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Forensic Engineering Analysis & Testing of Wood Truss/Wall Connections

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Abstract

Calculations, peer review, full scale truss system tests, and correlation of information has been completed to determine if a wood truss system requires shear blocking at typical wall/truss bearing connection.

Purpose

Structural engineers in Florida have been charged with "negligent engineering" for not providing shear blocking between the wood trusses at the wall/truss bearing. This research provides a basis for calculating the need for shear transfer blocking.

Scope

Determine the capabilities of a metal plate connected wood truss system to transfer the roof diaphragm wind load into the building wall WITHOUT the use of shear blocking between the trusses over the wall. Provide theory, procedures, calculations, and load tests to confirm the ability of wood truss systems to transfer the diaphragm load without blocking between the trusses at the building wall.

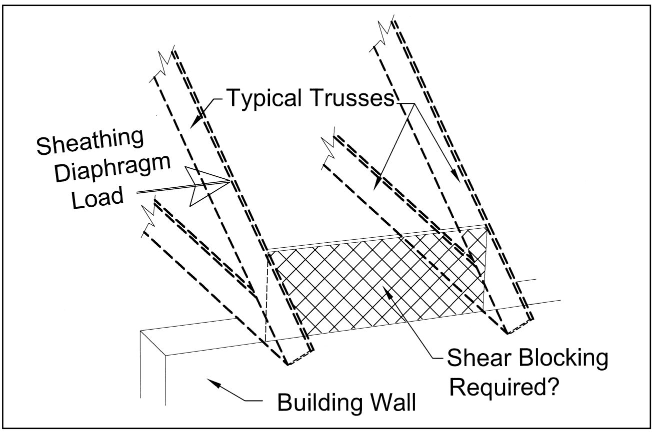

Figure 1
Typical Masonry wall with Metal Plate Connected Wood Trusses.
What conditions require Shear Blocking?

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Conclusion

THIS STRUCTURE DID NOT REQUIRE SHEAR TRANSFER BLOCKING BETWEEN THE TRUSSES AS CHARGED BY THE BOARD OF PROFESSIONAL ENGINEERS.

When an engineer has been charged with *"negligence in engineering"*, by the Board of Professional Engineers, it is the board's duty, thru their consultant, to consider all components and their resulting influences that contribute to the stability of the structure in question. It is not enough to simply use standard manufacture's catalog "safe" values in a simplified portion of the structure, or the board's reviewing consultant's method of "safe" design while ignoring other load resisting contributions. All load carrying items should be evaluated and considered in the analysis of the accused engineer's design to determine whether or not the design meets the applicable standards and building codes. The charge that wood truss/wall connections do not provide a mechanism to transfer the diaphragm load into the wall below, have been proved false by this research.

The building designed by the accused engineer of record has been used for this example:

W. T. Yaxley, PE produced standard engineering calculations and references to confirm this building did not need blocking between the truss ends over the wall to transfer the diaphragm load into the building wall below.

The calculations and appropriate references were sent to 12 structural engineers across the United States to verify the calculations were reasonable, and the procedures were credible. All 12 engineers agreed that shear blocking between the trusses were not required in this example.

Robbins Manufacturing[1] in Tampa, Florida provided two full size truss system tests to determine if the Yaxley calculations adequately predicted the performance of this system. Two tests; one for testing the system's ability to transfer the load from the sheathing diaphragm to the wall connection, and the second test verify the hurricane clip capabilities to transfer the shear load into the wall connection. The two load tests on the system proved the system would transfer more diaphragm load to the bottom of the truss than the hurricane clips, in this example, could transfer into the wall. Therefore, shear blocking between the trusses was NOT necessary in this example, as predicted by the Yaxley calculations. The limiting condition was the ability of the system anchorage to transfer the sheathing diaphragm load into the walls. The building truss roof system, and bracing configuration provided a roof that would act as a large rigid block, therefore all the shear connections were used to assess the total shear transfer available for this building. A calculation of all the shear transfer connections in this building[2] proved the total capacity of this system to transfer the loads into the walls, exceeded the code mandated diaphragm loads by at well over 400%[3].

General History

Engineers in Florida have been increasingly charged with "negligent engineering" for not providing blocking between the metal plate connected wood trusses bearing over the building wall. It has been common engineering practice in Florida for years, to NOT provide blocking between the trusses on normal houses and small commercial buildings. The limits of the size and conditions at the heel connection have not been evaluated and tested so the engineer had no verifiable procedure to follow to determine when the shear transfer blocking was required.

