



DESIGN VALUES FOR VERTICAL AND HORIZONTAL LATERAL LOAD SYSTEMS

SUMMARY: Developments in cold-formed/light-gauge steel (CFS/LGS) design over the past six years have resulted in a proliferation of new designs for lateral load resisting systems. The majority of these designs have been focused on the vertical lateral load resisting system. The objective of this Tech Note is to provide a concise, basic design reference for CFS/LGS framed lateral load resisting systems. As such, this Tech Note focuses primarily on Code (UBC, IBC and IRC) based design values and widely used standard design methodologies in the United States. More detailed information about the data presented here may be obtained from the references provided or by contacting the LGSEA.

Introduction:

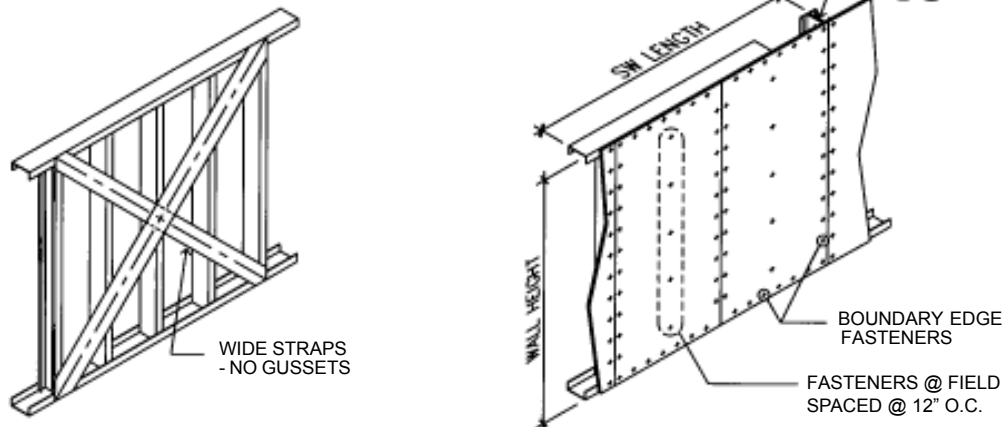
Advances in cold-formed/light gauge steel (CFS/LGS) framing as an alternative light framing material has resulted in the introduction of new design provisions for vertical (braced walls) and horizontal (diaphragms) lateral support systems. In addition, as the shape and configuration of buildings change (architecturally), the high performance demands placed on conventional, code approved, lateral load resisting systems (LLRS) have spawned the development of a host of proprietary LLRS to meet these demands. In this tech note, data applicable to design of the sheathed and diagonally braced lateral support systems is presented. The basis for development of the design values is also presented as an aid for designers who may be evaluating the claims of a proprietary system for compliance with the enforced building code.

Two basic light gauge steel frame vertical lateral load resist-

ing systems, as illustrated in Figure 1, are currently recognized by national building codes—shear walls and diagonally braced walls. The code-recognized shear wall systems comprise walls sheathed with wood structural panels (plywood and OSB), sheet steel, gypsum wallboard (GWB) and gypsum sheathing board (GSB). Diagonally braced walls typically utilized tension-only flat strap bracing, but compression bracing is also possible. In addition to providing design values (or procedures, as in the case of diagonal bracing), the building codes also make a distinction between design for wind and seismic loading. This distinction is based on the assumption that wind load effects on typical light framed structures are not dynamic or cyclic in nature. For all shear walls currently permitted in the codes, UBC and IBC, (plywood, OSB, GWB, GSB and sheet steel), tabulated design values are based exclusively on physical testing which included monotonic loading for wind design values and reversed cyclic loading for seismic design values.

