Tech Notes SBCA

Lumber Shear-out at High Tension Truss Plate Connections: (TPI 1 Section 8.9 Net Section Lumber Check)

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Issue:

With the adoption of ANSI/TPI 1-2002 (TPI 1), an additional design check was added in the joint design process, and can be found in Section 8.9.2. The TPI 1 Project Committee (PC) intended for this provision to address a possible design mode of failure where high tension forces create a shear failure parallel to the grain of the wood directly beneath the connector plate teeth. The effect of this provision is that the number of chord and or web members has had to be increased (e.g., single webs to double webs, double chords to triple chords, etc.) in truss designs that otherwise have historically good performance records. This prompted questions about the basis of the provision.

Key Definitions:

Narrow Face: The nominal 2-inch face of a 2x_ member.
Lumber Shear-out: A failure that begins at the interface between the truss plate and the wood. Other appropriate technical terms include chunk-out, shear-plug, and tear-out.

Background:

With the adoption of the 2002 edition of TPI 1, a new design check was added to the metal plate connector joint design provisions, Section 8.9.2, which states in part *(see Appendix A for complete text)*:

> ANSI/TPI 1-2002 Section 8.9.2 …For wood thickness greater than two inches with plates embedded only on the surface normal to the thickness, the tension, T, introduced by a single joint into a wood member, shall not exceed 1600 pounds per inch of wood width,...

Recently, after a rigorous review and discussion, the Truss Plate Institute's Technical Advisory Committee (TPI TAC) approved the following interim guideline:

> **ANSI/TPI 1-2002 Section 8.9.2 Interim Guideline.** It is recognized by the metal plate connected wood truss industry that block shear (failing of the lumber beneath the connector plate teeth), is a failure mode that needs to be checked in truss design. The current design value of 1600 lbs/inch was established based on limited information and is considered conservative. Values of up to 3100 lbs/inch have been justified by some in the industry based on engineering experience, full and small scale truss testing and engineering analysis. There are many variables that affect this issue, including the species and grade of the material, and the length of the connector plate involved in the connection. Empirical evidence, field experience and engineering judgment may be used to consider design values significantly higher than the current design value stated in the standard.
>
> Block shear for axial tension members has only been identified as a concern at the ends of a member. The provisions of section 8.9.2 do not apply to joints in the middle of a piece, such as chords that are continuous through a joint.

This interim guideline clarifies two issues found within Section 8.9.2 of TPI 1. The first issue is that this design check does not apply to the chord members that run through the joint. The second issue allows

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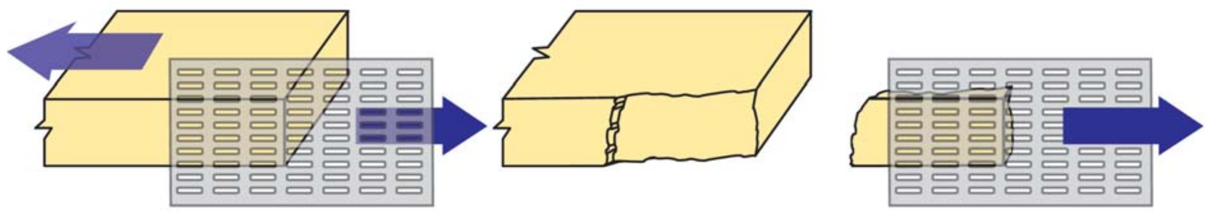
other limiting values to replace the 1600 pounds per inch of wood width when test data or engineering can justify an alternative value. Needless to say, this provision is welcomed by those in the truss design community who have experience with truss designs affected by this provision and the background to rationally adjust their design approach for this condition.

Analysis:

Chunk-out, Shear-out, Shear-plug, Tear-out?