Several engineers have been required to defend their decision not to block between the trusses to transfer the wind load from the roof sheathing diaphragm into the wall of the building. Usually the charge has been made by the Florida Board of Professional Engineers, on advice from their expert consultants, that …'*no mechanism was available to transfer the diaphragm load into the wall since the trusses are not designed specifically for torsional resistance*'. A simple force diagram was used in some cases to show that the roof diaphragm, above the wall plate line, separated by the truss would not provide the load transfer to the wall below. The trusses '*would simply roll over like dominoes*' was a common justification. No reasonable calculations or tests were ever provided by the board's consultants to justify their method of determining the truss system was '*not capable of transferring the loads into the wall*'. When the calculations by the accused engineer in this case were provided to the board's consultant, they were rejected without a reasonable review or an analysis of the fallacy of the accused engineer's procedural steps of design.

This example was taken from a project designed by an engineer charged with '*negligent engineering*', because he did not provide the '*required*' shear transfer mechanism. The truss setup and conditions in this presentation replicate the truss system specified on the drawings the engineer had produced for a commercial project. The engineer was charged with '*negligent engineering*' for failing to provide the shear transfer blocking between the trusses over the wall.

Standard engineering calculations were performed by Yaxley to provide a basis to conclude shear blocking was not required in this example. The calculations and references were then sent to 12 structural engineers across the United States to review the procedures and reasonableness of the engineering judgements used by Yaxley. All 12 engineers agreed that shear blocking was not required in this example.

Actual load tests were desired by Yaxley to confirm the adequacy of the calculations to predict the requirement for shear blocking. Two load tests, performed by Robbins Manufacturing, Tampa, Florida confirmed the calculations by Yaxley provided adequate factors of safety for the system to transfer the loads from the sheathing diaphragm into the wall below without the installation or use of shear blocking.

An engineering textbook[4] and a article in the Structural Engineer magazine[5] have hypothesized that a shear block between the trusses must be provided to transfer the diaphragm load into the wall below. This study provides a basis for an engineer to determine, with standard engineering calculations, when the shear transfer blocking is necessary.

Information Furnished

1. Complaint by the Florida Board of Professional Engineers against the engineer of Record. "An assembly allowing for the proper transfer of the roof diaphragm forces into the masonry shear walls should have been provided."
2. Plans, calculations and information from the accused engineer's records on this project.
3. Meeting with the accused engineer to review the details of the his original calculations and plans for this project.
4. The accused engineer submitted the Yaxley calculations in response to the charge by the board. The submittal was not persuasive to the board's expert consultant.
5. Partial transcript of the board's meeting, after response by the accused engineer, confirmed their continuing concern, '*a lack of shear transfer mechanism for this project*'.

Authoritative References

1. 1997 Standard Building Code.
2. American Institute of Timber Construction, Fourth Edition, 1994.
3. Engineered Wood Association, Plywood Design Specifications.
4. Simpson Strong Tie catalog for specifications for hurricane clips H-3 & MGT anchors.
5. 12 Engineering colleagues[6] peer review for the calculations and methods used for analysis.
6. Full scale testing, data collection, and interpretation by Robbins Manufacturing[7], Tampa, Florida.
7. Consultation with several structural engineering colleagues[8] reviewing and confirming the test correlation.

Summary of Analysis

The calculations, reviews, and load tests all confirm the shear blocks between the trusses were NOT required for this project.

The engineer of record's, analysis of the most critical portion of this building required the transfer of 5,700 pounds between the roof sheathing and the wall below at a particular point in the building. The wood truss system was a hip roof, loaded parallel to the ridge. The system provided 26 truss to wall plate connections and four large MGT[9] connectors on the girder trusses to wall plate within a portion of the building. The Yaxley calculations addressed this portion of the building only. After the rejection by the board through their expert consultant further calculations, peer reviews, and tests were designed and undertaken to determine if the original calculations by Yaxley were credible.

> *Calculations* indicated a total load resistance to be 19,500 pounds from the truss end connections[10], and 18,000 pounds available from the 4 MGT connectors[11], for a factor of safety = 6.6[12]. The analysis was conservative in that it did not consider the fact that the roof framing was a hip roof and would provide further resistance to the lateral deflection of the truss system. The effect of the over framing in the middle of the building, which adds to the stiffness of the connection at the wall plates, was not included in the analysis or testing and is another conservative factor in the analysis.

> *Standard manufacturer's* hurricane clips, plus the heavy girder connections provided allowable resistance to meet the wind load. H-3 clips (125# x 2 x 26 = 6,500#); 4-MGT connectors (3965# x 4 =15,860#) for a safety factor of 3.9 based on the manufacturer's allowable resistance[13].

