2nd INTERNATIONAL CONFERENCE
TIMBERBRIDGES
LAS VEGAS 2013

2nd International Conference on Timber Bridges

Proudly hosted by:

USDA

FOREST SERVICE U.S. DEPARTMENT OF AGRICULTURE

NATIONAL CENTER FOR WOOD TRANSPORTATION STRUCTURES

WoodWorks

U.S. Department of Transportation
Federal Highway Administration

Thanks for your support!

Silver Sponsors

ALAMCO WOOD PRODUCTS, LLC. 1410 WEST 9TH STREET ALBERT LEA, MN 56007

ASCE AMERICAN SOCIETY OF CIVIL ENGINEERS

Canadian Wood Council / Conseil canadien du bois

Guardian Bridge RAPID CONSTRUCTION INC.

WESTERN WOOD STRUCTURES

Bronze Sponsors

ANTHONY ANTHONY FOREST PRODUCTS CO.

APA

FPInnovations

LCI LAMINATED CONCEPTS INC.

WCLB

WESTERN WOOD PRESERVERS INSTITUTE

WWPA

SWG MY-TI-CON Timber Connectors Inc. Made in Germany | Distributed in North America

NORDIC Building the future

RINNTECH

StructureCraft

1865 THE UNIVERSITY OF MAINE

2nd INTERNATIONAL CONFERENCE TIMBERBRIDGES LAS VEGAS 2013

2nd INTERNATIONAL CONFERENCE

TIMBERBRIDGES

LAS VEGAS 2013

September 30 – October 2, 2013 ● Las Vegas, Nevada

Edited by: James Wacker & Jaime Krohn

Copyright© 2013 WoodWorks – Wood Products Council, Washington DC

Electronic Proceedings Only

Sample Citation:

In: Wacker, J.; Krohn, J., eds. Proceedings, 2nd International Conference of Timber Bridges. Washington, DC: WoodWorks – Wood Products Council, 2013.

186 m footbridge in Kalmar, Sweden

Rune B Abrahamsen
Structural engineer
Sweco Norway
Lillehammer, Norway
rune@sweco.no

M.sc. in structural engineering from NTH Trondheim 1995. Worked for Sweco since 1998. Group leader for bridges in Sweco.
Involved in the design of 60 timber bridges since 1996.

Gunnar Veastad
Structural engineer
Sweco Norway
Lillehammer, Norway
gunnar.veastad@sweco.no

M.sc. in structural engineering from NTH Trondheim 1986. Worked for Sweco since 2004. Involved in the design of 30 timber bridges since 1996.

Summary

Hansabron is a 186 m long footbridge in Kalmar, Sweden. The bridge crosses the E22 motorway and was opened for traffic in May 2013. A clearance of only 4.7 m above the motorway complicates the design due to a vehicular impact load of 500 kN. Unusually much steel was needed to pre-stress the deck to handle the load. Special screws were used to reinforce the timber in areas were crossbeams run through the 600 mm thick deck. Large deck elements were prefabricated and lifted into position on site to minimize installation time. The bridge was subjected to extensive control by the Swedish Road Authorities. Some rules from the Norwegian Public Road Authorities' bridge handbook were allowed to better the design.

Keywords: timber deck, accidental loads, reinforcement screws, prefabrication, assembly

1. Introduction

1.1 Hansabron, Kalmar

Hansabron is a 186 m long and 4.5 m wide footbridge in Kalmar, Sweden. The bridge crosses the E22 motorway and connects the city's pedestrian and bicycle network with the Hansa City shopping area. It has separate lanes for pedestrians and bicyclists. The bridge also connects the new football arena with the city. Hansabron was opened for traffic on May 17th 2013.

In 2011 the municipality of Kalmar (Kalmar kommun) developed a bridge design proposal and held a competition to build the bridge. The Swedish construction company NCC won this competition.

The proposal showed a bridge with only 4.7 m clearance above the motorway and crossbeams that ran through the timber deck. This complicates the design of the bridge considerably, as will be

discussed in this paper.

The structure is divided into 10 spans with a main span of 50.4 m over the motorway between axis 9 and 10. There is an expansion joint in axis 7 which serves to separate the bridge's two different structural systems. All timber components are pressure treated with copper. Exposed members are protected with steel claddings.

- Between axes 1-7 there is a continuous stress-laminated T-beam timber deck. The largest span is 15.8 m. This part of the bridge was designed by Finnmap Consulting.
- Between axes 7-11 there is a continuous stress-laminated glulam deck which is supported by a cable-stayed steel structure. The glulam deck height varies from 600 to 642 mm. This part of the bridge was designed by Sweco. This paper is about this part.

Fig. 1 Hansabron looking east

Fig. 2 Hansabron looking north

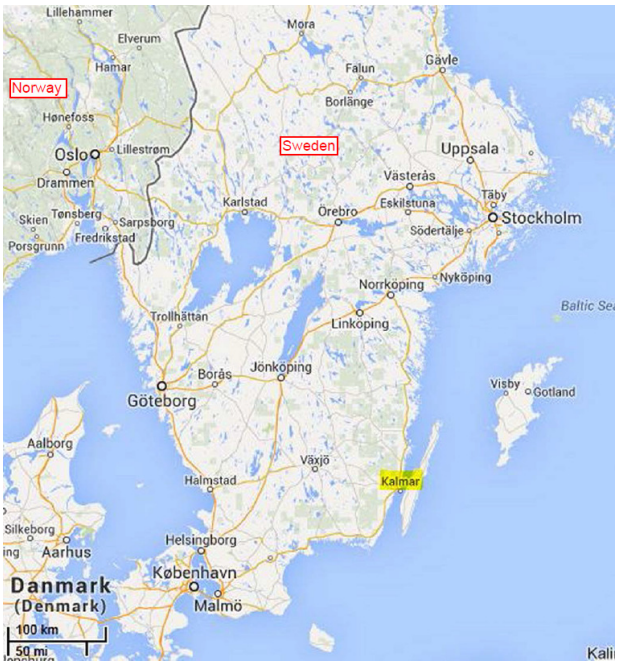


Fig. 3 Map where Kalmar is highlighted

1.2 Location

Hansabron is located in the southeast of Sweden, approximately 400 km south of Stockholm and 330 km east of Copenhagen. To locate the bridge follow motorway E22 outside of Kalmar and you cannot miss it!

1.3 Swecos assignment

NCC hired Sweco and Finnmap Consulting to do the bridge design. Finnmap was responsible for all foundations and concrete structures, as well as the stress-laminated T-bridge timber deck between axes 1-7. Sweco was responsible for the steel structures (Sweco Stockholm) and timber structures (Sweco Norway) between axes 7-11, as well as design management.

2. Accidental actions and pre-stressing of timber deck

2.1 Accidental actions

Kalmar kommun and Kalmar kommun's architect made the condition that the bridge deck should be placed as low as possible over the motorway. Consequently the bridge deck was designed with the lowest permissible clearance above the road, which was 4.7 m at the time. During the design period Swedish authorities published new regulations with increased clearance, but this was not to be used for this bridge. The design was done according to TK Bro 2009 [4]

Swedish bridges must be designed for accidental actions according to Eurocode 1991-1-7 [1]. The vehicular impact load is 500 kN. The E22 motorway presently has 2 lanes in each direction, but can be widened to 3 lanes in each direction. The impact load can hit the bridge deck within the shoulders of the future 2x3 lane configuration.

Horizontal forces on the bridge between axes 7-11 are transferred to the substructure through the pylons in axis 9 and 10, and the abutment in axis 11. The highest shear force was calculated to 414 kN. This force was decisive for the design of the pre-stressing of the deck.

2.2 Pre-stressing the timber deck

Chapter 6.1.2 in Eurocode 5-2 [3] specifies a method for design of stress-laminated deck plates. In addition, rules from the Norwegian Bridge Handbook no 185 [5] are used. We have utilized the following formula for assessment of longitudinal shear:

$F_{v,Ed} = V/(0{,}9 \text{ x deck width})$

When calculating the compressive stress due to pre-stressing the coefficient of friction between the lamellas plays an important role. The Eurocode has the following design values for μ_d:

Tabell 6.1 – Dimensioneringsvärden på friktionskoefficienten μ_d

Lamellernas ytråhet (endast barrträ)	Vinkelrätt fiberriktningen		Parallellt fiberriktningen	
	Fuktkvot ≤ 12 %	Fuktkvot ≥ 16 %	Fuktkvot ≤ 12 %	Fuktkvot ≥ 16 %
Sågat virke mot sågat virke	0,30	0,45	0,23	0,35
Hyvlat virke mot hyvlat virke	0,20	0,40	0,17	0,30
Sågat virke mot hyvlat virke	0,30	0,45	0,23	0,35
Trä mot betong	0,40	0,40	0,40	0,40

Fig. 4 Coefficient of friction from the Swedish Eurocode 1995-2

The Norwegian Bridge Handbook no 185 [5] states that the friction factor for glulam in bridge decks can be taken as μ_d=0.30 perpendicular to grain and μ_d=0.25 parallel to grain. The Swedish Road Authorities accepted that these friction coefficients could be used for the design of the bridge deck, since they are recommended by the Norwegian Public Road Authorities. The Norwegian Bridge Handbook also recommends a maximum 50% bar force loss for pre-stressed glulam decks with a sealing layer. This also was accepted for this bridge.

Using these values the calculated minimum initial pre-stressing force per bar was 409 kN. We chose to use 419 kN. This equals an initial compressive stress due to pre-stressing of 1.4 N/mm^2. Dywidag WR 26.5 bars spaced 500 mm were used to achieve this unusually high compression stress in the deck. If we would have had to use the Eurocode only, the whole side of the timber deck would have been covered with steel anchor plates.

Page 11 of European technical approval ETA-05/0123
Validity from 14.11.2011 to 18.09.2015, replaces
ETA-05/0123 with validity from 19.09.2010 to 18.09.2015

Member of EOTA

Designation	Nominal bar diameter	Nominal cross-sectional area	Maximum prestressing force 1)	Maximum overstressing force 1), 2)
—	d_s	S_n	$0.8 \cdot F_{pk}$	$0.95 \cdot F_{p0.1k}$
—	mm	mm^2	kN	kN
Threadbar				
18 WR	17.5	241	204	219
26 WR	26.5	552	464	499

Fig. 5 Capacity of Dywidag 26 WR pre-stressing bar

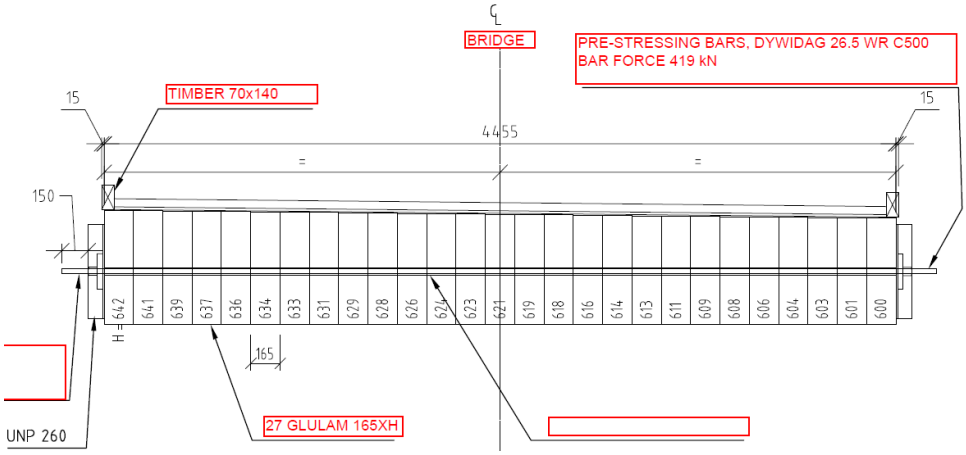


Fig. 6 Typical cross-section of bridge deck

The allowable compression stress perpendicular to grain at installation is 3.43 N/mm^2. Channel sections UNP 260 with a height of 530 mm were used to distribute the bar forces correctly to the deck. See fig. 8

3. Crossbeams through the deck

One of the main challenges for this bridge was to design the deck according to Kalmar kommun's bridge proposal. The deck was not allowed to rest on underlying crossbeams; hence the beams were forced to run through the deck.

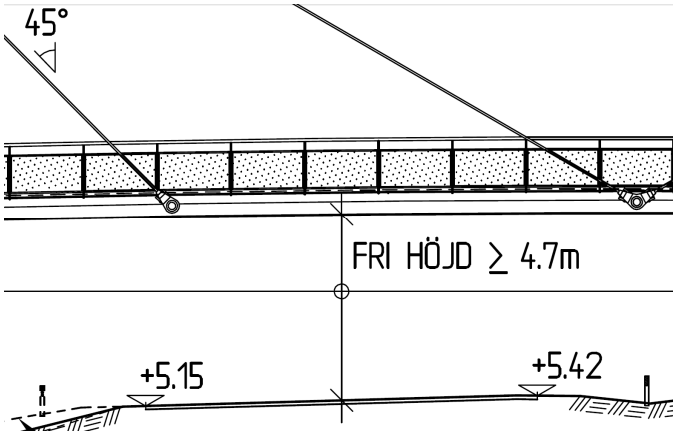


Fig. 7 Cut-out from the proposal drawing showing crossbeams through the deck and clearance of 4.7 m.

Initially we tried to place the crossbeam in a recess in the lower part of the deck, but this was discarded due to dramatically reduced longitudinal stiffness. There was a need for rigid crossbeams in relation to the available cross-sectional height. We selected to use a beam welded from 40 mm steel plates. We positioned the beam eccentrically in the vertical direction in order to have sufficient support for the glulam beams.

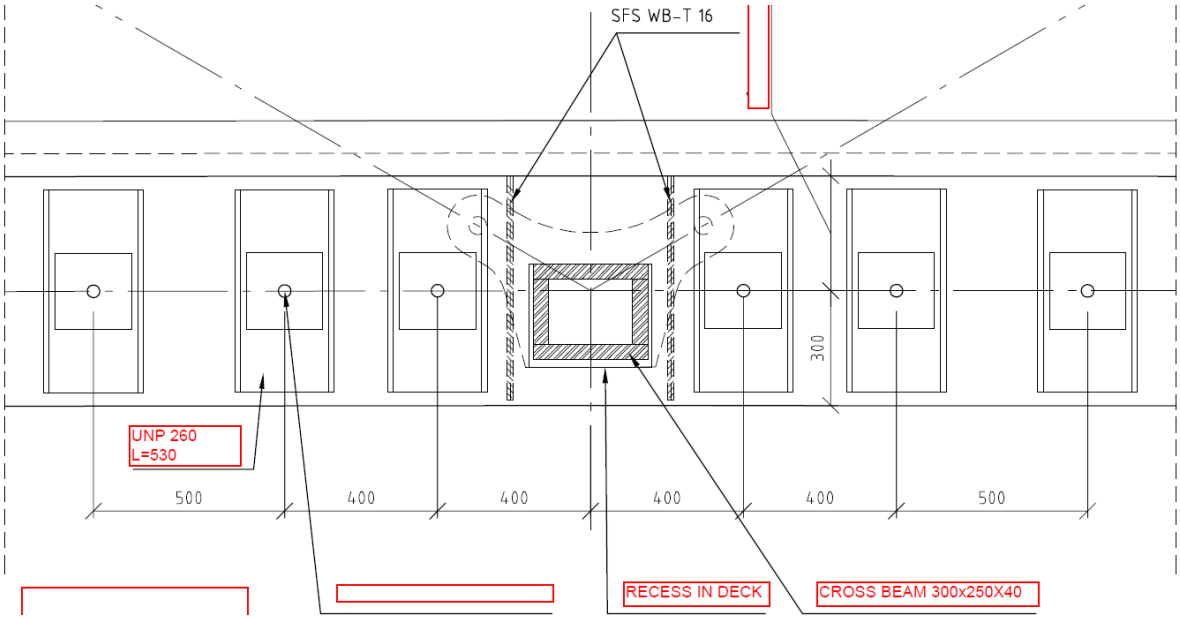


Fig. 8 Detail of the deck in the area around a crossbeam

The bridge between axes 7-11 is modeled using FEM-Design 3D Structure [7]. Using this program we could assess the stiffness of the crossbeam in relation to the deck stiffness in the transverse direction. Since the beam is relatively soft compared to the deck, the forces are concentrated around the deck edges, as shown in Fig. 9.

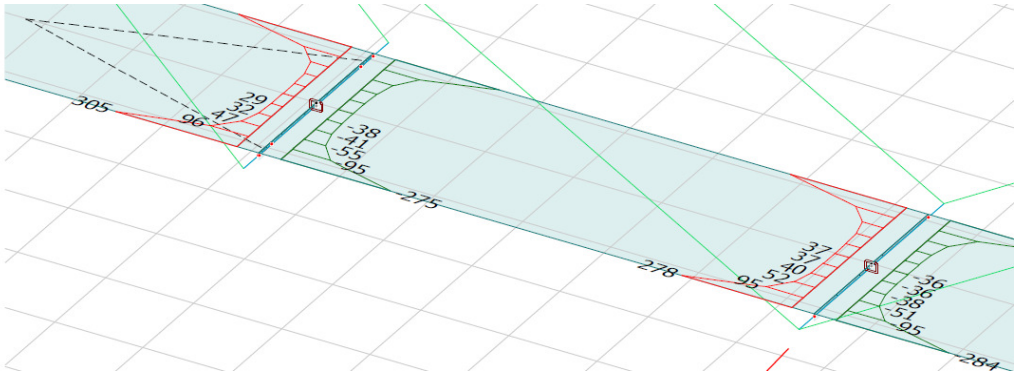


Fig. 9 FEM-Design plot of vertical shear force in the deck

It is well known that beams with a notch at the support have reduced capacity, ref Eurocode 5 chapter 6.5.2 [2]. However, the kind of support we have in this case is not handled by the Eurocode. We had to seek other sources to verify the solution, and chose to use SFS Intec WB threaded rod [6] for this purpose. These screws are approved according to the German standard DIN 1052.

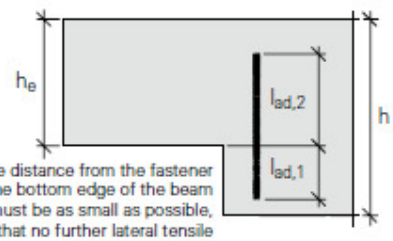
Proposal of design according to DIN 1052: **2004-08**

The reinforcements must be calculated for the following tensile force $F_{t,90,d}$

$$F_{t,90,d} = 1.3 \cdot V_d \cdot [3 \cdot (1-\alpha)^2 - 2 \cdot (1-\alpha)^3]$$

$$\text{mit } \alpha = \frac{h_e}{h}$$

V_d design value of transverse force



The distance from the fastener to the bottom edge of the beam must be as small as possible, so that no further lateral tensile proof is necessary.

$$\frac{F_{t,90,d}}{n \cdot R_{ax,d}} \leq 1$$

n number of fasteners (only one fastener is permitted in the longitudinal direction of the beam)

$R_{ax,d}$ design value of the load-bearing capacity of a fastener

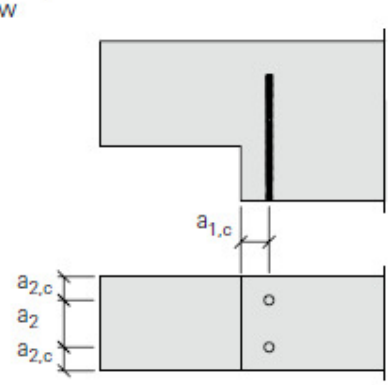
Values $R_{ax,d}$ can be found in the table below

$l_{ef} = \min [l_{ad,1} ; l_{ad,2}]$

Arrangement

Minimum spacing

	WB-T-16	WB-T-20
a_2	48 mm	60 mm
$a_{1,c}$	40 mm	50 mm
$a_{2,c}$	40 mm	50 mm



Please note that in the case of notching only one rod may be allowed in the longitudinal direction of the beam.

Fig. 10 SFS guidelines for using WB-T timber screws as reinforcement

System WB gemäss Z-9.1-777

			Rohdichte ρ_k [kg/m³]	350	380	410	430	450
WB-T-16xL [mm]	l_{ef} [mm]	100	Auszieh-widerstand aus Holz $R_{1,k}$ [kN]	13,7	16,2	18,8	20,7	22,7
		200		27,4	32,3	37,7	41,4	45,4
		300		41,2	48,5	56,5	62,1	68,0
		400		54,9	64,7	75,3	82,8	90,7
		500		68,6	80,9	94,1	103,5	113,4
		600		82,3	97,0	113,0	124,3	136,1
		700		96,0	113,2	131,8	145,0	158,8
		800		109,8	129,4	150,6	165,7	181,4
	Zugbruch Befestiger $R_{2,k}$ [kN]			91,5				

Fig. 11 Characteristic capacity of SFS WB-T screws

By inserting 16 mm SFS WB-T screws on both sides of the opening in every 165 mm glulam lamination, as shown in Fig. 8, we achieved a vertical shear capacity of 207 kN/m.

Fig. 12 Picture of crossbeam and bar anchor plates

4. Assembly procedure

The bridge crosses the heavily trafficked E22 motorway. A lot of effort was put into doing the assembly as quick and hassle free as possible. NCC, Sweco and the Finnish glulam manufacturer Versowood chose to emphasize on a high degree of prefabrication. Versowood preferred to split the 94 m long bridge deck into three large prefabricated elements. The longest element was 36 m.

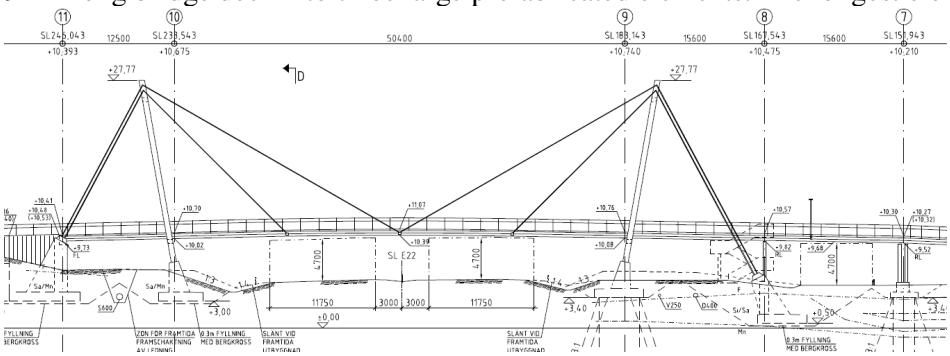


Fig. 13 Elevation of bridge between axes 7-11

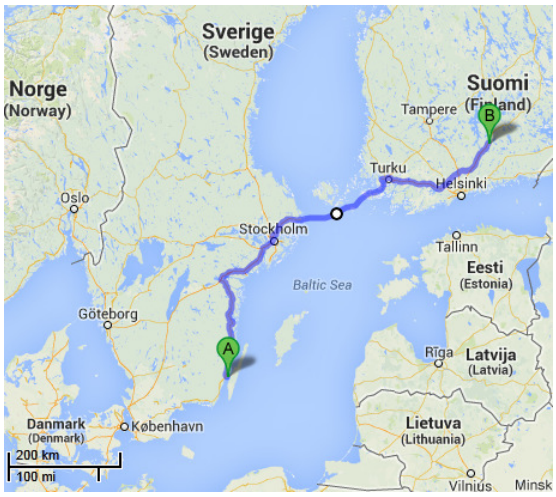


Fig. 14 (left) Transportation of bridge deck elements. The elements were manufactured at Versowood's factory in Vierumäki, Finland and transported about 750 kilometers by truck.

Fig. 15 (right) The crossbeams were pulled through the deck on site before the large elements were lifted into position by two large mobile cranes.

Fig. 16 The prefabricated elements are joined in a 3,0 m long connection zone. Not more than one butt joint occurs in any four adjacent laminations

Fig. 17 A detailed procedure was developed for the installation of pylons, deck and stays.

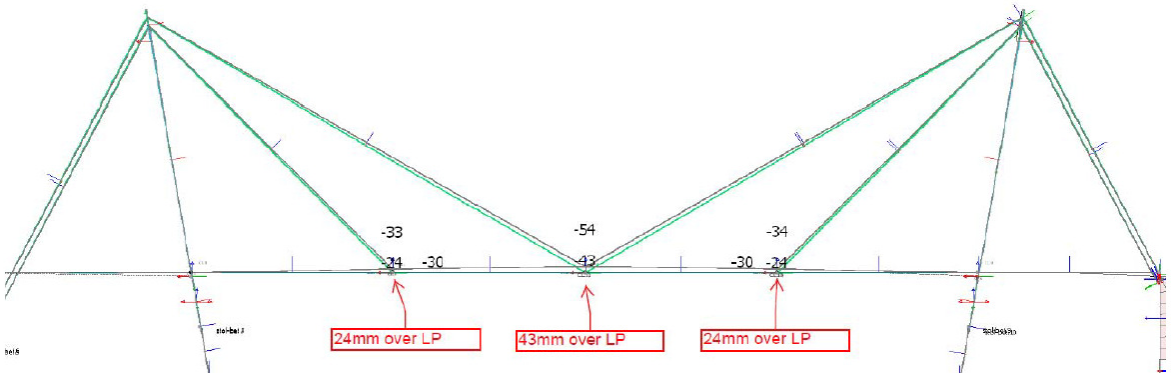


Fig. 18 The stays had to be adjusted upwards prior to adding the asphalt wearing course, to compensate for the vertical deflections

Fig. 19 After installation the bridge deck's sides were covered with thin steel plates

5. Conclusions

Swedish bridges must follow Eurocode when designing stress-laminated timber decks. In this project Swedish Road Authorities allowed us to use alternative values for friction values and bar loss. These are values that are well documented in Norway and Sweden. Future timber bridges in Sweden should be given the opportunity to use these values in order to increase the cost competitiveness of timber bridges.

Unusually much steel was needed to pre-stress the deck for Hansabron. This was a result of the impact load from vehicles. In general considerable amounts of timber and steel can be saved if one ensures that the bridge deck has sufficient clearance above an underlying road. Calculations we have done show that by lifting the bridge to avoid the impact load, and positioning the crossbeams under the deck, the deck thickness of Hansabron could have been reduced from 600 mm to 450 mm.

Letting the crossbeams run through the deck complicates design and installation significantly as well as increases the costs. This is hardly a smart move to make timber bridges cost competitive with other bridges. On the positive side crossbeams through the deck give the bridge a somewhat smoother look, and improve the aesthetics. Kalmar kommun have gotten a nice looking bridge that can be used for many generations to come. Congratulations!

6. Acknowledgements

The authors would like to thank all the people and companies that took part in the design process of Hansabron, and in particular...

NCC Construction Sverige AB. *Gustaf Dahlgren, Per Karlsson.*

Sweco Sweden. *Håkan Lundqvist. Frans Cleveson. Kerstin Andersson*

Sweco Norway. *Magne Bjertnæs. Trond Egil Nyløkken. Endre Vangskåsen.*

Versowood. *Esa Hynninen. Harri Mäenpää.*

Finnmap Consulting. *Pertti Kaista*

The municipality of Kalmar. Kalmar kommun. *Anders Berg*

Swedish Transport Administration. *Karl-Magnus Krona*

7. References

[1] Eurocode 1991-1-7. Eurocode 1: Actions on structures – Part 1-7: General actions – accidental actions. 2006

[2] Eurocode 1995-1. Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings. 2004

[3] Eurocode 1995-2. Eurocode 5: Design of timber structures– Part 2: Bridges. 2004

[4] Swedish Transport Administration. *TK Bro. Design rules for bridges.* Version 2009-07-01.

[5] Norwegian Public Roads Administration. *Handbook 185. Design rules for bridges.* Version 2011-11.

[6] SFS Intec. *SFS WT, WR, WB. Verstärkung ausgeklinkte Träger. Datenblatt Nr. 06.* 2008

[7] Strusoft FEM-design version 11.0

Chambers Covered Railroad Bridge Salvage and Rehabilitation

Gregory W. Ausland, PE
Senior Project Manager
OBEC Consulting Engrs
Eugene, Oregon, USA
gausland@obec.com

Greg has more than 27 years of civil/structural design and project management experience, and since 1989 has had the distinctive experience of being the designer and/or project manager on rehabilitation and repair projects for 32 of Oregon's 50 covered bridges.

Tony LaMorticella,PE, SE
Sr. Project Engineer
OBEC Consulting Engrs
Eugene, Oregon, USA
tlamorticella@obec.com

Inspired by the graceful bridges on the Oregon Coast, Tony became an engineer after building custom furniture for 15 years. He's been engaged in structural design for 20 years, and has been lead design engineer on 18 covered bridge rehabilitation projects.

Summary

The Chambers Covered Bridge was built in 1925 and is the last remaining covered railroad bridge in Oregon, possibly the only one west of the Mississippi. Therefore, its rehabilitation was critical to historic preservation efforts of the country's remaining covered rail bridges. Before rehabilitation, the 27.4 m (90-foot) long timber structure was in danger of collapse after decades of neglect. In 2010 a windstorm threatened to destroy the bridge. This unique, single-span, four-leaf Howe truss structure was dismantled and rebuilt using almost all the original iron and hardware, and 25 percent of the original timber. The bridge was reconstructed on dry ground and launched onto the existing concrete piers. The rehabilitated Chambers Covered Bridge now serves as a landmark pedestrian and bicycle crossing that provides safe access across the Coast Fork of the Willamette River. Locomotive-inspired artwork within and around the structure evoke the bridge's history.

Keywords: Howe Truss, railroad, covered bridge, rehabilitation, new use, pedestrian bridge

1. Introduction

1.1 Historical Background and Context

J.H. Chambers built the Chambers Covered Bridge as part of the development of his new sawmill in Cottage Grove, Oregon, in 1925. The bridge carried a railroad spur from timber land west of the city to the Chambers mill. In August 1942 the sawmill burned to the ground, and J.H. Chambers passed away before he could finish rebuilding it. His son sold the mill to Warren Daugherty, who reorganized it under the name Lorane Valley Lumber Company. Six years later, the mill burned down again, and Daugherty sold the railroad line to a scrap firm in Seattle, at which time the tracks were removed. No maintenance was performed after 1951, and the bridge

Fig. 1 J.H. Chambers Sawmill, ca. 1928

Fig. 2 Bridge, 1975

remained abandoned at the edge of the old mill site, suffering the ravages of neglect, weather, and vandalism.

The City of Cottage Grove obtained ownership of the bridge in 2006 and hired OBEC Consulting Engineers (OBEC) to inspect, inventory, and document the structure; and to assist in the grant application process. Funding was obtained in 2009, and OBEC was hired to design and oversee the rehabilitation into a pedestrian and bicycle bridge. Then, after sixty years of unchecked deterioration and with the design process underway, a strong wind during a winter storm in early 2010 brought the Chambers Covered Bridge to near collapse. This made it even more crucial to quickly execute the safe emergency removal of the derelict structure. Before disassembly began, a model was produced from a laser scan as final documentation.

1.2 Design

A variety of alignment and structure type alternatives for the bridge approaches were explored, as well as options for lighting and interpretive displays. Structural modeling was done with finite element analysis to optimize the design of the approaches and to establish the actual capacity of the rehabilitated timber truss.

The design of the approaches had to consider impacts to the floodplain and be sensitive to the historic context while providing a safe and aesthetically pleasing structure. The west approach also had to fit within the tight space between the river bank and River Road. It now has a 26.8 m (88-foot) long ramp providing Americans with Disabilities Act (ADA) compliant access for wheelchairs as well as bicycles, and a gracefully curved concrete stairway for pedestrians. The concrete ramp has a very thin profile and is elevated above the ground to reduce floodplain impact. The east approach is a short, level, earth-filled ramp.

2. Physical Description of Bridge

The covered bridge spans 27.4 m between concrete piers and is a tall, massive structure originally designed to carry loaded freight trains pulled by steam locomotives. It is the only covered bridge in Oregon with four-leaf trusses. The Howe trusses are approximately 7.6 m (25 feet) tall with 3.8 m panels. The floor beams are on top of the bottom chords, and originally there were six floor beams in each panel spaced less than 600 mm on center. On the floor beams are six lines of evenly spaced stringers. Originally there was no deck; instead, railroad ties were attached to the stringers, which were grouped closely under each track, and tracks to the ties. All of the wood in the bridge is local Douglas fir, and the original material was all very high quality from old-growth trees.

Fig. 3 Bridge in 2008

Where diagonal timber truss members intersect with chords, forces are transmitted through massive cast-iron hardware that is carefully let into the timber members with precisely cut dados, efficiently transferring forces while removing the smallest possible amount of wood. Similarly, splices in the bottom chord are joined using elaborate but efficient cast- and wrought-iron hardware that requires minimal disruption of wood fiber. Each truss has three hanger rods at each panel point, varying in diameter from 50 mm at mid-span to 60 mm at the ends. The ends of the rods have forge-rolled upset threads that are larger than

the rod so that the root diameter of the thread is the same as the body diameter of the rod, an extremely material-efficient design which is no longer used.

The house siding along the sides of the bridge is 8.5 m tall and was never painted; it has no windows. The portal openings are 4.9 m wide and 6.7 m tall measured from the elevation of the tracks (now measured from the deck). The roof has always been corrugated metal. The metal roofing is light and relatively inexpensive, and afforded some resistance to fire that could have been ignited by the shower of sparks spewing from the stacks of steam locomotives.

3. Chronology of Development and Use

As mentioned above, the bridge carried trains delivering logs to and lumber from the sawmill. It served this function for approximately twenty-five years until the mill burned and the rail line was abandoned.

Fig. 4 Chambers Bridge before rehabilitation

At the bridge location, the east bank of the river, where the mill once stood, is approximately 1.8 m higher than the west bank. As long as the bridge has existed there has been a frontage road, River Road, running parallel to the west bank. Originally the rail line was carried by a trestle over the road to the bridge, and the road was cut deeper to provide clearance. Since the river's floodplain extends beyond River Road, this depressed section of road was flooded for much of the year. Eventually the road was built up to create an at-grade crossing, but when the rail line was abandoned and the tracks were removed, the road was graded level, and the west end of the covered span was left dangling 1.8 m above ground.

Before its rehabilitation, the bridge site was an environmental hazard. From 1925 to 1951, Chambers Covered Bridge provided the railroad crossing for the J.H. Chambers sawmill in Cottage Grove. Throughout those years toxins associated with the operation of the mill had accumulated in soil around the east approach to the bridge. This was removed as part of the rehabilitation project.

Fig.5 Bridge deterioration

For over sixty years the bridge was neglected and attracted vandalism. Most of the siding was kicked off, and local youths would jump from the trusses into the river. Twice fires were built in the abandoned structure, but luckily it did not fall. Large sections of roofing were missing, so the structure was no longer protected from weather. The 2006 inspection revealed that at one location near mid-span of the bottom chord, the combination of decay and vandalism left only 7 percent of the original material intact. It is a testament to the original robustness of the structure that it continued to stand after such aggressive deterioration.

4. Description of Most Recent Rehabilitation Project

4.1 Emergency Removal

Fig. 6 Bridge being dismantled

Removal of the existing bridge was an emergency that could not wait for the next in-water work period. Furthermore, the bridge spans the Coast Fork of the Willamette River, an environmentally sensitive stream that is very popular with boaters and anglers. Therefore, no supports could be placed in the active channel during removal. The design involved construction of steel tracks attached to the existing piers and a rolling platform that spanned the river. The platform was built adjacent to the bridge then rolled under it, where it was anchored in place and used to stabilize the existing structure and to provide a work surface. Removal was very systematic, and complex hardware assemblies were kept together and in sequence to facilitate future reassembly.

4.2 Retaining Original Features

Almost all of the original hardware, including the hanger rods with upset threads, was cleaned by shot-blasting, then painted and reinstalled. Unfortunately, the hanger rod nuts had to be cut off; however, the machine shop in Cottage Grove that had fabricated the original nuts is still in business, and they were able to make replacements identical to the originals. Though most of the timber was in an advanced state of decay, a portable sawmill was set up on site, and all members that contained sound wood were resawn into smaller but usable sizes. Approximately 25 percent of the wood in the rehabilitated bridge was cut from old bridge timbers.

Fig. 7 Rehabilitated bridge being moved into position

Every member of the rehabilitated covered bridge is identical to the original in size, shape, number, orientation, and configuration except there are now only two floor beams per panel rather than the original six, the stringers are spaced evenly, and the ties and tracks were replaced with timber decking and bridge rail. When the bridge carried trains there were floor beams every 600 mm to control deflection. However, under pedestrian loading the stringers can easily span 3.5 m, and since there is now a solid deck they are visible only from the river. Installing more floor beams than were needed would have been an unnecessary added expense. Because the new function of the bridge is to serve pedestrians, the use of pressure-treated wood was deemed inappropriate except on the corbels that are in direct contact with the concrete piers.

4.3 Rehabilitation and New Features

All timber members and the siding were coated with a clear sealant that provides the appearance of the original bare wood but affords some protection from moisture and fire. The new roofing is corrugated metal like the original but with a continuous strip of translucent skylight along the ridge.

The skylight is not visible from outside the covered bridge but washes the interior with natural light.

Before rehabilitation began, a set of steel tracks was erected spanning the river from the east bank to the west pier on deep wide-flange beams. The bridge was erected on these tracks over dry land and rolled into position after completion, then lifted with hydraulic jacks and lowered onto the piers. Finally, the tracks and steel beams were removed, and the approach spans completed. A video of the launching can be found at http://www.obec.com/bridge/chambers-covered-bridge.html.

Fig. 8 Bridge lighting at night

High-efficiency electric lights were added, discretely concealed above the top chords of the trusses and controlled by exterior light sensors. They are aimed upward and reflect off the underside of the metal roofing, providing pleasant, indirect interior lighting after dark. There are also LED strip lights tucked under the eaves that softly wash the sides of the covered bridge, making it visible at night to passers-by.

To evoke images of the railroad, alternate deck planks are stained to resemble ties, and two continuous steel strips are imbedded into the wood spaced at standard track gauge. Artists from Sea Reach, Ltd. created two full-scale sculptures of one of the locomotives that used to traverse the bridge. One of the sculptures is on an inside wall and one stands outside on a short section of train tracks that are aligned with the track pattern on the bridge deck. Informational placards are placed on the handrail throughout the length of the covered bridge.

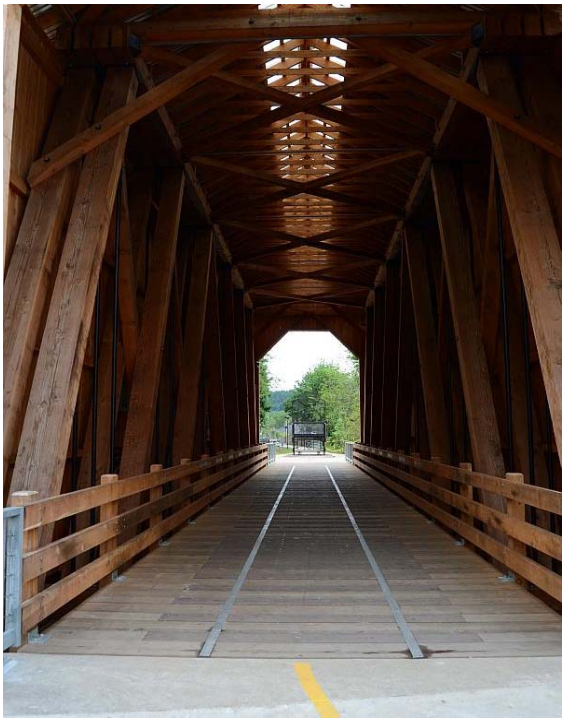


Fig. 9 Interior with "tracks"

Although most residents in Cottage Grove do not remember trains actually using the bridge, they can appreciate the historical depth Chambers Covered Bridge brings to their community. Part of the rehabilitation effort included creation of an interpretive center displaying pictures and descriptions of the bridge and its history. This interactive feature provides an opportunity for residents of Cottage Grove as well as visitors to learn the story of this magnificent bridge.

4.4 Awards

Since rehabilitation, the bridge has won numerous awards and received international attention. The bridge rehabilitation has garnered an ACEC Oregon 2013 Grand Award, 2012 Oregon Heritage Excellence Award, and 2012 First Place Renovation Award for Oregon top projects. That attention is bringing increased tourism to Cottage Grove.

5. Conclusion

When rehabilitation work began, the bridge was near collapse. The fact that such a severely deteriorated structure could be completely rehabilitated and repurposed within the awarded budget was a noteworthy achievement made possible by the innovative approach taken to both the

emergency removal and the repair of the bridge, and by the amount of original material that was salvaged and reused.

The dire condition of the bridge made it necessary to replace a good deal of the existing structure. Careful documentation of existing conditions provided accurate detail on the original design prior to deconstruction and was critical to the accuracy of the subsequent rehabilitation effort. Damaged material was, where possible, re-milled into smaller dimension elements, maintaining character and historic fabric,and retaining integrity. Any material too damaged to repair was replaced in kind. Changes in use that required modification from the original construction, such as the installation of a continuous deck and rail, were designed with respect to the bridge's historic function. Translucent roof-top panels that were installed to provide natural illumination, also a safety requirement, were integrated into the historically-based metal roofing and held back from the eave and gable ends, to reduce visual impact from virtually all external viewpoints.

The bridge now provides safe passage for pedestrians and bicyclists, connecting the local high school with the rest of Cottage Grove and providing convenient access to a city park currently under development. It has become a venue for local gatherings and weddings. In addition, as the only remaining railroad covered bridge in Oregon and perhaps the only one left west of the Mississippi River, it is a tourist destination and, as such, a boost to the local economy.

Fig. 10 Bridge entrance after rehabilitation

5.1 Rehabilitation Project Team

OBEC Consulting Engineers – Prime Consultant
Hamilton Construction Company – Contractor for emergency removal
Heritage Research Associates, Inc. – Cultural and Historical Resources
Sea Reach, Ltd. – Interpretive Exhibits Design and Fabrication
Cascade Earth Sciences, Ltd. – Hazardous Materials
Foundation Engineering, Inc. – Geotechnical
Pacific Habitat Services, Inc. – Environmental
Wildish Construction Co. – Contractor for reconstruction

Goodpasture Covered Bridge Rehabilitation

Gregory W. Ausland, PE
Senior Project Manager
OBEC Consulting Engrs
Eugene, Oregon, USA
gausland@obec.com

Greg has more than 27 years of civil/structural design and project management experience, and since 1989 has had the distinctive experience of being the designer and/or project manager on rehabilitation and repair projects for 32 of Oregon's 50 covered bridges.

Tony LaMorticella, PE, SE
Senior Project Engineer
OBEC Consulting Engrs
Eugene, Oregon, USA
tlamorticella@obec.com

Inspired by the graceful bridges on the Oregon Coast, Tony became an engineer after building custom furniture for 15 years. He's been engaged in structural design for 20 years, and has been lead design engineer on 18 covered bridge rehabilitation projects.

Summary

The Goodpasture Covered Bridge, crossing the McKenzie River, is a 72.5 m (238-foot) five-span timber structure that provides lifeline access to a large community. The main span is a 50.3 m (165-foot) covered heavy timber Howe truss. The truss chords have three leaves, each made of segments spliced together. Eight of the sixteen bottom chord splices were broken, and the truss sagged 100 mm at mid-span. Site constraints dictated that all repair work be performed under traffic and without temporary support beneath the bridge. The solution was a post-tensioning system for the bottom chords to provide an alternate load path. To lift the center of the bridge to the proper camber without support from below, a pair of tube-steel trusses were constructed inside the house to bear the loads. The bridge was lifted 200 mm at mid-span to achieve 100 mm of positive camber, and held by the steel trusses while the hanger rods were tightened and post-tensioning system installed.

Keywords: Covered bridge, rehabilitation, Howe truss, post-tensioning, repair under traffic

1. Historical Background and Context

The Goodpasture Covered Bridge was constructed by Lane County at its present location under the supervision of veteran bridge builder A.C. Striker in 1938. The main span is a housed eleven-panel, three-leaf Howe truss. It was built using a standard design developed by the Oregon State Highway Department, which became the Oregon Department of Transportation in 1969. The total cost for the original construction of the bridge was $13,155.

In 1925 there were 450 covered bridges in use in Oregon. By 2003, replacement, removal and destruction had reduced that number to 51. Among the survivors, Goodpasture is the longest covered bridge in Oregon still open to vehicular traffic on an active roadway, and the second-longest covered bridge in the state. It is located just south of Highway 126, making the bridge highly visible along this popular east-west travel route.

2. Physical Description of the Bridge

The Goodpasture Covered Bridge is a five-span timber structure crossing the McKenzie River in eastern Lane County, Oregon. The river is one of America's most pristine waterways. It is home to many protected species of aquatic life and is very popular with boaters and anglers. The bridge is a lifeline link providing the only access to a neighborhood of approximately 300 residents. A short approach span from Oregon Highway 126, one of the major east-west state highways, leads to the main span, which is a 50.3 m-long covered Howe truss followed by three simple timber approach spans. The total length is 72.5 m. It is a single-lane bridge that has served to carry logging, recreational, and local traffic. For several years it has been weight-restricted because of structural distress. It still carries approximately 750 vehicles per day, of which approximately 75 are trucks.

Fig. 1 Goodpasture Covered Bridge

Each of the inner and outer leaves of the bottom chords of both trusses are four-piece members. Each of the middle leaves is made from three pieces. Of the sixteen resulting bottom chord splices, eight are broken and were repaired with heavy steel tie-rod and plate assemblies lag-screwed to the timber chord members, first in the 1970s and again in the 1980s. The repairs were only marginally effective. When OBEC Consulting Engineers inspected the bridge in 2010 there were gaps between the ends of timber members at all of the repaired splices, some over 13 mm wide, and the bridge sagged over 100 mm at mid-span.

3. Chronology of Development and Use

The original design was for H-10 loading. However, for many years and especially from the post-war period until the mid-1980s, logging in Oregon was very heavy. For a few years in the late 1970s there was more timber passing through nearby Eugene, Oregon, than through any other city in the United States. A fair amount of it was coming from the McKenzie River valley, and much of that was crossing the Goodpasture Covered Bridge. Traffic across the bridge included many log trucks and other heavy-haul vehicles weighing as much as 360 kN (80,000 pounds) each, four times the design capacity of the structure. Exactly when the bottom chord splices of the bridge trusses began to break is unknown, but the first attempt to repair them was made in the 1970s. Other repairs in the 1980s were part of a major rehabilitation. Yet as recently as 1998 overloads as big as 787 kN, almost nine times the design capacity, were being approved. By 2010 the overall condition had again deteriorated to the point that the state inspection report from that year gave it an overall sufficiency rating of 49 out of a possible 100. That same year the existing heavy composite roofing was replaced with a much lighter cedar shingle roof, allowing the posted weight restriction of 133 kN to remain in effect.

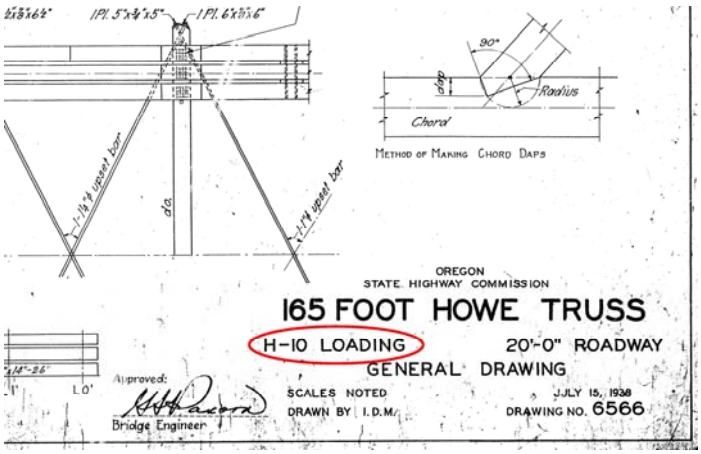


Fig. 2 Oregon Standard Drawing, Howe Truss, 1938

Because of site constraints and concerns of local residents, an alternate crossing was not possible even as a temporary detour. Consequently, all work on the

bridge had to be performed under traffic with only short-term night closures. To further complicate matters, the in-water work period recommended by the Oregon Department of Fish and Wildlife is July 1 through August 15. In other words, all work in the active channel had to be complete within a period of only six weeks. In this location the river bottom is rock and over 10 m below the main span. The water here is swift, carrying debris that sometimes includes large trees, and the water level can rise several feet in a few days of heavy rain. Therefore, even if a work bridge or temporary support could be erected during the in-water work period, leaving it in place through a rainy season would be extremely hazardous.

4. Description of Most Recent Rehabilitation Project

4.1 Retuning

Although the bridge was originally designed to carry only an 89 kN live load, it was sufficiently robust that only the bottom chords proved inadequate to carry legal loads. By strengthening the bottom chords, weight restrictions could be removed. However, replacing chord members would not be possible without closing the bridge for an extended period of time, and because this is a lifeline bridge with no serviceable detour route, a long-term closure was not a viable option. Therefore, an alternate load path had to be provided to relieve the bottom chords; the method selected was to post-tension with high-strength steel strand. However, the geometry of the truss had to be corrected before any compression could be applied to the bottom chords or the "repair" would have magnified the sag. The challenge then, in light of access limitations outlined above, was to lift the center of the bridge without any support from below while keeping the bridge open to traffic.

This was accomplished by constructing a pair of tube-steel trusses that fit inside the covered bridge and had sufficient capacity to bear the entire weight of the covered bridge and live loads up to 133 kN. To make room for the steel trusses, 600 mm were cut from each edge of the bridge decking, the bridge rail was removed, and temporary guardrail was installed. The temporary steel trusses bore directly on the concrete piers, narrowing the roadway from 4.9 m to 3.7 m during this phase of construction. This narrowing of the roadway was not a major inconvenience to users as the bridge had always functioned as single-lane; however, this did require additional work at the bridge rail to guardrail transitions.

Fig. 3 Jack on temporary steel truss

Each steel truss was manufactured in three pieces and assembled inside the timber bridge. Then a series of lifting platforms was installed below the floor beams of the timber bridge suspended by steel rods from hydraulic jacks placed on the steel trusses. The timber bridge was lifted 200 mm at mid-span to achieve 100 mm of positive camber. While held in the correct shape by the steel trusses, the vertical hanger rods of the timber bridge were tightened, and the post-tensioning system consisting of six 13 mm-diameter Grade 1200 strands on each side of each bottom chord was installed. Each strand was jacked to 89 kN, yielding a total compressive force of 1068 kN on each bottom chord. This reduces tensile stresses in the bottom chords enough to enable the covered bridge to safely carry legal loads. Then the tube steel trusses were disassembled and removed.

Finally, a new deck was installed, damaged siding was replaced, and the entire exterior of the bridge was repainted.

4.2 Interpretive Display and Lighting

To complete the project, bridge lighting was installed, and an interpretive display devoted to the history of the bridge was added at the top of an existing stairway leading to the river bank at the southeast corner of the bridge.

Fig. 4 Interpretive display

Goodpasture Covered Bridge is located on a blind curve of a heavily-travelled road passing through a dense Douglas fir forest, which is very dark at night. The bridge was difficult to see at night, and there is no turn lane and no shoulder for west-bound traffic. Multiple accidents have resulted from vehicles slowing suddenly to make the turn. Lighting was added to make the bridge more visible at night and to improve safety. High-efficiency LED lights were installed in the windows, under the eaves and behind the barge rafters at the portal ends, activated by an external light sensor and discretely illuminating the ends and side walls from concealed fixtures.

During the past several years local residents have established a tradition of decorating the bridge for the winter holidays. Large wreaths are hung over the portals and colored lights were placed in the windows making them alternately red and green. The bridge owner (Lane County) had reservations about allowing the public to install electric lights on an old wooden structure. To satisfy residents and simultaneously relieve the County of some liability, the window lights were programmed so they can be changed from white to alternate red and green by the flip of a switch located in a locked panel concealed behind a hidden door in the wrap-around siding. The residents can still hang their wreaths and are provided access to the switch at the beginning of the holiday season. The installation is both safe and efficient.

Fig. 5 LED lighting on bridge

Fig. 6 Portal of bridge at night

5. Conclusion

Maintaining the function of the Goodpasture Bridge during the rehabilitation was a key element of the project. The design solution of installing temporary tube-steel trusses made it possible to raise the existing truss elements prior to the installation of the post-tensioning system. This allowed the bridge to remain in use and did not require support from the ground beneath the bridge. All modifications and repairs to the bridge were designed in compliance with the US Secretary of the Interior's Standards for Rehabilitation, preserving the key design characteristics and replacing deteriorated elements in-kind.

Structural augmentation, the post-tensioning system, which allows the bridge to meet required load capacity, was located on the sides of the lower chords below the level of the deck and has minimal visual impact from most accessible viewpoints. The installation of lighting, improving the function of the bridge and reducing the potential for future damage, was undertaken to have minimal visual impact during daylight hours and is entirely removable.

The setting of the Goodpasture Covered Bridge is particularly dramatic and picturesque. It is said to be the most photographed covered bridge in Oregon and is a popular tourist destination. It continues to be a vital resource, functionally, historically and aesthetically.

5.1 Rehabilitation Project Team

OBEC Consulting Engineers – Prime Consultant

DKS Associates – Traffic Study

Heritage Research Associates, Inc. (HRA) – Cultural and Historical Resources

Sea Reach Ltd. – Interpretive Exhibits Design and Fabrication

Balzhiser Hubbard Engineers, Inc. – Lighting Design

Lois Cohen Associates – Public Involvement

David Place Consulting – Construction cost estimating and value engineering

Wood-Concrete Composite Beams under Low-to-High Cycle Loading

Jeno Balogh
Associate Professor
Metropolitan State University of Denver
Denver, USA
jbalogh@msudenver.edu

Massimo Fragiacomo
Associate Professor
University of Sassari
Sassari, Italy
fragiacomo@uniss.it

Richard Gutkowski
Emeritus Professor
Colorado State University
Fort Collins, USA
r_gutkowski@yahoo.com

Rebecca Atadero
Assistant Professor
Colorado State University
Fort Collins, USA
rebecca.atadero@colostate.edu

Peter Ivanyi
Professor
University of Pecs
Pecs, Hungary
peteri@morpheus.pte.hu

Summary

The wood-concrete composite beams investigated in this paper consist of a wood layer overlapped by a concrete layer and interconnected by notched shear-key connections to prevent the relative slip. When this type of structural system is applied to bridges with high traffic it is essential to know their fatigue behavior. This paper presents the result of a joint research at Colorado State University and Metropolitan State University of Denver, attempting to establish the S-N curve for fatigue conditions of notched wood-concrete connections based on low/high-cycle, repeated loading tests. Experimental results are reported on fourteen 1524 mm span composite beam specimens of 191 mm width, with a 89 mm thick wood layer consisting of five "2x4" boards and a 63 mm concrete layer interconnected by embedded anchor screws at the notch locations. Five specimens were only statically loaded while the others were cycled to failure with a maximum to minimum pulsating load ratio of 10. Points on the S-N curve were determined for three levels of the maximum load as a function of the average static failure load. Typical observed failure modes were block-shear of the wood at the notch and tension failure of the wood at mid-span. The obtained S-N curve is compared to the respective provisions of Eurocode 5 for timber beams and with the results of other fatigue tests performed in Germany.

Keywords: Wood-concrete composites, Fatigue, Design codes.

1. Introduction

Wood-concrete composite systems are constructed by interconnecting a wood layer with a concrete layer placed atop. In this way it is possible to exploit both materials at the best since the wood is subjected to tension coupled with bending, and the concrete slab is mainly compressed. The composite system, in fact, exhibits larger stiffness and strength, no problem of vibration, better seismic performance, reduced dead load, better appearance, and the possibility to use the wood layer as a permanent formwork for the concrete slab [1]. A notched interlayer connection detail [2] was adopted in this study, with a vertical bearing surface [3]. The interconnection between wood and concrete is achieved by direct bearing of the concrete in the notch on the wood surface. Advantages of the notch detail include the higher composite action (i.e. stiffness of the composite system) and larger achievable load capacity. The stiffness of the interlayer connection determines the degree of the composite behavior achieved [4, 5, 6].

Wood-concrete composite structural members are usually applied in building structures. There are a relatively small number (in the low 100s) of known bridge applications involving wood-concrete composites. A problem with using these novel composite members in bridges with high traffic is that the fatigue behavior of such members under high-cycle repeated loading is not yet well known due to the small number of test data available [7, 8, 9]. Knowing the fatigue performance of these members is essential in bridge applications. Nevertheless, a number of bridges have been built [10, 11], showing the adequacy of the wood-concrete composite system to bridge applications.

2. Experimental Setup

The research was performed at Colorado State University and the testing was conducted in the laboratory of the Metropolitan State College of Denver, in 2011 and 2012 [12]. The research program was conceived in coordination with work at University of Stuttgart.

2.1 Geometry

Fourteen 1626 mm (64 in) long wood-concrete composite beam specimens were built for this test program. The width of the specimens was 191 mm (7.5 in) and each had a 89 mm (3.5 in) thick wood layer consisting of five “2x4” boards with nominal size of 38 mm x 89 mm (1.5 in x 3.5 in), and a 63 mm (2.5 in) concrete layer, as shown in Fig. 1.

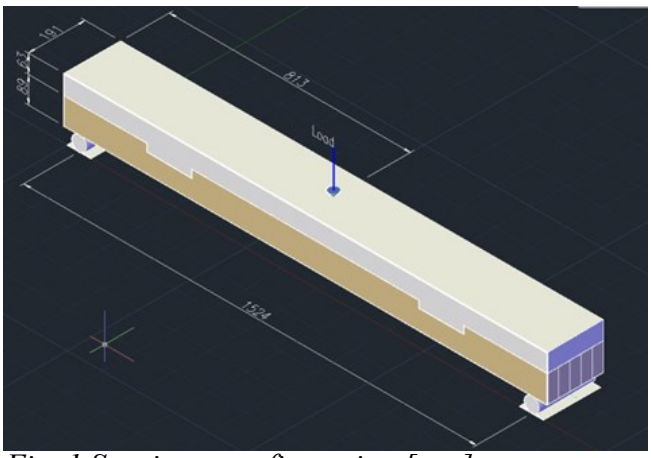


Fig. 1 Specimen configuration [mm]

The wood and concrete layers were interconnected by embedded anchor screws, of 12.5 mm (0.5 in) diameter and 102 mm (4 in) long with an embedding depth of 50 mm (2 in), at the notch locations. The notches were cut into the wood 25 mm deep (1 in) and were 150 mm (6 in) long over the entire width of the specimen. The wood boards were attached to each-other by 4 in deck screws 150 mm (6 in) apart. The wood layer and the end of the wood boards were coated with a sealer paint (shown as white in the picture) to reduce the moisture entering the wood during the casting of the concrete layer.

2.2 Material Properties

The wood material was premium-grade No.2 or better kiln dried Hem-Fir. The MOE value was determined by testing 28 wood samples according to ASTM D143 using an Instron 5569 device [12]. The moisture content of the wood was determined with a Delmhorst 11212 moisture mapping meter. The dry conditions are typical to the Colorado climate (the tests were conducted between the months of September and March). The concrete material contained glass fibers [12].

2.3 Test Setup

Simply supported specimens were configured with a span of 1524 mm (60 in). Mid-span (three-point loading) loading was applied using an MTS hydraulic loading and digital data acquisition system. Roller supports were configured at each end to avoid horizontal forces straining the actuator, resulting in a symmetric configuration, see *Fig. 2*. Some of the specimens were subject to static loading while others to cyclic pulsating loading.

All loading was displacement controlled. The controlled displacement component was the deflection of the member at the point of the load application (mid-span) [13].

Fig. 2 Typical test setup

3. Results

3.1 Static Test Results

In order to determine the average static failure load (P_{max}) a total of five specimens were loaded statically. The load-deflection characteristics recorded are shown in *Fig. 3*. It was observed that a drop in the resisted load occurs when block-shear failure develops first and that the member is usually able to take additional load. More severe and irrecoverable drops in the resisted load are caused by the failure of a wood board in tension at the mid-span . The average total failure load was $P_{s,max}$=56.4 kN (12.7 kip). Initial failures loads were recorded and stiffness data was derived [12].

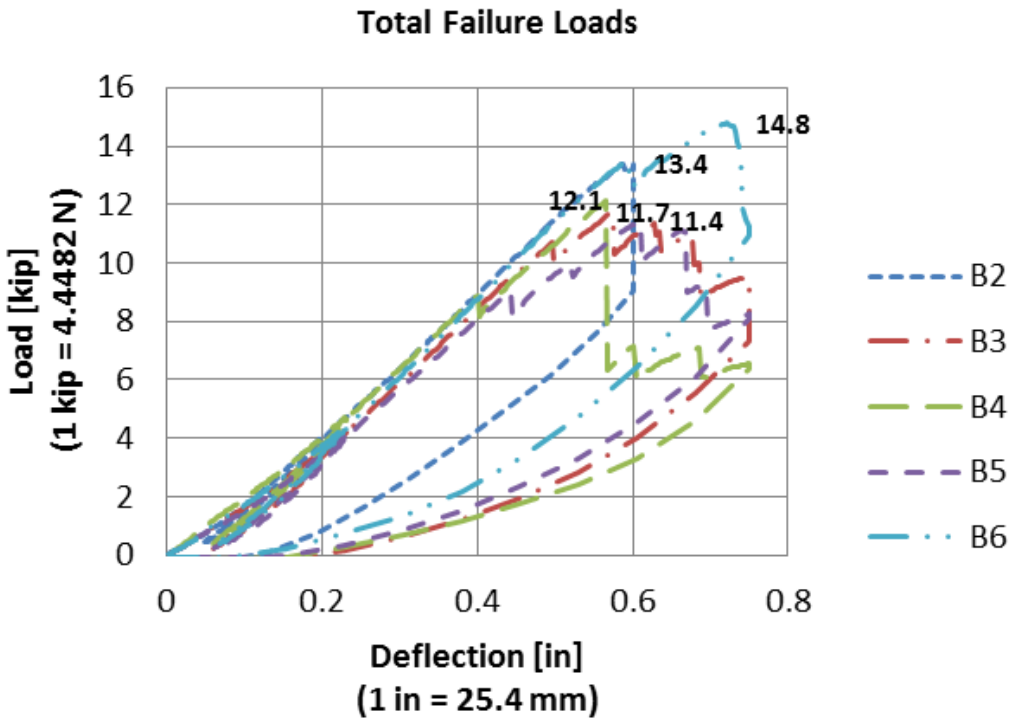


Fig. 3 Typical test setup

3.2 Fatigue Test Results

The load was cycled sinusoidally, at a frequency of 1 Hz, maintaining a maximum load, P_{max}, to minimum load, P_{min}, ratio of $R=P_{min}/P_{max}=0.1$.

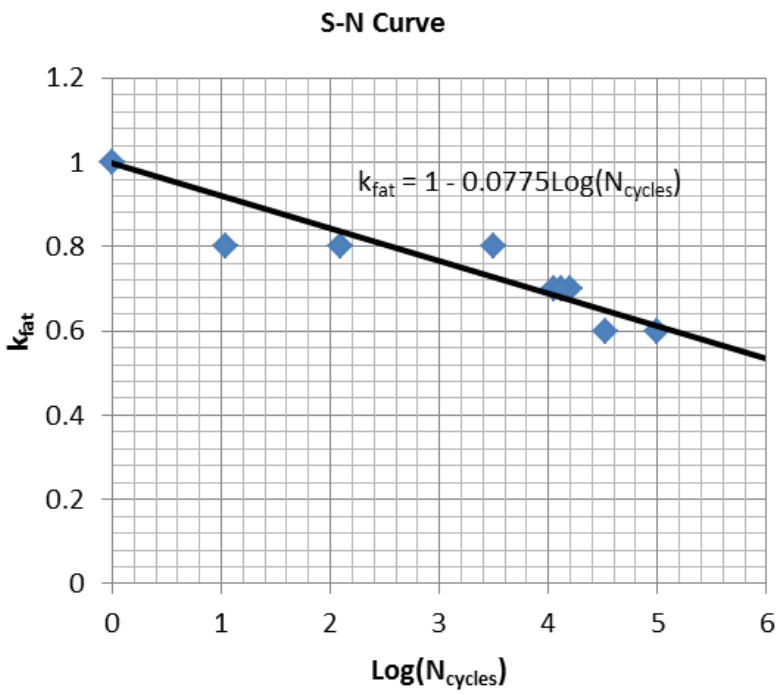


Fig. 4 Mean S-N line approximation

Points on the S-N curve are determined for three levels of the maximum load as a function of the average static failure load, $P_{max}/P_{s,max}$ of 0.8, 0.7, and 0.6. The results [12] are summarized in *Fig. 4*, showing the fatigue test results (blue bullets) and an approximation of S-N line on a logarithmic scale.