The provision in Section 8.9.2 of TPI 1 has caused considerable confusion as to exactly what type of failure mode or problem it is intended to prevent. The appropriate technical term for this type of failure mode is shear-out, block shear or shear-plug failure. This type of failure can potentially occur when a high tensile force must be transferred from a metal connector plate to a wood member and the plate is fastened to the narrow face at the end of a member. It is this transfer of load and the resulting non-uniform stress distribution that can prompt the wood to shear underneath the plate, leaving a significant portion of wood embedded in the plate as the joint pulls apart. *Figure 1* illustrates this failure mode. Although there has been no complete test data set or field experience presented that clearly demonstrates this failure mode and its likelihood of occurrence, some members of the TPI 1 PC and TPI TAC have concluded that there are circumstances where this failure mode can occur at a load below the tensile strength of the connecting wood member, the plate tooth withdrawal and associated plate values.



Shear out failure mode
(plate shown on one side only for simplicity)

FIGURE 1

"Chunk-Out" Re-examined

The TPI 1 PC and TPI TAC believe that Section 8.9.2 presents a necessary truss design check and recent additions to the *National Design Specification® (NDS®) for Wood Construction* adds analogous design checks for bolted connections. The corresponding problem in trusses, however, is more complicated because the teeth in the plate do not penetrate the full depth of the wood member. This results in a complex stress analysis problem that involves how the stresses dissipate from the wood adjacent to the teeth at the surface to the center of the wood member. Whether these stresses will be large enough to cause failure and control the truss design depends on the shear strength of the wood as influenced by grade and species, and potentially even the ring structure of the wood and the occurrence of other high stresses in the truss. In other words, there is still much more to learn about when and where a shear-out failure mode check should control the truss design.

What Got Us Here?

Ultimately, understanding the shear-out problem requires a better understanding of the magnitude and distribution of the stresses in the vicinity of the plate and comparing this to the wood shear strength in this area. One can then determine the likelihood that these conditions reach critical levels in everyday truss design and target design provisions accordingly. Since we cannot "see" stresses or easily measure them, mechanical load tests provide only an end-result, single-outcome indication of the problem or the lack thereof. If a test produces a shear-out failure, obviously one knows that the stresses exceeded the wood shear strength for the conditions of that test. Generalizing test results to a wider variety of situations without the stress data is an engineering art due to the need for nearly exhaustive testing across the design spectrum of joint configurations, and the existing limitation was established with some application of that art, and not an absolute value in accordance with any definitive standard.

Small scale tests formed the original basis for the 1600 lbs/inch of width limitation currently in TPI 1, but these tests were few in number and variation in species/grades/sizes and therefore did not provide conclusive evidence of the conditions or stresses associated with shear-out. Given the outcome of, and conditions examined in the tests, the 1600 lbs/inch of lumber width is considered conservative by TPI TAC.

The shear-out failure mode is most often associated with top chord bearing, parallel chord trusses of considerable span that require large plates to attach the first panel 4x2 (or larger) tension web to the top and bottom chords of the truss. Some engineers have reported what they believe to be the shear-out failure mode in truss tests of this configuration but the observations have been few and not well documented in the public domain. California Truss Company, a WTCA member, conducted a series of 44 truss tests in 1986 to evaluate the general performance of these types of trusses. To date, this is the best and most extensive set of data available although at the time, shear-out was not a consideration and the tests did not specifically include webs as narrow as 4x2. WTCA sincerely appreciates the willingness of California Truss Company to share the data, because without it, clarification of the section 8.9.2 provision would have been more difficult to make. The testing was conducted on top chord bearing trusses manufactured with Douglas-fir lumber, using either double 4x2 or single 4x6 chords, with 4x4 or 4x6 webs in the first panel. These 44 trusses ranged in span from 31- to 60-ft and were tested to failure. None of the 44 trusses tested yielded a shear-out failure even though, in several cases, the tensile force in the first tension web resulted in an equivalent design value well in excess of the 1600 lbs/inch width limitation.

