Hold Down Systems Key to Shear Wall Performance

Part 1 — Basics

By Alfred D. Commins

Loose holddown connections allow shear panels to lift. The result is cross grain mudsill failure as shown, or nails bending and breaking.

The promised lateral capacity provided by shear walls is seldom achieved in finished buildings, because one or more factors are missing. For shear walls to perform, four factors must be correctly and completely evaluated. These factors are: component strength, system stretch or elongation, building settling/shrinkage and component serviceability (reliability). All elements must be correctly evaluated if shear panels are to perform. If any factor is lacking, panels will not perform as expected.

This article discusses basic holddown performance as it relates to shear walls, evaluates current holddowns and provides insight into what works and why. Part One covers shear wall performance and the elements needed to allow shear walls to perform as intended. Part Two compares standard holddowns (straps, holddowns, stacked holddowns), complete systems (continuous rod, rod and cage and cable systems) and shrinkage compensating devices. Part Three covers "Designing Continuous Tie-Down System.

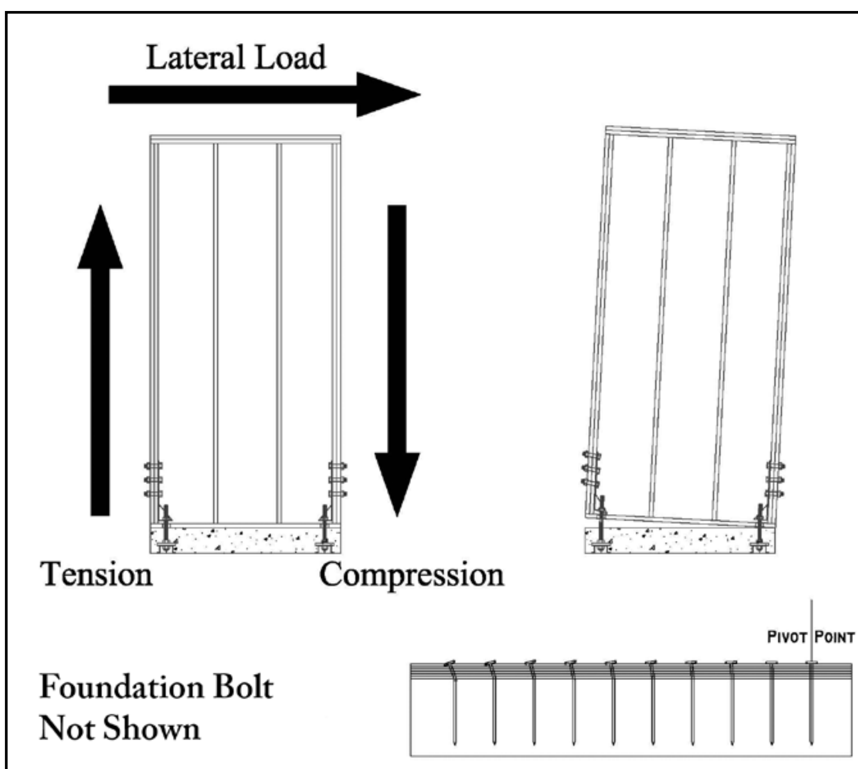
Are Shear Walls Needed?

Thousands of buildings have been built without "proper" shear walls and without proper holddowns. Most of those buildings are still standing and functioning after 50 years or more. So we need to ask the question: Are shear walls needed?

Large seismic and extreme wind events are low probability, high risk events. The probability a building will see the expected design load is perhaps 1% or less. Every 100 years or so, the building will see the design conditions. Because of the low probability, we tend to overlook the importance of the shear walls and shear wall connections. When the event occurs, there will be a high risk of serious building damage and a high risk of serious injury or death. So, even though there is a low event probability, the risk is such that we need to look at shear walls and shear wall connections very carefully.

How Shear Panels Fail

Numerous test observations and on-site field examinations of failed shear panels show three common failure modes. Mudsill failure is the most common. Under lateral load, panels tend to lift. The panel bends, rotates and twists the mudsill until the mudsill fails in cross grain bending. Engineers and code agencies have addressed this failure by installing large plate washers. The large plate washers move the failure point from the mudsill to the nails in the shear panel. The performance of the panel won't change, but the failure location will. The real solution is to tie the shear panel to the lower floor or foundation with a stiff, tight connection.



Unequal Nail loading due to loose or flexible connections.

Solid sawn wood can experience shrinkage exceeding ¾ inch per floor. This photo shows shrinkage nine years after installation.

The second failure mode is a splitting of the vertical studs when excess wood has been removed from studs and/or knots have weakened the studs. Under cyclic loading, studs are repeatedly put into compression and tension. There are two solutions to this problem: either use holddowns without bolts, or keep the studs in compression by using a continuous rod system for tension.