In addition, specimen F10 was investigated at a load level $P_{max}\approx 0.5P_{s,max}$, of 29.4 kN (6.6 kip) and no signs of failure were detected in the first 10^5 cycles.

Typical observed failure mode was by initial block-shear, shown in *Fig. 5*, followed by a tension failure of the wood in bending, see *Fig. 6*.

Fig. 5 Typical block-shear failure

Fig. 6 Typical tension failure

4. Conclusions

This paper presents the result of a research at Colorado State University and Metropolitan State University of Denver, performed in coordination with work at University of Stuttgart, attempting to establish the S-N curve for fatigue loading of notched wood-concrete connections based on low-to-high-cycle, repeated loading tests. Experimental results are obtained on fourteen composite beam specimens. Five specimens were statically loaded to failure, while the others were cycled to failure.

Points on the S-N curve were determined for three levels of the maximum load as a function of the average static failure load. Typical observed failure modes were block-shear of the wood at the notch in some cases followed by tension failure of the wood at mid-span. The obtained S-N mean line was determined. This allows for comparison with the EC-5 EN 1995-2:2004 [14], Annex A which addresses the fatigue verification [12]. The results tentatively indicate that the provisions of EC5 EN 1995-2:2004 for fatigue verification could be used for the type of composite member studied herein provided the expected failure mode is by block shear of the wood at the notches. This conclusion is tentative only, due to the small number of tests it is based on, and is consistent with findings by Kuhlmann and Aldi [15].

Acknowledgements

The U. S. Department of Transportation has provided funding via the Mountain Plains Consortium, which is federally sponsored through the University Transportation Centers Program. The contents of this paper reflect the views of the authors only, and the above agencies assume no liability for the contents or use thereof.

Authors are thankful to research aid student Aaron Andrews and students Jasmine Bott and Samuel Csonka for assisting in the MSCD laboratory with the preparation of the specimens and handling.

References

[1] Ceccotti, A., "Timber-concrete composite structures", *Timber Engineering, Step 2*, First Edition, Centrum Hout, The Netherlands, 1995, pp. E13/1-E13/12.

[2] Natterer, J., Hamm, J., and Favre, P., "Composite wood-concrete floors in multi-story buildings", *Proceedings of the International Wood Engineering Conference*, New Orleans, Louisiana, October 28-31, 1996, pp. 3431-3435.

[3] Kuhlmann, U., Schanzlin, J., "Grooves as shear connectors for timber-concrete composite decks", Joints in Timber Structures, Edited by Aicher, S., Reinhardt, H.-W., *Proceedings of the International RILEM Symposium*, University of Stuttgart, 2000, pp. 283-290.

[4] Clouston, P., Bathron, L.A., and Schreyer, A., "Shear and bending performance of a novel wood-concrete composite system", *Journal of Structural Engineering*, 131(9), 2005, pp. 1404-1412.

[5] Dias, A.M.P.G., "Mechanical behaviour of timber-concrete joints", *Ph.D. Thesis*, University of Coimbra, Portugal, 2005.

[6] Balogh, J., Fragiacomo, M., Gutkowski, R. M., and Fast, R. S., "Influence of repeated and sustained loading on the performance of layered wood-concrete composite beams", *Journal of Structural Engineering*, 134(3), 2008, pp. 430-439.

[7] Kuhlmann, U., Aldi, P., "A study on fatigue strength in timber-concrete-composite beams", *Report*, Universität Stuttgart, 2008.

[8] Miller, N., "Long-Term and repeated load behavior of wood-concrete composite beams incorporating utility poles", *M.S. Thesis*, Colorado State University, Fort Collins, CO, USA, 2009.

[9] Mueller, J., Rautenstrauch, K., "Fatigue behaviour of stud connector used for timber-concrete composite bridges", 44th Meeting of International Council for Research and Innovation in Building and Construction, *Working Commission W18-Timber Structures*, Alghero, Italy, 2011.

[10] Jutila, A., "Wood Concrete Composite Bridges – Finnish Speciality in the Nordic Countries", *Proceedings of the International Conference Timber Bridges ICTB2010*, Lillehammer, Norway, September 12 -15, 2010, pp. 383-392.

[11] Dias, A.M.P.G., Ferreira, M.C.P., Jorge, L.F.C., Martins, H.M.G., "Timber–concrete practical applications – bridge case study", *Proceedings of the ICE - Structures and Buildings*, Volume 164, Issue 2, March 2011 , pp. 131-141.

[12] Balogh, J., Fragiacomo, M., Gutkowski, R., Atadero, Ivanyi, P., "Low-to-high cycle fatigue behavior of wood-concrete composite beams with notched interlayer connections", *Pollack Periodica - International Journal for Engineering and Information Sciences,* Volume 8, No. 1, 2013, pp. 3-14.

[13] EN 26891: 1991, "Timber Structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics", *European Standard (ISO 6891-1983)*, February 1991.

[14] EN 1995-2: 2004, "Design of timber structures, Part 2: Bridges", *Eurocode 5*, November 11, 2004.

[15] Kuhlmann, U., Aldi, P., "Prediction of the fatigue resistance of timber-concrete composite connections", *42nd Meeting of the Working Commission W18-Timber Structures*, CIB, International Council for Research and Innovation, Dübendorf, Switzerland, August 24-27, 2009, Paper No. CIB-W18/42-7-11, 10 pp.

FATIGUE PERFORMANCE OF SINGLE SPAN WOOD-CONCRETE-COMPOSITE BRIDGES

Leander Bathon[1], Oliver Bletz-Mühldorfer[2]

ABSTRACT: Wood-concrete-composite structures in residential and commercial applications are gaining market shares in recent years. The application of the wood-concrete-composite bridges however is very limited. This is mainly because there is a lack of knowledge on the fatigue behaviour of the wood-concrete-composite bridges. The authors believe there is a great potential to design these bridges using wood-concrete-composite cross sections with single span of 30 meters and more.

KEYWORDS: wood-concrete-composite, glued in shear connectors, bridges, fatigue, cyclic loading test

1 INTRODUCTION

The use of wood-concrete-composite systems for highway bridges in Germany was not possible in the past due to a lack of code approval of the shear connectors. The research work of this paper however was the base for the first code approval given in Germany for this kind of application. The authors believe that there is now a great potential for highway bridges using the wood-concrete-composite technology.

2 TESTING

The performance of a wood-concrete-composite bridge depends solemly on the performance of the shear connection. The shear connetors used in this application is a hbv-shear connector which consists of a steel mesh of S 235 (equvalent to A36). On half of the hbv-shear connector reaches 40 mm into a 3,2 mm wide channel within the timber and is secured through adhesive action. The other half (50 mm) is reaching into the concrete. In order to provide a durable bridge system it is desirable to use this stiff but ductile connection system.

2.1 SHEAR TEST

The shear tests were conducted as push-out-tests. The specimens were 600 mm long. The wooden cross section for the beam solutions was 80 by 140 mm and for the plate solution was 400 by 80 mm. The concrete cross section was 70 by 400 mm. The hbv-shear connector reaches 40 mm into the wooden and 50 mm into the concrete part of the composite system. All shear tests show steel failure within the shear connectors under ultimate load conditions (Fig. 1). Figure 1 shows a great plastic performance with almost now variation due to the steel failure of the shear connectors.

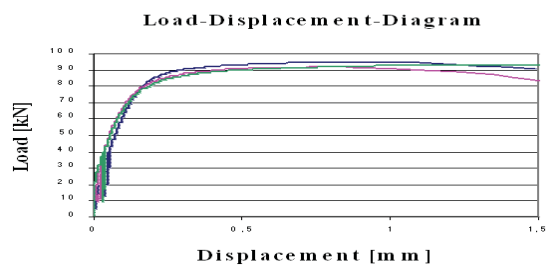


Figure 1: Shear testing on shear connector

2.2 FATIGUE TESTING

The fatigue testing was performed on the shear connector itself (tension shear) and the wood-cocnrete-composite system (compresion shear). Again all test showed steel failure within the shear connectors. Figure 2 shows the results of the fatigue tests in the hbv-shear connector. The shear connectors were tested under a sinustype tension ramp-load and a frequency of 2 to 20 Hz. The load was applied at constant strain ratios $\kappa = \sigma_u / \sigma_o$. Two test-series differing in a gap between the two clampings were conducted (Δ = 2 mm, Δ = 4 mm).

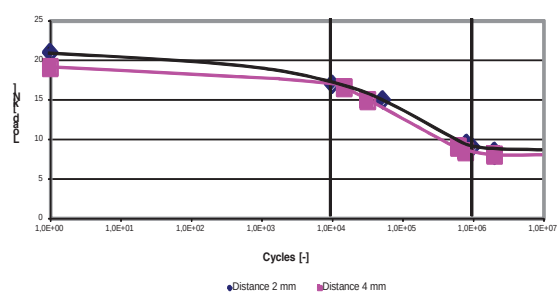


Figure 2: Fatigue testing on shear connector

[1] Leander Bathon, Faculty of Civil Engineering, Timber structures and building technology, HS-RM Wiesbaden University of Applied Sciences, Kurt-Schumacher-Ring 18, 65197 Wiesbaden, Germany, Email: leander.bathon@hs-rm.de

[2] Oliver Bletz-Mühldorfer, Ph.D. student, TU Darmstadt and HS-RM Wiesbaden University of Applied Sciences, Kurt-Schumacher-Ring 18, 65197 Wiesbaden, Germany Email: oliver.bletz@hs-rm.de

The ultimate load in the short-time tests arise in dependancy of the gap to $F_{u,2mm}$ = 20,7 kN respectively $F_{u,4mm}$ = 19,2 kN. The fatigue strength – identified for a an expected load cycles of 2 x 10^6 – shows a value of 8,47 kN at a gap of 2 mm and 7,98 kN for the gap of 4 mm.

3 ANALYSIS

3.1 TRUSS MODEL

The analysis of the wood concrete composite bridge is performed through a truss model. The model is shown in Figure 3. The truss model consists of a top and bottom member representing the concrete and wood respectively. The spring diagonals represent the shear connector based on the shear test of the steel mesh. The truss model provides both the prediction of the test performance as well as a design tool for the commercial application of the system.

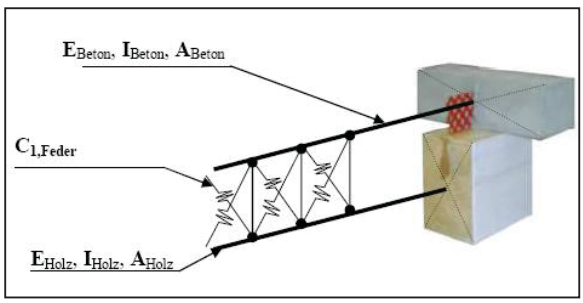


Figure 3: Truss model

The analysis showed that the wood concrete composite system produces a nearly fully composite action. The fatigue design is shown in the EC 5. In order to perform the fatigue design you need the factor **a** and **b** (Fig. 4) for the shear connector in use. The code approval of the hbv-shear connector shows **a=2,5** and **b=4**.

$$\sigma_{d,max} \leq f_{fat,d} \quad (1)$$

$$f_{fat,d} = k_{fat} \frac{f_k}{\gamma_{M,fat}} \quad (2)$$

$$k_{fat} = 1 - \frac{1-R}{a(b-R)} \log(\beta N_{obs} t_L) \quad (3)$$

Figure 4: Fatigue design (Eq. 1,2,3) of EC 5

3.2 FATIGE ANALYSIS

The fatigue analysis is based on the EC 5. It uses the cumulative linear fatigue theorie of Palmgren-Miner. Figure 5 shows the comparison of the Palmgren-Miner Rule for a strain ration $\kappa = \sigma_u / \sigma_o$.of κ =0,09. Figure 5 shows that the application of the Palmgren-Miner-Rule will produce conservative design values for the fatigue design of the wood-concrete-composite bridges.

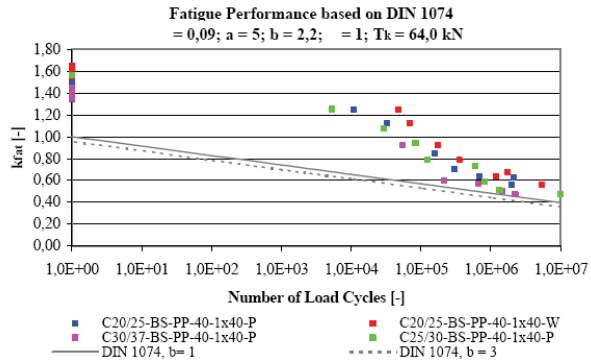


Figure 5: Fatigue analysis vs. test data

In recent years a number of wood-concrete-composite bridges have been build using the data presented (Figure 6). The authors are convinced that with the knowledge gained based on this research there will be more opportunities for the timber industry to build wood-concrete-composite bridges throughout the world.

Figure 6: WCC-bridge in Winschoten (NL)

4 CONCLUSIONS

This paper introduces a fatigue design approach for a single span wood-concrete-composite bridge. The design is based on 200 shear test as well as 60 fatigue test that have been performed at the Test Laboratories in Wiesbaden, Germany. The test data provides the information needed to design the wcc-bridges according to the code approval of the system used. The code approval paper determines the stiffness and strength of the shear connector for static loading conditions as well as the factors a/b for fatigue loadings conditions according to EC 5.

ACKNOWLEDGEMENT

The reasearch was supported by the following companies: Derix, Gröber, TiComTec, Duscheck & Duscheck. In addition the federal agency of AIF supported the research and therefore the Ph.D. work of the co-author Oliver Bletz-Mühldorfer. The authors are very grateful of all support brought to them.

REFERENCES

[1] Bathon, L.; Bletz, O. (2008): „Holz trifft Beton", bauen mit holz 6/2008, Seite 28 – 33

[2] Bletz, O.: „Beitrag zur Entwicklung von Holz-Beton-Verbundkonstruktionen mit eingeklebten Streckmetallen", Dissertation in Arbeit, TU Darmstadt, Institut für Stahlbau und Werkstoffmechanik

[3] DIN - Deutsches Institut für Normung (2006): „DIN 1074, Holzbrücken"

[4] DIN - Deutsches Institut für Normung (1978): „DIN 50100, Werkstoffprüfung, Dauerschwing-versuch, Begriffe, Zeichen, Durchführung, Auswertung"

[5] DIN - Deutsches Institut für Normung (2003): „DIN-Fachbericht 101, Einwirkungen auf Brücken"

[6] Kuhlmann, U.; Aldi, P. (2008): „Fatigue of timber-concrete-composite beams: characterisation of the connection behaviour through push-out tests", Proceedings of the 10th World Conference on Timber Engineering, Miyazaki, Japan

[7] Bathon, L.; Bletz, O. 2006. Long term performance of continous wood-cocnrete-composite systems. Proc. of the 10th World Conference on Timber Engineering, Portland, Oregon, USA 2006

[8] Bathon, L.; Bletz, O., Bahmer, R., 2006. Concrete bearings – a new design approach in wood-concrete-composite applications. Proc. of the 10th World Conference on Timber Engineering, Portland, Oregon, USA, 2006

[9] Bathon, L.; Clouston, P. 2004. Experimental and Numerical Results on Semi Prestressed Wood-Concrete Composite Floor Systems for Long Span Applications. IN: Proc. of the 8th World Conference on Timber Engineering, Lathi, Finnland, June 14 – 17, 2004, Volume 1: page 339 – 344

[10] Fa. Duscheck & Duscheck GmbH Ingenieurholzbau, A-3032 Eichgraben, www.d2-duscheck.at

[11] Bathon L.A. and Graf, M. (2000). "A Continuous Wood-Concrete-Composite System". Proc., World Conference of Timber Engineering, Whistler, B.C.

[12] Fa. TiComTec GmbH, D-63808 Haibach, www.holz-beton-verbund.de

Älvsbacka Bridge – A Bridge in Cold Climate

Niclas Björngrim
PhD student
Luleå University of Technology
Skellefteå, Sweden
Niclas.bjorngrim@ltu.se

PhD studies at Wood products engineering, LTU 2011-
MSc Wood technology at LTU 2005-2010

Xiaodong Wang
Associate Professor
Luleå University of Technology
Skellefteå, Sweden
Alice.wang@ltu.se

Olle Hagman
Professor
Luleå University of Technology
Skellefteå, Sweden
Olle.hagman@ltu.se

Summary

Älvsbacka Timber Bridge was erected during the summer of 2011. The bridge is equipped with a health monitoring system with GNSS-receivers, accelerometers, MC-sensors, weather station. The timber bridge is situated in Skellefteå, Sweden close to the artic circle. The location and the monitoring system of Älvsbacka Bridge gives us a unique opportunity to study the behavior of glulam and the bond of glulam in a cold climate. The bond of the adhesives is usually more brittle than wood. Bond line performance at elevated temperatures is well documented, but little information is reported on the behavior on sub zero temperatures. The bond line performance is of concern for outdoor timber constructions in regions such as: the Nordic countries, northern US, Canada, Russia and northern Japan. The monitoring system of the bridge gives us opportunities to study the effect of cold climate on timber constructions. Together with lab tests on pine and spruce and the bond stability of the glue line down to -60° we will have a better understanding on how engineered wood products and timber constructions behave at extreme temperatures.

Keywords: Glue line, Engineered wood, Cold climate, Timber bridge

1. Introduction

Engineered wood is increasingly used in large structures in Europe, though little is known of its behavior in cold climate. This paper presents the structural health monitoring system of Älvsbacka Bridge, a newly built suspension timber bridge with a deck of glulam timber, as well as a bond stability study regarding cold climate performance of engineered wood. The bridge is located in Skellefteå in northern Sweden, and connects two parts of the city situated on opposite shores of the

Skellefteå river. In an ongoing study of the timber-bridge, a structural health monitoring system is employed to verify structural design and long-term performance.
Wood construction is increasingly using engineered wood products. Engineered wood application in bridges, sports centers and timber buildings are common in Europe and North America. Adhesives are the key part of these engineered wood products and play an important role in the performance of engineered wood products. How do the bond lines of engineered wood behave under extreme cold climate? The concern is stronger in regions like Scandinavia, Canada, Alaska, Russia, and North China and Japan etc. The bond of most adhesives is more brittle than wood. The performance of bond lines at elevated temperatures is well documented [1, 2, 3], but not much information is available on the stability of bond lines at low temperatures and especially under extremely cold temperatures, although some studies of timber bridges in cold climates have been reported [4, 5, 6].
The main objective of this project is to determine how engineered wood behaves when exposed to temperatures between 20 °C to -60 °C, and study how these features should be described and measured in an objective way. Bond stability of glulam structures in cold climate is also examined in a range of experiments ranging from small glued wood joints to full size glulam bridge performance over time. In this paper the first test on small specimens is presented.

1.1 Älvsbacka Bridge

Älvsbacka timber footbridge was built over the Skellefteå River in Sweden 2011. It is a cable-stayed bridge with a span of 130 m. In general today the health of the bridges is assessed at regular intervals by visual inspections and if necessary some minor local tests. Continuous measurements could complement the inspections and provide a better basis for planning maintenance activities and evaluating the remaining service life. An ongoing project is developing monitoring tools for timber structures to guide the planning of maintenance and to evaluate the performance of the bridge. The monitoring of the bridge contributes to wood research on specific areas such as durability of timber bridges and vibrations of wooden deck plates, research on measurement and data transmitting techniques as well as performance of timber structures in cold climate. The Älvsbacka Bridge is monitored using GNSS receivers, MEMS accelerometers, wireless moisture content sensors, strain gauges and a weather station. Data from the monitoring systems is analyzed regarding accuracy, complexity, costs and reliability for long time use. In this way, the measurements deliver complete and unique data from long-term monitoring. Wireless sensor networks are tested for the bridge monitoring. Temperature, wind and rain are measured at the site. The bridge is monitored for temperature, moisture, movements, deflections and vibrations at different points enabling analyses of the bridge health. Vibration measurements using accelerometers, natural frequencies and modes will be used to evaluate the structure.

1.2 Wood in cold climate

The wood constructions are increasingly using engineered wood products (such as glued-laminated timber (glulam), laminated veneer lumber (LVL), structural-composite lumber (SCL), or cross laminate timber (CLT)) [1]. Adhesives are the key part of these engineered wood products and play an important role in the performance of engineered wood products. The response of glue line to temperature changes and extreme temperatures as regularly occurs in normal use raise some concerns on the integrity of wood structure. It is believed that the most commercial wood adhesives are vulnerable to the extreme temperatures and temperatures changes. The concern is stronger in the regions having extreme weather like Scandinavia, Canada, Alaska, Russia, and North Japan etc. Wood constructions in these areas are exposed to low temperatures during a quite long period each year. Despite this, thermal effects are usually not a consideration in the design of wood constructions. Wood and adhesives are very different in terms of swelling and shrinkage coefficients. Glue line of most adhesives is also more brittle than wood. If not compensated for, any differential between wood and adhesives thermal properties can lead to performance problems

when the construction is exposed to large temperature changes. In these cases, the design needs to compensate for differential movement of components while preserving structural integrity.
This research project takes advantage in that the Älvsbacka Bridge is located where temperatures varies from 30° to -35° during the year together with temperature data from the health monitoring system.

2. Materials and method

2.1 Materials

The wood used was Norway spruce (*Picea abies*). The equilibrium moisture content was 12% and the average density 450kg/m^3
The adhesives used were: One-component polyurethane (PUR), polyvinyl acetate (PVAc), emulsion-polymer-isocyanate (EPI), melamine-formaldehyde resin (MF), melamine-urea-formaldehyde resin (MUF) and phenol-resorcinol-formaldehyde resin (PRF).

2.2 Methods

The boards were conditioned at 20°C and 65%RH until EMC of 12% was reached.
The specimens were made of two lamellas glued together, after gluing the lamellas together the specimens where cut to correct size, 150mm x 20mm x 10 mm. For each of the six adhesives and for each of the six temperatures 15 specimens were produced. 15 reference specimens of solid wood were also produced.

Figure 1. Universal testing machine with wood specimen, in climate chamber.

The specimens were tested for shear strength according to European Standard EN302-1. The tests were done with a universal testing machine inside a Vötsch industrietechnik vcv7120-5 climate chamber (Figure 1). The feed speed was 2mm/min. the whole test procedure was conducted as recommended in EN 302-1. Data were analysed with IBM SPSS Statistics 20. An analysis of variance with 5% level of significance was conducted on the data. When significant differences were found a Duncan's multiple-range test were conducted to find differences caused by different adhesives at different temperatures.

3. Results

From the data presented in table 1 it can be seen that the trend is when temperature is decreased so is the shear strength of both specimens with adhesives and solid wood specimens. PUR showed the strongest shear strength of the tested adhesives. PRF and PVAc was the adhesives that had the lowest change in shear strength, both just below 20% while the other adhesives lost 25-27% in shear strength. However the shear strength of PVAc decreases a lot between 20°C and -20°C and then behaves rather homogenous between -20°C to -60°C. MUF showed the lowest shear strength of the tested adhesives. Shear strength of solid wood decreased from 10,1 to 8,0 going from 20°C to -60°C.

Table 1 Shear strength data (in MPa)

Temp./Glue	1-PUR	2-PVAc	3-EPI	4-MF	5-PRF	6-MUF	7-Wood
20°C	10.9 (1.0)[1] A[2]	9.6 (0.6) A	10.0 (1.1) A	9.8 (0.9) A	9.8 (1.0) A	8.7 (0.7) A	10.1 (0.9) A
-20°C	10.8 (2.3) A,B	8.1 (1.2) B	8.3 (1.6) B	9.4 (2.3) A	9.7 (1.8) A	7.9 (1.1) A,B	9.5 (1.1) A,B
-30°C	10.2 (2.9) A,B	8.2 (1.2) B	8.3 (2.0) B	8.1 (2.5) A,B	9.2 (2.0) A,B	7.2 (1.9) B,C	9.0 (0.9) A,B,C
-40°C	9.9 (1.5) A,B	7.9 (1.9) B	7.3 (1.5) B	7.7 (1.4) B	9.3 (1.4) A,B	7.2 (1.3) B,C	8.7 (1.3) B,C
-50°C	9.2 (1.3) B,C	7.9 (1.8) B	7.8 (1.4) B	7.5 (1.2) B	8.4 (1.7) A,B	6.6 (0.9) C	8.8 (1.5) B,C
-60°C	7.9 (1.4) C	7.7 (1.1) B	7.2 (1.5) B	7.3 (1.7) B	8.0 (1.4) B	6.6 (1.3) C	8.0 (0.9) C
Total shear strength change (%)[3]	27.1	19.3	27.5	25.0	18.7	24.5	27.1

[1] Values in parentheses are standard deviations based on 15 specimens.
[2] Values in the same capital letter in each column are not statistically different at the 0.05 significance level.
[3] Total shear strength change (%)=(Shear Strength $_{20°C}$ - Shear Strength $_{-60°C}$)/ Shear Strength $_{20°C}$*100.

4. Discussion

Specimens of Norway spruce and PUR, PVAc, EPI, MF, PRF and MUF adhesives were tested for shear strength at 20°C, -20°C,-30°C,-40°C,-50°C and -60°C. The shear strength of the tested adhesives at room temperature is relatively stable. As the temperature drops the variation of the shear strength increases. All tested specimens except for PUR have a shear strength below 10 MPa already at -20°C, not meeting the EU requirement (EN 301). This calls for more attention on how low temperature impacts solid wood and engineered wood products.

Next step of this project is to study how glued finger joints behaves in cold climate.
Data acquired in this ongoing project will be used to model temperature behavior in the glue line and compare with bridge data.

5. References

[1] Clauss S., Joscak M., Niemz P. Thermal stability of glued wood joints measured by shear tests. *European Journal of Wood and Wood Products* 69(2011): 101-111.

[2] Falkner H, Teutsch M (2006) Load-carrying capacity of glued laminated wood girders under temperature influence. *Bautechnik* 83(6): 391-393.

[3] Frangi A, Fontana A, Mischler A (2004) Shear behavior of bond lines in glued laminated timber beams at high temperatures. *Wood Sci. Technol* 38(2): 119-126.

[4] Kainz J, Ritter M (1998) Effect of cold temperatures on stress-laminated timber bridge deck. *Proceedings of the 5th World Conference on Timber Engineering*, 1998, Montreux, Switzerland. WCTE, August 17-20,, 1998 Swiss federal institute of Technology Lausanne Switzerland Montreux. Eds.., Natterer, J. and Sandoz, J. L. Vol. 2: 42-49.

[5] Wacker J.P. (2003) Cold temperature effects on stress-laminated timber bridges-A laboratory study. Res. Pap. FPL-RP-605. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 22p.

[6] Wacker J. P. (2009) Performance of stress-laminated timber highway bridges in cold climates. P. E. M. ASCE. Copyright ASCE 2009. Cold Regions Engineering 2009.

Comparison of accelerometric response records on a timber footbridge

Daniele Bortoluzzi[1]

Fabio Casciati[1]

Sara Casciati[2]

Lucia Faravelli[1]

[1] **DICAr, University of Pavia, via Ferrata 1, 27100 Pavia, Italy.**

[2] **DICA, School of Architecture, University of Catania at Siracusa, P.za Federico di Svevia, 96100 Siracusa, Italy.**

fabio@dipmec.unipv.it

Summary

The purpose of this paper is to report the results of two experimental campaigns on a timber footbridge. They were carried out by the authors, in order to estimate the mechanical behavior of the footbridge, in two different seasons: late Autumn and late Spring. The bridge under study is made by two main parallel timber beams, linked by a sequence of transversal steel elements. The geometry of this structure sees a length of about 83 m length and a width 3.8 m width. To cover the whole length, a special wireless telecommunication architecture was used.

Keywords: Pedestrian bridge, Structural monitoring, Timber bridge.

1. Introduction

The behavior of wooden elements is very sensitive to some environmental conditions as for an example the moisture content in the air, the temperature of the air and so forth…[1]-[2]-[3]. Thus, the idea behind this paper is to identify structural response modifications (if any) from modifications due to different environmental conditions. Moreover timber structures, and wooden elements in general, require a continuous maintenance in order to preserve their strength, but also to fix their architectural appearance. The best approach would be to rely on a continuous structural monitoring system [4]-[5]. But this solution is only possible if the structural system offers adequate lodgment to the equipment and the possibility to check the behavior of the data acquisition system. So the only viable and practicable way is a periodical structural monitoring which holds if and only if one is to separate, once again, structural modifications (if any) from the response modifications associated with different environmental situations.

2. The case study

The authors were offered the chance to collect data on a timber footbridge located in Trasaghis (UD), a small village in the North-East part of Italy. The bridge, built between 2009 and 2010, is used to link the two sides of the emissary channel of a natural lake named Cavazzo Lake (see Fig. 1).

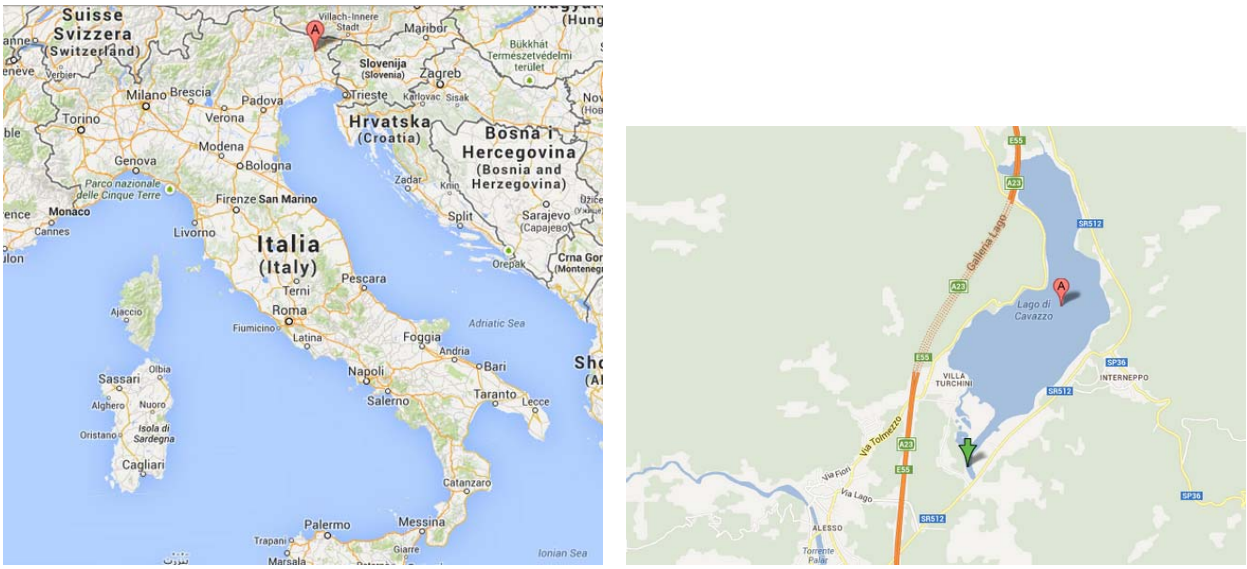


Fig. 1 Map of the region (left) and a zooming on the lake area (right).

To harmonize the structure with the naturalist park surrounding the lake, the designer, in accordance with the Municipality, decided to employ only timber and steel as construction material, limiting the use of the reinforced concrete to the foundations (see Fig. 2).

Fig. 2 Lateral view of the pedestrian timber bridge under study.

The static scheme of the bridge can be included within he family of “cable-stayed bridge” where, however, the typical steel cables are replaced by tubular steel elements anchored to the two antennas located on the opposite sides of the channel. The span is 83 m and the double- beam deck width is 3.82 m, of which 3.22 m represents the free crossing width.

Glued laminated timber of high strength GL28c is adopted for all the main structural timber elements, while glued laminated timber of strength GL24c is adopted for the walking deck. For the steel elements only strength S355JR was adopted. The two main lateral curved beams of section (0.20 by 1.941 m) are mounted on neoprene supports at the ends, and are linked by a sequence of “H” shape transversal tubular steel elements. These elements supports five timber beams on which the walking deck is mounted and fixed by high-strength screws (see Fig. 3).

Fig. 3 A detail of the beam support (left). View of the deck from below: one can see the linking steel transversal beams and the steel braces (right).

The beams are anchored, at the thirds, to the steel antennas on the two sides. The height of the antennas is about 15 m, and they are made by elements of tubular steel section of external diameter 457,5 mm and thickness 14,2 mm. The link antenna-beam (tubular steel oblique element) is made by tubular steel elements of section of external diameter 273 mm and thickness 8 mm (see Fig. 4).

In this structure great importance has been given to the durability of the timber elements, especially against the environmental attach. In fact, as one can see in Fig. 5, the external surface of the two main timber beams, are covered by a cladding made by larch planks. Moreover, in its upper side there is a layer of synthetic material used as water proof system against the rain.

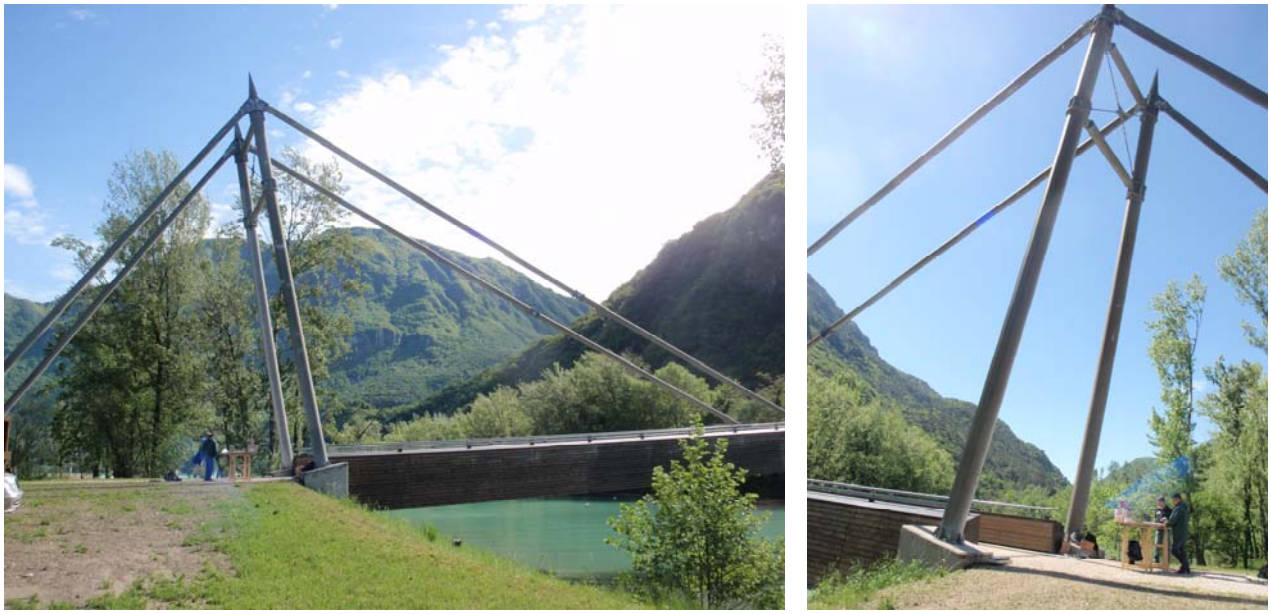


Fig. 4 A detail of the tubular steel oblique elements (left). A lateral view of the antenna (right).

Fig. 5. View of the upper protection system of the beam made by copper (left). A detail of the external larch cladding (right).

According with the Italian code [6], the pedestrian bridge under study can be regarded within "category 3"; this means that two main static loads have to be considered:

- $q_{1.d}$ = isolated load of 10 kN over an area of 0.70 by 0.70 m;
- $q_{1.e}$ = distributed load of 4 kN/m^2 resulting from a dense people gathering.

In addition, the effects of snow, wind and earthquake were taken into account in the design.

A numerical model of the system was built within MSC Marc-Mentat2010 environment [7]. Moving from the design technical documents, particular attention has been paid to modeling every single element of the structure. The two main timber beams were modeled with *shell* elements (0.20 m by 1.941 m). In the same way the deck has been modeled by *shell* elements 6.3 cm thick. To represent the static scheme of the deck, steel elements with H shape were introduced. The deck modeling was finally completed introducing five squared timber beams (employed to carry the wooden deck), and the bracing elements under the deck. To better simulate the interaction between the deck and the beams discussed above, *ad-hoc* rigid links were used. The model was completed by introducing the main steel element of the antenna and the tubular oblique elements, simply as beam element (see Fig. 6). More details about the structural modeling are reported in [8] and [9].

Fig. 6. View from the bottom of the deck (left), and details of the antenna and the tubular oblique elements (right) as implemented in the MSC Marc-Mentat2010 software.

3. Experimental campaign: plan and configuration of the instrumentations

The experimental campaign was planned in two stages, both focused on acquiring the accelerations in sensitive points of the bridge under environmental loads. Standard tri-axial and uni-axial Kinemetrics accelerometers were deployed. The position of the sensors was selected as resulting by numerical analysis *ad hoc* planned.

Two challenges were soon met. Indeed the use of a standard wired acquisition system would have covered a distance up to the 30 m. To overcome this experimental gap, the authors employed a wireless technology (see Fig. 7) developed in [10]. The second, and main, challenge was to develop a reliable process to clean the effects of current environment conditions, that might afflict different experimental campaigns [11].

The authors, in agreement with the Municipality owner of the bridge, defined several experimental campaign in different periods of the years in order to cover the purpose of their research activities. In this paper the results of the only two available tests on field are discussed. In particular the first campaign was carried out in November 2012, while the second one in May 2013 (the further test are at the moment just planned).

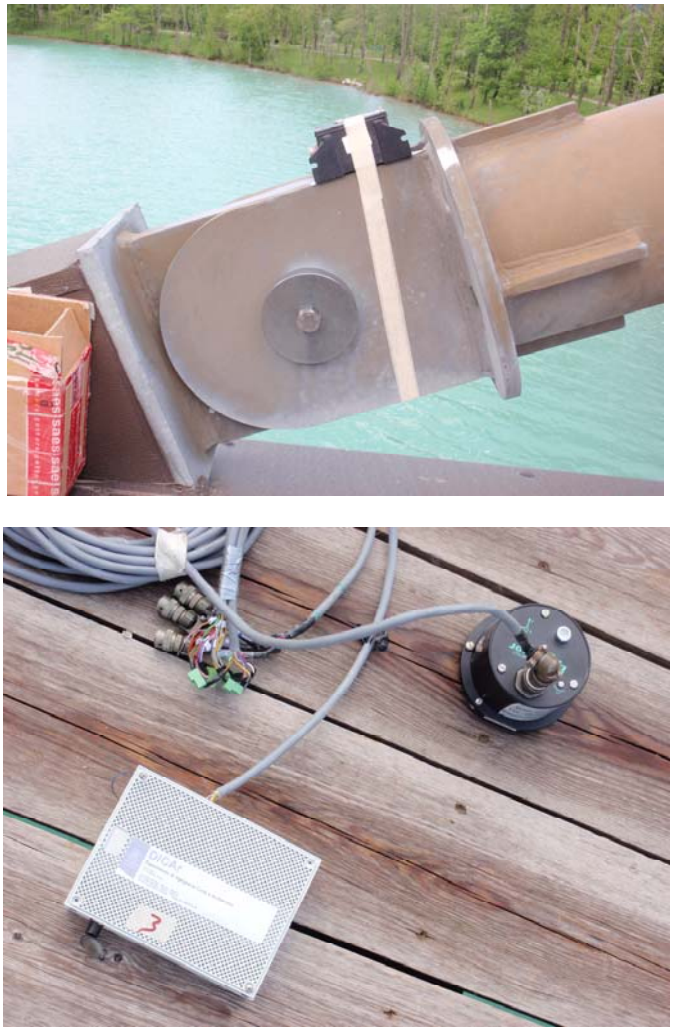


Fig. 7 Uni-axial accelerometers located over the tubular steel elements (above).The tri-axial accelerometer connected with a wireless sensor unit (WSU) posed over the bridge's deck (below).

The first set of experiments was carried out on November 30, 2012. The average temperature of air along the data acquisition was about 10 °C. The configuration of the sensors is given in Fig. 8.

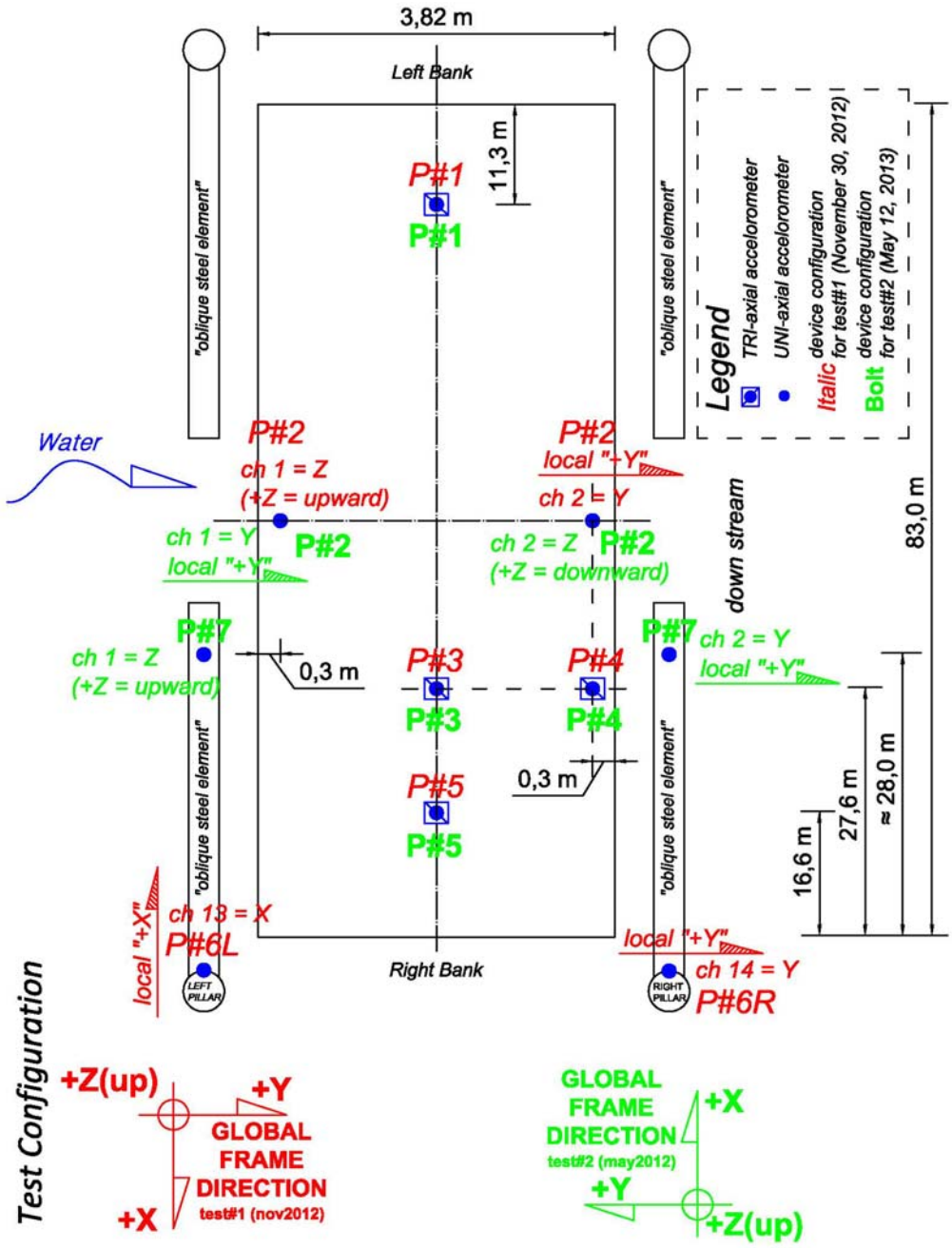
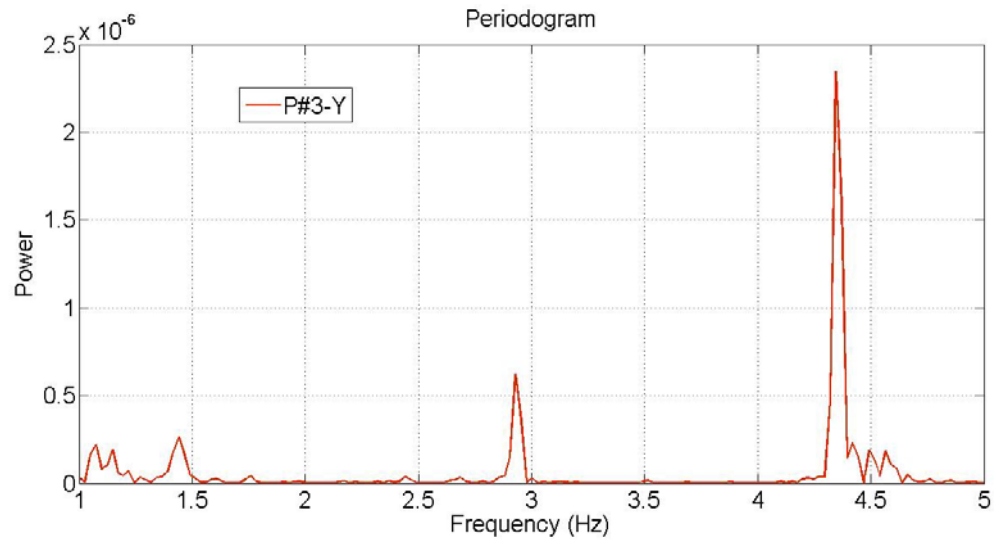


Fig. 8 Sensors configuration for the two experimental campaign (November 30, 2012 and May 12, 2013).

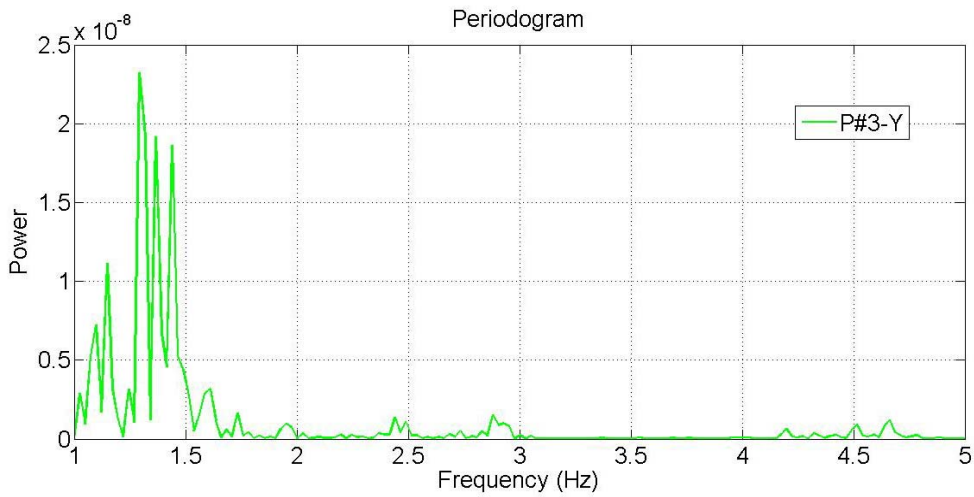
The second campaign of experiments was carried out on May 12, 2013 under the sensor configuration sketched again in Fig. 8.

Two set of data are reported: the first one was collected in the morning (9.30 a.m. – temperature of air about 15.5 °C), and the second one at noon (12:00 a.m. – temperature of the air about 25 °C). This test repetition was planned for investigating changes in the structure behavior related to environment condition variations.

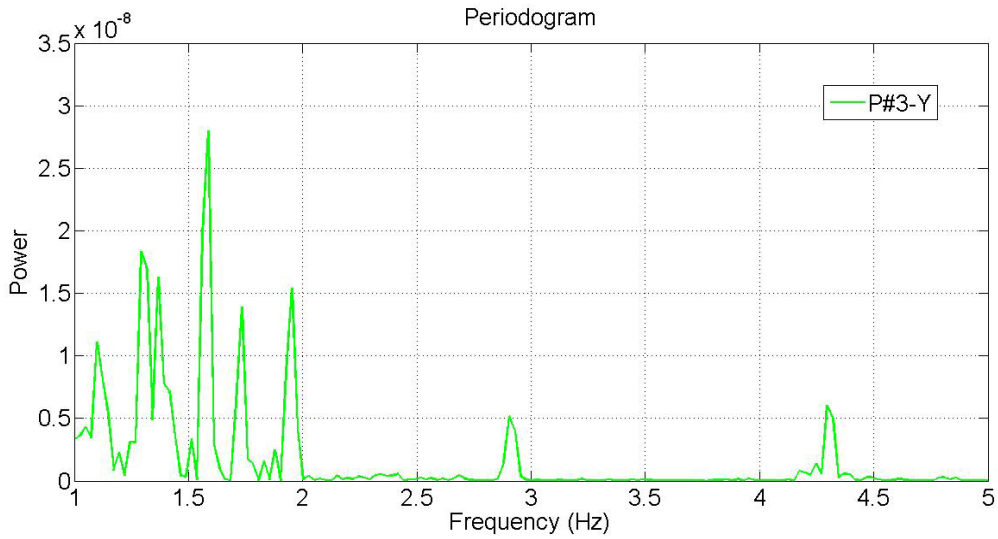
The periodograms for the acceleration components along the transversal and vertical axes are given in Fig. 9 and Fig. 10. The three tests provides the same ranges for the frequencies which correspond to the vibration of the different structural elements.



(a)



(b)



(c)

Fig. 9 Periodograms of the signals recorded at position 3 (P#3): from the top to bottom, one sees the elaboration of the signals (along direction Y) collected in November, in the early morning of May 12, 2013 and in the late morning of the same day. The number of points in the temporal window of the signal is 2170.

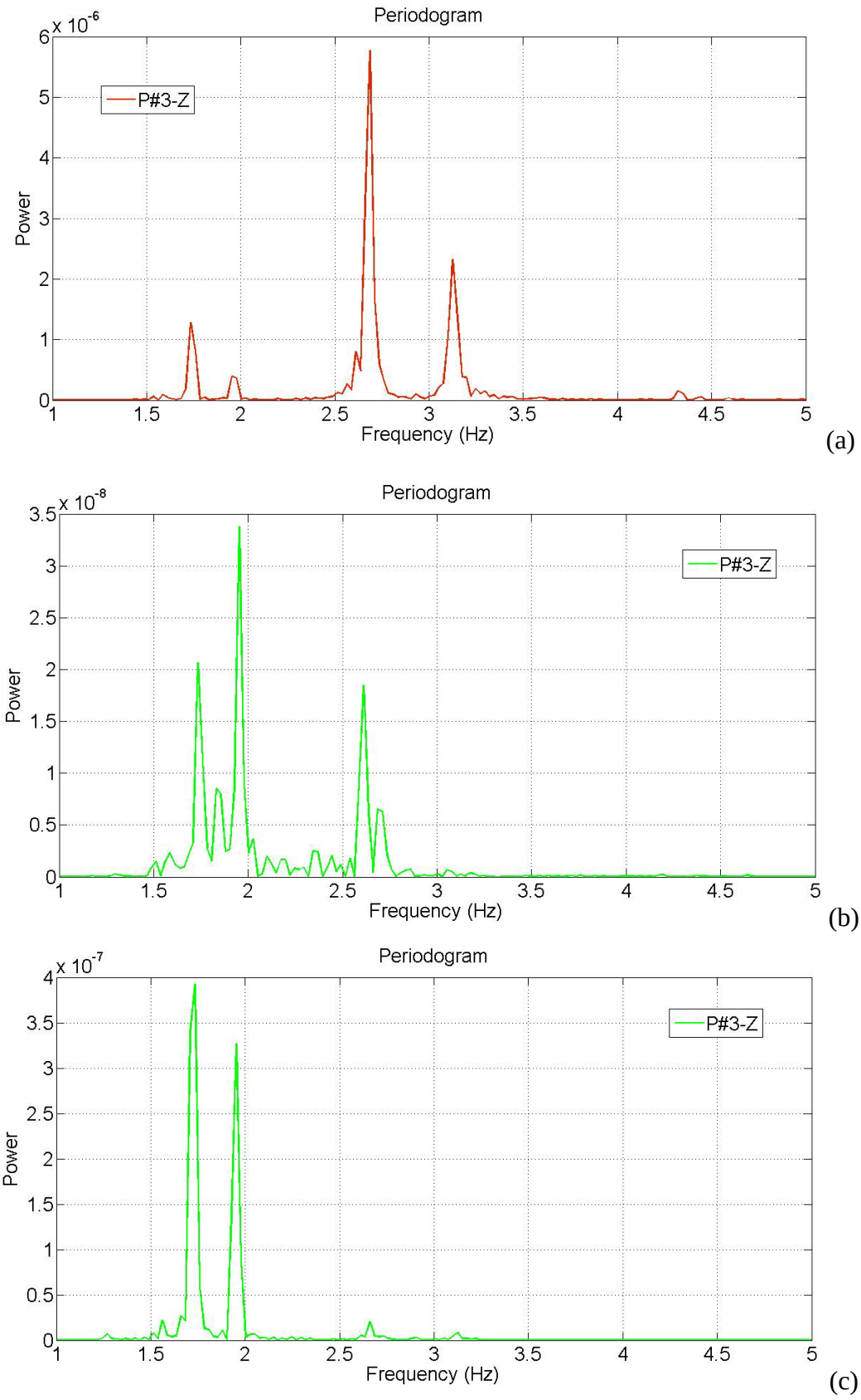


Fig. 10 Periodograms of the signals recorded at position 3 (P#3): from the top to bottom, one sees the elaboration of the signals (along direction Z) collected in November, in the early morning of May 12, 2013 and in the late morning of the same day. The number of points in the temporal window of the signal is 2170.

4. Conclusions

The wireless structural monitoring system adopted in the experimental campaign reported in this paper was working in a very satisfactory way. Indeed standard wired solutions, applied to structures like the one under study (with span length of 83 m), would have obliged to a suitable design of the cables, with intermediate storage stations.

Despite the two longitudinal beams realized in wood, the structural system has a hybrid nature, with all the antenna and the transversal links on the deck being realized in steel. The experimental results reported in this paper confirmed by the low sensitivity of the dynamic response to temperature variations, i.e., the prevalence of the steel skeleton on determining the footbridge vibration.

5. Acknowledgements

The cooperation of the designer, Dr. Ing. Stefano Boranga, and the timber elements producer Rubner Holzbau S.p.A. of Bolzano, Italy, is acknowledged for providing them access to the technical documents. Athenaeum grants from the Universities of Catania and Pavia are eventually acknowledged.

References

[1] G. Giordano, A. Ceccotti and L. Uzielli, *Tecnica delle Costruzioni in Legno* [in Italian], Hoepli, Milano, Italy, 1999, 850 pp.

[2] M. Piazza, R. Tomasi and R. Modena, *Strutture in Legno* [in Italian]. Hoepli, Milano Italy, 2005, 736 pp.

[3] S. Casciati and M. Domaneschi, Random imperfection fields to model the size effect in laboratory wood specimens, *Structural Safety*, 29 (4), 2007, pp. 308-321.

[4] H. Wenzel, *Health monitoring of bridges*, John Wiley & sons, Chichester, 2009, 621 pp.

[5] C.F. Beards, *Structural vibration: analysis and damping*, Arnold, London, 1996, 276 pp.

[6] Norme tecniche per le costruzioni - D.M. 14 Gennaio 2008 (NTC2008) [In Italian].

[7] http://www.mscsoftware.com.

[8] Sara Casciati, Lucia Faravelli, Daniele Bortoluzzi, *Pedestrian Timber Bridges: Experimental Investigation and Modelling*, The 10th International Conference on Damage Assessment of Structures, Dublin, Ireland, 8-10 July 2013.

[9] Sara Casciati, Lucia Faravelli, Daniele Bortoluzzi, *Human induced vibrations in a pedestrian timber bridge*, The 4th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN2013), Kos, Greece, 12-14 June 2013.

[10] S. Casciati and Z. Chen, *A multi-channel wireless connection system for structural health monitoring applications*. Structural Control and Health Monitoring Journal, 18 (5), 2011, pp. 588–600.

[11] C.M. Casado, I.M. Diaz, J. de Sebastian, A.V. Poncela and A. Lorenzana, Implementation of Passive and Active Vibration Control on an In-service Footbridge, *Structural Control and Health Monitoring*, 20,2013, pp. 70-87.

Field Performance of Timber Bridges: A National Study

Brian K. Brashaw
Program Director
University of Minnesota Duluth, Natural Resources Research Institute
Duluth, MN, USA
bbrashaw@nrri.umn.edu

Brian Brashaw serves as the Director of the Wood Materials and Manufacturing Program, with and emphasis on non-destructive inspection technologies for timber structures including buildings and bridges.

James Wacker
Research Engineer
USDA Forest Service, Forest Products Lab, Madison, WI, USA
jwacker@fs.fed.us

Jim Wacker has focused his research efforts mainly on timber bridge inspection, durability, engineering, design and performance monitoring. He is the current chair of the ASCE Bridge Technical Committee.

Frank Jalinoos
Federal Highway Administration Office of Infrastructure R&D
McLean, VA, USA
frank.jalinoos@dot.gov

Frank Jalinoos is a structural research engineer on the Hazard Mitigation Team and serves as the FHWA lead for this cooperative research project.

Summary

As many engineers begin to implement life cycle cost analyses within the preliminary bridge design phase, there is a significant need for more reliable data on the expected service life of highway bridges. Many claims are being made about the expected longevity of concrete and steel bridges being 75 years or more, but few are based on actual performance data. Because engineers are least familiar with timber bridges, their expected longevity is often unfairly estimated at 20 years. A national scale project has been developed for the United States, headed by the USDA Forest Products Laboratory and the U.S. Federal Highway Administration. In this project, national cooperators including Federal Highway Administration, the U.S. Forest Service, Louisiana Department of Transportation, University of Minnesota Duluth, Iowa State University, Mississippi State University, Laminated Concepts, Inc. and Tom Williamson Timber Engineering, LLC will each inspect 15-25 timber bridges, using visual, probing, stress wave, and resistance microdrilling. The study results will help to provide a better understanding of the design, performance, and durability characteristics of timber bridge structures, which can improve future bridge design and preservation practices and ultimately extend service life. Lastly, the findings should assist with timber bridge service life expectancy when compared with alternative bridge materials.

Keywords: timber bridge, inspection, nondestructive evaluation, service life

1. Introduction

1.1 Background

As many engineers begin to implement life cycle cost analyses within the preliminary bridge design phase, there is a significant need for more reliable data on the expected service life of highway bridges. Many claims are being made about the expected longevity of concrete and steel bridges being 75 years or more, but few are based on actual performance data. Because engineers are least familiar with timber bridges, their expected longevity is typically unfairly estimated at 20 years.

Additional research is needed on a national scale that provides more reliable data about the true longevity of timber bridges in the U.S. and allows for more accurate life-cycle assessments.

In order to generate more quantitative and unbiased bridge performance data, the Federal Highway Administration (FHWA) recently launched an initiative called the Long-Term Bridge Performance Program (LTBP). The LTBP is national program and includes detailed inspection, periodic evaluation and monitoring of approximate 200 bridges over a 20-year period [1]. The LTBP program concentrates on "work-horse" highway bridges. This includes steel, reinforced and pre-stressed concrete bridges of stringer/multi-beam or girder, multiple box beam or girders and deck design types constituting 75% to 80% of the National Bridge Inventory (NBI). A representative sample of bridges will be evaluated in a cluster/reference bridge sampling method. Each reference bridge will anchor a cluster of 5 or more bridges that are within a small geographical location (along linear highway corridor or scattered about a geographic region approximately 30 miles wide) and subject to similar climate and traffic conditions. The reference bridges will involve 1) detailed and arm-length visual inspection (VI); 2) advance nondestructive evaluation (NDE); 3) global testing (load testing, modal testing, and continuous monitoring); and, 4) destructive material sampling. The cluster bridges will involve arm-length VI. The LTBP has developed an open, scalable, and extensive data management system called the "Bridge Portal". This database is capable of integrating LTBP data with other sources, such as NBI, PONTIS, weather, and traffic. Unfortunately, the LTBP currently does not include timber bridges in their current scope. This research study was developed to generate more reliable bridge performance data and establish a framework for long-term timber bridge performance monitoring. This background information will be needed if timber bridges are to be incorporated into the LTBP in the future.

1.2 Timber Bridge Statistics

Timber bridges are an indispensable component of the U.S. highway system especially on secondary rural roadways. The current December 2010 National Bridge Inventory (NBI) database [2] includes 604,426 bridge structures (including culverts) with a span length greater than 20 ft (6 m), with approximately nearly 8 percent utilizing timber as a primary structural component for the superstructure (Table 1). Many of these structures have been in-service for decades and have performed well structurally, but have been deemed structurally deficient or functionality obsolete. In addition, many bridge inspectors are not as familiar with timber as a bridge material and tend to downgrade their condition rating in the NBI. The net result is that many timber bridges have been assigned poor load ratings, or posted for restricted loads, when their actual condition is satisfactory. The long-range impact is that many engineers hold the misconception that timber is a low performance bridge material having an estimated service life of less than 20 years.

Table 1 – NBI bridge statistics as of December 2010.

Concrete	Steel	Timber	Masonry	Aluminum	Other	Total
391,161	185,148	24,267	1,743	1,459	648	604,426

All timber superstructure	Timber deck on steel stringers	Total
24,267	24,492	48,759 (8% of total)

There are promising examples of length-of-service that support the long-term durability of timber bridges. The Forest Service (FS) maintains nearly 3,000 timber road bridges in their transportation network. Many of them are sawn timber superstructures that were installed in the post-WWII era and are still in-service after several decades. The FS also has a fairly large population of early glulam bridges built in the 1950s that are still providing vital transportation links in the Pacific Northwest. The railroads have used timber components for bridges for over 100 years and have several sawn timber

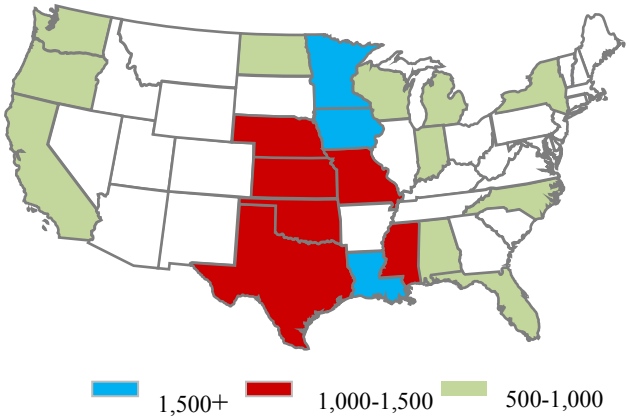


Figure 1 – States with a significant number of timber bridges in their bridge inventory.

structures that have been in-service for more than 75 years. However, what is lacking in the literature is a scientific study on the long-term performance of timber highway bridges in the U.S.

The geographical distribution of the NBI inventory data shows that 19 States have greater than 500 bridges (Figure 1). States with more than 1,500 timber bridges include Louisiana, Iowa, and Minnesota. States having 1,000–1,500 timber bridges include Texas, Oklahoma, Mississippi, Kansas, Missouri, and Nebraska. States having 500–1,000 bridges include North Dakota, California, Oregon, Washington, Wisconsin, Michigan, Indiana, Alabama, New York, North Carolina, and Florida. The remaining 31 States have less than 500 timber bridges in their respective inventory.

There are a variety of timber bridge superstructure types. The main categories are longitudinal decks, stringer systems, and others. Longitudinal decks include nail-laminated, spike-laminated, stress-laminated (using either sawn lumber or glulam laminations), and longitudinal glulam bridges. The spike-laminated and longitudinal glulam deck superstructures consist of partial width panels that are interconnected with transverse stiffener beams attached at intervals along the bridge length. Stringer systems include sawn lumber, glulam, steel, and fiber-reinforced polymer (FRP) glulams. Other timber bridge superstructures include arches, trusses, and cable-supported structures. Photographic examples of several timber bridge superstructure types are provided in Figure 2.

Nail-laminated deck.

Glulam stringer and deck.

Longitudinal glulam deck.

Stress-laminated deck.

Glulam deck arch.

Glulam suspended arch.

Figure 2 – Photographic examples of various timber bridge superstructures.