In order to compare the computed tensile force in the web at ultimate load with the 1600 lbs/inch width limitation, the tensile force is divided by a factor of 2.1 and the web width (i.e., the width of the web in contact with the plate). The 2.1 adjustment factor is used to adjust the test value, to a design value and accounts for duration of load, manufacture, and use. Normally, the 2.1 adjustment factor is used to adjust the fifth percentile test value, meaning the strength at which 5% of test samples will have failed. In this case, given that there were no test failures of any of the web members in any of the 44 trusses tested, we used this as a point estimate design value to make rough comparisons for evaluation of efficacy purposes. In 18 of the 44 tests, the adjusted tensile force in the first web at the time the truss failed exceeded the 1600 lbs/inch limit. In each of these 18 cases, the truss failed outside of the first panel. Had these trusses been designed with the Section 8.9.2 limitation, the required size of the first web would have been larger than what was actually used, even though the smaller webs performed more than adequately in these tests.

The highest web design force recorded for these trusses was 2285 lbs/inch of web width, or 1.42 times higher than the current limitation. In the remaining 26 tests the load levels applied to the trusses at failure produced corresponding tensile forces in the first webs that were less than the 1600 lbs/inch. All the failures in these 26 tests were tension or compression lumber failure, or plate withdrawal failures of the chords or webs in the center portion of the trusses. There were no failures at all with any of the first web members, where there were the largest plates and the highest axial tension and shear forces. These are precisely the conditions that have been suggested cause the shear-out failure mode to occur. The results of these tests provide evidence that tension webs can safely sustain forces at and beyond 1600 lbs/inch, but only in the conditions associated with these tests. Additionally, none of these tests were designed to prompt failure in the first panel. Conversely, these trusses demonstrate the anecdotal feedback that has been received suggesting that the shear-out type of failure is a rare occurrence.

Current Design Considerations

As noted above, the TPI 1 PC and TPI TAC members believe that there are conditions where the shear-out failure mode could control the design of a tension member in a truss. TPI TAC has provided the guidance needed to recognize that the current design value of 1600 lbs/inch is to be considered conservative and that by working with your truss design engineer, other design values could be used so that there is not an inappropriate limit on the truss design process when we know that the truss being designed can safely and effectively carry the applied loads. Experience, full and small scale truss testing, joint testing and engineering analysis by those involved in the design and manufacture of purlin trusses suggest that the Section 8.9.2 design limit can be justifiably adjusted up to and potentially beyond 3100 lbs/inch of width. Until additional analysis and data are available to further shape design guidelines, the key to implementation will be to work with your truss design engineer and use the experience that you have with these types of trusses to determine the Section 8.9.2 design limit for your truss designs.

One approach to guide selection of the design limit is to apply the allowable adjusted shear strength from the *NDS*® for the appropriate species and grade times the contact area of the plates on both faces of the connection to yield a computed shear-out design limit. Sometimes the wood shear values will be less than the corresponding plate withdrawal values and therefore will control. Such an approach will yield alternative design limits in the approximate range of 2000 lbs/inch of width to 3600 lbs/inch of width depending on the minimum plate length necessary to prompt the shear out failure mode. One limitation of such a calculation is that it assumes that the shear stress in the wood under the plate is uniform when in fact it is not. Nonetheless, higher values are appropriate for species with higher allowable shear strengths and vice versa

The typical truss design conditions where the Section 8.9.2 limits will come into play include:

- Short span, top chord bearing floor trusses with heavy loads.
- Floor trusses with tension webs at very shallow angles.
- Heavily loaded commercial floor trusses.
- 4x2 purlin trusses in panelized roof systems.

Finite element stress analysis and testing planned for the future offer a path to define the nominal stresses associated with the shear-out condition. Such an analysis combined with verification tests is one of the steps in defining when the shear-out failure is likely to control member design and the appropriate limits that should apply. Computing stresses is much easier than attempting to measure them in tests. But testing will also be necessary to affirm computations. Testing of discrete truss loading and configuration conditions will provide additional evidence of when the shear-out failure mode should control a given design.

Conclusion:

There are no known in-service truss failures in the public domain that have been associated with the shear-out failure mode as defined in Section 8.9.2 of TPI 1. In contrast, there have been lumber tensile, shear and tooth withdrawal failures even though there are design properties readily available for these failure modes. Until additional analysis and data are available to further shape design guidelines, the key to implementation will be to work with your truss design engineer and use the experience you have with these types of trusses to determine the shear-out design limit for your truss designs.

