Cold-formed Steel Framed Shear Wall Assemblies

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Since the early 1990s, cold-formed steel shear wall design has evolved in step with the increased understanding of their performance. Significant advances and additions in the provisions for these systems have occurred since their first inclusion in the 1997 *Universal Building Code (UBC).* These include the addition of values for steel and gypsum sheathing, allowance for thicker framing members, shear walls with openings, and deflection equations.

The 2006 International Building Code (IBC) Section 2210.5 requires that the design of light-framed cold-formed steel shear walls be in accordance with the 2004 edition of the American Iron and Steel Institute's (AISI's) *Standard for Cold-Formed Steel Framing–Lateral Design* [LATERAL-04]. The Lateral Design standard provides further information and clarifies design and detailing requirements of lateral-force resisting, cold-formed steel systems than what was previously available. This article discusses and illustrates the information and requirements of this relatively new standard.[1]

Light-framed Shear Walls

A typical light-framed shear wall transfers lateral loads, in the plane of the wall, through the mechanically attached sheathing, and into the framing members. The in-plane shear loads are transferred from the wall to the floor framing or foundation along the length of the bottom horizontal member (bottom track). The induced overturning forces are transferred through the vertical boundary members (end studs) and over-turning restraint system (hold-downs) at the ends of the wall as shown in Figure 1.

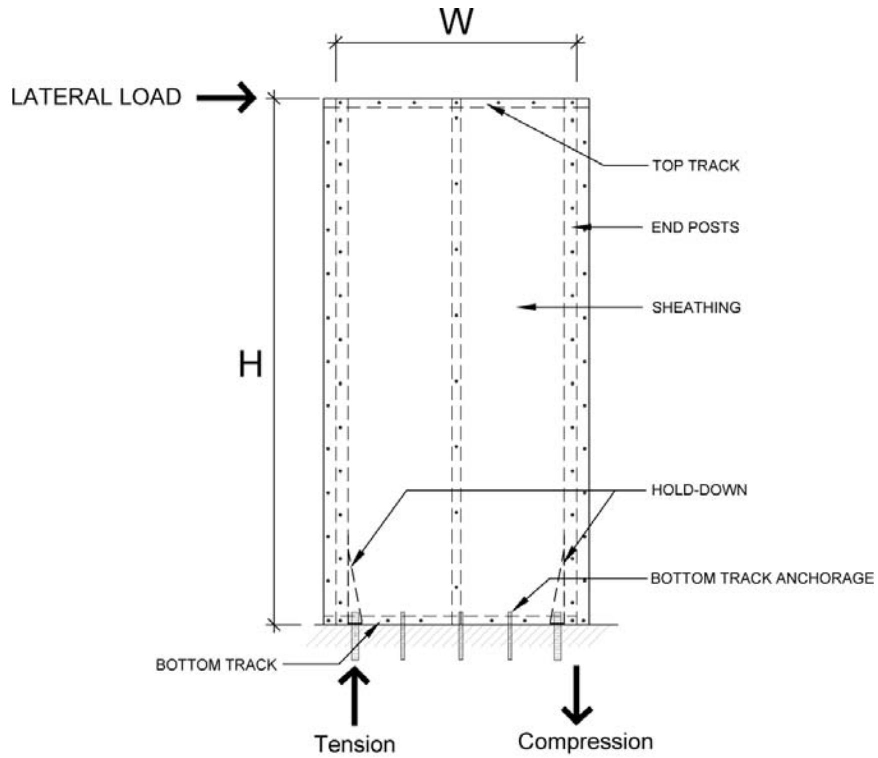

Figure 1

Typical lateral loads on shear walls result from either wind or seismic demand. Design wind loads are the actual expected forces, whereas design seismic loads are reduced based on the type of lateral system used, how many lateral elements are employed in the structure, and the level of seismic detailing performed. Designing for a reduced seismic load can significantly lower the cost of construction, but the tradeoff is damage in the structure during a major earthquake.

