Connectors for Seismic Resistant Wood Construction – North American Design Philosophy and Trends

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1. Introduction

Large, damaging earthquakes do not seem like rare events anymore. This is largely because of the global, near real-time communication that we enjoy today. There is, in fact, "an app for that", and the one on my phone is constantly reminding me of earthquakes that are happening all over the world (there have been three events with M_w>5.0 just while writing this paragraph). In spite of this, the chance that any specific building will suffer damage from an earthquake during its lifetime is rather small. The consequences to individuals and to society, however, can be devastating if we do not design our buildings to survive large, rare earthquakes.

While it is certainly possible to design structures to sustain little, if any, damage during most large earthquakes the cost of doing so can be high. Consequently, public policy makers have allowed that proper seismic-resistant design can also result in structural damage to our buildings, even to the point where they must be demolished and rebuilt after an earthquake. This is viewed as an acceptable trade-off between protecting the public and making our infrastructure too expensive to live in. What is not acceptable, however, is significant loss of life through things like complete building collapse, such as that shown in Figure 1.

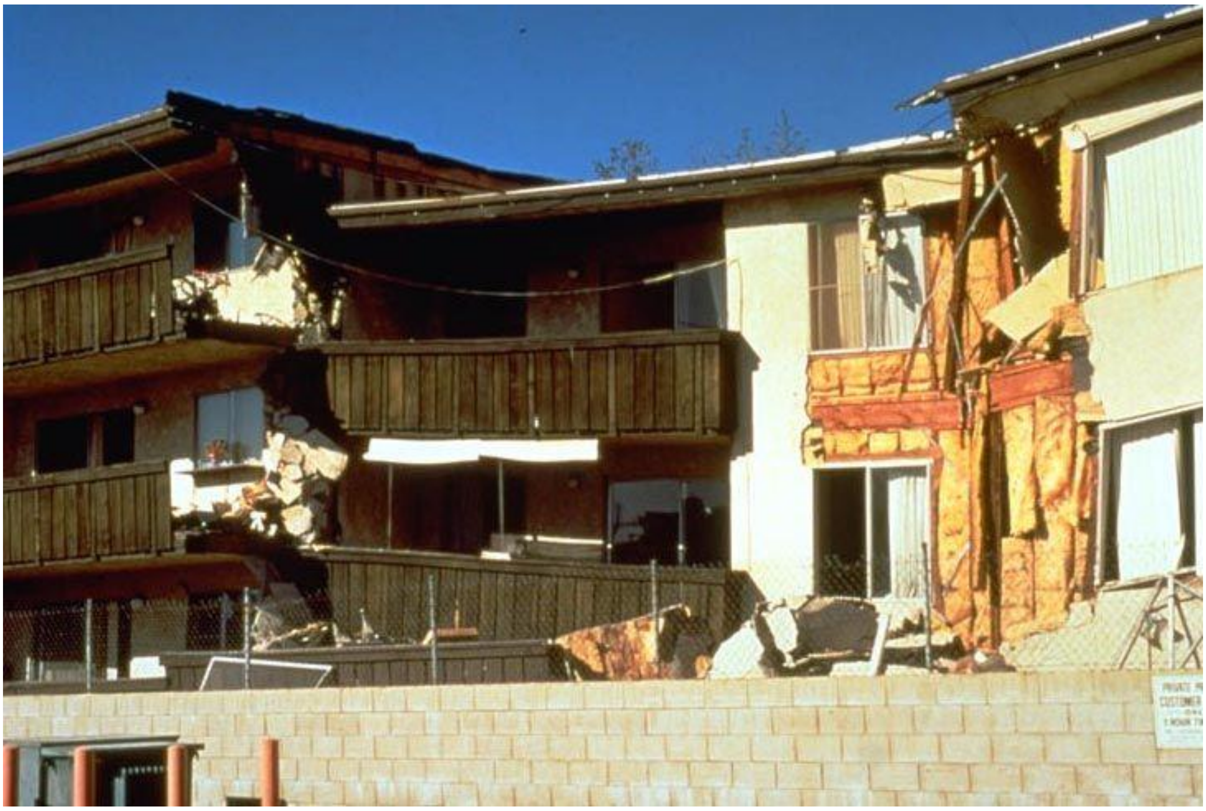

Figure 1: The Northridge Meadows apartment complex where sixteen residents died in the 1994 Northridge earthquake of southern California. The collapsed first story contained a mixture of living spaces and parking units. (Source: EERI)

2. Earthquake loads

While a full discussion of potential ground motions and how they are determined is beyond the scope of this paper, a basic knowledge is helpful in understanding the overall seismic design philosophy for buildings. It is intuitively understood that the ground will shake harder when you are closer to the source of the earthquake (fault) than when you are farther away, and that bigger faults are capable of producing bigger earthquakes than smaller faults. But what if your building is relatively close to a smaller fault and farther away from a bigger fault? Which earthquake/fault rupture scenario is going to cause harder shaking under your building? To help answer this question, seismologists from around the world have produced maps of anticipated ground shaking for designing buildings that consider this question (and more).

Another important consideration for these maps is the frequency of occurrence for a given size of earthquake (or level of ground shaking). Small earthquakes happen more frequently than large ones. If the maps are based on more frequent events, buildings will be under designed for the larger less frequent events. The maps for both Europe and the U.S. (approximately) provide "design level" shaking that is associated with an event that has only a 10% chance of happening in a given 50 year period. Put another way, design level shaking is associated with an event that happens, on average, every 475 years. It is the same as saying that what a building is designed for has a small chance of happening during the life of the building. This can lead to a lot of push back from owners and builders, particularly in areas with new, more stringent seismic design requirements, to the extra requirements engineered into the building construction to sustain this level of shaking without collapse. One of the benefits, though, is that the performance during smaller, more frequent earthquakes is expected to be quite good.

Whereas wind directly loads a structure by applying force to the outside of the building, earthquakes do not directly load a structure above the foundation. Instead, the foundations of the building move according to the ground movements, and if properly designed the building will have the ability to, more or less, stay up with the movement of the foundations. This means that every molecule in the building is going to be affected by the ground shaking, and this is why codes have seismic requirements to tie every part of the building together, including the walls, to keep the building from collapsing. Additionally, the fact that earthquake motions are quickly changing direction during an event allows buildings to be designed differently than for wind forces that tend to blow largely in one direction during an event. During earthquakes, buildings can be allowed to give, or yield, over several centimeters of movement because the earthquake motion that caused yielding in one direction will likely be replaced by another in the opposite direction, pushing the building back toward where it was. If we were to allow buildings to yield during wind events we might end up with buildings that get blown over.

3. Seismic design philosophy

When earthquake motions strike a building, a complex, dynamic, usually nonlinear vibration response occurs in the building. The mass of the building tends to be lumped around floors and roofs, and these masses tend to vibrate back and forth in a horizontal plane, sometime in phase with each other and sometimes out of phase. Controlling these movements is what is known as the lateral force resisting system (LFRS). In the U.S. this is further broken down into horizontal elements of the LFRS (floors and roof diaphragms) and vertical elements of the LFRS (the structural elements in the walls). Ultimately it is the movement between adjacent stories, or between the foundation and the roof for single story structures, that causes most of the damage to modern timber buildings in an earthquake. This differential movement is known as interstory drift (ISD), and it primarily impacts the vertical elements of the LFRS. As mentioned above, keeping ISD below the threshold that causes permanent damage can be expensive because it means adding considerable strength and stiffness to a building. The ultimate goal, then, of the seismic design philosophy is to provide a building with a LFRS that allows for controlled ISD past the yield point of the system (prevents collapse in the large, very rare earthquake) while not compromising those elements that hold the building up (the gravity system). See

Figure 2. While the gravity system may not play much of a role in the LFRS, ensuring that it has displacement compatibility with the expected interstory drift is an important part of the design. Fortunately for timber structures, most typical connection details in the gravity system also permit a large amount of ISD while maintaining gravity support. It is something that should be considered, though, when developing new connections.