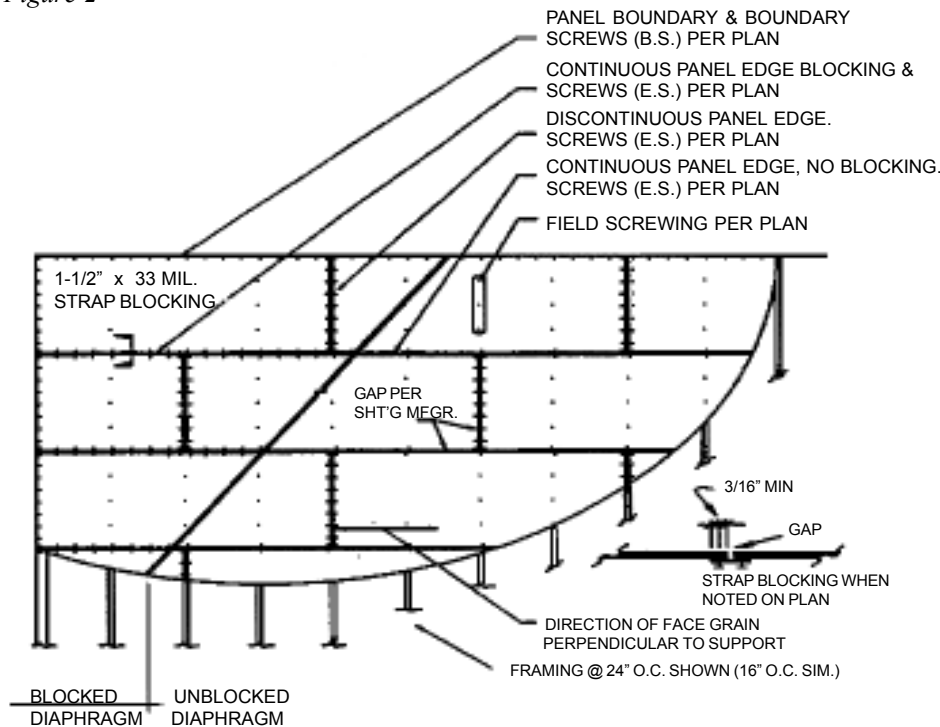
Code recognized vertical lateral load resisting systems

Figure 1



Basic layout of a cold-formed steel framed diaphragm

Figure 2



for seismic events.

As indicated earlier, the code design values in the 1997 UBC and 2000 IBC for wind and seismic design are based on testing. The scope of testing and some of the limitations imposed by the codes are a result of the need, at the time the codes were being balloted, to provide basic codified design information where nothing existed. Since that time, a number of issues have been raised regarding design (some specific to the code limitations). Some of these issues require additional testing to confirm what sound engineering practice recommends and others can be directly ad-

For light gauge steel framed diaphragms (horizontal lateral load resisting systems), as illustrated in Figure 2, the building codes provide no design values. In general, the same basic parameters that influence the performance of the vertical LLRS also influence the performance of a diaphragm. However, in addition, the panel layout configuration and blocking requirements also have a direct impact on the strength of the diaphragm. Although the codes currently provide no design guidance, some design information does exist in the form of a tech note (LGSEA 1998) published by the LGSEA. The design values recommended in LGSEA tech note are based on calculations using basic principles of mechanics (that is, connection strengths and adjustment factors similar to those adopted for wood-framed construction, Tissell 1993).

In the following sections, code provisions for walls, and the LGSEA recommendations for diaphragms, are referenced and discussed.

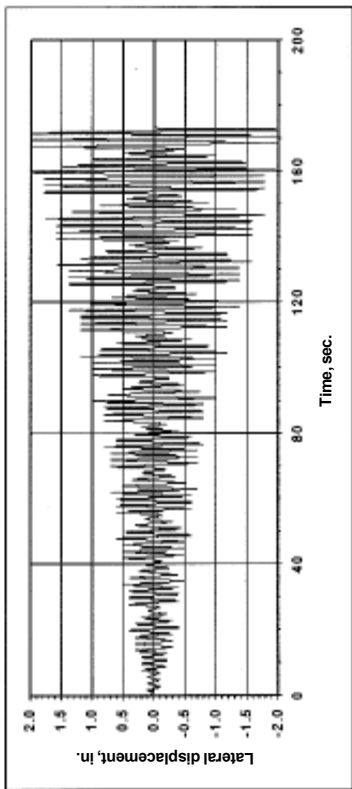
Code Approved Design Values for VLLRS: The main factors that influence the performance of shear walls appear to include the rate of loading (monotonic or reversed cyclic), framing thickness, screw size (shank diameter), center-to-center screw spacing, screw edge distance and the material used to sheath the wall. For diagonally braced wall systems, the code permits design to be based on a rational set of design procedures with specific force and drift limitations imposed

as a rational extension to existing work. Before discussing the background to the code values, it is of historical interest to note that testing for development of code design values occurred in two phases. The results of the first phase of testing were incorporated in the 1997 UBC and the results of the second phase of testing (combined with the results of the first phase) were incorporated in the 2000 IBC and IRC.

