Codes and Standards

updates and discussions related to codes and standards

In November of 2009, Coming Up with Tie-Downs, Part 1, appeared in STRUCTURE® magazine. It detailed the history and current state of design for the continuous rod tie-down system industry where these systems are used for wind uplift restraint in light-frame wood construction. That article explained that, as detailed at the time, the wood top plate's bending capacity often controls the design of these systems, though this design constraint is often overlooked. It also pointed out other limitations, such as top plate deflection, top plate rotation, rod elongation, bearing plate capacity and wood shrinkage that must be considered in proper system design. In addition, the article described the lack of design guidance for these limitations – leaving designers, contractors, and manufacturers to rely solely on their own judgment for wind uplift tie-down system design.

Coming Up with Tie-Downs Part II

ICC-ES Passes AC391 Providing Industry Guidance for Continuous Rod Tie-down System Design

By Bryan Wert, M.S., P.E., SECB

Design Guidance Arrives

In June of 2010, the International Code Council Evaluation Services (ICC-ES) passed Acceptance Criteria 391 (AC391). This document provides the industry the desired guidance for the design of continuous rod tie-down systems resisting wind uplift in light-frame wood construction. As with any other Acceptance Criteria, AC391 was developed by ICC-ES technical staff in consultation with multiple manufacturers and with input from other interested parties at open public meetings, and was approved by an Evaluation Committee made up entirely of Building Officials.

For manufacturers, AC391 established guidelines for running proper calculations and/or tests in order to receive a product Evaluation Report for either:

- The steel components comprising continuous rod tie-down *runs* (CRTR) only.
- The entire continuous rod tie-down *system* (CRTS), which includes CRTR and the light-frame wood structure used to resist wind uplift.

What does this mean for engineers and architects designing projects that want to use rod tie-down systems for wind uplift restraint? Until manufacturers actually have Evaluation Reports from ICC-ES, project designers can use the guidelines set forth in AC391 when evaluating the

