Another View of Shear Wall Hold-down Systems

By Ronald F. "Rawn" Nelson, S.E.

STRUCTURE® magazine has published a four-part series of articles on shear walls, hold-down systems and shrinkage compensation by Mr. Alfred D. Commins. These articles appeared in the August & November 2007 and March & April 2008 issues.

The series made several statements about the design and behavior of wood frame shear walls. Much of the same information has been presented at public hearings of the International Code Council Evaluation Service (ICC-ES). The following article contains alternative viewpoints regarding some of the opinions expressed by Mr. Commins. Responses from Mr. Commins to the author's comments below may be found online at www.STRUCTUREmag.org.

Promised Lateral Capacity

Parts 1 and 2 both stated, "The promised lateral capacity provided by shear walls is seldom achieved…." As readers, we might incorrectly infer that many shear walls are unsafe. If this was the author's intent, then the statement should have been accompanied by the author's understanding of what is considered to be the "promised lateral capacity" and supporting information for the claim that this capacity is "seldom" achieved.

Part 1 included the statement, "For shear walls to perform, four factors must be correctly and completely evaluated." The text went on to identify those four factors as "strength, system stretch or elongation, building settling/shrinkage and component serviceability (reliability)." Limiting shear wall performance to these four factors is somewhat naïve from a design perspective.

Are Shear Walls Needed?

Part 1 also stated, "Because of the low probability, we tend to overlook the importance of shear walls and shear wall connections." This suggests that important structural details are routinely neglected. However, design standards, product standards, test standards and evaluation criteria are regularly used by competent licensed professionals to ensure that designs, including shear walls and their connections, are appropriately specified and safe.

Shear Wall Failures

Field observations following catastrophic events and controlled laboratory testing have shown that not all shear wall damage occurs as a result of a deficiency in the hold-down system. However, Part 1 stated that "failed shear panels show three common failure modes." Let us examine each:

1) Splitting of sill plates: While hold-down deflection is one contributor to the splitting of sill plates, tests of shear walls with extremely rigid hold-downs also resulted in split sill plates. Even with the current code requirement of large washers on sill plates, increasing the hold-down stiffness by itself does not solve this issue. Part 1 suggested that adding such washers would "move the failure point from the mudsill to the nails in the shear panel." However, testing indicates that this is not always the case.
2) Splitting of vertical wood studs: Splitting at shear wall boundary members may occur with or without cyclic loading due to an earthquake. For example, splitting has also occurred in monotonic testing that has been associated with wind load conditions. Splitting is normally not due to compression as Part 1 suggests.
   Shear wall boundary members and hold-downs have been discussed in several articles and papers (*see the online version for references*).
   The common conclusion of these articles and papers was that boundary members can be designed at their net section for tension and bending when the connector is eccentric. In addition, following the 1994 Northridge earthquake, larger washers are required on eccentric bolted connections to reduce the bolt pull-through that had been observed to cause boundary member splitting. All of these conditions that could cause damage or failure to boundary members can be accounted for in the design process.
3) Nail pull-through, bending or breaking: The structural sheathing fasteners in the lower corners of shear walls will resist greater combined vertical and lateral loads. This has been shown to cause these fasteners to yield or fail sooner than other fasteners. This condition occurs whether the hold-down is rigid or not, and for both concentric and eccentric hold-downs.
   The ICC-ES acceptance criteria for hold-down devices (AC155) and shrinkage compensating devices (AC316) both effectively have a ⅛-inch maximum deflection limit at ASD capacity. This limit includes any fastener or device looseness. Shear walls do not fail from ¼-inch vertical deflection.

Shear Wall Hold-down Checklist

Part 1 did not describe the "expected performance level" of a shear wall when it presented a checklist. However, performance requirements are defined by building code requirements for strength, displacement and compatibility and are used by competent licensed professionals on a daily basis to create safe structures. Shear wall connections are not "evaluated based solely on system strength," as Part 1 suggested.

⅛-Inch Stretch and Loose Shear Walls

The series repeatedly cited "a system stretch or elongation" limit of ⅛ inch, which is not supported in any of the articles or papers or by any of the data from AF&PA, APA, CUREE or COLA-UCI testing.

