Fatigue Design Criteria of MPC Wood Trusses for Bridge Applications

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Abstract

Lightweight metal plate connected (MPC) wood trusses are commonly used in residential and commercial roof construction. These trusses also offer advantages for low-volume rural bridges such as ease of construction, high stiffness and wide availability. MPC trusses can be used in bridge applications provided that proper consideration for fatigue, corrosion protection, and detailing to prevent connector plate 'back-out' are incorporated into the MPC joint design. Using fatigue test results of 172 individual MPC joints and 34 fill-scale trusses, recommendations for fatigue design are given. Two MPC truss bridges, 46 ft and 39 ft long, built in Maine in 1993 and 1994 are described briefly.

Keywords: bridges, timber, trusses, fatigue, connectors, metal, corrosion

Introduction

In many parts of the United States the highest-capacity wood structural member, fabricated locally, is the metal plate connected (MPC) truss. MPC trusses are lightweight wood trusses assembled using dimension lumber and metal connector plates. They are commonly used for framing roofs and floors in both commercial and residential buildings.

MPC trusses are cost-effective because they are constructed using widely available dimension lumber, are simple to fabricate, structurally efficient and easy to handle. MPC trusses also offer high stiffness, which is particularly important in bridge construction. Until recently, however, these trusses have not been used for constructing timber bridges (Trish et al., 1994; Dagher et al., 1992, 1994, 1995; Altimore, 1995). Several ways in which MPC trusses can be used in timber bridges are shown in Dagher et al. (1994).

In 1991, the University of Maine and the USDA Forest Products Laboratory initiated a feasibility study to investigate the use of MPC trusses in low-volume rural bridge applications. The one-year study examined a number of issues including economics of MPC truss bridges, potential structural systems, and fatigue and corrosion of MPC joints. The positive results of the feasibility study warranted a more detailed investigation (Dagher et al., 1992). Through the detailed investigation in 1993-94, fatigue design criteria for MPC joints in timber bridges were developed (Dagher et al., 1994). In addition, two demonstrations stress-laminated MPC

truss bridges were constructed in Maine and tested (Altimore, 1995). In 1995 and 1996, the proposed fatigue design criteria have been refined through fatigue testing of full-scale trusses.

This paper summarizes the research results and addresses the following design concerns:

1. Fatigue: Truss joint connector plates are commonly made from 16, 18 or 20 gage steel. Plates are punched from sheets with a die to form teeth that are an integral part of the plate. The punching process causes stress concentrations in the thin plate and makes them more susceptible to fatigue.

2. Corrosion: Steel sheets are commonly galvanized before plates are punched. After punching, unprotected steel is exposed in the critical areas around the roots of the teeth. Stainless steel plates would reduce corrosion concerns, but are prohibitively expensive.

3. Plate 'back-out': Like nails, connector teeth can 'back-out' (i.e., partially withdraw) when the wood in which they are embedded is subjected to repeated cycles of wetting and drying.

Fatigue Testing of Individual MPC Joints

A limited amount of high-cycle fatigue test data on MPC joints is available in literature. In the United States, Sletteland (1976) conducted experiments on the fatigue of metal plate connectors. Hayashi and colleagues (1979, 1980, 1981, 1983 and 1984) conducted fatigue studies on truss connector plates at the Wood Research Institute in Kyoto, Japan. Significant differences in the results obtained by Sletteland and Hayashi made it difficult to draw solid conclusions on fatigue design of MPC joints based on the available test data (Dagher et al., 1992). As a result, the University of Maine embarked on a comprehensive study of the fatigue of MPC joints. One hundred seventy-two high-cycle fatigue tests on MPC wood joints were conducted. The tests were typically carried out to two million cycles, the residual strength of the connector in tooth holding, plate tension and plate shear were determined (TPI 1978, 1989; CSA, 1980).

Test parameters included:
a) load and plate direction with respect to the grain of the wood;
b) plate manufacturer;

c) galvanized versus stainless steel plates;
d) plate thickness;
e) plate confinement (various levels of mechanical confining pressure were applied to the joints)
f) size and geometry of the joint, including the number of rows of teeth in the connector;
g) speed of fatigue testing; and
h) magnitude of fatigue load.

Also, some specimens were moisture-cycled prior to fatigue testing. Moisture cycling was conducted by R. Wolfe at the Forest Products Laboratory. The effects of moisture cycling on the static strength of MPC truss joints has been studied by Wilkinson (1966), White (1977) and Groom (1994).