Figure 2: Collapsed parking structure at California State University Northridge, 1994 Northridge earthquake. While extremely ductile concrete moment frames around the outside formed the vertical elements of the LFRS, the interior gravity support structure was not detailed for displacement compatibility to accommodate the interstory drifts permitted by the moment frames, leading to collapse of the structure even though the vertical elements of the LFRS were very good. (Source: EERI)

3.1. Seismic design loads and design resistance

Because our light-frame seismic design philosophy allows buildings to be pushed beyond their elastic limit during an earthquake, the building response becomes inelastic, or nonlinear. Performing nonlinear analysis of structures is very difficult, whereas a linear (or elastic) analysis is comparatively simple. To facilitate this simpler analysis approach building codes allow, for most regular structures, the use of an 'elastic pseudo acceleration design response spectra', or just (elastic) Design Response Spectrum, coupled with a 'seismic response modification coefficient'. This response modification coefficient is referred to as the 'q factor' in European standards and the 'R factor' in the U.S. The use of the Design Response Spectrum and the response modification coefficient is best visualized graphically as shown in Figures 3 and 4.

As stated previously buildings vibrate in response to an earthquake. The time it takes to complete one cycle of vibration is known as T, the period of vibration. It is important to know T because the vibration of the building actually amplifies the ground motion. This amplification varies but is proportional to T except at periods below T_S as shown in Figure 3. The horizontal axis of Figure 3 is the period of vibration starting at zero (i.e., infinitely rigid), and the vertical axis is building acceleration. Once T has been determined, the (elastic) Design Response Spectrum is used to determine what the maximum building acceleration will be – if it remained elastic. Since force = mass x acceleration, knowing the building mass and maximum acceleration allows us to determine a pseudo force for which to design the building for. This is known as the Base Shear, and it is represented by V in U.S. codes. At this point, though, the Base Shear,