1.2 Objective and Scope

The primary objective of this study was to assess the condition and performance of approximately 100 existing timber highway bridges representing the main climate regions in the continental United States. A secondary objective of this study was to establish the baseline framework for evaluating future performance of timber bridges nationwide. The study results will help to provide a better understanding of the design, performance, and durability characteristics of timber bridge structures, which can improve future bridge design and preservation practices and ultimately extend service life. Lastly, the findings should assist with timber bridge service life expectancy when compared with alternative bridge materials.

2. Research Methods

2.1 Project Team Overview

The overall project was jointly managed by James Wacker of the Forest Products Laboratory (FPL) and Frank Jalinoos of the Federal Highway Administration (FHWA). The regions that were

selected for bridge inspection were based on the American Wood Protection Association (AWPA) wood decay hazard map shown in Figure 3. In order to assess these regions, a diverse project team was created that included: Tom Williamson Timber Engineering LLC, Rogue River-Siskiyou National Forest, Iowa State University, University of Minnesota Duluth, Mississippi State University, Louisiana Transportation Research Center, and Laminated Concepts Inc. Each organization represented a U.S. region and would be responsible for inspecting bridges in their respective regions. Figure 3 also shows the general geographic region for each organization.

A project kickoff meeting was held in Madison, Wisconsin and included several presentations and discussions with a goal of providing project information on bridge selection, inspection protocols, safety guidelines, and data processing for the project. All teams received information on inspection protocols and equipment. An inspection demonstration was completed on a timber bridge.

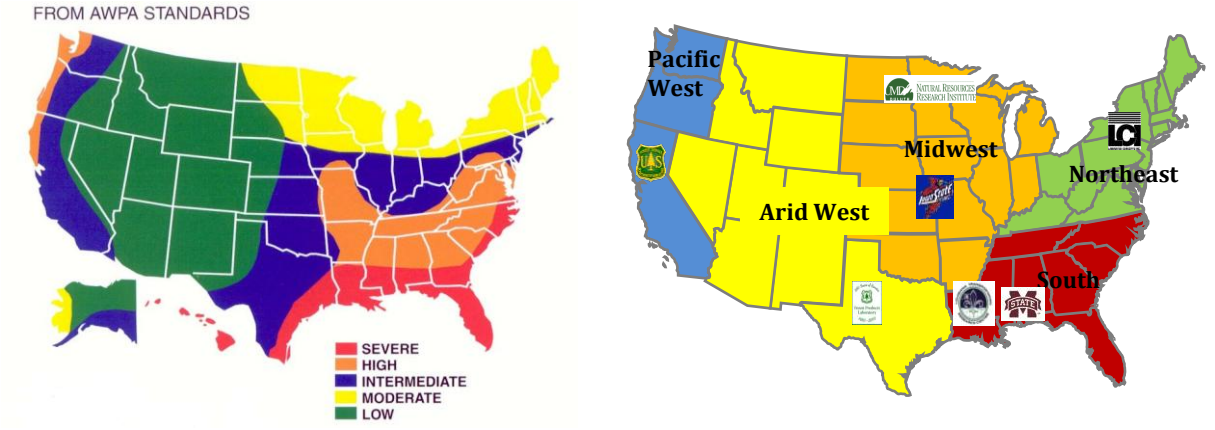


Figure 3 – Wood decay hazard map and team inspection assignments in the continental U.S.

2.2 Bridge Selection

Team members were asked to assess the inventory of timber bridges in their region and select candidate bridges for field evaluations. Teams were required to select timber bridge clusters within a multi-county region that had the same superstructure design type and were preferably of similar age. Each project team selected 15-25+ bridges for inspection. This list was reviewed and approved by the project coordinators and used to assemble a national perspective for the candidate bridge selections and determine age and bridge type overview.

Teams used several options for identifying candidate bridges within their region. The National Bridge Inventory (NBI) [2] should be the primary source for identifying timber bridge populations by state and county. A good resource for navigating the NBI database is available at the National Bridges internet portal [3]. It is a non-governmental website based on December 2012 NBI figures and allows for rapid data sorting and downloading to spreadsheets for additional analysis. Another effective method was to contact specific counties that have a sizable population of timber bridges directly. In most cases, they have developed their own bridge database separate from any NBI requirements.

The criteria for the selection of timber bridges included the following requirements:

- Bridge superstructure must consist of timber as structural component;
- The timber bridge must be located on a public roadway;
- The timber bridge must have been in-service for more than 15 years;
- Background files must include adequate records about the design, inspections, and major repairs;
- Accessibility for conducting an “arm’s length” bridge inspection in a safe manner;
- In selecting reference bridge clusters, vicinity to a WIM station for ADT data and to a weathering station for temperature/moisture records is desirable.

Teams were encouraged to conduct pre-inspection site visits to confirm the inspection location and understand the site conditions include key parameters like traffic conditions, depth of water and vertical distance from bridge underside to water.

2.3 Bridge Inspection

Each team conducted their bridge inspection using the same protocol that was provided during the kick-of meeting and in the detailed study plan. This ensured that a reliable dataset was produced regarding the performance of timber bridges. The recommended protocols for pre-inspection, general inspection, and NDE inspection are described below.

2.3.1 Pre-Inspection Protocol

Prior to inspection, a careful review of design plans and any prior inspection and maintenance work was recommended. This identified bridge areas that have been problematic in the past. These areas were then a focus area during the project inspection. Teams were also encouraged to interview the bridge owner about the bridge and any previous problems noted and to obtain previous reports.

Methods for recording inspection work included digital photographs, sketches/diagrams, and tabulated NDE data. Based on the NBI data or design drawings (or recent as-built drawings, if available) it was recommended that schematic drawings (plan, profile, and section) were developed along with tabulated data sheets. Tracking the location of data collection points was critically important. Photos were taken with high-resolution digital cameras and not compressed when inserted into the report files. A detailed spreadsheet template was provided for all inspection teams to assist with creating consistent data. This spreadsheet had the following sheets: inspection checklist, bridge information, raw data and sketches, bridge photos, reduced data, NBI ratings, inspection and maintenance history.

2.3.2 Inspection Protocol

Inspection procedures included visual observations and supplementary NDE tools including hammer, probe, moisture meter, stress wave timer, and resistance microdrill. Detailed information about these procedures is noted in section 2.4 of this paper. All data was documented through onsite sketches, photos and video.

2.3.3 Topside Inspection Process

Specific field tasks were provided for the topside inspection.

1. Record the bridge location and orientation of the timber bridge. Drawings should be marked for principal directions and stream flow.
2. Visually inspect wearing surface for problems.
3. Visually inspect the rail and curb system for problems.

2.3.4 Bottom side Inspection Process

Specific field tasks were provided for the bottom side inspection.

1. Label bridge components prior to inspection and photographic documentation. Abutments should be labeled BOB (beginning of bridge) and EOB (end of bridge). Girders should be labeled A, B, C, etc. with A on the upstream side of the bridge.
2. Conduct an initial visual assessment and use a hammer for “sounding” bridge components. Record locations with visible deterioration and mark those areas suspected of internal deterioration, so further investigation using NDE tools can be performed.
3. Conduct a survey of moisture contents in typical areas prone to decay (bridge ends, tops of stringers, etc.) to identify areas conducive to decay activity with mc levels above 20 percent. Readings should be taken at pin penetrations of 1-, 2-, and 3-inches at each location.
4. Record the types of preservative treatment used and describe how well it has performed. This information may be documented on bridge plans.
5. Establish baseline NDE data, by using the stress wave timer and resistance microdrill unit in an area of the bridge with no expected internal deterioration. This will be useful in comparing NDE data from other areas with various levels of internal deterioration.

6. Investigate all suspected areas using a two-step approach. Utilize the stress wave timer when you have access to opposite sides of a member. Record the transmission distance and the resulting transit time (microseconds). When stress wave data is significantly higher than baseline values, or when only one side of a bridge component is accessible, use the resistance microdrill unit. Make sure to record the location of all data points and drilling locations (and file no.) on drawing or sketches.

2.3.5 Performance Data and NBI Ratings

1. Identify performance issues that caused decay or poor performance addressing design type, construction quality, durability, serviceability, functionality, and structural integrity.
2. Determine possible causes for decay or poor performance.
3. Assign NBI ratings for the bridge deck and superstructure according to he NBI bridge condition code rating description shown in Table 2.

Table 2 – NBI bridge condition code ratings description.

FHWA - SI&A Sheet Condition Rating Code	FHWA - SI&A Sheet Condition Rating Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION - New or like new condition.
8	VERY GOOD CONDITION - No problems noted.
7	GOOD CONDITION - Some minor problems but no structural defects at critical locations (wood decay is a defect).
6	SATISFACTORY CONDITION - Structural elements show some minor defects and/or deterioration at critical locations. No measureable section loss.
5	FAIR CONDITION - All primary structural elements are sound but may have minor to moderate defects and/or deterioration with measurable section loss at critical locations. No significant reduction in primary structural member load carrying capacity.
4	POOR CONDITION - Primary structural elements show moderate to serious defects, deterioration, corrosion, cracking, crushing, and/or scour. Advanced section loss at critical locations. Diminished load carrying capacity of members is evident.
3	SERIOUS CONDITION - Serious and widespread defects have substantially reduced load carrying capacity of primary structural members. Local failures may be evident. Deflection/misalignment of members may be evident. Signs of severe structural stress are visible. Fatigue cracks in steel, shear cracks in concrete, and severe decay, checking, splitting, and crushing of beams or stringers in wood elements may be present.
2	CRITICAL CONDITION - Advanced deterioration of primary structural elements. Defects have now resulted in significant local failures. Scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	IMMINENT FAILURE CONDITION - Major deterioration or section loss present in critical structural components and/or obvious vertical or horizontal movements affecting structure stability. Bridge is/should be closed. However, correction action may put bridge back in light service.
0	FAILED CONDITION - Out of service. Beyond corrective action.

2.4 Nondestructive Inspection Techniques

Comprehensive inspection protocols for timber bridges include a wide variety of techniques to assess the condition of wood in service. Visual, moisture content, mechanical probing, resistance microdrilling and stress wave or ultrasound-based technologies are all used individually or in combination by inspectors [4], [5], and [6]. These techniques are based on solid technical information, supporting research, and field experience. The following sections provide background information for visual inspections, moisture content, mechanical probing, resistance microdrilling and stress wave or ultrasound-based technologies that were incorporated into inspection protocols.

2.4.1 Visual Inspection

The simplest method for locating deterioration is visual inspection. An inspector observes the bridge elements for signs of actual or potential deterioration, noting areas that require further

investigation. Visual inspection should never be the sole method used. Visual inspection requires strong light and is useful for detecting intermediate or advanced surface decay, water damage, mechanical damage, or failed members. Visual inspection cannot detect early stage decay, when remedial treatment is most effective. During an inspection, the following signs of deterioration should be investigated: fruiting bodies, sunken faces or localized collapse, staining or discoloration, insect or animal activity, plant or moss growth, checks and splits, weathering or impact damage, or rotation of members. Figure 4 shows several visual indicators evident during bridge inspection.

2.4.2 Drilling, Coring and Probing Techniques

Simple mechanical tests are frequently used for in-service inspection of wood members in timber bridge structure. For example, probing or drilling-type tests are used to examine the condition of a structural member that is identified as potentially having decay or deterioration. Further, the use of probes can allow the inspector to determine the depth of any checks or splits. The underlying premise for such tests is that degraded wood is relatively soft and will have low resistance to probe penetration or withdrawal.

Figure 4 – Visual inspection conditions including discoloration and staining, insect activity, mechanical damage and build-up of sand and gravel.

Probing with a moderately pointed tool, such as an awl or knife, locates decay near the wood surface as indicated by excessive softness or a lack of resistance to probe penetration and the breakage pattern of the splinters. A brash break indicates decayed wood, whereas a splintered break reveals sound wood. Although probing is a simple inspection method, experience is required to interpret results. Care must be taken to differentiate between decay and water-softened wood that may be sound but somewhat softer than dry wood. Probes can also be used to assess the depth of splits and checks in timber bridge elements. Flat bladed probes like pocket knives or feeler gauges are recommended for use in this process. It is important to understand the impact of checks and cracks in other advanced techniques such as stress wave inspection.

Another drilling technique that has been commercially developed is the resistance drill system. This system was originally developed for use by arborists and tree care professionals to evaluate the condition of urban trees and locate voids and decay. It is now being utilized to identify and quantify decay, voids, and termite galleries in wood stringers, columns, poles, and piles. This technique is now the preferred drilling and coring technique for timber bridge elements.

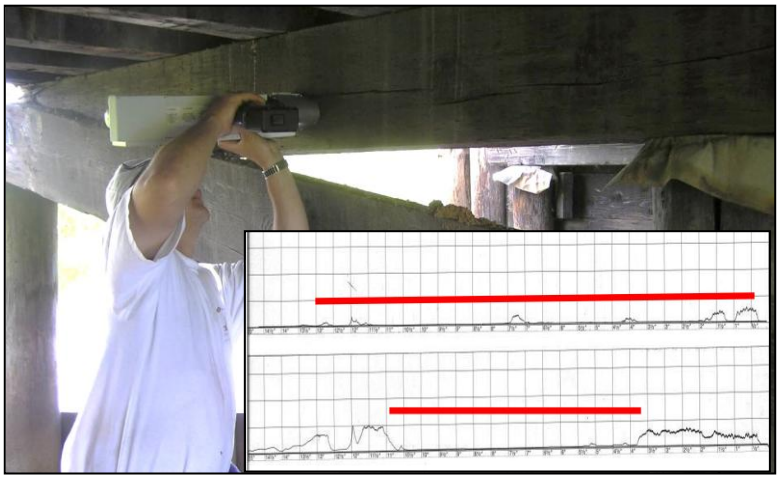


Figure 5 – Resistance microdrilling techniques were used to inspect timber bridge elements.

The resistance drill system measures the resistance of wood members to a 1.5-mm drill bit with a 3.0-mm head that passes through them. The drill

bit is fed at a fixed movement rate allowing the inspector to determine the exact location and extent of the damaged area. This system produces a chart showing the relative resistance over its travel path. This chart can be produced either as a direct printout or can be downloaded to a computer. Areas of sound wood have varying levels of resistance depending on the density of the species and voids show no resistance. The inspector can determine areas of low, mild, and high levels of decay. Figure 5 shows a timber abutment cap being assessed with a resistance microdrill and the resulting chart image showing minimal drilling resistance that indicating that the majority of the cap is decayed.

2.4.3 Moisture Content Inspection

Moisture meters can effectively be used in conducting inspections of timber bridge elements. It is well documented that the presence of moisture is required for decay to occur in timber. Typically, moisture conditions in timber of less than 20% will not allow decay to occur. However, as the moisture increases above 20%, the potential for decay to occur increases. Serious decay occurs only when the moisture content of the wood is above 28-30%.

2.4.4 Stress Wave Inspection

Stress wave timing is an effective method for locating and defining areas of decay in timber bridges. Stress wave propagation in wood is a dynamic process that is directly related to the physical and mechanical properties of wood. In general, stress waves travel faster in sound and high quality wood than in deteriorated and low quality wood. By measuring wave transmission time through a timber bridge stringer, pile cap or piling in the radial direction, the internal condition of the structural element can be fairly accurately evaluated. As an introduction, a photograph and schematic of the stress wave concept for detecting decay in a timber piling are shown in Figure 6. A stress wave is induced by striking the timber member with an impact device or with an ultrasonic pulse instrumented with an accelerometer that emits a start signal to a timer. A second accelerometer, held in contact with the other side of the timber member, senses the leading edge of the propagating stress wave and sends a stop signal to the timer. The elapsed time for the stress wave to propagate between the accelerometers is displayed on the timer. This measured time, when converted to a transmission time on a per length basis (or wave propagation speed), can be used as a predictor of the physical conditions inside the timber bridge member.

Stress wave transmission times are shortest along the grain (parallel to fiber) and longest across the grain (perpendicular to fiber). For common timber bridge species Douglas fir and southern pine at dry conditions, stress wave transmission time is approximately 200 µs/m (60 µs/ft) parallel to grain, but ranges from 850 to 1000 µs/m (259 to 305 µs/ft) in the perpendicular or cross-grain direction. The presence of deterioration from decay can greatly affect stress wave transmission time in wood. Transmission times for decayed wood are much greater than that for nondecayed wood. For example, transmission time for nondegraded Douglas fir is approximately 800 µs/m (244 µs/ft), whereas severely degraded members exhibit values as high as 3,200 µs/m (975 µs/ft) or greater.

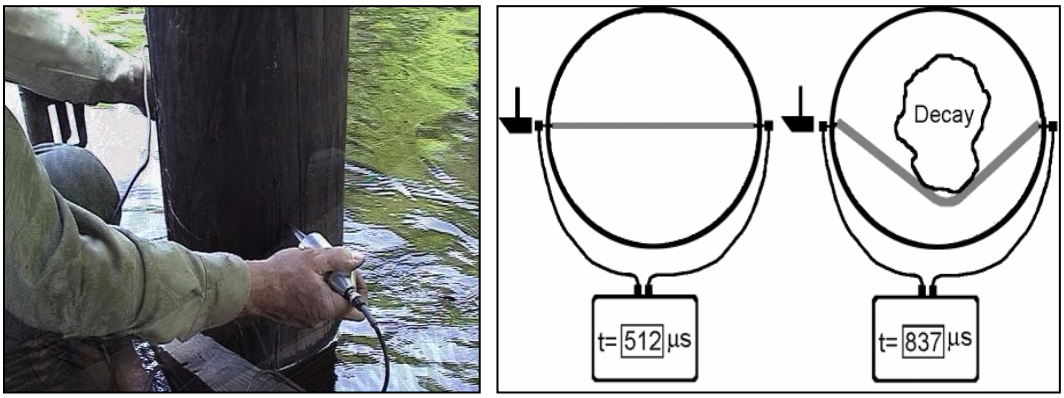


Figure 6 – A stress wave timer is used to inspect timber bridge elements to identify the presence of internal decay that is not visible.

3. Upper Midwest Inspection Region

3.1 Bridge Selection

The UMD project team was responsible for the upper Midwest. Minnesota has over 1,500 bridges and Wisconsin has between 500 and 1,000 bridges containing timber superstructure members. Both states are located in AWPA wood decay deterioration zone 3 - moderate. The project team selected bridges based on type and then focused the inspection in one -three adjoining counties. Specifically, the team selected solid sawn timber stringer, glulam timber stringer, steel stringer with timber deck, and spike-laminated deck bridge systems for inspection. The solid sawn timber stringer bridges were located in two separate geographic locations. One set was owned and maintained by the USDA Forest Service Chequamegon-Nicolet National Forest in northern Wisconsin. The second set was owned and maintained by Freeborn, Faribault and Jackson Counties in southern Minnesota. The glulam timber stringer bridges were owned and maintained by Faribault County in southern Minnesota, while the steel stringer timber deck and spike laminated bridges were owned and maintained by St. Louis County in northern Minnesota. The average climate data for northern Wisconsin shows annual precipitation of 33-34 inches [7]. The normal annual precipitation amounts for northern Minnesota are 28-30 in and from 29-33 in. in southern Minnesota [8]. Table 3 provides detailed information about the bridges selected for inspection.

Table 3 – Timber bridges inspected in the upper Midwest region during the project.

County/ ID	Superstructure Type	Year Built	Span/ Lengths m (ft)	Average Daily Traffic	Width m (ft)	Wearing Surface	Substructure Material
Freeborn/ 8587	Solid sawn timber stringer	1946	6.0 (19.8)	51	7.4 (24.4)	Gravel over timber deck	Timber piling
Faribault/ 22512	Solid sawn timber stringer	1967	9.1 (30)	67	8.4 (27.5)	Bituminous over timber deck	Timber piling
Jackson/ 5937	Solid sawn timber stringer	1940	9.4, 9.4, 9.4 (31)	1,050	8.5 (28)	Bituminous over timber deck	Timber piling
Jackson/ 5938	Solid sawn timber stringer	1940	9.4 (30), 10.4 (34)	1,050	8.5 (28)	Bituminous over timber deck	Timber piling
Taylor/FS 108-05.3	Solid sawn timber stringer	1980	7.3 (24)	50	7.9 (26)	Timber plank	Timber piling
Taylor/FS 112-4.1	Solid sawn timber stringer	1953	5.2 (17)	30	4.6 (15)	Timber plank over laminated deck	Timber piling
Price/FS 147-1.3	Solid sawn timber stringer	1956	4.9 (16)	20	4.6 (15)	Timber plank	Timber piling
Ashland/ FS/ 164-24.3	Solid sawn timber stringer	1953	7.7 (25.3)	20	4.9 (16)	Timber plank	Timber piling
Faribault/ 22508	Glulam timber stringer	1968	10.2 (33.5)	95	10.1 (33.3)	Bituminous over timber deck	Timber piling
Faribault/ 22514	Glulam timber stringer	1968	12.2 (40)	35	7.9 (26)	Gravel over timber deck	Timber piling
Faribault/ 22518	Glulam timber stringer	1969	11.7 (38.5)	70	10.1 (33.1)	Gravel over timber deck	Timber piling
Faribault/ 22519	Glulam timber stringer	1969	10.2 (33.5)	539	9.7 (32)	Bituminous over timber deck	Timber piling
Faribault/ 9967	Glulam timber stringer	1951	11.0 (36.2)	175	8.4 (27.4)	Bituminous over timber deck	Timber piling
St. Louis/ 540	Spike laminated deck	1991	5.4, 5.4, 5.4 (17.8)	519	11.6 (38)	Bituminous over timber deck	Steel H piling

St. Louis/ 497	Spike laminated deck	1988	4.3, 5.5, 4.3 (14, 18, 14)	230	10.4 (34)	Bituminous over timber deck	Cast in place (CIP)
St. Louis/ 221	Spike laminated deck	1981	5.3, 7.0, 5.3 (17.5, 23, 17.5)	95	10.4 (34)	Bituminous over timber deck	Cast in place (CIP)
St. Louis/ 611	Spike laminated deck	1985	6.6, 6.6, 6.6 (21.8)	245	10.4 (34)	Bituminous over timber deck	Timber piling
St. Louis/ 92	Steel stringer timber deck	1978	8.7 (28.6)	80	8.7 (28.5)	Bituminous over timber deck	Timber piling
St. Louis/ 185	Steel stringer timber deck	1983	10.7 (35)	40	9.1 (30)	Bituminous over timber deck	Timber piling
St. Louis/ 194	Steel stringer timber deck	1977	9.5 (31)	35	7.3 (24)	Bituminous over timber deck	Timber piling
St. Louis/ 197	Steel stringer timber deck	1971	9.5 (31)	35	7.3 (24)	Bituminous over timber deck	Timber piling
St. Louis/ 811	Steel stringer timber deck	1987	10.0 (32.8)	4	6.7 (22)	Timber deck	Timber piling

3.2 Inspection Results

Detailed inspections were completed using visual, moisture content, mechanical probing, resistance microdrilling and stress wave or ultrasound-based techniques. Sketches were completed on site for each bridge inspected and they were later converted to AutoCAD images for record keeping. High resolution digital photographs were taken of the bridge with special emphasis on deteriorated or damaged areas. Raw data from stress wave timer and resistance drill were processed and recorded. Information was obtained from the bridge owner on previous inspections with special emphasis on any repairs or modifications to the bridge. Finally, the inspection team rated the bridge according to NBI ratings.

3.2.1 Solid Sawn Timber Stringer Bridges

Two sets of solid sawn timber stringer bridges were inspected. One group was located in northern Wisconsin and owned and maintained by the US Forest Service Chequamegon-Nicolet National Forest. The superstructures for these bridges were solid sawn, creosote treated Douglas fir stringers, with most having timber decking and timber rails. The NBI ratings assigned by the UMD NRRI team are shown in Table 4. The data for each bridge is reported along with the mean ratings for the group. It was found that the vast majority of the superstructure stringers were in satisfactory to good condition. There was minimal evidence of decay or deterioration in the stringers, but there was cracking and checking present in the outermost stringers that were exposed to weathering. Often, there was frequent mechanical damage to the railings caused by snowplows, road graders or other vehicles.

Table 4 – NBI condition ratings for the solid sawn stringer bridges inspected in northern Wisconsin.

NBI Condition Rating	Bridge Number				Group Mean
	USFS/ 108-5.3	USFS/ 112-4.1	USFS/ 147-1.3	USFS/ 164-24.3	
Deck	5	4	6	5	5
Superstructure	7	6	6	6	6.3

The second set of solid sawn timber stringer bridges was inspected in southern Minnesota. Three of the four bridges had a bituminous wear layer over timber decks and one had gravel over a timber deck. The decking was in generally good condition, with some deterioration and cracking in the bituminous wear layer, allowing for water infiltration. The solid sawn stringers were solid sawn, creosote treated Douglas fir stringers. The majority of the stringers were in good to satisfactory condition, but Freeborn 8587 had several significantly decayed girders. This bridge is slated for replacement in 2014. The NBI ratings assigned by the UMD NRRI team are shown in Table 5.

Table 5 – NBI condition ratings for the solid sawn stringer bridges inspected in Minnesota.

NBI Condition Rating	Bridge Number				Group Mean
	Freeborn/ 8587	Faribault/ 22512	Jackson/ 5937	Jackson/ 5938	
Deck	5	6	7	7	6.3
Superstructure	5	6	7	7	6.3

3.2.2 Glulam Stringer Bridges

A set of 5 glulam stringer bridges was inspected in Faribault County, Minnesota. Most of these single span glulam bridges were constructed in late 1960s and had creosote treated southern yellow pine stringers. These stringers did not contain modern fingerjoint construction within each laminate layer, but a modified scarf joint. Each of the bridges had a nail laminated timber deck with a bituminous wear layer. These stringers were found to be in excellent condition as inspected using visual, sounding, and stress wave inspections. The bituminous wearing surfaces showed cracking, allowing water to infiltrate through the deck. There were no high moisture conditions identified in the glulam stringers. The NBI ratings assigned by the UMD NRRI team are shown in Table 6.

Table 6 – NBI condition ratings for the glulam stringer bridges inspected in southern Minnesota.

NBI Condition Rating	Bridge Number					Group Mean
	Faribault/ 22508	Faribault/ 22514	Faribault/ 22518	Faribault/ 22519	Faribault/ 9967	
Deck	7	6	7	6	7	6.6
Superstructure	7	7	7	7	6	6.8

3.2.3 Spike Laminated Deck Span Bridges

Four spike laminated deck span bridges were inspected in St. Louis County, Minnesota. These bridges had been mostly constructed in the 1980s. These bridges were constructed of panels manufactured by Wheeler Consolidated. The main structural material was creosote treated Douglas fir. The bridges all had a bituminous wear layer over the spike-laminated deck. The wear layers showed significant cracking and deterioration in some cases. However, there was no decay or deterioration noted in the inspection of the superstructure. The railing systems showed some damage and deterioration from snowplows, road graders or other vehicles. The NBI ratings assigned by the UMD NRRI team are shown in Table 7.

Table 7 – NBI condition ratings for the spike laminated bridges inspected in northern Minnesota.

NBI Condition Rating	Bridge Number				Group Mean
	St. Louis/ 540	St. Louis/ 497	St. Louis/ 221	St. Louis/ 611	
Deck	5	5	5	4	4.8
Superstructure	7	7	7	5	6.5

3.2.4 Steel Stringer Timber Deck Bridges

Five steel stringer timber deck bridges were inspected in St. Louis County, Minnesota. These single span bridges were constructed from 1971 - 1987. The steel stringers were the main structural superstructure and four of the bridges had a nail laminated treated timber deck with a bituminous wear layer. The fifth bridge had a nail laminated timber deck without a bituminous wear layer. The bridges were in satisfactory to good condition according to the NBI condition rated completed. The bituminous wear layer showed cracking and deterioration on most bridges. Visual inspection, hammer sounding and probing of the decking showed it to be in satisfactory condition. The steel stringers showed some peeling paint and minor rust. The NBI ratings assigned by the UMD NRRI team are shown in Table 8.

Table 8 – NBI condition ratings for the steel stringer timber deck bridges inspected in Wisconsin.

NBI Condition Rating	Bridge Number					Group Mean
	St. Louis/ 92	St. Louis/ 185	St. Louis/ 194	St. Louis/ 197	St. Louis/ 811	
Deck	7	6	6	6	6	6.2
Superstructure	8	7	7	7	7	7.2

4. Conclusions

The purpose of this project was to create a timber bridge assessment project resulting in the inspection of approximately 100 timber vehicle bridges over 15 years old. These bridges were to be inspected using visual, probing, moisture content, stress wave, and resistance microdrilling. Completion of this work will help provide a better understanding of the design, performance, and durability characteristics of timber bridge structures, leading to improvements in future bridge design and preservation practices and ultimately extended service life. The following conclusions were evident from the testing, analysis, and data interpretation of the project to date:

- Approximately 100 bridges were inspected across the U.S. that had structural timber in their superstructure. Detailed arms length inspections were completed using visual, sounding, probing, moisture content, stress wave timing and resistance drilling techniques. Varied performance was identified for these bridges based on age, AWPA wood decay zone, and maintenance level.
- The majority of bridge superstructures inspected were stringer systems with solid sawn timbers. Additional groups of bridges assessed included glulam stringer, steel stringer timber deck, and longitudinal deck systems. Decks systems included sawn, nail laminated and glue laminated timber decks.
- Ongoing assessment and interpretation of the project results will be completed and reported in the next 9 months.

5. Acknowledgements

This study was funded and conducted under a joint agreement between the Federal Highway Administration – Turner Fairbank Highway Research Center, and the Forest Service – Forest Products Laboratory. Appreciation is expressed to all of the cooperative teams including USDA Forest Products Laboratory, Tom Williamson Timber Engineering LLC, Rogue River-Siskiyou National Forest, Iowa State University, University of Minnesota Duluth, Mississippi State University, Louisiana Transportation Research Center, and Laminated Concepts Inc. Bridge owners including State Departments of Transportation, County Public Works Departments and the U.S. Forest Service provided important assistance for this project.

6. References

[1] U.S. Federal Highway Administration. 2013. Long Term Bridge Monitoring Program. <http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/structures/ltbp/>

[2] U.S. Department of Transportation. 2013. Bridges and Structures, Tables of Frequently Requested NBI Information. <http://www.fhwa.dot.gov/bridge/britab.cfm>.

[3] National Bridges. 2013. The National Bridge Inventory Database. <http://www.nationalbridges.com>

[4] Ross, R.J., B.K. Brashaw, X. Wang, R. White, and R.F. Pellerin. 2004. Wood and Timber Condition Assessment Manual. Forest Products Society, Madison, WI. 73 p

[5] Ross, Robert J.; Pellerin, Roy F.; Volny, Norbert; Salsig, William W.; Falk, Robert H. 1999. Inspection of timber bridges using stress wave timing nondestructive evaluation tools—A guide for use and interpretation Gen. Tech. Rep. FPL–GTR–114. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 15 p.

[6] Brashaw, Brian K.; Vatalaro, Robert J.; Wacker, James P.; Ross, Robert J. 2005. Condition Assessment of Timber Bridges: 1. Evaluation of a Micro- Drilling Resistance Tool. Gen. Tech. Rep. FPL-GTR-159. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 8 p.

[7] National Weather Service. http://www.crh.noaa.gov/images/mkx/climate/avg_30_year_precip.png

[8] Minnesota Climatological Working Group. http://www.climate.umn.edu/doc/historical/precip_norm.htm

KINGS COVERED BRIDGE REHABILITATION
SOMERSET COUNTY, PENNSYLVANIA

Authors: **William J. Collins, RLA, V.P., Principal Landscape Architect,**
Simone Collins Landscape Architecture, Berwyn, PA, USA
wcollins@simonecollins.com

William Collins, RLA is a principal at Simone Collins Landscape Architecture of Norristown, Pennsylvania and received his B.S. from Ohio State University. He is a member of the Timber Framers Guild of North America and managed the Kings Bridge design.

Samer H. Petro, PE, Manager,
Herbert, Rowland & Grubic, Morgantown, WV, USA
spetro@hrg.com

Samer Petro served as structural engineer of record for Kings Covered Bridge rehabilitation. He obtained his Bachelor's and Master's degrees in civil engineering from West Virginia University and serves as managing engineer at Herbert, Rowland & Grubic in Morgantown, WV.

***Fig.1**: Kings Bridge over Laurel Hill Creek after rehabilitation, east portal.*

Summary

Kings Bridge spans 116 feet over the Laurel Hill Creek. The original multiple kingpost truss structure was constructed circa 1857 by an unknown builder and retrofitted with nail-laminated arches circa 1900. Fortunately, the historic structure was bypassed with a steel bridge in the 1930s, and the original structural systems were never "modernized" like many timber bridges were to serve increased vehicular loads and highway traffic.

The total rehabilitation of Kings Covered Bridge in Somerset County, Pennsylvania was completed in 2008, culminating an eleven-year effort by the non-profit Southern Allegheny Conservancy (SAC) and a private-public partnership utilizing 100 percent federal funding from two Federal Highway Administration programs – the National Covered Bridge Program and the Transportation

Enhancements Program. The covered bridge boasts a colorful history and provides examples of rare structural systems that are now conserved, including an original lattice floor joist system under a restored deck. Most of the major historic members and much of the historic fabric were conserved in place. Arches that had been retrofitted after initial construction and tied to the lower chords of the original multiple kingpost truss system were rehabilitated. Using an innovative engineering strategy, arches were extended through the lower chords to create a hinged arch that serves like a Burr arch structure by bearing on the abutment.

The structure remains a museum-quality artefact for pedestrian use. Bridge ownership was transferred from the Southern Allegheny Conservancy (SAC,) the non-profit that managed the rehabilitation project to the local municipality after site improvements for visitors were completed in spring 2009. The Kings Bridge site now serves as one two "twin covered bridges" in municipal parks – connected by a bike route to the Barronvale Bridge located one mile upstream in Middlecreek Township.

Keywords: *Multiple Kingpost Truss, Burr Arch Truss, retrofitted arch, Somerset County, Pennsylvania, FHWA, PennDOT*

1. CHRONOLOGY of DEVELOPMENT & USE

From the 1930s until 2002, the Kings, a local farming family of Middlecreek Township, owned and maintained the bridge. The family retrofitted "Kings" Bridge to serve as a livestock barn over the water. Former gates and fences from this agricultural use were saved to re-install on the bridge after rehabilitation. Remnants of rubber tire hinges still exist on the lower downstream chord where the Kings hung a "floating fence" to prevent livestock from wandering up the creek in low flow periods.

Fig. 2 *Human and livestock gates at east portal. Note the diagonal repair rods through the truss braces, which were installed early in the twentieth*

Fig 3*: Same gates from outside.*

Project engineers agree that it was the King Family who was responsible for ensuring the bridge's survival by maintaining the roof and installing a remarkably astute homespun system of tension rods when both lower chords began to fail. During the rehabilitation, these rods were un-tensioned and ultimately left in place to acknowledge the family's interventions and to interpret the full structural history of the bridge, in keeping with the project preservation plan.

By 1997, both lower chords had failed completely, and only the arches and the repair rods installed by the Kings prevented the bridge from imminent collapse. In 2000, a temporary support system

was engineered and installed with two longitudinal, queenpost-tensioned trusses supported on timber crib towers. Transverse needle beams were installed between the two steel queenpost trusses below the upper chords to bear the suspended weight of the covered bridge until rehabilitation could be fully funded, and engineering and construction completed. The falsework remained in place through 2007 and served as the construction staging for the rehabilitation contractor, which enabled the work to be completed in place with minimal disturbance to Laurel Creek below.

The temporary stabilization system was engineered and installed, and a funding strategy was developed under an initial state-funded project for $90,000. By 2004, SAC and partners had secured $860,000 in federal funds to engineer and carry out a total rehabilitation project.

2. PHYSICAL DESCRIPTION of BRIDGE

The Kings Bridge that existed before 1997 was constructed in at least two phases. The construction of a multiple kingpost truss is estimated to date from the middle of the nineteenth century circa 1860 or earlier. The arches were added to the truss around 1900. The following description is of the extant structural system at the time of rehabilitation:

2.1 Trusses: The lengths of truss bays between posts were neither identical nor were they constructed using the typical geometric conventions of the time. Traditionally, the longer bays were near both ends, but in the Kings Bridge, the longer spans were found in the center bays where tension forces were greater in the lower chords. Several hand-hewn posts remained in the trusses, but most members were sawn – indicating later repairs. Tie beams were dropped into slots at the tops of posts and pinned with timber pins or “trunnels.” Rafter sills were bearing on the outrigger ends of the tie beams. Knee braces were joined using mortises between tie beams and posts. Horizontal X-bracing was alternately nailed and “let in” between the tie beams of each bent. Lower “needle” beams were added later to the posts, below the lower chords, suspended from the arches for supplemental support.

2.2 Arches: Nail-laminated arches, constructed of circular-sawn boards were not “let in,” but were bolted to truss members – indicating their later addition. The combined structure of trusses and arches was intended to carry the live load. Because the arches were tied to the lower chords (not hinged), they imparted a significant horizontal thrust into those members and contributed to the failure of both in locations weakened by water damage. These subsequent chord failures changed the distribution of the loads through the arches dramatically by visibly deforming them, but not to failure.

2.3 Struts: Instead of extending the arches to create a hinged arch, the early re-builders installed diagonal struts from the bottom chords (in the general location of the arch connection) to the abutment faces. These struts were similar to arch extensions in appearance but able to carry much less force. The wood struts decayed at their abutment seats and ultimately transferred no forces from the arch directly into the abutment. This resulted in greater forces imparted from the arches horizontally into the lower chords. The deteriorated lower chords failed, but the trusses tied to the arches did not fail, and instead spread each chord apart toward both ends. The abutments resisted these forces, but not without damage to the substructures that were originally built to receive only vertical loading from the multiple kingpost superstructure.

2.4 Substructure: The bridge was constructed on cut ashlar limestone abutments that had “seats” for later struts carved into their faces. As suspected, rehabilitation excavation revealed that the

abutments and wingwall substructures were a single course deep, and backfilled with rubble and compacted earth.

2.5 Joists / Floor; A rare, lattice system of floor joists was used below the timber deck, where one layer of diagonal joists was overlaid by a second layer at an opposite angle. These light, circular sawn 5 x 6s were bearing on two levels of individual ledger blocks nailed to the inside face of the lower chords. Overlapping joists were not fastened at their intersections like lattice trusses. The longer joist spans created by this design appear to have combined the purposes of transverse beams, longitudinal stringer beams, and under floor diagonal bracing into one system that carried floor loads and provided lateral bracing. The lower layer of decking was laid transverse, while the upper was laid longitudinally.

Fig. 4*: Lattice floor joist system and lower tie beam added after original construction.*

2.6 Sheathing: The trusses were clad with board and batten siding. The roof consisted of deteriorated asphalt shingles installed on top of deteriorated wood shingles on circular sawn nailers that had been mounted on sawn rafters fastened at the apex with trunnels. Wainscoting and a canted cap were in place to protect the lower area of the inside of the trusses from traffic debris.

3. DESCRIPTION of STRUCTURAL ASSESSMENT / ANALYSIS

Prior to the rehabilitation project, an engineering investigation of the structural conditions of the bridge was conducted. Inspection revealed that water damage posed the greatest challenge to the bridge in three structural areas.

3.1 Lower chords: Breaches had occurred in the corresponding bays on the opposite ends of each truss. These weaknesses (breaks or failures) were recognized in time by the Kings, who skillfully made a series of vernacular repairs, including metal rods, wood splints and iron brackets to keep the bridge standing. Removing wainscoting during the engineering investigation revealed that wood scabs had been added across the faces of truss posts and braces in the area of failure. Several areas of deterioration occurred in those sections of the lower chords fastened by traditional “lightning bolt” tension splices. This complicated the repairs by requiring removal of the entire length of a chord section or splicing in short new chord sections.

3.2 Truss Posts: Roof leaks above the lower chord failures also resulted in damage to joints and several truss posts and shouldered truss braces in Kings Bridge. The heads of two posts failed in shear at the joints due to excessive new loading patterns resulting from the lower chord failures. Several of the posts were found to be only partially damaged, while one required total replacement.

3.3 Arches / Struts: The arches were visibly deformed above the locations of lower chord failures, exhibiting the transfer of loads from the adjacent truss posts through the arch and back into the trusses and lower chords to the foundations. Struts were installed below the lower chords and were never sheathed by the bridge siding. The hearts of these untreated heavy timbers decayed at the bearing seats and remained in place suspended by metal tie rods only

3.4 Non-destructive testing: In 2004, the rehabilitation design team inspected the structure, assisted by personnel from the USDA Forest Service and the Forest Products Laboratory (FPL) in Madison, Wisconsin. An FPL engineer performed a series of non-destructive evaluation tests on various bridge truss members that were identified as deteriorated or potentially deteriorated based upon a visual condition assessment. The non-destructive testing (NDT) techniques included moisture content testing, stress wave analysis, and resistance drilling. Samples were retrieved from the structure in locations where member replacement was imminent, and those were also tested by FPL for species and strength. It was determined that white oak was the original species to be matched where replacement of structural members was required. All adjusted moisture content measurements were less than 16 percent, except for one test location that was slightly higher at 19 percent. These results indicated that the truss members were drier than the threshold moisture content level required for decay. Stress wave velocities ranged from 180-220 ft/μ-second and were near the threshold level for the presence of internal deterioration. Several micro-drill resistance measurements reported consistently low wood density and confirmed the stress wave measurements. Both truss members had a relative drilling resistance below 15 percent, with the interface between members visible at approximately 7.5 inches drilling depth.

3.3 Modeling: The arch-truss system of Kings Bridge was modeled and analysed, assuming a linear-elastic behaviour using STAAD structural analysis software. The geometry of the bridge was developed based upon centerlines of the members measured directly from the bridge in its existing state. Section and material properties were also used to describe the members. The material properties of the white oak truss members were determined from small scale laboratory testing conducted by the Forest Products Laboratory. The tests conducted by FPL in October 2004 also included various non-destructive techniques conducted on site on selected bridge members to determine moisture content, decay, and defects (i.e., checks, splits, etc). In addition, mechanical testing conducted by FPL on small-scale samples retrieved from the bridge determined species, specific gravity, moisture content, MOE, and MOR values. An MOE value of 1.4 x 106 lb/in2 was used in the analysis.

Due to the complexity of the geometry and variety of connections, a conservative approach was followed in the modeling. A primary issue was the behaviour of the joints at the intersections of verticals, diagonals, chord members, and the arches. The ends of the diagonals and the ends of the posts were assumed to be pinned (i.e., free to rotate). The chord members and the arches were assumed continuous, and all other joints were assumed fixed. This was considered to be a conservative approach and an accurate model of the bridge, and is in line with earlier analyses of Burr arch-truss systems. The supports of the truss were modeled as pinned at the left end and roller supported (resisting only vertical movement) at the other end. This was considered to be the most probable state of the original construction and to have generated the greatest forces in the bottom chord. The arches were modeled as pinned at both ends and resting in a corner of the stone abutment. The arch, which actually is a double arch (i.e., each half of the arch is composed of nine nail-laminated 2" x 4" members) straddling the truss members, was approximated by a series of twenty straight members and was modeled as a continuous member.

Dead loads were approximated by measuring timber dimensions on site, including truss members, top and bottom bracing, deck, roofing, siding, etc. These volumes were multiplied by a unit weight of white oak approximated at 43 lb/ft3 and placed at upper and lower chord panel joints in a manner approximating the actual loading conditions. Physical testing of small samples retrieved from Kings Bridge indicated that the specific gravities ranged from 0.59 to 0.68, which corresponded well with published values of density in the Wood Handbook. The live load used was a pedestrian load of 85 lb/ft2 as specified by AASHTO Guide Specifications for Design of Pedestrian Bridges published in 1997. The live load was divided between the two trusses and placed at the lower chord panel joints in a manner approximating the actual loading conditions. Since the AASHTO bridge specifications and PennDOT DM4 Bridge Design Manual do not adequately address the issue of wind load or snow loads for covered bridges, a conservative approach was followed in selecting snow and wind loads and load combinations. A snow load of 35 lb/ft2 was used and placed at the upper chord panel joints. A wind load pressure of 12.5 lb/ft2 based on ANSI/ASCE 7, Minimum Design Loads for Buildings and Other Structures, Section 6.4, Method I Simplified Procedure, corresponding to a basic wind velocity of 100 mph was used and divided between the upper and lower chord panel joints in a manner approximating the actual loading conditions.

In this analysis, a combination of dead, live, wind, and snow loads was followed. The load combination used was in accordance with ANSI/ASCE 7, Minimum Design Loads for Buildings and Other Structures (Section 2.4), Dead + (Wind + Live + Snow)*0.75, which is considered conservative for analysing extant covered bridges (according to the Federal Highway Administration's *Covered Bridge Manual*) A three dimensional (3-D) space frame model of the bridge was developed using STAAD with the loads applied at the top and bottom panel joints as per the ANSI/ASCE 7 load combinations. The versatility of the 3-D model made possible the addition of the top X-bracing members and their analysis for the lateral loads. The 3-D model also made possible a very realistic representation of the actual structure. The continuity of the top and bottom chords and the arches was preserved, although the bottom chord was severely damaged (i.e., ruptured) at two locations that were not modeled.

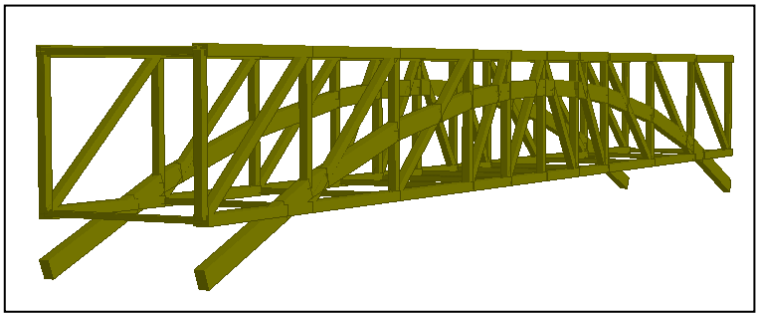


Figure 5*: 3-D Rendering (STAAD Model) of Kings Bridge*

The results from the STAAD analysis containing member forces, moments, and joint deflections were analysed with two objectives in mind: (1) to compare the computed stresses with allowable stresses (obtained from the National Design Specification (NDS) for wood construction) to ascertain the degree of safety of the member in question; and (2) to achieve some understanding of how the arch-truss system behaves, i.e., stress distribution and deflections. From the analysis it was determined that the most important structural characteristic of the arch-truss system of the Kings Covered Bridge when compared to the original multiple kingpost system was the stiffness associated with deflections. The addition of the Burr arch greatly reduced the deflection by a factor of approximately three under full load, suggesting the synergy of both the arch and truss producing a structure that is stiffer than the kingpost truss acting alone. For long span timber bridges, such as Kings Bridge, the arch provides a necessary stiffening of the truss so that deflections resulting from live and dead loads are controlled to acceptable limits. The addition of the arch also accomplished a reduction of member stresses and even the critical lower chord-arch joint member was satisfactory.

4. TREATMENTS / INTERVENTIONS

Rehabilitation methods and techniques were conceived and conducted in accordance with the Secretary of the Interior's Standards for the Rehabilitation of Historic Properties. The rehabilitation of Kings Bridge minimized interventions and required repairs in-place without dismantling and replacement of deteriorated members in-kind where possible.

The engineering strategy adapted conserved the remaining structural integrity of Kings Covered Bridge and focused on rehabilitating and stiffening the trusses by extending the arches to bear directly on the stone abutments while employing traditional nineteenth-century timber joinery methods such as "joggle" splice joints to replace deteriorated members. Epoxy adhesives and Glass Fiber Reinforced Polymer (GFRP) rebars and plates were also used to splice new members to existing ones, as well as to stiffen members. Only where rot damage was so severe that repairs could not be made in compliance with conservation best practices were irretrievable members replaced with other wood material. The first option for replacement timber was from salvaged members of the bridge that had been removed because they were deemed to be not historically significant and were not designated for repair. The rehabilitation project focused on the following areas:

4.1 Extend arches: The arches were extended below the lower chords by adding in-kind laminations to the existing nail-laminated arches. The cross-section area of the arch was removed from the corresponding section of each lower chord member to allow the new arch extension to bypass the chord unfastened.

***Fig. 6**: Arch "tied" to chord and strut below to abutment.*

***Fig 7:** Laminated arch extended to reinforced abutment.*

While the bridge was elevated for rehabilitation, the new arch extensions were installed and scribed to meet the new seats carved in the stone abutments before the bridge was lowered into place. The ends of the timbers were treated with wood preservative and elastomeric pads were used as compressive and thermal insulators between the stone and wood at the bearings. The new arch extensions were intended to remedy the damage to the trusses caused by the previous tied arch geometry. The deformed arch sections were separated, reshaped, and re-laminated in place. Repairs were also made by extending the laminations past the lower chords by dapping (notching) a portion of the bottom chord members. The new laminations follow the existing butt joint pattern and are also nail-laminated, stitched, and joined with epoxy. This repair includes the addition of a GFRP flitch plate with epoxy applied to both sides of the GFRP plate to reinforce the notched bottom chord.

4.2 Rebuild / reinforce stone abutments: A system of abutment reinforcements was designed to buttress the existing stone abutments. The abutment / wingwalls were completely excavated and

formed to receive flowable backfill to create the buttress that would withstand the new forces created by the extended arches. An estimated 20 percent of the substructure stone was removed and re-laid in the rehabilitation process. All former Portland cement grouting was removed, and all joints were re-struck with a softer mix using a greater percentage of lime to avoid spalling at the edges of the softer stone with the harder mortar. New white oak sills were installed to replace the previous sills that had deformed under the horizontal loads imparted after the chord breaches.

***Fig 8**: Excavated abutment before reinforcement.*

Replace posts, braces and chords (full and partial): Based on visual inspection, the non-destructive testing conducted by FPL, and the results of the structural analysis, several repair types were implemented. Glass Fiber Reinforced Polymer (GFRP) reinforcing rods embedded in epoxy were used to repair existing timber members where deterioration was such that an epoxy mortar repair was not appropriate, where a new section of wood was to be added to an existing original member, or where a fractured original member was restored. Another repair type intended to repair an existing fractured or missing timber section required the installation of a new white oak implant "joggle" at the splice location. This repair method represents a traditional timber joinery splice consistent with that period.

***Fig.9**: Joggle splice repair (above).*

***Fig. 10**: GFRP post repair (opposite).*

Restore other members and sheathing materials: The members of the lattice joist floor system were replaced in-kind. Some of the decking was reused as subflooring, but most of the remainder was replaced by rough-sawn hemlock boards. Both courses were laid transverse with staggered joints. Some rafters and all lath were replaced in-kind.

***Figure 11**: Interior rehabilitated.*

Rough-hewn cedar shingles were installed to the lath. Most of the siding was replaced in-kind, with some reused as battens. The wainscoting and cap material was stockpiled and then reinstalled.

Figure 12*: South elevation – rehabilitated.*

5. CONCLUSION

The abutments remained in place and a system of abutment reinforcements were installed to buttress the existing stonework. Archaeological al disturbance did not take place. The laminated arches were extended to the abutments and the bridge continues to operate as a self-supporting wood truss without modern support. The historic resource now serves as a pedestrian crossing of the Laurel Creek and is one half of a new "twin covered bridge" municipal park connected by a trail to the Barronvale Bridge one mile upstream, also owned by Middlecreek Township.

The visual character of the bridge was not compromised and all character-defining features were retained. The geometry of the bridge, such as the nail-laminated arches, was altered with minimal interventions to remedy fatal structural defects. The changes, such as the extension of the nail-laminated arches, were deemed structurally necessary to remedy the damage to the trusses caused by the previous arch installation. Every member of the truss was carefully examined and retained if possible. Any new materials match in-kind the materials they replaced or were based on historical antecedent. No changes were made that would create a false sense of historical development. All character-defining features were respected as products of their time within the period of significance. Glass Fiber Reinforced Polymer (GFRP) reinforcing rods were also used as a method of repairing existing timber members (*as opposed to total replacement*) where deterioration was found to such extent that an epoxy mortar repair was not appropriate and where a new section of wood was to be added to an existing original member.

The most important members of the truss were preserved. Replacement members were made from in-kind material and traditional craftsmanship including joinery techniques that replicated historic construction methods. Repair work was completed with in-kind materials and traditional craftsmanship techniques including joinery methods. Other non-traditional repair was carried out using GFRP rods to repair and strengthen existing members.

6. REFERENCES

AASHTO. *Guide Specifications for Design of Pedestrian Bridges*. American State Highway and Transportation Officials, Washington, DC, 1997.

American Forest and Paper Association. *National Design Specifications for Wood Construction (NDS), Supplement to the 1997 Edition*. Washington, DC, 1997.

ASCE. *Minimum Design Loads for Buildings and Other Structures*. American Society of Civil Engineers, Reston, VA, 2002.

Collins, William J., Samer Petro, et. al. "Kings Covered Bridge Rehabilitation, Somerset County, Pennsylvania." SEI Structures Congress, 2005, 1-10.

U.S. Department of Transportation, Federal Highway Administration. *Covered Bridge Manual*. Publication No. FHWA-HRT-04-098, McLean, VA, 2005.

7. REHABILIATION PROJECT TEAM

Rehabilitation Design:	Simone Collins Landscape Architecture
Structural Engineer:	Gannett Fleming Inc.
Stabilization Engineer:	DCF Engineering Inc.
Stabilization Contractor:	Arnold M. Graton Associates
Construction Contractor:	Allegheny Restoration
Technical Assistance:	USDA Forest Service
	Forest Products Laboratory
	Somerset County Conservation Service
Funding:	Federal Highway Administration
	PA Department of Conservation and Natural Resources
	PA Department of Community and Economic Development
	Somerset County
Project Partners:	Pennsylvania Department of Transportation
	Somerset County
	Middlecreek Township
	Rockwood Area Historical Society
Owner:	Middlecreek Township

THE TOHICKON AQUEDUCT:

Rehabilitating a Historic State Park Artefact

Authors: **William J. Collins, RLA, V.P., Principal Landscape Architect**
Simone Collins Landscape Architecture, Berwyn, PA, USA
wcollins@simonecollins.com
William Collins, RLA is a principal at Simone Collins Landscape Architecture of Norristown, Pennsylvania. He earned his BSLA from Ohio State University, and has practiced as a traditional timber framer, and served as bridge designer for the Tohickon Aqueduct rehabilitation.

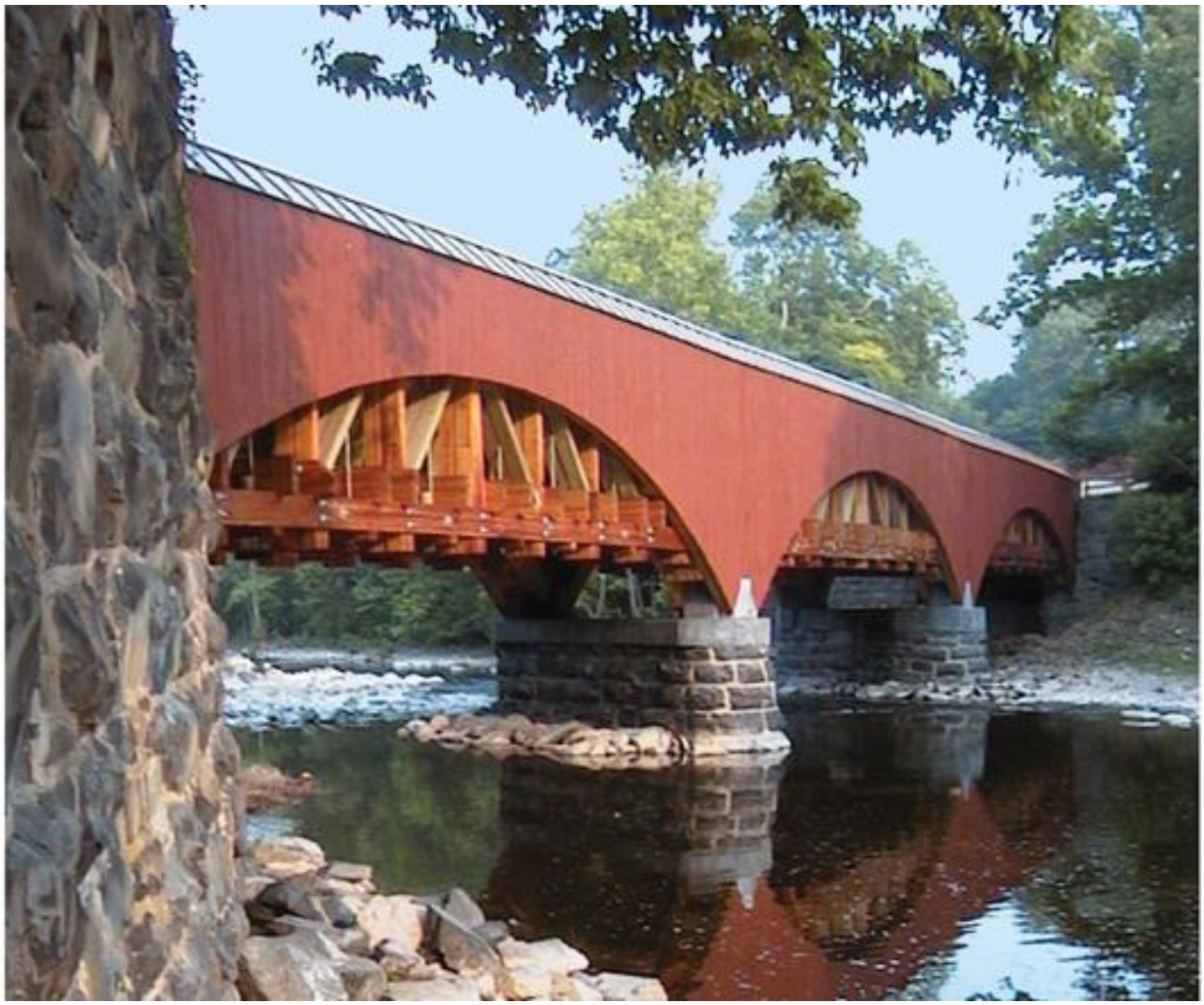


Fig. 1 The western elevation (upstream) of the rehabilitated Tohickon Aqueduct

Summary

The Tohickon Aqueduct is a heritage structure in Bucks County, Pennsylvania that was rebuilt to carry the historic Delaware Canal over the Tohickon Creek *(Fig.1)*. The rehabilitation project was completed in June 2001 as a local-state-federal partnership to re-establish water flow; reopen the national Delaware and Lehigh Trail; improve infrastructure aesthetics; and extend the life of a heritage investment in a state park bridge. The Tohickon Aqueduct is the largest structure contributing to the 60-mile Delaware Canal National Landmark in Bucks and Northampton Counties, Pennsylvania. The rehabilitation design combines a traditional Burr Arch Truss system with modern wood materials and treatments to create a new 3-span, 220-foot timber bridge that carries the historic “water road” and towpath trail on rehabilitated stone foundations.

The design imperative was to protect investments in the wood superstructure by synthesizing traditional forms, techniques, and materials with modern timber treatments. Specifications included a hidden impermeable liner to contain the waterway and traditional cedar bridge sheathing and copper caps to create a complete "covered bridge" system. Mechanical detailing was equally important to protect timber materials as the preservative treatments that were specified, based on member location, size, lamination, and species. A canted trunk wall design was borrowed from the historic Roebling Aqueduct to reduce water load, maintain canal boat clearances, and brace the trusses laterally. Aesthetics were considered integral to the attributes of the structural timber and the design echoes the context of the historic commercial waterway and surrounding natural environment. Signature architectural treatments include coping the bridge siding concentric to the arches to expose the primary trusses to greater air circulation; relieve flood obstructions; and feature the monumental superstructure behind the drip lines.

Fig. 2 The previous concrete and steel version of the Tohickon Aqueduct, built after WWII

The Tohickon Aqueduct is a milestone product of a decade-long partnership between the US Forest Service; Pennsylvania Department of Conservation and Natural Resources (DCNR); the local community; research institutions and private industry. The Forest Service provided financial and technical support over five phases of timber design development, including value-engineering of the final design to achieve a $1M cost reduction by replacing most solid timbers with glued-laminated (glulam) members while maintaining the integrity of the true Burr Arch Truss design. UMaine Advanced Wood Engineering Center (AWEC) designed and field-tested fiber-reinforced polymers (FRPs) on glued-lam transverse beams under the unique uniform live load conditions of a watered aqueduct. The Tohickon Aqueduct received four national, state and local design awards.

Keywords: *Theodore Burr, Arch, Truss, Roebling Aqueduct, Delaware Canal, State Park, covered bridge, DCF Engineering Inc., FRP, Fiber-reinforced polymer, Glued-laminated timber, glulam, Ithiel Town, Pennsylvania Department of Conservation and Natural Resources (DCNR), J.D. Eckman Inc., Simone Collins Inc., timber bridge, Tohickon, University of Maine Advanced Wood Engineering Composites Center (AWEC), USDA Forest Service, Economic Action Program (EAP) and Wood in Transportation Program (WIT).*

1. Brief History of the Delaware Canal

The 60-mile Delaware Canal was originally built in two years between Easton and Bristol, Pennsylvania, however craftsmanship was so poor in some sections that it took two more years to reconstruct the system so that that it would hold water. Commercial navigation began in 1834 and operated for nearly 100 years before a steel truss version of the Tohickon Aqueduct failed in 1931, ending the canal era in Pennsylvania *(Fig 3).* After World War II, the canal was acquired to serve as a Pennsylvania state park, and soon thereafter the Tohickon Aqueduct was rebuilt as a concrete and steel superstructure that remained functional until the early 1990s *(Fig 2).*

Fig. 3 The western elevation of the steel truss version of the Tohickon Aqueduct with cascade overflows

2. The Community Partnership Design Process

By 1990, the State was planning to replace the deteriorating concrete aqueduct with a new concrete superstructure. Members of the Point Pleasant Community Association (PPCA) began discussing the possibility of reconstructing the aqueduct "the way they originally built it, with wood." A local landscape architect with timber framing experience prepared concept plans for a timber aqueduct. The concepts were presented at a meeting in 1992 of community, with local and the state park officials. DCNR offered to partner with the community outside of the normal engineering selection process – asking the PPCA to deliver engineering plans for a timber aqueduct superstructure. DCNR would be responsible for all other project engineering and administration.

The PPCA received assistance from the Forest Service in 1992 – beginning a 10-year partnership with the first of five Forest Service grants needed to retained Simone Collins landscape architecture and DCF Engineering as the timber superstructure design team. The project design advanced slowly between 1992 and 1999 as phases of funding became available to develop engineering analyses; pre-final design; final engineering; and the timber construction documentation that married with DCNR substructure engineering.

3. Engineering a Modern Burr Arch Truss Timber Aqueduct

3.1 Geometric constraints of the historic waterway

Multiple design constraints controlled how a new timber structure would fit within the existing features and conditions. Critical original elevations needed to be maintained at bottom of the superstructure to prevent stream encroachment, and at the trunk inverts to preserve the hydraulics within the canal. The new structure was limited in width by the two existing stone piers that were to be preserved and re-used. The "centers" between piers were not identical and the substructures were skewed 30 degrees from the bridge transverse.

3.2 Selecting a structural truss system

The landscape architect researched the Delaware Canal historic documents at the National Canal Museum archives in Easton, and DCNR searched its own archives – neither yielding any plans of the four previous versions of the Tohickon Aqueduct. The lack of historic data freed the new structural system to be selected based on other considerations. All extant timber bridges in Bucks County are Town lattice trusses – making the Town the local preference. DCF questioned whether a Town truss would be stiff enough to carry the water load, but Town trusses also posed other issues.

Town lattices are known for their toughness, but they are created from many lighter built-up members, making the system more susceptible to potential decay on hidden surfaces. The Town lattice is also difficult to repair or reinforce in place because of its vast array of members. A Town lattice can be coupled with an arch to increase stiffness, but practical problems arise in constructing a Town lattice truss from pressure treated materials due to the amount of field boring required for timber "trunnels" (pins) that must be driven tightly through the members. Controlling moisture content within such pressure treated materials would have posed a problem. Additionally, a Town lattice truss would have needed to be constructed continuous over three spans and extend beyond the end supports for a distance equal to its depth – a structural reality that would increase the length of the Tohickon Aqueduct approximately 24 feet.