In addition to the fatigue testing, 155 static strength tests were conducted to provide a reference point to determine the residual strength of the fatigue-tested joints. Details of the fatigue testing study are given in Dagher et al. (1992, 1994).

Fatigue Design Methodology for MPC Joints

Using the results of the individual joints testing program, a fatigue design methodology for MPC truss joints for use in bridge construction was developed. The fatigue criteria are simple to use. It is proposed that fatigue design of MPC truss joints be conducted as follows:

First, the joints are designed for total stress using the plate manufacturer's allowable static design values.

Check 1: Static design for total stress:

(DL + LL + Impact + etc....) stresses < allowable stress of metal plate connector stated by maker

Next, empirically-derived reduction factors ϕ_{th}, ϕ_{pt}, and ϕ_{sh} for fatigue strength are applied as follows:

Check 2: Fatigue design for LL + Impact only:

(LL + Impact) stresses < ϕ_{th}(allowable static tooth holding)

< ϕ_{pt}(allowable static plate tension)

< ϕ_{sh}(allowable static shear values)

in which:

ϕ_{th} = reduction for manufacturer's allowable static tooth holding value(s)

ϕ_{pt} = reduction for manufacturer's allowable static plate tension value(s)

ϕ_{sh} = reduction for manufacturer's allowable static shear values

The net effect of the fatigue reduction factors ϕ_{th}, ϕ_{pt} and ϕ_{sh} is to increase the size of the MPC joints above and beyond what is required in static design, thereby increasing the fatigue resistance of the trusses.

For timber bridge applications, it is proposed that the three fatigue reduction factors ϕ_{th}, ϕ_{pt} and ϕ_{sh} b e selected to provide adequate strength of the MPC joints after the application of one to two million cycles of the design truck loading. This is a severe fatigue design requirement for most timber bridge applications, particularly since most timber bridges are on low-volume rural roads with low Average Daily Truck Traffic (ADTT). According to AASHTO LFD (American Association of State Highway and Transportation Officials - Load Factor Design) requirements for fatigue design, the two million cycles fatigue design requirement applies to ADTT counts comparable to interstate highway bridges. Most timber bridges intended for this application would have ADTT counts that would place them in the 300,000 cycles or less AASHTO fatigue design category. The one-to-two million cycles design suggestion used here accounts (indirectly) for the interaction between fatigue and corrosion of the MPCs.

Fatigue Testing of Full-Scale Trusses

Using the fatigue test results of the individual MPC joints, preliminary empirically-derived reduction factors ϕ_{th} = 0.60, ϕ_{pt} = 0.35 and ϕ_{sh} = 0.35 were selected for further evaluation. The evaluation program included fatigue testing of full-scale trusses designed using the proposed fatigue reduction factors, as well as the fatigue testing of static-designed trusses. The general configuration of the trusses used in the foil-scale laboratory fatigue study is given in Figure 1.

The fatigue testing was conducted at a rate of five Hz. Clearly, in the interest of time, it was desirable to conduct the testing as quickly as possible since it requires 4.6 days to reach two million cycles at five Hz. However, five Hz was the fastest value that could be used while keeping the dynamic effects related to the inertia of the truss within acceptable limits. To study stiffness degradation over time, the test control equipment was programmed to interrupt the fatigue testing at regular intervals and to conduct a quasi-static, data-taking load cycle. LVDTs were used to measure maximum truss deflection during the data-taking cycles. All fatigue testing was conducted up to two million cycles or up to failure, which ever occurred first. For trusses that reached two million cycles, the fatigue testing was interrupted and the static residual strength of the trusses was determined.
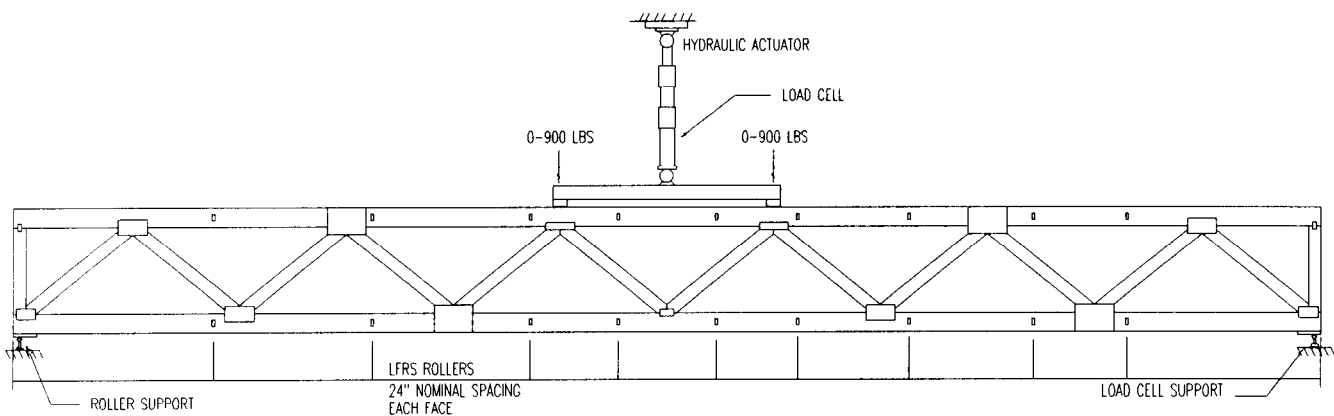