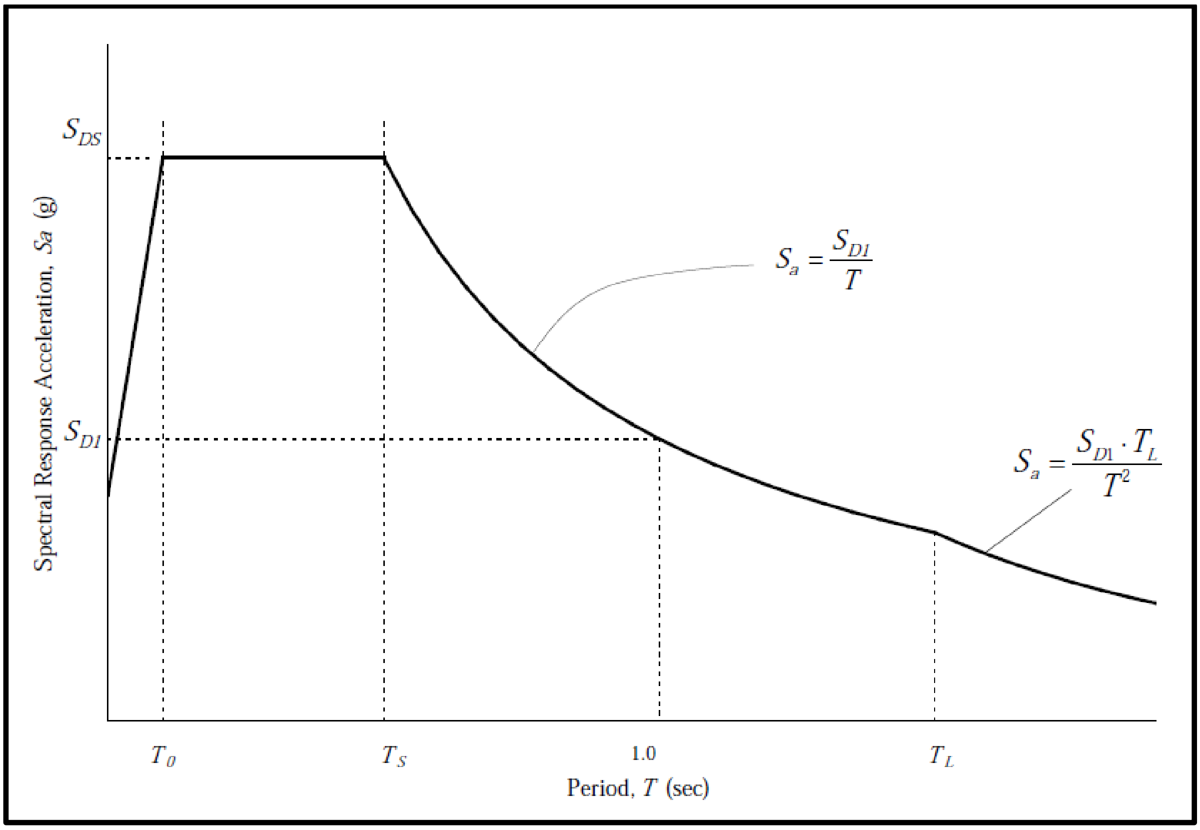

Figure 3: (elastic) Design Response Spectrum

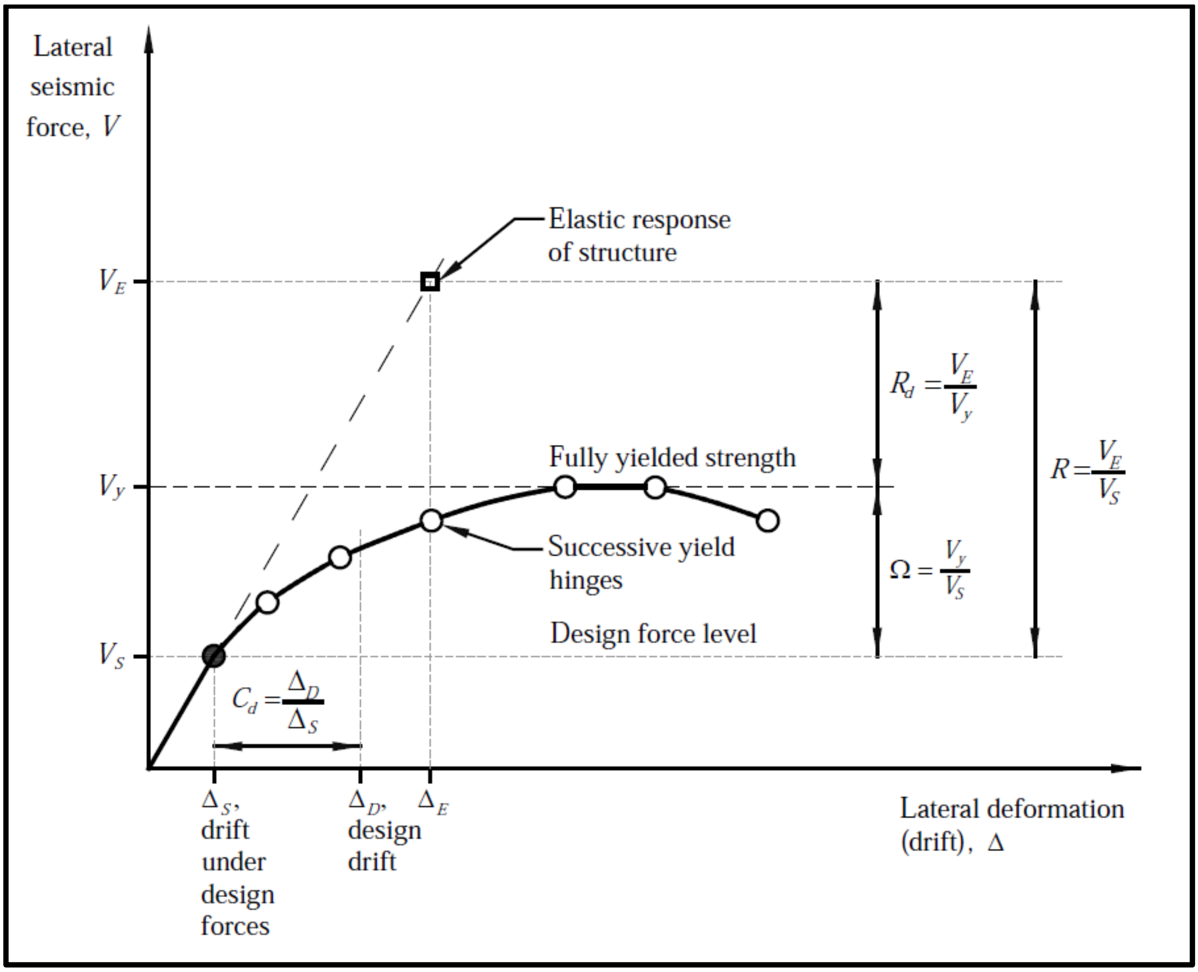

Figure 4: Determination of design loads through reduction of elastic response

V, is still a force that corresponds to the building remaining elastic – which it won't – and this is where the R factor (or q factor) comes into play (see Figure 4).

Figure 4 shows the relationship between the Lateral Seismic Force in the building (base shear), V, and the horizontal deformation of the structure, Δ. The dashed line shows the response of the structure were it to remain elastic vs. the actual inelastic response of the structure (solid line) as a result of designing it with less strength than it needs to remain elastic. V_E is the lateral strength the building would need to have if it were to remain elastic. The design strength, V_S, however, is determined by dividing V_E by the R factor. The R factor can be thought of as having two parts to it, one that accounts for the inherent overstrength above the design strength (Ω), and one that accounts for a reduction in demand due to the ductility available in the system (R_d). Once a building begins to yield its stiffness drops, which in turn lengthens the period of vibration, T, which as can be seen in Figure 3 leads to a reduction in demand on the building. In the U.S. the R factor = 6.5. This means that our design lateral demand is only 1/6.5 = 15.4% of the strength necessary to maintain an elastic response! Why does this work?

There are a number of reasons why this works for the U.S. First, it should be noted that the value of 6.5, while changed somewhat through successive versions of the U.S. building code, is largely derived from historical practice and observation of acceptable performance of light-frame structures when designed this way. Second, if you consider the "pushover" curve in Figure 4 to be just the response of a single nailed plywood wall, the ratio between the "fully yielded strength" as noted in the graph and the stated design strength, V_S, is approximately 2.0. This alone would leave us with an anticipated R_d factor (which is similar to the European q factor) of 6.5/2 = 3.25. But beyond this there is another very important reason for this practice leading to acceptable performance in earthquakes: the strength and stiffness contributions of "non-structural" sheathing (such as gypsum wall board and exterior cladding over the plywood or OSB) are ignored in the determination of V_S. This means that the actual value of Ω for typical light-frame structures is much larger than 2.0 It also means than when a building doesn't look like a "typical" light-frame structure, with lots of interior "non-structural" walls (laterally speaking, not part of the designated LFRS), then using R=6.5 should be reconsidered. Another case where R=6.5 may be the wrong value is in the lower stories of multi-story wood structures (say, greater than three stories) where more of the walls are designated as part of the LFRS, and the relative strength of the those "non-structural" walls that are left is much lower than in a typical structure.

One final thing to note in Figure 4 is the estimated inelastic drift, Δ_D. Because buildings are designed elastically for the reduced (fictitious) load $V_S = V_E/R$, the resulting calculated elastic displacement Δ_S is also fictitious. To give designers a better understanding of what the real seismic displacements will be like, the calculated values of Δ_S are multiplied by a displacement amplification factor, C_d. The displacements (drifts) thus computed (Δ_D) are then checked to make sure they are below code-defined acceptable inelastic drift levels, which vary normally between 2.0% and 2.5% of the story height. Successful seismic mitigation through yielding of the structure only works if: a) the building can accommodate the yielding drifts without losing (too much) strength; and b) the building does not drift so far that gravity then pulls the building down. This doesn't happen by accident and requires a carefully thought out system of sacrificial mechanisms and associated geometry that can sustain the yielding and drift in the structure.

4. Connectors for timber frame lateral force resisting systems

In North America this style of construction is called light-frame construction. It consists of walls framed with a repetitive placement of wood studs, 38 mm (1.5 in.) thick and either 89 mm (3.5 in.) or 140 mm (5.5 in.) deep, and placed at 40.6 cm (16 in.) on center. Additional studs or posts are used in areas of concentrated loads. Floors and roofs are likewise framed with a repetitive series of structural wood elements, ranging from solid sawn to engineered wood to metal plate connected wood trusses. Floors and roofs are typically sheathed with plywood or OSB, providing the necessary strength and stiffness for the horizontal elements of the LFRS. Depending on the earthquake hazard at the building site, walls may be either fully or partially sheathed with plywood or OSB as needed to provide the required strength and stiffness in the vertical elements of the LFRS.

As discussed in the previous section, somewhere in framing system a designated LFRS with mechanisms that can yield and sustain building drift without losing too much strength must be implemented. Both tests and observation from actual earthquakes have demonstrated that horizontal plywood diaphragms at roofs and floors tend to remain elastic in earthquakes for typical light-frame structures. This means that all of the yielding and inelastic response needs to be designed into the walls, or vertical elements of the LFRS. Fortunately there is an easy way to do this: create wood shear walls by nailing plywood or OSB sheathing to the vertical studs and top and bottom plates with smooth shank nails. Smooth shank nails are specified because they result in a more ductile response with more displacement capacity. Figure 5 shows a typical nailed wood structural panel shear wall arrangement. Resistance to sliding is provided by anchor bolts through the sill plate to the foundation, and resistance to overturning is provided by holdowns.

