FAILURES OF IN-SERVICE MPC PARALLEL-CHORD WOOD FLOOR TRUSS COMPONENTS REVEAL DEFICIENCIES IN ANSI/TPI STANDARDS

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ABSTRACT: *The increased use of metal-plate-connected (MPC) parallel-chord wood floor trusses calls for a corresponding emphasis on the need for fully disclosed research and testing of web/chord joint behavior. Standardized industry procedures utilizing parallel and perpendicular-to-grain joint tests have resulted in improper design values. Instances of connector plate tooth-withdrawal in minimum five-year-old trusses have led to expensive repairs. Thirty-seven truss joint specimens removed from an existing apartment project were tested. Web/chord connections were found to have a mean ultimate strength of 165 N (37 lbF) per tooth, a value much lower than predicted by accepted standards. Trusses in additional projects were inspected and failing web/chord connections were calculated to have a mean ultimate capacity "at slip" of 60 N (13.5 lbF) to 67 N (15.1 lbF). In some cases, application of standardized safety factors to tested ultimate strength values leads to allowable design values greater than observed slip values. Until adequate industry testing is performed and disclosed and adequate standards are developed, a maximum allowable design force of 20 N (4.5 lbF) per tooth for 1.0 mm (20 gauge) plates, regardless of plate manufacturer, is suggested.*

INTRODUCTION

The evolution of the metal-plate-connected (MPC) wood-truss industry has resulted in economical and efficient construction of wood floor and roof structural systems throughout the United States. Floor trusses constructed with diagonal web members connected to top and bottom parallel-chord members comprise a significant portion of the total annual production of wood trusses. In spite of several decades of MPC wood truss production, joint behavior of parallel-chord floor trusses is still only partially understood.

In 1983, the author investigated sagging floors in several existing Dallas-area wood frame apartment and townhouse buildings. Gypsum board ceilings were removed at several locations to expose the structural wood floor trusses. The metal connector plate "teeth" that had once joined the truss members to each other had partially withdrawn from the wood. The plates themselves appeared to have bent, or "peeled" from the wood. Small gaps existed between the ends of the wood diagonal members and their abutting members. Limited resources prevented further investigation. In 1987, after observing additional structures with similar characteristics, the author, assisted by others (see acknowledgments), was able to conduct a reasonable visual, analytical and testing investigation of joints taken from five-year-old wood floor trusses in various buildings of a San Antonio, Texas apartment complex. The results suggested that wood trusses generally produced in the marketplace do not achieve the level of theoretical perfection hoped for by the industry standards, and that erroneous results result by using conventional industry standards that depend on formulas derived from Hankinson's formula. Additional visual investigations of other San Antonio projects thereafter provided similar investigative insight, as did investigations of wood-framed houses and apartment, townhouse, and condominium buildings in Austin, Houston and other Texas cities. Unfortunately, none of the additional property owners were willing to bear the expense of having the trusses tested.

During early investigations, the author corresponded with, and met on-site with, various representatives of the truss and connector plate manufacturing industry concerning observable connector problems with MPC floor trusses. In 1992, the author met with a representative of a large metal plate manufacturer who expressed dismay at the extent of connector plate failure he could observe in wood

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floor trusses that had utilized his company's product. Both parties agreed that deficiencies might exist in the industry standards used to design MPC floor trusses, and subsequently the representative supplied the author with limited, but helpful, information from his files, expressing confidence that a new truss manufacturing standard, ANSI/TPI 1-1995, would help improve design and testing standards over those found in TPI-85. As discussed further in this paper, with regard to parallel-chord floor truss metal plate connectors, the latest standards offer almost no improvement.

While seeking testing results from manufacturers, the author learned that manufacturer's test information, whether or not derived from third party testing, was unavailable, and the information was considered privileged or proprietary. The author was then told that the proprietary information was being used as the basis for recommendations to national building code authorities and others, for adoption of wood truss standards. Proprietary testing used as the basis of any code or standard is inappropriate. No legitimate reason exists, other than profit, which justifies the truss industry limiting the disclosure of research and test results while gaining standards-and-product acceptance by code authorities, engineering licensing boards, and the courts. Annually, MPC wood trusses encompass approximately 80% of homes built, 3 billion board feet of softwood and 120,000 tons of steel (Hoover, 1995). The industry commands an estimated $2.5 billion market, with the connector plate market comprising about $150 million (Grossthanner and Brakeman, 1993). Federal research funding is limited, with available funds divided among projects other than just MPC wood truss research. The time has long since past for the wood truss industry to sponsor adequate, open research and testing of trusses, not only for new projects, but to establish performance history in older projects. Engineering practitioners aware of possible shortcomings in MPC wood truss standards, as well as code officials, should now factor that awareness into project specifications and code evaluations, possibly recommending significant increases in connector plate sizing requirements until the industry is able to publicly justify smaller sizes. The industry will probably protest and increase costs to the end user, but the industry is also aware of enormous liability exposure (Grossthanner and Brakeman, 1993) and the cost of repair to deficient trusses far outweighs the cost of connector plates.

The subject is timely and controversial, and the author seeks thorough, objective comment and criticism of this paper. The author also invites the recollections by other structural engineers of similar patterns of failure in existing buildings, and hopes that the procedures outlined in this paper will lead to a significant insight into the true characteristics of MPC wood trusses.