DCF modelled both Town lattice and a Burr trusses, and concluded that a Burr truss system would provide the stiffest traditional timber structure, and be more suitable for an aqueduct structure than a Town truss. The Burr truss-arch is known to be statically indeterminate, but has been demonstrated that the arch provides great stiffness to the system and the truss affords stability. DCF designed the Burr trusses *(Fig 4)* for the Tohickon Aqueduct as a redundant system so that either the trusses <u>or</u> the arches could carry the loads independently. In the 19th Century, builders would simply proportion each to carry the load, and then "yoke the two together." Arch and truss were analysed separately and together by DCF, and each section of Burr Truss was designed to support 4000 plf or a total of 8000 plf per span.

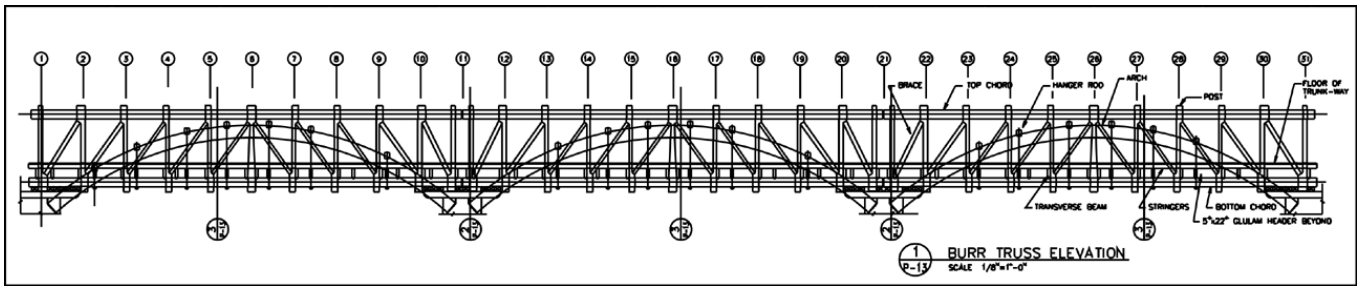


Fig.4 DCF construction drawing – elevation of the Burr Truss

3.3 Integrating engineering and bridge architecture

The bottom elevation of the new trusses was set at the bottom of structure elevation of the previous aqueduct structure. Tall trusses were needed to provide adequate stiffness in the superstructure and this reality required the new timber trusses to extend above the towpath elevation – physically separating the towpath from the waterway. The landscape architect designed the towpath as an

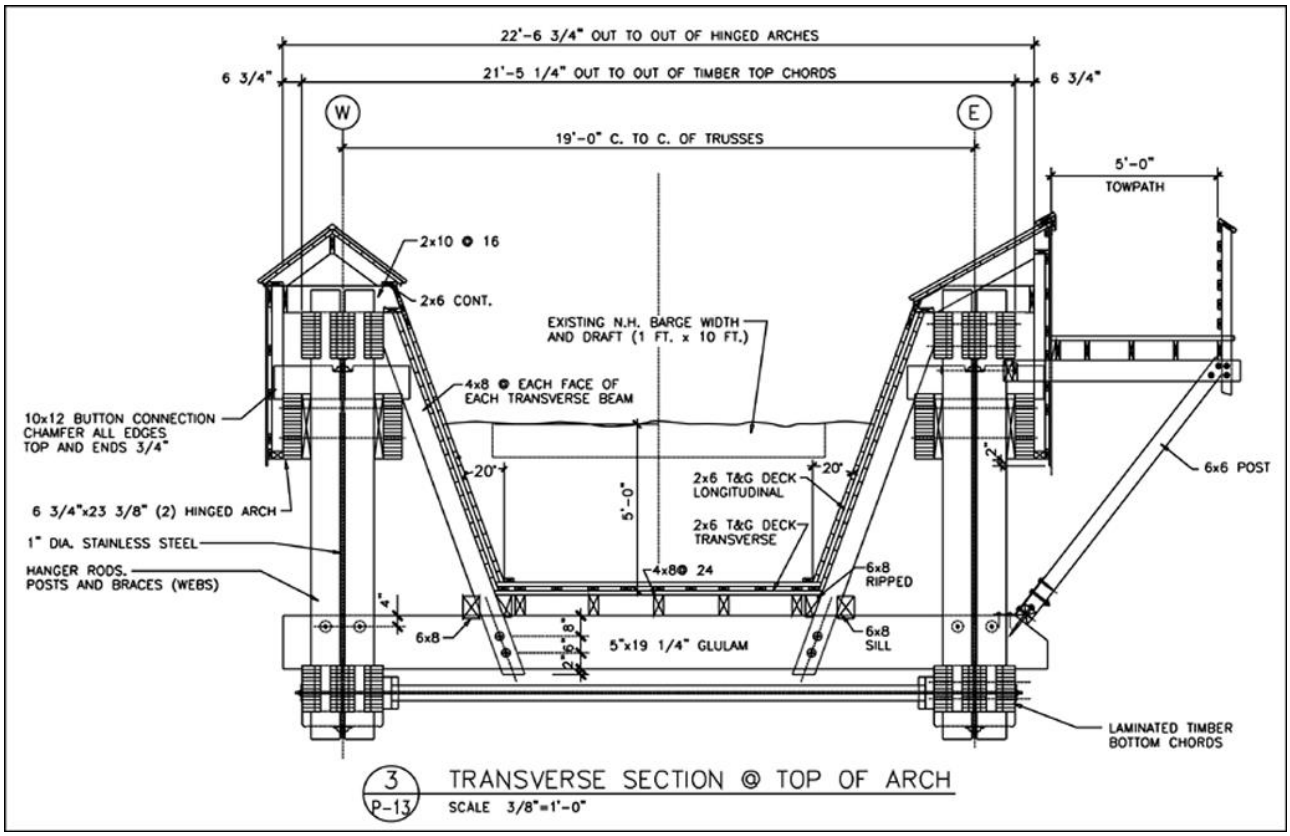


Fig.5 DCF construction drawing – transverse section through the superstructure

outrigger deck, supported by canted posts bearing on the downstream truss to allow canal boat towropes to slide over the truss top *(Fig. 5)*. DCF and SC collaborated on the trunk design that would bear on the transverse beams and conduct water across the bridge without leaking. The canted trunk post design that was documented and reconstructed by the National Park Service in the renowned Roebling Aqueduct over the Upper Delaware River was adapted for the Tohickon Aqueduct design. The trapezoid cross section of waterway reduced the design load, and the angled posts serve as transverse braces for the trusses.

3.4 Using modern materials within a traditional structure

DCF realized that glulam members would be required for the long truss chords and transverse beams to carry the water load to the trusses, however the first specifications for the truss posts and braces included several timber species, kiln drying, and preservative treatments to meet the particular demands for each member. Ultimately, only truss braces and the trunk/wall braces were constructed of solid timber. All partners envisioned that an impermeable liner system was absolutely essential to contain the canal water and protect the structural timbers below. The landscape architect designed the trunk system with a structural wood sheathing and wood façade liner to hide and protect the geotextile liner laid in between. The façade liner was designed as an interlocking, removable “pallet” system that was fixed in place without piercing the impermeable liner. The tops of the trunk liner system and the exterior sheathing were all covered by copper caps to create a completely “covered” bridge. Although covered and mechanically protected, all timber materials were also pressure treated to resist decay because moisture derived from leaks, condensation, floods, and splashing were judged to be unacceptable risks to an untreated structure. The landscape architect prepared the schematic plans for the stone and concrete abutments and piers to initiate the design interface between the community “superstructure” and State “substructure” engineering teams.

3.5 Value Engineering / Technology Transfer

The collaborative design-documentation process did not end in 1999 when the project partnership was faced with a $3.1 M construction bid as the lowest submitted by J.D. Eckman Inc.(JDE) of Chester County, PA. The State determined that a $3.1M timber aqueduct exceeded the budget available for the project, and directed DCNR to develop an alternative design for bid using concrete. The PPCA petitioned DCNR to consider a "value-engineered" timber aqueduct as an alternate bid. The Forest Service agreed to fund the value engineering tasks and suggested that Fiber-Reinforced Polymers (FRP) tensile material be considered as a cost savings measure to reduce the sizes of timber members. DCNR agreed to re-bid two alternates – timber and concrete.

Value engineering included two tasks: cost-cutting design measures and technology transfer. The landscape architect coordinated the work and the team identified areas where the timber construction could be simplified and costs reduced. The traditional Burr truss form was accepted as the "character-defining" feature and was retained for the timber alternate. Using the same Burr truss structural plans, specifications and details were value-engineered as follows:

- All timber framing labor qualifications were eliminated.
- All traditional handcrafted mortise and tenon joinery (including lightning bolt splices) were eliminated and replaced with steel fasteners.
- Solid timbers were eliminated and replaced with glued-laminated (glulam) materials for major members. (This was extremely cost-effective for the shouldered truss posts to eliminate the problems of material supply, shrinkage, and cross-section tolerances.)
- Multiple species were eliminated and southern pine was specified.
- Arches were redesigned as glulam members (Shear blocks, stitch bolts, dual species, and special treatment of the bearing laminations were eliminated.)
- Pentachlorophenol (Penta) wood preservative was specified as the preservative treatment for all glulam members. (Except for the arches that were too large to treat after fabrication, so a Penta type C was used - which is a Penta in light petroleum solvent instead of oil - that allows gluing after treatment.)
- The State did not want creosote treatment in the structure, so bearing sills and sleepers were treated with CCA.
- The half-lap joints at the interfaces of the arches and trusses were eliminated.
- The different span lengths between existing foundations were made identical by re-engineering the concrete caps on the pier tops. Forming and placing the concrete thrust pads after the arches were suspended in place accommodated the remaining differences.
- The continuous 3-span truss of the original design was replaced by three separate and identical span trusses (to simplify fabrication.)
- The primary suspension rods and transverse rods remained as stainless steel, but some stainless steel hardware items were replaced with galvanized steel. The State requested that the decking hardware remain as stainless steel as a remedy to the local acid rain.
- Contractors were permitted to submit shop drawings for the superstructure and fasteners.
- FRPs were introduced as a potential new material

The landscape architect compiled and distributed the new technical specifications to potential general contractors, material suppliers, and fabricators in advance of the State rebid bid notice. PPCA mailings and phone conferences helped communicate the project simplifications to bidders. The design team goal was to remove contractor uncertainties and reduce construction costs by $.5M below the original $3.1M low bid. The second round of construction bids returned in 2000 with another low bid by JDE at $2.1M – one million dollars less than its previous low bid. The State selected JDE and construction began without the FRP-glulams included in the contract, because DCNR would not allow transverse beams to be downsized based upon new FRP-glulam technology

that was yet to be field-tested. A compromise with DCNR allowed FRP-glulam specifications to be included in the construction documents prepared by DCF and AEWC, but only for transverse beams that were sized to bear the full load without FRP reinforcement. This created the opportunity to monitor field performance of FRP-glulam members in the Tohickon Aqueduct.

3.6 Introducing Fiber-Reinforced Polymers into the Project

An aqueduct is an unusual structure because it is uniformly loaded almost perpetually. The Tohickon Aqueduct supports 350 tons of water within its trunk. Any traversing vessel displaces an amount of water equal to its weight, resulting in a constant total uniform weight on the aqueduct. DCF and AEWC determined that the high bending moments in the transverse beams offered the best FRP testing opportunity. Approximately one third (24 total) of the transverse beam members within the bridge were fabricated with fiber-reinforced polymers (FRP) for the purpose of monitoring this application under extended duration of a "uniform load," as created by the unique field conditions of a working aqueduct filled with water. The monitoring regime was designed to measure "creep" of the FRP material as a function of deflection over time under the sustained loading of the water-filled aqueduct, in an environment where temperature and relative humidity are uncontrolled.

4. Constructing the Timber Aqueduct

Fig.6: The eastern elevation of the Tohickon Aqueduct during construction

DCNR agreed to allow FRPs to be used in the structure with the following stipulations:

1. The project construction schedule would not be delayed.
2. The contractor was voluntarily willing to participate.

The FRP-glulam work did not fall in the critical path and the contractor agreed to participate with the cooperation of the glulam manufacturer and supplier. The contractor determined that he could build two of the spans while waiting for FRP-glulam beams for the third span. FRP reinforcement was applied to 24 of the transverse beams, and to an additional six (6) floor beams for laboratory testing at AEWC. It was thought the FRP reinforcement could be applied to the beams at the manufacturing plant of Alamco Inc., in Minnesota. Unfortunately, the fabricator could not secure a

Fig. 7 View to north between trusses with transverse beams in place and trunk framing under construction

commercially available supply of pre-made layers of FRP, as specified by AEWC. Instead, the glulams designated for reinforcement, plus six extra test beams were shipped to Maine where AEWC manufactured and applied the FRP reinforcement. Meanwhile, the remaining two-thirds of the unreinforced glulam beams were shipped to the project site for assembly.

After the FRP reinforcement was applied, the six extra beams remained at AEWC for testing and the twenty-four FRP-glulams were sent for preservative treatment and then delivered to the project site. The beams arrived at the site in time for the contractor to install them without adversely affecting the construction schedule. When the structure was essentially completed, staff from AEWC arrived on site to attach instrumentation to eight of the FRP-glulam beams and eight of the unreinforced glulams. AEWC worked with JDE to install catwalks beneath the structure to make reading the instruments easier by data collection volunteers

4.1 FRP Field Monitoring and Data Analysis

Most deflection monitoring conducted in a laboratory setting uses digital (e.g. electronic data acquisition) or analog (e.g. dial gage) equipment. In using analog dial gauges, there needs to be a non-moving surface below the object onto which the gauge is mounted. With digital equipment, there needs to be a power supply, or solar panels to provide power to the equipment and data acquisition system. In the case of the aqueduct, there was not a suitable surface onto which a dial gauge could be mounted, and there was not a power supply available. As an alternative, “string and ruler” deflection gages were developed. This entails mounting a ruler to the neutral axis of the beam at mid-span and stringing a thin wire along the length of the beam from the neutral axis at each support. As the beam deflects, the mid-span “ruler” gauge deflects with the supports (line) remaining fixed, allowing for a deflection reading to be taken *(Fig. 8).*

Fig. 8 Field instrumentation on the Aqueduct (rods and rope are the catwalk system)

4.2 Conclusion

The Tohickon Aqueduct is both a cultural heritage structure and a modern timber structure. It continues Theodore Burr's brilliant 19th Century design legacy that combined the multiple king post truss with a two-hinged arch, as it showcases the possibilities to synthesize traditional construction methods with modern technology by featuring an array of hybrid details to suit unusual demands. As a demonstration project, it provided a real world laboratory for timber technology. The 10-year project satisfied the visions of community organizations, elected officials, heritage development promoters, historians, federal forestry administrators by employing the multi-disciplinary technical expertise of landscape architects, structural engineers, state agency engineers, and timber research engineers to create a compellingly beautiful structure as the next chapter in a remarkable historic structural genealogy. The Tohickon Aqueduct received the following awards:

- 1st Place – National Timber Bridge Award – Rehabilitation, 2002
- Honorable Mention – National Rail-Trail Design Recognition Awards, 2001
- Honor Award – ASLA, Pennsylvania/Delaware Chapter, 2001
- Excellence In Design, Franklin Wood Award – Bucks County AIA, 2002

4.3 Acknowledgements

- The late David C. Fischetti, P.E., engineer of record Tohickon Aqueduct superstructure
- The late Eugene Comoss. P.E., engineer of record Tohickon Aqueduct substructure
- Point Pleasant Community Association
- US Forest Service
- Pennsylvania Department of Conservation and Natural Resources
- Pennsylvania Department of Environmental Protection
- Pennsylvania Rural Development Council
- Pennsylvania Forestry Association
- UMaine AWEC
- National Canal Museum

Fig. 9 View north of the Tohickon Aqueduct prior to 1931. Note the boat and mules in operation.

Figure 10 View north of the new Tohickon Aqueduct, partially watered

REHABILITATION AND UPGRADING OF A HERITAGE LISTED TIMBER TRUSS BRIDGE

Keith Crews
Professor of Structural Engineering & Assoc Dean
University of Technology Sydney,
Australia
keith.crews@uts.edu.au

Keith is an internationally recognised expert in the area of Sustainable Infrastructure – such as timber bridges, buildings & infrastructure. Keith has published extensively, and is an Editor for the Construction and Building Materials Journal.

Summary

The Tharwa Bridge over the Murrumbidgee River was originally constructed in 1895, and is the oldest and largest surviving four span, timber Allan truss bridge in Australia. A key objective of the rehabilitation of the old bridge was to maintain the integrity of it's national heritage value, by keeping as close as possible to the original design, rather than providing an alternative bridge using modern materials such as prestressed concrete or steel.

In order to accomplish this, and upgrade the load carrying capacity to modern standards, the analysis and design methods utilised were based upon research that was undertaken previously at the University of Technology, Sydney – particularly in regard to use of a 110m long stress laminated timber deck and advanced techniques for modelling the buckling characteristics of timber truss members.

This paper presents an overview of the design and construction innovations in timber truss bridge design that were undertaken for successful completion of the project in 2011. It also highlights the fact that successful application of new techniques and innovative design can be directly attributed to the collaborative efforts between owners, designers, advisors and construction teams that made this happen.

Keywords: Heritage Bridges, stress laminated timber, timber trusses, buckling.

1. Introduction

1.1 Background

The original Tharwa Bridge was 180m long and for over 100 years it has been the main crossing over the Murrumbidgee River for the local community and visitors to Tharwa, which is located some 30 kilometres south of Canberra. The bridge was initially constructed entirely of timber; with timber piles supporting timber piers up to 12m high for the four main 90ft (27.4m) truss spans and simply supported span approaches (each approx 9m) on either side. Subsequent rehabilitation works during the 1960s replaced the timber substructure with concrete piers, as well as replacing most of the approach spans with reinforced concrete decks.

Degradation as a result of inadequate maintenance and construction details that trap moisture, has meant that the principal timber truss elements have been periodically replaced throughout the life of the bridge. However, by the turn of the 20th century significant deterioration of the structure eventually caused a loss in capacity of the main truss elements, resulting in installation of Bailey support trusses in 2005 and then finally closure of the bridge in 2006. Subsequently, in response to community support for the original bridge design and recognition of its significance, Roads ACT proposed a $25m rehabilitation in 2007, involving upgrading of the Allan truss structures rather than the replacement of the bridge.

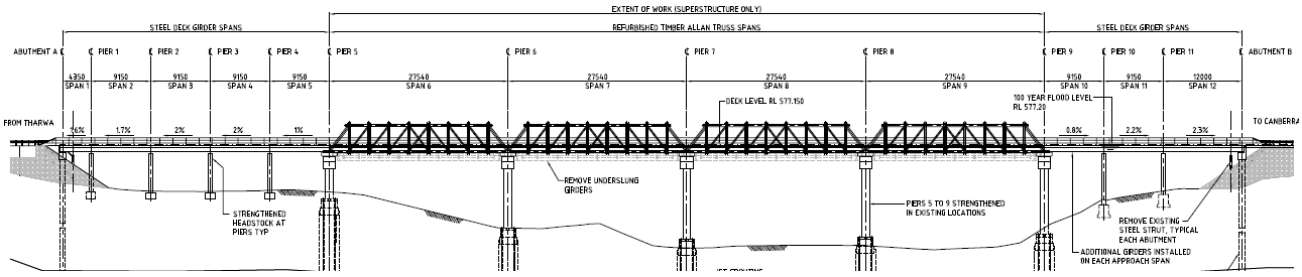


Figure 1 – Elevation of Tharwa Bridge

Figure 2 – Bailey Trusses in 2005

Figure 3 – Temporary support structure 2008

2. Overview of the Project

The project was split into four stages to enable traffic to use the facility throughout the upgrade works:

2.1 Stage 1

The first stage involved temporarily supporting the deck using steel under-slung girders so as to remove the Bailey and timber trusses, and allow traffic access after nearly two years of closure. The bridge was re-opened with a 5 tonne load limit in August 2008.

2.2 Stage 2

Due to the degradation that had occurred, refurbishment and strengthening of the existing concrete approach spans and piers for an increased T44 live load capacity (44 tonne vehicle) was required. Strengthening of the main span piers was undertaken using jet grouting around the perimeter of each pier to provide scour protection of the foundation and stability of the piers during a flood event was increased through micro-piles tied back to the pier face and anchored into bedrock.

Strengthening of the approach span piers was required for the increased traffic loading. This was achieved by the addition of two longitudinal I-beam girders under the deck in each span and installing carbon fibre containment strips wrapped around each concrete pier headstock to provide additional shear strength.

2.3 Stage 3

This stage of the works involved the replacement of the timber trusses, deck, and traffic barriers by the Roads and Traffic Authority of NSW (RTA), who have replicated as near as possible the original methods of construction for the trusses. This work forms the main focus of the restoration and is discussed below in further detail.

2.4 Stage 4

The final stage involved removal of the under-slung girders and temporary steelwork, including rehabilitation of the surrounding public area. The entire reconstruction was undertaken while still providing traffic access during the refurbishment operation. With the exception of several short term bridge closures, access interruption for Tharwa residents and visitors to the area was kept to a minimum.

Figure 4 – Strengthening of the approach spans

Figure 5 – FRP strengthening of the headstocks

3. Design Innovations

3.1 Challenges

One of the significant challenges in undertaking restoration of historic timber truss bridges is the fact that structural behaviour and interaction of elements is not accurately modelled using conventional analysis techniques and the prediction of truss capacities tends to be quite conservative when applying procedures in design codes that were essentially developed for residential buildings. The combination of these two factors has meant that designers have usually produced designs that require truss elements that are significantly larger than the original members, which in turn has resulted in changes to the structural form and loss of heritage value – even though the trusses had been safely carrying the required vehicles prior to rehabilitation.

The solution to this problem has been to develop more refined analysis methods, particularly in regard to prediction of buckling capacity and modelling of connections within a truss, which is the outcome of several research projects undertaken at the University of Technology Sydney since 2008 [1], [2], [3].

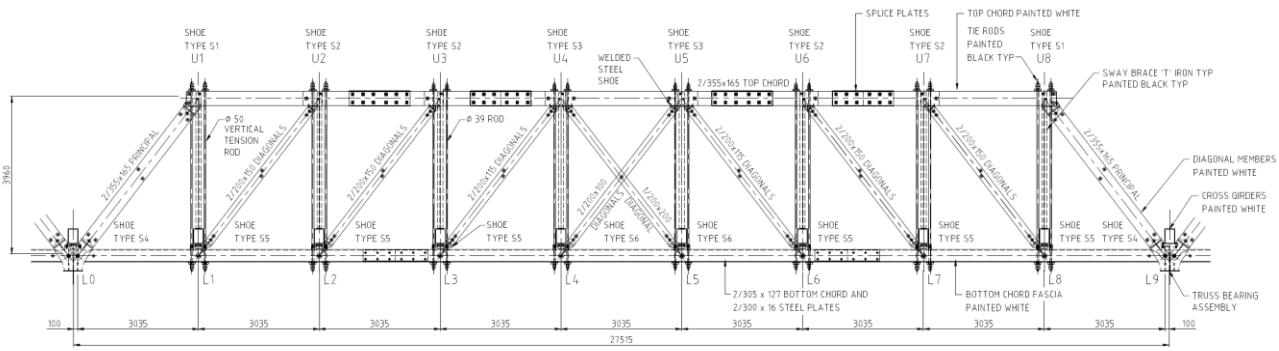


Figure 6 – Side Elevation of the Allen Truss

3.2 Improving Truss Performance

Design of the Tharwa trusses was based on this research which led to a number of innovations that utilise the timber very effectively. Additionally since timber is generally more suited to compression than tension, the bottom chords were replaced by two steel plates which carry the full tensile load. The diagonals are then connected at the plates through a simple pinned steel connector (shoes). Timber members were connected to the outside of the steel plates to maintain the visual appearance of the original bridge without acting as primary structural elements. All critical steel elements were replaced with low temperature, high strength grade steel suited to the temperatures experienced in Tharwa, limiting the possibility of failure as a consequence of brittle fracture. This material replaces the original cast iron shoes historically used in construction.

3.3 Designing for Durability

Durability of exposed timber in bridge trusses is always a major consideration since timber deterioration is influenced by many factors, particularly exposure to weather and water. The interface between steel and timber tends to be an area of degradation due to the propensity to act as moisture traps. To address these issues, specific details were developed that reflect “best practice” for creating inherent durability in the construction.

Timber/steel interfaces were fully painted to reduce the probability of decay resulting from fungal infestation. Steel flashing covering the top of the truss top chords, end diagonals and bottom chords limited the deleterious effects of moisture by providing a protective cover and appropriate air movement. All timber elements are painted white, a simple yet effective method of sustaining the life of the timber elements through reflection of light energy, and a detailed maintenance regime has been established to ensure that the paint and fungal decay protection systems remain effective.

3.4 Other Design Issues

New requirements in bridge design to current standards have been merged with maintaining the aesthetics of the original design. Traffic barriers are shaped to match the original design but are constructed of stronger steel sections. The four truss spans are fixed to the piers to promote load sharing between the piers and to prevent the vertical movement of the deck during flood events.

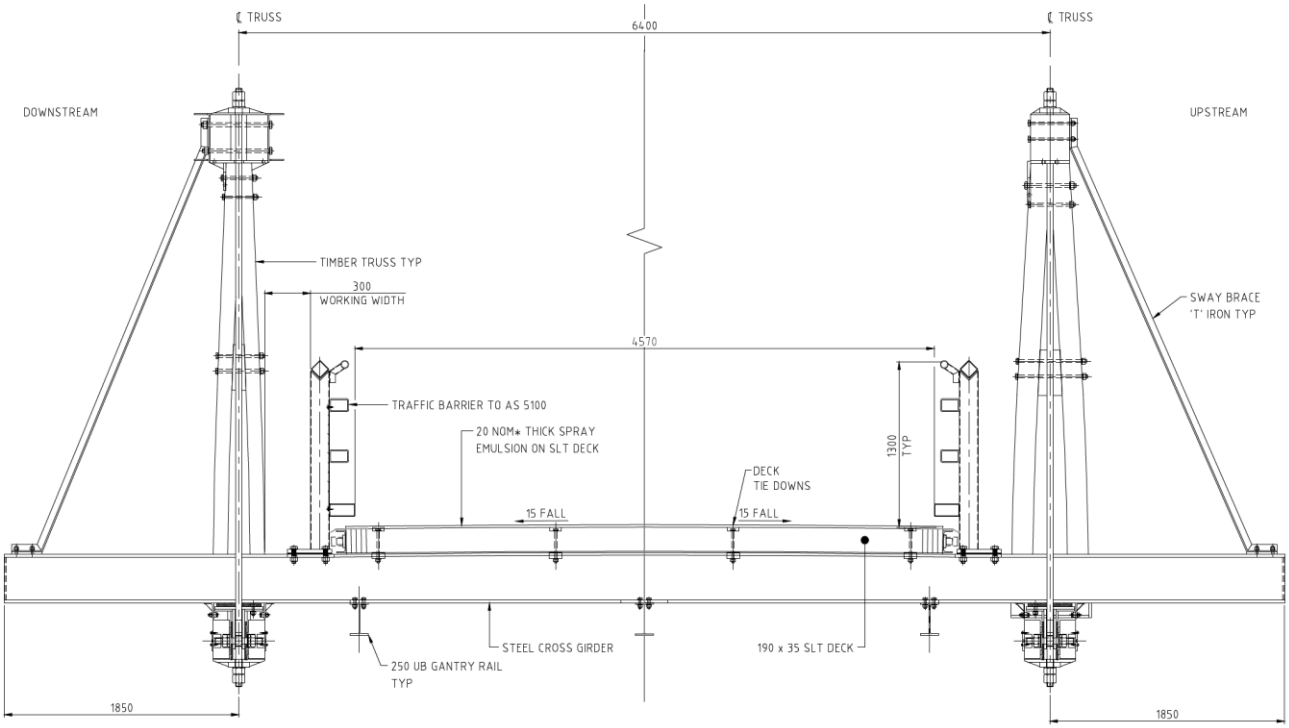


Figure 7 – Typical Cross Section of the new trusses, showing traffic barrier & SLT deck

3.5 Deck Design

The traditional construction method for decking timber bridges involves using a multitude of timber elements fastened to beams using bolts or spikes. This technique requires frequent maintenance (retightening of the connections) and contains provides numerous areas where degradation can occur at each connection and the interfaces between members, since the gaps tend to become inherent moisture traps.

The modern high performance alternative to traditional decking is the stressed laminated timber plate deck, which consists of kiln dried (in this case native hardwood) timber laminates compressed together using high strength strands or bars. In the case of the Tharwa bridge, the stress laminated timber (SLT) deck is laid directly on and fixed to the cross girders, which have been constructed of steel to ensure durability. Timber cross girders, used in the past, are one of the most frequently replaced elements due to the rate of degradation in this location, and they are known to be one of the first modes of failure in a timber Allan truss. Differential movement between the deck and the trusses is taken up at elastomeric bearings under each cross girder, with appropriate tie downs that prevent deck vibration movements and permit horizontal shear transfer into the main structure.

Figure 8 – Truss Assembly on site

Figure 9 – Assembling the SLT deck on site

The SLT deck was constructed as a monolithic orthotropic plate, 110 metres long, which eliminated the need for joints between the truss - a reduction in joints from a possible five to just one at each end. Deck joints in timber decks tend to be prone to high rates of degradation under traffic loading and this consideration justified the adoption of a full length deck. However, this length of SLT deck is double that of any previously launched in Australia and as such, posed new challenges in the ensuring uniform stressing and correct positioning of the deck. The deck was constructed on site, aligned with the bridge centreline for eventual launching using swivel castor rollers. Careful planning and monitoring of both stress levels and alignment of the deck ensured that it was launched and installed in place successfully.

4. Procurement of Timber and Fabrication of the Trusses

4.1 Material Sourcing and Preparation

Quality management of timber sourcing and the seasoning process were critical processes in ensuring that the timber was suitable in terms of strength, stiffness and durability. The trusses required procurement of 86 m^3 of high grade native hardwood timber (Eucalyptus) in December 2008 which was subsequently seasoned for 12 months in storage. This process involved surveillance of all activities in the supply chain - including identification of the logs in the forest, felling of trees, sawmilling, inspection and confirmation of grade of the sawn members, and seasoning.

4.2 Truss Fabrication and Installation

A camber of 31mm was designed and built into each truss, since the truss shoes would have been overstressed if the truss was built square and level prior to inducing the camber. A truss assembly pad was designed and built on site to allow assembly of the eight trusses in a narrow line which also enabled the crane to reach all of them from a single set up. Progressive installation of temporary props allowed progressive truss assembly to occur. Temporary truss supports were also installed to prevent the trusses pulling apart during lifting. Planning for truss lift commenced some over 12 months prior to the lift date.

The fully assembled trusses (each 22 tonnes) were installed in three days using a 400 tonne mobile crane with a reach of 45m, allowing installation from a single site set-up adjacent to the Murrumbidgee River. The weight of each timber member was confirmed during truss fabrication to accurately verify the assumed timber density and calculated truss weight. Joint site inspections were undertaken with the crane operator and a specific truss lifting Safe Work Method Statement was developed. Whilst six days were programmed for the installation of the trusses, all eight lifts were completed within three days.

Figure 10 – Truss being lifted into place using a 400t mobile crane

5. Construction of the Stress Laminated Timber (SLT) Deck:

5.1 Design Constraints

The SLT deck was fully fabricated on site (on the eastern end of the bridge approaches) between May and July 2010. The design required a variety of timber lengths for the SLT deck laminates, to address the following constraints:

1. the stressing elements had to “fit” within the truss geometry to ensure appropriate access for stressing hardware in order for re-stressing to occur
2. the deck needed to utilise readily available standard sized timber
3. The butt joint pattern was not to exceed one joint in every four laminates across the deck width (1 in 4 pattern).

The deck was designed as a continuous plate in conformity with the new RTA standard [4] using kiln dried F27 blackbutt, with design properties of f'_b = 80 MPa and MOE = 18500 MPa. Construction consisted of 3,500 individual laminates which were fabricated (docking and drilling of holes) off site, then sorted with each bundle delivered to site in a planned sequence to optimise construction. Once on site the workmen laid out the laminates in the order they were to be nailed, saving sorting time on site and reducing exposure of laminates to wet weather. A specifically designed SLT support bed and work area was constructed, including key blocks along the centreline of the deck to provide lateral stability during stressing. The SLT deck was built 200mm longer than specified to allow it to be cut to the final "actual" length.

Figure 12 – Construction of the SLT deck showing laminate lay-up

5.2 Stressing

Initial stressing was completed using 36 x 30 tonne hydraulic jacks with associated hoses and fittings, and a single hydraulic pump. The 144 strands were numbered from 1 to 4 to represent 4 jacking stations. The process of stressing entailed the repeated force application of 25%, 50% and 100% of the maximum permissible design jacking force. Final restressing was completed in April 2011 and achieved the modified target force of 191kN. The width of the deck reduced 65-75mm from its initial value after completion of the final stressing.

5.3 Launching

Launching of the SLT deck was carried out in 3 days in early May 2011 using a 24 tonne truck with an 8 tonne winch pulling from the front end, with another 24 tonne truck and winch restraining the deck from the rear end. The winch steel wire rope was attached to a specially designed and fabricated launching nose and pulling frame. The actual maximum pulling force exerted on the 100 tonne deck was 2 tonnes. The deck was then winched along on swivel castors which were guided by a channel fixed to the underside of the deck to keep it in line.

Figure 13 – SLT deck Launching Nose

Figure 14 – Launching of the SLT deck

6. Conclusions

This project presented a unique opportunity to rehabilitate a 100 year old timber bridge which has highly significant heritage value for Australia. The challenges of maintaining the inherent structural integrity and form of the trusses, whilst incorporating new design and construction technologies to re-instate the bridge to carry modern vehicle loadings, were successfully addressed and the bridge was opened in June 2011.

The two major project milestones were the truss lifting in November 2010 and SLT deck launching in May 2011, and both were achieved successfully through detailed planning and preparation, with a superb safety record for the duration of this stage of the project. The safe, correctly fitting truss and deck installation process was testament to the great efforts of the RTA construction team. Working with a 116 year old structure and with a natural material like timber will always create challenges. Despite this, the required truss tolerances of 1mm to 5mm and the SLT deck width tolerance of 30mm over 110m were both achieved, as a consequence of meticulous attention to detail by the AURECON designers, ACT Heritage and the RTA construction team.

The new timber trusses and deck are rated to safely carry an enhanced traffic loading up to 42.5 tonnes (T44) legal load. The visual appearance is now almost identical to the original Allan trusses and has preserved the historical significance of the four span Allan truss in Tharwa. Success of this project can be measured not only by the restoration of this valued structure, but by the initiative taken to meet the challenges of working with new solutions, and the cooperation and dedication of all parties working in an Alliance relationship, to achieve a best for project outcome.

7. Acknowledgements

The author wishes to acknowledge Ms Marcia Prelog and Mr John Hilton, AURECON Consulting Engineers, Sydney and Mr Sam Millie, Roads and Traffic Authority, NSW for their assistance in preparation of this paper.

8. References

[1] Crews (2008) - *Rehabilitation Designs on Some Timber Heritage Bridges in Australia*; Structural Faults & Repairs, Edinburgh, 2008.

[2] Nicholas (2011) - *Capacity of Compression Members in RTA Timber Truss Bridges*; Master of Engineering (Structural) Thesis, University of Technology, Sydney

[3] Nicholas & Crews (2012) - *Capacity of Compression Members in Heritage Timber Truss Bridges*; IABMAS 2012 – 6th International Conference on Bridge Maintenance, Safety & Management, Stresa Italy, 2012.

[4] Crews (2012) - *Guide for the Design of Stress Laminated Timber Bridge Decks*; Roads and Traffic Authority, NSW, Australia – ISBN: ISBN: 1 86365 146 2

Timber Abutment Piling Rehabilitation and Repair

Justin M. Dahlberg
Bridge Research Engineer
Iowa State University
Ames, IA USA
dahlberg@iastate.edu

Summary

This paper provides an overview of the completed project for the Iowa Department of Transportation and Iowa Highway Research Board by Iowa State University's Bridge Engineering Center (BEC) regarding the repair and rehabilitation of timber abutment piling [1]. A large number of bridges on the county-level roads around Iowa are constructed using timber piles and these structures are increasingly being labeled as structurally deficient because of the condition of these components. Repair and rehabilitation methods specifically aimed at piling could potentially upgrade a progressively aging fleet of timber bridges thereby extending their service life. The objective of this study was to identify effective methods of repair that are currently being used around the state and to improve upon these methods or develop other methods that can be economically implemented.

Keywords: Timber Piling, Repair, Timber Strengthening

1. General

1.1 Introduction

Based on previous National Bridge Inventory data, the state of Iowa has nearly 25,000 bridges which rank it number 5 in the nation behind Texas, Ohio, Illinois, and Kansas. In Iowa, close to 80 percent of these bridges are on low volume roads (LVRs) and, thus, are the responsibility of the county engineers. Of the bridges on the county roads, 24 percent are structurally deficient and 5 percent are functionally obsolete. A large number of the older bridges on the LVRs are built on timber piling. In many cases, as timber abutments and piers age, the piling deteriorate at a rate faster than the bridge superstructure. As a result, a large percentage of the structurally deficient bridges on LVRs are classified as such because of the condition of the timber substructure elements. This situation is especially common for bridges constructed in the period 1950-1970 that have reinforced concrete stringers and decks or reinforced concrete decks with steel stringers and timber substructure elements. The soil/water/air interface area of the piling is particularly prone to severe cracking and rot. Because there have been instances where bridges with legal rated superstructures and no load posting have failed under traffic loading due to pile failure, this represents a critical infrastructure situation.

As funds for replacing bridges decline and construction costs increase, effective rehabilitation and strengthening techniques for extending the life of the timber substructures in bridges with structurally sound superstructures has become even more important. Several counties have implemented various techniques to strengthen/repair damaged piling, however, there is minimal data documenting the effectiveness of these techniques. There are numerous instances where cracked and failed piling have been repaired. However, there are no experimental data on the effectiveness of the repairs or on the percentage of load transferred from the superstructure to the sound pile below.

1.2 Research Objectives and Scope

The objectives of this investigation were to:

- Review existing products for timber preservation and repair and to document their effectiveness in extending the life expectancy of various bridge components.
- Determine techniques used by county engineers and other engineers to repair and restore load carrying capacity of piling damaged by deterioration and cracking.
- Review methods used to repair failed piling.
- Determine/develop effective methods for transferring bridge loads through the failed portion of the pile.
- Determine that safe load capacity is restored by the repair methods (existing or new) determined to be structurally efficient.

2. Deterioration Mechanisms and Condition Assessment

In Iowa, low volume bridge foundation problems are often associated with timber substructures [2]. Timber piles are subjected to deterioration, which, at initial stages, can be difficult to detect. Furthermore, information regarding the soil profile and pile length at a given bridge site is often unavailable. There are currently no reliable means to estimate the residual capacity of an in-service deteriorated pile; and thus, the overall safety of the bridge cannot be determined with confidence. Although the majority of inadequate substructures have timber piling, there are numerous cases in which the steel substructures are inadequate (problems with corrosion, misalignment, damage due to impact, etc.). If procedures can be developed to assess the integrity of existing timber substructures and rehabilitate/strengthen inadequate substructures components, it will be possible to extend the life of those bridges and have increased confidence in predicting their performance.

2.1 Biological Deterioration

In most timber bridge applications, decay fungi are the most destructive organisms [3]. Fungi are microscopic thread-like organisms whose growth depends on mild temperatures, moisture, and oxygen. There are numerous species of fungi that attack wood, and they have a range of preferred environmental conditions. Decay fungi are often separated into three major groups; brown rot fungi, white rot fungi, and soft rot fungi. Soft-rot fungi generally prefer wetter, and sometimes warmer, environmental conditions than brown or white rot fungi.

Termites rank second to fungi with respect to damage to wood structures in the US [3]. Their damage can be much more rapid than that caused by decay, but their geographic distribution is less uniform. Termite species in the US can be categorized by ground-inhabiting (subterranean) or wood inhabiting (non-subterranean) termites. Most damage in the US is caused by species of subterranean termites.

Other types of insects such as powderpost beetles and carpenter ants can cause notable damage in some situations, but their overall significance pales in comparison to the decay caused by fungi and termites. Other organisms, including bacteria and mold can also cause damage in some situations, and several types of marine organisms degrade wood placed in seawater.

The two greatest factors influencing regional biodeterioration hazards are temperature and moisture [4]. The growth of most decay fungi is negligible at temperatures below 2 °C and relatively slow at temperatures below 10 °C. The growth rate then increases rapidly, with most fungi having optimum growth rates between 25 °C and 35 °C. The natural range of native subterranean termites is generally limited to areas where the average annual temperature exceeds 10 °C. Decay fungi require a moisture content of at least 20 percent to sustain any growth, and higher moisture contents (over 29 percent) are required for initial reproduction [4]. Most brown and white rot decay fungi prefer wood in the moisture content range of 40 to 80 percent. In almost all cases, wood that is protected from ground contact, precipitation, or other sources of water will have insufficient moisture to sustain growth of decay fungi. In contrast, wood that is in contact with the ground often has sufficient moisture to support decay, even in relatively dry climates. On the other hand, wood can be too wet to support fungal growth. For example, void spaces in the wood are increasingly filled with water as the moisture content exceeds 80 percent. The lack of oxygen and build-up of carbon

dioxide in the water limits fungal growth [3].

2.2 Physical Deterioration

There are several forms of physical deterioration that occur in timber bridge piling. In almost all cases the physical deterioration causes exterior damage that breaks down the protective preservative barrier and allows entry of biological decay mechanisms into the untreated wood. One of the most common types of physical deterioration is abrasion or debris damage. This generally occurs by the impact of floating debris and/or ice in a channel [2]. The velocity of water moving past the pile and the quantity, shape, size, and hardness of particles being transported have been linked to the rate of abrasion [5].

Overloading of piles can result from continuous heavy loads, infrequent severe loads, loss of the pile structural capacity, or more frequently, complete loss of adjacent supports [2]. Failure of one pile requires the adjacent piles to carry additional load. Overloading can be caused by vertical and/or horizontal loads. Continuous overloading results in several modes of compression failure including splitting of the top portion and misalignment or “mushrooming” at a hollow portion after breakage [6]. These stages include development of initial entry holes, active deterioration of the inner core with a significant increase in the size of the hollow space, compression failure of the shell, and finally separation of the hanging top portion of the pile from the pile cap [7]. In addition, overloading can occur from the mechanical fasteners used to connect bridge elements. Many times fasteners are over tightened causing bulging around the head, nut, or washer. The bulge generally leads to entry holes for deterioration of the inner portions of the timber to occur.

Fire is a threat to all timber bridge elements and has the potential to destroy an entire bridge in a matter of hours. However, thermal degradation of wood occurs in stages. The degradation process and the exact products of thermal degradation depend upon the rate of heating as well as the temperature [2]. A timber pile has a generally uniform strength throughout its cross section. Thus, the unburned section of the timber pile retains its strength, and its load carrying capacity is reduced in proportion to the loss of cross section. When exposed to high temperatures, wood will decompose providing an insulating layer of char that retards further degradation. Therefore, the amount of charring of a cross section controls the fire endurance of a timber pile [6].

Other noteworthy physical agents that damage timber piles are connection failure, which exposes untreated wood allowing entry for fungi or insects, ultraviolet (UV) degradation, chemical degradation, and foundation settlement [8].

2.3 Condition Assessment

A number of tools exist to assist the inspector with the diagnosis of deterioration and preventative maintenance [3]. The tools vary considerably in the amount of experience required for reliable interpretation, accuracy in pin-pointing a problem, ease of use, and cost. No single test should be relied upon for inspection of timber bridge components. Rather, a standard set of tools should be used by inspectors to ensure conformity in inspections and consistency between inspectors. These tests and tools include: visual assessment, probing and pick test, moisture measurement, sounding, stress wave devices, drill resistance devices, core boring, and preservative retention analysis.

3. Pile Maintenance State of Practices

Ritter [9] divided pile maintenance activities into three categories. The first category is preventative maintenance, in which the repair involves keeping the structure in a “good state”. At this stage, deterioration has not started, but the conditions or potential are present. The second category is early remedial maintenance. At this stage, deterioration is present; however, the capacity or performance of the structure is not affected. More severe damage is imminent unless corrective action is taken. The last category is major maintenance, which involves immediate corrective measures to restore the structure to its original condition [2].

Preventative maintenance includes such things as moisture control, in-place treatments (surface treatments, pastes, and fumigants), and small crack repair by epoxy injection. Remedial maintenance includes posting/splicing, concrete jacketing, PVC wrapping, FRP wrapping, and epoxy injection. Major maintenance includes the addition of supplemental piles and/or removal of deteriorated existing piles.

4. Field Testing of Existing Timber Repair Methods

Upon completion of the field reconnaissance, the researchers along with the technical committee selected four bridges for live load testing. The goal of the testing was to determine how each repair performed when loaded and how that performance differed from that of a non-repaired pile in good condition. Of all bridges considered, three repair systems were tested; these include 1) encasing the weak pile in concrete, 2) posting, and 3) installing additional piles. Each of the tested bridges included at least one of these repairs; the results are discussed in the following sections.

4.1 Bridge Test 1

Bridge 1 is a 38.4 m long bridge with three equal spans. The continuous span superstructure consists of three steel girders and a concrete deck, while the substructure consists of timber piles, five at each abutment and pier. The only timber pile repairs were located at a single pier; two of the five piles were repaired using the concrete encasement method. One of the repaired piles was completely encased, while the other was only partly encased.

One can assume by visual inspection that the concrete encasement stiffens inadequate piles, yet the need exists to quantify the actual force transferred to the encasement. Each of the five piles within the pier were instrumented with multiple strain gages placed to enable quantification of the force carried by the concrete encasement and, when accessible, the timber piles. It is evident by the strain data that the concrete encasement did carry part of the total load imposed on the repaired piles. As one might expect, the strain values measured on the encasement were considerably less than those measured on the timber piles alone. This can be attributed to the substantial difference in total cross-sectional area between the timber pile and concrete, along with the greater modulus of elasticity of the concrete. It is assumed that the concrete encasement does not carry the entirety of the load imposed on the pile. This phenomenon would most likely happen only in circumstances where the entire cross-section of the timber has been lost; this method of repair would not have been appropriate if that were the case. The bridge geometry is mirrored on the centerline of the bridge. Likewise, the load paths of the test vehicle were mirrored on the bridge centerline. Subsequently, when comparing the strain values measured in the fully encapsulated pile to the timber pile on the opposite side of the pier, the percentage of total load introduced to the concrete encasement can be derived. After calculating the force induced into the piles given the strains and cross-sectional properties, it was determined that the concrete encasements carried between 50 and 70 percent of the total load imposed on the respective piles. That is not to say the repaired piles were only capable of carrying 30 to 50 percent of the total load. Rather, it more likely reflects the stiffness of the concrete encasement with respect to the timber and its inherent tendency to carry a greater portion of the load.

4.2 Bridge Test 2

Bridge 2 is a single span 5 m long Greenwood flume bridge. The superstructure consists of timber decking and 21 timber stringers bearing on a timber pile cap, while the substructure at each abutment consists of eight original timber piles and six added timber piles. Additionally, the base of all piles at the waterline was encapsulated in concrete.

The added piles were essentially the same size as the existing piles and were placed directly adjacent to them. The bearing conditions appeared to be consistent between all existing and new piles. After plotting the strain data, it was observed that the strains in the new and old piles were very nearly the same. Assuming that adjacent piles receive equal load, this gives evidence that the load is approximately split between the two piles, thus reducing the load capacity required by any one pile to half.

The condition of the original piles was unknown prior to the addition of new piles and concrete footing. Nonetheless, the remaining visible portion of the original piles is in good condition. With that said, the strain data also gives evidence to the good condition of the visible portion and the effects of the concrete encasement at the base of all piles. With near equal strain values, the pile stiffness and, therefore, condition must be nearly equal. Additionally, the formed concrete footing that encapsulates the bases of each pile provides a solid base for which the load can be transferred from the piling.

4.3 Bridge Test 3

Bridge 3 is a 16 m long bridge with simple spans of 4.5 m, 7 m, and 4.5 m for the first, second, and third spans, respectively. The superstructure consists of 20 timber stringers and timber decking bearing on a timber pile cap; the substructure at each abutment consists of four timber piles, a timber backwall, and timber wingwalls. Additionally, the bottom 0.6 m of piles above ground have been encapsulated by timber planking and concrete infill. Each of the two piers has six piles. Three piles in the eastern pier have been encapsulated in a corrugated metal pipe with concrete infill, whereas in the western pier, one pile has been encapsulated.

As the objective of the load test was to determine how the repaired piles respond to applied load, the researchers decided that strain sensors would be placed on each pier pile and the piles within the eastern abutment. Strain sensors on the pier and abutment piles were placed above and below the encapsulated portions where applicable.

At the piers, the measured strains revealed that part of the applied load was distributed to the formed cast portion around the existing pile. The strain values measured on the cast portion were generally smaller than those measured on the timber-only portion, which should be expected given the difference in total cross-sectional area and combination of materials used at the casted portion. Moreover, where compression and tension strains were measured in the timber-only portions of the pile (top), the strain values in the strengthened portion followed. The total load applied to the piles was calculated assuming the piles were primarily in axial compression. It was also assumed the strains measured at the strengthened portion were uniform throughout each respective cross section. A comparison was made of the total load calculated in each pile to the calculated load distributed to each component (concrete and timber) within the strengthened portion. The observable differences in total load between the top of the pile and strengthened portion can be attributed to such unknown attributes as the modulus of elasticity of each material, slight variances in cross-sectional area, or bending behavior. Nonetheless, it is evident that a significant portion of the load is distributed to the concrete within the strengthened portion of the pile.

At pier 2, the only repaired pile carried very little load in all load cases. It is possible the bearing condition between the pile cap and pile has separated enough to inhibit immediate load transfer. It may also be possible that the non-viewable portions of the pile below the ground line have deteriorated to a condition that prevents load transfer to the ground. Given the apparent load transfer within the casted portions of the piles in pier 1, it can be assumed the cast effectively strengthens and restores stiffness to the pile when the pile is in otherwise good condition beyond the casted portions.

The behavior seen in the abutment piles was consistent with the vehicle configuration and load path traveled. Strain data collected at the face of the timber planking mirrored the data collected near the top of the pile, though the strain magnitudes were different. This phenomenon gives evidence for load sharing between the piles and timber planking; it is likely the load is shared with the concrete infill as well. By visual observation, it was clear the timber planking and concrete infill system shortens the effective length of the pile in the transverse direction, protects the bottom half of the piles from damage due to debris flow, and provides support to the existing backwall.

4.4 Bridge Test 4

Bridge 4 (total length of 18 m) has three spans of equal length. The superstructure consists of 13 timber girders and a timber deck, while the substructure consists of timber pile caps and timber piles, five at each abutment and four at each pier. The only timber pile repair was located at the easternmost pier; one of the four piles was repaired by removing the timber pile from between the ground and the pile cap and replacing it with a steel H-pile.

Only the pier where the pile repair is located was instrumented with strain gages. Strain gages were placed on each pile and the pile cap. Strain results from three load paths were obtained; 0.6 m from left curb, centerline, and 0.6 m from right curb.

The strain results obtained from the test on Bridge 4 were typical of those anticipated by the researchers based on the fairly simple overall structure (simple spans and vertical timber piles with diagonal cross-bracing). However, the results obtained from the steel pile, were unusual especially given that the only connections to the pile were at the pile cap and at the cut end of the original

timber pile, i.e., no diagonal bracing. The researchers anticipated significant compression loading in the steel pile and, at a minimum, compression loading on one side of the pile if bending occurred. Given the position of one load path in particular with respect to the position of the steel pile, it would appear very unlikely the pile would be in tension. However, this was the case. Moreover, the strain results in the original timber pile to which the steel pile was connected indicated compression loading. The researchers questioned the validity of the results because of this unusual and seemingly illogical phenomenon; thus, the researchers decided to retest the pile in question. After retesting the pile, the results from the original test were verified to be correct. Without a much greater amount of instrumentation and significant investigation, this puzzling occurrence may be left unsolved.

This incident may be attributable, at least in part, to the connections of the steel pile to the original pile and pile cap. The pile cap did not achieve full bearing on the post. Rather, the pile cap achieved bearing only on one edge of the post. Additionally, lag screws were used to connect each component. It is possible that when a load was introduced to this connection, the localized load path induced tension into the pile. Regardless of the unusual results, and possibly despite little structural assistance of the repaired pile, the bridge has been able to carry vehicular loads.

5. Development of New Strengthening System

The researchers, after having developed several potential strengthening systems, with the purpose of creating constructible and economical solutions to timber pile strengthening and/or improvement to existing solutions, summarized details for these schemes and subsequently, consulted the TAC to propose lab testing on a selected few. Three were of particular interest; two of which could be completed in the ISU structures lab, while the third would be a field demonstration.

The two options selected for laboratory testing were 1) steel channel attached to opposite sides of the deteriorated portion extending to sound timber above and below, and 2) revision of steel posting connection to enable field adjustment for full bearing.

5.1 Control Specimens

Prior to completing axial load tests on the two selected potential strengthening systems, three control specimens were created using timber piles obtained from former bridge structures. Each was cut to simulate 50 percent cross-sectional area loss. These control specimens were created to compare the results of a strengthened pile load test to that of a non-strengthened pile. Information regarding the material properties and, maybe even more noteworthy, the capacity of a reduced section and failure mechanism is determined from the axial load tests.

The minimum modulus of elasticity was found to be 5.2 Gpa, while the maximum was found to be 9.0 Gpa. The maximum stress calculated for the three tests varied from 21.3 Mpa to 24.7 Mpa.

As was previously mentioned, the piles used to create the specimens were from former bridges within the state of Iowa. Additionally, the sizes used are typical of those at existing bridges. Subsequently, the total capacity of the specimens in and of itself, even with the simulated 50 percent decay was noteworthy. The smallest failure load measured for any of the specimens was 498 kN – a load significantly greater than that individual pile are currently subjected to at existing bridges. One should not assume that piles even in a decayed state can withstand loads to this magnitude, as the field conditions most likely differ from that of a controlled test, i.e., length and degree of decay, lateral unbraced length, induced bending, etc. Rather, one could assume that a significant amount of reserve capacity exists in piles that have experienced moderate decay.

Also noteworthy was the way each of the piles eventually failed. During loading, the reduced section began to balloon in the radial direction until most exterior fibers would splinter and peel away from the specimen. If any checks were present prior to loading, the size of the checks was magnified and propagation often ensued.

5.2 Steel Posting Connection

The ability to remove deteriorated portions of an existing pile and replace it with a steel pile that fits exactly in a given location can be difficult given the conditions beneath bridges and tools required. That is not to say it is impossible because that task has been successfully completed many times. However, a solution that provides field adjustment capabilities could improve the process and

hopefully would achieve full bearing on the replaced pile. In the previous section, the bridge which was the subject of bridge test 4 could have potentially benefitted from a repair method with such field adjustability. As previously noted, the pile cap did not entirely bear on the replacement steel pile and therefore could have potentially contributed to the unusual load transfer through the pile observed during testing.

The researchers created a mockup of a connection that exhibits field adjustability. This mockup consisted of a timber pile section, steel H-pile section welded to a base plate, four 2.5 cm diameter threaded rods, and four 1 cm thick steel angles. Each steel angle was bolted to the timber pile using 1.5 cm diameter lag bolts and leveling nuts were placed between the angles and base plate on the threaded rods. The leveling nuts enabled the adjustment of the base plate.

The connection was tested in axial compression using the laboratory's universal testing machine. The load versus deflection behavior provides evidence the connection has the capacity required in most timber piles. However, the deflection values were higher than desired. This issue could be easily remedied by using a thicker base plate or base plate stiffeners, as a majority of the deflection was a result of base plate bending. The recommended base plate thickness is greater than the 0.5 cm thickness used in the mockup. Near the end of the test, the slope of the load deflection curve significantly increased. This was the result of the base plate coming into contact with the top of the timber pile. This also provides evidence the capacity of the connection would be greater with a thicker base plate as the total load had not yet reached the capacity of the pile.

5.3 Steel Sisters

Commonly, when loss of section is discovered in timber piles and where visual inspection may indicate an inadequate pile, only a short length of the pile has decreased load carrying capacity. A majority of the pile may very well be intact and able to withstand the desired vehicular loading. Where this is the case and localized section loss has advanced to a degree where replacement or reinforcement is desired, a method that could be considered is to sister steel sections to the pile. This method is comprised of spanning and reinforcing the damaged or decayed portion of the pile with steel "sisters" which are anchored above and below within the remaining solid portions of the pile.

The researchers included this method is their laboratory investigation and testing. A pile was modified to simulate a 50 percent cross-sectional area loss over a one foot length. Two M6x4.4 (A =8.32 cm^2) steel sections were used to span the section loss and were anchored by four 2.5 cm diameter threaded steel through rods. Both steel sections were instrumented with strain gages on each flange at the midpoint of simulated decay.

The specimen was axially loaded for purposes of comparison with the control section without the steel sisters. It was found in this test that the steel resisted minimal load until the failure of the pile was imminent and the steel sisters became engaged – the tolerances around the anchor rods were enough that a certain amount of deformation in the pile was required before the rods would bear on the sisters. Even with the simulated 50 percent section loss, the pile performed very well before the sisters were engaged. In fact, the performance was well enough to withstand loads commonly seen by these piles due to vehicular traffic. Nonetheless, with minor changes in how the sisters are attached to the pile, it is likely that one would be able to engage the steel sections almost immediately upon loading.

The observed maximum stress of 25.6 MPa in the timber is on the same order as that observed in the control specimens, thus providing assurance that the sistered pile was a representative specimen. Additionally, upon failure of the pile, the maximum stress observed in the steel was only 80.6 MPa – well below the yield stress of steel. This would lead one to believe that the overall capacity of the pile would be even greater than the capacity achieved in the load test if the steel were fully engaged earlier in the loading process. Even more, the possibility exists that if the test was not stopped the steel would have become fully engaged and the total load would have reached levels corresponding to the yield stress of the steel.

Similar to the load versus deflection curves of the control specimens, the rate of deflection increased significantly only after reaching approximately 7.6 mm of total deflection. This gives more evidence that the specimen was representative of other timber piles and the fact that the steel was not becoming fully engaged until the timber began to fail.

6. Summary, Conclusions, and Recommendations

Tens of thousands of bridges within Iowa are maintained at the county level. Of those bridges, a significant percentage are constructed using timber piles, girders, and decks. The superstructures are often sufficient to carry the traffic for which they are required. Even so, the advancing decay of substructure elements, such as timber piles, pose a problem to the longevity of the overall structure. With such a large number of bridges requiring attention and the fact that available funds are decreasing while maintenance costs are increasing, it becomes important to improve or identify the best currently used maintenance methods for timber substructures.

The objectives of this research were to complete the following:

- Review existing products for timber preservation and repair and to document their effectiveness in extending the service life of various bridge components.
- Determine techniques used by county and other engineers to repair and restore the load carrying capacity of piling damaged by deterioration and cracking.
- Review methods used to repair failed piling.
- Determine/develop effective methods for transferring bridge loads through the failed portion of the pile.
- Determine that safe load capacity is restored by the repair methods (existing or new) determined to be structurally efficient.

Deterioration of timber substructure elements can be attributed to either biological or physical deterioration mechanisms. Included in the biological mechanisms are decay fungi, termites, powderpost beetles, and carpenter ants. These mechanisms often have a direct correlation with the temperature and moisture conditions present. Alternatively, physical deterioration mechanisms include abrasion, debris contact, and overloading.

Condition assessment should be conducted using a multitude of tools. These tools include 1) visual assessment, 2) probing and picking, 3) moisture measurement, 4) sounding, 5) stress wave devices, 6) drill resistance devices, 7) core boring, and 8) preservative retention analysis. Any single method may give an incomplete or inaccurate assessment of the given substructure element.

Maintenance activities depend entirely on the extent of deterioration present within the substructure element. Depending if the deterioration is minor, moderate, or severe, the maintenance activities will either be preventative, remedial, or major, respectively. Preventative maintenance includes moisture control, in-place treatments, and/or epoxy injection of small to medium sized cracks. Remedial maintenance includes posting/splicing by means of mechanical splicing, concrete jacketing, FRP or PVC wraps, and/or injection of epoxy. Major maintenance corrective measures are conducted when deterioration has progressed to the point where major structural components have experienced moderate to severe strength loss and repair or replacement is mandatory to maintain the load carrying capacity. Often the only method that can be employed with this level of decay is to install supplemental piles.

Four Iowa bridges utilizing different methods of repair or strengthening were subjected to live load testing. In each of these tests, the repairs proved to be effective in that the desired stiffness was restored.

At the first of these bridges, corrugated metal pipe was used to create a form around the decayed or damaged portions of the pile which was filled with concrete, thereby creating a cast and providing additional stiffness. The near term performance of this method appears to be adequate to maintain a functioning bridge. Being as the method of repair has not been observed over the long term, conclusions regarding its indefinite performance cannot be made.

At the second bridge, supplemental piles were placed adjacent to each existing pile. Though seemingly a more expensive option, when installed correctly, this method effectively restores the bridge substructure system to its original condition. Theoretically, the original piles would not require additional maintenance procedures and could progressively lose bearing capacity without any adverse effects on overall bridge performance.

At the third bridge, a cast system similar to that used in the first bridge was used to stiffen the pier

piles, whereas at the abutment piles, timber planking was installed across the stream-side face of the piles and the created void between the planking and existing backwall was subsequently filled with concrete. This method, at a minimum, provides much greater protection to the piles from debris flows. Even more, the piles are reinforced in the transverse direction and, as such, may have a greater bearing capacity.

At the fourth bridge, a posting method of repair was used. One pile had been partially removed and replaced with a steel section extending from the sound portion of the existing pile near the ground surface to the pile cap. If installed correctly and proper bearing is achieved at the pile cap and existing pile, the method is quite adequate. One should note that only select piles in any one pile bent should be repaired using this method, as the lateral stiffness in the piles and, therefore, the bridge would be lost at the pile/post connection.

Following the completion of field testing of four Iowa bridges, details of new strengthening systems were developed with the purpose of creating or improving constructible and economical solutions to timber pile strengthening needs. Laboratory testing was completed for two of these solutions.

The first solution involved modifying the existing method of posting with a steel H-pile or the like. Field adjustment of the spliced portion can be necessary when not fabricated to exactly fit the removed portion of pile. A base plate and leveling bolts were implemented to allow for vertical adjustment at the connection between the existing timber pile and new steel post; the connection detail proved to be a promising solution.

The second strengthening system entailed adding steel “sisters” to a decayed or damaged pile. Each sister was bolted to the pile opposite of each other and extended beyond the simulated section loss. In the end, the “sisters” only aided in the strengthening when failure in the remaining portion of the pile was imminent, though it is assumed that modification to the connection details would engage the sisters earlier in the loading process.

The researchers provide the following recommendations regarding the assessment, preservation, repair, and rehabilitation of timber substructure elements.

- Utilize multiple methods to more accurately assess the condition of timber substructure elements including any or all of those previously mentioned in the summary.
- Make provisions for physically protecting timber structure elements from environmental conditions (e.g., precipitation), debris, and other damage-causing objects.
- Adhere to the AWPA Standards for the treatment and care of timber bridge elements.
- Be cognizant of applying preservative treatments to cut or fastened portions of timber substructure elements to avoid point of entry for biological decay mechanisms.
- When decay or damage is present, conduct maintenance activities at earliest possible stage to avoid increased cost associated with maintenance postponement.
- The addition of mild-steel reinforcement in the form of angles, channels, w-shapes, or similar has the ability to provide increased load capacity to mild or moderately decayed existing pile.
- Field adjustability can be achieved with few minor and relatively inexpensive parts when completing the posting method of repair.
- The current method of casting a single pile with corrugated steel pipe and concrete effectively restores the desired stiffness within the casted portion of the pile; this method has been used in numerous locations around the state of Iowa.

7. References

[1] Dahlberg, J., Phares, B., Bigelow, J., and Klaiber F., "Timber Abutment Piling and Back Wall Rehabilitation and Repair", IHRB Project TR-616, InTrans Project 10-372. Institute for Transportation, Iowa State University. Ames, IA, 2012.

[2] White, D., Mekkawy, M., Klaiber, W., and Wipf T., "Investigation of Steel-Stringer Bridges: Substructure and Superstructure, Volume II", IHRB Project TR-522; CTRE Project 04-184. Center for Transportation Research and Education, Iowa State University. Ames, IA, 2007.