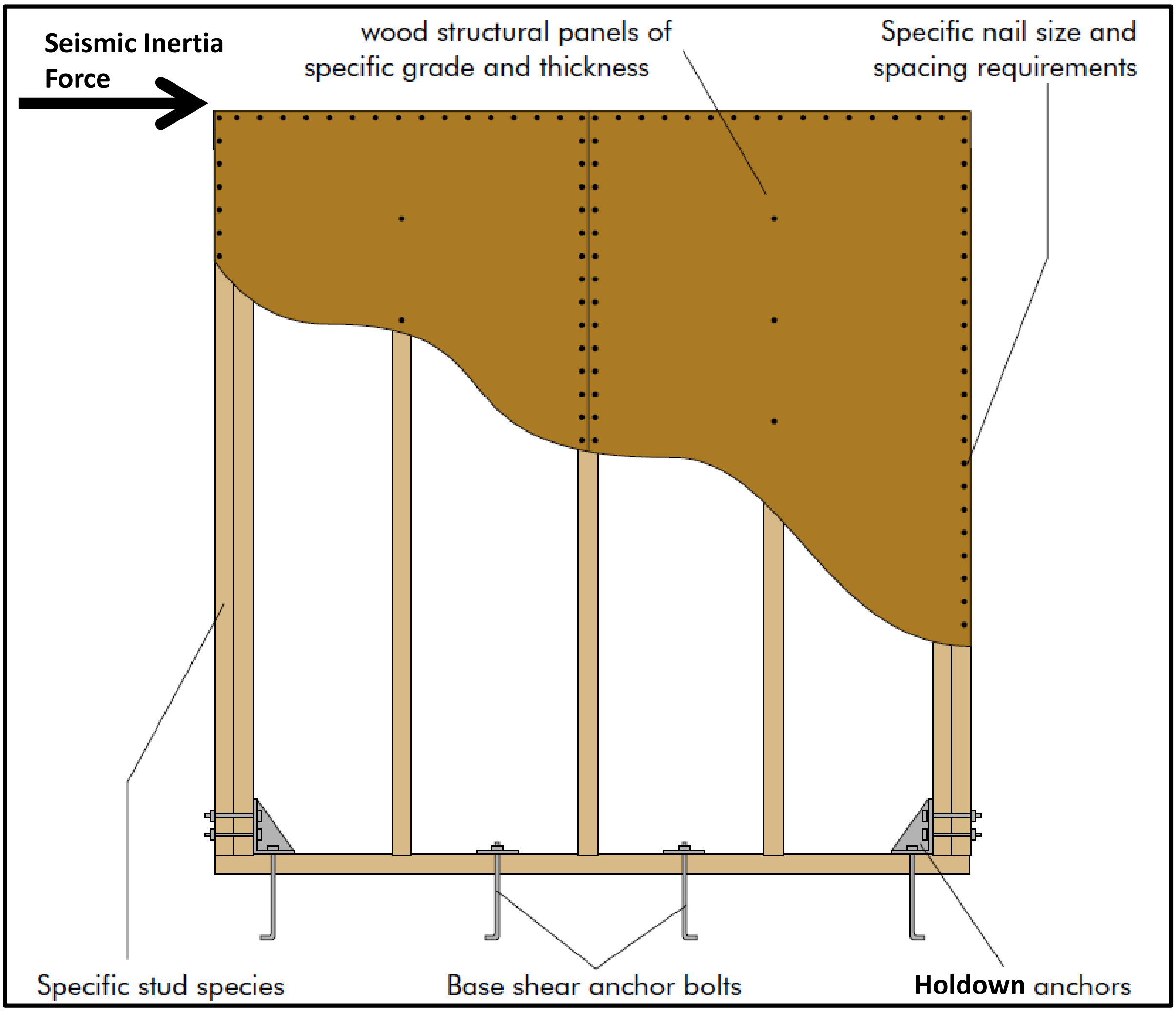

Figure 5: Nailed wood structural panel shear wall

Because the yielding of the nail joint connecting the sheathing to the framing provides enough ductility to the building to perform adequately in an earthquake, yielding of the anchor bolts and holdowns is not required. In fact, if the holdowns are too flexible, the end post will lift up off of the sill plate, which could prematurely fail the sheathing nail connection at the bottom of the wall, compromising the ability to move shear forces through the wall. Typical holdowns used in one- to three-story light-frame structures in North America are show in Figure 6. Above three stories the most common type of holdown is the continuous rod system as shown in Figure 7.

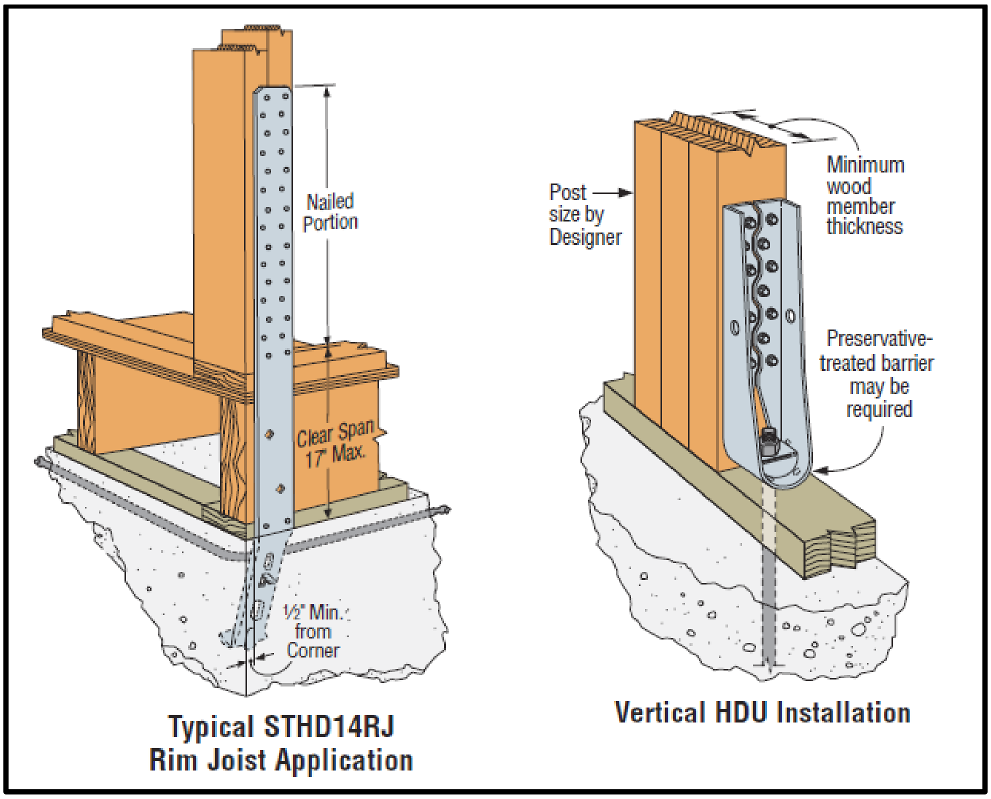

Figure 6: Typical holdowns used to resist shear wall overturning: cast-in-place nailed holdowns and holdowns connected to the post using self-drilling lag screws.