> Therefore, the only question remaining was the question of shear transfer[14] between the plywood sheathing diaphragm to the wall below without shear blocking. That question has been answered by standard engineering calculations, peer review, and load test confirmation. SHEAR BLOCKS BETWEEN THE TRUSSES AT THE WALL WERE NOT REQUIRED.

The full size truss tests confirmed the calculations were safe and provided an adequate safety factor against failure. It is not uncommon for an engineer to over design and ignore some of the items that contribute to the stability of a structure for ease of design complexity; but when evaluating the *'negligence'* of an accused engineer, all relevant contributions to the stability must be considered to fairly assess the capability of the system to resist code mandated loads.

This building was safe and met the applicable Standard Building Code requirements without blocking between the truss ends. The calculations and conclusions were confirmed with full scale testing of the truss system at the Robbins Manufacturing in Tampa, Florida. The factor of safety of the test over the calculations was in agreement with the Simpson allowable loads and provide a reasonable factor of safety.

Analysis of Information

1. This building, a T configuration, consisted of masonry walls with a lintel block, reinforced with 1 #5 horizontal, and concrete filled cells with 1 #5 each cell tied to the foundation and tie beam. (No question was raised about the wall's ability to transfer the load to the foundation)
2. The documents specified a 2" x 8" Southern Pine plate bolted to the lintel with 1/2" anchor bolts at 24" o.c.
3. The hip roof was framed with metal plate connected wood trusses. The roof had a 6:12 pitch and 12" overhang. Each truss was specified to be attached with 2 Simpson H-3 clips. Each H-3 clip to be nailed with 4 - 8d common nails into the plate and 4 - 8d common nails into the truss, the clips were specified to be on opposite sides of the truss and wall plate.
4. The portion of the roof in question also included: three sets of double girder trusses with 4 large MGT anchors that transfer force to the wall below. Additionally 2 girder trusses framed into the girder truss below the T portion of the building.
5. Roof sheathing was specified to be 5/8" C-C plywood fastened with 10d common nails spaced 6" o.c. at the supported edges and 12" o.c. in the field of each sheet.
6. The sub-fascia[15] was specified to be a 2" x 6" nailed with 2 - 16d common nails into each truss end.
7. The truss material and the wall plate was specified to be #2 or better Southern Yellow Pine.
8. 2x6 truss blocking was specified between the trusses at the ridge.
9. 1/2 inch drywall ceiling was specified to be attached to the bottom chord of the trusses.
10. 2x4 bottom chord braces at 10 feet on center were required, with diagonal bracing at 15 foot intervals.
11. Seven 2x4 web member lateral braces were required, with diagonal bracing at 15 foot intervals.
12. Permanent bracing within the truss system was specified, 3 rows each truss direction, with diagonal bracing at 15 foot intervals.
13. 1-Steel Wide Flange beam, perpendicular, below the main roof section with connections at each truss intersection.

14. 2- 6x6x1/2 steel columns under the girder truss at the 'bottom of the T portion' with a significant connection.

Theory of Analysis

A critical design load to be transferred from a portion of the roof diaphragm to the wall below was agreed to be 5,700 pounds from the wind load analysis performed by the accused engineer of record. The main items utilized for contributing to the load transfer were: 4 MGT anchors at the girder truss ends, 26 truss ends with plywood sheathing in bending, fascia at the truss ends, the moment couples created by the H-3 hurricane clips at the truss to plate connection. NOT considered in the tests were the ridge blocking, full roof sheathing, ceiling, web, bottom chord and permanent bracing. No contribution by adjacent framing was considered in the calculations or testing.

The following items were calculated to provide a basis to conclude that NO shear transfer blocking was necessary for horizontal shear of 5,700#:

(Factor of Safety=3.9)

4 - MGT Girder Anchors provide (3965# x 4)	15,860#
26 sets of two H-3 anchors[16] provide (125 x 2 x 26).........................	6,500#
AMOUNT OF SHEAR RESISTANCE BETWEEN TRUSSES AND WALL ...	*22,360#*

The following items were calculated to provide a basis to conclude that NO shear transfer blocking was necessary for transferring the diaphragm (roof sheathing) load to the wall below. Only the 26 truss ends were considered in this overturning resistance per truss end:

Plywood resistance to bending, Upper Portion.....................	541 inch pounds
Plywood resistance to plywood, Lower Portion	4067 inch pounds
Fascia resistance to rotation	656 inch pounds
Resistance of H-3 hurricane clips to wall............................	988 inch pounds
TOTAL MOMENT RESISTANCE PER TRUSS END	*6,252 inch pounds*