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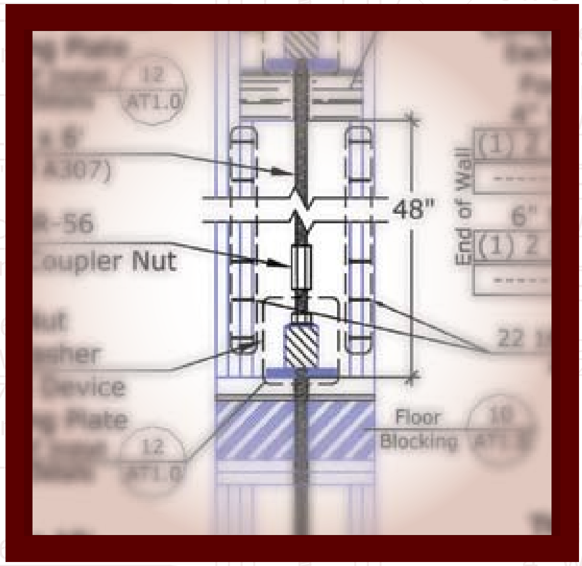
Part 4 referred to the 2001 COLA-UCI Light Frame Test Committee Report in support of the author's opinion that "Loose Shear Walls Don't Perform." A closer look at the COLA-UCI report reveals that loose nuts on hold-downs, intentionally allowing 0.2 and 0.4 inches of free movement, reduced initial wall stiffness but had no effect on ultimate capacity of the walls. These walls still performed the same as walls with tight nuts at the maximum capacity ranges.



This indicates that a shear wall will deflect more with a loose hold-down system, but will not necessarily fail prematurely as the series suggested. Engineers are required by code to limit shear wall deflection under seismic loads to minimize damage. These deflection calculations address hold-down deflection (including fastener slip and device looseness), rod elongation, wood crushing and shrinkage effects.

Building Settling and Shrinkage

Part 1 included a table of "Worst Case shrinkage for several typical building types." This table is prone to misapplication because it did not include any definitions of framing layout. Many of the shrinkage estimates in this table could not be reproduced using the information provided by the series. Estimates tabulated for LVL and PSL appear to be more than 50% greater than what would be consistent with the table footnotes.

A more complete reference on wood shrinkage is *Shrinkage Calculations for Multistory Wood Frame Construction*, published by the Western Wood Products Association as Technical Note (TN) 10. TN 10 includes a shrinkage calculation example for a specific construction configuration that accounts for framing orientation and provides framing placement suggestions to reduce shrinkage. TN 10 also includes considerations for connections, finish materials, brick veneer, doors, windows, plumbing and electrical and mechanical equipment.

Shear Wall Designs

Shear walls are designed based upon demand, not "the weakest element in a series". Additionally, design must consider minimum requirements of the building code and applicable standards. More information on the design of continuous tie-down systems in shear walls can be found in past issues of STRUCTURE magazine (*see the online version for references*).

These articles provide a comprehensive explanation of the procedures involved in the design of continuous tie-down systems, including examples, formulas used for calculating shrinkage and rod elongations, boundary post compression (parallel and perpendicular to grain), effects of skipping stories, drift, and structural details. There is no mention in either article of a ⅛-inch limit on system stretch, because it is far from being an agreed-upon design requirement. Examples of various limits are given; however, without an industry consensus, the final design choice is left to the engineer of record, subject to approval of the code official.

Both a subcommittee of the Building Seismic Safety Commission (BSSC) and the SEAOC Light Framed Wood Committee are working on common language for the design of continuous tie-down systems that will include suggested rod elongation limits. Neither group will ask the engineer of record to depend on tables with "overestimated floor shrinkage to avoid loose shear panels."

Rod Elongation

Few agencies currently impose rod elongation limits, and the requirements vary among the different jurisdictions that do have them. The one thing that should be commonly agreed upon is that rod elongation is properly measured between connectors. As shown by the testing referenced by Ghosh et al., the rod must properly account for configurations that skip stories or the shear wall may experience excessive deflection. A rod elongation "limit" by itself does not eliminate the need to include the rod elongation in the shear wall drift.