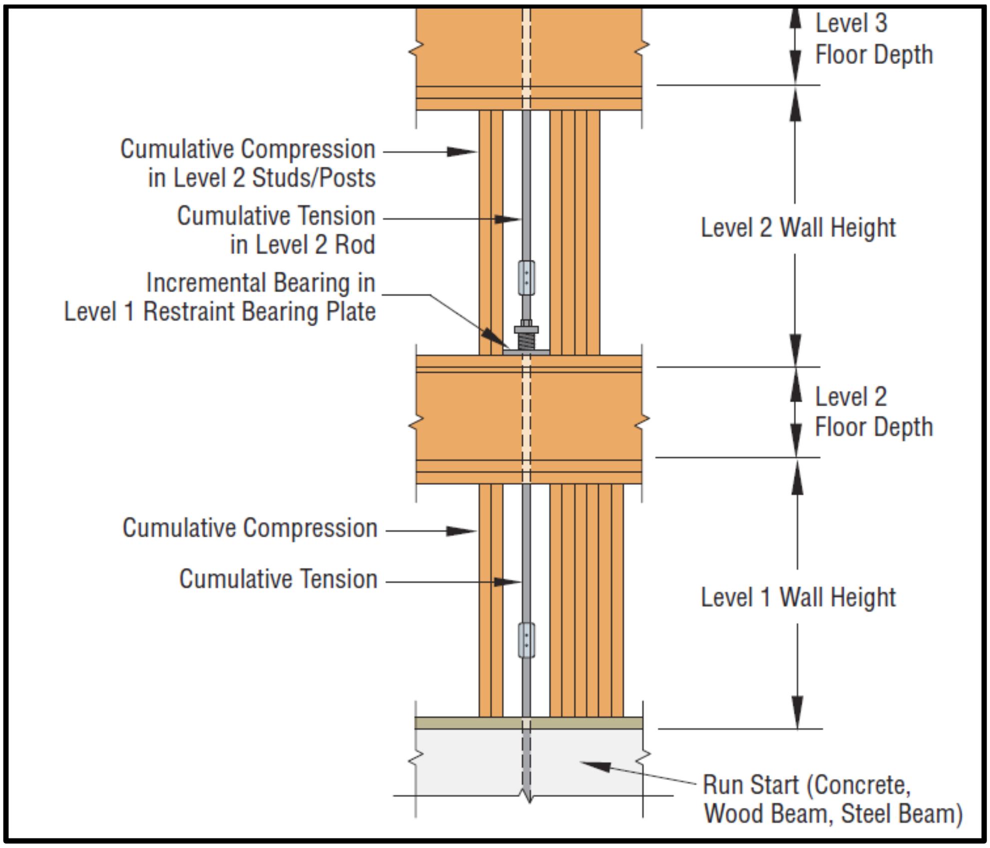

Figure 7: ATS continuous rod tie-down system

Often taken for granted in the seismic performance of light-frame structures is the standardized aspect ratio of plywood and OSB panels. These roughly 2:1 aspect ratio panels permit the wall to adequately deform in-plane without losing strength too quickly during an earthquake, even in a very long section of wall sheathed with many 2:1 panels. If they were made differently such that the panels were much larger and fewer were used, the wall performance would not be as good. This underscores the point that it is not enough to have a ductile connection in a wood building – the geometry of what the connection allows to move must be considered when assessing the seismic design and detailing rules for the system.

It is also important to remember that the shear wall is just one component of the LFRS. The job of the shear wall is to move the mass of the structure when the foundations move in response to an earthquake, so it is critical that there is a good load path into the mass of the structure to do this. Wood diaphragms support much of the mass of the structure, but they are designed for uniform shear forces along their edges. This requires a collector element, or "drag strut", to absorb the uniform shear force and push (or pull) the collected force to the shear walls. In light-frame construction these struts are usually either double 38 mm thick wood plates at the top of the wall or an element of the gravity framing system in the roof or floor system. Double top plates generally have staggered splices, and when necessary flat strap connectors are used to add tension capacity to the spliced plate as shown in Figure 8. Other common connectors used to tie collectors to each other or to walls are shown in Figures 9 - 11.