Figure 1 -Configuration of Truss Fatigue Test

Table 1 - Results of full-scale fatigue testing of trusses.

Truss Group	Number of Trusses	MPC Joint Design	% Trusses that Exceeded 2 Million Cycles	% Trusses that Exceeded 1 Million Cycles	Mean Stiffness at 50,000 Cycles (lbs/in)	Mean Stiffness at 2 Million Cycles (lbs/in)	% Stiffness Loss at 2 Million Cycles
1	15	Fatigue, tension chord splices at panel points	66	86	5,020	3,515	30
2	9	Fatigue, tension chord splices between panel points	78	100	4,930	3,765	24
3	10	Static design	0	0	4,610	0	100

The 34 trusses that were fatigue tested were divided into three groups as shown in Table 1. All joints were sized to resist the two 900 lb. concentrated loads shown. Therefore, every joint was critical for the load configuration shown. The only difference among the three groups of trusses is how the joints were designed.

The MPC joints in the fifteen Group 1 trusses were designed for fatigue using the reduction factors $\phi_{th} = 0.60$, $\phi_{pt} = 0.35$ and $\phi_{sh} = 0.35$. Also, the tension chord splices were placed at the panel points as is often done in practice today. The trusses in Groups 1 and 2 were identical except that the tension chord splices in Group 2 were relocated between the panel points to reduce the stresses in the joint at the panel points. This was done because the fatigue failures in Group 1 trusses tended to occur at the tension chord splices located at the panel points. The joints in the Group 3 trusses were not designed for fatigue, In other words, all the reduction factors used in designing the joints in the Group 3 trusses were $\phi_{th} = \phi_{pt} = \phi_{sh} = 1.0$, as is commonly done in static truss design today. The size of the MPC joints in the Group 3 trusses were therefore smaller than those in Groups 1 and 2 and less resistant to fatigue.

The results of the fatigue testing are summarized in Table 1. The Table gives average test results for each of the three groups of trusses. The variability of the results was significant, as one would expect in fatigue testing. In general, the Table shows that the proposed design criteria, using the reduction factors $\phi_{th} = 0.60$, $\phi_{pt} = 0.35$ and $\phi_{sh} = 0.35$, give a mean fatigue strength between 1 and 2 million repetitions of the static design load. Smaller values of the reduction factors increase the fatigue strength. As indicated by comparing Groups 1 and 2 trusses, attention to joint detail to minimize stresses within the joints will also improve the fatigue design strength.

Developing rigorous probability-based fatigue reduction factors is difficult at this time considering the high variability of the fatigue strength data and the limited amount of test data currently available. It should also be pointed out that other more rigorous approaches to fatigue design were considered. For example, a fatigue design approach may attempt to estimate the maximum stress ranges within the MPC connectors, accounting for factors such as stress concentrations, eccentric moments, shear tension and compression stress combinations within the joints.

Such an approach, while more proper, would be too cumbersome for everyday design, particularly since the proposed semi-empirical methods appear to work well.