[3] Bigelow, J., Clausen, C., Lebow, S., and Greimann L., "Field Evaluation of Timber Preservation Treatments for Highway Applications" IHRB Project TR-552, CTRE Project 06-252. Center for Transportation Research and Education, Iowa State University. Ames, IA, 2007.

[4] Highley, T.L. Ch. 13. Biodeterioration of Wood. In "*Wood handbook-Wood as an Engineering Material", Gen Tech. Rep. FPL-GTR-113.* Madison, WI: USDA, Forest Service, Forest Products Laboratory, 1999.

[5] U.S. Army Corps of Engineers, Naval Facilities Engineering Command, and Air Force Civil Engineering Support Agency. (2001). *Unified facilities criteria (UFC)—Maintenance and Operation: Maintenance of Waterfront Facilities*, Publication No. UCF 4-150-07, Washington, D.C.

[6] U.S. Department of Agriculture, Forest Service, Forest Products Laboratory (1999). *Wood Handbook: Wood as an Engineering Material, Gen. Tech. Rep. FPL-GTR-113*. Madison, WI.

[7] Buslov, V. M., and Scola P.T., "Inspection and Structural Evaluation of Timber Piers: Case Study." *Journal of Structural Engineering*, 117(9), 1991, 2725-2741.

[8] Manuel, F. S. "*Evaluation and Improvement of Existing Bridge Foundations", Report No. FHWA/RD-83/061*, 1984, Federal Highway Administration, Mclean, VA.

[9] Ritter, M. A., "*Timber Bridges: Design, Construction, Inspection, and Maintenance. United States Department of Agriculture",* Forest Service, Forest Products Laboratory, Madison, WI, 1992.

Fiberglass Composite Bridges

Authors:C,Dewar, J.Dewar, B.Dewar
Occupation: Engineering, Manufacturing
Organisation: Guardian Bridge Rapid Construction Inc.
Stratford ON Canada
crawford@bridgedecks.ca

Engineered Wood
Advanced Fibers
Advanced Thermosets
Hybrid Composite structures

Summary

Fiberglass Composite Bridges as structural prefab elements offers tremendous potential to meet owners critical needs at bidders price, for rehab or new construction. At approx. 1% bridge deck inventory, wood bridges in Canada and the United States have lost market share primarily to concrete which is not due to a lack of wood, but rather technical issues regarding environmental, durability and capacity which can be resolved using hybrid means/methods to save time and money.

Keywords: fiberglass composite bridges at bidder's price

1. Introduction

1.1 Problem

Engineering firms hired by bridge owners assign the design engineer to produce three design concepts and associated costs which more than often does not include wood because service loads with timber low modulus of elasticity radially results in higher fabrication costs of due to more superstructure elements such as girders, diaphragms compared to concrete decks. Timbers increased depth of section compared to concrete results in disqualification during 100 year flood, hydrology affects.

1.2 Solution

Concrete is poor in tension and engineers compensate with internal reinforcement. Wood benefits from external reinforcement to solve the problem of durability and capacity. Since 2000 the Canadian Highway Bridge Design Code Section 16 has allowed consideration of fiberglass composite. 70% of the bridge inventory is short and medium span bridges. The following case study is an example of our rigid span structure in fiberglass composite compared to concrete.

1.3 Implementation

Concrete rigid span structures are vetted in design contracts because in the opinion of engineering consultants it has many attractive features such as prefabricated and benefits in conformance to the CHBDC CL625 ON Truck with a design service life of 75+ years. Rigid Span structures are used by engineers concerned about the environmental assessment fit, hydrology and 100 year flood obligations. The Rigid Span structure by design is well suited to mitigate flooding/damming compared to slab on girder designs where the potential for flooding debris can lodge up into/ between the girders compared to a flat bottom design with Rigid Span structures which often deflects the material through the length and width reducing the risk of the bridge from acting as a dam. To further understand the compelling argument of fiberglass composite in comparison to precast concrete, refer to Figure 1 & Figure 2.

1.4 Figures

Figure 1

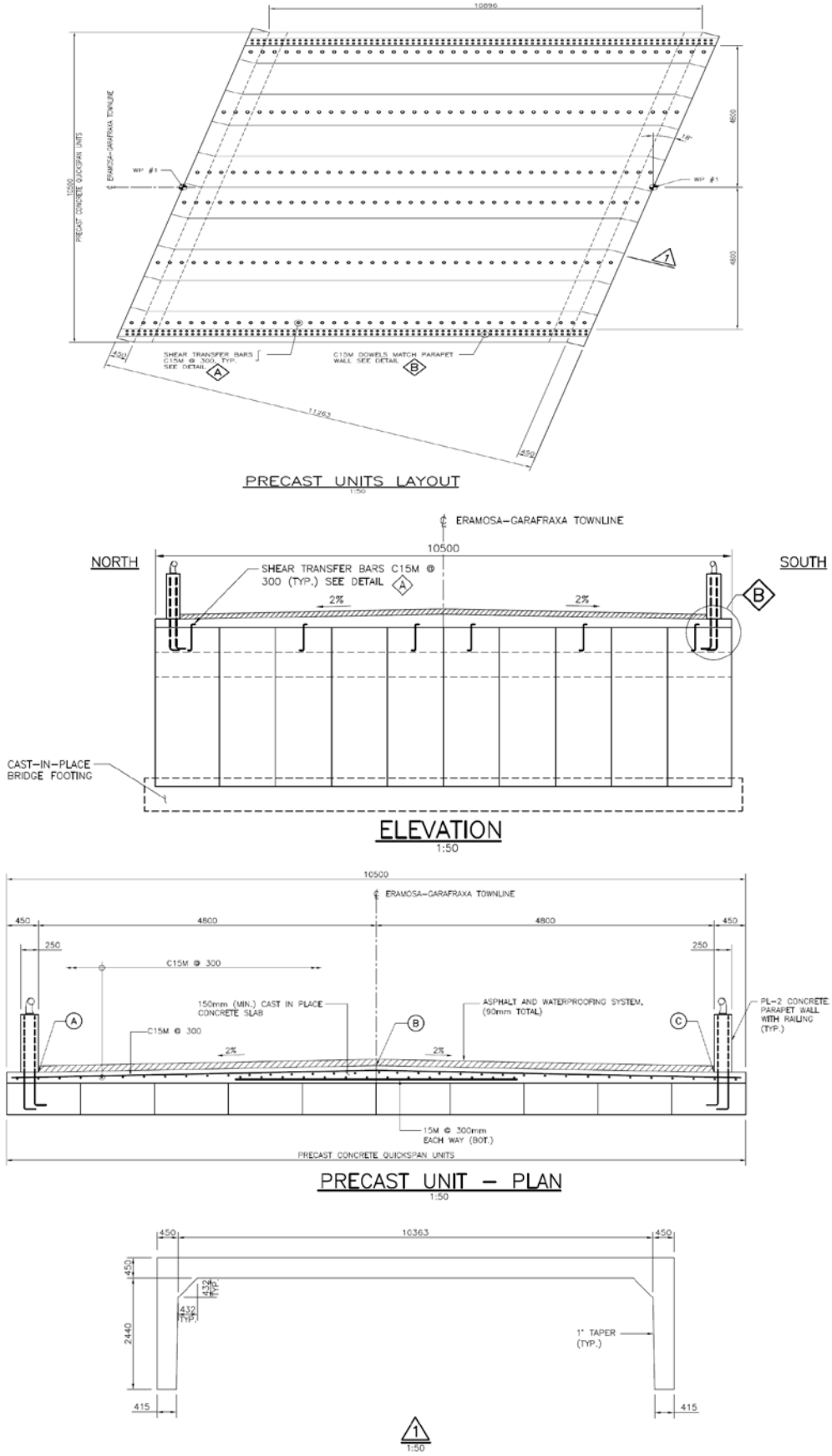


Figure 2

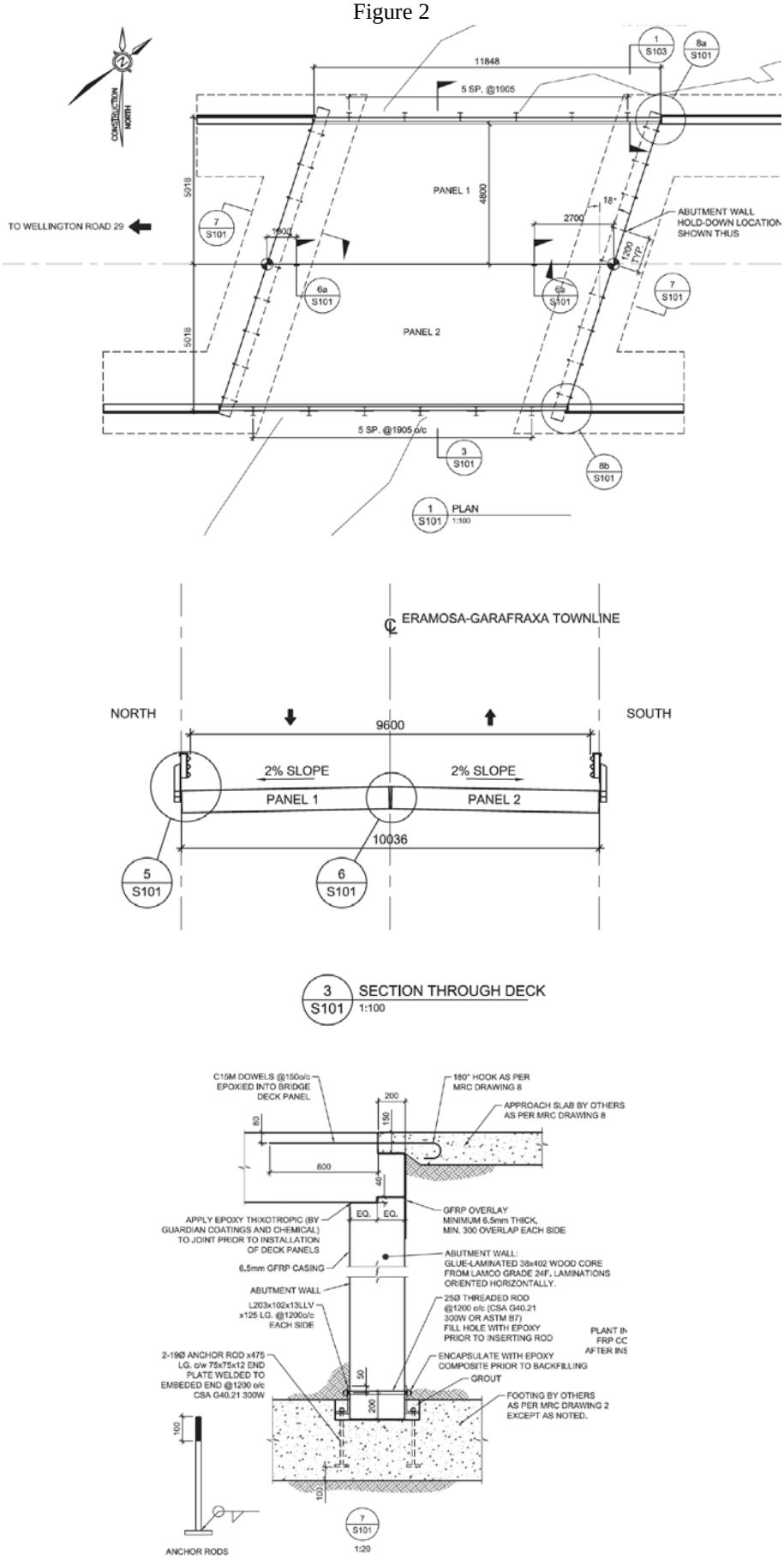

CONSTRUCTION NORTH
11848
5 SP. @1905
PANEL 1
PANEL 2
4800
5018
5018
TO WELLINGTON ROAD 29
2700
18°
ABUTMENT WALL HOLD-DOWN LOCATION SHOWN THUS
1200 TYP
5 SP. @1905 o/c
1 S101 PLAN 1:100
ERAMOSA-GARAFRAXA TOWNLINE
NORTH
SOUTH
9600
2% SLOPE
2% SLOPE
PANEL 1
PANEL 2
10036
3 S101 SECTION THROUGH DECK 1:100
C15M DOWELS @150o/c EPOXIED INTO BRIDGE DECK PANEL
180° HOOK AS PER MRC DRAWING 8
APPROACH SLAB BY OTHERS AS PER MRC DRAWING 8
APPLY EPOXY THIXOTROPIC (BY GUARDIAN COATINGS AND CHEMICAL) TO JOINT PRIOR TO INSTALLATION OF DECK PANELS
GFRP OVERLAY MINIMUM 6.5mm THICK, MIN. 300 OVERLAP EACH SIDE
6.5mm GFRP CASING
ABUTMENT WALL: GLUE-LAMINATED 38x402 WOOD CORE FROM LAMCO GRADE 24F, LAMINATIONS ORIENTED HORIZONTALLY.
ABUTMENT WALL
L203x102x13LLV x125 LG. @1200o/c EACH SIDE
25Ø THREADED ROD @1200 o/c (CSA G40.21 300W OR ASTM B7) FILL HOLE WITH EPOXY PRIOR TO INSERTING ROD
2-19Ø ANCHOR ROD x475 LG. c/w 75x75x12 END PLATE WELDED TO EMBEDED END @1200 o/c CSA G40.21 300W
ENCAPSULATE WITH EPOXY COMPOSITE PRIOR TO BACKFILLING
GROUT
FOOTING BY OTHERS AS PER MRC DRAWING 2 EXCEPT AS NOTED.
ANCHOR RODS
7 S101 1:20

Figure 1 shows a precast concrete design with 10 rigid span structures at 450mm deep plus the added work scope item of dowels, rebar with a 150mm cast in place concrete deck.

Figure 2 has a fiberglass composite design with 4 prefabricated elements being two abutment walls and two slab spans with a depth of section of 508mm.

The bridge owner chose the fiberglass composite design proposal for the following reasons

- Bidders price cost in place 20% less than concrete
- 92mm less depth of section
- Depth of section ideally suited for hydrology
- Depth of section conforms to 100 year flood
- Conformance to CHBDC CL625 ON Truck
- Durability at 100+ years
- Product immune to water ingress and de-icing materials

The disadvantage overcome in the design was the PL3 Jersey type barrier specified in the concrete design as to our knowledge there exists no proven crash test method for PL3 to a timber deck design. We gratefully acknowledge the USDA , Forest Products Lab, U.S. DOT FHA for the "Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks" which was submitted an approved for use by Designer Cory Zurell Ph.D P.Eng Blackwell Bowick Engineering in Waterloo Ontario. In our opinion, PL3 remains a major disadvantage as AADT volumes demand the use of PL3 thus disqualifying PL1 or PL2 designs. It would highly advantageous for a PL3 design using timber.

1.5 Manufacturing to Site

Our manufacturing allows us to press timber up to 1.2m deep x 5.5m wide x 15m long with an extension frame to a length of 30m.

The fiberglass composite slab spans have a manufactured SPF wood core, vertically laminated 495mm deep x 5.018m wide x 11.848m long with a 18 degree skew to be cut after pressing.

Example of vertically laminated wood deck

The fiberglass composite slab manufactured as a monolithic slab to span from abutment to abutment. Each panels weighs 19,660 kg or approx. one fifth the weight of concrete.

Fiberglass Composite Bridge Slab

Fiberglass composite bridge slab on edge

Robust material can be picked up from CL Span

And rotated using slings.

Picture also shows skew.

Fiberglass composite bridge slabs

Both panels can be prefitted at shop

Prior to shipping

Payment Certifier can inspect

Reduces risk on site

Promotes confidence and credibility

Fiberglass composite abutment

Two abutments

Dimensions:

400mm thick

2.46m high with a 2% cross slope from CL

12m long

Weight 5498 kg

Panel One Fiberglass Composite Bridge Slab

Using a 400 ton crane

Panel one is lifted in place in thirty minutes

Each abutment bearing has adhesive applied

Set in place requires four rigging persons

Fiberglass composite abutments, Panel One & Two

Installation using four rigging persons

After panel one set in place

Rig panel two and apply adhesive to panel one

longitudinal joint and abutments in prep for panel

two lift and set in place.

Fiberglass Composite Rigid Span installed

Installation complete

Longitudinal joint with adhesive

Abutments joints with adhesive

Exotherm set adhesive in 160 minutes permanent

No mechanical connections required

Anchor rods used for footings and wing walls

Project completed

PL2 Railing

Asphalt on approaches and bridge

Completed on time and budget

1.6 Niche Market

Truss structures, bascule, swing bridges benefit greatly from our lightweight high strength ratios as we are able to remove all steel stringers and span from floor beam to floor beam with a typical dead load savings up to 40% compared to open steel grating, steel orthotropic decks, 80% less than concrete and were desired build in a 2% cross fall from CL. In the US there are approx. 7,800 of these structures with 5.3 million square meters of bridge deck to be replaced with a construction value of 8 Billion dollars.

1.7 Conclusion

The wood industry in North America has done a superb job of cutting big sticks into little sticks but has lost market share in the bridge industry at approx. 1% in the US but it would be very beneficial for all wood manufacturers to support value added wood programs, such as Wood Works in North America, at the expense of the concrete industries market share as bridge owners are looking for alternative design solutions in tandem with compelling evidence of value added engineering at bidder's price that saves them time and money.

Fiberglass composite structural elements are readily available off the shelf, shovel ready and offer tremendous potential to meet the critical needs of rehab or new construction because hybrid design, using engineered wood, advanced composite materials in tandem with a proprietary process to produce merchant quality robust, durable products with high strength, low weight ratios lend themselves well to prefabricated, modular parts in width and length, with a cost in place less than concrete, as constructability on site is accelerated and completed in days versus months with the value added features of immune to water ingress, salt and de-icing materials with a service life 2 to 3 times greater than concrete and provide redundant load paths that satisfy in service stiffness and exceed section strength requirements as ultimate loads provide for safety factors in excess of five when compared to in service conditions.

1.8 References

[1] Canadian Highway Bridge Design Code

[2] FHWA

[3] AASHTO

[4] USDA Forest Service, Forest Products Laboratory

1.9 Acknowledgements

- County of Wellington Gord Ough P.Eng Mark Eby P.Eng
- Centre Wellington Township Council
- McCormick Rankin Bob Stofko P.Eng
- Blackwell Bowick Engineering David Bowick P.Eng Cory Zurell PhD P.Eng

National Historic Covered Bridge Preservation Program

Sheila Rimal Duwadi, P.E
Team Leader, Hazard Mitigation
Federal Highway Administration
McLean, VA 22101
sheila.duwadi@dot.gov

Sheila, in addition to being a Team Leader, manages the Timber Bridge and the National Historic Covered Bridge Preservation RD&T Programs for the Federal Highway Administration at the Turner Fairbank Highway Research Center. She is a registered professional engineer.

Summary

The National Historic Covered Bridge Preservation (NHCBP) program administered by the Federal Highway Administration since 2000 has included preservation, rehabilitation and restoration of covered bridges; and research, education and technology transfer. The program has provided opportunity to save many of our covered bridges from destruction by providing funds for restoration; and through research, education and technology transfer, develop better means to preserve, educate and inform the community. For the period from 2000 to 2012, more than 200 bridges in 24 States, and over 50 individual research projects have been funded. This paper will provide an overview of the NHCBP program over this year period.

Keywords: covered bridges, timber bridges, bridges, wood, historic structures.

1. Introduction

The 1998 *Transportation Equity Act (TEA21) for the 21st Century* as amended by the TEA21 Restoration Act established the National Historic Covered Bridge Preservation (NHCBP) program. It was subsequently continued in the succeeding 2005 Highway Legislation, the *Safe, Accountable, Flexible, Efficient Transportation Equity Act – A Legacy for Users* or SAFETEA-LU for short. However, the program is not included in the current 2012 Highway Legislation, *Moving Ahead for Progress in the 21st Century* or MAP-21. The NHCBP program included preservation, rehabilitation or restoration of covered bridges that were listed or were eligible for listing on the National Register of Historic Places; and research for better means of restoring, and protecting covered bridges, development of educational programs, and technology transfer to disseminate information on covered bridges as a means of preserving our cultural heritage. In implementing the program, grants were awarded to the States through annual solicitations by the Federal Highway Administration to preserve, rehabilitate and/or restore these structures. Emphasis on research, education, and technology transfer was focused on finding means and methods for repairing, strengthening and treating covered bridges; developing historical documentation and educational videos and manuals; and on workshops and seminars.

The NHCBP program provided opportunity to save many of our covered bridges from destruction, and through research, ways on improving preservations practices. Research has also provided opportunity to develop detailed engineering records of these bridges so that we have the opportunity to learn from the past in order to build innovations for the future. What follows is a synopsis of the program.

2. Grants for Preservation

Administered through the FHWA Office of Bridge Technology, the FHWA awarded grants to States through annual solicitations for preservation of eligible bridges awarding approximately $87M to save, protect, and rehabilitate many covered bridges across the Nation. More than 200 bridges in 24 states have received funds.

Some of the bridges the FHWA has funded follow with a complete list to be found at www.fhwa.dot.gov/bridge/covered.cfm

Clarkson/Legg Covered Bridge over Crooked Creek- Cullman Co, Alabama -This is a 82.3m (270 ft.) two span Town lattice truss bridge, built in 1902, and the second longest covered bridge in the southern U.S. It represents an era of civil engineering dating from 1840 to early 1900.

Comstock Covered Bridge - East Hampton, Connecticut - This is one of the three remaining covered bridges in Connecticut. Of the three, it is the only one still carrying its own weight with the original timber superstructure. The main span is a 27.4m (90 ft.) long Howe truss built in approximately 1840. It is one of the oldest, if not the oldest, remaining original Howe truss in existence. The truss and floor beams date from the original time of construction, including the original iron rods. The approach span is a 11.0m (36 ft.) long Queen post pony truss constructed in 1791.

Thompson Mill Covered Bridge over Kaskaskia River- Shelby Co, Illinois - This is a five span 49.1m (161.2 ft.) long bridge built in 1868. It is one of five 19th century covered bridges in Illinois and one of only two of the Howe truss types.

Norris Ford Covered Bridge - Rush County Bridge #100, Rush County, Indiana - This is a 45.1m (148 ft.) long single span Burr Arch truss built in 1916. The bridge was built by E.L. Kennedy & Sons and is one of 13 remaining bridges of the 58 built by the Kennedys.

Six surviving covered bridges of Madison County, Iowa

1. Imes covered bridge is a 25.0m (82 ft.) long Town lattice truss built in 1870 - 1871 by J.P. Clark. It has two wood trestle approaches.
2. Cutler - Donahue covered bridge is a 25.0m (82 ft.) long Town lattice truss built by Eli Cox in 1871. It has two wood tress approaches.
3. Hogback covered bridge is a 29.9m (98 ft.) long Town arch lattice truss with three wood approaches totaling 48.8m (160 ft.), built by H.P. Jones in 1884.
4. Holliwell covered bridge is a 33.2m (109 ft.) Bowed arch truss with two wood approaches totaling 47.2m (155 ft.) built by H.P. Jones in 1880. (*see Fig. 1*)
5. Roseman covered bridge is a 31.7m (104 ft.) Town arch lattice truss with four wood approaches totaling 62.5m (205 ft.) built by H.P. Jones in 1883.
6. Cedar covered bridge was a 21.3m (70 ft.) town Town arch Lattice truss with three wood approaches totaling 39.6m (130 ft.) built by H.P. Jones in 1883. This bridge has subsequently been destroyed by fire and a replica structure was completed in 2002.

Bennett's Mill Covered Bridge - Greenup County, Kentucky - This 1855 bridge is a 48.3m (158.5 ft.) single span Wheeler truss. It is one of 13 remaining covered bridges in Kentucky.

Bennett Covered Bridge - Oxford County, Maine - Built in 1901, this is a single span 28.3m (93 ft.) Paddleford truss. It is the latest of five identified Paddleford truss bridges from 1857 to 1898.

Figure 1. Holliwell Bowed Arch Covered Bridge in Madison County, Iowa.

Fitches Covered Bridge - Delaware County, New York - This 30.5m (100 ft.) single span Town lattice truss was originally built in 1870 and located within the Village of Delhi. In 1885 it was dismantled and relocated to its current site about 4.8km (3 mi.) north of Delhi to make room for a 'modern' iron bridge.

Ponn Covered Bridge - Vinton County, Ohio – This bridge built in 1874 spanned 54.9m (180 ft.) across Raccoon Creek. It has a humpback feature which made it the only one of its kind in Ohio. The south end of the bridge was a double Queenpost Truss, the north end was a Multiple Kingpost Truss and the middle was a combination of a Multiple Kingpost with an arch. The bridge was destroyed by fire in June, 2013.

Fisher School Covered Bridge - Lincoln County, Oregon - This bridge built in 1919 is the fifth oldest Howe truss in Oregon. The bridge's unique features include semi-elliptical portal arches, ribbon openings, and flared side walls. It is the longest span covered bridge in the county at 22.0m (72 ft.).

Knapps Covered Bridge - Bradford County, Pennsylvania - This is a 27.1m (89 ft.) single span Burr Truss built in 1853.

Sanderson Covered Bridge - Rutland County, Vermont - This bridge built in 1840 is a single span 40.2m (132 ft.) long Town lattice truss.

Greenbanks Hollow Covered Bridge - Caledonia County, Vermont - This 23.8m (78 ft.) long Queenpost truss was built in 1886 as a single span. When it was reconstructed in 1976 two concrete piers were built at approximately 2/3 of the span making it into a 3 span bridge.

Humpback Covered Bridge – Alleghany County, Virginia – This 30.5m (100 ft.) long single span covered bridge was built in 1857, and is the oldest remaining covered bridge in Virginia. It received its National Historic Landmark Status in 2012.

Fletcher Covered Bridge - Harrison County, West Virginia - This bridge built around 1891 is a 17.6m (57.7 ft.) single span Kingpost truss. It is one of seventeen remaining covered bridges in the state of West Virginia, and one of nine still carrying vehicular traffic.

3. Research, Education and Technology Transfer

Administered through the FHWA Office of Infrastructure R&D, the FHWA worked through partnership and close collaboration with the National Park Service – Historic American Engineering Record and the Forest Service – Forest Products Laboratory to identify, prioritize and conduct RD&T. More than 50 projects have been initiated and are at various stages of completion.

While the NHCBP program made available dedicated funding for preservation, restoration and rehabilitation of historic covered bridges, a practical 'how to' guide of use to engineers and preservationists alike to rehabilitate and restore these bridges was lacking. As a result effort was undertaken to develop such a resource. The *'Covered Bridge Manual'* was published in 2005 and is available through the Federal Highway Administration, both in print and electronic version at: www.fhwa.dot.gov/publications/research/infrastructure/structures/04098/04098.pdf

Wood if unprotected can deteriorate due to biological activity. During the era when covered bridges were built there were no chemical treatments available to prevent fungi and insects from attacking and colonizing the wood and thereby deteriorating the structure. In order to prevent this deterioration, the method used was to cover the bridge with roof and sides thereby eliminating moisture. Although covering a bridge keeps the wood dry and prevents wood components from deterioration, overtime moisture can migrate and lead to suitable conditions for colonization by wood destroying organisms. There are many covered bridges that have had be repaired due to deterioration. Therefore, knowing that chemical preservatives exist today which can prolong the life of wood components and also reduce maintenance work, it became a priority to identify treatments that could be used to preserve wood members while still being acceptable to the historic preservation community. The second concern was 'fire' and the need to identify fire retardant treatments as many wooden structures are burnt down by vandals. While large timbers have inherent fire resistive characteristics that eliminate the need for Fire Retardant Treatments (FRTs), these treatments add extra layer of protection giving more response time for authorities. Two studies were initiated, one dealing with identification of preservative treatments and fumigants for treating historic covered bridges and the other on effects of fire retardant treatments on covered bridges. The *Guide for In-Place Treatment of Wood in Historic Covered and Modern Bridges* published in 2012 compiles results of these studies and is available online at http://www.fpl.fs.fed.us/documnts/fplgtr/fpl_gtr205.pdf

Protecting wood member from deterioration by use of chemical treatments is important to prevent decay, but many of the older, proven preservative treatments have use restrictions, and may not be acceptable for historic preservation. Some of the newer systems that are in use today have had

tendencies to accelerate the corrosion of metal fasteners. The *corrosion of fasteners in wood treated with newer wood preservatives* study was initiated to evaluate corrosion performance of ferrous and nonferrous fasteners and degradation of non-metallic fasteners in wood treated with newer preservatives, specifically, alkaline copper quaternary (ACQ), Copper azaole (CuAz) and micronized copper quaternary (MCW) and examine feasibility of rapid test to evaluate corrosion; and examine role played by moist environment.

In addition to identifying suitable chemical wood preservatives that can be used to prolong member life, and yet not negatively impact connectors and connections, a study has been underway to *evaluate naturally durable wood species* where the wood would not have to be treated. Outdoor tests are being conducted on a number of wood species replicating the covered bridge environment.

In order to enhance transportation education at all levels and to build a bridge between America's youth and the transportation community, a study was undertaken to develop a comprehensive guide on covered bridges in the United States to be used by educators in developing lesson plans for K-12 for math, science and American history courses. The *educational guide on the history of covered bridges in the United States* available in hard copy and online at www.woodcenter.org/covered_bridges_education is formatted in sections for grades K-5, 6-8, and 9-12 and includes compilation of all different types of covered bridges used in the United States, and their history. It includes a pictorial coverage, and descriptive drawings including bridge type, year built, design loads, traffic, wood species used, information on the designer/builder, and much more. The Guide has two components: printed lesson plans for educators and a companion disk with interactive and animated elements for students.

Additionally, a documentary entitled *Bridge to the Past* was funded that has been broadcast Nationwide on Public Television stations.

Accurate documentation of historic structures is essential, as often these structures are old and fragile, and can be destroyed as in the case of the Bartonsville covered bridge swept away from raging waters during tropical storm Irene in Vermont. A large effort has gone into developing documentation for historically significant covered bridges. Through the *HAER level 1 documentation* measured and interpretive drawings, large format photographs, and written historical reports have been produced for many bridges. The documentation continues to be developed using experts in the field of cultural resources. Included in the historical reports are detailed engineering analyses of both the unusual (Haupt, Post, Paddleford, Smith) and standard (Town, Howe, Burr) truss configurations, assessing their behavior and performance characteristics. The documentation provides a rich history of the covered bridges. Completed works are available through the Historic American Engineering Record (HAER) Collection, Prints and Photographs Division at the Library of Congress at www.loc.gov/pictures/collection/hh/

Using the detailed local and regional information generated by HAER Level I documentation of significant covered bridges, as well as material contained in existing HABS/HAER documentation the Program is supporting a study linking covered bridge construction techniques with local and regional wood construction practices, and a book on *Covered Bridges and Vernacular Construction Practices* is being developed. The book will provide information on the extent to which local and regional practices of barn and house construction influenced bridge construction techniques and materials. In addition, a collective biography of bridge builders and their links to local and/or regional building practices will be reported.

Accurately documenting covered bridge as is done under the HAER Level 1 documentation requires taking field measurements of the bridges and its various components in the detail necessary to develop interpretative drawings. *Use of Laser Scanning Technology to obtain As-Build Records* is studying the feasibility of utilizing advanced cameras and laser scans to generate as-built drawings that are less labor intensive, yet produce the same level of detail. If feasible, this could greatly shorten the recording process.

The *World Guide to Covered Bridges* is a database of all covered bridges in the United States. An interactive site has been developed where the data can be searched to provide location, condition, descriptive information and the history of covered bridges. It can be accessed online at www.woodcenter.org website. For the covered bridge enthusiasts, it gives the location and history of the bridge, and can be an useful tool if planning a trip to visit covered bridges.

The highest official recognition of historic properties is a designation as a National Historic Landmark. The NHCBP program has supported the *National Historic Landmark Theme study* to assess and nominate covered bridges for National Historic Landmark status. Through this effort two bridges have now been granted the National Historic Landmark status. These include the Knight's Ferry covered bridge in California a 1864 Howe truss, and the Humpback covered bridge in Virginia a 1857 multiple kingpost truss.

The *Secretary of the Interior's Standards for the Treatment of Historic Properties* and accompanying Guides are the documents used in preservation, restoration, rehabilitation and reconstruction of historic properties. These documents provide the do's and the don'ts in historic preservation, but have been developed with emphasis towards buildings rather than transportation systems. Therefore, a study is being supported to develop similar document focused on covered bridges. The *Covered Bridge Preservation Guide* will be similar in concept and of use to any agency or group undertaking a covered bridge preservation project. The *Guide* will list treatments that should be embraced as well as avoided.

Because covered bridges are older structures designed to carry much lighter loads than the heavy vehicles of today, several studies have been funded to understand their load carrying capacity, develop methods to strengthen members, develop more accurate rating procedures, and identify ways to decrease dead load. *Improved ratings for covered bridges through load testing* will develop rating procedures for reliably determining safe load-carrying capacity of these bridges. *Strengthening historic covered bridges to carry modern traffic* assessed the use of Fiber-Reinforced Polymers as a means to strength components. *Improved analytical techniques for historic covered bridges* will recommend improved analysis methods that have been validated through field load test data to determine behavior of various covered bridge types. *Lightweight floor replacement systems* will assess commonly used systems that have been proven for bridge applications for covered bridge use. *Residual capacity of structural wood members in historic covered bridges* will provide guidance on determining remaining capacity of members.

Unlike steel and concrete, engineering properties of wood can vary both within and between species. Additionally, wood is orthotropic which means material properties vary depending on grain direction. Therefore, wood design values given in specifications and standards are based on statistical samples with large amount of safety factor to account for the variability. Often the type, source and grade of existing wood in old covered bridge are difficult to identify, but the current practice is to guess the species and select wood values from existing specifications. In *Species identification and field-grading* a field manual is being developed which can be used to more easily

determine the timber species and help in identifying the grade, and therefore more reliably predicting engineering properties.

Timber trusses that make covered bridges were engineered structures that set the stage to modern day trusses. The bridge builders of the period were ingenious in designing the truss and arch systems that make up these bridges to carry and transfer loads safely. Projects are being supported to study several of these designs. The original design of William Howe who patented the Howe Truss permitted tightening of the tension rods to induce compression in the wood diagonals. *Howe truss bridge design and performance* study will provide a better understanding of the design of William Howe by studying the Moose Brook Howe Boxed Pony Truss bridge through both analytical and full scale experimental tests. Similarly, *Burr truss arch systems* will be studied to have a better understanding of the design and connection details of Theodore Burr's designs. The recovered Bartonsville covered bridge damaged by Tropical Storm Irene will be used to study the performance of *Town lattice trusses* and the interaction of the lattice system to carry and distribute loads in order to develop more precise calibrated models.

Historic covered bridges need to be protected from vandals as many of these bridges have been lost due to arson. *Best practices guide on security of historic covered bridges* will provide cost effective, tested systems that can be used to protect against vandalism.

Condition assessment of all bridges is essential to determine bridge safety. In accordance with the National Bridge Inspection Program all bridges greater than 6.1m (20 ft.), on public roads are required by law to be inspected every two years. While visual inspection to determine condition is by far most prevalent for all bridge types, more advanced technologies for bridge inspection continues to be developed, but mostly for steel and concrete structures. *Advanced field evaluation tools for condition assessment of wood members in covered bridges* is a new study that will be underway to identify, adapt and evaluate low cost innovative technologies that can be used by bridge inspectors to inspect wood members and more accurately determine the condition of covered bridges.

In addition to damages to covered bridges that are caused by age, environment, arson, and increasingly heavier loads, natural hazards such as floods have also played a role in destroying these historic structures. Hurricane Irene as it moved up the northeastern US in 2011 caused major rainfall, and subsequent flooding damaged many roadways and bridges in Pennsylvania, New York, Massachusetts, Vermont and New Hampshire. Several covered bridges were washed away and destroyed during this event. *Improved hydraulic safety of covered bridges* will evaluate hydraulic forces on covered bridges and develop retrofit strategies to save these structures from future hydraulic events.

To further knowledge transfer, and educated engineering and architecture students about structural behavior and the evolution of wooden truss designs *educational models for four significant truss designs: Burr, Town, Long & Howe* will be developed. Designs for scale models of 100ft trusses will be developed to allow users to construct them and demonstrate the aspects of each design.

4. Summary

The National Historic Covered Bridge Preservation Program, from the years 2000 to 2012 funded more than 200 covered bridges. These funds went into preservation, restoration and rehabilitation of the Nation's covered bridges that are listed or are eligible for listing on the National Register of

Historic Places. The NHCBP Program during this period funded over 50 research, educational, and technology transfer projects many of which are still ongoing. The research, education and technology transfer has been conducted as a partnership between the FHWA, NPS and FS, and has worked well. As studies are completed, information resulting from the program will be made available at the National Center for Wood Transportation Structures site at www.woodcenter.org.

Covered bridges proliferated in the United States in the mid-nineteenth century. Today 500 to 600 remain, and represent the technological heritage of the United States. Restoration of historic structures requires that the projects be carried out in the most historically appropriate manner preserving the historic fabric of the structure. This is often a challenge as the covered bridges of the past were not designed to carry today's loadings. Developing techniques and methodologies utilizing the right blend of the new and traditional, and resulting in preserving historic integrity including the original architectural, structural and material characteristics of the bridge has been the thrust of the program.

5. References

[1] Duwadi, S.R.; Wacker, J.P. (2008) *Covered Bridges in the United States and the Preservation Program*. In: Conference proceedings of the 10th World Conference on Timber Engineering, June 2-5, 2008, Miyazaki, Japan.

[2] Duwadi, S.R. (Publication in Process). *Saving the Past*, In: Public Roads, Department of Transportation; Federal Highway Administration.

Stress-Laminated Timber Bridge Decks: Non-linear Effects in Ultimate and Serviceability Limit States

Mats Ekevad
Luleå University of Technology
Division of Wood Science and Technology
SE-931 87 Skellefteå
Sweden
mats.ekevad@ltu.se

Ph.D. Associate Professor. Research focus is on finite element simulations of wood material and engineered wood products behaviour during loading.

Peter Jacobsson
Martinsons Träbroar AB
Kroksjön 230
SE- 93195 Skellefteå
Sweden
peter.jacobsson@martinsons.se

M.Sc. Development Manager. Focus is on increasing competiveness for timber bridges.

Robert Kliger
Chalmers University of Technology
Division of Structural Engineering
SE-41298 Gothenburg
Sweden
Robert.kliger@chalmers.se

Ph.D. Chair Professor. Research focus on structural behaviour of built-up wood-based panels with semi-rigid connections, wood properties, timber products, wood-based components and timber bridges.

Summary 10 lines max

Stress-laminated timber bridge decks made of glulam beams are advantageous when it comes to their strength, production and construction costs. The thickness of a deck with a specified span and width is typically determined by considering requirements relating to strength and stiffness stipulated in a design code and specifically in Europe the Eurocode 5. Linear stress calculations do not have inherent potential to simulate slip or gaps between beams, but experience from finite element (FE) simulations and full-scale tests shows that these nonlinear effects from slip and gaps between glulam beams exist. In this paper, comparisons between linear and non-linear FE results for two bridge decks with different spans, widths and thicknesses are made. Separate comparisons are made in the ultimate limit state (ULS) and the serviceability limit state (SLS). It is shown that non-linear effects may be important and should be considered, especially in the case of thin decks.

Keywords: prestressed, stress laminated, timber bridge deck, finite element, FEM

1. Introduction

Stress laminated timber bridge decks in Sweden are now days designed according to the rules in Eurocode 5 [1] in the serviceability limit state (SLS) and the ultimate limit state (ULS). Limits for strength of a bridge deck are determined by specified design values for load and wood material strength in ULS. Limits for deformations in SLS may vary depending on the specific application in question. Design values for loads and material properties to be applied are different for SLS and ULS and normally taken as mean values for SLS and 5-percent percentile values for ULS, i.e. in general higher design loads for ULS.

Eurocode does not explicitly specify calculation methods which mean that linear manual calculation

methods or linear FE (finite element) methods may be used. Traditionally early developed US effective width beam methods with correction factors for plate effects have been used, see [2].

Recent full-scale tests and FE-simulations with nonlinear models of frictional slip included have shown that slip between beams in stress laminated bridge decks start to appear at relatively low load values, [3], [4], [5]. At low loads frictional slip is mainly horizontal due to twisting moments and appears in the corners of the bridge deck but later at higher load levels also vertical slip appear, normally close to the load application points. The slip does not lead to failure or instability but is visible as a deviation from linearity of the load vs. displacement curve.

A number of calculations with different calculation methods have been conducted recently in a project for 5 different bridge deck geometries and for 3 different load cases. The methods are a linear FE shell method, a nonlinear 3D (three-dimensional) FE contact method and a nonlinear 3D FE elastic-plastic method. In this paper results for the elastic-plastic FE method is shown for two of these bridge decks. The purpose of this paper is to show differences between linear and nonlinear FE results and to show for which bridge deck geometries and for which of ULS and SLS these differences are large and thus needs to be taken into consideration. Also shown are the consequences of nonlinearity such as the size of frictional slip.

2. Materials

Two different single span bridge deck geometries were chosen for this paper, one short and one long. Dimensions were 5x4 m with thickness 315 mm and 25x12 m with thickness 1125 mm, see Table 1. The deck thicknesses were beforehand chosen fulfil approximately acceptable load-bearing capacity in ULS and this was then checked afterwards.

Table 1. Bridge deck geometries.

Span *l* (m)	Width *b* (m)	Thickness *t* (mm)	*b/l*	*t/l*	*t/b*	
5	4	315	0.80	0.16	0.079	short
25	12	1125	0.48	0.045	0.094	long

The decks were built up from Norway spruce (*Picea abies*) glulam beams having a width of 90 mm and being produced from lengthwise finger jointed boards with thicknesses 45 mm. Prestress value was set to 0.35 MPa which was the lowest acceptable prestress value.

3. Methods

Finite element (FE) modelling of the deck geometries were made with the commercial FE code ABAQUS, [6] and a specially written user subroutine for treating orthotropic plasticity [7]. The method to handle frictional slip was described in [3]. The deck is treated as a continuum with no borders between beams and frictional slip is modeled applying an elastic plastic material effect. Frictional slip appear in horizontal and vertical directions with friction coefficients 0.29 and 0.34, respectively. Gaps that may appear due to transverse bending were modeled as local reductions of all elastic constants to near zero when local transverse stresses exceeded zero. Density of the glulam beams was 400 kg/m^3 and moduli of elasticity 12000 and 240 MPa in the fiber and cross directions, respectively. Shear moduli were 720 MPa for bending shear in two shear planes and 72 MPa for rolling shear modulus. Poissons ratios were all set to zero, all of this according to Eurocode standard. Design values for strength were taken as 22.18, 1.94, 2.52, 0.58 MPa for bending stress, transverse compressive stress, bending shear stress and rolling shear stress, respectively. Supports were modeled as very stiff and prestress 0.35 MPa was applied as a constant pressure on the side surfaces of the bridge decks. Half models were used due to symmetry conditions see Fig.1.

The only load case that was studied here was a selected combination of a number of loads. Gravity load of wood, pavement load 2.5 kN/m^2, a distributed load of 9 kN/m^2 on the whole top surface of the deck and finally 4 wheel loads each of 225 kN on 0.4x0.4m surfaces on the top surface of the deck, i.e. a double axel load. The wheel loads center was positioned in the middle section of the span but laterally close to the edge see, Fig.1. These levels of loads combined together defined the ULS design load in this paper and its relative load value was designated 1.0. For reasons of

simplicity and to be able to compare between SLS and ULS and also in order to find out what happens for even greater loads, the combined loads were increased proportionally from zero to the relative value 2.3. This was made in order to find out where nonlinear effects and/or instabilities starts to occur. This is not the exact way load cases are to be treated in Eurocode 5 since design load combinations differ between ULS and SLS and also the relative proportion of each load varies between ULS and SLS. Also, this is only one of many load cases that have to be treated when performing a full examination according to Eurocode. However, as a rough estimate used here to compare linear and nonlinear results, relative load levels 0.436 and 1.0 were found to approximately represent the SLS load case and the ULS load case in Eurocode 5 for the 5x4m deck, respectively. In the same way relative load levels 0.222 and 1.0 were found to approximately represent the SLS load case and the ULS load case in Eurocode 5 for the 25x12 m deck, respectively. Limits for stresses in ULS according to Eurocode were checked and points where limits were reached were marked on the load vs. displacement curves. Vertical slip values were calculated as plastic vertical shear strain times the beam width 90 mm in order to get a specific value for vertical slip between beams. Horizontal slip values were generally much lower.

4. Results

4.1 Bridge deck 5x4 m

Figs.2a and 2b show linear and nonlinear load-displacement curves for bridge deck 5x4 m for loads up to 2.3 times the ULS-load. The maximum deformation in the nonlinear solution was about 22% and 12% higher than the linear solution at the ULS load and SLS load, respectively. The displacement was measured at a point on the very edge of the bridge deck in the midsection. This point was the point of maximum displacement for relative load levels below 1.19 but for higher load levels a point in the midsection between the wheels close to the side achieved larger displacement due to slip between beams, see Fig.3. No instabilities occurred for relative load levels up to 2.3 but maximum slip between two beams increased up to 20 mm. ULS limits for bending stresses in fiber direction were reached at relative load levels 0.97 and 0.84 for the linear and nonlinear case, respectively. A limit of span/400=12.5 mm was used as the SLS limit for deformation. Horizontal slip started early at about 0.1 relative load and over large areas close to the corners of the deck at the top and bottom of the deck. At load level 2.3 the horizontal slip was at the most 1.70 mm. Load vs. displacement curve started to deviate from linearity at load level 0.1. Vertical slip started at about load level 0.4 close to the wheel load application areas and in the middle of the height of the deck and it reached values shown in Fig.2. The maximum gap between two beams is shown in Fig.2a.

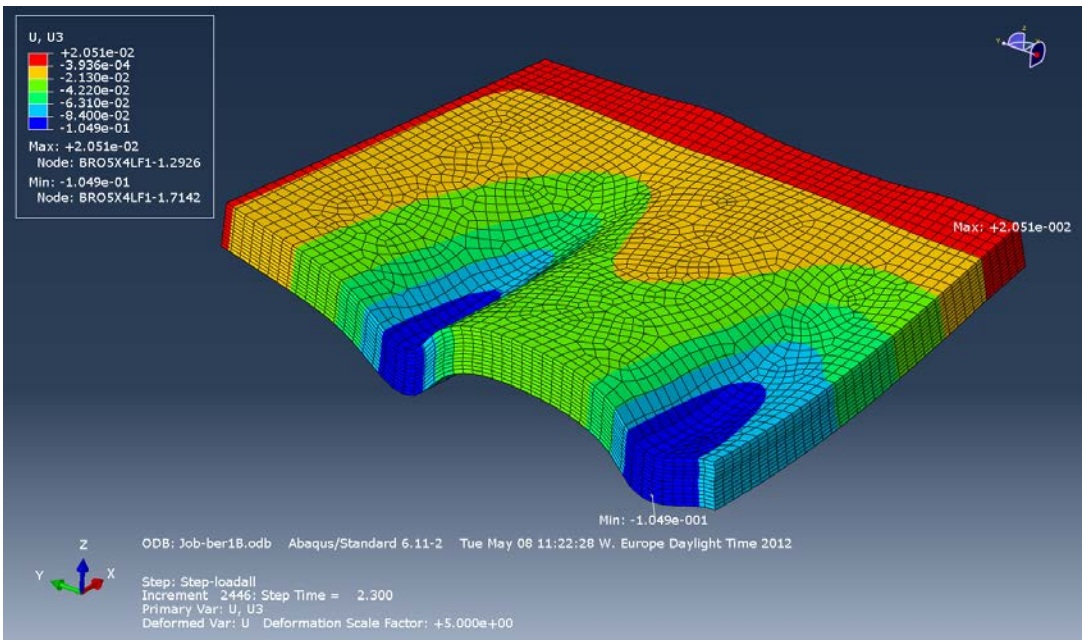
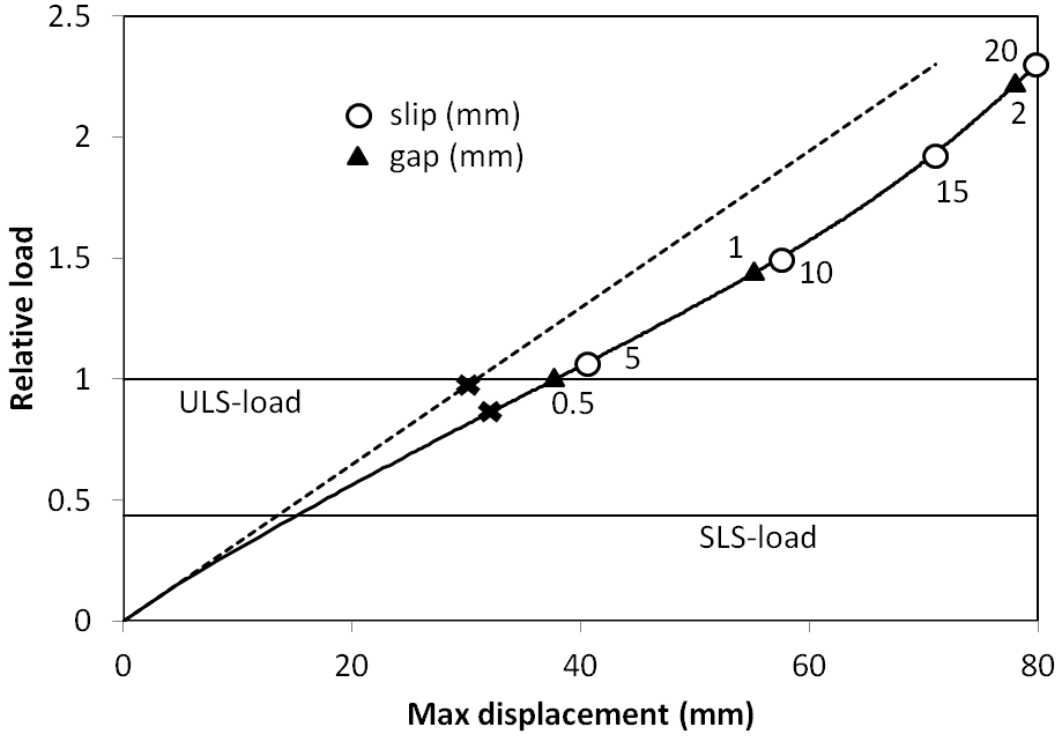


Fig.1. FE half-model of bridge deck 5x4 m. Vertical displacements (m).



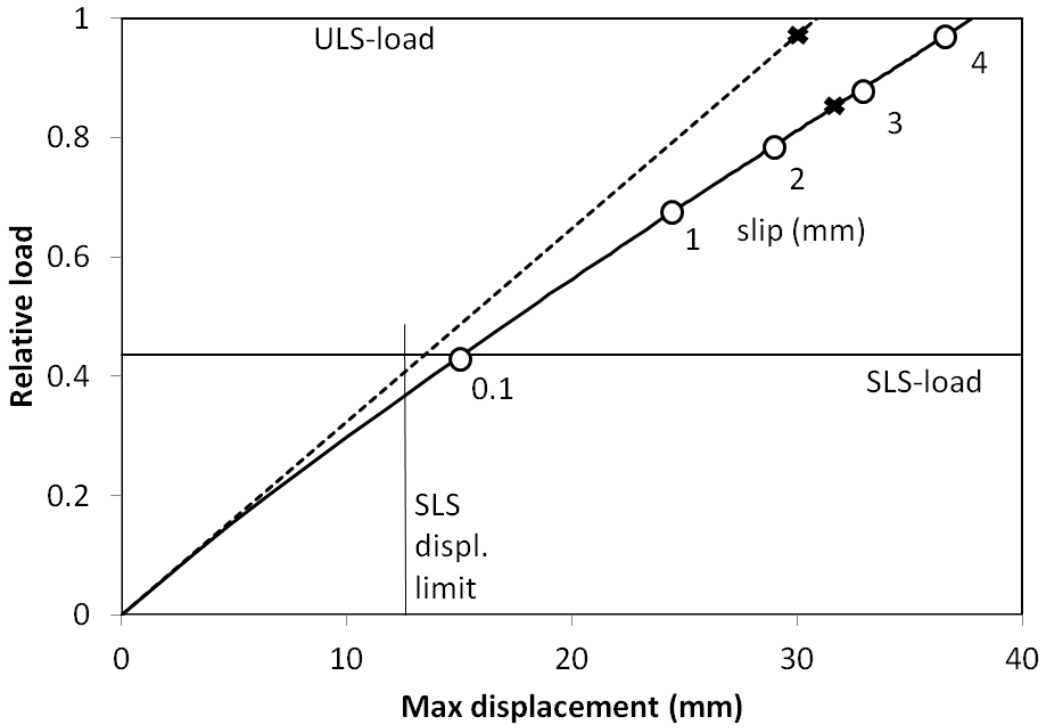


Fig.2a and b. Load vs. displacement curve for bridge deck 5x4 m (span x width), thickness 315 mm. Slip is maximum vertical slip between two beams and gap is maximum local gap between two beams. Crosses denotes where maximum ULS-design stresses are reached locally. Dotted lines are results from the linear model and solid lines are results from the nonlinear model.

Upper a) load level 0 to 2.3

Lower b) load level 0 to 1

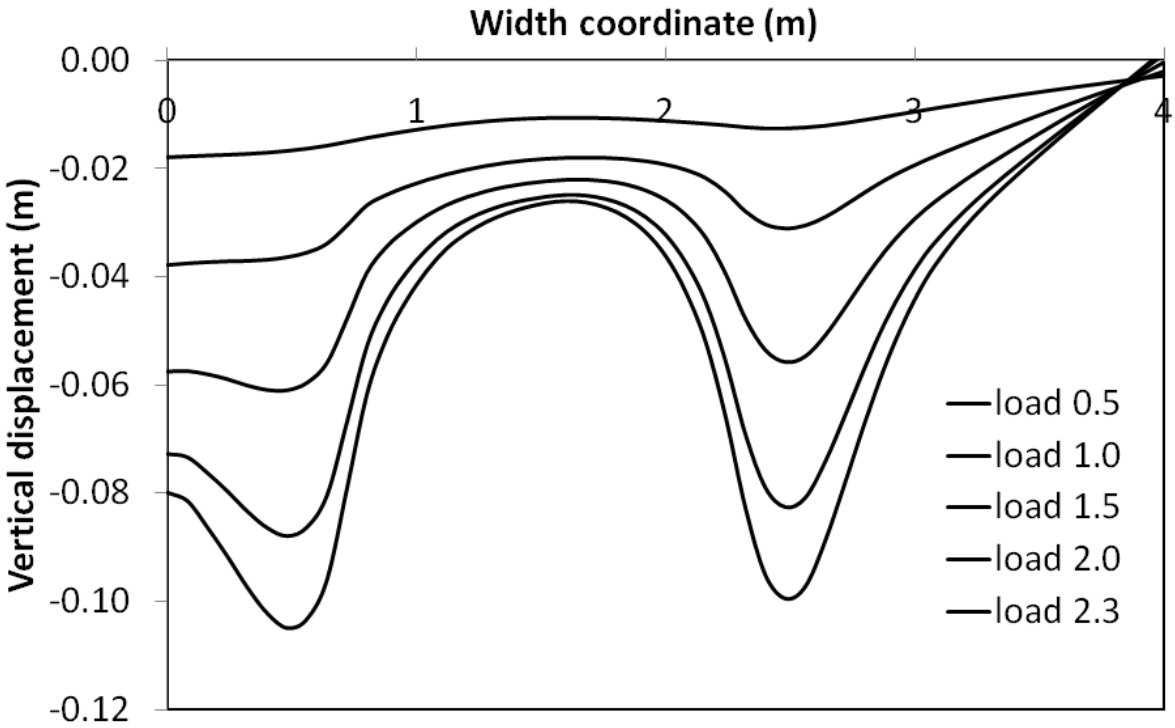
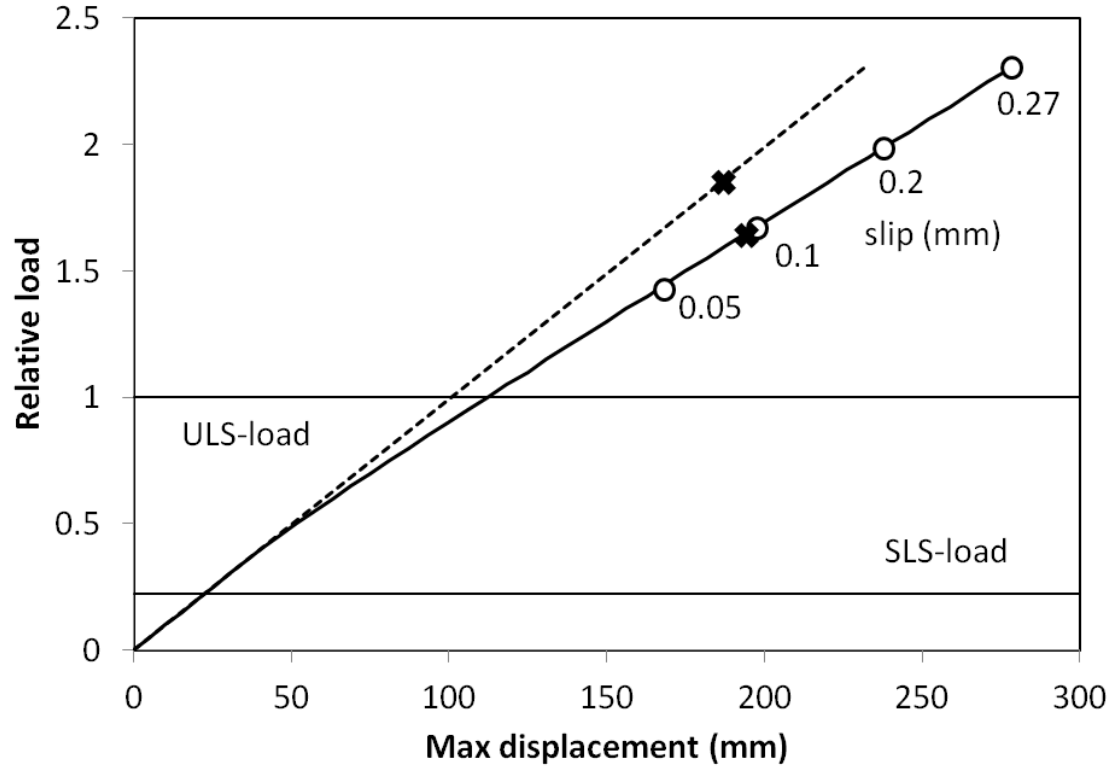


Fig.3. Bridge deck 5x4 m. Midsection displacement for different relative load levels from the nonlinear model.

4.2 Bridge deck 25x12 m

Figs.4a and 4b show linear and nonlinear load-displacement curves for bridge deck 25x12 m for loads up to 2.3 times the ULS-load. The maximum deformation in the nonlinear solution was about 11% higher than the linear solution at the ULS load and did not differ at all at the SLS load. The displacement was measured at a point on the very edge of the bridge deck in the midsection and this did not change for loads up to 2.3. No instabilities occurred for relative load levels up to 2.3 and maximum vertical slip was at the most 0.27 mm at relative load 2.3. ULS limits for bending stresses in fiber direction were reached at relative load levels 1.89 and 1.67 for the linear and nonlinear case, respectively. A limit of span/400=62.5 mm was used as the SLS limit for deformation. At load level 2.3 the horizontal slip was at the most 0.21 mm. Load vs. displacement curve started to deviate from linearity at load level 0.3. A gap started to form at load 2.2 and became at the most 0.01 mm in the midsection in the middle of the width on the upper deck surface.



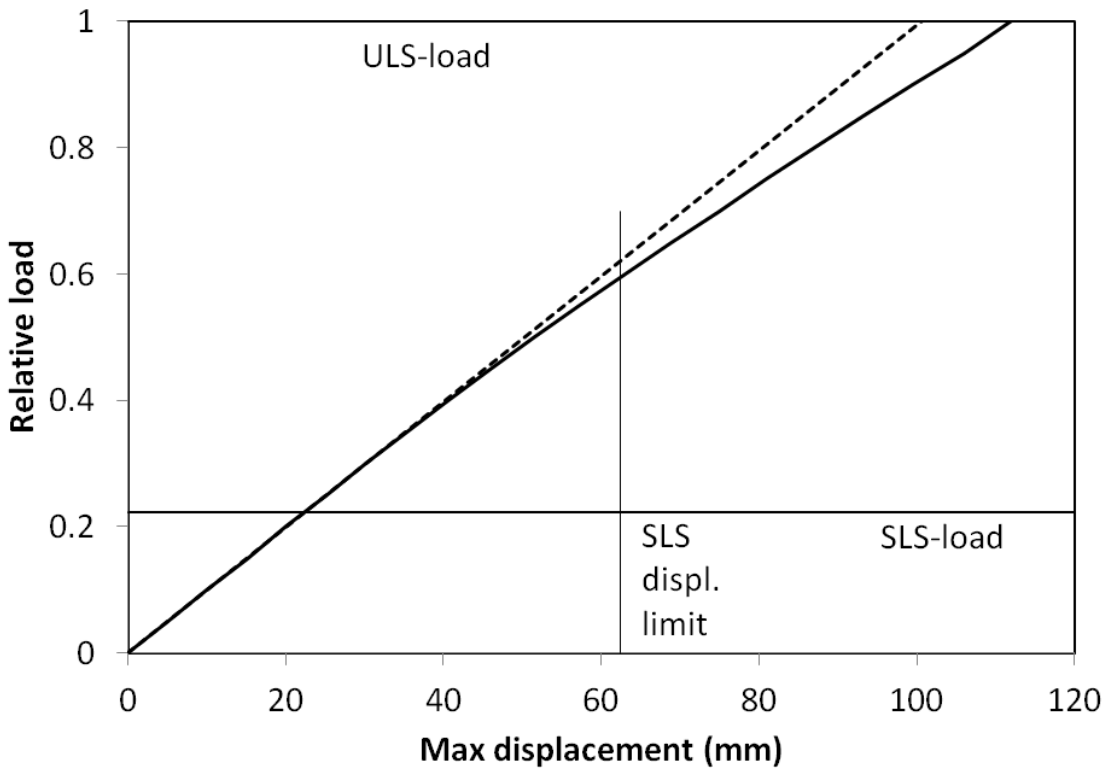


Fig.4a and b. Load vs. displacement curve for bridge deck 25x12 m (span x width), thickness 1125 mm. Slip is maximum vertical slip between two beams and gap is maximum local gap between two beams. Crosses denotes where maximum ULS-design stresses are reached locally. Dotted lines are results from the linear model and solid lines are results from the nonlinear model.

Upper a) load level 0 to 2.3

Lower b) load level 0 to 1

5. Analysis and discussion

The thickness 315 mm that was beforehand chosen for the 5x4 m deck was well suited and almost exactly fulfills the requirements according to ULS limits in the linear calculation (load 0.97, Fig.2a). However the nonlinear analysis showed that the linear calculation was non-conservative (ULS nonlinear limit load 0.84, see Fig.2a). Also shown from the nonlinear calculation results was that at the ULS limit the vertical slip was about 4 to 5 mm (Fig.2a and 2b) which may lead to failure of the deck surface sealing layer and/or paving on the deck. The maximum deformation in the nonlinear solution was higher than the linear solution due to slip. Vertical slip at SLS was much less than 1 mm which perhaps could be acceptable from viewpoint of integrity of sealing layer and paving. Gaps started to evolve just before reaching the relative load 1.0 and became 0.5 mm at load 1.0 but then increased up to 2 mm at load 2.3. Nonlinear effects in the midsection displacement curves make the point of maximum deformation change from a point on the edge to a point under a wheel load due to local vertical slip between beams at high load (see Fig.3).

The thickness 1125 mm for the 25x12 m deck resulted in lower than maximum allowable design stresses in ULS (Fig.4a), thus this deck thickness could have been reduced. Also for this deck the linear calculation in ULS gave allowable loads that were higher than the allowable loads from the nonlinear calculation (load 1.89 cf.1.67 allowable load in ULS). SLS load (0.222) was lower for this deck than for the 5x4 m deck due to the larger relative contribution from gravity load of wood compared to wheel load for this deck. SLS limits for deformation were easily disposed of with a large margin (Fig. 4b). There was no vertical slip at load 1.0 and only 0.27 mm vertical slip at load 2.3. Gaps were negligible and did only appear above relative load 2.2.