Appendix A

The language in RED signifies sections of the code or law that have been used in the foregoing document to make it easier for the reader to see the language in context.

ANSI/TPI 1-2002
National Design Standard for Metal Plate Connected Wood Truss Construction
Chapter 8 – Metal Connector Plate Joint Design

8.9 NET SECTION LUMBER CHECK (h')

8.9.1 At all joints, members shall have metal connector plates sized and positioned so that the allowable axial tension stress, F'_t, of any wood member, or the allowable axial compressive stress, F_c^*, of any wood member at any joint without wood-to-wood bearing in the direction of the axial force, is not exceeded on the reduced net section, h' times d^2 (see Figure 8.9-1).

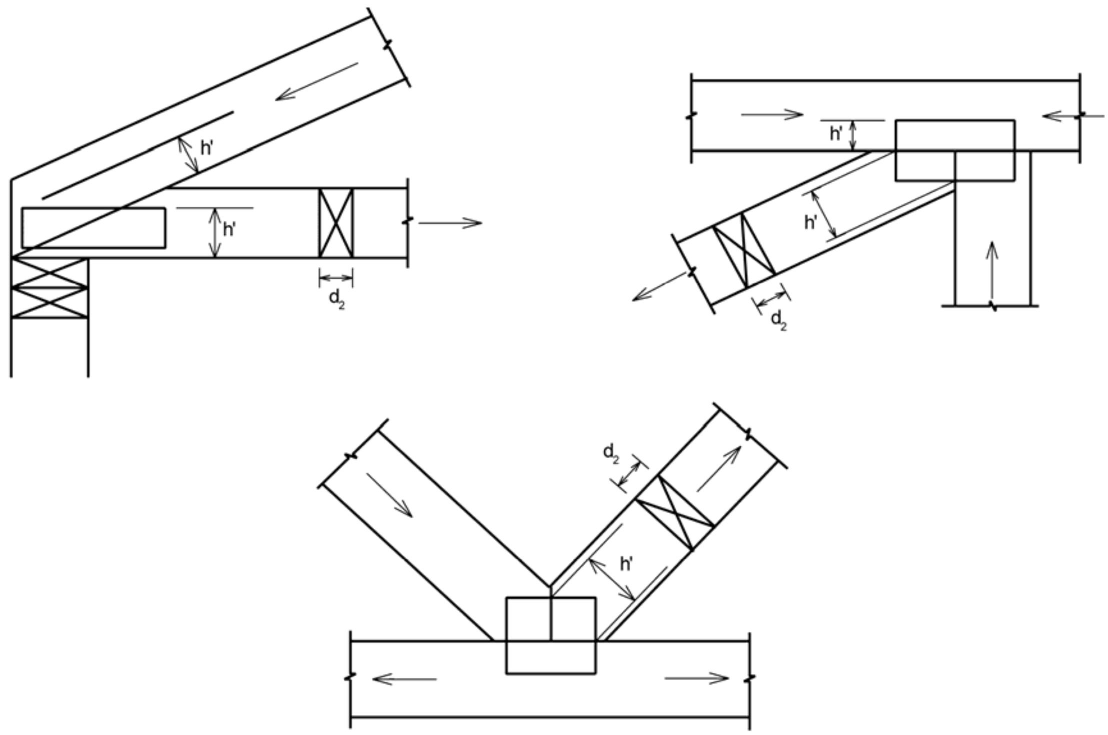


Figure 8.9-1 Reduced Net Section Checks

8.9.2 For wood thickness greater than two inches with plates embedded only on the surface normal to the thickness, the tension, T, introduced by a single joint into a wood member, shall not exceed 1600 pounds per inch of wood width, where wood thickness is the wood cross-section dimension perpendicular to the plane of the truss and wood width is the wood cross-section dimension in the plane of the truss, and this tension limit shall be adjusted per Section 6.4.

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