Typically, light-framed shear wall assembly strengths are determined through monotonic tests per ASTM International E 564, *Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings*, for wind load resistance. Another monotonic test standard that is used to determine sheathing strength and not system performance or strength, is ASTM International E 72, *Standard Test Methods of Conducting Strength Tests of Panels for Building Construction.*

Cyclic tests are performed to determine strengths for light-framed shear walls used to resist seismic forces. Cylic test protocols that have typically been used for light-framed shear wall assemblies are the Sequential Phase Displacement (SPD) protocol and the CUREE protocol for seismic resistance. The SPD protocol was originally developed for masonry walls and then required to be used for light-framed wood shear walls as well. However, research from around the world (e.g., Forintek Canada, Inc., Dan Dolan of Washington State University, Ario Ceccotti in Italy) has shown that the energy demand of this protocol is up to seven times of the actual demand to a low-rise building under during a design level earthquake. The Curee protocol is discussed in detail in Curee publication No. W-13 entitled "Cyclic Response of Woodframe Shearwalls: Loading Protocol and Rate of Loading Effects" with a printing date of May 2002. This cyclic test protocol considers a history of minor to moderate seismic events prior to the design level earthquake and is considered by many to be a more realistic test protocol.

Member strengths and system failure modes are important considerations for seismic design. Generally, a system that will fail suddenly is classified as 'brittle,' while a 'ductile' one is detailed to sustain more deformation without loss of load-carrying capability. This is typically done by designing the connections and members that are not supposed to yield—or are incapable of yielding (*e.g.* compression columns)—with a strength in excess of what is needed to fully develop the strength of the designated yielding elements. Use of the special load combinations that employ the 'over-strength' factor to determine the design level demand of the non-yielding components is one way to ensure yielding of the designated ductile elements. Codes encourage the use of ductile systems by assigning them a higher R-value, which results in lower required design loads.

Codes and Standards

Cold-formed steel-framed shear wall assemblies have been in the codes for several years. In the 1997 *UBC*, recognition was given to wood sheathed, 0.8- and 1.1-mm (33- and 43-mil) cold-formed steel-framed shear walls for wind and seismic resistance.

The aforementioned AISI *Standard for Cold-Formed Steel Framing–Lateral Design* includes further design information and requirements based on the latest research. These additions include:

- gypsum-board-sheathed assemblies;
- steel-sheathed assemblies;
- shear walls with openings (Type II [as explained later in this article]);
- 4:1 aspect ratio allowances in regions of moderate to high seismic risk;
- shear wall and diaphragm deflection equations; and
- wood-sheathed, cold-formed steel-framed diaphragm assembly strength.

Shear Wall Types

The AISI standard recognizes two basic types of cold-formed steel-framed shear walls (Figure 2). Type I is defined as a fully sheathed shear wall resisting in-plane forces, with hold-downs at each end of each wall segment, and where "detailing for force transfer around the openings is provided" if the wall has openings. A Type II shear wall contains multiple wall segments resisting in-plane forces, with wood or steel sheathing that contains openings between wall segments, and with hold-downs only at the ends of the wall. There is no requirement to detail for shear transfer around openings in a Type II wall.