For the studied deck geometries nonlinear effects were only significant for the 5x4 m deck since the 25x12 m deck did cope with all restrictions (limits) both in the linear and nonlinear calculation. The nonlinear calculation of the 5x4 m deck showed that two different potential problems arouse. The first was that the ULS design stresses were too high (only 0.84 load was allowed) and the second was that the vertical slip in ULS was 4 to 5 mm. The first problem could be a non-relevant problem if it is that the Eurocode design stress limits are set in a way to cope with the difference between linear and nonlinear stress results. Regarding the other potential problem related to vertical slip then it is not yet known whether this is a real problem or not or if this can be solved by the bridge producers.

6. Acknowledgement

Most of the financing of this work was from VINNOVA-Swedish Governmental Agency for Innovation Systems and industry in the project “Competitive Bridges”.

7. References

[1] Anonymous, Eurocode 5: Design of timber structures-Part 2: Bridges. SS-EN 1995-2:2004.

[2] Ritter, M. A. (2005). *Timber bridges: design, construction, inspection and maintenance (Part one)*. University Press of the Pacific, ISBN1410221911.

[3] Ekevad M.; Jacobsson P.; Forsberg G.. ”Slip between glulam beams in stress-laminated timber bridges: finite element model and full-scale destructive test”. *ASCE Journal of Bridge Engineering* 16:188-196. 2011.

[4] Ekholm K.; Kliger R.; Crocetti R.; “Full-scale ultimate load test of a stress-laminated timber bridge deck”. *ASCE Journal of Bridge Engineering* 17:691-699. 2012.

[5] Ekholm K.; Crocetti R.; Kliger R. “Stress-Laminated Timber Decks Subjected to Eccentric Loads in the Ultimate Limit State”. *ASCE Journal of Bridge Engineering* 18:409-416. 2013.

[6] Anonymous, Abaqus 6.11-2 Users manual. Dassault Systemes Simulia Corp. Rising Sun Mills, 166 Valley Street, Providence, RI 02909-2499, USA. 2011.

[7] Ekevad, M. (2006). ”*Modelling of dynamic and quasi-static events with special focus on wood-drying distortions.*” Doctoral dissertation 2006:39. Luleå University of Technology, Division of Wood Science and Technology, Skellefteå, Sweden. ISSN:1402-1544.

Case Study of the Longest Single Span Timber Bridge for Highway Loads in Sweden

Kristoffer EKHOLM
Bridge designer, PhD-student
WSP Bridge & hydraulic design
Gothenburg, Sweden
kristoffer.ekholm@wspgroup.se

Kristoffer Ekholm is a bridge designer employed by WSP. For the past five years, he has conducted research of stress-laminated-timber decks. Kristoffer will defend his PhD thesis called "Performance of stress-laminated-timber bridge decks" in the end of October 2013.

Peter NILSSON
Bridge designer
WSP Bridge & hydraulic design
Gothenburg, Sweden
peter.c.nilsson@wspgroup.se

Peter Nilsson works as a bridge designer with a focus on analysis of complex structures. Peter has conducted several analyses of dynamic wind loading on bridges for WSP.

Erik JOHANSSON
Bridge designer
Moelven Töreboda
Töreboda, Sweden
erik.johansson@moelven.se

Erik Johansson works as a structural designer with a focus on wood constructions. Erik has several years of experience in timber bridge design at Moelven Töreboda

Summary

A 47.4 m long glulam arch bridge was built over the Nissan River when Highway 27 was upgraded in Sweden. The bridge consisted of two individual arches spaced 13.9 m apart which were unconnected at the top due to architectural requirements. The lateral stability of the bridge was obtained through stiff vertical hangers which were bolted to a stiff transverse girder underneath the road level. Extensive analyses were made to ensure the stability and capacity of the arches against accidental, wind and vehicle loads. The deck was constructed as a continuous stress-laminated-timber deck using glulam. A total of approximately 230 000 kg of untreated Swedish glulam was used in combination with approximately 100 000 kg of steel. The bridge could, to a large extent, be pre-assembled in the factory before being transported to the construction site. The entire assembly of the bridge took approximately two months.

Keywords: Glulam, arch bridge, stress-laminated-timber deck, FE analyses, wind analyses, buckling

1. Introduction

A new stretch of Highway 27 has been constructed to primarily reduce the amount of traffic through the town of Gislaved,Sweden, but also to increase the safety of the highway. The largest bridge on this particular new stretch of highway is the one crossing of Nissan River. The new highway was constructed with a width of either 9 or 14 m. The 9 m highway width had two lanes (one in each direction) while the 14 m wide one had three lanes, which is a common solution in Sweden including road shifts between two lanes in one direction and one in the opposite direction.

The Swedish Transport Administration (Trafikverket) decided early on that they would like a timber-arch bridge crossing the river. The required clear span of the bridge was 45 m. There were several architectural requirements related to the appearance of the arches that had to be fulfilled. Firstly, a rather small radius was required for the arches and they also needed to resemble an old steel-arch bridge in the area. A second requirement was that the two arches were not allowed to be connected above the road level. In essence, the architects aspired for the drivers crossing the bridge to have an unobstructed view unlike that provided by a tunnel.. The combination of a small radius for the arches and the lack of cross bracing in the top posted a challenge for the design of bridge. A stress-laminated-timber (SLT) deck made out of glulam beams was used for the bridge deck. Drawings of the bridge are shown in Figure 1.

The contract for the construction of the highway stretch went to Svevia (a Swedish contactor) which ordered the timber bridge from the manufacturer Moelven Töreboda (Moelven). Moelven contracted the consulting company WSP Civils Sweden (WSP) for the analysis and design of the bridge. The bridge was assembled during May and June of 2013.

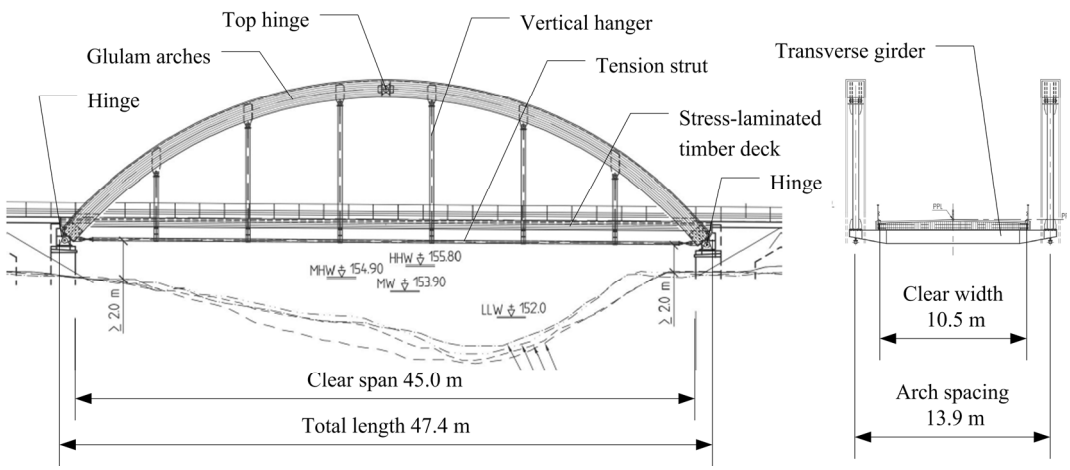


Fig. 1 Elevation and section drawing of the bridge.

2. Structural analysis of the bridge

The structural analysis was performed using the finite element (FE) software Brigade/Plus 3.1-5 which is based on the solver from Abaqus. The structural analysis made by WPS was limited to the superstructure. The obtained reaction forces from the analyses were used for the design of the substructure.

The density of the SLT deck is larger than for the glulam arches in order to include the weight of the prestressing system and bridge railing. The stiffness values for glulam of grade CE L40c (Sweden glulam) was taken from the Swedish glulam manufacturers [1]. The orthotropic plate material was calculated using the defined ratios in EN 1995-2 [2].

Table. 1 Material property values used for the wood parts in the analysis

Material property	Glulam arch	SLT deck
E_1 [MPa]	13 000 [1]	13 000 [1]
E_2 [MPa]	410 [1]	0.02 E_1 = 260 [2]
E_3 [MPa]	410 [1]	0.02 E_1 = 260 [2]
G_{12} [MPa]	760 [1]	0.06 E_1 = 780 [2]
G_{13} [MPa]	760 [1]	0.06 E_1 = 780 [2]
G_{23} [MPa]	760 [1]	0.1 G_{12} = 78 [2]
ρ [kN/m^3]	4.2 [1]	4.5 [1)]

1) Combined weight of glulam, prestressing and bridge rail

2.1 Loads

The bridge was loaded by the self-weight of all steel and glulam components as described above in Table 1. Another dead load included was the surfacing of the deck, which was composed of a 110 mm thick asphalt with a density of 23 kN/m^3 [3].

Vehicle loads from Eurocode EN 1991-2 [4] were applied on the bridge using load combinations from EN 1990 [5]. In addition to the vehicle loads in Eurocode, fourteen classification vehicles should be used according to Trafikverket. These classification vehicles are often governing the design of bridges in Sweden. As well, both vertical and horizontal effects from the vehicle loads needed to be considered in design.

The geographical position of the bridge together with the geometry of the arches and hangers resulted in the need for extensive wind analyses. The wind loads on the bridge should conform with the requirements in TK Bro [3] and EN 1991-1-5 [6]. A quasi-static wind load which could be applied in the static analysis was calculated from the dynamic wind analyses. Special considerations had to be made to the stiffness of the vertical hangers in order to avoid severe fatigue problems from vortex shedding.

Temperature loads were applied to the bridge in the form of a uniform temperature load of +25ºC and a temperature difference of -43 ºC according to EN 1991-1-5 [7]. Loads from temperature gradients were also applied on the bridge.

An accidental load was applied with a force magnitude of 1000 kN in the direction of the bridge (direction 1) and 500 kN perpendicular to the direction of the bridge (direction 2). The load should be applied on the arches at a distance of 1.25 m above the top of the surfacing. The magnitude and the position of the load was taken from EN 1991-2 [4].

2.2 Arch

The two arches (spaced 13.9 m apart) had a total length of 46.6 m between the points of rotation in the bearings. The radius of the arches centerline was 29.9 m. The arches were modeled with hinges at the top and over the support. The tension strut ties the two halves of the arch together, cf. Figure 2.

The arches were modeled using beam elements with anisotropic material properties, cf. Table 1.

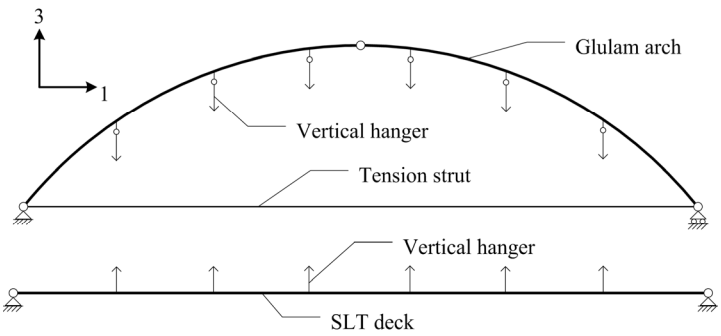


Fig. 2 Principe sketch of the structural behavior of the arches.

Direction 1 was designated for the arch defined as the tangent to the centerline of the arch. A simplified buckling analysis was initially made on the arches. The critical buckling load and mode shape showed that the hangers and especially the stiffness of the SLT deck needed to be included in the analysis. A second buckling analysis was made on the entire structure. The critical load for the structure was obtained through an analysis where a load was applied in one of the hangers. The critical load analysis was repeated for all of the hangers. Eight different combinations of loading the hangers were evaluated in order to find the critical buckling load for the entire structure.

2.3 Hangers and transverse girder

The vertical hangers and transverse girders were modeled using beam elements with isotropic material properties. The stiffness of the steel in the model was 210 000 MPa and the density was 78.5 kN/m^3 , taken from EN 1991-1-1 [8]. In the design, the hangers and the transverse girder are supposed to stabilize the arch for horizontal loads. The connection between these components was modeled as a stiff frame corner, cf. Figure 3. The connection between the hanger and the arch was modeled as pinned around axis 2, cf. Figure 2. The girders needed to be very stiff due to the stiff connection between the girder and the hanger. A small deformation of a weak girder would result in a large transversal displacement on the arch. Hence, the girders had to be very stiff, cf. Figure 3.

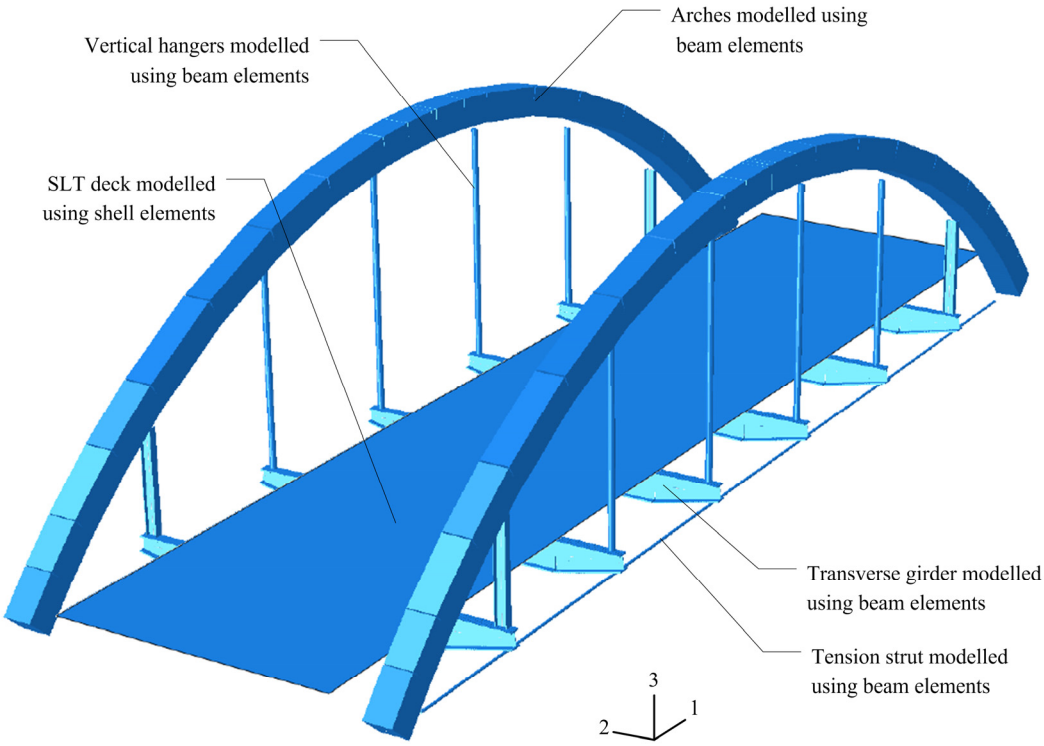


Fig. 3 Numerical mode from Bridge-Plus. The parts of the bridge were modeled using beam or shell elements. All beam elements in the figure are rendered to show their correct profile. The coordinate system used is also shown.

2.4 Deck

The SLT deck was modeled as a continuous slab with six intermediate supports (the transverse girders) and two end supports, cf. Figure 2. The total length of the deck was 47.4 m and the width was 10.5 m, cf. Figure 1 and Figure 3. The thickness of the deck varied between 450 mm and 580 mm with the deepest section in the middle and 450 mm along the longitudinal edges. The SLT deck was modeled using rectangular shell elements with orthotropic material properties, cf. Table 1. Connections between the deck and the girder were modeled with restrained movement in directions 1 and 3 in order to measure the uplifting force. The middle point of the girder restrained the movement of the deck in direction 2, thus simulating the slotted-in plate restraining translation the deck in the transverse direction.

2.5 Other components

The tension strut was modeled using beam elements and the same material as for the hangers and the transverse beam. A small weightless and infinite stiff element was used to couple the arch and the tension strut at the correct position, cf. Figure 3.

3. Design of components

The design of the arches, hangers, girders and the deck was an iterative process in combination with the structural analyses. All of the major components in the bridge had to be analyzed and designed simultaneously in order to achieve better material utilization as well as to avoid problems with wind induced fatigue.

All detailing of connections and other components could be made after the main structure was designed. Since the glulam is untreated, the design of the weather protection for the glulam components is crucial. The design of the bridge was made using the 3d-software Tekla Structures. Both 3d and 2d drawings were generated from the 3d model.

3.1 Arch

Each of the glulam arches were constructed using five parallel arched glulam beams with a cross section of 1170 x 215 mm (depth, width). The glulam beams were glued and screwed together forming a 1075 mm wide block. The cross-section of the arch was deigned to fulfill the criteria for compression, bending, combined compression and bending, shear, torsion and tension perpendicular to the grain. The shear capacity of the arch governed the depth of the cross-section, which was due to the crack factor k_{cr} = 0.67 for glulam in EN 1995-1-1 [9]. The magnitude of the crack factor has been widely debated in Sweden for a couple of years now, however, recent studies by SP Technical Research Institute of Sweden have shown that a smaller reduction of the capacity should be used [10].

A steel detail was created to connect the glulam arch to the bearing using doweled slotted-in steel plates, cf. Figure 4. The arch end detail also allowed for the tension strut to connect the two arch halves. The outer measurements of the steel detail are approximately 1.3 x 1.2 x 1.2 m (length, width and height).

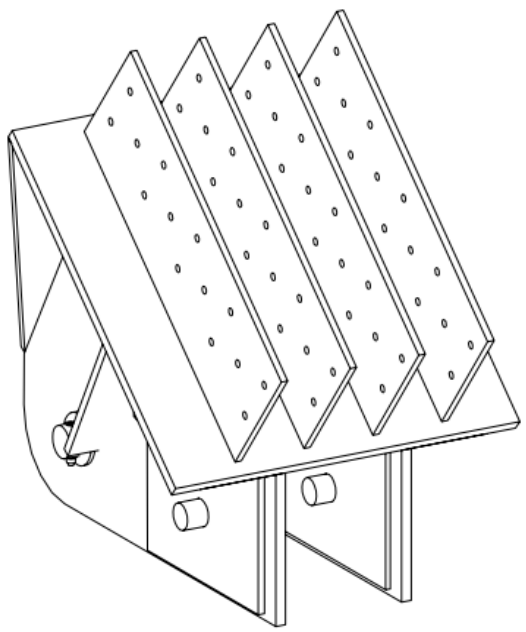


Fig. 4 Detail of the connection between the glulam arch and the bearing. The connection for the tension strut is also seen in the figure.

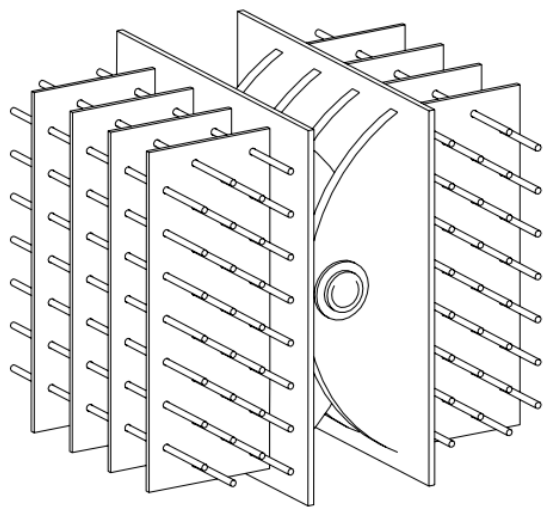


Fig. 5 Detail showing the hinge used for connection the two halves of the arch together. The steel dowels are shown in the figure.

Another steel detail related to the arch included the pinned connection between the two halves of the arch. Again, doweled slotted-in plates were used in this detail, cf. Figure 5.

The tension perpendicular to grain was larger than the capacity for glulam CE GL40c, as suchthe glulam beams were reinforced using fully-threaded screws with the dimensions 13 x 800 mm (diameter, length).

3.2 Deck

The deck was designed as a continuous deck using staggered butt joints. One in five adjacent beams was butt joined. The length of the glulam beams used for the deck varied between 12.3 m and 19.5 m. The width of the deck beams was 140 mm and the depth varied between 450 mm and 580 mm. The deck was designed to fulfill the criteria in ULS for bending, shear, interlaminar shear, compression perpendicular to grain, and compression parallel to grain for the break plates. The bending stresses in the deck governed the design of the deck depth. The smallest depth was 450 mm at the longitudinal edges of the deck. A fatigue verification in accordance to Appendix A in EN 1995-2 [2] had to be executed.

The deflection of the deck itself as well as the entire bridge had to be checked in SLS. The deflection of the deck in between the transverse girders was small. However, the global deflection of the deck was 50.5 mm, which corresponds to a deflection of $L/940$, where L is the span between the two end supports. The deflection criterion for bridges in Sweden which needs to be fulfilled is a deflection less than $L/400$.

Two rows of prestressing bars were used with a longitudinal spacing of 600 mm. Dywidag bars with a 20 mm diameter were used. The longitudinal edge of the deck including the glulam arch or a steel hanger had to be stressed using coupled 1 m long bar sections. The coupling of shorter bar segments was used in order to maintain the possible interchangeability of any part of the deck.

The deck was bolted to the transverse girder to prevent separation between the deck and the girder. Four M24 bolts were equally distributed over the width of the deck.

3.3 Hangers and transverse girders

The hangers were made out of HEB450 steel profiles. Eight out of twelve hangers had additional stiffeners welded onto the flanges to increase the stiffness.

The transverse girders were made out a welded steel I-profile. The 9.5 m profile in the middle of the girder had a depth of 1200 mm while the outer 2.66 m profiles on each side of the girder had a varying depth of 1200 – 600 mm. The width of the girder flanges was 440 mm. As previously mentioned, the transverse girders had to be very stiff in order to avoid generating larger transverse displacements on the arches.

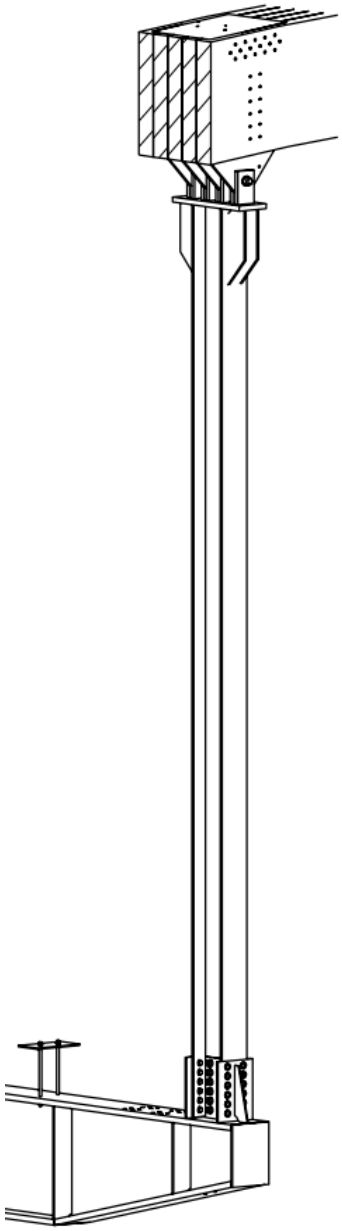


Fig. 6 Detail showing the hanger with connection to the arch and the transverse girder. The increasing depth of the girder is shown. The bolts preventing separation between the deck and the girder are also shown.

The connection between the hanger and the transverse girder was a bolted connection such that the hanger or girder could potentially be replaced. The bolted connection consisted of 36 M36 bolts, cf. Figure 6. The fatigue verification governed the design of the bolted connection.

The connection between the vertical hanger and the glulam arch was a pinned connection around axis 2, but could transfer moment around axis 1 and 3. The connection consisted of four doweled slotted-in plates that were welded to a steel plate, cf. Figure 6. The purpose of the added platewas to reduce the amount of tension perpendicular to grain in the arch. A M48 bolt was used for transferring the forces from the arch to the hanger, cf. Figure 6.

The amount of steel used for the hangers and the transverse girders would have been significantly reduced if cross-bracings were allowed between the arches. However, the aesthetics of the bridge would be significantly changed.

3.4 Other components

The tension strut consisted of several joined M100 steel bars that were 6.3 m, 7.6 m or 10.2 m long. It was more economical to use joined bars instead of a high strength steel cable.

The railing was connected to the deck with a capacity equal to a load that is twice the size of the load which would cause the rail posts to plasticise. The two M20 bolts that will primarily be loaded in tension had to be anchored within the deck. A 450 x 150 x 15 mm (width, height, thickness) steel plate was used to anchor the force at a distance equal to 4 x 140 mm = 960 mm from the edge of the deck.

4. Durability

All wood parts of the bridge were constructed using untreated glulam made out of Norway spruce. The wood had to be protected from moisture in order to ensure the required durability. The deck was covered with a 5 mm sealing board to prevent moisture from penetrating the deck. A 110 mm asphalt layer with 2.5% slope covered the top of the deck. Eight inlets were used on the bridge together with a rail gutter on the side of the deck, cf. Figure 7. The gutter prevents some of the melted water from entering the river Nissan underneath the bridge.

A piece of bent sheet metal that runs along the entire length of the deck on both sides transports the water away from the deck, cf. Figure 8. A ventilated side panel is used to prevent the edge beams of the deck and the prestressing anchorage from rain and sun. The panel consists of painted 120 x 21 mm z-panel made out of wood. The panel is fastened to the deck through wood block spacers.

Fig. 7 Stainless steel inlet (before sealing board and asphalt are added)

Fig. 8 Bent sheet metal to divert water (before sealing board and asphalt are added)

Fig. 9 Ventilated wood panel and steel sheeting to protect the glulam arches.

The glulam arches were also covered with a ventilated wood panel. The top of the arches was covered with large painted steel sheet metal, cf. Figure 9.

The prestressing bars and nuts were made out of galvanized steel.

5. Bridge assembly

The bridge was fabricated in Moelven's facilities in Töreboda, Sweden. About six weeks was required for the manufacturing of the arches and the SLT deck. All steel connections were inserted into the glulam arches at the plant, cf. Figure 10. Most cuts in the bridge (i.e. slotted-in plates) were made by skilled labor. In total, approximately 230 000 kg of glulam and 100 000 kg of steel were used for the bridge.

Fig. 10 Insertion of dowels in the arch

The bridge was transported 180 km south from Töreboda to Gislaved in a meticulous order. One crane on each side of the bridge lifted the two halves of the arch in place and the pins could then be inserted, cf. Figure 11. Steel cables were used to stabilize the arches until a temporary HEA300 steel beam was mounted, cf. Figure 12. Pontoons were temporarily used for easy access to both sides of the construction site

The vertical hangers and transverse girders were assembled next to the construction site. The six frames were lifted into position using cranes, cf. Figure 13. The deck could be assembled as soon as the arches were stabilized by the hangers and girders.

Fig. 11 First half of the arch lifted into position using a crane.

Fig. 12 Both arches in place with temporary support beam. The pontoons are visible to the left in the figure.

The deck beams were transported to the construction site as pre-assembled blocks containing five glulam beams. The deck was constructed from the middle towards the sides. When all glulam beams were in place, a prestressing force of 1000 kPa was applied to the deck. An overview of the deck in place is shown in Figure 14.

The guard rail posts were mounted when the deck was stressed. The moisture barriers could be assembled after the guard rail was mounted. Some of the moisture barrier details are shown in Figure 7 and 8. The entire deck was covered with a sealing board. The bridge will be surfaced at the same time as the highway, cf. Figure 15.

Fig. 13 The second frame is lifted into position.

Fig. 14 SLT deck in position.

Fig. 15 View of the completed bridge without the asphalt surfacing.

6. References

[1] Svenskt Trä. Hållfasthetsklasser för limträ (Glulam strength classes) - In Swedish. Föreningen Sveriges skogsindustrier; 2012.

[2] EN 1995-2. Eurocode 5: Design of Timber Structures - Part 2: Bridges. European Standard EN 1995-2:2004: Comité Européen de Normalisation CEN; 2004.

[3] TK Bro. TK Bro (Swedish design rules for bridges published by the Swedish Transport Administration) - In Swedish. TK Bro (2009:27). Vägverket, Borlänge, Sweden2009.

[4] EN 1991-2. Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges. European Standard EN 1991-2:2003: Comité Européen de Normalisation CEN; 2003.

[5] EN 1990. Eurocode - Basis of structural design. European Standard EN 1990:2002: Comité Européen de Normalisation CEN; 2002.

[6] EN 1991-1-4. Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions. European Standard EN 1991-1-4:2005: Comité Européen de Normalisation CEN; 2005.

[7] EN 1991-1-5. Eurocode 1: Actions on structures - Part 1-5: General actions - Thermal actions. European Standard EN 1991-1-5:2003: Comité Européen de Normalisation CEN; 2003.

[8] EN 1991-1-1. Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight and imposed loads. European Standard EN 1991-1-1:2002: Comité Européen de Normalisation CEN; 2002.

[9] EN 1995-1-1. Eurocode 5: Design of Timber Structures - Part 1-1: General - Common rules and rules for buildings. European Standard EN 1995-1-1:2004: Comité Européen de Normalisation CEN; 2004.

[10] Pousette A. Skjuvhållfasthet för limträbalkar - balkar utsatta för extrema fuktvariationer (Glulam shear strength - beams subjected to extreme variations of moisture) . (SP Rapport 2012:68) - In Swedish: SP Sveriges Tekniska Forskningsinstitut; 2012.

Footbridge over the Bow River in Banff

Gerald Epp
President, StructureCraft
Partner, Fast + Epp
Vancouver, BC, Canada
gepp@fastepp.com

Gerald Epp, born 1956, received his Bachelor's and Master's degree in Structural Engineering from the Univ. of British Columbia. He co-founded Fast + Epp in 1989, and established sister design-build firm StructureCraft Builders in 1998. Gerald sits on the committee for the CSA 086 Timber Design Code
.

Adam Gerber
Project Engineer
StructureCraft Builders
Delta, BC, Canada
agerber@structurecraft.com

Adam Gerber, born 1988 is a recent graduate from the Univ. of British Columbia department of Civil Engineering and currently working towards his M.Eng. He has worked with StructureCraft Builders for 8 years, starting as a carpenter before pursuing a career in Structural Engineering.

Lucas Epp
Project Engineer
StructureCraft Builders
Delta, BC, Canada
lepp@structurecraft.com

Lucas Epp, born 1985, received his civil engineering degree from the Univ. of British Columbia. He worked for Fast + Epp and StructureCraft Builders prior to joining a prominent engineering consultancy in Auckland.

Summary

Set over the Bow River in the mountain setting of Banff is a slender 113m long timber bridge both serving pedestrian traffic and providing a sensitive sanitary crossing replacement. Its minimal form was designed to blend into the natural beauty of Canada's first national park.

Completed in June 2013, the bridge was designed and constructed by StructureCraft Builders in collaboration with Fast + Epp Structural Engineers for the Town of Banff, under a design-build contract. Key to success of the project was careful attention to detailing, mitigation of dynamic behavior, and a high degree of pre-fabrication.

Figure 1 - Bridge superstructure after erection

Keywords: timber footbridge, long span, dynamics, tuned mass damper, design-build, Banff

1. Introduction

Given the highly visible and historically significant location, the client was very careful about the form the footbridge would take, the views it would create both from its deck and from surrounding vistas (of which it would form a part), the materials it would be made of, and the connection it would make with local trails. Driving the agenda, however, was a more pressing concern that existing sanitary pipes installed below the river at this location some fifty years earlier could fail, spilling raw sewage into the pristine Bow River. So the bridge also needed to carry new pipes, creating an important dual use for the new crossing. The bridge is designed to carry medium weight emergency vehicles, addressing another need in the community, as there is only one other crossing to the populated south side of the river. Being situated in a national park, environmental constraints were particularly stringent for both the final structure and the construction process.

In late 2011, the Town created a design-build competition attracting a number of respondents; choosing a winner who addressed the constraints, with a cost which fit within the budget. The final design features an 80m clear span, which for a timber bridge is one of the longest of its kind. This with a relatively slender 4m width create the primary design challenge for the bridge: its dynamic behaviour. Not uncommon with long span footbridges, the primary vertical and lateral frequencies fall directly in the range of susceptibility to pedestrian excitation. To sufficiently damp the response, several damping systems were investigated, with the final system comprising two custom-designed tuned mass dampers suspended visually with cables beneath the bridge. Monitoring of the structure in-place has allowed tuning of the masses to the measured natural frequencies of the bridge.

2. Design

The design-build delivery method enabled both the designers and the Town to pursue a variety of concepts based on a simple set of guidelines:

1. Design within a 135m crossing length, with maximum 5% slope for accessibility.
2. Carry two sanitary and one water line, critical for the Town of Banff.
3. Create a minimal, unimposing design.
4. Minimize work within the river channel.
5. Accommodation of medium sized emergency vehicles.

The structural system is simple: Propped by drilled piers located (to minimize disturbance) just outside the normal river channel, two 40m tapered haunch glulam girders cantilever from either side to support a central 34m suspended span. Tension rods tie the cantilevers down into concrete abutments at either end of the bridge. The bridge cross section comprises twinned sets of stepped glulam girders trussed with diaphragm steel webbing. These are designed to follow the flow of forces, and range in depth from 2.6m at the piers to 0.9m at the suspended span. The 4m wide path is made of solid timber panels which span between the tops of the girders, and are removable to provide access to the service pipes hidden below.

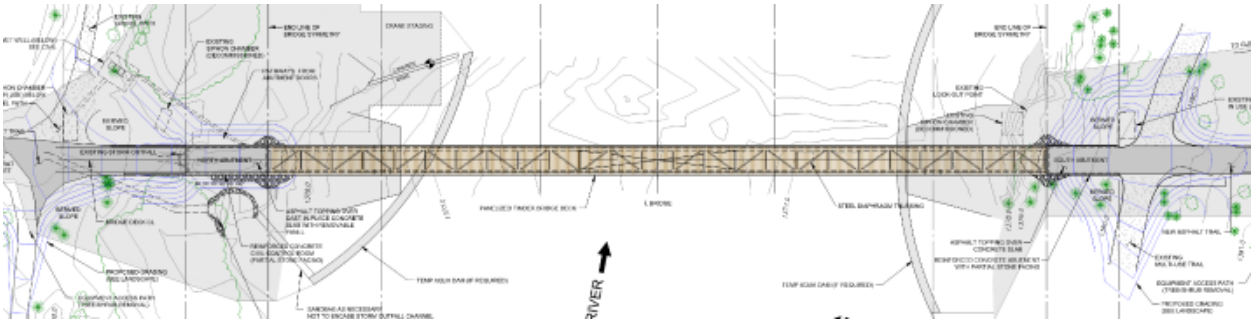


Figure 2 - Site plan

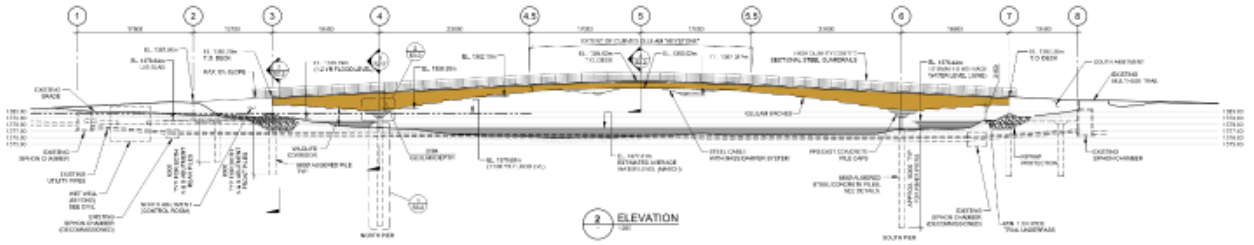


Figure 3 - Bridge elevation

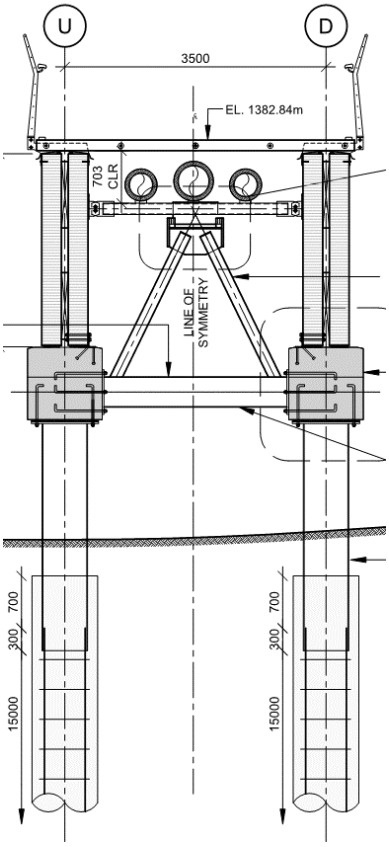


Figure 4 - Section at north pier

The concrete abutments at either end of the crossing build up the pathway to the bridge level, but also house the machine room for the mechanical systems required to run the lift station, eliminating the need for any additional above-grade structures. Thanks to careful positioning of the bridge, existing trails on both sides of the river were maintained without any substantial grade or alignment changes.

Piling in a national park and close to a township was a delicate process, requiring careful selection of foundations. Noise and soil constraints ruled out driven piles, and environmental restrictions within the river basin required the installation of a containment system around the entire piling area. The two sets of piles in the river take large compression loads, as the pivot of the cantilever, while the two front piles at each abutment hold down the cantilever's back-span. Each of the river piles has a steel pipe anchored to augured concrete piles and fastened to the precast concrete pile caps above.

The glue-laminated beams contribute to the bridge's character and natural feel, however wood beams of this scale are not without their challenges. At up to 2.6m deep, 43m long and 11.5T per piece, simply finding manufacturers capable of producing beams this big was a challenge. The girder supplier needed to perform a full survey of their own facility to ensure

Figure 5 - Pile installation through the ice

Figure 6 – Concealment of service pipes

Figure 7 - Drop-in span detail

that once loaded onto a tractor and steerable rear axle rig, the beams with their stepped profile maintained adequate clearance leaving their yard.

Durability was a topic of significant interest to the client. The Town wanted to ensure that the bridge continued to look and perform well for the extent of its design life. A tremendous amount of energy was spent detailing for durability. The glulam pairs were separated by laminated veneer lumber (LVL) blocking and capped with a heavy gauge galvanized steel flashing. The spacing allows full ventilation between members and the flashing creates a drip edge shedding water 100mm from the face of the glulams. All steel components are hot dip galvanized (HDG) or stainless steel, and rubber spacers or grommets separate the two where they interface. The glulams were coated with a high quality system which behaves like a membrane, allowing vapour to pass through while preventing liquid water from penetrating the wood. A darker tint was selected to provide improved UV protection and all three coats were applied in shop prior to shipping. The town has committed to maintenance of the coatings with the goal of increased longevity and performance of the structure.

The steel diaphragm trussing serves a dual purpose, both connecting the pairs of glulam beams and creating support for the three service pipes sitting just underneath the bridge deck. Rollers are attached to the steel webs, allowing the pipes to be easily deployed across the bridge, and accommodating thermal movement of the pipes during the year.

The central drop span sits on neoprene bearing pads on notches in the receiving ends of the cantilevered Glulam beams. This detail is achieved by using long screws to reinforce notch. Diaphragm chord forces are transferred through the connection into the beams via steel drag straps along the tops of the beams, concealed beneath the deck panels and flashing. This forms an elegant, concealed connection which was easily assembled on site and left plenty of tolerance during erection.

3. Structural Analysis

The structural analysis and design of the bridge and associated abutments was in some ways relatively simple – as a determinate system, the basic forces in the cantilevers and drop span due to gravity loading can be calculated quickly by hand. To capture the more complex behaviour

due to stepping of glulam, a finite element model was built incorporating the beams and pile caps as shell elements, and the trussing and piles as beam elements.

The long span and slender section of the bridge make it susceptible to both vertical and lateral excitation from human traffic on the bridge. The use of timber, which is less stiff than steel, also contributed to creating a more flexible structure. Early discussions with the client highlighted the fact that this was likely to be a lively bridge, and mitigation measures were proposed.

The 3D finite element model was also used to investigate the response of the bridge to dynamic excitation. Preliminary modal analyses showed that the fundamental frequencies of the bridge both vertically and laterally fell within the critical range for human excitation. The primary modes of concern and their natural frequencies were as follows:

Table 1 - Critical modes and frequencies

Mode Description	Predicted Frequency (Hz)	Measured Frequency (Hz)	Mechanism
First Lateral	0.8	0.8	Walking
Second Vertical	1.9	1.5	Walking
Second Torsional	3.1	3.3	Jogging

It seemed evident that some form of damping system would be advantageous for user comfort. Detailed dynamic loading models for each of the potential excitation scenarios were imposed on the finite element model, and analysis of the time history acceleration response provided the first estimates of performance under pedestrian excitation and the level of damping required.

Figure 8 - Tuning of mass dampers before erection

Several damping systems were investigated, including viscous dampers fastened to the bridge diaphragm, and combined viscous and tuned mass dampers suspended below the bridge deck. The final system adopted comprised two asymmetric custom-designed tuned mass dampers suspended on cables at quarter points beneath the central span of the bridge. The dampers were created using a cradle which had the capacity to hold different numbers of tuning mass plates. This cradle was suspended from the bridge above with two sets of cables, of particular lengths and stiffnesses.

4. Fabrication

A parametric 3D solids model of the entire bridge was created early on in the design process, allowing rapid and detailed investigation of a multitude of design decisions, providing visual feedback to both designer and client. The model grew through the design process to include every component of the bridge – from the surrounding topography to the giant glulam beams to the minimal guardrails. The parametric nature of the

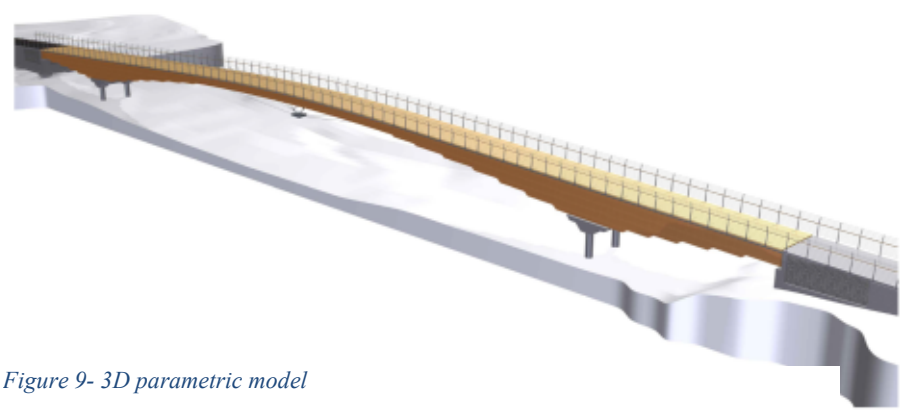


Figure 9- 3D parametric model

model allowed it to respond seamlessly to changes in global form and reflect them down to the smallest level of detail.

The model was critical to many parts of the design process, from early visualization and rendering, to cut-and-fill calculations on the site, trucking viability, and fine tuning the aesthetics of the tuned mass damper beneath the bridge. Most importantly the model was used to create detailed fabrication drawings for each piece of the bridge. The link between model and drawings ensured drawings remained accurate even as the design evolved.

As the Glulam beams are curved, the parametric model contained both 'states' of the beams – pre-bent (straight, prior to cutting) and post-bending (arched and cut to length), allowing accurate dimensional checks to be performed at each of the manufacturing stages and an accurate appraisal of erection weights.

Comprehensive part numbering and component scheduling within the model provided accurate material take-offs and ensured all pieces were assembled correctly on the site.

Fabrication of all timber elements occurred in the shop. The direct proximity of the fabrication shop and the design team allowed full-scale 'rapid prototyping' of many of the key components of the bridge. Handrails, stanchions, and deck panels all went through several mock-ups and design iterations before settling on a final design optimised for both form and aesthetic.

Throughout the design and fabrication process, the delivery model led to a natural desire to create efficiency: similar to the manufacturing industry, conceptualisation, design, modelling, testing, fabrication, and erection takes place in-house, allowing each step of the process to be influenced by the other steps.

Figure 10 - Glulam delivery at fabrication shop

5. Erection

A tight, remote site, huge structural elements, and harsh winter weather all combined to make the bridge erection a challenge. This, coupled with a desire to complete the lifts before spring thaw and a firm arrival date for a large mobile crane put a huge emphasis on ease and accuracy of assembly in the field.

The main structural elements of the bridge were too large to be transported to the site in modules ready for installation; and fitting up the pieces over the river with a smaller crane would have presented significant environmental and safety challenges. In order to accurately assemble and

erect the bridge, the individual elements were prefabricated in the shop and shipped to site as a kit of parts. All cutting, drilling, sanding and finishing was performed indoors under controlled conditions so that members were protected from the elements both in transit and on site. Jigs were built to ensure accurate assembly of the main bridge components in the field. The stepped glulam girders were paired up and joined by long fully threaded screws through pre-drilled holes. Once the beams were levelled and squared to each other, the diaphragm webbing was connected with friction grip bolts to the receiving plates which were fastened to the innermost glulam face.

Figure 11 - Erection of south girders

During the final preparations for the big lift, temporary walking planks and safety lines were installed to provide immediate access to the bridge sections after installation. Additionally, several survey targets were installed to the girders, both at the abutment end and at the tips which would receive the drop-in span. This allowed real-time verification and placement of the bridge sections and fine tuning of the hold-down rods at the abutment ends to compensate for dead load deflection of the girders and the drop-in span.

Figure 12 - Erection of drop-in span

In all, the entire bridge superstructure was erected in 3 lifts over 2 days with an additional day spent relocating the crane to the North bank of the river. The tapered girder assemblies weighed in at over 50T while the drop-in span, complete with tuned mass dampers, maxed out the 500T crane at just over 20T and a radius of 45m. The bridge sections rested within 12mm of their target coordinates and the drop-in span "fit like a glove," according to the erector.

Figure 13 - Dropping over tie-down rods at abutment

6. Dynamic Testing

The desire for field testing of the bridge arose early in the design phase and gained traction as the importance of its dynamic behaviour came into focus. The testing was primarily required to provide experimental validation of the structure's natural frequencies; but a number of additional tests were performed which provided estimations of modal damping as well as measurements of pedestrian induced accelerations for comparison with user comfort criteria. This information allowed for precision tuning of the dampers and quantification of their effect on user comfort.

A variety of tests are available to determine dynamic properties of structures, however, the Ambient Vibration Test stood out as the most appropriate and convenient method. Due to the current availability of high precision sensing equipment and sophisticated data analysis software packages, experimental modal

estimation, which once required extensive equipment, custom software, and precise knowledge of the input forces and frequencies, can now be performed with a few accelerometers and a laptop. The test setup involves positioning of accelerometers at locations of maximum estimated modal displacement which are set to record simultaneous data sets of the accelerations measured under ambient vibration of the structure.

The testing of the Banff Bridge utilized a commercially available software package for the determination of the natural frequencies and associated mode shapes. The software applies local degrees of freedom to the global geometry where accelerometers are to be placed. Higher accelerations resulting from resonance at the structure's natural frequencies appear as peaks on a frequency vs acceleration plot, which made the modal frequencies easily identifiable. Finally, by selecting the peaks on the graph, the software animates the dominant motion of the structure as measured by the accelerometers. In this way we were able to visually confirm which mode shapes we were measuring and compare with the analytical model.

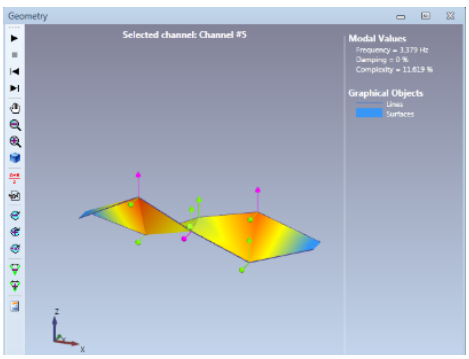


Figure 14 - 2nd torsional mode (from AVT)

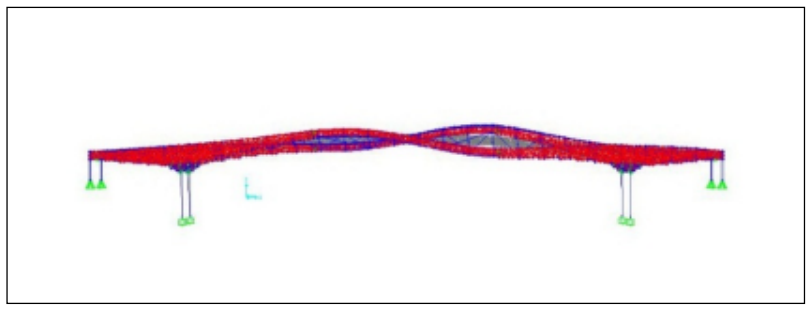


Figure 15 - 2nd torsional mode (analytical model)

Once the true natural frequencies were known, it was time to tune the dampers. Instead of simply tuning to the measured frequencies, baseline testing was performed to estimate the modal damping of the structure before and after tuning. Resonance tests, which seek to excite only specific modes, were performed by jumping up and down at the exact natural frequency of the mode being excited. With the assistance of a metronome, it was relatively easy to activate the modes of interest by jumping at a location of high modal displacement. Once resonance was achieved, the excitation was stopped and measurements of the free decay response were taken.

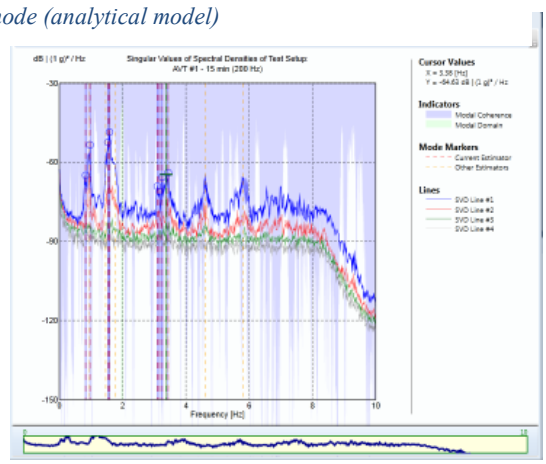


Figure 16 - Frequency domain decomposition graph

Estimation of modal damping was determined by fitting of an exponential curve to the free decay response according to equation (1).

$$a(n+1) = a(n)e^{-2\pi\xi\sqrt{(1-\xi^2)}} \tag{1}$$

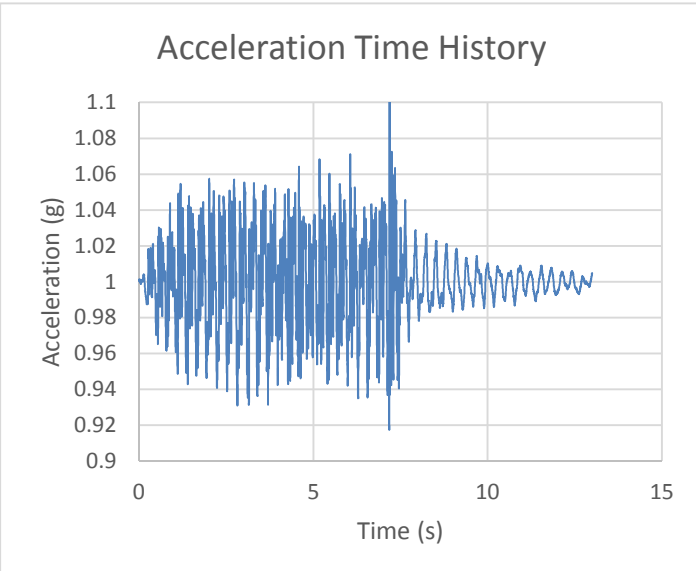


Figure 17 - Excitation and free decay of 2nd torsional mode

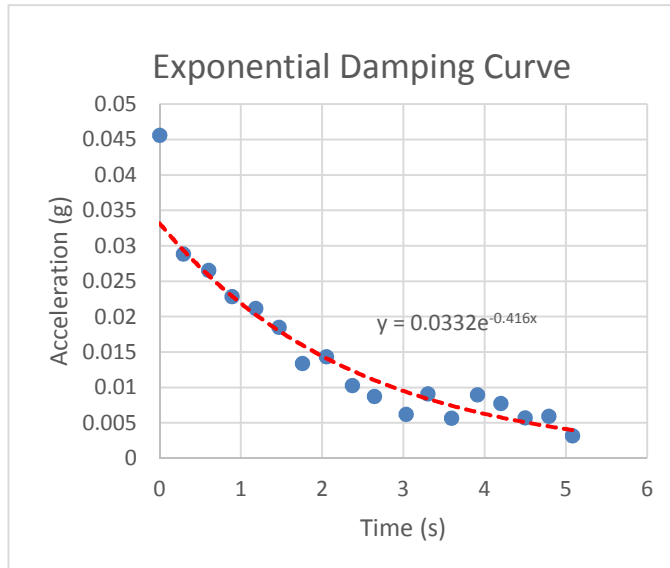


Figure 18 - Fitting of exponential curve to free decay response

Since the two tuned mass dampers were designed to mitigate motions excited by the 2nd vertical and 2nd torsional modes of vibration, these were of particular interest. The 2nd vertical mode was predicted to be at 1.9 Hz and the damper's vertical stiffness & mass were proportioned to have the same natural frequency with room to add or remove mass for tuning. The measured frequency of the 2nd vertical mode was significantly lower than anticipated at 1.5 Hz; meaning the structure was softer than predicted, likely due to uncertainty in modelling the boundary conditions. Frequency follows equation (2) and being unable to alter the stiffness provided by the

$$f = 2\pi\sqrt{\frac{k}{m}} \tag{2}$$

cables suspending the carriage, the mass needed to be increased by over 600 kg (or 60% of its predicted mass) for which we had made sufficient allowance in the damper carriage. By contrast, the 2nd Torsional mode was predicted to be 3.1 Hz and was measured at 3.3 Hz. This required an increase of only 50 kg to 375kg total. Once the masses of the dampers had been calibrated to the bridge's natural frequencies, an additional set of resonance tests was performed for both the 2nd vertical and 2nd torsional modes. The accelerations and structural damping were compared with the baseline tests performed before tuning.

The tuned mass dampers improve the dynamic performance by preventing the accelerations from reaching unacceptable levels by participating in and out of phase at the natural frequencies; thus preventing a resonant response. The results of the before and after tuning resonance tests showed that the damping ratio increased slightly but remained at a relatively low 1.5-2% depending on the mode under consideration. Vertical accelerations were reduced by over 50% to less than 1 m/s^2 during excitation of the 2nd vertical mode.

In order to calibrate the model to match the field tested frequencies, very small adjustments to suit the boundary conditions between the drop span and haunched girders as well as the bearing surface between the girders and the precast pier cap brought the analytical frequencies precisely in line with measured values. This sensitivity to actual site conditions is a major reason why there is a continued need for field testing of vibration controlled structures.

7. Conclusion

Footbridges are an excellent opportunity to exploit the use of timber, especially in beautiful settings. However great care in design and execution must be taken to account for its unique properties, and to ensure the structure will remain an enhancement to the setting which all can enjoy for many years to come. Proper credit goes to all involved, including a sympathetic client with great aspirations!

Figure 19 - Finished bridge south bank

Figure 20 - Finished bridge south elevation

Figure 21 - Opening day crowds

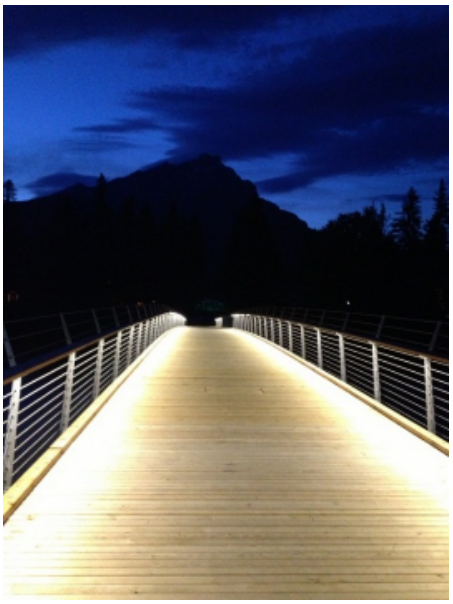


Figure 22 - Illuminated walkway

Simplified Analytical Model for a Queen-Post Covered Timber Bridge

By

F. Fanous
Professor
Iowa State University
Ames, Iowa, USA
fanous@iastate.edu

D. Rammer
Research Engineer
Forest Product Laboratory
Madison, WI, USA
drammer@fs.fed.us

T. Wipf
Professor and Chair
Iowa State University
Ames, Iowa, USA
tjwipf@iastate.edu

Summary

During the 19th century, the economic material to build bridges was timber due to its abundant availability, cost, and ease of construction. Many of the well-known timber bridge types are the Burr arch, Town lattice, Howe, Queen and King type of trusses. This paper summarizes an investigation that was sponsored by the Federal Highway Administration and the USDA Forest Products Laboratory to recommend simple but accurate analytical techniques for improving the analysis of historic covered timber bridges. Within this manuscript, field testing results of displacements and strains for covered Queen-Post-Truss timber bridges was utilized to validate the results of the developed finite element model.

A two dimensional model that included the properties of the truss individual members was utilized by the STAAD finite element program to carry out the analysis. Eccentricities that existed at the connectivity between timber members, i.e., details of the connection between the different members in the truss systems were included when calculating the stresses and the stains in each member. The moving load option that is available in the STAAD software was utilized to represent the moving load of the vehicle that was used in the field test.

The finite element results compared well with the field measured results. However, one needs to pay attention when modeling the applied load and the boundary conditions. In addition, care must be taken when calculating additional strains due to the eccentricity of the joints between the individual members.

Keywords: historical, timber, Queen Post, finite element, splice joints; joint eccentricities.

1. Introduction

During the 19th century, the economic material to build bridges was timber due to its abundant availability, cost, and ease of construction. Due to the availability of timber and the need for a safe way to cross the lands, thousands of these timber covered bridges were built during the 19th and early 20th centuries [3]. Many of the well-known timber bridge types are the Burr arch, Town lattice; Howe, Queen and king type of trusses. These types of bridges have many of the same characteristics, but each is uniquely different enough to cause concern when evaluating the structural behavior of each bridge. Also, these differences add some complexity to accurately analyze these types of bridges. For example, there are several eccentric connections, various load paths, connection uncertainty between the subassemblies (trusses and arches), and interaction between the trusses and their housing. When these are combined with material variability, it is easy to question the use of simplified truss analysis to design these types of bridges. This is recognized in the Federal Highway Administration's (FHWA) publication

FHWA-HRT-04-098, Covered Bridge Manual [6]. The manual states that there are inconsistencies with the assumptions of traditional simple static analysis of these covered bridges using simple analysis of trusses.

This manuscript summarizes the results of using a simple but more accurate analytical technique for the analysis of historic covered timber bridges. The STAAD [5] finite element program was utilized to accomplish this objective by analyzing the Moxely Bridge , and the results were validated using the results that are documented in a report of the field study that was conducted by Hosteng, et. al. [4].

2. Field Test of Moxley Covered Bridge

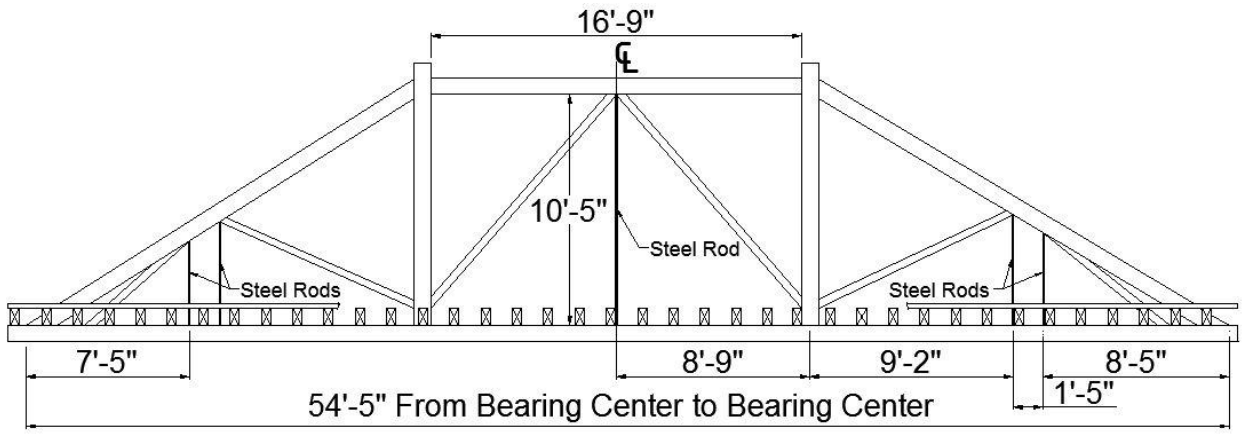
2.1 Bridge Descriptions

The Moxley covered bridge is located 2.5 miles south of junction then 0.1 mile in the town of Chelsea in Orange County of the State of Vermont. The bridge was constructed in 1883. Fig. 1 shows different photographic views of the bridge.

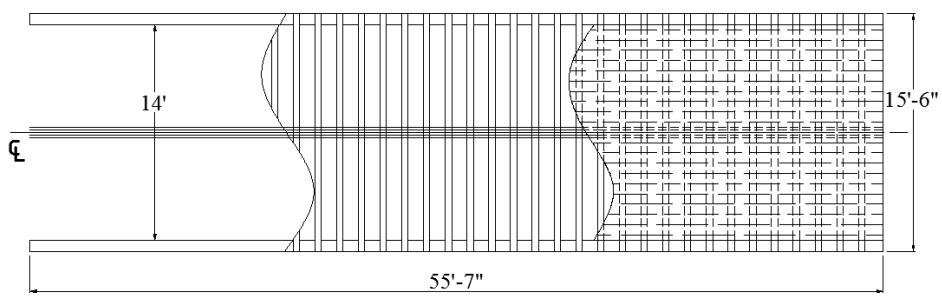
Fig. 1 Different views of the Moxley Bridge

The bridge span is approximately 54 ft. - 5 in. (see Fig. 2) and is 13 ft.-9 in. wide. The bridge deck is made of continuing 7 to 10 in. wide, 2 in. deep and 15 ft. long timber planks placed closely to each other. The bridge deck is seated on 5 in. by 9.5 in. floor beams which are about 1 ft. - 3 in. spacing from each other. All floor beams are supported on the two trusses bottom chord and a beam that is located at the middle of the bridge deck. The bottom chords of the two trusses of the bridge are 9 in. by 9 in., while the middle beam is formed by four 2 in. by 10 in. that are bundled together. However, the contribution of the middle beam to transfer any of the applied loads to the end supports was excluded in the following analysis. This was justified by the small flexural stiffness of such a long beam.

Figure 2 shows schematic elevation and plan views of the Moxley Bridge. Notice that Queen-Post truss system was provided by tension rods and were provided to act as additional supports to the trusses bottom chords. In addition, diagonal members were also provided as additional supports to the trusses top chords as shown in Fig. 2. The dimensions of structural members are listed in Table 1.



a. Elevation View



a. Plan View

Fig. 2 Schematic elevation and plan views of the Moxley covered Timber Bridge

Table 1 Measured member dimensions of the Moxley Bridge

Structural Member	Width (in.)	Height (in.)
Bottom & Top Chords	9	9
Floor Beam	5	9 ½
Verticals	9	9
Diagonals	4	9
Tension Rods	1 in. diameter	

Note: 1 inch = 25.4 mm, 1 feet = 762 mm.

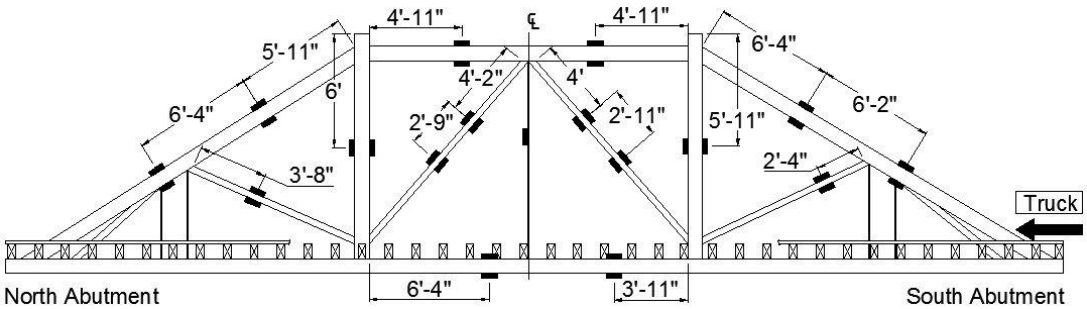
2.2 Vehicles Used in the Field Test of the Moxley Bridge

The vehicle that was utilized in the load testing of the Moxley Bridge is shown in Fig. 3. The front and back axles of the truck weigh 20.9 kN and 50.9 kN, respectively. The distance between the two axles was spaced at 4,470 mm.