LITERATURE REVIEW

Wood is an organic, cellular material exhibiting higher strength properties when loaded parallel to the grain than when loaded perpendicular to the grain (Scofield, et al., 1963). When a force is applied to a wood member at an angle other than parallel or perpendicular to the grain, the unit allowable strength will have an interpolated value determined using the well-established Hankinson (1921) formula. The formula was derived by the United States Air Force in 1921, at a time when there was a need for heavy timber materials used in aircraft hangers and railroad bridges. In order to transmit tension forces between members, connections consisted of steel bolts with washers, or steel split ring connectors held together with bolts. The steel components were fitted into bored or grooved wood members, and the success of load transmission for both types of connectors depended on unyielding steel surfaces compressing directly against internal wood fibers. Modern metal plate connectors do not have through-member bolts bearing directly against the wood fibers. Instead, MPC joints have individual teeth embedded a few millimeters into the face of the wood. Friction, not bolting, maintains the teeth/wood contact. Laboratory studies by researchers and the author indicate that as a tension load is incrementally applied to a connector plated joint, and the ultimate load is reached, the wood member tends to "slip" with respect to the plates and the opposing member, and slide past the connector plate. This slip and slide action is manifested by an incrementally increasing force of the wood bearing directly against the metal teeth, forcing the teeth out away from the wood, and bending the metal plate into an "arch" configuration.

The term “tooth withdrawal” is the technical term for this type of failure, but the term “plate peeling” is more descriptive in that it connotes the actual arching action of the plate itself.

It is well recognized that the organic and cellular structure of wood differs from other structural materials comprised of crystalline materials, but researchers have nevertheless developed theories and computer modeling techniques using approaches similar to those used for analysis of other materials. In conjunction with this type of research, a number of researchers have studied MPC truss joints assuming perpendicular and parallel grain orientations, with most assuming Hankinson-type behavior for intermediate orientations while citing the need for experimental truss testing. Foschi (1977) noted that metal plate connector properties are derived from those for a single plate tooth, taking into account the nonlinear character of the load-deformation relationship. Foschi derived tooth properties assuming four test grain orientations: two perpendicular to the grain and two parallel to the grain, theorizing that the use of Hankinson’s formula for intermediate angles would not reduce the accuracy with which a connection could be analyzed. McCarthy and Wolfe (1987) derived parameters for Foschi’s truss joint model and conducted tension tests with plates applied at various angles to the grain and oriented parallel to the diagonal members rather than to the chords. Triche and Suddarth (1988) expanded Foschi’s work by modifying a computer program to give design information for lumber members as well as plate connectors utilizing a nonlinear finite element approach on the entire tooth array, noting the expense of conducting full scale truss performance tests and the proprietary nature of the final results. Crovella and Gebremedhin (1990) developed finite element and elastic foundation theoretical models, assuming each tooth acting as a cantilever beam, in order to provide a practical substitute for extensive testing. The computer output showed that the load applied to the joint is not equally shared among the teeth. Predicted joint stiffness values were sensitive to foundation (wood) modulus but not sensitive to tooth width and moment of inertia.

Many researchers have emphasized the need for research and testing of truss joints and trusses. Although some testing of metal connector plate joints has been undertaken by manufacturers, only limited data which addresses web-chord plate behavior in actual trusses has been made available to other researchers and engineering practitioners. One manufacturer (Gang Nail Systems, 1981) reported that a type of 1.0 mm (20 gauge) metal plate, embedded in Southern Yellow Pine and tested in accordance with a Truss Plate Institute butt-splice tension test, has an ultimate tooth pullout value of 427 N (96 lbF) per tooth. Gupta and Gebremedhin (1990) conducted joint tests and addressed the semi rigid nature of truss joints and the brittle failure characteristic of tension splice joints. No studies had been reported on testing of web at the lower chord joints, and the researchers, citing the proprietary nature of truss testing, listed several previous researchers who had emphasized the continued need for testing of actual truss joints to determine their structural characteristics and failure modes. Vatovec, et. al. (1996), utilized three-dimensional finite-element analysis software to model MPC joint behavior and predict joint forces and displacements, conducting tension tests on conditioned Douglas-Fir lumber with plate slots oriented at various angles to the grain and plate edges aligned with chord edges. During testing, connector plates were clamped on one side of the joint. Vatovec, et.al (1997) also tested a full size scissors truss intending to use Hankinson’s formula to interpolate intermediate angle capacities; however, previous test results by Vatovec had detected inaccuracies in the use of Hankinson’s formula, so the joint per-tooth wood-plate stiffnesses were modeled incorporating linear interpolation. Comparisons between the modeled and tested scissors truss results were not made because the final tested truss-member force data was not disclosed.

Researchers have expressed concern with test procedures recommended in standards organizations. Gupta and Gebremedhin (1990) noted shortfalls in ASTM and Canadian Standard Association joint testing methods. Grossthanner and Brakeman (1993) noted that Truss Plate Institute (1985) standards incorporate parallel or perpendicular-to grain testing procedures, while observing that there are no specific standards on how to interpolate values for intermediate plate orientations. They noted that some designers use linear interpolations, others use Hankinson’s formula, and others such as ICBO (1979) suggest a step function incorporating either parallel or perpendicular to grain values. They also noted that TPI design standards, although not adopted as part of all codes, have been used by all major building codes as a basis for the product approval for connector plates. Although the more recent ANSI/TPI 1-1995 standards depart from the ICBO step approach, the standards still rely on parallel and perpendicular to grain testing and not on testing of true force-orientations.

Published standards do not adequately address long-term truss performance. Eccentric loadings on connector plates, intermittent overloads, variations in material quality and fabrication, and environmental effects, contribute to construction unknowns that are normally addressed by factors of safety. Gebremedhin and Crovella (1991) and TPI-85 defined the allowable design load as the ultimate load divided by a factor of three. This factor includes a load duration factor for test load duration of 5 minutes, a reduction to the fifth percentile specific gravity, a quality factor, and a safety factor. A recent ANSI standard has increased this factor slightly to 3.2 (ANSI/TPI 1-1995).

Theoretically, a truss joint may be analyzed as a pin, but in reality, eccentric loadings can create bending conditions in connector plates. The effects of eccentric loads have led some researchers to conclude that member or plate capacity should dictate plate design. Gupta (1994) tested tension splice wood truss joints for axial and bending conditions, suggesting that joints should be sized so that failure occurs in the wood or steel instead of depending on tooth-holding capacity. O'Regan et. al. (1998) excluded the perpendicular-to-grain force component entirely, expressing difficulty in predicting the force induced in the teeth based on plate geometry and steel properties, and proposed formulas to ensure that the joint will fail in the steel net-section.