Therefore the equivalent horizontal force above the plate equals 6252 in. lbs. ÷ 14.37in. = 435 lbs each truss end[17] (Two H-3 clips). *(Two H-3 clips are rated by Simpson for 250 lbs., perpendicular to the truss).*

- 26 truss ends connected with 2 H-3 clips each would provide 6,500 # resistance (125lbs. x 2 ea. truss x 26 truss connections), Simpson catalog allowable of 125# per clip.
- Test results would allow 9,620 lbs by the H-3 clips before failure. However the deflection criteria of 1/8" movement yielded (3700# ÷ 20 = 185# per H-3), the same limiting criteria used by Simpson.

- 4 MGT anchors would provide an additional 15,860 # (3985lbs each x 4 MGT) of shear load[18] transfer.
- This does NOT consider the effect from the hip framing and the spreading of the load into the intersecting roof at the 'T' portion of the building.

Analysis of Tests

Five trusses were fabricated in accordance with the project shop drawings. 4'x8'x5/8" CDX plywood was nailed to each eave condition in accordance with the project drawings. The remaining sheathing was not installed, the ridge blocking was not installed, no ceiling was installed, and no permanent bracing was installed. Two Simpson H-3 clips were installed at each truss/plate intersection. The H-3 clips were placed on opposite sides of the wall plate and truss. Each H-3 clip was installed with 8-8d nails, 4 into the wall plate, 4 into the truss. The plywood was attached to the trusses with 10d nails spaced at 6 inches on center at the supported edges, and 12 inches on center in the field of the sheathing.

The test truss system was installed vertical to allow loading parallel to the roof sheathing by 6 load cells placed on the floor. The information from the loading and deflection was recorded with a data recorder. The load cells were connected with a manifold to equalize the loading.

First Test:

The first test documented the capability of the system to transfer the load to the wall plate. It was a hypothesis that the H-3 clips would fail first; therefore a block was installed at each truss wall connection to prevent movement more than 1/4" along the wall plate. This allowed documenting the total load transfer capability of the system.

The truss system transferred a peak of 9288[19] pounds of force before starting to exhibit nail withdrawal. The nails in the H-3 clips on the tension right side exhibited significant nail withdrawal while the nails on the left tension side did not exhibit the nail withdrawal. The system did NOT fail catastrophically but continued to hold the load.

Second Test:

The second test allowed the H-3 clips to the plate to fail. The system was re-nailed slightly and new H-3 clips installed to allow testing the ability of the H-3 clips to transfer the load to the wall. 3 deflection monitors were connected to the data recorder with two supplemental deflection gages that were read during the loading. The loading was temporally stopped to record the supplemental gages. This was noted by the load/time curves being dis-continuous by the time delay but no drop in load carrying capability.

The right side exhibited greater deflection at the truss/wall connection. A deflection of 1/8" was reached at approximately 3700[20] pounds; deflection of 1/4" was reached at approximately 5200 pounds. The deflections at the left side were significantly lower.

Close inspection of the H-3 clips at maximum load revealed compression collapse on the right side, while collapse on the compression side was noted only in the three upper H-3 clips on the left side. However, the upper two tension clips on the left side failed entirely by tension rupture; meanwhile two tension clips on the right side were ruptured approximately 1/4 of the cross section. The failure in the clips began in the most distant clips from the load application.

Close inspection, post loading, reveled significant nail withdrawal from the truss end, at the fascia/truss connection. Moderate nail withdrawal was noted in the connection between the sheathing and the top chord of the truss. Plywood sheathing exhibited only slight deflection in bending during the tests, most of the plywood failure was due to nail withdrawal.

Test 1 & 2 Results:

The test results show the above described wood roof system can safely transfer a significant amount of shear between the roof sheathing and the wall plate below without the addition of shear blocking. Even, deflected and twisted, the trusses in the structure[21] did not fail in the often described *'domino effect'*, but continued to transfer load in excess of 8000 pounds (10 truss ends). The shear transfer between a single truss end and the wall plate was approximately 800 pounds[22] (2 Simpson H-3 clips were rated at 250 lbs), providing a safety factor of 3.2[23] over the simple capacity of the 2-H-3 clips at each connection.

