. Because the Type II walls are an alternate configuration, and because of the very high overstrength, Type II walls are deleted from the data set for the balance of parameter evaluation in this section of the report. An “all walls” Type I data set is provided in Appendix G and used in the remainder of Section 2 of this report.

A data set for wood structural panel (WSP) sheathed shear walls only is provided in Appendix H, and a comparison of parameters to the “all walls” Type I parameters is provided in Table 13.

Table 13. Wood Structural Panel (WSP) Walls Only

	V_P / V_{ASD}		$\Delta_{0.80P} / \Delta_{ASD}$		$\Delta_{0.65P} / \Delta_{ASD}$		Drift Ratio 0.80P		Drift Ratio 0.65P	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV
All Walls (Type I)	3.41	34%	17.78	92%	18.48	85%	2.58	22%	2.77	20%
WSP Only	3.63	32%	20.17	84%	20.57	78%	2.61	23%	2.76	20.0%
Ratio All/WSP	0.94	1.06	0.78	1.10	0.77	0.90	0.99	0.96	1.00	1.00

Ratios are only moderately different for WSP only, except the ratios of $\Delta_{0.80P} / \Delta_{ASD}$ and $\Delta_{0.65P} / \Delta_{ASD}$, which have more notable changes.

Table 14. Wood Structural Panel (WSP) Walls Only Without Canadian Softwood Plywood (CSP)

	V_P / V_{ASD}		$\Delta_{0.80P} / \Delta_{ASD}$		$\Delta_{0.65P} / \Delta_{ASD}$		Drift Ratio 0.80P		Drift Ratio 0.65P	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV
All Walls (Type I)	3.41	34%	17.78	92%	18.48	85%	2.58	22%	2.77	20%
WSP Only	3.63	32%	20.17	84%	20.57	78%	2.61	23%	2.76	20.0%
WSP w/o CSP	3.56	33%	21.69	91%	21.39	88%	2.51	27%	2.62	23%

Table 14 evaluates the effect of softer, lower-capacity Canadian Softwood Plywood (CSP) on the overall data group, based on Appendix I data for WSP shear walls without CSP for Appendix J for CSP only. Including or excluding the CSP group has a moderate impact.

To further explore the effect of the lower-capacity CSP sheathing, Vieira and Shafer (2009) conducted small scale fastener tests comparing fastener peak capacity and deformation for Canadian CSP and Canadian Douglas Fir Plywood (DFP) and U.S. Structural 1 plywood. Resulting data representing the average of five tests for each sheathing type and given in Table 15.

Table 15. Wood Structural Panel (WSP) Walls Only Without Canadian Softwood Plywood (CSP)

Specimen	Peak Test Capacity (lb)	Tabulated Nominal Capacity for 6"oc (plf)	Ratio of Nominal to Test	Deflection at Peak Test Capacity (in)
CAN-2-DFP	660	795	1.205	0.61
CAN-1-CSP	500	651	1.302	0.66
USA- STR I	630	780	1.413	0.55

Interestingly, the tabulated ratios of nominal capacity to test capacity show CSP to be between DFP and US STR I. There does not seem to be any performance-related reason to exclude these tests from the data used to derive seismic parameters. On the other hand, the CSP plywood is a product with very limited use in the US, and Table 14 has illustrated that their inclusion somewhat impacts parameters; as a result it is recommended that they be excluded from data sets used to derive parameters for US construction.

Because differences in steel sheet sheathed shear wall performance have been identified (see discussion of Table 1), Table 16 considers seismic parameters for steel sheet shear walls, based on Appendix K & L.

Table 16. Steel Sheet Sheathing Only

	V_P / V_{ASD}		$\Delta_{0.80P} / \Delta_{ASD}$		$\Delta_{0.65P} / \Delta_{ASD}$		Drift Ratio 0.80P		Drift Ratio 0.65P	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV
All Walls (Type I)	3.41	34%	17.78	92%	18.48	85%	2.58	22%	2.77	20%
Steel Sheet Only	2.60	18%	6.00	27%	7.22	31%	2.39	17%	2.83	18%
Steel Sheet Five (27 mil) Representative Tests	1.89	12%	4.75	37%	4.76	16%	2.22	9%	2.43	10%

There are two lines of data for the steel sheet shear walls. This is because only five of the 23 test specimens closely reflect configurations included in the AISI S213-07 nominal shear strength table. The five specimens reflected have 27 mil steel sheet sheathing. The other eighteen specimens have steel sheet of 30 or 33 mils, which do not currently have tabulated values. As such, attention needs to be focused on the five representative tests reflecting AISI S213-07 intent. Ratios of V_P to V_{ASD} are significantly lower for the five representative tests, and are clearly lower than the 2.5 minimum criterion proposed by the AC322 task group. This suggests that the overstrength of steel sheet sheathed walls needs to be recognized as being lower than wood structural panel sheathed walls, given current detailing requirements and design values.