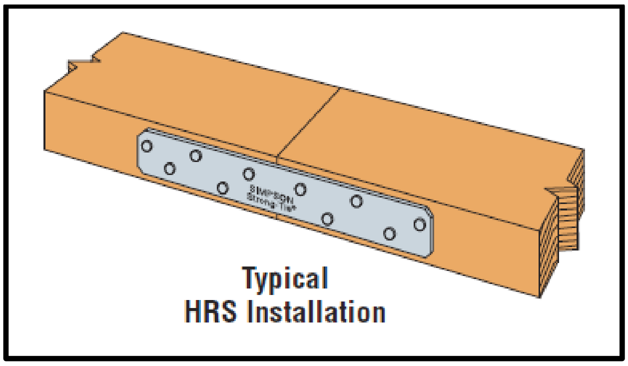

Figure 8: Flat strap tension splice at edge of 38 mm wall plate

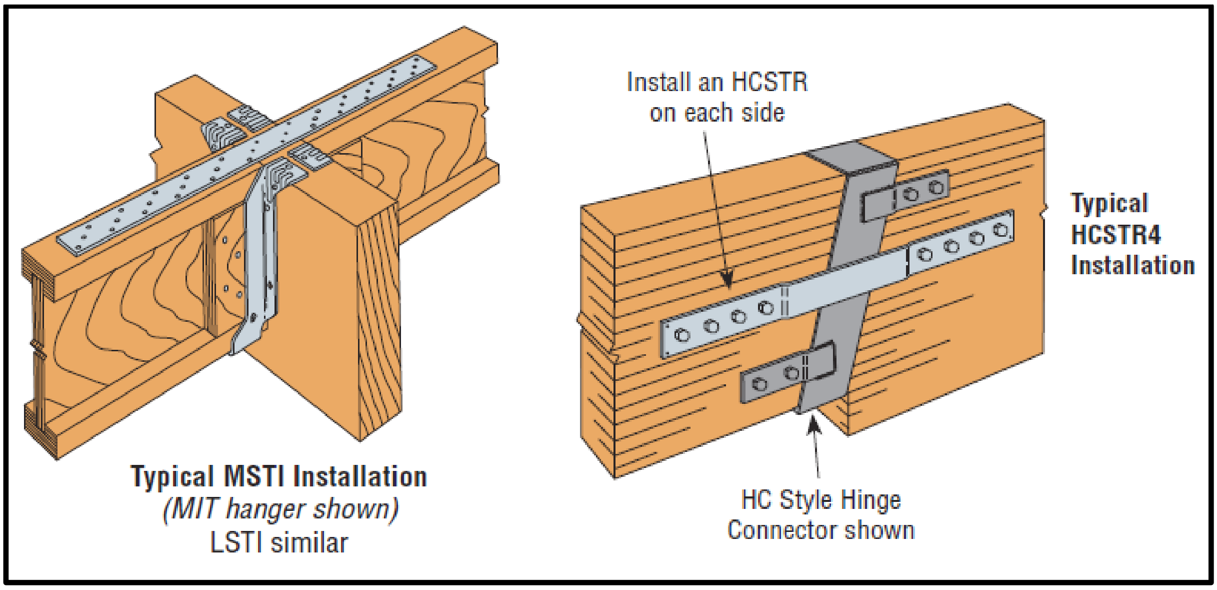

Figure 9: Common drag strut and collector splices

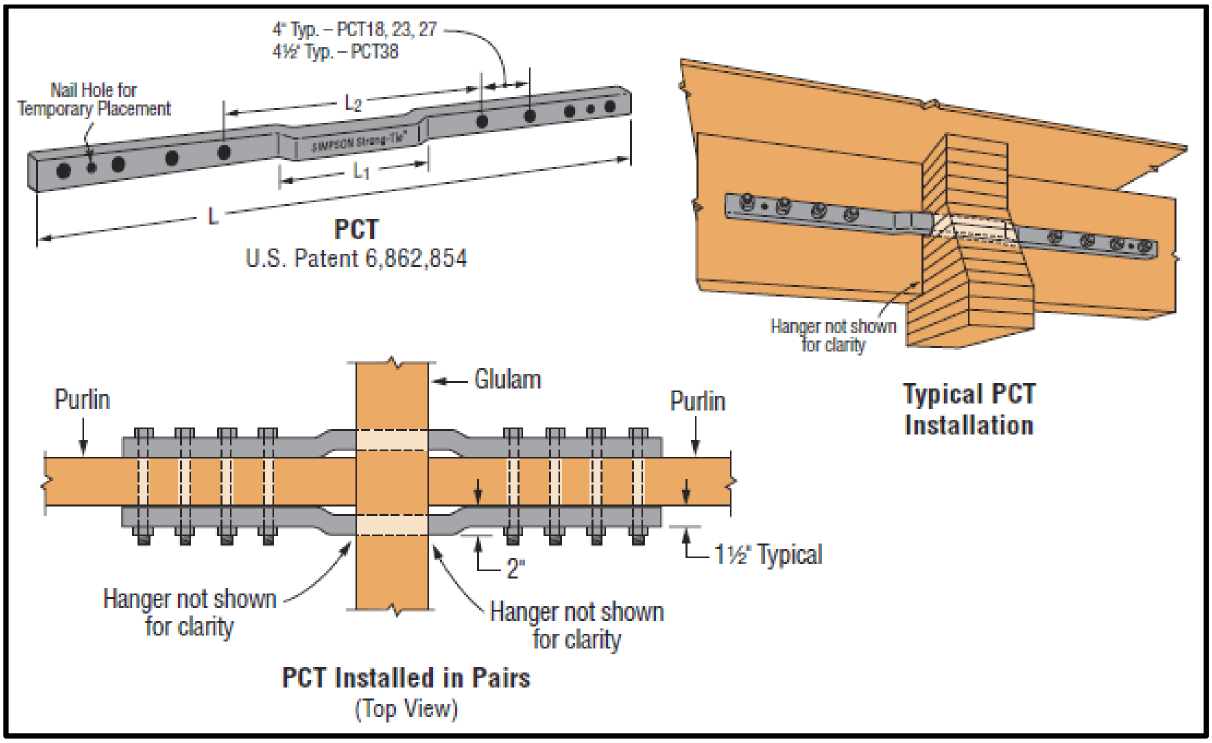

Figure 10: Heavy duty collector tension splice connector

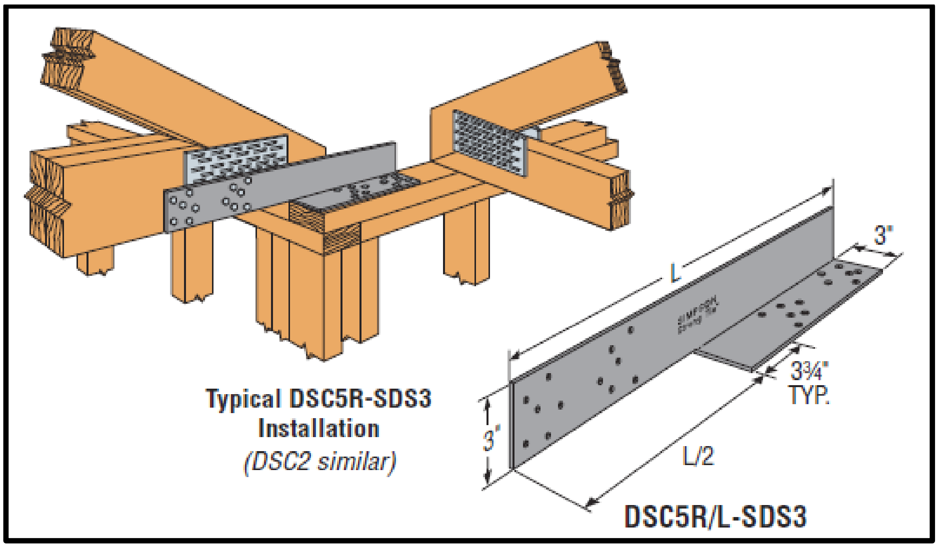

Figure 11: Drag strut connector between roof truss and wall line

In light-frame construction the plywood or OSB diaphragm is often separated from collectors and struts by the depth of the framing member. Since the shear in the diaphragm needs to be transferred to the collector, either the framing member or blocking between the framing members is used to transfer the load to the collector. At the bottom of the framing member or blocking a metal connector is typically used in seismic areas to transfer shear into the collector as shown in Figure 12. Past earthquakes have revealed that toe nails can experience brittle failures in these connections and should be avoided.

Occasionally the architectural demands of the structure restrict the available wall space to the point that traditional nailed wood structural panel shear walls can longer provide the needed strength and stiffness, requiring another solution for the vertical elements of the LFRS. When this happens designers often turn to one of two specialized solutions: proprietary high-capacity pre-fabricated shear walls (Figure 13) or ductile steel special moment frames (Figure 14).

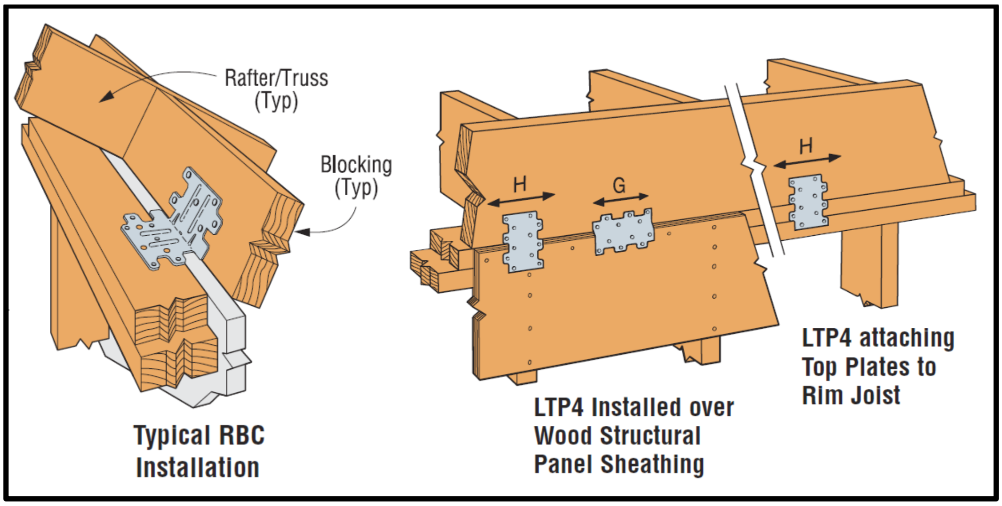

Figure 12: Typical seismic shear transfer connections in light-frame construction