The third failure mode is a nail pull through, or a bending and breaking of the nails that connect the plywood to the panel chords (studs and plates). When a shear panel rocks, the mudsill tries to stay straight because it is nailed to the shear panel. At the same time the mudsill is bolted to the foundation. The rocking motion loads the nails in an unequal manner. The result is a progressive failure of shear wall attachment nails. The solution is to use a low stretch system with shrinkage compensators, and to keep system stretch under ⅛-inch.

Shear Wall Holdown Check List

For shear walls to approach the expected performance level, all holddowns should be designed to the following:

1) All elements are designed based on the strength of the weakest element in series.

2) Shrinkage and settling on a floor-by-floor and cumulative basis shall be evaluated. Shrinkage and settling are considered stretch without strain and are part of the system elongation. Shrinkage/settling are considered in all cases where the starting MC of any component exceeds 10%.

3) The elongation of all elements for a given connection are added together. Total system elongation shall not exceed ⅛ inch per floor. Runs may skip floors if the elongation does not exceed ⅛ inch. Elongation is determined at the system design load.

4) Components are evaluated for durability. Durability includes the ability to resist aging and corrosion, and to be unaffected by reversed loading. Elements that are difficult to install properly or which have a catastrophic failure mode shall not be used.

Discussion

Tie down systems are seldom limited by system strength. System strength is typically used by engineers and is the first step in design; however, other factors such as shrinkage, system stretch or system durability are the real limits to shear wall performance. When designing a hold down system, always start with system strength and expand from there.

Depending on the system, items to consider for system tensile strength may include nails, bolts, strap area, rod area, holddowns, wood cross section, bolt bearing, bolt rotation, connection eccentricity and bearing plates.

Every system consists of a series of connecting elements arranged in series to resist tensile forces. Because shear panels are alternately loaded in two directions, connections must not only carry the required tensile loads but must accommodate compression loads without being adversely affected. This does not mean the connection itself must carry the compression load; rather, the system compression loading will not weaken the connection.

Building Settling and Shrinkage

As builders combine studs, plates, plywood, etc., construction gaps are inevitable. Often evident are misaligned parts with cumulative gaps of ⅛ inch per floor. Some gaps disappear as the building is loaded and as framing continues, but gaps always exists. Settling is often combined with shrinkage for convenience, but it is separate and distinct. Even light frame steel stud buildings will see ⅛ inch per floor settling.

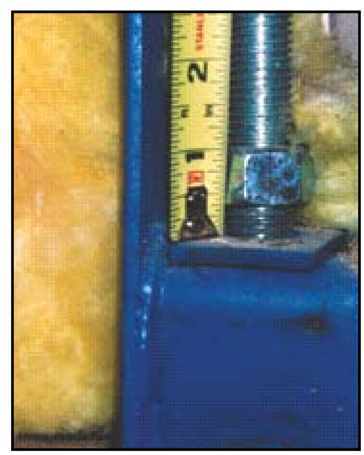
All light framing shrinks or settles. Depending on the materials, material orientation, weather etc., the shrinkage can be as little as 1/16 inch or as much ¾ inch per floor. Evidence of shrinkage is seen in bulging straps, buckled siding and cracked gypsum wall board. We sometimes dismiss movement as cosmetic because we don't see the gaps in the hold down system and don't understand the significance of loose connections.

If one considers shrinkage as a simple "Elongation without Load" (in terms of how it affects the building) and then further consider the lengths taken to limit stretch, one begins to understand the significance of shrinkage. Shrinkage is even more important when cumulative shrinkage is considered, as in a multiple floor rod system.

Simplifying Shrinkage Estimates

With literally 100s of variables, including wood species, manufactured or solid sawn joists, specimen size, grain orientation and moisture change, proper shrinkage determination can appear daunting. To successfully aim for a true average shrinkage, by definition, half of the connections will be loose and half will be tight. Half of the connections will carry the whole load, which is not a good plan.

However, if estimates are "rounded up", and shrinkage is overcompensated for, all the connections can be "caught". Every connection can be loaded in an even manner. Some may consider overcompensation a waste. The excess shrinkage capacity should not be looked as a waste, any more than a safety factor is a waste.



Wood shrinkage resulted in ¼ inch to 3/8 inch of shrinkage on every floor of a 5 story condo. This photo shows shrinkage 5 years after installation.