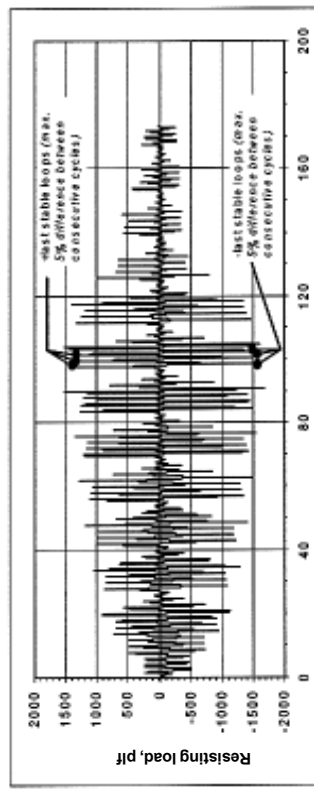
The typical test assembly in both phases of testing comprised 8-ft. high walls having a width of 2 ft., 4 ft. or 8 ft. corresponding to aspect ratios (ratio of the wall height to width) of 4:1, 2:1 and 1:1. For the 1997 UBC (the first phase of testing), all walls were 4 ft. wide except for the gypsum sheathed walls that were 8 ft. wide. The 2-ft. walls were part of the second phase of testing. In each test, the wall was anchored at both ends for overturning and along its base for shear transfer. The sheathing was oriented parallel to framing (all edges blocked), except for the gypsum wallboard/sheathing tests where the panels were installed perpendicular to framing with strap blocking at the abutting edges. The test protocol and typical results for the reversed cyclic and monotonic load tests, and the interpretation of these test results, are illustrated in Figures 3 and 4, respectively. Note that the resisting load referred to in Figures 3 and 4 is a measured load (distributed along the top of the wall) that resulted from a known imposed displacement at the top of the wall.

Reversed-cyclic wall load and response

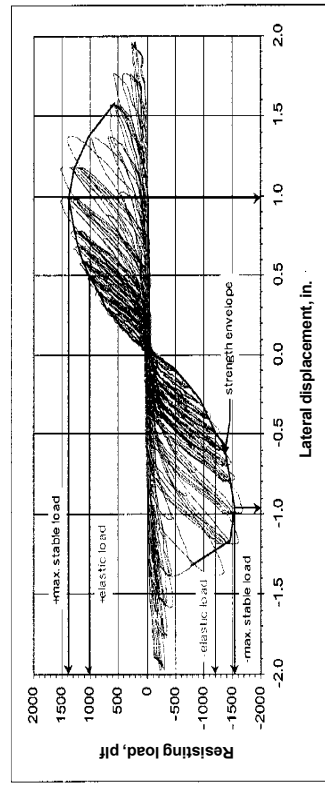
Figure 3



A. Applied displacement-time history (test protocol)



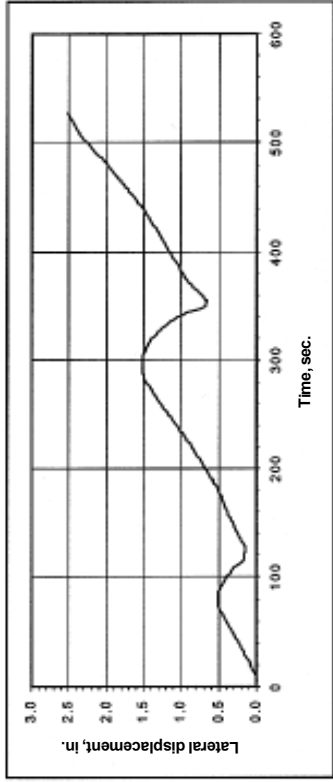
B. Resisting load-time history (force response)