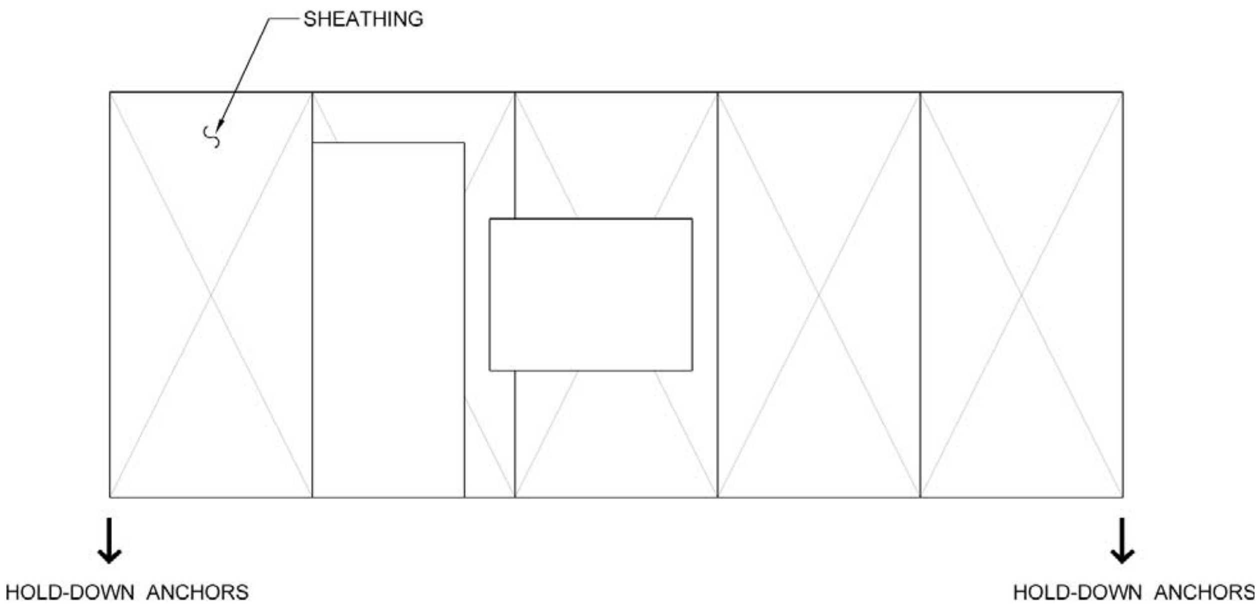

Figure 2 - Typical Type II Shear Wall

Basically, Type II shear walls use Type I published strength values modified by a coefficient based on wall and opening height (shear resistance adjustment factors). Type II shear walls also have special considerations, including design for a uniform uplift force along the wall bottom plates, in addition to the typical Type I design for uniform shear at the bottom plates.

Shear Wall Tables

The Lateral Design standard has three shear wall tables tabulating nominal strengths based on sheathing material, fastener spacing, framing thickness, and seismic or wind loading. The first table focuses on wood- or steel-sheathed assemblies resisting wind loads, while the second deals with gypsum board-sheathed assemblies resisting wind or seismic loads. The final table is for wood- or steel-sheathed assemblies that resist seismic loads.

The values in the tables represent the nominal (or in this case, ultimate) wall capacities. They have to be adjusted to obtain the appropriate design resistance—this is done by multiplying by a resistance factor (φ, phi) to obtain a load- and resistance-factored design (LRFD)-based resistance, or dividing by a safety factor (Ω) to obtain an allowable strength design (ASD) level resistance. Ω is 2.0 for wind and 2.5 for seismic, whereas φ is 0.65 for wind and 0.60 for seismic.

LRFD is defined by the Lateral Design standard as a "Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations." ASD is defined as a "Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations." LRFD load combinations include load factors where ASD typically does not, except that a reduction factor is applied to the dead load for the combination including dead plus seismic load and also a reduction factor is applied to the seismic load that is calculated per the IBC at an LRFD level and, thus, when ASD is used it must be converted down to the ASD level.

General Requirements and Design Procedure

Some of AISI's Lateral Design standard shear wall basic requirements include use of framing members with a minimum thickness of 33 mil, no shear panels less than 305 mm (12 in.) in width, and 610-mm (24-in.) maximum framing spacing. For seismic applications, framing member thickness cannot be beyond the limits set in the table. Summing the strength of shear walls with different sheathing material on the same wall face is not permitted. The wood-sheathed shear wall strength can be increased by 30 percent if gypsum board is used on the opposite side as permitted by the standard's first table.

Per the AISI Lateral Design standard, the wood- and steel-sheathed shear panels may be installed either perpendicular or parallel to the framing members and all panel edges are to be blocked. However, gypsum board sheathing must be installed perpendicular to the framing members. Additionally, where the shear wall tables permit, when the height (h) to width (w) aspect ratio of a shear wall segment may exceed 2:1, but not greater than 4:1, if the tabulated nominal shear wall strengths are reduced by 2w/h. Using an aspect ratio greater than 2:1 is not permitted for gypsum sheathed shear wall assemblies.

