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INTRODUCTION

In a highly competitive and aggressive construction market, designers are always looking for new, proven ways to design safe, economical building structures. This is even more relevant in tract residential construction where a few dollars saved on one detail can affect who is awarded a project.

In residential construction, a complete load bearing light gauge steel (LGS) system is now somewhat commonplace in the United States. Architects and engineers who once designed almost exclusively with other conventional materials are now consider LGS as an alternative. Although, it is feasible to make a direct substitution of LGS for conventional wood framing, the response of the system (and its components) may not be similar. Thus, designers who are not familiar with LGS should make every attempt possible to become aware of the statistical variability of computed values determined from design guidelines. In this paper, a few of the important design criteria related to lateral load design are discussed.

Conventional light frame construction using LGS is similar to wood and in some cases one can make a direct "stick-for-stick" replacement of one material for the next. Where light gauge steel differs from wood framing is in the response of members to induced forces, and in some cases, flexibility and details of physical application/construction. One area worthy of consideration (post 1994 Northridge earthquake) is the lateral load response of LGS construction in high seismic zones, particularly the vertical lateral support system. For wood framed construction, vertical lateral resistance is typically provided by wood structural panels attached to the frame. In light gauge steel construction, the designer has at least four options for providing lateral resistance: wood structural panels, flat-strap X-bracing, metal sheathing, or a braced system. These systems can be generic or one of the many proprietary systems available in the residential market today.

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OUR CURRENT STATUS

Prior to 1997, engineers involved in seismic resistant design of light gauge (cold-formed) steel structures were required to demonstrate that vertical and horizontal diaphragms had sufficient capacity to resist code-based design forces. Until late 1994, this was accomplished primarily by reference to work reported by APA--The Engineered Wood Association (Tissell 1993) or Tarpy and Klippstein (1991). These early works were important for the industry, but as we now know they had severe shortcomings. Namely, the wall assemblies presented in the reports did not and do not represent current methods of construction and in some cases their use may result in undesirable structural responses. Additionally, the allowable design values were based on static testing and some engineers argued that the results may not be applicable to high seismic zones where dynamic or reversed cyclic behavior may result in reduced performance (ductility and strength).

In 1994 (post-Northridge), a series of static tests on shear wall assemblies with flat strap X-bracing, plywood, oriented strand board (OSB), gypsum wallboard (GWB), and gypsum sheathing board (GSB) were conducted at Santa Clara University. The tests were limited to 0.033-in. (20 GA) light gauge steel framing and the walls were either 6.00 in., 3.58 in., or 3.50 in. (stud depth). The tested assemblies were similar to those specified by designers and the results gave engineers more choices for their designs.

Following the 1994 Northridge earthquake, some jurisdictions (for example, the City of Los Angeles—COLA) implemented strength reductions on all code approved (UBC) wood-framed shear wall values with additional restrictions on edge distances and for 3-ply plywood. These reductions were based on observed damage to light framed wood structures. In the 1997 UBC, COLA's reductions were not adopted. However, limitations were imposed on the aspect ratio for high seismic zones and on the size of some framing members when design loads exceed 300 lb./ft. Subsequent discussion and preliminary reversed cyclic testing conducted by APA (Rose 1998) has demonstrated that COLA's strength reductions may be too severe.

Although there has been no evidence to date of poor lateral load performance of LGS framed structures during a seismic event, a limited two-phase research program (Phase I—Serrette et al. 1996 and Phase II—AISI 1998) was undertaken at Santa Clara University. The program was sponsored by the American Iron and Steel Institute (AISI) with support from many manufacturers. The research program provided some parity with wood and took design a step further by considering reversed cyclic response of wall assemblies. Over a two-year period, more that 70 static and cyclic shear wall assembly tests were completed. The tested assemblies incorporated plywood, OSB, flat strap X-bracing, thin metal sheathing, and GWB/GSB as the lateral resisting components. The assemblies covered aspect ratios that ranged from 1:1 to 4:1, and stud thickness that ranged from 0.033 in. (20 GA) to 0.054 in. (16 GA)—parameters that are common to the construction practice in the United States.

The first phase of the test program resulted in the development of design values by AISI, which were subsequently adopted in the 1997 UBC. The values were presented in nominal terms to prepare designers for a transition from allowable stress design to limit states design and provide engineers with a better understanding of capacity versus demand. Results of the second phase on the test program have been published by AISI (AISI 1998) and are expected to be submitted for code approval under the International Building Code (IBC).

For applications that involve horizontal diaphragms, design loads are typically low and designers have found that by using principles of mechanics, with appropriate modifications for steel, reasonable strength values can be determined.