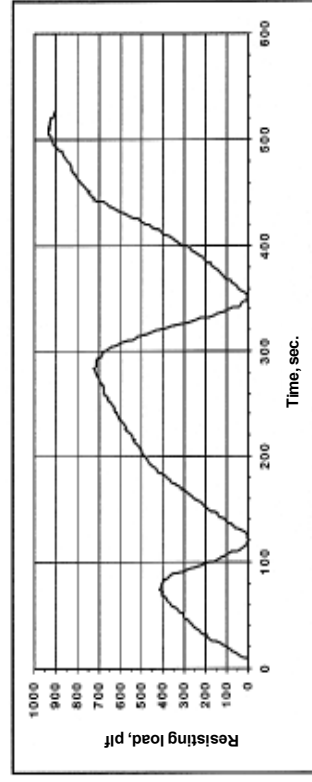
C. Overall wall response: resisting load versus lateral displacement

Monotonic Wall Load and Response

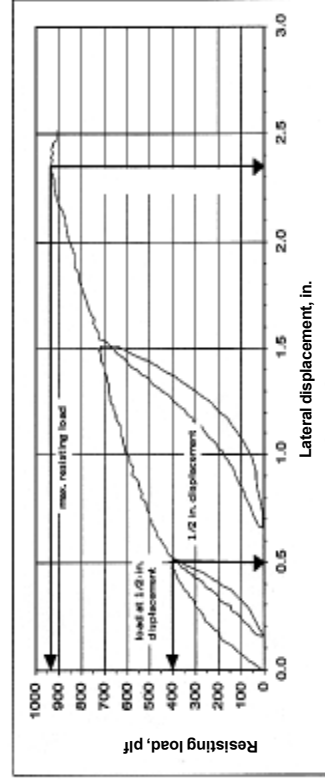
Figure 4



A. Applied displacement-time history



B. Resisting load-time history



C. Overall wall response: resisting load versus lateral displacement

Using the measured wall response, as illustrated in Figures 3(c) and 4(c), the code tabulated nominal design strengths for the wall configurations tested were based on the:

- Tabulated nominal design values for seismic design: Using Figure 3(c), the nominal wall strength was taken as the smaller of (i) 2.5 times the envelope strength (averaging positive and negative values) at 0.5 in. of lateral displacement and (ii) the maximum envelope strength (averaging positive and negative values).
- Tabulated nominal design values for wind design: Using Figure 4(c), the nominal wall strength was taken as the smaller of (i) 3.0 times the strength at 0.5 in. of lateral displacement and (ii) the maximum strength of the wall.

In the 1997 UBC and 2000 IBC, design values are tabulated in terms of a nominal capacity, R_n . Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) capacities are computed as R_n/Ω and ΦR_n respectively, where Ω is a safety factor and Φ is a resistance factor. The tabulated design values for seismic design are found on page 2-267 for provisions under the 1997 UBC and page 537 of the 2000 IBC. The safety and resistance factors for the UBC and IBC are summarized in Table 1.

Resistance and safety factors: 1997 UBC and 2000 IBC			
<i>Table 1</i>			
LOADING CONDITION	FACTOR	1997 UBC	2000 IBC
Seismic Design	Ω (safety factor)	2.50	2.50
	Φ (resistance factor)	0.55	0.55
Wind Design	Ω (safety factor)	3.00	2.50
	Φ (resistance factor)	0.45	0.55

Comparing the 2000 IBC tables with the 1997 UBC tables, it is evident that more design options are permitted under the IBC. This difference as indicated earlier is the result of additional testing (second phase) that was completed and approved by AISI after publication of the 1997 UBC.

The seismic and wind design values in the tables noted above assume that walls are fully sheathed and holdown anchors are provided at each end of the wall as required by calculation. All panel edges are assumed blocked for the shear wall applications. Unpublished monotonic test data has shown that one unblocked edge may reduce the fully-blocked capacity by as much as 50 percent.

The following comments regarding the tabulated design values are offered to assist engineers with extrapolation of the code design values in a manner that meets the intent and overall safety implied by the code:

- Some unpublished (proprietary) data exists which sug-

gests that for walls with the same material on both sides (and identically attached) the shear values listed in the tables may be doubled. Although these findings are encouraging, additional testing is needed to independently verify performance.