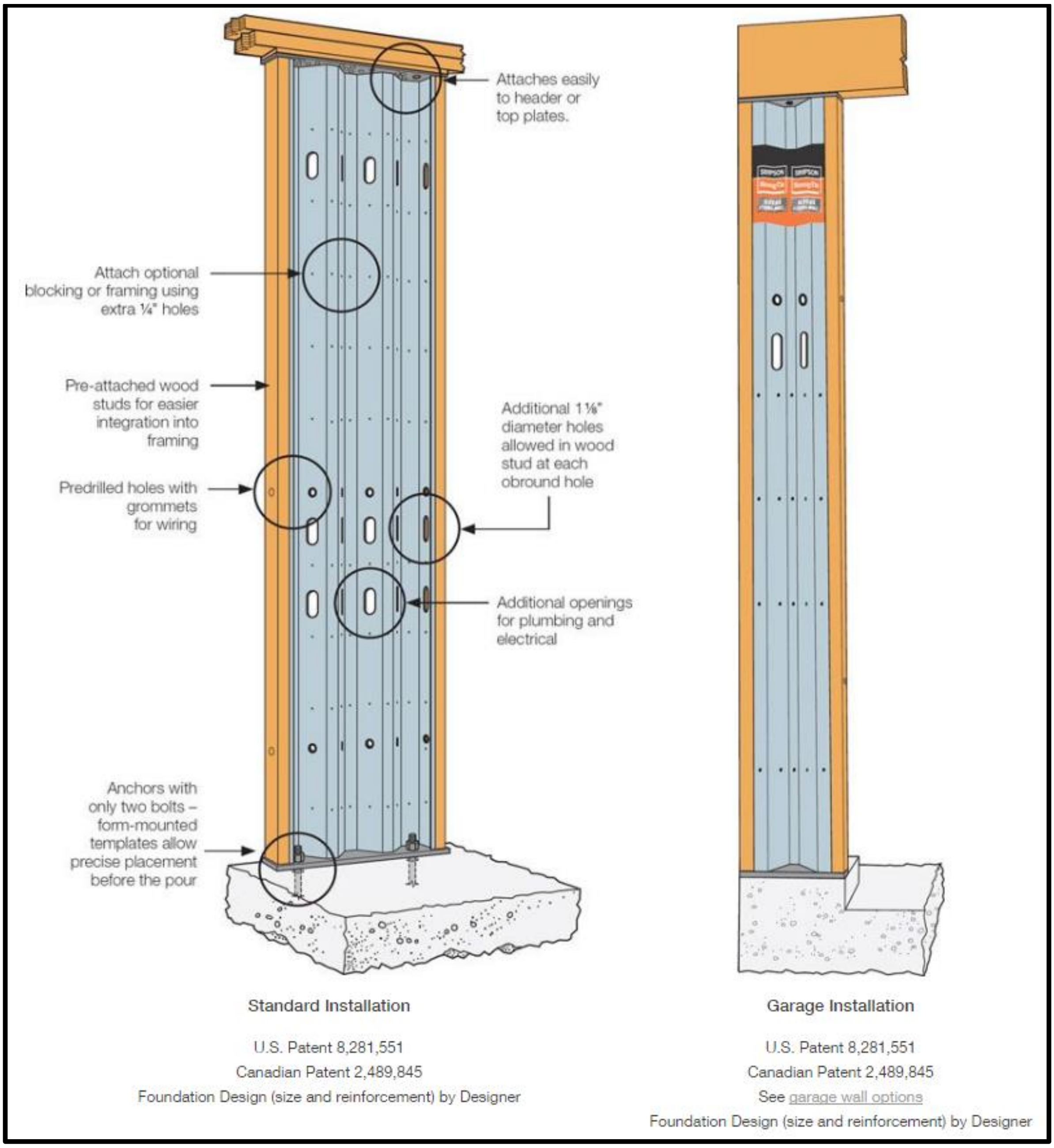

Figure 13: Prefabricated steel Strong-Wall®

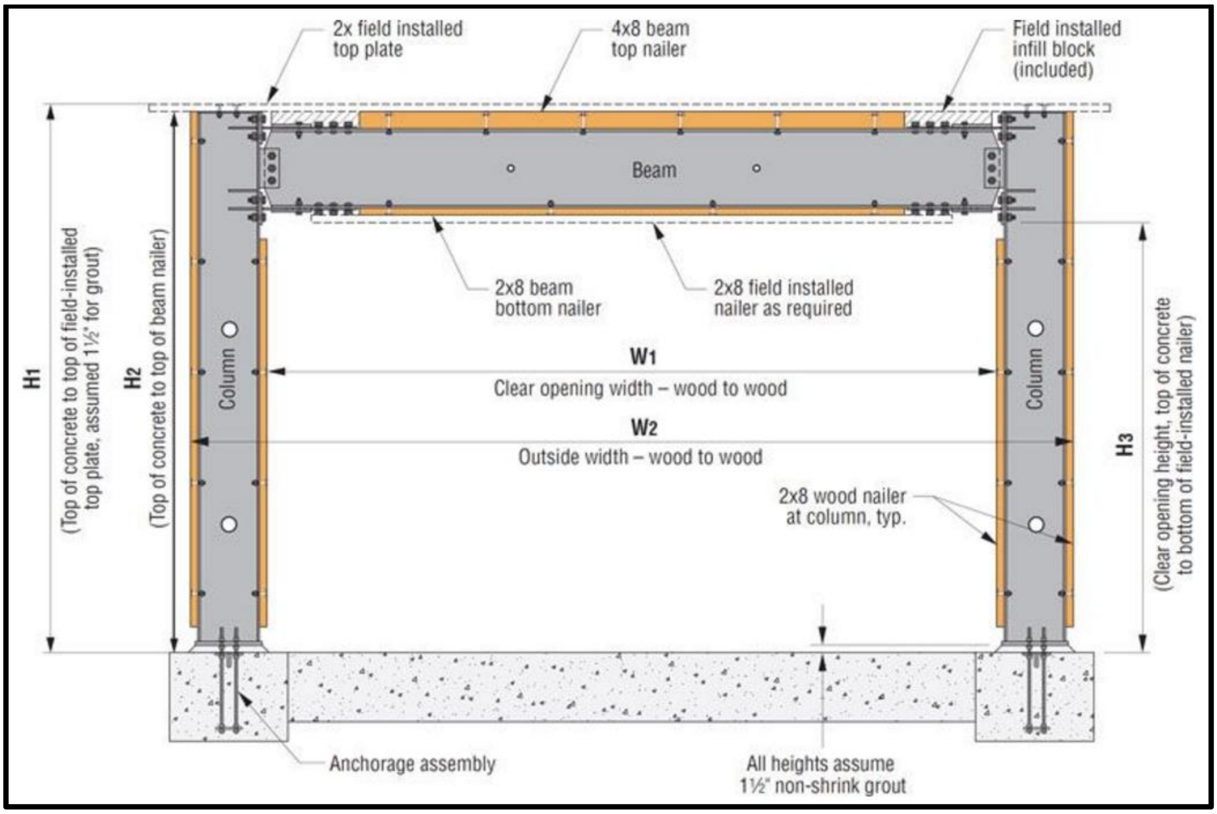

Figure 14: Patented Strong-Frame® ductile steel special moment frame (SMF) for integration with timber frame construction

In the U.S. code there are four different ductility classes of structural steel moment frames. Only the most ductile, which is classified as a "special moment frame" (SMF) is ductile enough to use in wood light-frame structures without penalizing the building design as a whole (mixing in a system that is less ductile than light-frame requires using higher design forces in the building). In other SMF's, ductility under lateral loading is achieve through yielding of the steel beam's entire cross section adjacent to the column flange. This yielding promotes instability in the beam, and consequently stringent out-of-plane bracing is required to control lateral-torsional beam buckling. It is very difficult to properly anchor this bracing to a surrounding wood structure to achieve the required strength, and importantly, stiffness, necessary for the braces to function adequately. The Strong-Frame uses patented Yield-Link® technology to move ductile yielding out of the beam and into bolt-on / bolt-off replaceable elements of the beam-to-column connection (see Figure 15). This maintains a stable beam and allows the beam to be designed without worrying about lateral-torsional buckling bracing, greatly simplifying integration into timber structures. This SMF framing technology is being widely used in multi-story light-frame wood construction where large openings are needed on the ground floor to accommodate commercial space or parking, or where retrofits of existing buildings with inadequate strength are needed, such as in San Francisco. It is now included as Chapter 12 in ANSI/AISC 358-16, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*.