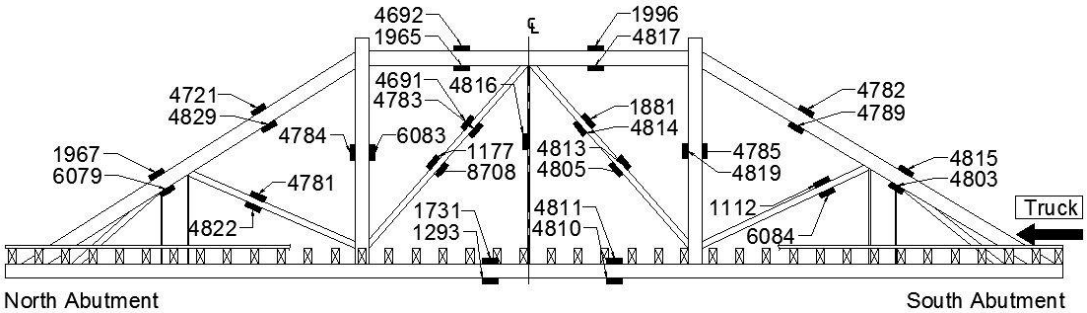
Fig. 3 Photo of the Truck used in testing of the Moxley Bridge

2.3 Bridge Instrumentation

Instrumentation used in testing the Moxely historical bridges consisted of BDI [1] strain sensors (referred to hereafter as strain sensors) with a resolution of 1 micro-strain ± 1 and the BDI testing software from Bridge Diagnostics Inc. Displacement data were obtained from Unimeasure PX string potentiometers having a resolution of 0.001 inches ± 0.001 connected to a Megadac logger. Both systems recorded data at 20 samples per second per sensor. Syncing of the data between the two systems was accomplished with a simultaneous data mark inserted into the files at the appropriate truck location positions. Several members of the Moxley Bridge were instrumented to measure the induced strain when a truck travels over the bridge. The locations and the identification number of these sensors are shown in Fig. 4. Sensors were also provided to measure the global displacements at the quarter and mid span of the Queen-Post truss.



i) Strain Sensor Locations



ii) *Strain sensor numbers*

Fig. 4 Strain sensor truss locations for the Moxley Bridge

3. Analysis of the Moxley Bridge

3.1 Finite Element Model

The STAAD computer software was selected to perform the analysis of the covered timber bridges studied in this work. This program was selected due to its simplicity in creating the required input file, to model the splice connections and to simulate a moving load. The Bridge was analyzed using two dimensional idealizations (see Fig.5). Beam elements were used to idealize the top and bottom chords of the Moxley Bridge. All diagonals were modeled as axial load members. Tension members only were utilized to represent the tie rods that were used in the construction of the Moxley Bridge. Figure 5 below illustrates the finite element model of the bridge as was utilized in the STAAD analysis. Internal hinges were inserted at specific locations to release the moments where elements were discontinued.

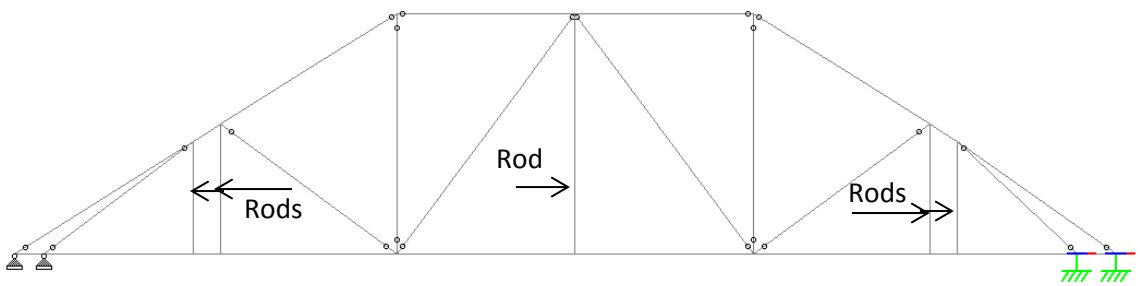


Fig. *5 STAAD two dimensional finite element model of the Moxley Bridge*

3.2 Boundary Conditions

Field inspection of the Moxley Bridge illustrated that each end of the truss structure was seated on a bearing surface that is approximately 1016 mm in length. This was characterized in the analysis by providing supports within the bearing length. All supports on one end were assumed pin type while roller supports were provided at the other abutment.

3.3 Loading

As previously mentioned, the Moxley Bridge was tested using a two axle truck. The load was driven at a slow speed across the bridge structure. This was simulated in the analysis using the moving load option that is available in the STAAD program. The truck's two axle loads were directly applied on the bottom chord of the truss system since the floor beams in this bridge were spaced approximately 50.8 mm on center.

3.4 Material Properties

The listed values for the timber members were estimated by the researchers of the Forest Product Laboratory after conducting moisture content, stress wave and resistor graph field tests. The tests resulted in young modulus for the bottom chord members of 15.3 GPa and 12.76 GPa for all other timber members. A young's modulus of 200 GPa was used for the steel rods.

4. ANALYSIS RESULTS and DISCUSSION

4.1 Displacement at Mid-span

Figure 6 shows the variation in the displacement at the midpoint of the truss bottom chord as the truck was driven across the bridge. The figure shows that there is a lag between the field measured displacements in the east and west trusses, but they are similar in magnitude. This lag is caused by the skew angle for the bridge structure. Also, the figure illustrates that there is a slight difference between the analytical and the measured displacements near the locations of the tension rods. These differences could have resulted from the behavior of the idealized connection between the tension rods and the truss top and bottom chords at these locations. In the analytical model, the two chords of the truss and the rods were assumed to be connected to these rods at common nodes. However, this idealization does not allow for local deformation that occurs in top and bottom chord members in the vicinity of the nuts at the end of these rods. These deformations result from the application of the tie rod forces which act perpendicular to the wood grain of the top and bottom chord members. Such deformation can be significant since the timber of the elastic modulus in the direction perpendicular to the wood grain is approximately one-tenth of the longitudinal elastic modulus in the direction parallel to the wood grain. The differences in the material moduli were not considered in the analytical model.

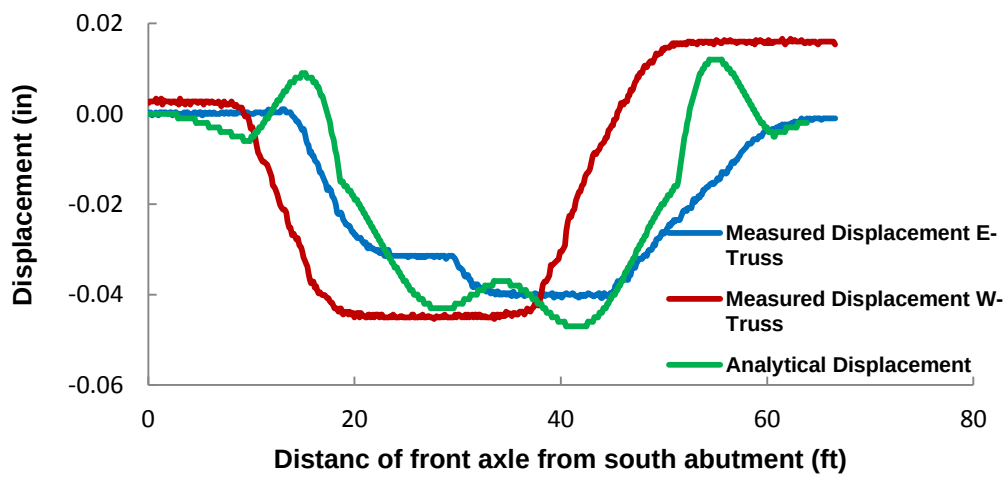


Fig. 6 Variation in the mid-span displacement as the truck travels along the Moxley Bridge

4.2 Comparison of Analytical and Field Strains

Figures 7 to 11 document the agreement between the measured and analytical strains in all members except in the bottom chord. For example, Fig. 7 depicts the variation in the strains in a diagonal member just to the right of the middle rod as truck travels along the center of the bridge deck. One may notice that there are differences of the locations of the peaks of the analytical and the recorded strains. This was due to the inability to match the exact truck locations when the field data was recorded to the analytical locations where the truck was positioned on the bridge in the STAAD analysis. Similar behavior was also noticed when examining the strains in the timber post to the right of the middle rod (see Fig. 8).

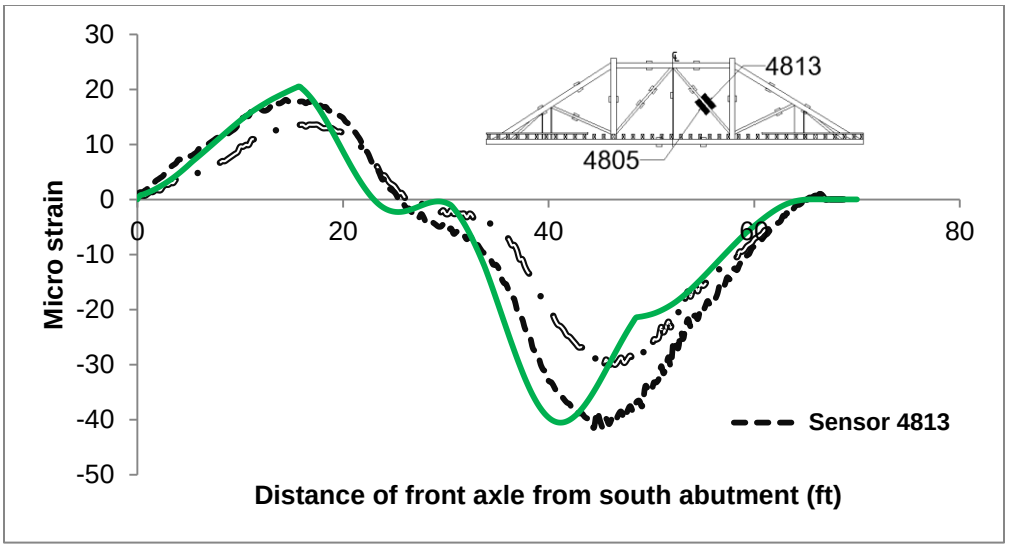


Fig. 7 Analytical and field strains in the diagonal member just to the right of the middle rod

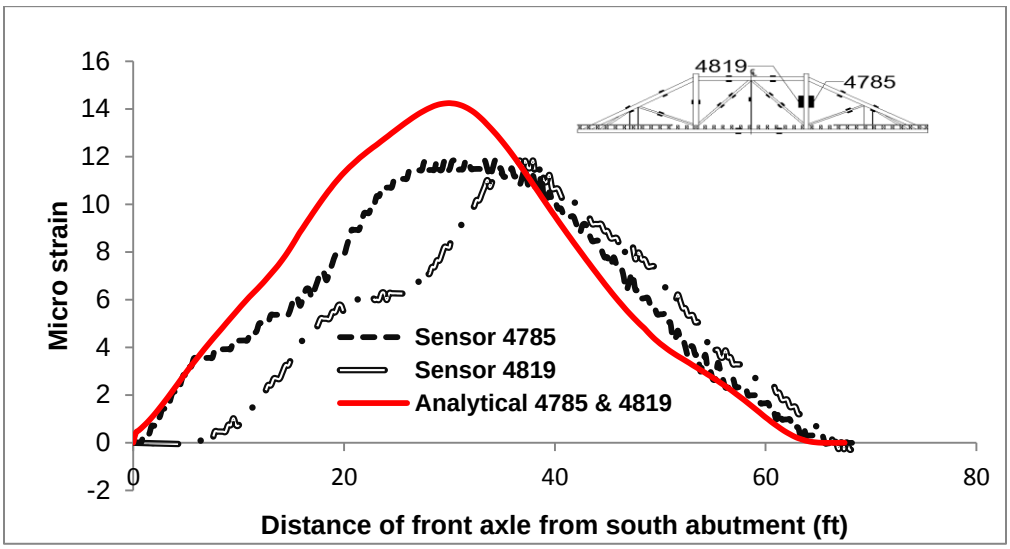


Fig.8 Analytical and field strains in the left timber post

Figure 9 illustrates that there is a slight difference in the analytical and recorded field strains in the first top chord member (see Fig. 9). These differences can easily result from not using the actual material modulus of the members in the analytical model. On the contrary, large differences between the field measured and the analytical strains were observed when investigating the performance of the middle rod (see Fig. 10). This could have resulted from the method used to mount the strain sensor on the element. In the field test two clamps were used to mount the flat sensor on the round bar. Another disagreement between the field measured and the analytical strains was noticed when examining the behavior of the truss bottom chord member just to the right of the middle rod (see Fig. 11). Both analytical and field measurements showed that the bottom chord is subjected to direct axial and bending strains (see Fig. 12). However, the effect of the bending strain was very noticeable in the analytical results. Examining the photo from the field of the location in the vicinity of these strain sensors showed

that there was a noticeable check and irregularity in the chord cross section in the vicinity of the sensors that were located on the bottom chord of the Moxley truss (see Fig. 12). This could be one of the reasons for the large differences between the measured and the calculated strains. Additional comparison between measured and calculated strains in other members is summarized in [2].

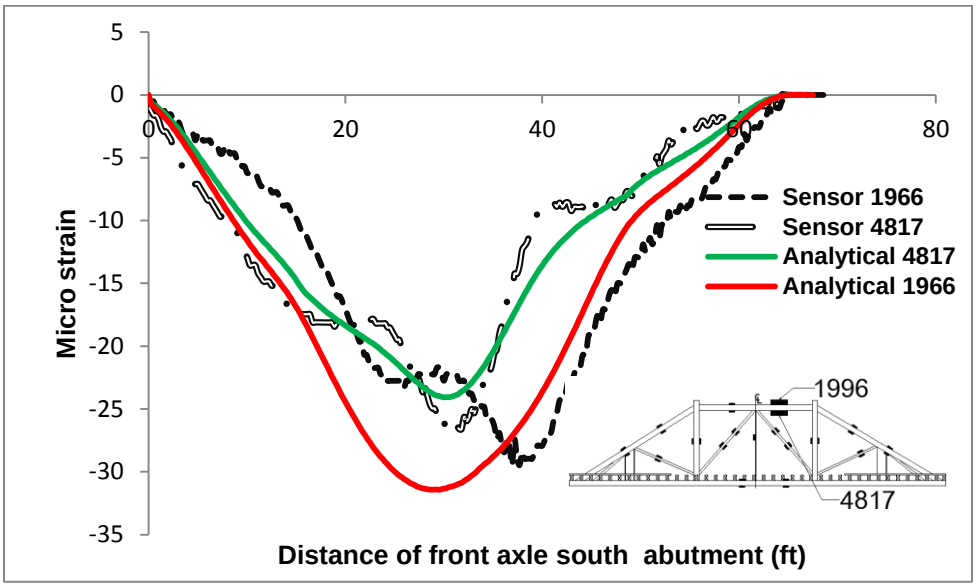


Fig.9 Analytical and field strains the left diagonal member

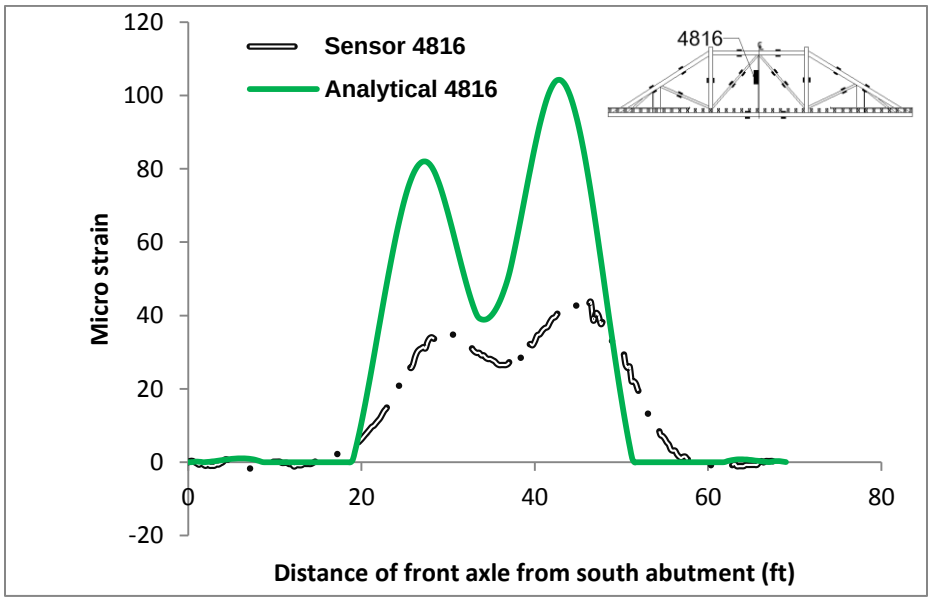


Fig. 10 Analytical and field strains in the middle rod

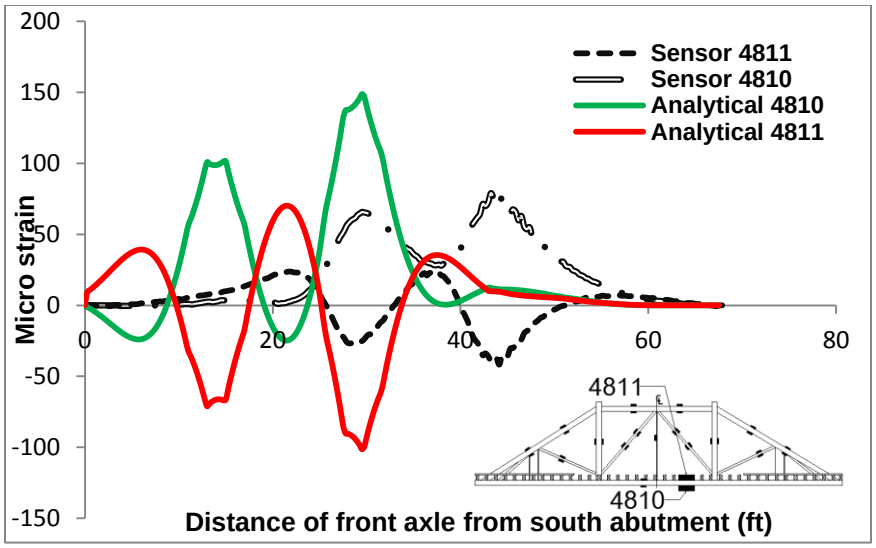


Figure 11 Analytical and field strains in the bottom chord member to the right of the middle rod

Fig. 12 Photo showing the condition bottom chord member of the Moxely- south truss

5. Summary and Conclusions

This paper summarizes an investigation that was sponsored by the Federal Highway Administration and the USDA Forest Products Laboratory to develop a simple but accurate analytical model to analyze a Queen-Post Truss bridge. A three dimensional model that included the splice joints that were used in the construction of the bridge was utilized in the analysis. The results of the analytical model were validated with the data obtained from the field testing of the same bridge structure.

The finite element results showed that idealizing the truss-arch structure excluding the eccentric in the connection between the vertical and diagonals, top chord and bottom chord members may yield a more stiff structure. In addition, the analytical model showed that the arch contributed to the load carrying capacity of the bridge. The strains obtained using the analytical model yielded very reasonable strain results; however, one must take into account the effect of the inherent eccentricity within the joints at both ends of members of the bridge structure.

6. Acknowledgement

The study presented in this paper was supported by the FHWA and the USDA Forest Products Laboratory (FPL) under the National Historic Covered Bridge Preservation Program. The authors wish to thank Mike Ritter, Assistant Director, USDA FPL, for his cooperation and assistance during the course of the work. Special thanks to Allison Lunde (Machtemes) for her assistance in performing some of the finite element analysis presented in this manuscript. Special thanks are extended to Douglas Wood, ISU Structural Engineering Laboratory Manager, for his assistance with data interpretation and conduct of the testing.

7. Bibliography

[1] Bridge Diagnostics, Inc., BDI. 1989. Bridge Testing, Load Testing & Structural Monitoring Solutions, Boulder, CO.

[2] Fanous, F., et. al., Improved Analytical Techniques for Historical Covered Bridges, Report submitted to Forest Product Laboratory, January 2013.

[3] "History of Covered Bridges." 2010. Welcome to the GDOT. Georgia Department of Transportation.<http://www.dot.state.ga.us/travelingingeorgia/bridges/coveredbridges/Pages/HistoryofCoveredBridges.aspx>.

[4] Hosteng, T., et. al., Covered Bridge Rating through Load Testing, Report submitted to Forest Product Laboratory, January 2013.

[5] STAAD.Pro, 2011. Structural Analysis and Design, Revision 8.0: Bentley Systems, Exton, PA. USA.

[6] US Department of Transportation Federal Highway Administration. 2005. Covered Bridge Manual. Rep. no. FHWA-HRT-04-098. McLean, VA: Federal Highway Administration, Print.

Simplified Analytical Model of a Covered Burr-Arch-Truss Timber Bridge

By

F. Fanous
Professor
Iowa State University
Ames, Iowa, USA
fanous@iastate.edu

D. Rammer
Research Engineer
Forest Product Laboratory
Madison, WI, USA
drammer@fs.fed.us

T. Wipf
Professor and Chair
Iowa State University
Ames, Iowa, USA
tjwipf@iastate.edu

Summary

Due to the importance of historical timber covered bridges throughout history, their preservation is necessary. However, conducting an accurate structural evaluation of these types of bridges has always caused difficulties to bridge engineers. This paper summarizes an investigation that was sponsored by the Federal Highway Administration and the USDA Forest Products Laboratory to develop a simple but accurate analytical model to analyze Burr Arch Truss bridges. A three dimensional model that included the splice joints that were used in the construction of the bridge was utilized in the analysis. The analysis was carried out using the STAAD finite element software.

The displacement and strain results that were obtained from the analytical models were compared to those measured in the field. The finite element results compared well with the field measured results if one models the connections and the eccentricities that existed at the connectivity between timber members.

Keywords: historical, timber, Burr-Arch, bridge, finite element, splice joints, joint eccentricities.

1. Introduction

During the 19th century, the economic material to build bridges was timber due to its abundant availability, cost, and ease of construction. Thousands of timber covered bridges were built during the 19th and early 20th centuries [5], yet a relative small number remain today. The top four surviving timber bridge types are the Burr arch, Town lattice; Howe, Queen and king type of trusses. These types of bridges have many of the same characteristics, but each is uniquely different enough to cause concern when evaluating the structural behavior of each bridge. These differences add some complexity to accurately analyze these types of bridges. For example, there are several eccentric connections, various load paths, connection uncertainty between the subassemblies (trusses and arches), and interaction between the trusses and their housing. When these are combined with material variability, it is easy to question the use of simplified truss analysis to design these types of bridges. This is recognized in the Federal Highway Administration's (FHWA) Covered Bridge Manual [9]. The manual states that there are inconsistencies with the assumptions of traditional simple static analysis of these covered bridges using simple analysis of trusses.

This manuscript summarizes the results of using a simple but more accurate analytical technique for the analysis of historic covered timber bridges. The STAAD [8] finite element program was utilized to accomplish this objective by analyzing the Zacke Cox Burr Arch Bridge, and was validated using the results that are documented in a report of the field study [6].

2. Analysis of Zacke Cox Covered Bridge

2.1 Bridge Descriptions

The Zacke Cox Covered Bridge is located in Parke County, Indiana. Elevation and end view photographs of the bridge are shown in Figure 1. The bridge was originally built in 1908 by Joseph A. Britton and its roof and deck were replaced in 1989. The deck was again replaced in 1991 and the bridge was restored in 2002 [7].

The Zacke Cox Bridge is a one lane, single span, simply supported double Burr arch truss with an approximate measure of 51 ft. and 2 in from the centerlines of the two end bearings. The truss consists of rectangular parallel chords, concentric arches enclosing the truss, two member lower chords, one member upper chord, one member diagonal and one member vertical. A schematic of the truss elevation is provided in Figure 2. Currently, the bridge is rated and posted for a thirteen ton load limit [2].

Fig. 1 Different views of the Zacke Cox Bridge

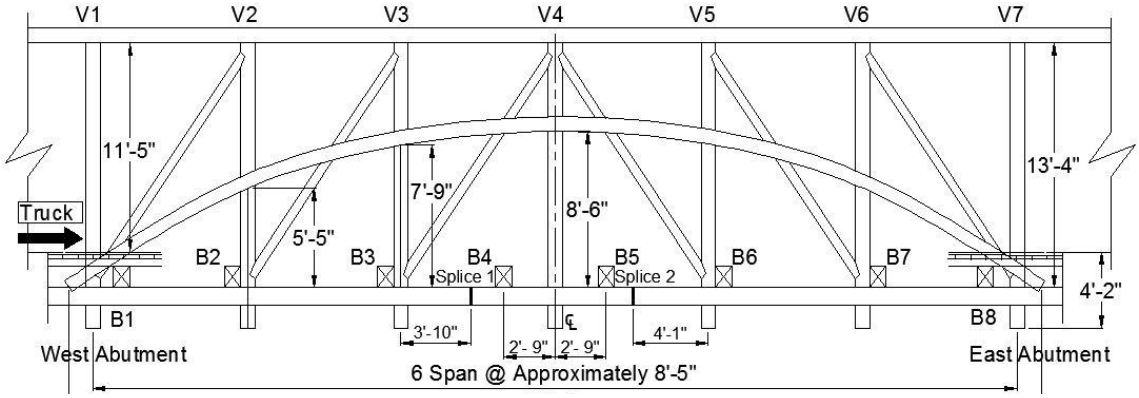


Fig. 2 Elevation view of the Zacke Cox Bridge

Table 1 summarizes the measured dimensions of each structural member used within the bridge and analytical models.

Table 1 Measured member dimensions

Structural Member	Width (in.)	Height (in.)
Bottom Chord (2)	5 ½	11 ½
Floor Beam (new)	10 ½	13 ¾
Verticals	7 ½	9 ½
Diagonals	7 ½	7 ½
Arch (2)	7 ¾	9 ½
Top Chord	7 ½	9 ¾
Stringers	5	5 ½

Due to the natural limitations of timber, splices are used to connect two timber members to create the bottom or top chords of the truss structure. The Zacke Cox Bridge utilizes single headed hook fishplate and iron shoe splice joints within the bottom chord. There are two such splice joints in the Zacke Cox Bridge. These joints are located one on each side of the center, i.e., between verticals V3-V4 and V4- V5 as shown in Figure 3.

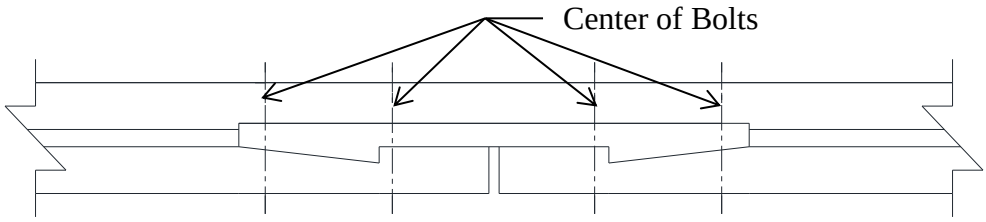


Fig. 3 Plan view of bottom chord splice joint between V3 and V4

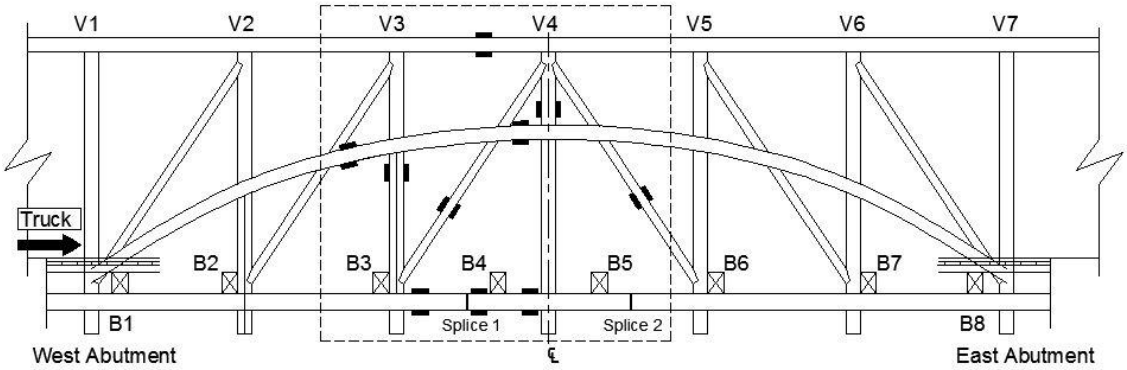
2.2 Vehicles Used for the Field Test

Fig. 4 Picture of large test vehicle

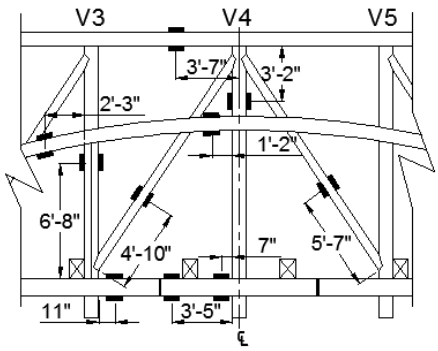
The two vehicles were used in the field testing but the analysis presented within will focus on the larger vehicle in Figure 4. This larger truck had a front axle weight of 447.5 kN, a rear axle weight of 68.7 kN, and a center to center axle spacing of 3.07m. The trucks were driven across the bridge down the center of the structure at a slow rate, approximately five miles per hour, to simulate static loading conditions.

2.3 Bridge Instrumentation

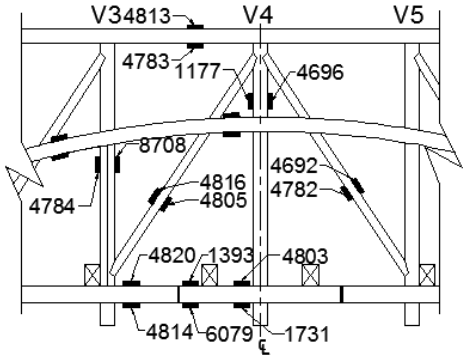
The locations of the BDI [1] strain sensors are shown in Figures 5 below (the listed numbers are used to identify the location of each sensor) for the south and north trusses; respectively. Unimeasure PX string potentiometers were used to measure the global vertical displacements at the mid and quarter span locations. Shown in Figure 6 are the locations of the strain sensors that were used near the splice joint in the bottom chord of the north truss. All measurements were recorded at 20 samples per second per sensor. More details regarding the field test of the bridge is given reference [6]. The truss top chord was spliced at verticals V2, V4 and at V6.



i) Instrumentation layout

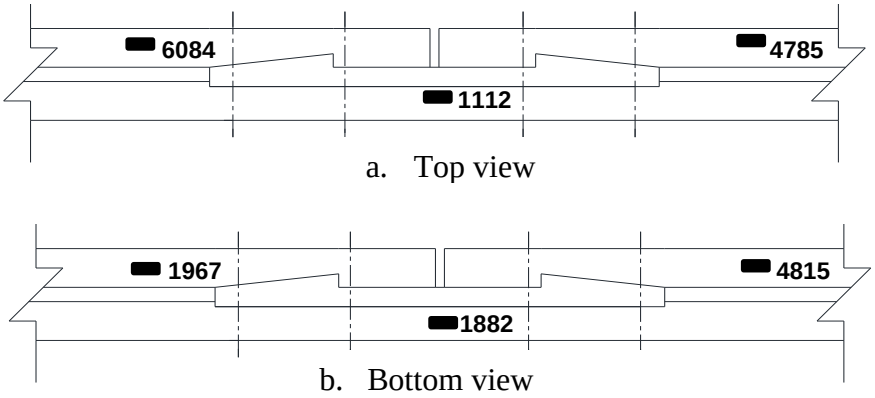


ii) Strain sensor locations



iii) Strain sensor details

Fig. 5 South truss strain sensor locations



a. Top view

b. Bottom view

Fig. 6 Strain sensor locations near splice1 between vertical V3 and V4

3. Analysis of the Zack Cox Bridge

3.1 Finite Element Model

The STAAD computer software was selected to perform the analysis of the covered timber bridges studied in this work. This program was selected due to its simplicity in creating the required input file, to model the splice connections and to simulate a moving load.

The bridge was analyzed using three dimensional idealizations (Fig. 7) of a single truss. Internal hinges were included in this model to represent the connectivity between the different members. The locations of these internal hinges are indicated with open circles in Fig.7. The splice joints were included in the three dimensional model by releasing the moment in the bottom chord members at the corresponding location of the splices. One end of the truss was assumed to be pinned, while roller support was imposed at the other end. The two ends of the arch were assumed to be pinned.

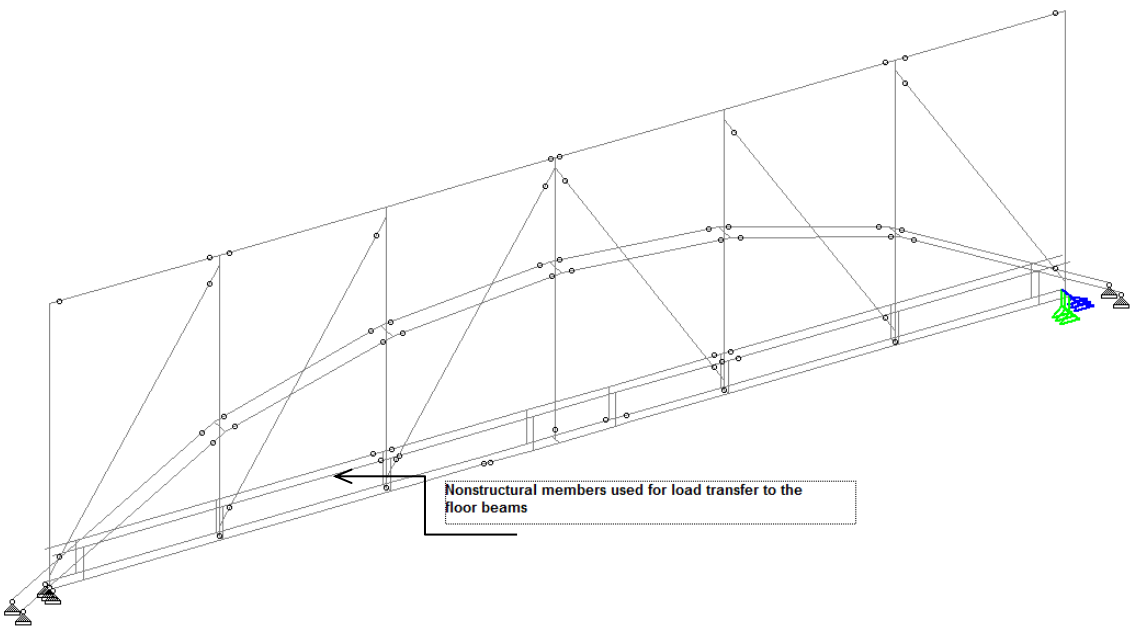


Fig. *7 STAAD three dimensional finite element model of the Zacke Cox Bridge*

The bridge model was loaded using the moving load option available in the STAAD program. To transfer the moving load to the truss bottom chord at the locations of the floor beam, it was necessary to add at the deck level, two longitudinal beams that were not part of the bridge structure. These members were added to facilitate the application of the moving loads. Each of these beams was connected using rigid links to the truss bottom chord at the locations of the floor beams. The properties of these beams were arbitrarily selected to match the dimensions of the two stringers that were used in the bridge.

The wood material was determined to be Eastern Hemlock and a modulus of elasticity of 8.27 GPa was assigned from the Wood Engineering Handbook [4]. For anatomical analysis of wood samples the trees species was identified as Tsuga. This species information, bridge location, and the Atlas of United States Trees [9] lead to the Eastern Hemlock selection. Steel bolts were assigned an elastic modulus value of 200 GPa.

3.2 Boundary Conditions and Loading

Visual inspection in the field indicated that the truss structure could be characterized as a simply supported structure while the arch portion of the bridge structure was pinned at each end.

The truck load was applied on the bridge using the moving load option available in the STAAD program. The truck axles were positioned on the bridge deck at several locations to emulate the strain and displacement data recorded in the field test.

4. Analysis Results and Discussion

4.1 Displacement at Mid-span

Shown in Figure 8 is a comparison between the average measured and the calculated displacements at the midpoint of the bridge structure. As previously mentioned, in an ideal situation the measured displacements in the north and south trusses should be similar. Therefore, the average of the two measured displacements was utilized in the comparison used in Figure 8. The results summarized in Figure 8 shows that the average measured displacement is in good agreement with the analytically calculated displacement.

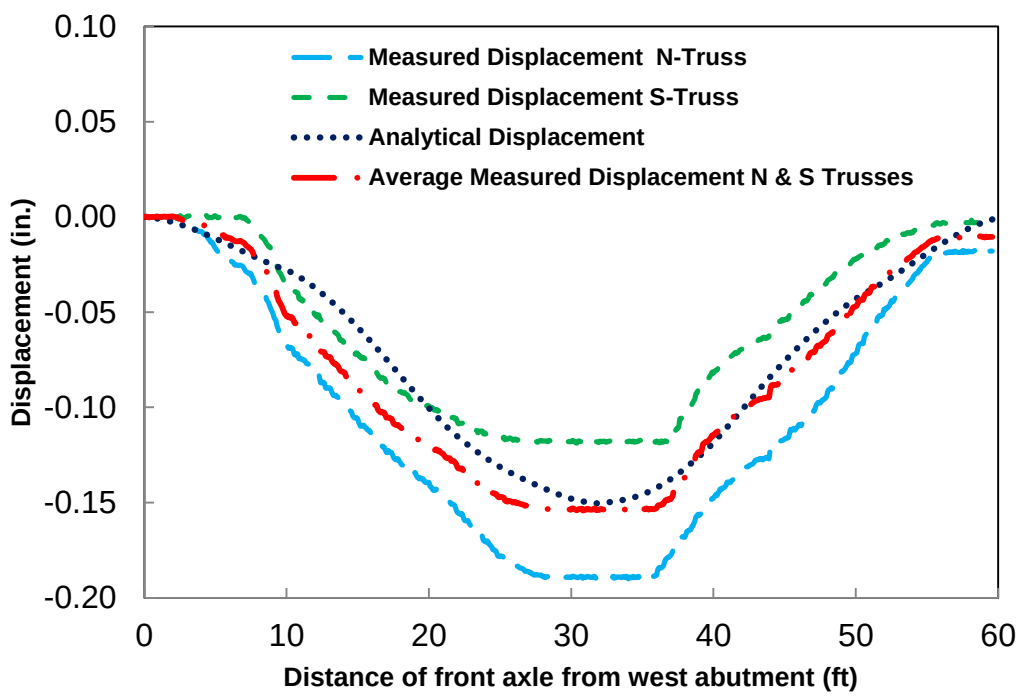


Fig. 8 Field measured and STAAD analytical displacement at mid-span

4.2 Displacements the Vicinity of the Splice Joint 1

Figure 9 illustrates the displaced shape of the bridge as the rear axle of the large truck was positioned on floor beam B4, just to the right of splice joint 1 (see Fig. 5). The figure shows the localized effects of the splice joints. As can be noticed, there is a discontinuity in the displaced shape of the bottom chord spliced member in the vicinity of splice joint 1. In addition, it can be noticed in Figure 9 that the two parts of the spliced bottom chord experience negative curvatures, i.e., they are concave downward. This indicates that the top surface of the two members to the left and right of the splice joint are subjected to tensile strains. However, one must realize that the degree of this curvature depends largely on the ability of the splice joint to transfer moment from one side to the other. Similar behavior was verified by the recorded field strain data. The field data indicated that the tensile strains in the top face of the bottom chord that were recorded by sensor 6084 was several times larger than the strains recorded by sensor 1697 on the bottom of the chord (see Fig. 6).

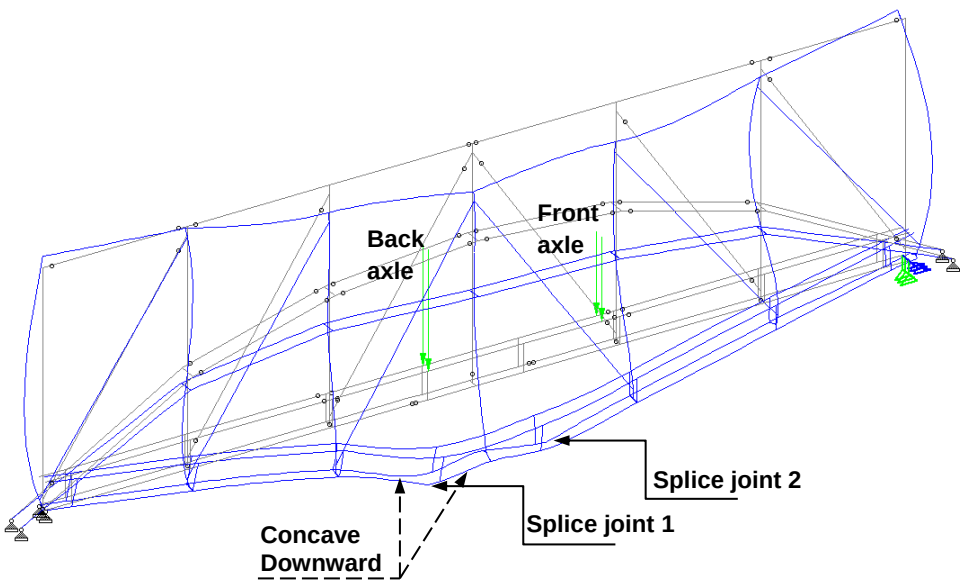


Fig. 9 Deflected shape of the Zacke Cox Bridge – large truck front axle to the right of splice joint 1

4.3 Comparison of Analytical and Field Strains

Figures 10 shows the measured and analytical strains at the top and bottom of the diagonal members located to the right and left of the middle vertical member V4. A small icon showing the location for which the strains were recorded is shown on the figures.

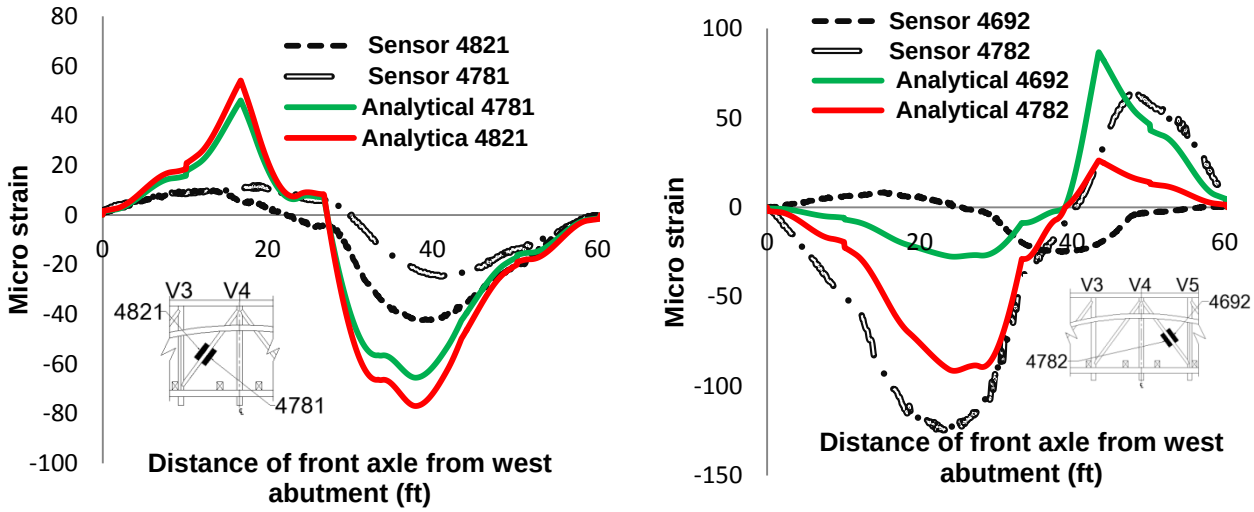


Fig. 10 Analytical and field strains in the diagonal just to the left and right of the middle post of the north truss

Fig. 11 Connection between V4 and the two diagonal members at the center span of the bridge

The differences in the strain magnitudes in the figures above are likely due to the inherent eccentricity within the joints at both ends of these members. The photo in Fig 11 shows a connection detail between vertical member V4 and the two diagonal members at the top chord panel point. Also, Fig. 11 shows that a gap exists between the vertical member V4 and the diagonal member located between vertical V4 and V5 resulting in an additional eccentricity at the joint. The eccentricity at these connections will induce bending moments at each end of these diagonal members. To include this type of detail in a finite element model, one needs to use 3-D solid modeling. Therefore, it was necessary to utilize beam-column strength of material formula to include the effects of the axial force and the bending moments that were induced by the eccentricity at each end of the diagonal member. This was accomplished using the aid of an Excel spread sheet to calculate the stresses and strains at the locations of the strain sensors.

The measured and the analytical strains in the bottom chord in the vicinity of splice joint 1 are shown in Fig. 12. These strains were measured in the continuous member of the truss bottom chord. The figure illustrates the good agreement between the analytical and the field strains. However, one may notice that the figure shows a sudden change in the analytical strains at the locations marked with solid squares. This could have resulted from the idealization used to transfer the truck load to the truss bottom chord. Factors that might have resulted in the differences between the measured and the analytical strains are summarized in the following

section. Additional comparison between measured and calculated strains in other members is summarized in [3]

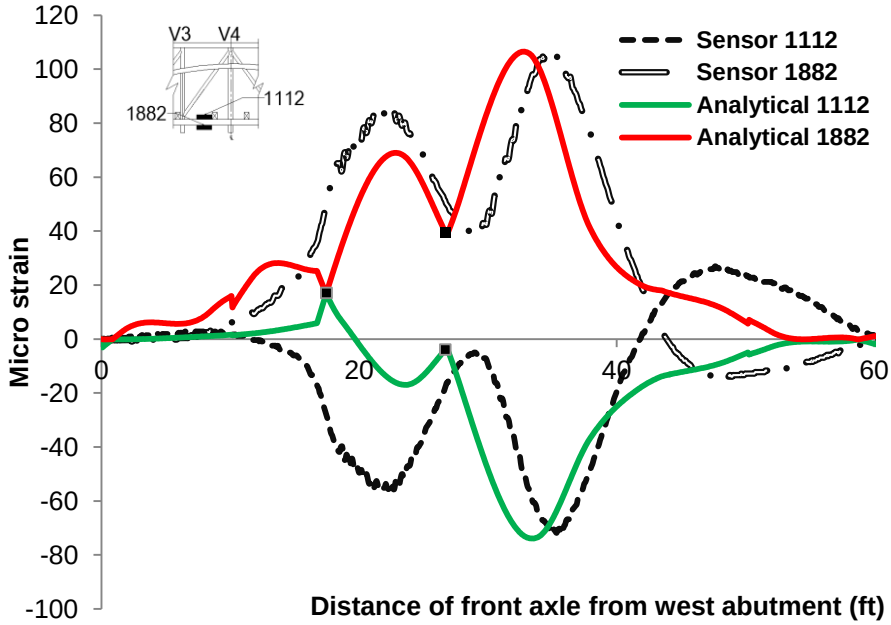


Fig. 12 Analytical and field strains in the vicinity of splice joint 1 of the north truss

5. Summary and Conclusions

This paper summarizes an investigation that was sponsored by the Federal Highway Administration and the USDA Forest Products Laboratory to develop a simple but accurate analytical model to analyze Burr Arch Truss bridges. A three dimensional model that included the splice joints that were used in the construction of the bridge was utilized in the analysis. The results of the analytical model were validated with the data obtained from the field testing of the same bridge structure.

The finite element results showed that idealizing the truss-arch structure excluding the eccentric in the connection between the vertical and diagonals, top chord and bottom chord members may yield a more stiff structure. In addition, the analytical model showed that the arch contributed to the load carrying capacity of the bridge.

The strains obtained using the analytical model yielded very reasonable strain results in the continuous bottom chord member in the vicinity of the splice joint. However, one must take into account the effect of the inherent eccentricity within the joints at both ends of members of the bridge structure and the actual structural properties of the splice joints. The later can be obtained from conducting laboratory and field tests on the different types of joints that are used in different types of historical timber bridges.

6. Acknowledgement

The study described in this manuscript was sponsored by the FHWA and the USDA Forest Products Laboratory (FPL) under the National Historic Covered Bridge Preservation Program.

The authors wish to thank Mike Ritter, Assistant Director, USDA FPL, for his cooperation and assistance during the course of the work. Special thanks to Allison Lunde (Machtemes) for her assistance in performing some of the finite element analysis presented in this paper and to Douglas Wood, ISU Structural Engineering Laboratory Manager, for his assistance with data interpretation and conduct of the testing.

7. Bibliography

[1] Bridge Diagnostics, Inc., BDI. 1989. Bridge Testing, Load Testing & Structural Monitoring Solutions, Boulder, CO

[2] Clark Dietz. 2009. Parke County Bridge No. 63 Special Feature Bridge Inspection. Indianapolis, IN: Clark Dietz, Inc. Print.

[3] Fanous, F., et. al., Improved Analytical Techniques for Historical Covered Bridges, Report submitted to Forest Product Laboratory, January 2013.

[4] Forest Products Laboratory.1990. Second Edition. Wood Engineering Handbook. Gen. Tech. Rep. Madison, WI: U. S. Department of Agriculture, Forest Service, Forest Products Laboratory. 4-13 p.

[5] "History of Covered Bridges." 2010. Welcome to the GDOT. Georgia Department of Transportation.<http://www.dot.state.ga.us/travelingingeorgia/bridges/coveredbridges/Pages/HistoryofCoveredBridges.aspx>.

[6] Hosteng, T., et. al., Covered Bridge Rating through Load Testing, Report submitted to Forest Product Laboratory, January 2013.

[7] Parke County, IN. Covered Bridges. 2001. Zacke Cox Covered Bridge. <http://www.coveredbridges.com/index.php/poi_detail?poiID=45>.

[8] STAAD Pro, 2011. Structural Analysis and Design, Revision 8.0: Bentley Systems, Exton, PA. USA.

[9] US Department of Transportation Federal Highway Administration. 2005. Covered Bridge Manual. Rep. no. FHWA-HRT-04-098. McLean, VA: Federal Highway Administration, Print.

Assessment and monitoring of the moisture content of timber bridges

Bettina FRANKE
Dr.-Ing.
Bern University of Applied Sciences, Biel, Switzerland

Robert WIDMANN
Research Associate
Empa
Dübendorf, Switzerland

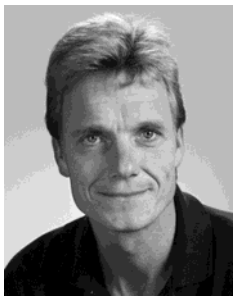
Andreas MÜLLER
Professor
Bern University of Applied Sciences, Biel, Switzerland

Thomas TANNERT
Assistant Professor
University of British Columbia, Canada

Bettina Franke

Civil Engineer and Doctoral degree in Timber structures

Research Associate at Bauhaus-University of Weimar, Germany, University of Auckland, New Zealand and since 2011 Bern University of Applied Sciences, Switzerland



Robert Widmann

Civil Engineer

Research Associate at Empa Swiss Federal Laboratories for Materials Science and Technology Structural Engineering Research Laboratory.

Field of work:
Timber engineering

Summary

Timber bridges are subjected to climate changes which result in the change of the moisture content of the wood and thus change of dimensions. But what happens with the wooden material due to the climate changes? Therefore four timber road bridges are long term monitored. In all cases, the moisture content in the wooden members was measured and analysed in relation to the prevalent climate. For the analyses of the distribution of the moisture content over the cross section or along the span, different sensors were positioned. The results confirm that the wood moisture content follows the climate changes. The reaction is delayed and with less variation compared to the calculated equilibrium moisture content. It was observed that the change of the moisture content differs over the cross section and along the span of the members.

Keywords: Timber Bridge, Monitoring, Moisture content.

1. Introduction

There are many historical timber bridge constructions around the world but also recently erected timber bridges in Europe and especially in Switzerland. Because of the hygroscopic material behaviour of wood, climate changes result in the change of the moisture content of the wood and thus change of dimensions as well (swelling and shrinking). For a reliable assessment, the material behaviour of wood due to the climate changes within a year or over the lifetime of the bridge has to be known. The changes of the moisture content have to be specified as well as the difference between the natural material axes or over the cross section of the structural members. The stress strain behaviour of wood under varying moisture content was mostly investigated in laboratory conditions, [1] - [4]. The investigations showed that there are differences in the moisture content over the cross section which leads to internal stress gradients. The research done by Häglund [5] - [8] showed that parallel to the absolute also the relative moisture changes and this influences the load capacity as well as the serviceability of timber structures. First investigations are also done for glulam and cross laminated timber, [9] - [15]. But the discussion for reliable approaches for the assessment or design is on-going and no recommendations have been published up to now.

Within a research project at the Bern University of Applied Sciences, four timber road bridges were long term monitored for two years and longer. The timber bridges are situated in different regions of Switzerland with various climate conditions. In all cases, the moisture content in the wooden

member is measured in relation to the prevalent climate. The paper presents the measuring equipment used and the results of the moisture contents measured. For a comparison between the different measuring groups and positions, sensors were positioned close to the surface and further inside of the cross section. Furthermore sensor groups are located close to the end grain of the timber members as well as in a certain distance to the end grain or at the mid span in order to evaluate the distribution of the moisture content in parallel to grain direction.

2. Material and Method

2.1 Hygroscopic behaviour of the material

The hygroscopic behaviour of wood describes the adsorption and desorption of moisture to maintain equilibrium depending on the surrounding climate in particular relative humidity and temperature. The adsorption of moisture occurs in two steps in the range from 0 % to 30 % where the moisture is transferred into the cell walls of the wood. Above 30 % moisture content, the cell walls are completely saturated and the moisture is transferred into the cavities of the cells. The moisture content of 28 - 30 % is called fibre saturation point. The fibre saturation point varies depending on the wood species. Changes in the moisture content below the fibre saturation point affect the physical, mechanical and rheological properties of wood, like the shrinking and swelling, the strength values or the modulus of elasticity or rigidity, [16], [17]. The dimension changes are different in the three material axes (longitudinal, tangential or radial) as principle shown in Fig. 1.

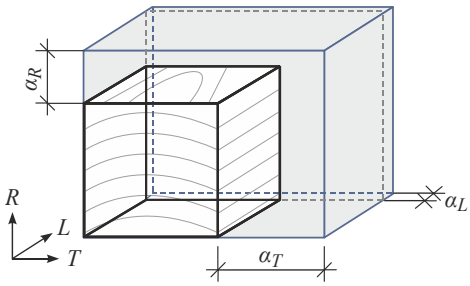


Fig. 1 Differential shrinking or swelling depending on the material direction

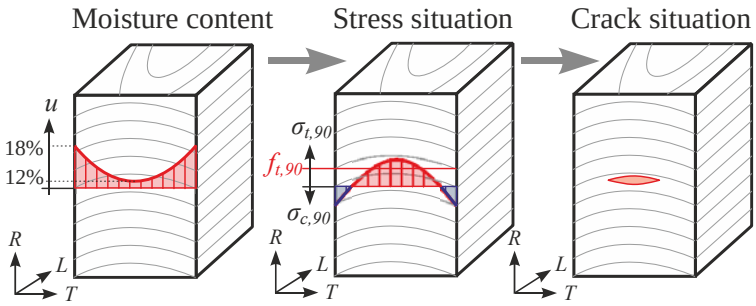


Fig. 2 Gradual increase of the moisture content and the stress reaction respectively crack growth

To reduce the initial change of moisture content in wooden members, they should be conditioned in such way that they meet the average moisture content which is expected in service. However, glulam and also block-glued glulam is been produced with a moisture content of 8 % to 15 % according to EN 386:2001 and will then mostly be installed with this moisture content, but the moisture content in service can be much higher depending on the application. Members in bridges in normal European climate conditions are expected to have a moisture content of around 15 % to 20 %, [18]. The moisture content within service class 2, for example, is allowed to vary of 8 % according to EN 1995-1-1:2004. It has to be noticed that a gradual increase of the moisture content of less than 1 % moisture content can theoretically already lead to excess of the material strength perpendicular to the grain (applying the characteristic properties for GL 24h). A gradual increase or decrease of the moisture content happens easily within the constructions and results in dimensional changes of the cross section and also in internal stresses, as shown in Fig. 2.

2.2 Bridges

The Bern University of Applied Sciences, the department of Architecture, Wood and Civil Engineering has been long term monitored four timber road bridges in Switzerland. The timber bridges are situated in different local regions with various climate conditions. The bridge “Horen” was erected in June 2008 and spans in total over 31 meters. The structural system consists of two main members of block-glued glulam four times supported and with secondary members spanning across for the deck construction, as shown in Fig. 3. The second bridge “Muotathal” is an arch bridge of glulam spanning a length of 32 meters, as shown in Fig. 4. The beam bridge “Obermatt” spans also over 32 meters. It is built with two main members with crossing secondary members, as shown in Fig. 5. The fourth bridge “Schachenhaus” is a timber concrete composite bridge, as shown

in Fig. 6, which spans over 20.4 meters. The characteristic and details of the measuring equipment for each road bridge are summarized in Table 1.

2.3 Measuring the moisture content

The measuring of the moisture content can be done directly by the oven drying process in the laboratory of wood samples taken from the structure (destructive), or indirectly using moisture meters (non-destructive). Moisture meters work with the relation of the moisture content to certain physical properties like e.g. the electrical resistance, capacity or microwaves. Moisture meters are commonly used in praxis and also for long term measuring. For the investigation of the bridges, the

Fig. 3 Bridge Horen

Fig. 4 Arch bridge Muotathal

Fig. 5 Bridge Obermatt

Fig. 6 Bridge Schachenhaus

Table 1: Monitoring details of the timber road bridges

Bridge	Erection	Characteristics	Measuring period/ Measuring rate/ Measuring system	Measuring data, system
Horen	2008	Beam bridge Spruce Glulam Block glued	since Oct 2009 every 6 hours local system Mulitsensor + Materialfox	20 moisture content sensors 1 air temperature sensor 1 relative air humidity sensor
Muotathal	2009	Arch bridge Spruce Glulam Block glued	Oct 2009 - Dec 2011 every 6 hours local system Mulitsensor + Materialfox	16 moisture content sensors 4 wood temperature sensors 2 air temperature sensors 2 relative air humidity sensors
Obermatt	2007/2008	Beam bridge Spruce Glulam	since Dec 2010 every 6 hours remote system Gigamodul + Thermofox	16 moisture content sensors 4 wood temperature sensors 2 air temperature sensors 2 relative air humidity sensors
Schachenhaus	2000	Timber-concrete composite bridge	since Mar 2011 every 6 hours local system Gigamodul + Thermofox	8 moisture content sensors 2 wood temperature sensors 1 air temperature sensor 1 relative air humidity sensor

electrical resistance measurement method is used in combination with local data loggers or remote systems. Chrome steel screws with insulated shanks are used as electrodes, as shown in Fig. 7. Screws with certain lengths are applied in predrilled holes to measure the moisture content in different member depths. One measuring position for the electrical resistance always consists of one pair of screws with a distance of 32 mm between them. The orientation of the screws per pair is perpendicular to grain as known for the use of common moisture meters. The measuring result, the electrical resistance, is logged by commercial data loggers Gigamodul or Multisensor from Scanntronik Mugrauer GmbH Germany. After collecting the data, the moisture content is then calculated by the relation to the electrical resistance and corrected depending on the temperature.

In addition to the measuring of the moisture content the theoretical equilibrium moisture content of the wood is calculated based on the isotherms of sorption for spruce, the Keylwerth-Diagram [19].

As an example, the measuring plan for the Obermatt bridge is shown in Fig. 8. The measuring points are positioned at both ends symmetrically to the span of the bridge. They are grouped to measure the air temperature (LT_S) and relative air humidity (LF_S) as well as the moisture content at the surface (HF_S_OF) or inside the structure with a depth of 200 mm (HF_S_T). Furthermore two temperature sensors (HF_S_T) for the temperature correction are installed next to the moisture sensor groups. For long term monitoring, a remote data transmission system was installed in cooperation with the company mageba SA, [18], [20]. The data is collected every 6 hours and remotely transmitted every week. This provides a sufficient period to react when extensive changes in the moisture contents happen, e.g. because of any damages or defects in the construction.

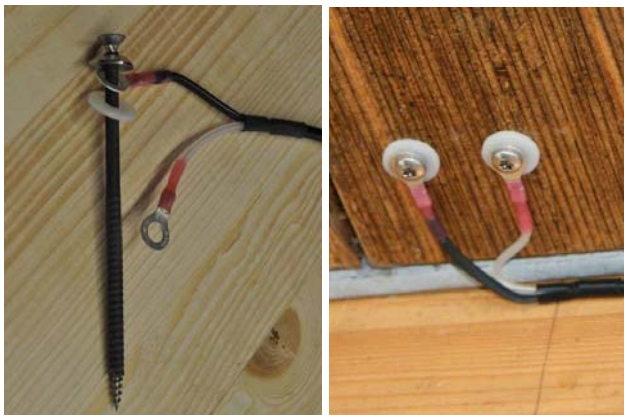


Fig. 7 Electrodes used with moister meter, uninstalled (left), and installed (right)

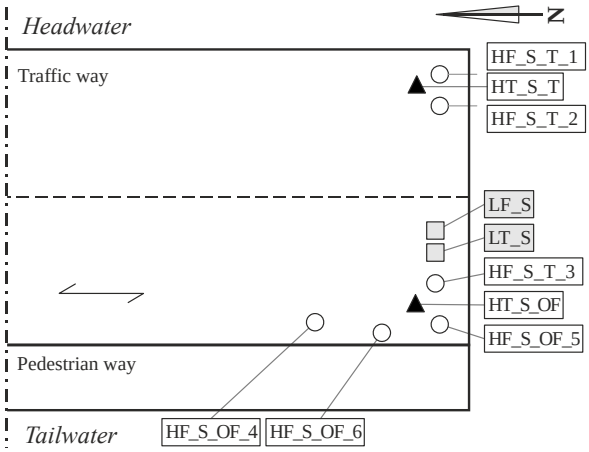


Fig. 8 Layout of measuring points including sensor names, south side of the Obermatt bridge

3. Results and Discussion

3.1 Climate data and moisture content

For the Obermatt bridge, the measuring results of the climate and moisture contents are shown in Fig. 9 to Fig. 11 for a period of 25 months. The climate data observed includes temperature and relative humidity of the air. The seasons of the year can be clearly distinguished, for the winter with low temperatures from -5 to 5 °C and high relative humidity, for the summer with temperatures of 15 to 20 °C and the two transition seasons spring and fall with the increase respectively decrease of the temperature and the relative humidity reversely. The equilibrium moisture content calculated is added in Fig. 9. The moisture content results theoretically between 10 % in summer and 32 % in winter.

In general, the moisture content measured in the timber follows the seasonal effective climate changes. The response is delayed and with lower variations for both sensor locations at the surfaces and inside. The variation of the moisture content at the surface between the summer and winter period is practically of about 5.5 % where theoretically the calculated equilibrium changes of 22 %,

as shown in Fig. 10. The moisture content varies between about 14 and 20 %. The phase shift between the theoretical calculated equilibrium and measured moisture content is about 2 to 3 months depending on the gradient of climate change and the phase of adsorption or desorption.

The moisture sensor results measured in a depth of 200 mm show a smaller seasonal variation than the results of the surface sensors, Fig. 10 and Fig. 11. The curves of the moisture content at the inner structure are more evenly distributed and compact to each other, with a range of the variations of 2.5 %. The difference between the sensors at the surface and inside is around 3 % of moisture content in average independent of the season. The difference between the inner and outer moisture content of 3 % results in internal moisture induced stresses.

In general the measured moisture content did not exceed 20 %. The behaviour determined and shown for the bridge Obermatt could, in a similar range, also be observed for the other three bridges, [20].