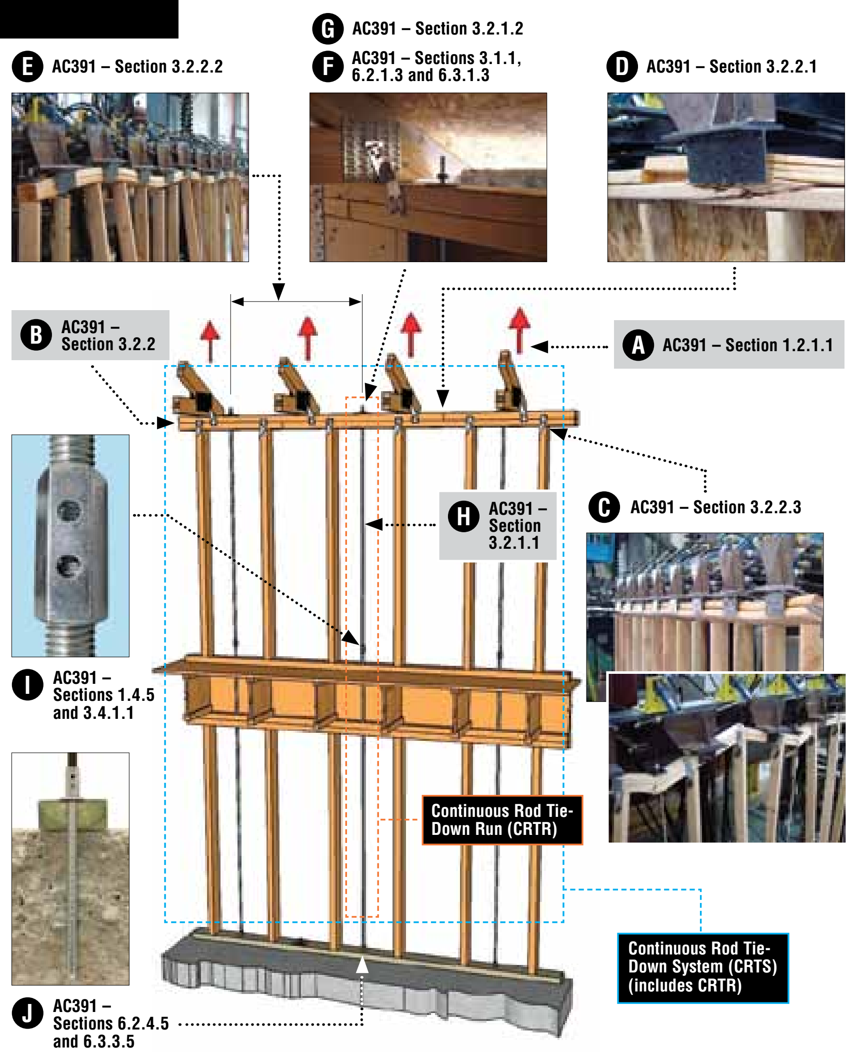

Figure 1: Illustrated Design and Detailing Requirements of AC391.

*Bryan Wert, M.S., P.E., SECB is a branch engineer for Simpson Strong-Tie in McKinney, Texas. He can be reached at **bwert@strongtie.com**.*

use of these systems in their structures. These guidelines, which are highlighted in *Figure 1* and described in *Table 1*, basically create a checklist of design requirements for this type of uplift restraint system.

Once manufacturers have obtained Evaluation Reports for their products, project designers will have some new tools to reduce their workload. However, the type of report obtained by the manufacturer may determine how useful that tool is. A CRT<u>R</u> evaluation report will leave the bulk of the design work up to the designer; only the steel components of the run are evaluated in the report, which means the designer must still analyze the wood structure and its ability to transfer uplift forces to the rod tie-down runs. Alternatively, a CRT<u>S</u> report should minimize the designer's workload responsibilities when creating a continuous load path for wind uplift restraint using rod tie-downs; all pertinent elements of the wind uplift system – steel components and wood structural members – are evaluated and included in user-friendly tables to lay out and detail the system.

Wood Structural System Limitations

Full-scale testing has proven that there are multiple limit states that could control the design of rod tie-down systems, and some of these limit states will typically govern system design over others. For instance, checking steel component capacities is irrelevant if the rod tie-downs are spaced too far apart for the top plates to adequately transfer the load to them. The capacities of the steel tie-down components (rods, bearing plates, coupler nuts, etc.) are unquestionably important, but wood component limitations and system effects will likely take precedence in design.

A traditional connector uplift restraint load path starts with a hurricane tie from truss/rafter to top plate and then attaches another tie from the top plate to the stud below on the same side of the wall. A small, but distinct difference when using rod tie-downs is the introduction of eccentricity when the load path from the top plate down is attached to a rod in the center of the wall, rather than to the stud on the same side of the wall.

Although small eccentricity in many structural designs may have negligible effects, the inherent eccentricity in this load path will cause the top plate to roll and thus reduce the amount of load transferred to the rod tie-downs. To combat this, Section 3.2.2.3 of AC391 requires top plate rotation to be prevented by a positive connection. What comprises "the positive connection" is not dictated by the Acceptance Criteria, but some clear choices are a top plate to stud connector or, if sheathing is on the wall, the rafter/truss to top plate connection could be installed on the sheathing side and fasteners could be added to the sheathing to help resist this rotation force.

Once top plate rotation is controlled, the next two top plate design considerations are bending and deflection. Bending capacity of a wood top plate can easily be found in the code-referenced *National Design Standard for Wood Construction®* (or NDS) published by ANSI/AF&PA. Finding the bending demand on the top plate is the moment in the top plate caused by the uplift forces divided by the section modulus. Choosing what moment equation to use for this calculation is where some common sense and experience are required. The American Institute of Timber Construction's (AITC) *Timber Construction Manual* provides diagrams and equations for moment, shear and deflection that can be used for calculating the moment demand (though most designers are likely more familiar with these same equations/diagrams in the American Institute of Steel Construction's (AISC) *Steel Construction Manual*).