Environmental effects alone have a very adverse effect on MPC wood trusses. Suddarth et. al (1981) conducted tests over a 3-1/2 year period of 17.0 to 22.7 foot-span full-size 4x2 trusses constructed with above-average lumber and good-to-excellent manufacturing quality. Truss deflection increased approximately ½ inch during the period due to creep attributed to environmental effects, but non-recoverable, "not observable" deformation was reported in the joints. McAlister (undated) tested trusses exposed directly to weather for a one year period and reported "serious plate backout" from No. 1 KD southern pine lumber and loss of joint strength. McAlister reported plates in one week-old control test specimens (MC = 10.2) failing by "peeling and tooth withdrawal", but in the weathered trusses (MC=9.8), the plates did not "peel", instead, the teeth crushed the wood. Reported test load was 17.8 pounds per tooth at critical slip for 20 gauge plates. Smulski (1993), in a study of eight year old parallel chord 4x2 floor trusses which had been installed in locations subjected to several cycles of extreme temperature and moisture within the floor system of a building, observed tooth withdrawal in tension chord splice plates but web-to-chord connections were essentially unaffected. Cyclic effects had ratcheted the connector plate teeth from the lumber, causing splice failure at the chords. Triche, et. al. (1994) initiated a study of design and performance attributes of an experimental MPC wood-truss bridge constructed of multi-truss girders and stress-laminated trusses utilizing a 30 percent reduction in tooth holding values. Plate "back-out" was controlled by a membrane used to limit lumber shrinkage and swelling, and by bolting the trusses together to provide perpetual compression to the faces of the connector plates.

SCOPE

The scope of this article is limited to three projects taken from the author's files. The scope encompasses (a) truss joint testing at one apartment project and (b) joint failure observations and analysis at a condominium project and an additional apartment project. Most buildings were approximately five to eight years old when structural problems were first reported by owners. All suspect trusses had been installed as second or third-story structural floor truss components supporting dead loads plus a specified 1.91 KPa (40 lb/ft2) residential live load. None of the spaces containing the trusses appeared to have suffered from previous temperature or humidity effects. Wood chord materials were stress rated Southern pine (SG = .55 taken from handbooks) and web materials were non-stamped Southern pine. The wood materials were the quality of lumber normally found in commercial lumberyards in the central Texas region. Moisture contents of various trusses were checked with a hand-held resistance-meter and were found to be similar, varying from seven to 10 percent. Cost-of-repair information was available on two of the projects. Over 4000 metal-plate-connected wood trusses were eventually inspected, of which over 2300 trusses were repaired.

Requests for engineering inspections to be conducted by the author were always initiated by property owners or property managers, usually as a result of tenant or management complaints. The complaints generally concerned “springy” and sagging floors, furniture or base cabinetry leaning away from walls, and excessive cracking in floor toppings and ceiling finishes. In some instances, a repair contractor would have already removed ceiling materials and had observed connector plate “peeling”. An inspection procedure was established for locating deficient trusses, since the original truss design or manufacturing drawings were not available. In some instances, the original architect or engineer’s floor load requirements could be found. Before ordering removal of floor or ceiling materials, the author took floor or ceiling level measurements throughout suspect living units. Floor systems with insitu deflection of 1.0 to 1.5 inches and 15 to 20 foot spans merited further investigation. Unit/deflection readings were prioritized, with small portions of the ceilings or floors then removed in the units exhibiting the most deflection. When trusses exhibiting connector plate peeling were found, additional ceiling materials were removed and stress calculations were then run based on the calculated known dead load on the trusses, plus an assumed live load of 40 psf in accordance with the building code. Loads were not factored. Embedded teeth were counted and calculated stresses reduced to ultimate unit load per tooth. Teeth partially embedded along the edges of the wood member were not included in the calculation. In all joints where plate peeling was discovered, the calculated actual ultimate load per tooth was less than the calculated ultimate allowable load obtained from factored ICBO evaluation data. The trusses were subsequently reinforced by a contractor.

Close inspection was made of all connector plates within individual trusses, not just the peeling plates. Some manufacturers have contended to the author that plate peeling is actually minor gapping that occurs during fabrication of the trusses. Some plates did exhibit minor manufactured gapping away from the wood, but in almost all instances the gapping was within allowable TPI specification. In addition, the peeling plates were found at joint locations with higher unit load per tooth values than at neighboring joints, and it was difficult for any of the observers to explain why peeling would be observed only in high stress joints and minor gapping would be observed in low stress joints within the same truss, unless the peeling had resulted from stress and not manufacturing quality. Some failed plates had been pressed directly into wood at knot or wane locations, but those instances were few. In general, based on the author’s previous intermittent experience inspecting local truss manufacturing facilities, the quality of the trusses found was considered to be typical of the area.

The effects of possible localized truss overloading during or subsequent to construction, as well as poor framing techniques, were investigated and discarded as primary contributors to the plate peeling process. Some trusses had been overloaded during construction, probably as the result of improperly stockpiled materials, but those locations were limited and usually confined to five or six adjacent trusses that exhibited fracture of the wood members as well as peeling of many plates. In almost all instances, the original building contractors had made repairs by reinforcing the trusses with conventional lumber. With regard to possible overloads during occupancy, those possibilities are normally included in standardized factors of safety, and occupancy overloads were discounted because joint failures were found to be widespread throughout the buildings and involved differing truss spans and depths. In many instances, particularly where apartment buildings had been designed with various standardized unit floor plans throughout the complex, joint failures were observed in near-identical trusses at near-identical locations. Missing hallway header supports at a few first floor locations, indicative of poor framing practices often found in local apartment projects, did contribute to truss plate failure because, for example, instead of spanning about 12 feet, the incorrectly supported truss might be required to span about 18 feet, but these discoveries were considered beneficial because information obtained from this data could be used for additional calculations for ultimate capacity of insitu connector plates. Strongback bracing across the truss chords was rarely found, another indicator of poor framing practices.