Designs should use the accumulated demand load (at skipped stories) for sizing the compression posts, anchor rod, bearing plates, couplers and connectors (including shrinkage control devices). The rod elongation and all effects of compression must be accounted for in shear wall drift design.

All rods of the same diameter elongate the same under an identical load, since rod elongation is a function of the elastic modulus, not the yield strength. AISC requires that the nominal area be used for rod capacity. However, for elongation of rods that are threaded over their entire length, net tensile area should be used, not the gross or nominal area. The net tensile area can be closely approximated by using 75% of the gross area. Exact values can be found in the AISC *Steel Construction Manual*, 13th ed., Table 7-18.

Strap, Hold-down, or Continuous Tie-down Systems and System Type Take-up Devices

It should be noted that the public hearing process used by ICC-ES has led to improved acceptance criteria for both hold-down devices (AC155) and shrinkage compensating devices (AC316). Researchers, engineers, builders and manufacturers all contributed. In response to a few of the points made in the series, consider the following:

1) Connectors are not typically designed to resist compression loads in shear walls.
2) Straps have been successfully used for years on thousands of structures. They can buckle in compression due to building settlement, wood shrinkage or overturning compression if any these effects are large enough. This bulging on rare occasion has created architectural problems and can add to the shear wall deflection like any hold-down system looseness. Before there is any reduction in strap capacity (50% was suggested), there should be adequate testing to justify such a reduction.
3) Eccentric hold-downs have been improved since their recorded performance during the Northridge earthquake. Boundary posts need to be designed at the net wood section for eccentric bolted connectors, as noted in AC155.
4) The Commins series expressed the opinion that large deflections that may occur at stacked hold-downs "will allow shear panels to fail at loads substantially lower than expected." As was previously discussed, testing by AF&PA, APA, the CUREE research project, the COLA/UCI and many others do not support this statement.

Shrinkage Control Devices

No shear wall hold-down system requires shrinkage compensation devices if engineers consider shrinkage and are able to control drift in accordance with building code requirements. However, such devices that comply with AC316 are a good addition to almost any project. Unfortunately, the Commins series included inaccurate claims regarding specific types of take-up devices to enable definition of one device as being preferable to another, when both comply with the provisions of AC316.

For example, the assertion in Part 3, Table 2 that ratcheting devices for a 1-inch rod have a 0.190-inches take-up backlash plus a 0.012-inch take-up deflection is totally inaccurate. The picture of this type device shown in Table 3 is of an improperly installed device, and the 0.322 inches of stretch is inaccurate. With respect to the referenced "backlash", this term is not found in AC316. However, these devices have been tested and reviewed in accordance with the provisions of AC316 so that the total device deflection plus looseness at allowable load cannot exceed ⅛ inch" as set forth in AC316 dated March 1, 2008. In most devices, the total is far less than the maximum allowed. Table 2 also has a line item for "Adjusted Capacity @ ⅛ inch"with what appear to be arbitrarily reduced capacities of shrinkage, with no supporting calculations.

Part 3 also stated that rod ratcheting devices could "freeze-up" after they have "partly advanced and then catch on the tips of the rod, stripping the threads." ICC-ES and COLA staff reviewed the operation of these devices with respect to this issue, and both agencies provided reports to aid code officials in accepting them. It should be pointed out that these devices do not require spring forces to engage the rod threads. Examination of the ratcheting devices will show that the geometry of the pieces cannot physically end up with more than half of the nut sections disengaged or at the tips of the rod.

Finally, Part 3 erroneously suggested that System #6 may freeze up with rod offsets. This is completely untrue. This system has been thoroughly tested full-scale with allowable rod offsets and is specifically engineered to prevent any lock-up under these conditions.

Conclusion

Shear wall designs in compliance with applicable codes provide reliable lateral resistance. Proper selection of connection hardware is an important factor in shear wall design. Shear wall connection hardware or devices may include a variety of types of hold-downs, connector types and shrinkage compensating devices to meet design requirements. There are several products approved by ICC-ES available on the market. No matter what devices or systems are used, proper design and construction are the most important ways to ensure the satisfactory performance of shear walls in buildings.▪

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