- Both the 1997 UBC and 2000 IBC call for gypsum board to be applied perpendicular to framing with strap blocking at abutting panel edges and solid block in the end bays. This limitation is based on the configuration used in the testing for development of the code values. However, the behavior of a shear wall suggests that it would be rational and safe to allow the use of the code values for gypsum board applied parallel to framing provided that all edges are, as required, blocked.
- For gypsum board shear walls, the code requires the use of both flat strap and solid end bay blocks at abutting panel edges (when the abutting edges does not occur at a stud). Because of the mechanism of load transfer at the abutting panel edge, solid-blocked end bays are not structurally necessary for the shear wall application. Note, however, that solid blocking may be required for torsional, lateral-torsional, and/or torsional-flexural buckling of studs.

- Under the 1997 UBC, an R-value of 4.5 would be required for sheet steel shear walls whereas R=5.5 is recommended for plywood or OSB shear walls. Under the 2000 IBC, the R-value for sheet steel, plywood and OSB shear walls is the same. The difference between the two code recommendations is due

to the fact that the sheet steel shear wall data was not available at the time when the revisions to the 1994 UBC were adopted for the 1997 UBC. Thus, under the UBC it is recommend that an R-value of 5.5 be used based on the information contained in the 2000 IBC. Engineers should however guard against using the higher R value for metal sheets with a thickness greater than those specified in the Codes.

In addition to the design data provided in the UBC and IBC, some basic research has been conducted on perforated shear walls. Based on this research, the 2000 IRC permits the seismic capacity of long walls with opening to be computed on the basis of the entire wall length with a penalty for the amount of openings (see Type II braced wall in Figure 5). The IRC provision (Section R301.2.2.1) is limited to structures located in Seismic Design Category (SDC) D_1 or D_2 . Using the perforated shear wall method, the maximum aspect ratio of all full height sheathed segments comprising the Type II wall is 2:1,

including the end segments of the wall. To apply this procedure, the required length of a fully sheathed wall (Type I is determined and this length is then multiplied by the appropriate adjustment factor (see Table 2) to get the required length of the perforated shear wall.

The primary advantage of the perforated wall method is a reduction in the number of holdowns required for design. In addition, as indicated in Table 2, for unrestrained opening heights less than or equal to one-third the wall height, the effect of the opening is negligible. The disadvantage of using this approach is the requirement for a longer wall (maximum 3 times the Type I length) for unrestrained opening heights greater than one-third the wall height.

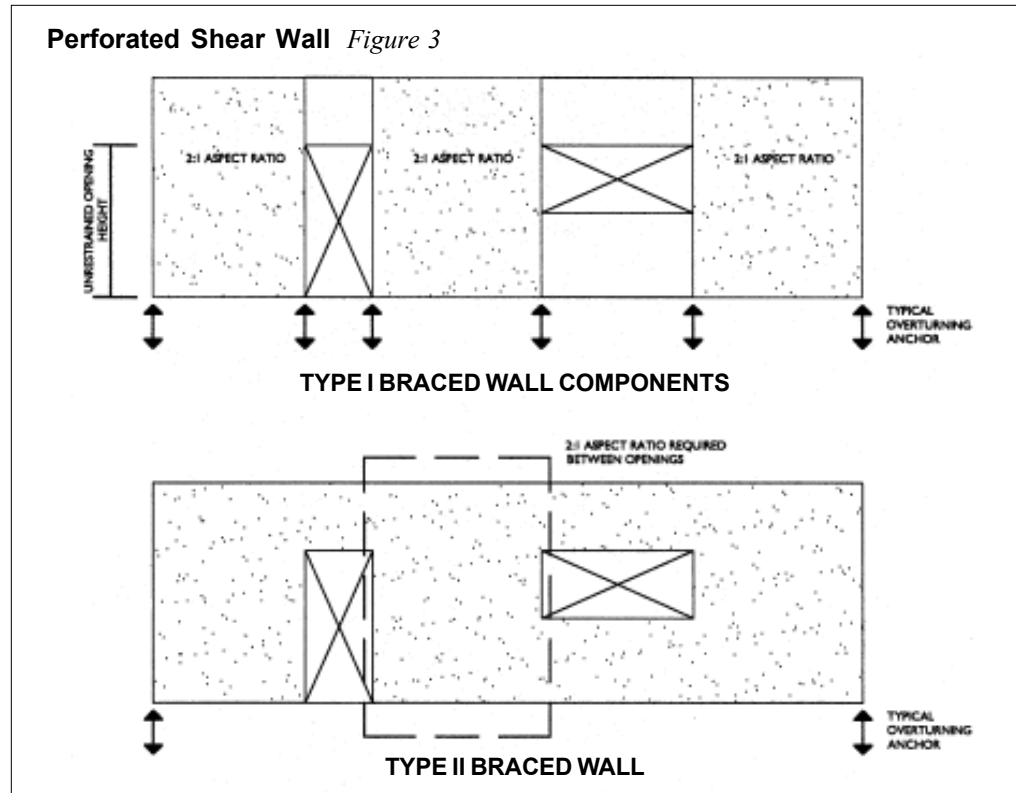
Turning now to the design of the flat strap diagonally braced walls, the basic considerations for the bracing element and the connection of the bracing element to the CFS/LGS frame are clearly provided in the codes. In addition to these provisions,