Trusses in the three projects had been constructed utilizing metal connector plates supplied by three different manufacturers. The manufacturers are not identified in this study in order to avoid improper comparisons. Instead, the connector plates are identified as Plate Types 1, 2 and 3. All three plate types were nominal 1.0 mm (20 gauge) and had a tooth density rating of 0.012 teeth per mm2 (8.0 teeth per in2). The end panels at some top-chord-bearing floor trusses exhibited failures at design loadings that were lower than failures observed at adjacent-panel web/chord joints. A complex load-deformation relationship involving combined shear and moment components was apparent at those types of end connections, and those joints were omitted from the study. Also omitted from the study were truss joints exhibiting water damage, pitched-slope roof trusses, and top and bottom chord butt splices.

Truss specimens utilizing Type 1 plates were selected for testing. The trusses had been spaced at 0.61 m (2 ft) centers. Plywood flooring supported a concrete topping with carpet. Thirty-seven non-damaged joint specimens, listed in Table 1, were selected and cut from parallel-chord floor trusses at known low-stress locations. Low stress specimens were found in trusses that were individually supported by several interior walls, which had substantially reduced the effective truss span in those trusses. Each specimen consisted of a chord member joined to two diagonal web members by metal plates on both sides as indicated in Figure 1. Trusses with Type 2 and Type 3 plates were visually inspected and analyzed in place but were not removed for testing due to owners' budget limitations. Type 2 plates were located in second story trusses encased between first floor drywall ceilings and plywood floor decking for the second story living units above. Carpet had been installed on top of the decking. Trusses with Type 2 plates were spaced at 0.61 m (2 ft) centers. Type 3 plates occurred in trusses that formed the floor structure for second story units located above automobile parking areas. Plywood had been attached to the bottom chords to form the ceilings, and plywood was also used as floor decking for the living units above. Concrete topping and carpet had been installed on top of the decking. Trusses with Type 3 plates were spaced at 0.61 m (2 ft) centers, although some trusses with Type 3 plates were spaced at 0.30 m (1 ft) centers to support concrete planters and firewalls. After removal of the ceiling materials, trusses were visually inspected for peeling connector plates. Many Type 2 and Type 3 plates exhibited peeling characteristics similar to those observed at failure during the Type 1 plate testing and two failure examples are shown in Figure 5. Truss height, span and characteristics (simple span, continuous span), connector plate sizes and tooth density were recorded. Wood defects and moisture contents were similar to those found in the Type 1 test specimens. Dead loads were then directly calculated from the actual conditions observed.

The term "progressive failure" suggests a loss of function of a structural member related to the rate of failure of that member. Based on the author's experience, MPC truss failures manifest themselves with time, and the question arises as to at what point in time has the truss or joint actually "failed". The author contends that failure has occurred once the final user has suffered a loss of function of the structure to the extent that continued use of that structure could be considered unsafe. In the case of MPC joints, some of the referenced researchers reported failure as occurring when the wood member actually pulls away from the peeled connection. Others relate the term to "slip". In this study, Type 1 tested plate failure is defined as having occurred with full withdrawal of the wood, and Type 2 and 3 observed plate failure is defined as having occurred at slip as well as at full withdrawal.

The testing and analysis discussed herein are based on load-per-tooth principles. Metal connector plates have differing tooth/area ratios that appear to lead to inconsistencies among manufacturers when relying on alternate net contact area or gross contact area methods. The load-per-tooth method was selected because it reflects the force at the actual connection point of the metal steel plate to the wood member. No teeth were deducted as a result of being located close to the edge or end of a member, since the observations had been based on counting the actual teeth penetrating into the member.

TEST PROGRAM - TYPE 1 CONNECTOR PLATES

Type 1 plate specimens were transported to the University of Texas (1987) at Austin and tested for their load-carrying capacities and their overall structural behavior. During removal, some specimens were numerically prefixed and placed in plastic bags to maintain moisture content until the specimens could be tested. The bagged specimens had moisture contents at the time of testing ranging from seven to ten percent. The remaining specimens were allowed to air dry before testing and had equilibrium moisture contents of less than eight percent at the time of testing. One objective of the tests was to determine the ultimate load-carrying capacity of the joints which might be used to give an indication of the relative reserve capacity compared with the design or service load capabilities. A second objective was to determine the characteristics of the deformation of the joint when a force is applied parallel to one of the web members. This assessment was made by measuring the amount of deformation along the line of action of the loaded web member from one side of the metal plate to the other. Each of the test specimens had the general configuration shown in Figure 1.

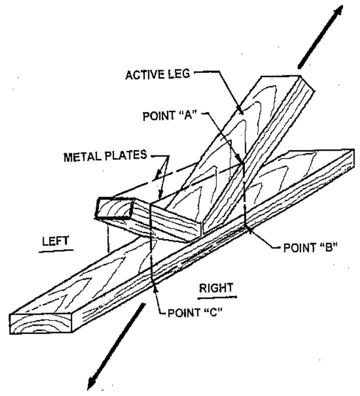

FIG. 1. Test Specimen Configuration - Type 1 Plates

Five different plate sizes were measured, ranging from 76 x 102 mm (3 x 4 in.) to 114 x 203 mm (4.5 x 8 in.). In general, the plate sizes on each side of the joints were the same, however, the plates on a given joint often were not symmetrically placed with respect to the members to be joined. In addition, the left and right plates on a given joint often had slightly different alignments or plate orientations. Minimum plate thickness found was 0.86 mm (0.034 in.) and the maximum was 1.04 mm (0.041 in.). Most plates were approximately 0.94 mm (0.037 in.) to 0.97 mm (0.038 in.) thick. Connector plates at most joints were found to be either fully embedded prior to test or had total gaps (gaps at both faces combined) not exceeding 1.59 mm (0.063 in.). Pre-existing defects in specimens consisted of slight plate bending attributed to fabrication (5 joints); wane (3 joints); wood splits (2 joints) with one specimen having different sized plates; ¾ inch plate contact at chord (2 joints) with one specimen having a long knot and chipping on the active web; and knot only (1 joint).