Ratios between post peak deflection and ASD deflection are significantly lower for steel sheet sheathing than for the “all walls” Type I data set. The variation between all steel sheet walls and the five representative tests, however, is not significant. The low ratios appear to be driven by higher ASD deflections. The higher ASD deflections are reflective of lower initial stiffness of the steel sheet shear walls. An alternate way to evaluate post-peak deflection capacity is the ratio of post-peak deflection to deflection at peak capacity. Table 17 illustrates use of this comparison for the wood-frame shear walls and five representative steel sheet shear walls.

Table 17. Steel Sheet Sheathing Only

	Δ_{ASD} (inches)		Δ_P (inches)		$\Delta_{0.80P} / \Delta_P$		$\Delta_{0.65P} / \Delta_P$	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV
Wood Framed	0.213	18%	2.69	16%	1.35	14%	--	--
Steel Sheet Five Representative Tests	0.504	28%	1.47	21%	1.54	17%	1.83	23%

Table 17 illustrates that the deflection at peak capacity appears to be more stable, making the ratios of post-peak deflections more stable. In addition it appears that the ratios of post-peak to peak deflections of steel sheet shear walls are reasonably in line with the wood-framed shear wall data set. It is recommended that ratios of post peak deflection to deflection at peak capacity be considered as an acceptable alternative to ratios of deflection at ASD capacity.

AISI Research Report RP07-3 (Yu, 2007) provides recommendations for nominal capacities for the 30 and 33 mil steel sheet sheathing. Table 18 provides AISI S213-07 values for 27 mil sheets, and RP07-3 values for 30 and 33 mil sheets together for comparison.

Table 18. Steel Sheet Sheathing Nominal Capacities (plf)

Source	Steel Sheet Framing Thickness (mils)	Max Aspect Ratio	Fastener Edge Spacing (inches)			
			6	4	3	2
S213-07-S1-09	27 / 43	4	-	1000	1085	1170
RP07-3	30 / 43	2	911	1014	1042	1070
RP07-3	33 / 43	2	1056	1169	1236	1304

Important to note is that the recommended nominal values for 30 mil steel sheet are very close to the values in AISI S213-07 for 27 mil. The values for a 33 mil thickness range from 11 to 17 percent more than the 27 mil, overall a minimal increase for the thicker sheet. Note that following this testing, further testing by University of North Texas has revisited and recommended revised nominal capacities for the 27 mil sheet sheathing (RP09-2, Yu, 2009), in addition to exploring variations in wall layout and detailing. Based on this additional information, Table 17, with the five 27 mil tests, best represents what is in AISI S213-07. Table 16, with all test values, better represents anticipated behavior if the University of North Texas recommendations for nominal capacities are followed.

It has been clearly observed in many shear wall tests that the aspect ratio (ratio of wall sheathing height to width) of the shear wall effects drift, and to a lesser extent peak capacity. Shear walls with high aspect ratios have lower peak capacities and higher drifts. Adjustments are made to allowable shears in high aspect ratio shear walls that compensate for the capacity and drift effects. Table 19 illustrates the effect of excluding higher aspect ratio shear walls from both the wood and CFS data sets. Interestingly, with CFS the only parameter that is significantly affected by eliminating high aspect ratio shear wall data is the coefficient of variation for the ratio of peak to ASD load capacity. All other parameters are moderately affected.

Table 19. Shear wall parameter variation with aspect ratios.