The example shown in Table 2 illustrates the application of the proposed fatigue design criteria. The example uses actual values for 20 gage plates in southern pine from two MPC manufacturers, Alpine and MiTek. The first two rows of the Table show allowable static design values for the plates as stated by Alpine and MiTek. The third row shows the proposed fatigue reduction factors. The fourth and fifth rows show the reduced allowable design values to account for fatigue. Values in the first two rows are then compared with total stress (i.e. with stresses resulting from any working stress design load combination). Values in rows four and five are compared with live and impact stresses only.

It is important to note that the proposed fatigue design criteria were developed primarily by testing Alpine and MiTek's 20-gage connector plates. It is likely that the reduction factors proposed here for fatigue will change from one type of plate to another and from one

manufacturer to another. This is because the stress distribution in the plate is a function of the location and geometry if the punched teeth, which vary from plate type to plate type (Mirsa 1964, 1966). Before utilizing these criteria, it is therefore essential that plate manufacturers verify the applicability of the criteria to their plates. Standard fatigue tests similar to the ASTM D1761 fastener static test (ASTM, 1986) should be developed in the future to accomplish this.

Demonstration Bridges

Two demonstration stress-laminated MPC truss bridges, funded through the ISTEA (Intermodal Surface Transportation Efficiency Act) Federal Highway timber bridge demonstration program, were constructed in Maine. The first bridge, located in Byron, is 46 ft. long, 32 ft. wide and has an 18 degree skew. Located in North Yarmouth, the second is 39 ft. long, 31 ft. wide and has a 12 degree skew. Both bridges were designed for AASHTO H25 loading and have similar configurations. Therefore, only the Byron bridge is described below.

Table 2 - Example of proposed fatigue design criteria

	Design Values	Tooth Holding (lb/2si)[a]		Plate Tension (lb/2li)[b]		Plate Shear (lb/2li)	
		AA	AE	AA	AE	AA	AE
Alpine 20 Gage (Net Area)	Static. Compare with any load combination	444	324	813	797	622	418
MiTek 20 Gage (Gross Area)	Static. Compare with any load combination	258	216	1,406	875	758	607
Fatigue Reduction Factors		0.60	0.60	0.35	0.35	0.35	0.35
Alpine 20 Gage	Fatigue. Compare with LL and Impact	266	194	285	279	218	146
MiTek 20 Gage	Fatigue. Compare with LL and Impact	154	128	492	306	265	212

[a] Lbs of load per square inch x two plates
[b] Lbs/linear inch x two plates

The Byron bridge is a stress-laminated, solid truss design, with equal numbers of structural and spacer trusses. The trusses were lifted onto the abutments in pre-assembled modules held together by nails and metal straps. Placed parallel to traffic, structural and spacer trusses are alternated side-by-side to form the width of the bridge. They are stressed in the transverse direction using epoxy-coated post-tensioned rods. A spacer truss is a 'stripped down' truss with only enough webs so that it can be handled safely as a unit. Spacer trusses prevent metal-on metal contact of the connector plates in adjacent trusses and prevent distortion of the bridge during stressing. Side-by-side placement of the trusses and post-tensioning also prevents the plates from backing out due to moisture content fluctuations in the lumber and fatigue loading. Metal connector plates were galvanized before punching and brush-coated with an epoxy paint after the trusses were manufactured. This protection system is one of five recommended for use on metal plate connectors by the Steel Painting Council to yield a 70 year life in a "sheltered marine environment" (Bruno et al., 1989). Additional protection of both the plates and wood against water is provided by a membrane placed on top of the trusses and paved with asphalt. The bridges have no separate deck since traffic runs directly over the top chords of the trusses.

The Byron bridge was load-tested and opened to traffic in November 1993. The North Yarmouth bridge was load-tested and opened in June 1994. The load tests verified the high stiffness of this bridge system. Under two 70,000 lb. trucks positioned so as to maximize deflection, the measured deflections in both bridges were less than L/2300 (Altimore, 1995).

Concluding Remarks

Metal plate connected wood trusses offer an attractive alternative for low-volume rural bridges. Their advantages include wide availability, ease of handling, speed of construction, and high stiffness. Proper fatigue prevention of plate 'back-out' and corrosion protection, as described in this paper, are essential to ensure longevity of the system. The fatigue design methodology described in this paper is based on tests conducted on individual joints and has been verified by conducting tests on full scale trusses. While the proposed design methodology applies to different makes of plates, the actual values of the fatigue reduction factors will vary from one plate manufacturer to another and will depend on the level of safety required in the design.

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