Test Apparatus, Instrumentation and Procedure - No suitable standard method, ASTM, ANSI or otherwise, was found which could be used to evaluate the truss joints and an alternative method for testing was developed. Reference at the time of test was made to TPI-85, Appendix C, the industry standard used for testing plates for pure tension, and where applicable those procedures were followed. The author subsequently compared ANSI/TPI 1-1995 to the procedure used and found no procedural conflicts in the test methods. The tests were conducted so that when tension load was applied to the active web, the amount of bending induced into the chord was minimized. This was achieved through the use of a specially designed steel sleeve used to encase the chord member. Figure 2 shows the steel sleeve placed around the two ends of the chord member and tied together with two rods. The active web is the vertical member in the figure and during the test it was gripped by wedges built into the testing machine. The steel plate between the two sleeves was welded to another plate containing a hole allowing the assembly to be bolted to a clevis attached to the testing machine. The hole and web were aligned during testing to increase the moment of inertia of the assembly in order to minimize any bending of the chord. Relative weight of the test assembly was inconsequential and observations during testing confirmed that the assembly functioned as desired.

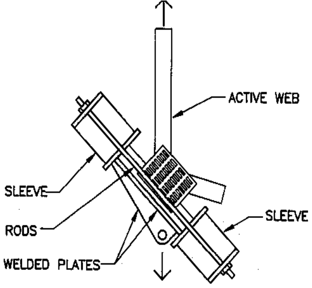

FIG. 2. Steel Sleeve Test Apparatus

A 36,320 kg (80,000 lbF) capacity Tate-Emery Screw-Type testing machine was used to tension load the specimens. The duration of each test was approximately ten minutes. Direct Current Displacement

Transducers (DCDTs) were used to measure the deformation in the joint. Two calibrated DCDTs were used per test, one on each side of the test specimen. Each DCDT had a 25.4 mm (1 in.) stroke and an accuracy of 0.025 mm (0.001 in.). The DCDTs were fixed to a metal bracket that in turn was connected by two set screws to the active web just outside the boundary of the metal plate. The spring-loaded plungers of the DCDTs rested on a small metal angle attached to the wood surface of the chord, thus providing an effective method for measuring web to plate deformations without actually touching the plate. In this way, the total deformation of the entire metal plate joint was found along a line parallel to the active web member. This total deformation thus includes the deformation of the wood, the deformation of the teeth interacting with the wood, and the deformation of the plates.

General Observations - The mode of failure noted most often was withdrawal of the teeth from the wood combined with plate bending. Specimens classified as web failures experienced tooth withdrawal from the active web (24 joints) and withdrawal at the web with web splitting (1 joint). Specimens classified as chord failures were chord (6 joints); both chord and web (3 joints); withdrawal at chord with chord splitting (2 joints); and chord splitting adjacent to the plate (1 joint). At lower test loads, minimal effect on the plate/tooth/wood interchange was noted. For web failures, when the load had increased to the level where approximately 0.38 mm (0.015 in.) slip occurred, a corner of the connector plate began backing away from the wood (Point A, Figure 1). The action of the wood pulling against the teeth resulted in the plate peeling outward. Peeling accelerated rapidly with additional load until full pullout of the web member from the connection was reached. For chord failures, peeling was initiated at Point B unless the plates were offset slightly toward the active web member, in which case peeling was initiated at Point C. The failure mechanisms reflect the complexity of joint behavior as well as a function of the location of the plate on the joint. There were no cases in which the plate itself was observed to fracture. In addition, before each specimen was tested, a line was drawn on the wood surface around the perimeter of each plate and no gross plate distortions were noted after testing other than those associated with tooth withdrawal.

Ultimate Strength – Table 1 lists the plate sizes tested and the test results, including ultimate strengths. Table 1 includes the acute angles between the axis of the active web and the top of the left metal plate shown in Figure 1, and pre-test average gaps of two plates at each joint. For the 76 x 102 mm (3 x 4 in.) plates, mean angle between the axis of the active web and the top of the left plate was 35.1 degrees and the mean ultimate strength was 7618 N (1712 lbF) with a standard deviation of 1753 N (394 lbF). Six specimens had 76 x 127 mm (3 x 5 in.) plates with a mean angle of 34.8 degrees and a mean ultimate strength of 9189 N (2065 lbF) with a 2238 N (503 lbF) standard deviation. Four specimens had 114 x 127 mm (4.5 x 5 in.) plates; angles of 28, 34, 29 and 36 degrees, and a mean ultimate strength of 10649 N (2393 lbF) with a standard deviation of 427 N (96 lbF). Five specimens had 114 x 203 mm (4.5 x 8 in.) plates, a mean angle of 35.8 degrees, and a mean ultimate strength of 17640 N (3964 lbF) with a standard deviation of 4788 N (1076 lbF). Specimen T10H had 114 x 152 mm (4.5 x 6 in.) plates. Specimen T3E had plates of different sizes, 76 x 102 mm (3 x 4 in.) on the left side and 114 x 127 mm (4.5 x 5 in.) on the right side.

Considering the acute angles as approximately equal, the results generally show that the specimens with larger plates had higher ultimate strength. The pattern of increasing strength with increasing plate size was broken by the 7962 N (1790 lbF) strength of the single specimen with 114 x 152 mm (4.5 x 6 in.) plates. The ultimate force per tooth remained within a comparable range for all plate sizes. The contact stresses between the plates and the active web were computed by dividing the force applied to the active web by the sum of the contact areas of the left and right plates with the active web. The contact stress values range from 1207 kPa (175 psi) to 3172 kPa (460 psi). The mean value for all specimens is 2088 kPa (303 psi) with a standard deviation of 508 kPa (74 psi).