A simple span moment may be too conservative for this application as top plates are usually 12 to 16 feet long and the "support" spacing is, in this case, the rod tie-down spacing, which may be 48 inches on center. Multiple designers have recommended the use of the 3 span uniform load moment equation; however, this is still the designer's choice. AC391 limits the depth of the top plate to a single member (or 1½ inches) for section modulus calculations. Why? Top plate splices. An exemption to this requirement is in Section 3.2.2.1. The manufacturer or designer may provide testing, calculations and details for splice reinforcement to utilize the same bending capacity as an

Table 1: Summary of Design and Detailing Requirements of AC391.

Connection Location	Requirement	Section
A	Use of Continuous Rod Tie-down Runs (CRTR) and Continuous Rod Tie-down Systems (CRTS) evaluated under AC391 is limited to resisting roof wind uplift in wood light-framed construction. Specifically excluded from AC391 is the use of rod tie-down runs to resist shear wall overturning forces or use in cold-formed steel framing.	1.2
B	CRTS allowable loads shall be evaluated and be limited by • Tie-down run steel component capacities per 3.1.1, or • Wood deflection limitations per 3.2.2.2, or • Flexural (bending) stress per 3.2.2.1, or • Shear stress perpendicular to grain per 3.2.2.4, or • Combined axial (chord/drag force) and flexural (bending) stresses per 3.2.2.5	3.1.1 and 3.2.2
C	Top-plate torsion (rotation) must be prevented due to offsets between the point of load application *(e.g. hurricane ties at the sides of the top plates)* and load resistance *(e.g. rods at the center of the top plate)*. This can be accomplished by providing a positive connection from the top plate to stud on the same side of the wall as the roof framing to wall connection.	3.2.2.3
D	Approved top plate splice details must be provided for the CRTS to utilize both top plates in bending, otherwise only the capacity of a single top plate may be used.	3.2.2.1
E	The deflection of the top plates in bending occurring between CRTR is limited to L/240, where L is the length of the top plates between tie-down runs. Additionally, the sum of the rod elongation and the deflection of the top plates between tie-down runs shall not exceed 0.25 inches at the applied (ASD) load.	3.2.2.2
F	The effects of wood shrinkage on the overall deflection of the CRTS shall be analyzed by a registered design professional, and a method of addressing wood shrinkage in the system shall be provided. If shrinkage compensating devices are used, they shall meet AC316 requirements.	3.1.1, 6.2.1.3, and 6.3.1.3
G	Steel bearing plates shall be sized for proper length, width and thickness based on steel cantilever bending action and wood bearing. Deflection from bearing compression (up to 0.04") must be included in overall deflection calculations.	3.2.1.2 and Figure 1
H	Rod elongation is limited to 0.18 inches for total rod length at the applied (ASD) load.	3.2.1.1
I	Proof of positive connection between threaded rod and threaded rod couplers shall be provided *(e.g. sight holes or other method)*. Rod couplers must also be tested to prove they can develop at least 100% of the rod's tensile strength and 125% of the rod's yield strength.	1.4.5 and 3.4.1.1
J	Design of the anchorage is the responsibility of the design professional and must be performed in accordance with the applicable code.	6.2.4.5 and 6.3.3.5

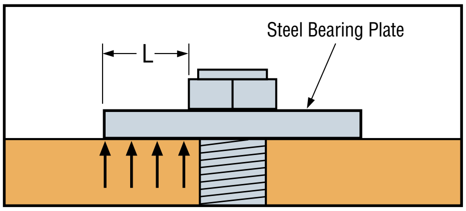

Figure 2: Steel Bearing Plate Cantilever Length.

un-spliced double top plate. If this burden of proof is met, then the section modulus can be doubled (the depth used in the section modulus equation cannot be increased to 3 inches, as this would more than double the capacity, and the top plate is not actually a homogenous 3 inch deep member).