Acknowledgment

Grateful acknowledgment is hereby made to the peer reviewers for their expertise, time, and comments on this presentation. The following structural engineers participated in this review process:

1. Robert Plowfield, PE, Winter Park, Florida.
2. Tony Pedonesi, PE, Brooksville, Florida.
3. Tony Huff, PE, Owensboro, Kentucky.
4. Richard Rice, PE, Morrow, Georgia.
5. John Lawrence, PE, Fayetteville, Arkansas.
6. Chris Banbury, PE, Brooksville, Florida.
7. Benjamin Carr, PELS, Calera, Alabama.
8. Edmund H. Bergeron, PE, Conway, New Hampshire.
9. Robert J. Lewis, PE, Pittsburgh, Pennsylvania.
10. William Bracken, PE, Tampa, Florida.
11. Richard Zaloum, PE, Bronx, New York.
12. Nick Nicholson, PE, Brooksville, Florida.

A special thanks to Robbins Manufacturing, Tampa, Florida; Tom Albini, PE, VP of Manufacturing; Phil O'Regan, PE, Director of Research; and George Petrov, Research and Development Engineer, for the construction, testing, and interpretation of the truss system results.

Reference

1. The tests were designed, built and tested with consultation from Phil O'Regan, PE and George Petrov, R&D Engineer with Robbins Manufacturing.
2. The very substantial construction and bracing specified would render the entire roof system a rigid system, therefore the shear of all the connectors can be considered.
3. Using only the H-3 connectors in the direction of maximum loading yielded almost 400% of the required shear transfer loading to the wall. This did not consider the 10 MGT connectors or the two C-2 column capitals, therefore the unity equation was not necessary.
4. The often quoted textbook reference was "Design of Wood Structures" by Donald E. Breyer, et al, 3rd Edition. The 5th Edition contains the same reference in Section 15.3 Connection Details—Horizontal Diaphragm to Wood-Frame Wall.
5. Structural Engineer, June 2000; "Better detailing means better performance", by Mike Romanowski, SE.
6. The peer reviewers were all structural engineers well qualified in structural engineering. 9 of the peer reviewers were forensic engineers.

7. Robbins Manufacturing is a supplier and manufacturer of wood trusses and the metal plates used to connect wood trusses. Robbins Manufacturing has testing facilities for testing full size truss components and plate performance.
8. Four engineer colleagues participated in the review of the test results and written report by Robbins Manufacturing concerning the tests.
9. MGT is a heavy uplift connector for girder trusses. Each MGT was anchored to the tie beam with a 5/8" bolt and to the girder with two 12 GA straps attached with 22-10d nails. The uplift allowable was rated by Simpson to be 3985 lbs each. 4 MGT anchors were used in this line of transfer in addition to the truss anchors at each wall intersection.
10. The calculations did not consider the failure of the H-3 clips in web buckling on the compression side, however the factor was very safe and it was not considered reasonable to perform a more detailed calculation.
11. The MGT connectors were evaluated using shear on the strap connectors to the girder trusses. Since the excessive capacity it was not necessary to provide a detailed analysis incorporating the unity factor method of assigning values to the connectors.
12. Since the factor of safety was great, the unity equation was not deemed necessary. A later calculation confirmed the total shear transfer for this roof system greatly exceed the code mandated wind loads, both horizontal and vertical when considering the unity equation.
13. Since the value was excessive a more detailed analysis was not warranted.
14. The ability of the wood truss system's ability to transfer the roof diaphragm load into the wall was a major concern and was the original basis for the complaint against the engineer of record.
15. The sub-fascia was required by the Architectural details and therefore has been included in the resistance calculations for the system.
16. This analysis did NOT use the contributions from the end hips or the perpendicular roof at the stem of the T portion.
17. This confirms the truss system can transfer more load from the roof diaphragm to the wall connection than can be transferred simply by the H-3 clips. To fairly assess the shear capacity of the system, all shear connections in the building were calculated to confirm the rigid roof system was adequately tied to the walls.
18. Since the capacity was far in excess of that required, the unity equation was not necessary.
19. The load dropped when reaching 7862lbs while the peak load was 9288lbs. "The decline coincides with the cylinders reaching maximum stroke, so the

assumption can be made that the structure had not reached the ultimate load resistance" (quote from the Robbins test engineer in his report of results).

20. 3700 lbs was the load on the system when some of the H-3 clips had allowed 1/8" movement between the truss & wall plate connection. Therefore, 3700 ÷ 20 clips = 185 lbs/H-3 clip (Simpson rated the H-3 @ 125 lbs per H-3 clip).
21. No permanent bracing or full sheathing was used during either test.
22. This would be designated as failure due to deflection of 1/8 inch and should be factored by 3 for the safe load allowable.
23. This factor of safety agrees with Simpson values and exceeds the Code mandated safety factor of 3.