TABLE 1. Results of Joints Tested in Tension Loading.

Joint no. (1)	Plate size (mm) (2)	Plate angle (deg.) (3)	Pre-test avg. gap (mm) (4)	Ultimate force (N) (5)	Contact area (mm2) (6)	Contact stress (kPa) (7)	Contact teeth (total) (8)	Ultimate force/tooth (N) (9)	Load at 0.38 mm slip (N) (10)
T3A	76x102	39	0.00	11343	3806	2979	43	264	7784
T1K	76x102	33	0.80	10231	3226	3172	44	233	4893
T10F	76x102	34	0.00	7384	2839	2599	34	217	5961
1004B	76x102	32	0.79	9897	3613	2737	46	215	7562
105C	76x102	33	2.78	9230	3677	2510	44	210	5756
T3K	76x102	35	0.39	5560	2194	2537	28	199	3692
704A	76x102	36	0.80	7251	3290	2206	41	177	6459
1501A	76x102	38	0.39	7340	3226	2275	41	179	5365
T3J	76x102	33	0.80	8363	3806	2199	49	171	6450
T1B	76x102	36	0.39	8452	4064	2082	54	157	5560
1004A	76x102	35	2.39	7251	3742	1937	47	154	5440
T3B	76x102	36	0.00	7784	4258	1827	53	147	5338
503B	76x102	35	1.19	7985	4581	1744	55	145	4079
T1J	76x102	33	0.80	7340	3935	1868	51	144	4893
T1A	76x102	40	0.80	7295	4516	1613	59	124	3692
T3E	(1)	35	0.80	4893	3097	1579	40	122	4003
T10B	76x102	35	3.98	7517	4710	1600	61	123	3559
T10C	76x102	34	0.39	7340	4710	1558	61	120	3114
105B	76x102	34	1.99	5627	3677	1531	47	120	4933
704B	76x102	35	1.60	4003	3032	1317	35	114	2651
T10M	76x102	36	0.00	5071	4193	1207	58	87	4893
907B	76x127	32	1.19	12322	4452	2765	57	216	7397
1605A	76x127	37	1.19	9519	3935	2420	48	198	8603
1501B	76x127	34	0.39	11121	4516	2461	57	195	7811
1605B	76x127	35	1.19	8407	4323	1944	53	159	5912
612A	76x127	36	1.60	6895	4000	1724	51	135	5436
612B	76x127	35	1.19	6850	4452	1538	60	114	5596
T1E	114x127	28	1.60	10720	4064	2641	48	223	9119
105A	114x127	34	0.79	10409	5871	1772	73	143	9452
503A	114x127	29	1.19	11209	5742	1951	79	142	6397
T10A	114x127	36	0.00	10231	7419	1379	98	104	6450
T10H	114x152	36	5.56	7962	3871	2055	60	133	6227
T3G	114x203	35	0.00	24020	8839	2717	104	231	19127
T3F	114x203	34	0.80	19394	7419	2613	95	204	17971
T10N	114x203	38	0.80	15791	6452	2448	79	200	10453
T1G	114x203	36	0.80	17971	8064	2227	105	171	13567
T1F	114x203	36	1.59	10987	7290	1510	92	119	(2)
Mean						2088		165	
Std. dev.						508		44	

(1) Differing plate sizes (see text). (2) Failed at 0.36 mm slip.

The plates had an average tooth/area value of 0.013 teeth per mm2 (8.2 teeth per in2). Table 1 indicates the total number of teeth embedded in the active web member from plates on both sides and provides the force per tooth at the ultimate load. The lowest force per tooth value at ultimate load was 87 N (20 lbF) and the highest value was 264 N (59 lbF). The mean force per tooth and standard deviation for all the specimens were 165 N (37 lbF) and 44 N (10 lbF) respectively. Table 2 summarizes average percent residual strength of each size of connector plate after 0.38 mm (0.015 in.) slip. The table indicates that as the connector plate size increases, the residual strength decreases.

TABLE 2. Average Percent Residual Strength After 0.38 mm (0.015 in) Slip

Plate size (mm) (1)	Plate size (in) (2)	1-(Pslip/Pult) x 100 (%) (3)
76 x 102	3 x 4	32.5
76 x 127	3 x 5	26.1
114 x 127	4.5 x 5	26.2
114 x 152	4.5 x 6	21.8
114 x 203	4.5 x 8	13.1

Load Deformation Behavior - Figures 3 and 4 summarize the load-deformation curves of each joint tested and reflect web and chord plate failures respectively. The curved line in each figure represents the average values obtained from the two DCDTs attached to each specimen. At 0.38 mm (0.015 in.) slip, the average load for the web-plate failure condition was 129 N (29 lbF) per tooth. For the chord-plate failure condition, the average was 102 N (23 lbF) per tooth. All of the responses reflected nonlinear behavior over some part of the load-deformation curve. Some responses were nonlinear throughout, while others had regions of linear behavior followed by nonlinear behavior. Combinations of linear and nonlinear responses for wood connector load-deformation behavior, including metal plates, are common.