INTERPRETATION OF CYCLIC RESPOSNSE FOR DESIGN

The current interpretation of cyclic test results for LGS framed shear wall assemblies is based on the idea that we can generate a load-displacement curve for an assembly using some form of an envelope of the cyclic test data. Figure 1 shows typical results from a reversed cyclic test with upper and lower bound envelope strength curves. Once these curves are defined, some criteria may be used to determine nominal strength values. One approach is to limit the nominal strength based on the lower of the ultimate strength and the strength at a specified displacement (typically considered the elastic displacement) amplified by some factor to account for inelastic behavior. The amplified displacement strength is based on the need to develop a minimum amount of ductility and provide some level of overstrength. The Structural Engineers Association of Southern California have proposed a more elaborate method of designing wood light framed shear walls and this method is discussed in some detail in a recent APA Research Report (Rose 1998).

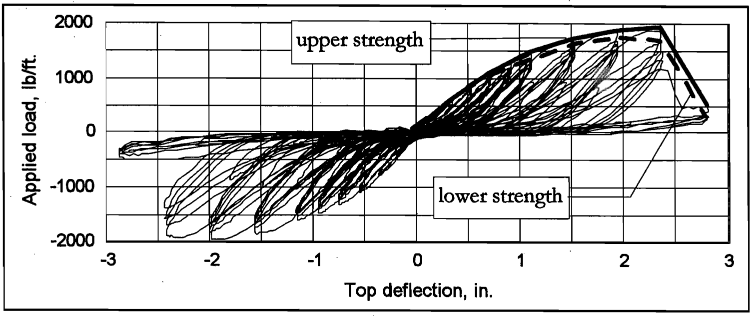

Figure 1. Shear wall hysteresis curve

Though expedient, the sole use of an envelope curve to define strength values may have neglected an important performance characteristic: hysteretic behavior. Two assemblies framed with identical sections and sheathed with the same material, but fastened with different fasteners, can exhibit identical enveloped curves. A closer evaluation of the hysteretic behavior may reveal significant differences internally. Because of this difference, the reduction factor used to compute design seismic forces should also be different. To address this issue, it may be more appropriate to use cumulative energy dissipated as the basis for establishing design loads. In this manner, the two systems with identical envelope curves can be assigned the same R-value (1997 UBC) but the resulting design strengths will be different.

OTHER DESIGN ISSUES

There are a few areas where work should be focussed to resolve issues faced by designers and set performance standards. As this industry forges ahead, more expedient construction techniques will be developed to aid contractors and reduce the overall cost of LGS framed structures. Paralleling these new developments, some effort will be needed to monitor recommendations made by different manufacturers. The following sub-sections highlight some of the areas in LGS framed design that may be useful to designers.

Shear Resistance of Gypsum Wallboard (GWB) in Seismic Zones

In all seismic zones in the United States, the 1997 UBC permits the use of GWB for vertical shear resistance in wood framed assemblies. Though not specifically prohibited, where seismic forces control design, no strength values are provided for GWB application in LGS framed assemblies. There may be some justification for limiting the use of GWB in seismic zones due to its low deformation compatibility. However, if lateral displacements are kept low enough, its appears that GWB can be depended on to provide lateral resistance in lower seismic zones (zones 1 and 2 per UBC). In addition to limiting lateral displacements, in low seismic zones overall structural redundancy may be higher than for the same structure in a high seismic zone. Thus, better performance may be obtained in low seismic zones.

Flat Strap X-braced Walls: Strap Overstrength

In the design of systems using flat strap X-bracing for lateral resistance, the straps are typically designed to reach their yield strength. Tests have shown that the actual yield strength of straps may be as much as 35 percent greater than the specified minimum strength. Thus, to limit the mode of failure to yielding in the strap, it is necessary to ensure that connections and other load transfer elements (chords, drag members, and anchorage) are designed to a load above that required to develop the actual strap yield strength (not the design load). In an effort to implement this concept in design, the 1997 UBC introduced an "overstrength factor", Ω_o, which is applied to the design load for evaluations of supporting components and connections. In the 1997 UBC, Ω_o is defined for all structural systems.

Walls Sheathed Both Sides

Another area that is not addressed in the current codes (for LGS) is applications with similar sheathing attached to both sides of the wall. For conventional light framed wood design, designers are permitted to double shear values. The same procedure may be applicable to steel frames. In all cases, however, designers are cautioned to ensure that all components and connections in a system are designed to capacity of the system or an amplified design load.

SUMMARY AND CONCLUSION

This paper discussed the current state of lateral load design for light gauge steel framed shear wall structures. Issues related to hysteretic behavior, deformation capacity, overstrength, gypsum wallboard, and sheathing both sides were presented. Although design values are available for different systems, it is suggested that a more detailed approach, using energy methods, be used to compare different systems.

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