	V_p / V_{ASD}		$\Delta_{0.80P} / \Delta_{ASD}$		$\Delta_{0.65P} / \Delta_{ASD}$		Drift Ratio 0.80P		Drift Ratio 0.65P	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV
Wood All Data	3.1	26%	20.30	45%	-	-	3.6	21%	-	-
Wood 2:1 Max Aspect	2.9	22%	12.8	27%	-	-	-	-	-	-
CFS All Walls (Type I)	3.41	34%	17.78	92%	18.48	85%	2.58	22%	2.77	20%
CFS 2:1 max aspect	3.01	12%	15.35	85%	16.66	79%	2.47	17%	2.67	17%

Of most concern in the effort to assign parameters for the CFS shear walls is the great variability in the ratios of 0.80 and 0.65 post peak deflection to ASD deflection. Bar charts shown in Figures 1 and 2 graphically show the variability in the 0.80 post peak ratio. Figure 1 includes all CFS Type I wall data. Ten tests show considerably more variability than the balance of the data; seven of these ten tests are 4:1 aspect ratios. The other three have 2:1 aspect ratios. The high ratios correspond to test numbers 33, 39, 40, 41, 45, 46, 4, 51, 52 and 53 in Appendix C. Parameter data for this figure is found in the third “CFS all walls” Type I line of Table 19.

Figure 2 shows the effect of removing the 24 shear walls with 4:1 aspect ratio shear walls. The fourth line in Table 18 provides parameter data for this group of walls. Three walls can be seen to still have much greater ratios than the balance.

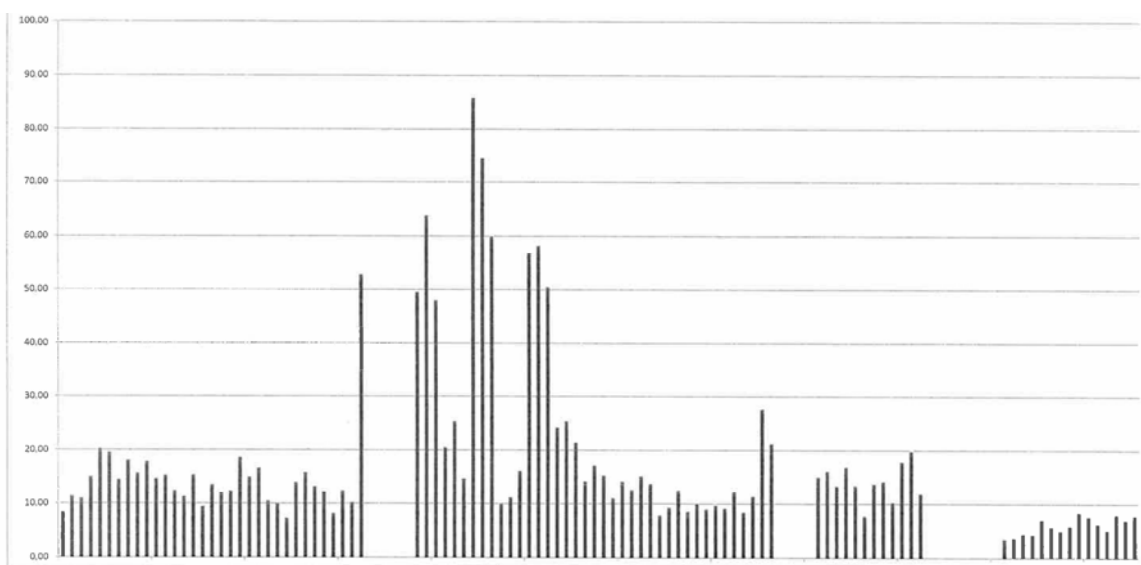


Figure 1. Bar chart of ratios of 0.80 post peak drift to ASD drift for all walls. Note that ratios for ten walls are much higher than the others (vertical axis plots ratio in percent, horizontal axis plots one bar for each data point).