- engineers should be aware of the following:
- The use of wide straps may eliminate the need for gusset plates. However, where wide straps are used attention must be given to load path and the development of high drag strut forces in the track.
 - Where a single strap is used on one side, attention must be given to possible out-of-plane (plane of the wall) bending in the chord studs and top track that may result from eccentric application of the strap load.
 - For seismic design, the strap end connections should

- be designed for a load larger than the yield capacity of the strap (the tensile capacity or an amplified design load).
- Finally, straps should remain taut under a structure's dead weight.

RECOMMENDED DESIGN VALUES FOR DIAPHRAGMS

On the basis of calculations (LGSEA 1998), allowable stress



Adjustment factors for application of the perforated shear wall approach

Table 2

MAXIMUM UNRESTRAINED OPENING HEIGHT (feet)						
WALL HEIGHT H, (feet)	$\frac{1}{3} H$	$\frac{1}{2} H$	$\frac{2}{3} H$	$\frac{3}{4} H$	$\frac{5}{6} H$	H
8	2.67	4.00	5.33	5.93	6.67	8.00
9	3.00	4.50	6.00	6.67	7.50	9.00
10	3.33	5.00	6.67	7.41	8.33	9.00
PERCENT FULLY SHEATHED WALL	ADJUSTMENT FACTORS					
10	1.00	1.50	2.0	2.22	2.50	3.00
20	1.00	1.36	1.67	1.79	1.92	2.14
40	1.00	1.25	1.43	1.49	1.56	1.67
60	1.00	1.15	1.25	1.28	1.32	1.36
80	1.00	1.07	1.11	1.12	1.14	1.15
100	1.00	1.00	1.00	1.00	1.00	1.00

design values for diaphragms have been developed for different diaphragm configurations. The computed values are based primarily on the capacity of the connections and the shear capacity of the attached sheathing (plywood or OSB panels) with modifications for the panel grade, low diaphragm loads, panel buckling, and load combinations. The ASD diaphragm values are found in LGSEA Technical Note 558b-1. For LRFD, a factor of 2.5 times 0.55 (resistance factor) may be applied to the tabulated ASD diaphragm values.

NEW PRODUCTS AND PROPRIETARY ASSEMBLIES

As indicated earlier, proprietary braced wall assemblies have been developed for use with CFS/LGS framing. The unique-

ness of these products makes them unsuitable for inclusion in design codes. In lieu of code specific provisions, these products typically carry some form of a recognized "evaluation service report" (example: National Evaluation Report (NER) number or International Conference of Building Officials (ICBO ES) Evaluation Report number). Possession of an evaluation report generally implies that a product has met some basic acceptance criteria for its recommended/intended application. Typically, the evaluation process is critical to ensuring that products intended for structural use meet the intent of the applicable design codes. When faced with the situation of evaluating new products or assemblies for compliance with the code or equivalence with code assemblies, it is useful to keep in mind the general concepts illustrated in Figures 3 and 4.

References

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4. IRC (2000). International Residential Code for One- and Two-Family Dwellings: 2000. International Code Council, Inc., Falls Church, Virginia, January.
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6. Tissell, J. R. (1993). Calculation by Principles of Mechanics. APA-The Engineered Wood Association, Tacoma, Washington.

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