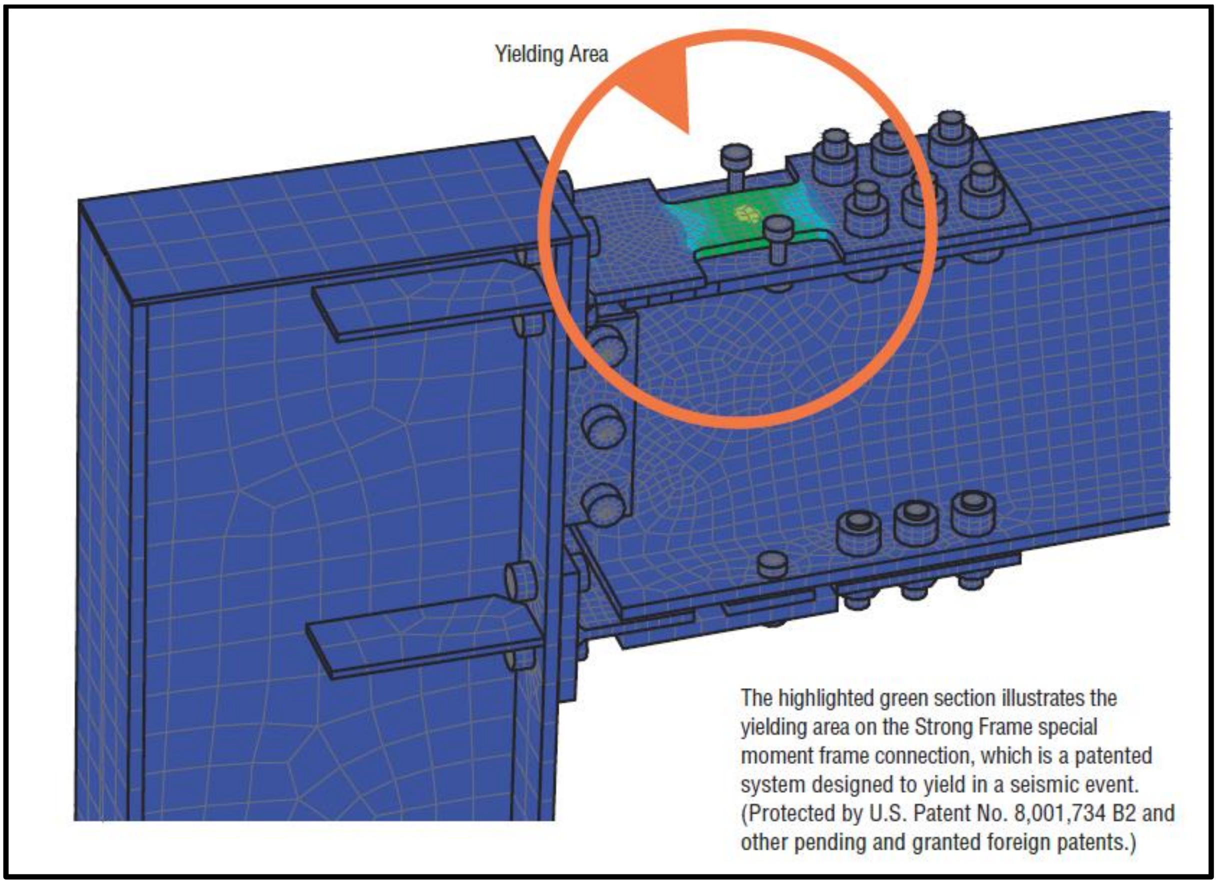

Figure 15: Proprietary Yield-Link connection technology in the Simpson Strong-Tie steel special moment frame

5. Connectors for CLT construction in seismic areas

Performance standards for connectors for use in seismically-resistant CLT construction are just now starting to be developed in Europe and in North America. As mentioned previously, it is not enough to simply place a ductile connection into a wood building and expect good seismic performance, and this is also true for mass timber buildings. Before the required connection performance can be given, design and detailing procedures need to be proposed and evaluated to ensure that the proposed method will achieve the desired performance in the building. The proposed design and detailing requirements would include things like limits on panel aspect ratio (if there are to be any), minimum connector performance and of course specification of the critical seismic design coefficients. This process is happening right now in the U.S. according to the procedures outlined in FEMA P695.

FEMA P695 is an iterative process of determining the seismic design coefficients and design and detailing requirements for new lateral force resisting systems. To use the process, design and detailing rules for the new system are proposed. These include specifying the values of R, Ω and C_d to use in the design process. After a considerable amount of testing on connections and components of the system has been completed, advanced nonlinear structural models of the entire building are created and the proposed system response is evaluated against a suite of ground motions from actual earthquakes. And not just for one building archetype, but for a suite of building archetypes covering the design space for which approval is being sought. After the analysis results are obtained, changes, if needed, are made to R, Ω, C_d, and the design and detailing rules, the buildings are then redesigned, reanalysed, and the new results studied. Once the probability of collapse is acceptably low and meets the requirements of FEMA P695 for all building archetypes the results are deemed successful. A very important part of the FEMA P695 process is the establishment of a peer review panel that acts as an unbiased voice in assessing the decisions of the team evaluating the new LFRS. This is because there are parts of the process that require judgment, and it is crucial to have unbiased oversight in these areas.

The connection scheme being proposed for seismic construction with CLT in North America is similar to what has been used in Europe. Angle brackets at the base of the wall transfer shear forces while providing a degree of overturning restraint (see Figure 16). As overturning forces become larger, discrete holdown connectors and continuous rod systems such as those shown above are also being considered. Individual panel aspect ratios are being considered at 1:1 maximum. This will allow the connector deformation to translate into yielding interstory drift, which helps to lessen the internal forces in the building.

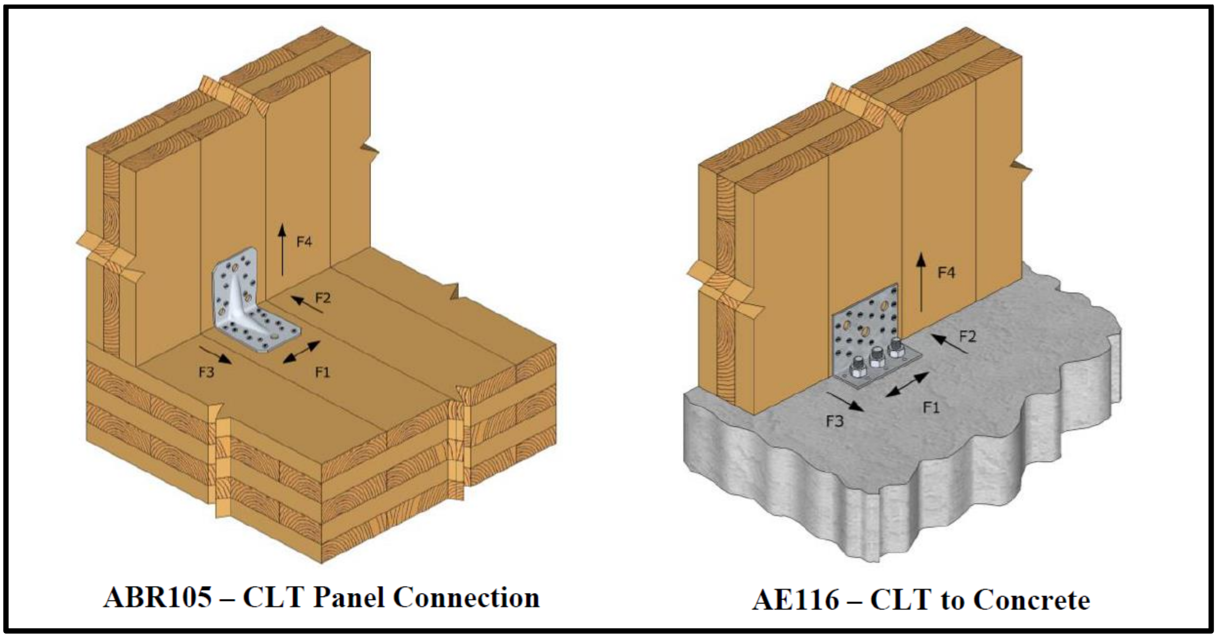

Figure 16: Base angle connections at CLT wall panels to floor panels and foundation

6. Conclusion

Having a complete load path is important in any structure, but buildings designed for earthquakes need additional connections that keep the building tied together, and tied to the lateral force resisting system, to ensure that there is neither partial nor full building collapse in large earthquakes. In wood structures, these connections are easily made with commercially available hardware. Architectural and structural demands may require the use of something other than wood, such as steel panels or frames, for the vertical elements of the lateral force resisting system in the lower stories of multi-story construction and in other areas with large openings and short wall segments.

The basic philosophy of seismic design in both North America and Europe is very similar. Because of the complex nonlinear nature of the building response, establishing design demand (via the q or R factor) directly impacts how design resistance is established (at the peak capacity of the component or with some overstrength left in). Successful seismic design typically means providing as much economy as possible in the building design while at the same time ensuring that there is a very low probability of collapse, even in very rare, large earthquakes. Properly designed and detailed, wood structures have a great track record of fulfilling both of these requirements.

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