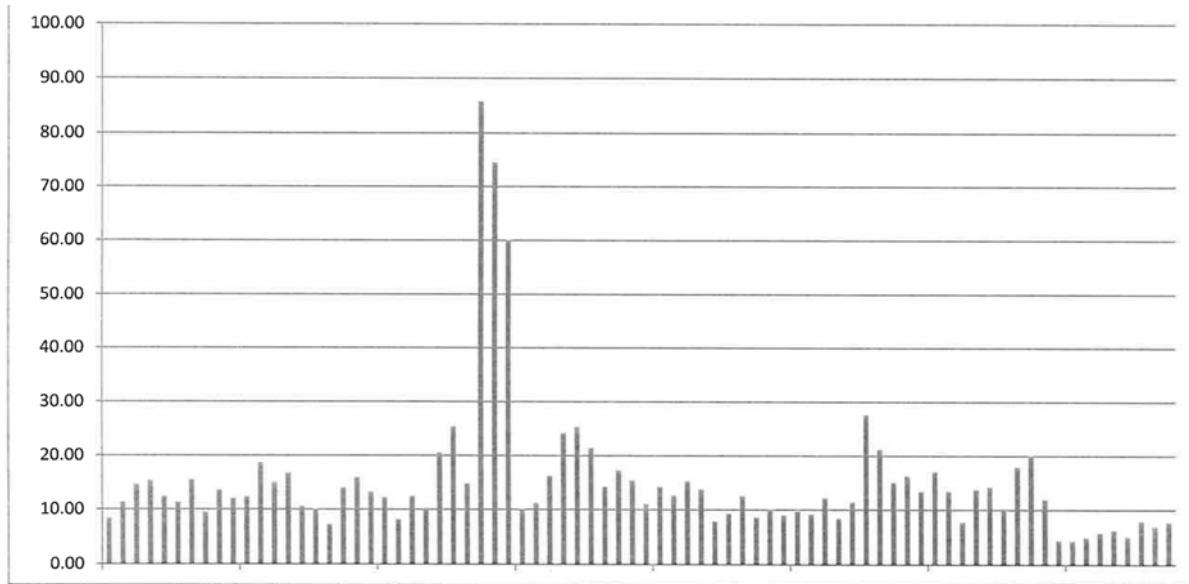


Figure 2. Bar chart of ratios of 0.80 post peak drift to ASD drift for all walls with aspect ratio of 2:1 or less. Note that ratios for three walls are much higher than the others (vertical access plots ratio in percent, horizontal axis plots one bar for each data point)

3. Recommended Parameters for Cold-Formed Steel Framed Shear Wall Seismic Equivalency

Based on the discussion in Sections 1 and 2 of this report, this section derives seismic equivalency parameters separately for CFS framed wood structural panel shear walls and CFS framed steel sheet shear walls, compares values to those used for the AC322 recommendations, and recommends equivalency parameters representative of these two groups of CFS framed shear walls. Evaluation is based on Type I walls only. The data used for comparison has not been adjusted for loading protocol.

Recommended Parameters for Wood Structural Panel (WSP) Sheathed Shear Walls

For derivation of parameters for wood structural panel sheathed CFS shear walls, the data group without Canadian softwood plywood (CSP) will be used, based on the assumption that the group without CSP is more representative of construction in the United States. Tables 20 through 24 explore the five seismic equivalency parameters with this data set, and compare to the AC322 wood-framed shear wall data set. Values are based on Appendix J.

Table 20. Ratio of peak capacity (V_p) to capacity at ASD (V_{ASD})

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Quantity Less Than 2.5	Quantity Greater Than 5
CFS WSP, no CSP	7.06	2.48	3.56	33%	4.73	2.39	1 of 62	10 of 62
Wood	5.4	2.3	3.1	63%	3.9	2.3	3 of 49	3 of 49

Table 21. Ratio of displacement at $0.80V_p$ ($\Delta_{0.8VP}$) to displacement at ASD (Δ_{ASD})

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 11
CFS WSP, no CSP	85	7.81	21.7	91%	41.4	1.97	12 of 58
Wood	43.4	6.4	20.30	45%	29.4	11.2	5 of 49

Table 22. Ratio of displacement at $0.65V_p$ ($\Delta_{0.65VP}$) to displacement at ASD (Δ_{ASD})

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 14
CFS WSP, no CSP	80	8.18	21.4	88%	40.1	2.65	22 of 56

Table 23. Drift ratio at 0.80 post peak load capacity ($0.8V_p$)

Framing	Maximum Ratio	Minimum Ratio	Average Ratio	COV	Average Plus 1 Standard Deviation	Average Minus 1 Standard Deviation	Number Less Than 2.8 percent
CFS WSP, no CSP	4.38	1.55	2.51	27%	3.18	1.84	38 of 58
Wood	5.5	2.3	3.6	21%	4.3	2.8	6 of 49