ICC-ES technical staff regarded deflection of the top plate to be a serious design concern, requiring in Section 3.2.2.2 that the deflection limit shall be L divided by 240, where L is the distance between rod tie-downs. Further, this section requires that the sum of top plate deflection and rod elongation be less than ¼ inch. Why such a stringent requirement? It is not only to control possible drywall and finish cracking, but also to ensure the studs are not too far removed from the top plate which, if unrestrained, could lead to failure from component and cladding wind loading perpendicular to the wall. Consequently, this deflection constraint can easily control system design.

Wood structural sheathing (OSB or plywood) should be able to aid in controlling top plate deflection between rod tie-downs. However, wall sheathing usually is designed for lateral shear forces only, so adding uplift force would stress both the sheathing and sheathing fasteners in multiple directions simultaneously. In addition, full-scale testing has revealed top plate rotation and cross-grain tension failure in the top plate when hurricane ties are installed on the opposite side of the top plate from the sheathing with no additional top plate reinforcement. A portion of this testing showed that gypsum board attached to the interior wall studs and top plate per code did not add any significant rotation resistance, as the fasteners quickly ripped from the edge of the panels. Though restraining top plate rotation and analyzing sheathing to resist shear and uplift does complicate the continuous load path, proper detailing of these connections could significantly stiffen the top plate, making wider rod tie-down spacing possible.

AC391 Section 3.2.2.5 requires the designer to consider wood member combined axial and flexural stresses, respectively. This design consideration exposes how lateral design can be affected by uplift design when using tie-down systems. The wood member under scrutiny is the top plate. The wood top plate only has limited capacity to handle multiple forces simultaneously. If part of that capacity must resist and transfer uplift load through flexure, less capacity remains available to resist axial forces.

Axial forces are typically what top plates are designed to transfer. Top plates usually act as drag struts, accumulating the shear forces axially until distributing these forces at each shear wall. Moreover, top plates also act as chords for floor and roof diaphragms, axially withstanding both compression and tension chord forces as the diaphragm flexes under lateral load. The splice transfer detail specified by designers for each project is typically governed by one of these axial force demands. Therefore, if the top plate has less capacity available for axial force resistance, this could cause the designer to place shear walls closer along the same line to reduce drag strut forces or require more shear wall lines to break up diaphragm forces.

It Shrinks Too?

AC391 Section 3.1.1 requires that the effects of wood shrinkage on the overall deflection of the system be considered. Unlike deflection from rod elongation, bearing plate crushing or top plate bending, deflection from wood shrinkage is not a result of the system being engaged due to a wind storm. In fact, it's deflection that can be set into the system starting the day the wood structure is built – long before any storm.

A rod tie-down is a stiff element in a wood-built structure, so when moisture escapes from the wood and the wood elements shrink, a gap will occur at the point of restraint – usually between the nut and the bearing plate on top of the wood top plate. That is, unless a shrinkage compensating device is installed at each tie-down run. Without this type of device, the components of the structure that have shrunk will need to deflect under uplift load in order to engage the rod tie-down restraints. Whether this deflection is minor or severe will depend on the number of floors in the structure, moisture content of wood components and the type of wood components comprising the structure. Designers should be aware that using engineered wood products for floor joists and rim boards, and kiln dried lumber for top and sill plates, can help minimize the effects of shrinkage. (A free shrinkage calculator is available at **www.strongtie.com/shrinkcalc**.)