Building Shrinkage Table

Table 1 (see page 18) lists Worst Case shrinkage for several typical building types. It assumes shrinkage compensating devices will be used with the shear walls. It further assumes worst case shrinkage both on a single story and multiple story basis. These two assumptions greatly simplify shrinkage compensator selection, and virtually insure 100% of the wood shrinkage and settling will be accounted for.

continued on page 19

Table 1 - Building Grade and Wood Shrinkage

Level	Wood Subject to shrinkage	Per floor	Cumulative	Design Shrinkage
Grade "S" Buildings		**Light Gauge Steel Studs**		
⅛" per floor for settling only				
Level	Wood Subject to shrinkage	Settling		Design
		Per floor	Cumulative	Shrinkage
5	0	0.125	0.500	½"
4	0	0.125	0.375	⅜"
3	0	0.125	0.250	¼"
2	0	0.125	0.125	⅛"
Grade "A" Buildings		**LVL or PSL Joists, KD Plates**		
(Plates: Starting MC = 19%, Final MC = 10%, Joists no dimensional changes)				
Level	Wood Subject to shrinkage	Shrinkage		Design
		Per floor	Cumulative	Shrinkage
5	15¾"	0.358	1.468	1½"
4	15¾"	0.358	1.110	1⅛"
3	15¾"	0.358	0.752	¾"
2	17¼"	0.394	0.394	⅜"
Grade "B" Buildings		**Glulam or KD Joists, KD Plates**		
(Starting MC = 19%, Final MC = 10%)				
Level	Wood Subject to shrinkage	Shrinkage		Design
		Per floor	Cumulative	Shrinkage
5	15¾"	0.503	2.049	2"
4	15¾"	0.503	1.546	1½"
3	15¾"	0.503	1.043	1"
2	17¼"	0.54	0.540	½"
Grade "C" Buildings		**Solid Sawn Joists, Std Plates**		
(Starting MC = 30%, Final MC = 10%)				
Level	Wood Subject to shrinkage	Shrinkage		Design
		Per floor	Cumulative	Shrinkage
5	15¾"	0.966	3.944	4"
4	15¾"	0.966	2.978	3"
3	15¾"	0.966	2.012	2"
2	17¼"	1.046	1.046	1"

The shrinkage table simplifies estimates by making several simple assumptions: The final MC (moisture content) is 10% in all cases. Initial conditions vary depending on building type S, A, B, or C. Stated shrinkage assumes worse case conditions using Douglas Fir (Coast Type) with shrinkage in the tangential (worse case) direction. These assumptions are believed to be conservative and will overestimate shrinkage. Each designer should review each building on a case by case basis.

Shrinkage coefficient 0.00267in/in/%. USDA Handbook 72, Wood as an engineering Material, 1987, Table 14- 3. MC = Moisture Content. KD = Kiln Dried at 19% MC.

Wood Subject to shrinkage: Plates: 3 @ 1½" (4 for level 2), Joist @ 11¼" Initial Conditions

Grade "S" = Steel Stud Buildings. Use ⅛" per floor for "Settling"

Grade "A" Buildings: Plates KD at 19% MC. Joist (LVL, PSL, Wood "I" Joist) manufactured, shipped and installed dry. A ⅛" has been added for settling and some moisture changes.

Grade "B" Buildings: Plates and Joist materials KD at 19% Maximum MC

Grade "C" Buildings: Plates and Joists supplied wet-As Sawn

For a single story connection, or for rod and cage systems, use the column marked "Per Floor". For a true continuous rod system, use the column marked "Design Shrinkage". Check the table notes, and if necessary adjust the shrinkage. This table tends to overestimate shrinkage so that all shear walls will work, and will work together.

Holddown Stretch, Back to Basics

Holddowns restrain shear panels and help reduce panel rotation and uplift. To understand the importance of system stretch on shear panel connections, it is important to review the original testing upon which shear panels were rated (ASTM E-72). This standard was designed to test and rate sheathing and nailing strength. The test specimen was an 8-foot by 8-foot assembly. This test used two 1 ¼-inch diameter rods at one corner to resist uplift. This test did not include adjustments for rod stretch, did not include information on adjustments for wood shrinkage and assumed 100% connection reliability. The uplift load and deflection were taken out of the test. Two assumptions are made with this test. First, the 8-foot geometry would be corrected for narrow shear panels (we routinely build 4-foot long walls). Second, the building designer would use the building weight, a steel connection or a combination of the two to resist the required uplift.

Today typical shear wall connections are evaluated based solely on system strength. Evaluations usually overlook the effects of stretch, building shrinkage/settling and reliability on the connection capacity.

In order to replicate the performance of the original testing, the author suggests take-up devices be used in all connections and system stretch should not exceed ⅛ inch at the design load. This is in line with current or pending San Francisco and San Diego Requirements. Some jurisdictions allow 0.200 inches of rod stretch per floor. Based on extensive testing and 20 years of shear wall observations, the author believes that the total *system* stretch should be limited to ⅛ inch. Stiffness pays large dividends.▪

Alfred D. Commins is President and Founder of Commins Manufacturing Inc. Mr. Commins manufactures the AutoTight® Rod Hold Down System. Previously, Al was manager of Research and Development for Simpson Strong-Tie. Mr. Commins has over 40 US and foreign patents. Al can be contacted through ***www.comminsmfg.com*** *or at 360-378-9484.*