A general procedure for design of shear wall assemblies is outlined as follows:
1. Design loads (*e.g.* gravity, wind, and seismic) are determined.
2. Shear wall sheathing, fastener, spacing, and framing types are determined, based on published strengths in the code or standard.
3. The connection of the member delivering the shear load to the shear wall (collector) is designed.
4. Boundary members and supporting elements of the structure are designed.
5. Wall stud bracing (see AISI's *Wall Stud Design Standard* [WSD-2004]) is designed.
6. The required overturning restraint (hold-down) and anchorage are determined.
7. Top of shear wall horizontal displacement (story drift) is analyzed (and adjusted as necessary) to ensure compliance with code requirements. One may have to verify the initial load distribution based on the final shear wall stiffness if a rigid diaphragm is used.
8. The foundation (including anchorage embedment and transfer of overturning compression) is designed for all the induced forces.

Deflection

Whether the wall is governed by wind or seismic forces, consideration of top of shear wall horizontal deflection is important, as excessive deflection can result in member or assembly failure and collapse. Additionally, excessive deflection can lead to unsightly cracks or failures in finish materials (*e.g.* stucco, gypsum board, and glass windows).

There is currently no code drift limit for walls loaded in-plane for wind. However, Commentary Section CB.1.2. in American Society of Civil Engineers (ASCE) 7-05 (with errata), *Minimum Design Loads for Buildings and Other Structures,* states:

An absolute limit on inter-story drift may also need to be imposed in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the inter-story drift exceeds about 9.5 mm (3/8 in.), unless special detailing practices are made to tolerate movement.

For seismic loading, the drift limit is checked at the anticipated 'real' position of the shear wall as it undergoes some sort of yielding, or inelastic, response to the design earthquake. To accomplish this, the codes require the amplification of drifts computed at the LRFD level by a factor that is 0.70R for *UBC* designs and denoted as 'Cd' for *IBC* designs. Drifts must be computed by amplifying the calculated LRFD deflections. At the ASD level, this translates to a limit of approximately 12.7 mm (0.5 in.) for a 2.4-m (8-ft) tall wall.

The Lateral Design standard provides a deflection equation for blocked, cold-formed steel-framed, wood- or steel-sheathed shear wall assemblies. This equation is a function of four basic parts:

- linear elastic cantilever bending;
- linear elastic sheathing shear;
- non-linear effects; and
- hold-down deformation.

The vertical deflection due to hold-down deformation is to be multiplied by the shear wall height-to-width ratio (h/w) to obtain the hold-down contribution to top-of-wall horizontal drift (Figure 3).

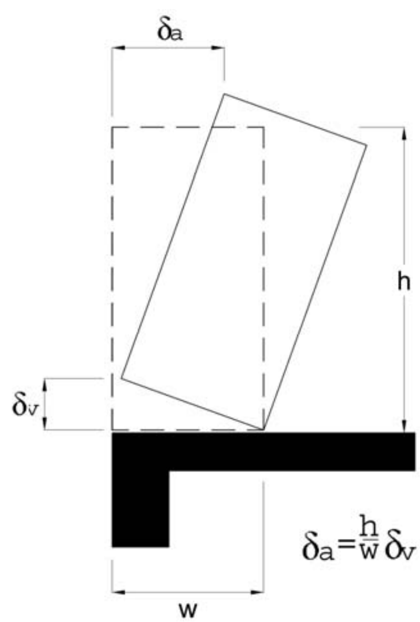

Figure 3 - Drift from Anchorage / Hold-down Deformation

The deflection equation is for a Type I shear wall. However, 2006 *IBC* Section 2305.3.8.2.9 states the deflection of wood-framed shear walls with openings (*i.e.* Type II) can be determined by taking the maximum individual deflection of shear wall segments and dividing it by the shear resistance adjustment factor used in the design of the wall. This same methodology perhaps may also be appropriate for cold-formed steel-framed shear wall assemblies.

Special Seismic Requirements

The 2006 *IBC* assigns an R-value of 6.5 for bearing wall systems using light-framed wood or steel-sheathed shear wall assemblies with no building height limit for Seismic Design Category (SDC) A through C and a 19.8-m (65-ft) building height limit for SDC D through F. As the R-value is high and the seismic design load consequently low, the codes and standards require special design and detailing considerations to better ensure the lateral system's ductility. The AISI Lateral Design standard specifies special requirements when one determines the design seismic forces for a cold-formed steel shear wall using an R-value greater than 3.0.