Steel Component Considerations

The strength of steel in the rod tie-downs is important, but system deflection caused by rod elongation may actually be what controls rod diameter and/or spacing. AC391 Section 3.2.1.1 limits rod elongation to 0.18 inches. The elongation or stretch of a steel rod is calculated with a simple equation, $\delta = PL/AE$, dependant on the tensile force (P), rod length (L), effective cross-sectional area (A) and modulus of elasticity (E), which is 29,000,000psi for all structural steel. The rod length (L) is not going to change – this is fixed by the building height. So from the variables that are left in this equation, it is easy to conclude that in order to control elongation the design must reduce the tensile (uplift) force (P) in each rod by spacing them closer, or increase the rod diameter to increase the cross-sectional area (A).

Another small, but important consideration in the overall deflection of the tie-down is to determine the effect of the bearing plate crushing into the top plate. Section 3.2.1.2 of AC391 draws from Section 4.2.6 of the NDS, and requires the designer to assume up to a maximum deflection of 0.04 inches from wood bearing compression based on load demand versus capacity of the plate. This 0.04 inches of deflection is definitely not a large number in general, but within the limit of 0.25 inches for the system deflection. It could be up to 16% of that total, and thus important to consider.

Bearing plate dimensions also are important. Clearly, the bearing area of the plate must be large enough so that the perpendicular to grain capacity of the lumber is strong enough to transfer the uplift force out of the top plate and into the rod. In addition, the steel bearing plate thickness must be considered in the design of the plate. AC391 Section 3.2.1.2 states that the steel bending capacity value shall be derived from cantilever bending action of the steel plate. The thicker the plate, the higher the bending capacity. *Figure 2* depicts this design constraint.

Another simple, yet extremely critical requirement of AC391 is proof of a positive connection in rod couplers. The easiest way to achieve this is positive stops, so the installer can verify the rod is threaded sufficiently into the coupler, and having witness holes makes for quick inspection. Without this requirement, the force in the rod above could not be transferred to the rod below. In addition, AC391 stipulates that rod couplers must be tested to exceed both 100% of the rod tensile strength and 125% of the rod yield strength – basically ensuring the coupler will not be responsible if the tie-down system fails.

Cables vs. Rods

Currently, AC391 does not address cables. In fact, it specifically excludes them. However, the specific requirements in the tie-down *system* section of the currently approved version of AC391 focuses on load path requirements and limitations – mainly dealing with top plate load distribution and deflection concerns. Thus cables could replace rods as the tie-downs in these systems.

Undoubtedly, additional constraints would need to be added for cables due to the differences in physical properties between cables and rods. Some of these properties, such as cable pre-stressing and subsequent relaxation, are already delineated in ICC-ES AC369 and could easily be referenced in AC391. Whether tie-downs are flexible cables or stiff rods, the AC391 guidelines for wood top plate bending, deflection and rotation must be addressed when designing these wind uplift restraint systems.

When Evaluation Reports are Available – What Then?

Evaluation reports that meet all the testing and calculation requirements for a CRTS will provide a comprehensive description of the system, including CRTR spacing, framing member requirements, and allowable uniform uplift load that can easily be reviewed and approved by engineers, architects, contractors and building officials. The designer then only needs to follow the tables provided by the manufacturer's evaluation report to design the wind uplift restraint system.

Evaluation reports that meet only the testing and calculation requirements for the steel components comprising the CRTR leave the majority of the design work for the project designer, including all the wood component stresses, deflections and shrinkage. In this type of report, ICC-ES has not evaluated anything but the steel CRTR component information.

Conclusions

Continuous rod tie-down systems give designers of light-frame wood structures an additional option to create the code required continuous load path to resist wind uplift. However, new guidelines must be learned and followed for proper design and installation. AC391 is that set of guidelines created by a consensus of engineers, manufacturers, building officials and other industry experts. The Acceptance Criteria defines the strengths and the limitations of these systems, exposing the unique rod tie-down system detailing that's required to design a safe structure capable of protecting lives in high-wind events. There is still plenty to be learned about rod tie-down systems, much of which will come to light as manufacturers obtain evaluation reports from ICC-ES. More testing will further our understanding of these systems – initiated in labs by manufacturers and researchers, and in the real-world by Mother Nature.▪