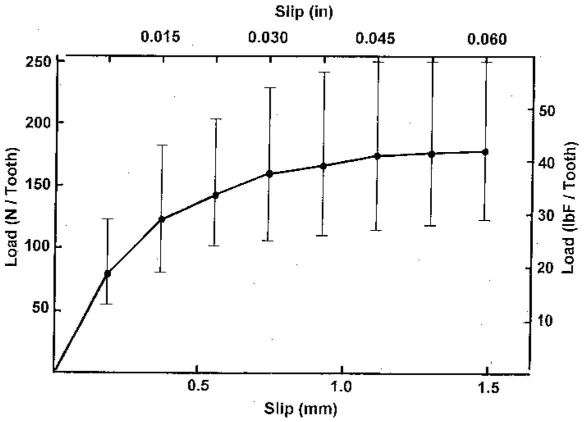

FIG. 3. Load-Deformation Curve for Web Plate Failure Condition

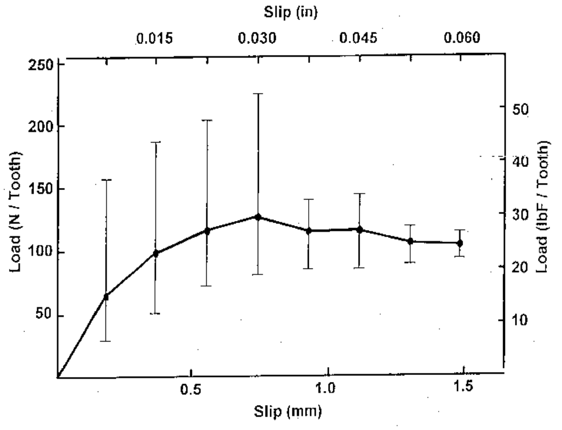

FIG. 4. Load-Deformation Curve for Chord Plate Failure Condition

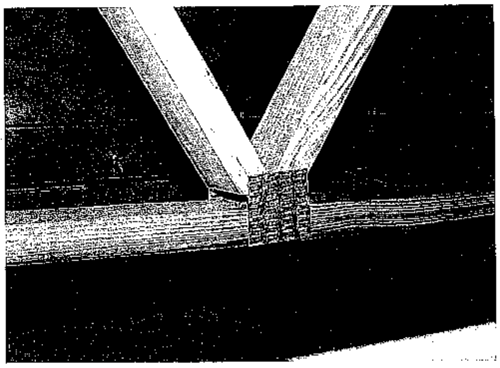

Plate peeling with web end separation

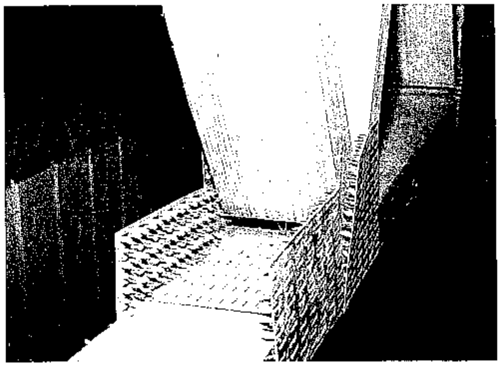

Plate peeling adjacent to bottom chord splice

Fig. 5. In-Service Joint Failure Modes

COMPARITIVE STUDY

The comparative study of Type 2 and Type 3 connector plates was undertaken to relate field observation results to Type 1 test results. Joint forces were calculated assuming known dead loads plus a 1.91 kPa (40 lb/ft2) residential floor load applied to the top chord of the trusses. Connector plates exhibiting peeling were considered to be in the process of failing or having totally failed. To simplify the calculations, no secondary analysis was made assuming force redistribution to other joints as a result of failure of a nearby joint in the same truss. Truss analysis was made using STAAD III software, assuming continuous chords and pinned web joints. Ultimate capacity "at slip" was calculated based on observed peeling. No attempt was made to quantify the amount of actual slip that had taken place. For Type 2 plates, the mean force at slip was 60 N (13.5 lbF). For Type 3, the mean force at slip was 67 N (15.1 lbF).

Figure 6 shows Gaussian distribution curves for the three plate types. For plate Type 1, a curve reflecting the actual ultimate load test results is shown. For plate Types 2 and 3, the calculated capacity at slip was used to construct the curves. Factoring the Type 1 mean force of 165 N (37 lbF) per tooth by a recommended factor of safety of 4, (ICBO 1982) results in an allowable design force of 41N (9.3 lbF) per tooth. Use of this value as a design value is unconservative for the Type 2 and Type 3 plates, many of which exhibited peeling at lower calculated forces.

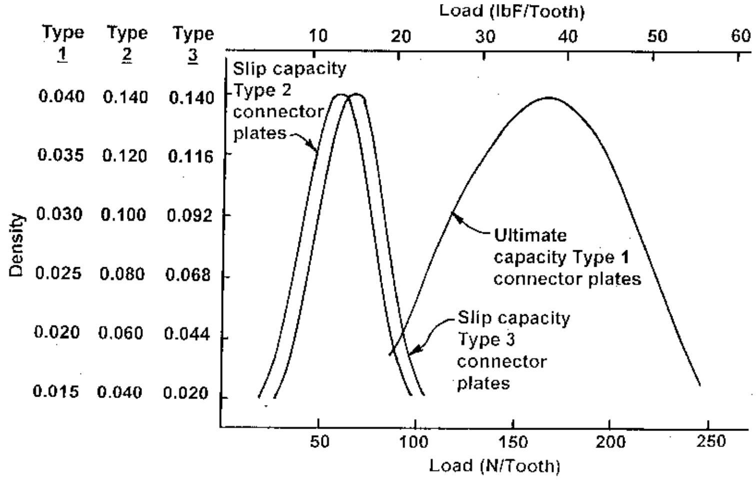

FIG. 6. Failure Distribution Curves for Types 1, 2 and 3 Plates

For comparison purposes, plate resistance values for three additional brands or styles of connector plate, expressed in terms of pounds per square inch of gross area plate contact and embedded into Southern pine (S.G.=0.55), were obtained from published ICBO (1997, 1998) evaluation reports. The three brands were Tee-Lok 20, Truswal Model 20, and Alpine Wave, and are termed "new brands". Each plate was rated 20 gauge steel with a tooth density of 8.0 teeth per square inch. With the density having been provided, an allowable load per tooth for each direction of grain and load with respect to length of plate, for the combined brands, could then be calculated. Average values of AA, EA, AE and EE for the three brands were then obtained, and the values then factored by 3.2 to obtain ultimate values. The calculated ultimate values, expressed in terms of pounds per tooth, are AA= 86.0, EA=69.6, AE=68.7, and EE=66.9. Utilizing expressions given in Section 11.2.1 of ANSI/TPI 1-1995, a web aligned 35 degrees with respect to a bottom chord, with the connector plate teeth aligned parallel with the bottom chord, should fail at an approximate ultimate load of 74.9 pounds per tooth.