The Lateral Design standard requires that the strength of connections (for top chord splices, boundary members, and collectors), the boundary members, and the anchorage be designed for the amplified seismic loads (over-strength factor, or Ωo) or the maximum force the system can deliver. As previously mentioned, this is to prevent sudden failure, such as end-post buckling or a connection problem, and to better ensure the assembly's ductile behavior.

The maximum force the system can deliver to the shear wall is typically considered to be the ultimate strength of the member delivering the load to the shear wall, with consideration given to the actual rather than the minimum specified material properties. The maximum force the system can deliver to the vertical boundary elements and the anchorage is typically considered to be the ultimate (*i.e.* tabulated nominal) shear wall strength. Under the 2006 *IBC,* for a light-framed shear wall in a bearing wall system with an over-strength factor of 3.0, only when the design seismic load is 56 percent or less of the shear wall strength does the over-strength factor requirement for the vertical boundary members and the anchorage become less restrictive than the 'maximum the system can deliver.' (This is considering the case when the shear wall strength is the maximum force.)

However, since the standard's seismic shear wall table values are based on SPD cyclic tests, it should also be stated the nominal strength of the cold-formed steel-framed shear wall assemblies, under seismic loading, may be greater than what is listed. For some light-framed shear wall assemblies cyclically tested under the CUREE protocol, higher strength was obtained on the approximate order of 20 to 30 percent when compared to similar assemblies cyclically tested using the SPD protocol. One of the researchers that noted this is Felix-Antoine Boudreault in his thesis entitled "Seismic Analysis of Steel Frame / Wood Panel Shear Walls" dated 2005 at the McGill University in Quebec, Canada and under the supervision of Professor Colin Rogers. It is important to note the AISI Lateral Design standard specifies the foundation need not be designed for the amplified seismic loads.

The 2006 *IBC* permits the use of an R-value of 3.0 (and not 6.5) for "steel systems not detailed for seismic" in SDC A through C. Therefore, the special seismic requirements do not apply and either the published wind or seismic shear wall assemblies strength table values may be used as one is increasing the seismic design load by decreasing the R-value. The designer must still multiply the shear wall table nominal values by the appropriate resistance factor (φ) or safety factor (Ω) to obtain the available design strength when LRFD or ASD, respectively, is used.

The 2006 *IBC* permits the use of "light-framed walls with shear panels of all other materials" such as gypsum board. However, they are assigned a low R-value of 2.0 due to the brittle nature of these sheathing materials. Further, they are restricted to a building height of 10.7 m (35 ft) in SDC D and not permitted in SDC E through F.

Conclusion

As discussed in this article, the current codes and AISI standards and commentaries developed over the last several years provide a wealth of additional information and clarification for the designer and builder of lateral-force-resisting cold-formed steel systems. The 2007 version of the Lateral Design standard is currently being worked on by the AISI Committee on Framing Standards (COFS) Lateral Design Task Group and will offer additional solutions and direction for the designer. Also, research is continuing throughout the world to continue to expand the current knowledge and possiblities for cold-formed steel framed shear walls.

Notes

[1] An earlier version of this article appeared in the December 2005 issue of *Structural Engineer*. This author has also presented the information as a technical seminar at the 2006 Metalcon conference in Tampa, Florida.

About the Author

Jeff Ellis, P.E., S.E., is a senior engineering project manager for Simpson Strong-Tie, and is a member of the Structural Engineers Association of California (SEAOC), the American Society of Civil Engineers (ASCE), the Building Seismic Safety Council Technical Subcommittee 6 (BSSC TS 6), and the American Iron and Steel Institute (AISI) Committee on Framing Standards (COFS). He is also the current chair of the AISI COFS Lateral Design Task Group and is the current president of the Cold-Formed Steel Engineers Institute (CFSEI) board of directors. He can be contacted via e-mail at jellis@strongtie.com.