In a similar manner, plate resistance values for Type 1 and Type 2 connector plates, expressed in terms of pounds per square inch of gross area plate contact and embedded into Southern pine (S.G.=0.55), were obtained from ICBO evaluation reports. No brand markings were found on the Type 3 plates and they were not considered further. For comparison, the Type 1 and Type 2 plate group is termed "old brands". Each plate was rated 20 gauge steel with a tooth density of 8.0 teeth per square inch. Resistance values for the old brands had been tabulated in terms of direction of load with respect to the length of the plate and the applicable standard was TPI-85. From average values an allowable load per tooth was then calculated. The values were factored by 4.0 to obtain ultimate values, as called for by ICBO (Gang-Nail Systems). The average ultimate value for zero to 45 degree orientation was 86.3

pounds per tooth, and for 46 to 90 degree orientation, 66.5 pounds per tooth. In order to obtain a reasonable comparison with ANSI/TP1-1995, it was then necessary to extrapolate comparative values of "new brand" plates in order to obtain values of AA, EA, AE and EE since those values had not been published for "old brand" plates. Utilizing expressions given in Section 11.2.1 of ANSI/TPI 1-1995, a web aligned 35 degrees with respect to a chord, with the connector plate teeth aligned parallel with the bottom chord, should fail at an approximate ultimate load of 74.8 pounds per tooth utilizing the "old brand" plates, almost exactly the same load as that for the "new brands".

Another comparison was made for the old brands, utilizing Hankinson's formula. For parallel to grain loading, an ultimate load value of 86.3 pounds per tooth was used, and for perpendicular to grain loading, the value used was 66.5 pounds per tooth. Applying these values directly to Hankinson's formula, with a web/chord angle of 35 degrees, the old brand plates could have been designed for 78.6 pounds per tooth.

Prior to ANSI/TPI 1-1995, ICBO allowed for a "step procedure" to be used for "old brand" plates, thereby allowing the full allowable tension parallel to the grain value to be used until the web/chord angle exceeded 45 degrees. Translating this allowable load to an ultimate load, the designer at the time could have based the plate design on an ultimate load of 86.3 pounds per tooth for webs aligned 35 degrees with respect to the chord.

When the 74.8, 78.6 and 86.3 pound per tooth potential plate design values are compared with the Type 1, 2 and 3 ultimate load or slip bell curve values, it can be seen that the use of ANSI/TPI 1-1995, Hankinson's formula, and ICBO, can be a very unconservative practice when designing web/chord connector plates for parallel chord floor trusses.

The question then becomes how to establish an appropriate design value. Assuming that standardized factors of safety are appropriate (considering the lack of published research on the long term performance of MPC wood trusses, the author does not contend this is true), then applying a factor of safety of 3.2 from ANSI/TPI 1-1995 to the mean result for type 1 plates in Southern pine parallel chord floor trusses, the design allowable value would be about 52 N (11.6 lbF) per tooth. Applying a factor of safety of 1.6 to the type 2 and 3 slip values results in type 2 plates designed for 37.5 N (8.4 lbF) per tooth, and type 3 plates designed for 41.8 N (9.4 lbF) per tooth. The problem with the lowest value being used as design values is that both type 2 and type 3 plates exhibit slip at values lower than 37.5 N (8.4 lbF) per tooth. A factor of safety of 3.0 applied to the mean force for Type 2 plates, results in an allowable design force of approximately 20 N (4.5 lbF) per tooth, which is lower than the lowest joint failure range of all curves shown in Figure 6. The value suggests that decades of research based on plate-tooth-withdrawal restrained by friction with wood, should be channeled into another direction, including designing connector plates to ensure steel or wood net-section failure before tooth-withdrawal failure.

REPAIRS

Trusses were repaired in place using plywood plates. One repair method proposed was to nail individual plywood gussets into place over the connector plates; however, oversized gussets would have been required because nails could not be driven through the plywood and then through the metal connector plates. All trusses were less than 24 inches deep and singular ripping of plywood sheets involved less labor; therefore, trusses were redesigned as I-beams or box beams and repaired with plywood panels glued and nailed into position.

Repair cost information was not available for Type 1 connector plates. Repair costs for the period 1989 through 1994 for Type 2 plates averaged \$22.67 per plate and Type 3 plates averaged \$18.81 per plate. Type 2 repair costs include drywall replacement and carpet reinstallation. Type 3 repair costs were lower than Type 2 costs because trusses with Type 3 plates had been installed over parking areas and the cost of interior finishes was not a factor. All costs exclude rent loss and engineering fees. Economies of scale were evident for projects with 150 or more failing trusses. Higher average repair costs were experienced at other projects requiring repairs to a smaller number of trusses.

CONCLUSION

Suggested investigative and testing procedures for in-service floor trusses have been provided in this paper, for use by other practitioners and researchers. MPC wood truss connector plate values recommended by manufacturers or condoned by building code officials are unconservative when compared to values obtained by test and comparative observation, and licensed engineering practitioners designing web/chord truss connections must exert restraint in minimizing the size of connector plates for the sake of economy. Until adequate industry testing is performed and disclosed and adequate standards are developed, a maximum allowable design force of 20 N (4.5 lbF) per tooth for 1.0 mm (20 gauge) plates, regardless of plate manufacturer, is suggested. An extensive research and testing program for in-service MCP trusses is suggested, and further exploration into environmental effects and factors of safety should be included. Abandoning the practice of designing metal connector plates for tooth withdrawal instead of ensuring steel or wood net-section failure before tooth-withdrawal failure should be explored.

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