3.2 Moisture transfer/distribution in the different material directions

With the measuring set up at the bridge Horen, the moisture transfer over the depth (in radial direction) and width (in tangential direction) was also observed. Fig. 12 shows the cross section of one block-glued glulam member with the positions of the sensors grouped in two measuring lines 755 and 1475 mm from the outside and 400 mm from the end grain. Each measuring group includes 5 sensors for the moisture content distributed over the beams depth. The distance between each

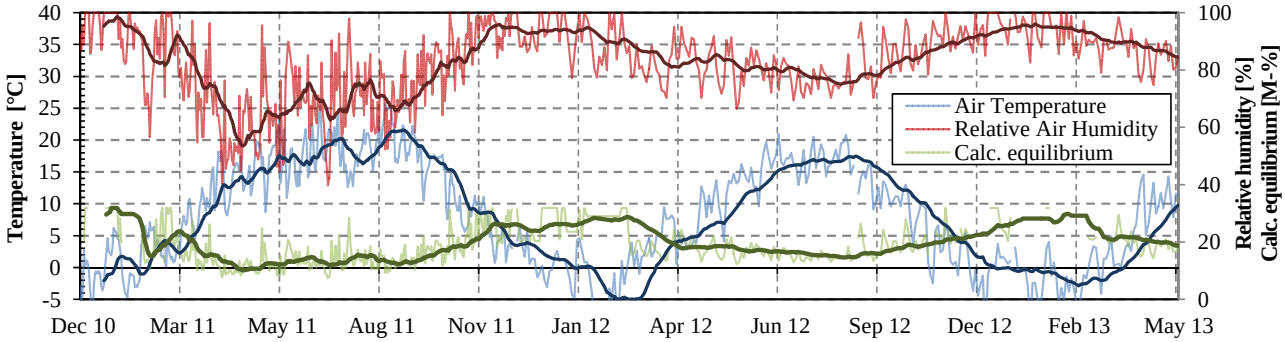


Fig. 9 Climate results and calculated equilibrium for the bridge Obermatt

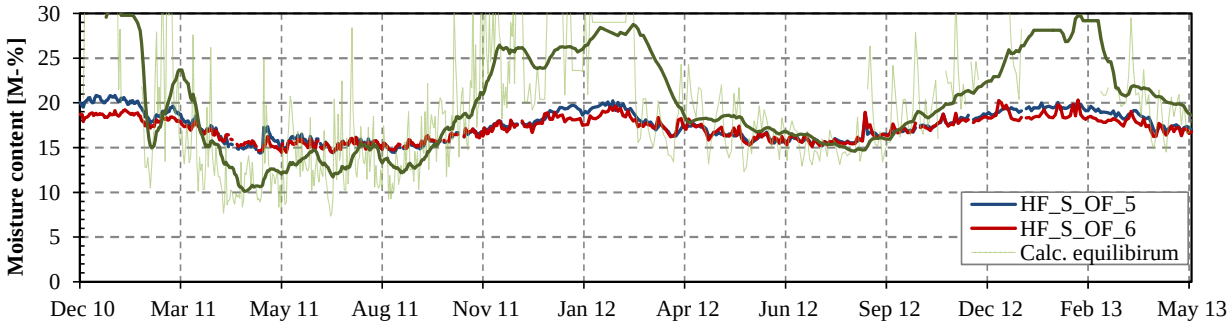


Fig. 10 Moisture content of the sensors at the surface

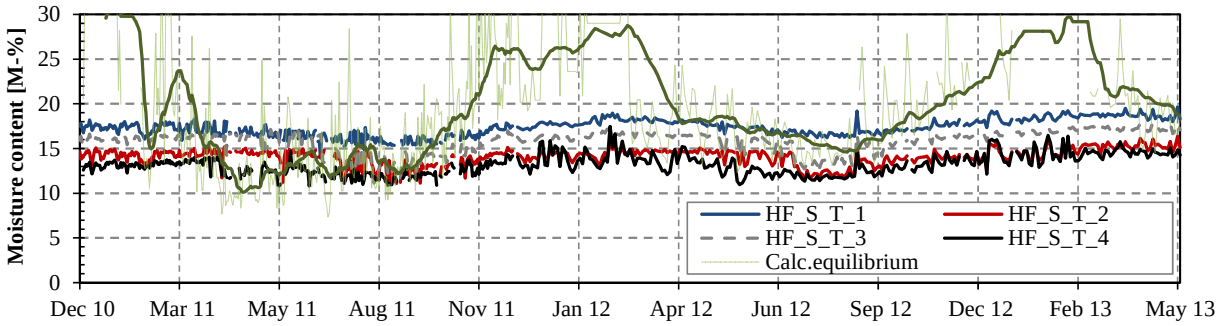


Fig. 11 Moisture content of the sensors in the inner structure, with a depth of 200 mm

other is around 100 mm. The moisture content in the wood was analysed for a period of 5 months beginning in May 2011. Within this period the increase of the mean relative humidity represents a theoretical increase of the moisture content from 12 to 27 %, as shown in Fig. 13.

However, the moisture content within both measuring groups, outside and inside, shows almost no differences in the variation due to the change of the outer climate, as shown in Fig. 14 and Fig. 15. The small changes are within the range of the measuring accuracy of around 1.5 %, [15]. Also, the construction at the positions of these sensors seems to be well protected against rain or other wet situations. Even wet conditions due to rain do not influence the moisture content for these measuring positions. Therefore the hazard of decay is minimized or can even be excluded.

For the investigation of the moisture transfer in longitudinal direction, the measuring results observed at the bridge Muotathal are used. The measuring setup is shown in Fig. 16, where the first measuring line is installed in longitudinal distance of 800 mm from the end grain and the second line with a distance of 2000 mm. The cross section is width x depth =1000 mm x 800 mm and the sensor groups were installed in a depth of 200 mm (tangential direction). Each measuring group includes 5 sensors for the moisture content. The vertical distance between each sensor is around 175 mm. The moisture content was analysed for a desorption period of almost 5 months beginning in December 2010. Within this period the decrease of the mean relative humidity represents a theoretically decrease of the moisture content from 25 % to 9 %, as shown in Fig. 17.

The results show at both positions a decrease of the moisture content in the wood of about 2 %, as shown in Fig. 18 and Fig. 19. No major differences in longitudinal direction could be observed. The measuring position of the first group shows already a large distance from the end grain in order to

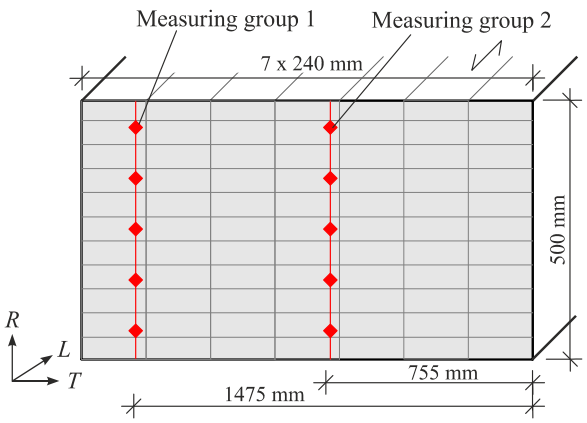


Fig. 12 Measuring plan of bridge Horen, cross section at the support

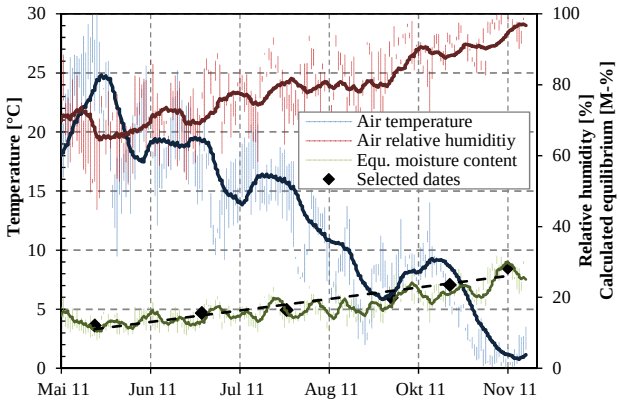


Fig. 13 Climate data and equilibrium moisture content of bridge Horen for a adsorption period

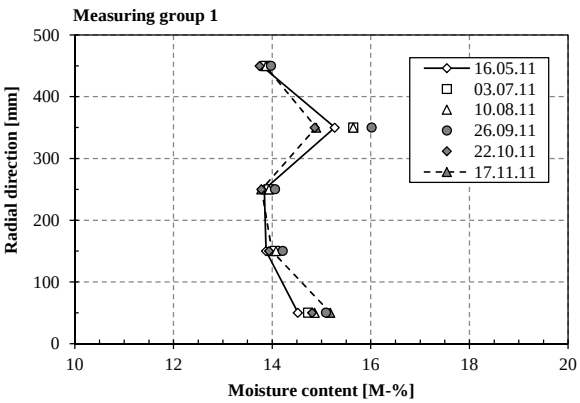


Fig. 14 Distribution of measured moisture content over the beams depth at 1475 mm from the beams side

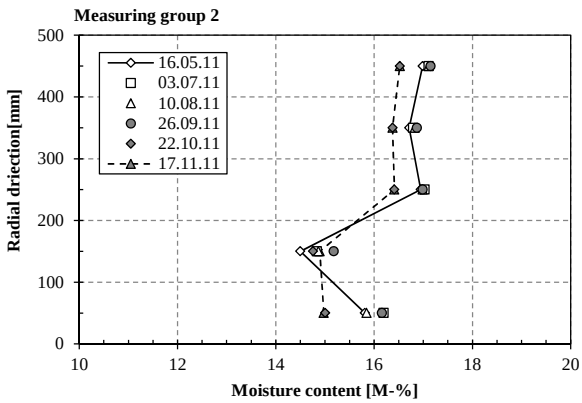


Fig. 15 Distribution of measured moisture content over the beams depth at 755 mm from the beams side

detect a dependency of the moisture content variations in longitudinal direction. The less influence of moisture adsorption/desorption from the end grain is in this case also reduced due to the steel plate as a part of the connection covering the end grain area and thus to a certain extent acting like a seal.

4. Conclusions and view

The electrical resistance measurement method used in the case studies is capable to determine the moisture content in the context of long term monitoring of timber structures. In case of construction deficiencies or later damages, the long term monitoring gives the possibility to observe extensive and unusual moisture accumulations at an early stage to avoid decay/fungal development. However, the monitoring can realistically not be done at every position of the construction, but could be done at important locations.

The monitoring of four timber road bridges and the analyses of their moisture contents confirm generally that the wood moisture contents follow the climate changes. The reaction against the calculated equilibrium moisture content due to the climate changes is delayed and with less variation depending on the distance to the surface. It could be shown that the moisture content in timber members subjected to the shown outdoor climate conditions varies between about 12 % and 22 %, which is below the critical moisture content of about 25 % for decay hazards. But a well-planned and applied structural protection of the wood against rain or snow is required, so that a general penetration of moisture into the timber members/construction is prevented.

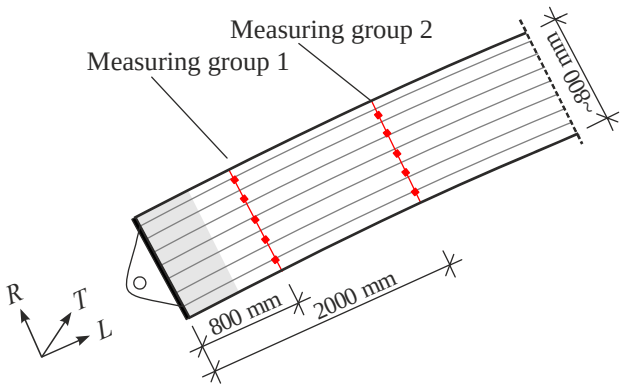


Fig. 16 Measuring plan of arch bridge Muotathal

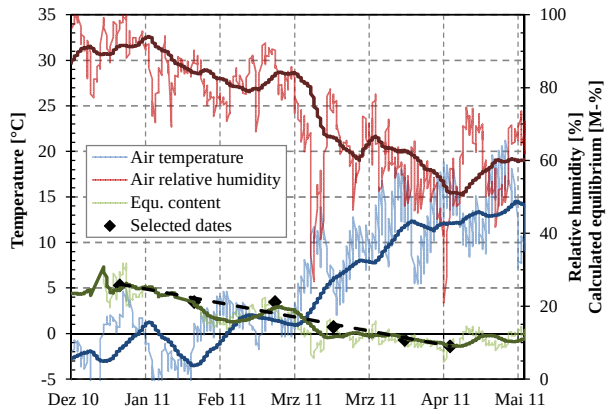


Fig. 17 Climate data and equilibrium moistore content of bridge Muotathal for a desorption period

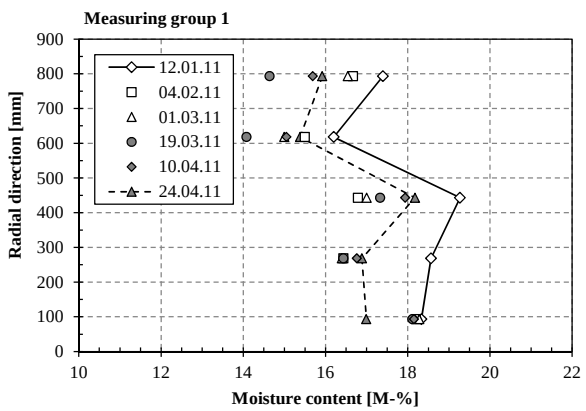


Fig. 18 Distribution of moisture content over the beams depth in a distance of 800 mm from the beams end

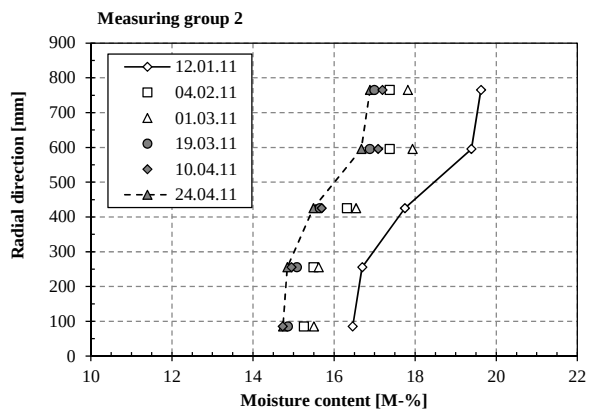


Fig. 19 Distribution of moisture content over the beams depth in a distance of 2000 mm from the beams end

In first steps, the behaviour of the moisture content could be determined over the cross section of block glued glulam members in radial and tangential direction and along the span of the member in longitudinal direction. In the cases observed, no major differences could be detected between the positions measuring sensors because of too large distances from the surface or the end grain. For a prediction model, the sensors should be positioned in a closer range for specification of differences in material axes and also investigated in specified and controlled climate changes.

The measuring results observed show that it is important to respect the moisture of the wood according to the climate change. Especially close to the end grain, cross section and in the outer glulam members of block-glued members the moisture gradients observed can lead to moisture induced stresses and furthermore to internal and/or external cracks.

Acknowledgment

The project work was supported by the Federal Office for the Environment (FOEN) and the project partners Makiol + Wiederkehr, Hüsser Holzleimbau AG, Decentlab GmbH and the Cantonal Civil Engineering Department Bern. Thank you for the support.

References

[1] McMillen, J. M., “Drying stresses in red oak.” *Forest Products Journal*, Vol. 5, 1955, pp. 71-76.

[2] Toratti, T., “Moisture induced stresses”, Presentation at Cost E55 meeting Trondheim (2009)

[3] Svensson, S., Toratti, T., “Mechanical response of wood perpendicular to grain when subjected to changes of humidity.” *Wood Science & Technology*, Vol. 36, 2002, pp. 145-156.

[4] Svensson, S., Martensson, A., “Simulation of drying stresses in wood - I: Comparison between one and two dimensional models.”, *Holz als Roh- und Werkstoff*, Vol. 57, 1999, pp.129-136.

[5] Häglund, M., “Moisture content penetration in wood elements under varying boundary conditions.”, *Wood SciTechnol*, Vol. 41, 2005, pp. 477–490.

[6] Häglund, M., Thelandersson, S., “Consideration of moisture exposure of timber structures as an action”, In Proceedings CIB-W18/38-11-2, Karlsruhe, Germany, 2005.

[7] Häglund, M., “Varying moisture content penetration in wood elements under varying boundary conditions.”, *Wood Science & Technology*, Vol. 41, 2007, pp. 477-490.

[8] Häglund, M., “Varying moisture content penetration in wood elements under varying boundary conditions.”, *Wood material science and engineering*, Vol. 3, 2008, pp. 38-45.

[9] Gereke, T., Niemz, P., “Moisture-induced stresses in spruce cross-laminates.” *Engineering Structures*, Vol. 32, 2010, pp. 600-606.

[10] Gereke, T., “Moisture-Induced Stresses In Cross-Laminated Wood Panels.”, Doctoral thesis, ETH Zurich, Switzerland, 2009.

[11] Angst, V., Malo, K. A., “The effect of climate variations on glulam-an experimental study.”, *European Journal of Wood and Wood Products*, Vol. 70/ 5, 2012, pp. 603-613.

[12] Angst-Nicollier, V., “Moisture Induced Stresses in Glulam.” Doctoral thesis, Norwegian University of Science and Technology, Trondheim, 2012.

[13] Tannert, T., Müller, A., Vogel, M., “Structural health monitoring of timber bridges.”, International Conference on Timber Bridges, Lillehammer, Norway, 2010.

[14] Frese, M., „Wechselwirkung zwischen der Anisotropie der Schwind- und Quellmasse sowie der Holzfeuchte-Änderung in der Ebene von gekrümmtem Brettschichtholz.“, *European Journal Wood and Product*, Vol. 69, 2011, pp. 259-367.

[15] Winter, S., Sieder, M., Gamper, A., Dietsch, P., Merk, M., “Gebäudeklima - Langzeitmessung zur Bestimmung der Auswirkungen auf Feuchtegradienten in Holzbauteilen,”, Research report, Technical University Munich, Germany, 2012.

[16] Neuhaus, F. H., „Elastizitätszahlen von Fichtenholz in Abhängigkeit von der Holzfeuchtigkeit,“, Technical Reports, Ruhr-University of Bochum, Germany, 1981.

[17] Niemz, P., „*Physik des Holzes und der Holzwerkstoffe.*“, DRW-Verlag, Leinfelden-Echterdingen, Germany, 1993.

[18] Scharmacher, F., Müller, A., “Erfahrungen und Konsequenzen aus der Langzeitüberwachung von Holzbrücken.”, Bridge Symposium – Grünbrücken aus Holz, Stuttgart, Germany, 2012.

[19] Keylwerth, R., Noack, D., “Die Kammertrocknung von Schnittholz.”, *Holz als Roh- und Werkstoff*, Vol. 22, 1964, 29-36.

[20] Franke, B., Müller, A., Vogel, M., Tannert, T., “Langzeitmessung der Holzfeuchte und Dimensionsänderung an Brücken aus blockverleimten Brettschichtholz.”, Research report, Bern University of Applied Sciences, Biel, Switzerland, 2012.

[21] EN 386:2001, Glued laminated timber – Performance requirements and minimum production requirements, European Committee for standardization, Brussels, Belgium, 2001.

Case Study: Design of pedestrian timber bridges in an AE Studio

Mikhail Gershfeld, SE
Prof. Practice Professor
CE Department
Cal Poly University
Pomona, CA 91768
mgershfeld@csupomona.edu

Expertise in design of wood structures with over 30 years combined experience in structural design, engineering and manufacturing management and engineering education. Chair ASCE Wood Education Committee, Chair Wood Education Institute.

Judith Sheine, RA
Professor and Dept. Head
Department of Architecture
University of Oregon
Eugene, OR 97403-1206
jesheine@uoregon.edu

Expertise in architectural design and studio teaching with over 30 years in architecture and architectural education.

Summary

California State Polytechnic University, Pomona recently approved an interdisciplinary design course (AE Studio) in which architecture and engineering students work in teams to design a pedestrian timber bridge. The course is structured as a progressive competition and the project sites are selected based on real needs. For the first three years of the course, the project site was on the Cal Poly Pomona campus. For the past two years, the project sites were located in the Angeles National Forest at locations in which pedestrian river crossings were needed. In one case the bridge improved access from parking and day-use facilities to a hiking and biking trail; on the second site the bridge provided an alternative to fording a river to continue on a scenic hiking path. Both project sites were selected through collaboration with the local U.S. Forest Service engineering office based on their future needs and opportunities for funding. In this paper the architecture and engineering faculty who developed this course share selected works of their students and discuss their pedagogical experiences in teaching this type of course, including a discussion of the benefits of using timber as a material and the pedestrian bridge as a type of structure for this kind of class.

Keywords: timber, bridges, pedestrian, education, studio, wood, architecture, engineering, collaboration

1. Introduction

Wood is one of the oldest and most environmentally sustainable construction materials. Approximately 90% (U.S. Department of Housing and Urban Development, 1994)[1] of all residential and 11% of non-residential (USDA Forest Service, 2008) [2] structures in the United States are built using sawn lumber and engineered wood products. These modern engineered wood systems require specialized design and materials specification knowledge. However, despite the need for education in this area, a significant number of civil engineering programs do not offer timber design at all, or offer it only sporadically, or as part of another course. A recent survey (Cramer, 2011)[3] indicated that just slightly over 50% of civil engineering programs offer a wood design course.

In architectural programs, the use of wood as a structural material, other than light wood framing in residential projects of four stories or less, is not common. This can be traced to a limited understanding of engineered wood as a structural material and the perception that it lacks durability and flexibility in form and has low fire resistance qualities. However, recent conservation efforts and sustainability concerns have appropriately elevated wood as the only sustainable structural material. Advances in engineered wood products development and product enhancements have renewed interest in non-residential uses of wood as well as in the use of mass timber in mid- and high-rise residential buildings. Engineered wood is being successfully used for multi-story construction, large span structures and pedestrian and vehicular bridges.

The lack of knowledge of a material can often cause reluctance to use it by the design professionals, or, when used, result in improper use and subsequent disappointment with its performance. It is important that architects and engineers are well educated in the strengths and weaknesses of materials and work collaboratively to ensure their proper and effective implementation. Most, if not all, of the learning related to collaboration between the two professions currently occurs in the workplace. It would be valuable to provide engineering and architecture students with an educational experience that will launch them on the path of mutually beneficial collaboration as they enter the professions.

This interdisciplinary design course (AE studio) provides an educational experience in AE collaboration, the design of long span structures and the creative use of wood as a structural material. The choice of a pedestrian timber bridge for this type of course was made for several reasons: (1) it requires close collaboration between architects and engineers from the beginning of the design process; (2) pedestrian bridge loading is simpler and lighter than that for vehicular bridges; (3) the use of wood required the students to understand its material and structural properties and their impact on the forms they chose to develop. The AE Studio is an exceptional educational tool that should be promoted and supported by the wood industry. The timber projects designed by the students during this course are presented in this paper in an effort to: (1) encourage collaboration between architects and engineers during the educational process; (2) demonstrate the creative designs of pedestrian bridges using wood as a structural material and; (3) involve the community of design professionals and the building industry in the educational process.

2. AE Collaboration

The idea of a collaborative relationship between architectural and civil engineering programs is by no means a new concept, but its implementation in undergraduate education is certainly not trivial. It requires engineering faculty with a level of expertise different from that of a traditional academically trained faculty member, an educational methodology that works for both disciplines and the selection of projects that are responsive to the educational objectives. Cal Poly Pomona presently offers civil engineering degrees within the College of Engineering and architecture degrees within the College of Environmental Design. The emphasis on structural engineering is typically addressed through technical electives in civil engineering. The AE Studio (CE471 – Interdisciplinary Architectural Engineering Design) is one such elective.

3. Course Structure

The CE 471/471L is a 3/1-unit 10-week course divided into three phases. The first phase is two weeks long and is devoted to case studies of existing timber pedestrian bridges by inter-disciplinary teams of architects and engineers and to the development of individual design concepts for the assigned project by architects, with input from engineers. At the end of the first phase the students present their case studies and their design concepts, and around eight or ten concepts are selected and approved by the faculty for the next phase of the course. At this time, AE teams are formed to develop preliminary designs of the selected concepts, which they work on for three weeks. At the end of this second phase, the AE teams present their designs to a panel of practicing engineers and architects. Each project is rated on ten criteria (Gershfeld, Sheine and McGavin 2011) [4] and four or five projects are selected to proceed to the third phase. The selection is made through a combination of the panel's objective scoring and the faculty's subjective input. The teams are expanded by absorbing students whose projects did not make it into the next phases. The final phase of the project is five weeks long and involves design development work. The teams develop ¼"=1'-0" scale models of their designs, produce renderings, animations, structural models, structural calculations, and details, build a 1"=1'-0" scale model of a section of their bridges (or, in some cases, a 3"=1'-0" scale model) and prepare posters and digital presentations. Their work is presented again to the panel of practicing architects and structural engineers, who again rank the projects based on ten criteria, and the winning design is selected based on the ratings of the professional panel. The ten criteria on which the projects were evaluated were aesthetic quality, originality, functionality, response to context, structural integrity, structural modeling, constructability, AE collaboration, cost effectiveness and finally, overall impression.

4. Student Projects

It is not possible to present the totality of the work generated by the AE students over the five years this course was taught. Thus, only selected works of the last three years of projects are presented: Winter 2010, 2011 and 2012.

4.1 Winter 2010: College of Engineering Bldg 9 and Bldg 17

The project site is located on the campus of California State Polytechnic University in Pomona (Figure 1).

Fig. 1 Project Site, California State Polytechnic University Pomona, College of

The project involved the design of a pedestrian bridge connecting two College of Engineering buildings at the second floor. The bridge had to: (1) accommodate pedestrian traffic; (2) provide social areas for students to congregate between classes; (3) serve as a learning tool for various civil engineering classes; (4) be sustainable; and (5) become a recognizable symbol of the College of Engineering. The distance between the buildings was 72 ft and a minimum of 14 ft vertical clearance and 25 ft horizontal clearance for emergency vehicle access was required between the supports. In addition, it was desirable to avoid transferring loads to existing buildings. Students performed their own survey of the site and prepared a site plan and a ¼”=1-0” scale site model. The structural design criteria for the project was established based on the latest IBC, ASHTO and ASCE 7 code requirements and were as follows: Live Load - 80 psf, Wind Load - 110 mph (3sec gust), Exposure B, Seismic Load: S_s and S_1 based on USGS Hazard Maps, Soil Class C. The engineering students were required, based on the initial architectural designs, to evaluate alternative structural systems, perform preliminary analysis (napkin level design) without the use of any structural analysis software, followed by simplified, but more detailed, 2D analysis; complete 3D computer analyses were performed for the final designs. In cases in which the design was too complex, early use of computers was allowed and in those in which the design was too simple, 3D computer analyses were not required (these were standard requirements for the course and will not be repeated for the projects presented from other years). The two projects presented here are, Space Truss Bridge and Ridge Bridge.

4.1.1 Space Truss Bridge

Fig. 2 Space Truss Bridge (1st Place), Architects: Nathan Houck, Greg Sagherian, Robert Yamnitz, Elane Yiu, Engineers: Bethany Lopez, Daniel Mourad, Ryan Turner, Samson Wong. Original design concept by Robert Yamnitz.

The winning design for this project was a space truss bridge. The rendering of the final design is shown in Figure 2. The bridge support structure is a space truss formed by three irregular triangles with their bases forming an upper triangle and opposite vertices providing bridge supports. The space truss provides a stable structural system and a symbolic representation of engineering. The bridge deck is suspended from the three vertices of the upper triangle in some places and supported by the legs of the triangles in others. The stringers that support the deck are suspended from the space truss vertices with steel cables. The bridge deck was designed using CLT panels to provide a slim profile and to develop the necessary horizontal stiffness. The final computer structural model of the bridge using

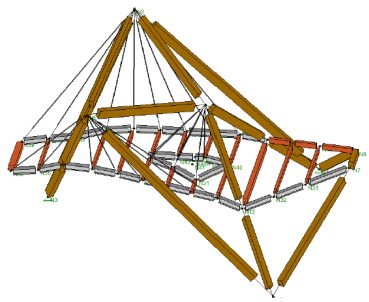


Fig. 3 Computer Structural Model of the Space Truss Bridge in RISA 3D

RISA 3D software is shown in Figure 3. All supports are modeled as pins, members as frame elements and connections as hinges. All components of the space truss are continuous members.

The members of the space truss were Alaska Yellow Cedar Glulam beams, a locally available naturally decay-resisting wood species, with the longest member approximately 50 ft in length. The unique angles of each joint of the space truss required the design of customized connections and each vertex was designed to accommodate steel hardware capable of transferring structural loading and providing aesthetically appealing details. The students explored a number of possible solutions to ensure that the connections were visually appealing and also provided acceptable load paths. The 10-week course allowed limited time for connection development and only the conceptual design of key connections was completed.

Fig.4 Deck stringers cable support connection

In designing the deck suspension system students had to address wood's weakness in resisting tension perpendicular-to-grain and designed connections to avoid this type of behavior by using steel straps around the stringers. (See Figure 4)
The bridge uplift issues were addressed by introducing a continuous stiffening beam above the deck along the perimeter of the bridge that also served as a bench. This project successfully addressed the project requirements and received the highest total score for all categories and the highest overall impression score from the panel of reviewing professionals; the faculty concurred with the professional panel.

4.1.2 Ridge Bridge

Fig. 5 The Ridge Bridge(2nd Place), Architects: Bridget Flecky, Eubie Han, Edward Kung Engineers: Gean Na, Alex Quinonez, Fernando Sesma. Original Design Concept by Eubie Han

The runner up project, the Ridge Bridge, was equally impressive. The continuous curved glulam beam supported by inclined glulam columns provided an aesthetically interesting support structure for a suspended bridge deck, conceptually similar to a ridge beam-rafter roof system. The bridge deck was suspended from the ridge line. The foundations supporting the sloped columns also served as sitting areas for students, thus creating a gathering space under the bridge. The detailed description is omitted and only an overall image of the design is provided to illustrate the aesthetic quality and creativity of the students' work.

4.2 Winter 2011: Angeles National Forest, West Fork Station, San Gabriel River

This project site was selected through discussion with the local U.S. Forest Service engineering office. The site had an existing parking lot and day-use facilities; however one of the most popular hiking trails was located on the other side of the river. Visitors used the nearby highway bridge to walk across the river, creating potentially dangerous situations for themselves and drivers, as the bridge was not designed to accommodate pedestrian traffic. The U.S. Forest service proposed to build a pedestrian bridge from the day-use facilities to the hiking trail in order to improve access to the trail and increase safety. The site plan of the proposed design (note the ramp) is shown in Figure 6. The U.S. Forest Services required that ADA standards were complied with for this bridge, since the trail quality, also serving as fire access from the main road, could accommodate handicapped persons. This constraint, due to the large difference in elevation between the day-use facilities site and the trail on the other side of the river, as well as insufficient space for visually appealing ramps, required a creative solution. The students were permitted to choose a bridge location that best accommodated their designs. The bridge spans, depending on the selected location, ranged from 110 to 140 ft. The winning design, Quadra Bridge, is presented here.

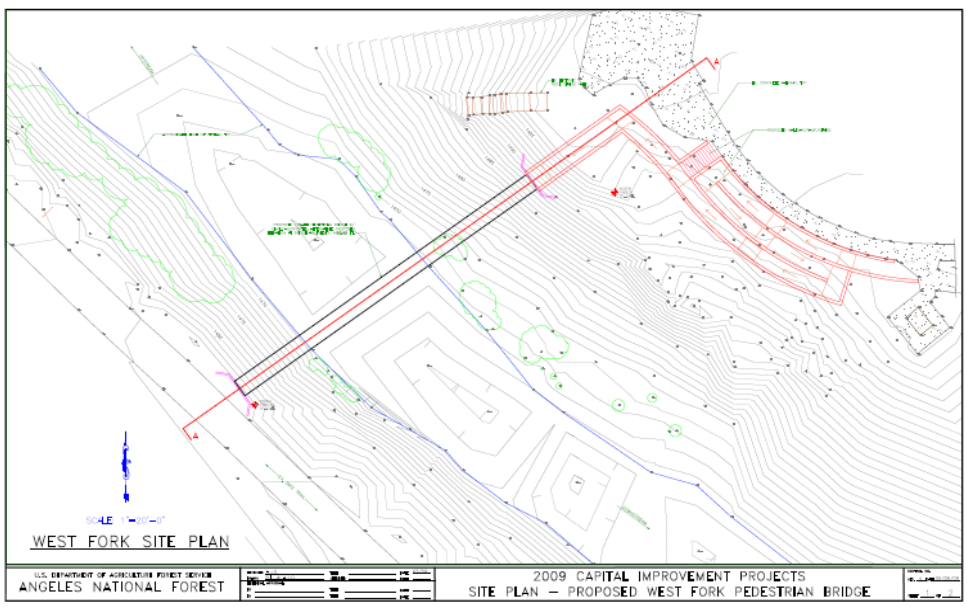


Fig. 6 Site plan and proposed bridge alignment for the site provided by the U.S. Forest Service

4.2.1 Quadra Bridge

The winning design for this project site was a 140 ft span bridge nicknamed Quadra Bridge. The design team addressed ADA requirements by providing a sufficiently long winding walkway to accommodate wheelchairs, sloping at 5%, the maximum allowed for a walkway that did not require intermediate landings. The portion of the walkway crossing the river was suspended from a 3D four-leg frame formed by the edges of the square pyramid. The two legs on each side of the river were joined together by steel structural tubing and, under the bridge deck, a chevron brace was provided as part of the lateral load resisting system. The ¼”=1-0” scale model of the bridge is shown in Figure 7.

Fig. 7 Quadra Bridge ¼"=1-0" scale model

This shape of the bridge support structure provided an aesthetically pleasing design and accommodated a very effective structural system. The perspective shows the bridge in the site context, including the access to the hiking trail from the day-use site and the aesthetics of the design (Figure 8).

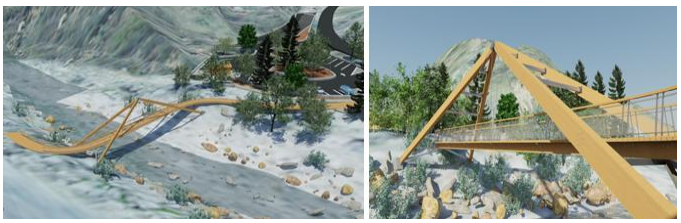


Fig 8 Quadra Bridge Rendering (1st Place) Architects: Maro Asipyan, Matthew Terry, Engineers: Christian Hainds, Francisco Perez, Bryan Strege. Original Design Concept by Mathew Terry.

The students also constructed a ½"=1'-0" scale model of the pyramid frame, shown in Figure 9 (while students typically constructed 1'=1'-0" details of the bridge, in this case it was determined that it made more sense to construct the entire frame at ½'=1'-0"). The legs of the pyramid structure were specified as glulam girders built from naturally decay-resistant species. The bridge deck was supported by coated steel cables connected to saddle straps wrapped around continuous wood girders and the deck assembly; the girders also acted as stiffening beams for uplift loading. Stainless steel structural tubing was used to connect the legs of the pyramid. The two-leg frames were intended to be constructed on each bank of the river and joined together to form the apex of the pyramid. The design of walkways leading to and from the bridge used concrete supports spaced along the river bank to accommodate the girder and decking assembly. The project design criteria was: LL=80 psf, Wind Load, 120 mph (3sec gust) wind, Exposure C and Seismic Loading based on USGS Hazard Map for this location and on geological information provided by the U.S. Forest Service. These design criteria were used by all teams for their bridge designs.

Fig. 9 Quadra Bridge ½"=1'-0" scale model showing support and apex connection

The SAP structural model of the bridge showing the layout of the main load resisting system and the distribution of

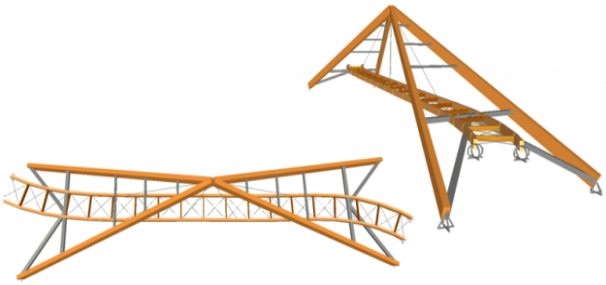


Fig. 9 SAP 3D Rendered Structural Model

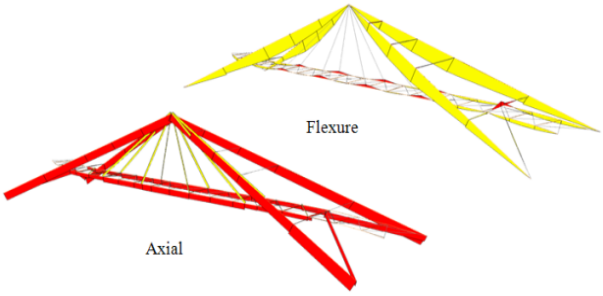


Fig . 11 SAP Structural Model Axial Load (in red) and Bending Moment (in yellow) Diagram

axial loads and bending moments throughout the bridge's main force resisting system due to vertical loading are shown in Figure 10 and Figure 11, respectively. The connection at the apex of the pyramid was designed to accommodate the construction sequence, structural requirements and also to be an aesthetically pleasing element of the bridge. The renderings shown in Figure 12 are the side view, top view and the conceptual representation of the cable to plate connection.

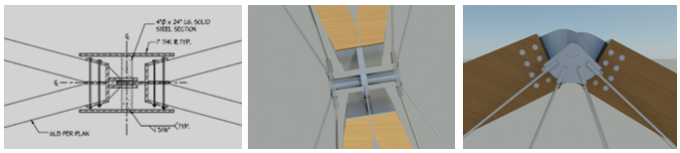


Fig. 10 Connection at the apex of the pyramid frame: engineering drawing, top view and rendering of the elevation,

The assembly of the bridge deck support connection consisted of a structural steel tube at deck level that resisted the compression load generated by sloped cables and a combination of steel rods and steel plates wrapping around the bottom of the beams resisting tension loads. The conceptual engineering detail of this connection is shown in Figure 13.

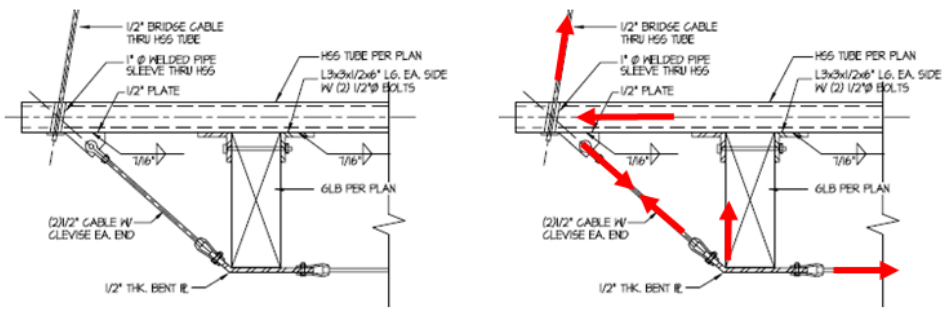


Fig.11 Bridge Deck Assembly saddle strap support conceptual connection detail and force diagram

This project earned the highest scores from the professional panel as well as from the faculty. It addressed in a simple and effective way the challenging architectural and engineering constraints of the project.

4.3 Winter 2012: Angeles National Forest, East Fork, San Gabriel River

Fig.12 Los Angeles Forest, East Fork, San Gabriel River, Bridge to Nowhere trail.

This project site was also identified by the U.S. Forest Service as needing a bridge. It was located along a very popular hiking trail called the Bridge to Nowhere (the trail terminates in a bridge designed as a link in a roadway that was not completed), where the U.S. Forest Service wanted an alternative to having hikers cross the river by jumping on rocks and logs. The proposed new bridge had to be visually appealing and contribute to the trail experience. The U.S. Forest Service did not require the students to design the bridge above the minimum flood level, but one important constraint was the ability to deliver materials to the

site. The available access was either by foot or helicopter delivery. Some of the photos of the site taken by students during the site visit are shown in Figure 14. A site plan was not available for this project, so students performed their own survey of the site, prepared a site plan and constructed a ¼" =1'-0" scale site model. The U.S. Forest Service did not specify a fixed bridge location, thus allowing the students significant flexibility in setting their bridge. As a result, the bridge spans varied from 45 – 75 ft. Eleven designs were proposed during the preliminary phase of the project and five designs were selected for the final phase of the class. The designs were nicknamed and are referred to here by these names: (1) Truss Bridge, (2) The Crossing, (3) Knee Brace Bridge, (4) Arco Iris and (5) No Where Bridge. The winning design, Truss Bridge, is presented here. The design criteria were similar to the previous year's project: LL=80 psf, Wind Load 120 mph wind, Exposure C, Seismic Loading based on USGS Hazard Maps and latest ASCE 7.

4.3.1 Truss Bridge

The Truss Bridge was the winning design for this project. The bridge spanned 65-ft and featured a space truss using wood members for web members and the top chord and steel cables and rods for the bottom chord. The top chord also acted as a stiffening beam designed to resist uplift loading. The renderings of the bridge are shown in Figure 15.

Fig.13 Truss Bridge (1st Place) Architects: Richard Delarosa, Candice Myers, Harold Ornelas, Leo Rodriguez, Johnny Tran; Engineers: Kun Chang, Henry Chi, Huong Vu. Original Design Concept by Harold Ornelas

The bridge design used the curvature of the bottom chord to provide an optimized truss design and to provide more attractive connections at the support. The structural analysis of the bridge was performed using SAP analysis. The engineering students created a structural model of the bridge using frame elements and analyzed the structural model for dead, live, seismic and wind loads. The SAP model and an axial force diagram under vertical load are shown in Figure 16.

The detailing of the connections was important from both the architectural and structural perspective. Selected connections developed for this design are shown in Figure 17.

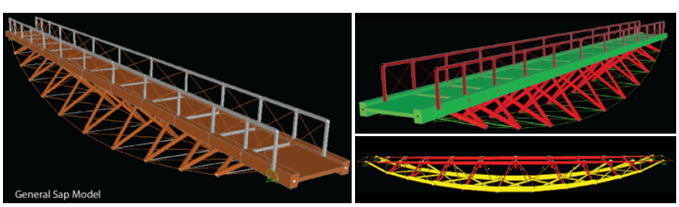


Fig. 14 Structural SAP Model and Vertical Loads Axial Load Diagram

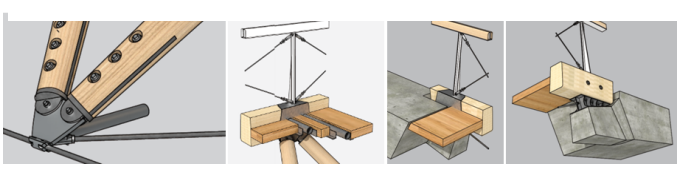


Fig.15 Connection details: truss web bottom connection and top connection, support top view and bottom view.

The students also built a 3"=1'-0"model of a section of the truss. The large scale of the model allowed students to experience some of the construction issues they would encounter in actually building the bridge and to address these as well as cost effectiveness. The image in Figure 18 shows the detail of the truss web members' connection to the top chord of the truss.

Fig. 16 Truss Bridge 3”=1’-0”scale model detail

5. Discussion, Conclusion and Acknowledgements

The student work presented in this paper demonstrates that collaboration between architects and engineers can produce aesthetically pleasing designs without sacrificing structural performance or cost effectiveness and constructability of the project. The use of pedestrian timber bridges as the class design topic, the progressive competition format and the involvement of the architectural and engineering professionals creates a truly inspirational learning environment full of creativity and an impressive student work ethic. The students were all highly motivated by the competitive nature of the project and devoted significant energies to delivering high quality work.

The AE interdisciplinary course provides a unique opportunity for architecture and engineering students to experience design collaboration in an educational setting. The course was highly rated by students and practicing professionals serving on the review panel. The course allowed engineering students to develop an understanding and appreciation of the aesthetics of design, to explore their creative side and learn to collaborate in developing and designing structural systems to accommodate architectural forms. The architectural students learned to develop a better understanding of the impact of structural requirements on their designs and to collaborate with engineers in developing forms that are conducive to constructible and economical designs.

The use of the studio format for engineering education is an excellent mechanism for developing many skills the industry desires in engineering graduates. These include creativity, collaboration, oral and written communication skills and the ability to understand the language and the needs of another discipline.

The development of this class was supported by the Colleges of Engineering and Environmental Design and the chairs of the Departments of Civil Engineering and Architecture.

6. References

[1] 2009 Legislative Session: 1st Session, 39th Parliament. (2009). *Bill 9 - 2009, Wood First Act.* BC Legislature.

[2] U.S. Department of Housing and Urban Development. (1994). *Alternative Framing Materials in Residential Construction: Three case studies.* Upper Marlboro, MD: NAHB Research Center.

USDA Forest Service. (2008). . Madison, WI 53726-2398: USDA Forest Service, Forest Products Laboratory.

[3] Cramer, S., Weat, D, (2011). “Education in Wood Structural Design: Who needs it?”. *STRUCTURE Magazine.*, June 2011, p5.

[4] Gershfeld, M., Sheine, J., and McGavin, G. (2011) “AE Studio – Beyond Pedestrian Access: Creating Bridges for Learning”, ASEE 2011 Annual Conference Proceedings.

Mile Long Boardwalk Enhances Estuary Restoration

Paul C. Gilham, P.E., S.E.
Chief Engineer
Western Wood Structures, Inc.
Tualatin, Oregon USA
paulg@westernwoodstructures.com

Paul Gilham has worked at Western Wood Structures, Inc. for the past 31 years. His responsibilities include design of engineered timber structural systems including bridges, trusses arches and domes. He inspects and designs repairs and upgrades for existing timber structures.

Summary

Timber boardwalks have seen an increase in use over the last 10 years as regulations regarding the disturbance of the ground in wetlands and sensitive environments have tightened. Such was the case in an estuary restoration project in south Puget Sound, Washington. This project included the removal a earthen dike allowing the area to revert back to a saltwater marsh. A timber boardwalk over 1.6 km (1 mile) long was constructed in the tidal zone to provide hiking, viewing and photography opportunities. The boardwalk included an observation tower, two viewing platforms and a bird viewing blind. Diamond Pier foundations, which are installed with minimal soil disturbance, were used to support the boardwalk. Tidal channels, formed after the removal of the dike, were spanned using timber pony truss bridges.

Keywords: Timber Boardwalk, Diamond Pier Foundations, Glulam, Timber Frame, Pony Truss.

1. Introduction

Figure 1. Boardwalk at high tide

The US Fish and Wildlife Service (FWS) partnering with Ducks Unlimited (DU) and the Nisqually Indian Tribe removed a 100 year old dike as part of an estuary restoration project. The removal of the dike returned 762 acres of land to a tidal marsh, vital to the environmental health of the estuary. The estuary provides nesting areas for waterfowl and important resting areas for migratory birds. More than 200 species of birds have been tallied in the area.

As part of the restoration effort, a timber board walk was installed to provide access for birdwatchers and hikers. Where most boardwalks are built to allow passage over a wetland or sensitive area from

one end to the other the Nisqually boardwalk invites visitors to linger and gaze at the surroundings, waterfowl and other wildlife. The boardwalk is located in the tidal flats allowing hikers to walk above the water at high tide and above the mudflats and the waterfowl feeding areas when the tide is out.

DU developed the layout for the boardwalk and the concept for the several ancillary structures. Western Wood Structures of Tualatin, Oregon took these concepts and provided the structural design for the boardwalks and ancillary structures.

2. Boardwalk Framing

The boardwalk is designed to carry a live load of 4.07 kPa (85 psf.). The wind speed at the site is 38 m/s (85 mph), Exposure D due to the unobstructed areas and water surfaces. The seismic base shear was 42%g using a seismic response modification factor of 1.5 for timber frames. The boardwalk is constructed using timber bents which are designed to carry the dead and live loads as well as resist the lateral loads due to wind and seismic forces. The glulam deck panels and sawn lumber handrails are made from Port Orford cedar (POC), a naturally decay resistant species and the balance of the timber is preservatively treated Douglas fir (DF).

2.1 Diamond Pier Foundations

figure 2. Installation of Diamond Piers.

The boardwalks are supported by Diamond Piers. These are precast concrete pier blocks with galvanized pipe pins that are driven into the soils. Bearing resistance of the piers is based on the horizontal projected area of the pins. The length of the pins is determined by the bearing characteristics of the soil and the load to be resisted. These piers are especially suited for sensitive areas such as wetlands and the tidal marsh. They require only a few shovels of soil to be disturbed to set the piers and they can be installed without heavy equipment. A 60 lb. electric jack hammer drives the pins into the soil. The spacing of the timber bents was set at 3.048m (10 foot) on center based on the 30.11 kN (6.72 kips.) capacity of the piers using 2 in. schedule 40 pipes, 2.1336m (7'-0") long. Each pier had a lateral capacity of 8.2kN (1.83 kips.) The maximum lateral load was 1.93kN (432 lbs.) due to seismic loading. A total of 940 Diamond Piers were used on this project.

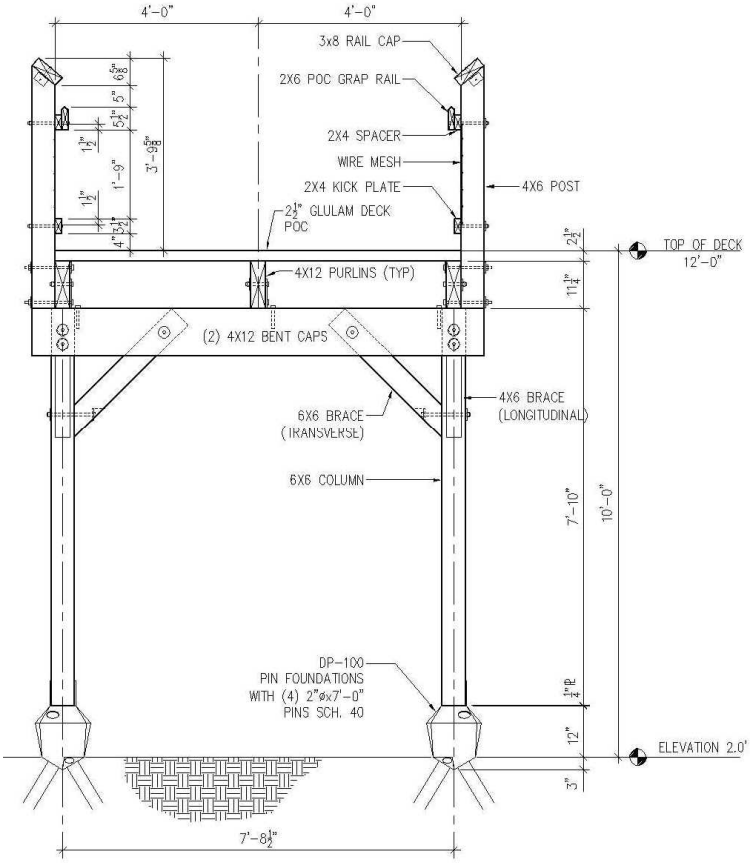


Figure 3. Typical Boardwalk Section

2.2 Timber Bents

The timber bents consisted of 6x6 DF#1 posts ranging in length from 2.44m (8'-0") to 4.88m (16'-0"). 4x12" DF#1 bent caps sandwiched the posts. 4x6 and 6x6 diagonal braces were used in the longitudinal and transverse directions respectively. These diagonal braces provide the lateral restraint necessary to resist lateral loads due to wind and seismic loading. The bents elevate the walkway to 3.66m (12'-0") above sea level.

All of the bent material was preservatively treated with ACZA meeting the AWPA use category UC5A. This treatment is used for lumber located in brackish or salt water environments and subject to marine borer attack. All of the preservative treatments were in accordance with the "Best Management Practices for the Use of Treated Wood in Aquatic Environments". This specification ensures that the minimum amount of treatment required to protect the wood is used, that the treatment is fully fixated in the wood and that the surfaces are clean before delivery to the jobsite.

2.3 Walkway framing

Three 4x12 DF Select Structural purlins support 6.35cm x 38.1cm (2½"x15") glulam deck panels. These panels are manufactured by ripping a 13.0175cm x 38.1cm (5 1/8"x15") beam lengthwise with a bandsaw. The rough sawn face is placed up and provides a skid resistant surface. The glulam deck panels are made with POC to reduce the amount of preservative in the project available to leach into the environment.

Originally DU specified Reinforced Plastic Lumber (RPL) for the deck and for the top rails. Preliminary bids for RPL came in almost twice as expensive as the glulam deck panels. Since the bending strength of the RPL is significantly lower than the glulam panels, three additional purlins would also be required adding to the expense of this system. RPL also continues to creep with time and over the years will sag between the purlins. Clearly the POC glulam deck panels were the best choice for this application.

The posts, kick plate and cap rail are made with DF sawn lumber. All of the DF lumber in the walkway was preservatively treated with ACQ meeting the AWPA use category UC4B which is appropriate for lumber subject to saltwater splash. The rail system features a 10.1cm x 10.1cm (4"x4") wire mesh to meet the required 10.1cm maximum opening size and to allow a greater degree of visibility for children and those in wheelchairs. A grab rail made with Port Orford cedar was chosen to provide a rail free of preservative chemicals.

2.4 Ancillary Structures.

The project included one observation tower, two viewing platforms, two walkway pushouts and a bird viewing blind. Each of these features invites visitors to stop and take in the views using observation scopes or rest on the benches provided by FWS. All of these structures are supported by Diamond Pier foundations and are made with pressure treated DF framing members and POC deck and grab rails. Roofs at the observation tower and viewing platforms provide a place to get out of the rain. DU specified a four sided hip roof for the Observation Tower, a gable roof for the first observation tower and an eight sided roof at the octagonal platform at the end of the boardwalk. Each of these roof structures were design to provide column free space. At the observation tower and octagonal platform, the hip beams acted as arches with a tension ring designed and detailed at the eave and a steel compression assembly at the peak.

2.4.1 Observation Tower

Figure 4. Observation Tower

The 3.6576m (12'-0") square observation tower, located at the beginning of the boardwalk has a floor height of 6.096m (20'-0") above sea level. From this height, visitors have a commanding view of the entire estuary. Full height columns at the corners support a gable roof. These columns are wrapped with cedar boards. Ramps slope up to the tower at 8.33% slope to allow handicap access to the platform. The ramp is 20.87m long (68'-6") with two switchbacks and two flat landings at the turns as required by the American with Disabilities Act.

2.4.3 Octagonal Platform.

Figure 5. Octagonal Shelter at end of boardwalk

The highlight of the boardwalk is an octagonal shelter at the end that provides a 360 degree view of McAllister Creek, the Olympic Mountains to the west, Mount Rainier to the east and several islands in Puget Sound. Measuring 7.3152m (24'-0") corner to corner, this structure provides 36.4m^2 (391.9 ft^2) of open space. The 6x6 columns in the corners extend to the tension ring at the eave level and support the eight hip beams that meet at a compression ring at the apex.

3. Tidal Channel Bridges

Figure 6. 15.24m (50'-0") Pony Truss Bridge

The construction of the boardwalk began shortly after the dike was removed. It soon became apparent that tidal channels were forming in several locations. These had not been accounted for in the layout of the boardwalk and it was not possible to predict where they would be located before the dike was removed. In four locations these channels were too wide to span with the standard 3.048m (10'-0") bent spacing. Timber Pony Truss Bridges were designed and installed to span these channels. These bridges ranged in length from 9.144m to 15.24m (30'-0" to 50'-0"). Each bridge consisted of timber pony trusses using 6x12 chords and 6x6 webs. These bridges were supported on timber bents with two Diamond Piers at each support. The trusses were shop assembled after preservative treating and shipped to the site where they were lifted into place with a helicopter.

4. Summary

The Nisqually National Wildlife Refuge boardwalk provides visitors with a unique opportunity to view a restored tidal estuary up close and personal. The project includes over a 1.6km (1 mile) of elevated boardwalk, viewing platforms, a bird viewing blind and an observation tower. Pressure treated Douglas fir and naturally decay resistant Port Orford cedar was used to provide durability for the boardwalks and other structures. Pony truss bridges provide clear spans over the tidal channels formed when a century old dike was removed. The light weight of the timber structure resulted in very small vertical reactions and base shears, allowing for the use of Diamond Pier foundation blocks.

The timber boardwalk fits beautifully into this setting. Its light footprint minimizes site disturbance while providing public access to the newly restored estuary.

Figure 7. Boardwalk at high tide with bird viewing blind and viewing platform

Stretching the Limits – Modern Timber Bridge Case Studies

Paul C. Gilham, P.E., S.E.
Chief Engineer
Western Wood Structures, Inc.
Tualatin, Oregon USA
paulg@westernwoodstructures.com

Paul Gilham has worked at Western Wood Structures, Inc. for the past 31 years. His responsibilities include design of engineered timber structural systems including bridges, trusses arches and domes. He inspects and designs repairs and upgrades for existing timber structures.

Summary

The trends in timber bridge design during the past decade have been to:

- increase span lengths,
- increase loading requirements, and/or
- leverage the aesthetics of timber.

Part of the trend toward increased span length is due to an added emphasis on protecting wetlands and flood plains. The time, effort and expense of permitting an encroachment into a flood plain often make a clear-span option the most cost-effective method of spanning a waterway. Additionally, removing abutments from the flood plain eliminates exposure to scour.

In terms of increased loading requirements, the latest AASHTO LRFD Bridge Design Specification [1] increased the vehicle loading requirements by adding a 9.34kN/m (640 plf) lane load, and a multiple presence factor of 1.2. The multiple presence factor alone effectively increased the weight of the HS20 design vehicle from 320kN (72,000 lbs) to 384kN (86,400 lbs), and the addition of the lane load adds an additional 285 kN (64,000 lbs) to each lane of a 30.48m (100 ft) bridge.

AASHTO also recently added guidelines and recommendations for bridge aesthetics. The specification states, "Bridges should complement their surroundings, be graceful in form and present an appearance of adequate strength." [2]

The following case studies illustrate how the Modern Timber Bridge design process has kept up with these trends, striding confidently past what was once considered the limits of timber bridge capabilities, while maintaining the functionality, economic advantages and stunning aesthetics inherent in timber construction.
Keywords: Glulam, Pedestrian Bridge, Vehicle Bridge, Timber Bridge, Timber Truss, Timber Arch

1. Introduction

The following four case studies detail the design challenges presented, and solutions arrived at to make these Modern Timber Bridges a reality. The three-span, 125m (410 ft) Cosumnes River Bridge features a center span of 61m (200 ft). The Lower Burnett Road Bridge spans 122m (400 ft), using three glue-laminated (glulam) arch spans. At a remote Alaskan site on the Placer River in the Chugach National Forest, the Whistlestop Bridge offers an 85m (280 ft) clear span, making it the longest clear-span pedestrian bridge in North America. And finally, a pair of arch spans at Overpeck Park in New Jersey each cover 43m (140 ft), and carry two lanes of vehicular traffic.

2. Cosumnes River Bridge – Rancho Murieta, California

2.1 Overview

The Cosumnes River Bridge near Sacramento, California, is a 3.66m x 124.96m (12'-0" x 410'-0") pedestrian bridge that links two halves of a residential development built on both sides of the Cosumnes River. Wetland restrictions prohibited intermediate piers in the existing streambed, requiring a 60.96m (200'-0") clear span. The layout also required two 32m (105'-0") side spans to reach the top of the bank on each end and clear the 100-year flood elevation. The bridge was built to provide pedestrian and light-vehicle access across the river. It is designed to carry 4.07kPa (85 psf) pedestrian loading and a 35.85kN (8000 lb) vehicle. The wind loading was per the AASHTO Standard Specifications for Highway Bridges. This results in a minimum wind load of 4.38kN/m (300 plf) in the plane of the windward chord and 2.19kN/m (150 plf) in the plane of the leeward chord.

2.2 Truss Configuration

The first design decision for this truss bridge was to settle on the basic shape of the truss. The AITC Timber Construction Manual [3] suggests a span-to-depth ratio of between 8 and 10 for a parallel chord truss. A span-to-depth ratio of 9 results in a depth of 3.48m (11'-5") for the side spans and 6.86m (22'-6") for the main span. An abrupt change in truss height at the piers was not aesthetically pleasing, and using the 6.86m (22'-6") depth for the side spans looked out of proportion. AASHTO recommends that, “Engineers should seek more pleasant appearance by improving the shapes and relationships of the structural components themselves…abrupt changes in the form of components and structural type should be avoided.”[4]

To avoid an abrupt transition, a curving transition from the 3.48m (11'-5") depth at the side spans to the 6.86m (22'-6") depth at the center span was implemented, which provided a visually pleasing solution as shown in Figure 1. The top chord members were built with a reverse curve, resulting in a continuously curving profile for the bridge and avoiding the abrupt change from a shallow side span to a deeper center span.

Figure 1. Curved transition between spans

With the 9:1 span-to-depth ratio for the main span, the live load deflections were greater than what is considered acceptable, so a method of stiffening the center span was needed. Using larger truss members was considered but deemed economically prohibitive. The decision was made to make the trusses structurally continuous over the supports. The three-span continuous arrangement effectively reduced the main span deflections by half.

2.3 Design Challenges and Solutions

There were consequences to providing this continuity over the supports that needed to be addressed. With the continuous spans, the truss members experienced stress reversals with unbalanced loading that had to be accounted for in the member and connection designs. For example, the bottom chord splice connection at the piers was in compression under balanced load cases, but in tension under the unbalanced case.

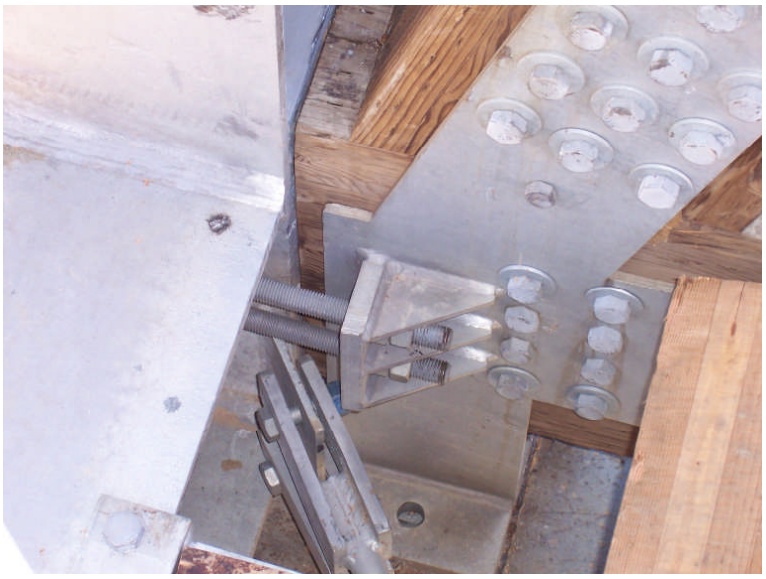


Figure 2. Tension/compression connection at pier

At this location, the connection had to be designed for forces in both directions. This was accomplished by designing the connection to provide end-grain bearing of the chord members when stressed in compression, and using tension ties similar to a hold-down, to resist the tensile forces (See Figure 2).

The wind load resulted in extremely high lateral reactions at the interior piers. This was another consequence of providing continuity of the bridge. The bridge is designed with horizontal trusses between both top and bottom chords.

These trusses transfer the lateral loads due to wind and seismic to the interior piers. The lateral reaction at the end of the top chord had to be transferred vertically to the pier. The configuration of the intermediate piers did not allow for a traditional knee brace or cantilevered column to support the ends of the top chord.

Figure 3. U-shaped, moment-resisting frame at piers

Therefore a rigid U-shaped, moment-resisting steel frame was designed to transfer lateral forces from the top chords to the piers. This frame was designed to fit inline with the truss structure, making it visually transparent in the final structure.

2.4 Erection Methods

Each span was fully assembled on the river bank before being lifted into the final position. In this way the workers are able to work in man-lifts as opposed to being above the water, tied off to the bridge structure. This sequence of erection, while requiring higher-capacity cranes, increases the safety and speed of the erection process and ultimately reduces the cost of the erection. The design of the connections between spans was done in coordination with the installation procedures. The truss connections at the piers and abutments were designed so that all bolts through the wood could be installed before the sections were lifted. This eliminated the time it would have required to align multiple rows of bolts while working over the river with the crane supporting the section.

Connections attaching one section to the next were made using steel pins or hold-down type connections for tension connections, and end-grain bearing of the wood on compression type connections. For example, the top chord splice over the pier is a tension splice that was made using 63.5mm (2½") diameter pins. This connection was easily and quickly made during the erection process. The bridge sections were lifted into place using a 300-ton crane.

The center span was picked as one unit. This lift weighed 576.46kN (129,600 lbs) – the heaviest single lift performed to date by a Western Wood Structures crew. By paying close attention to the connection details, the entire bridge was set in one day, minimizing crane costs and worker exposure to falls.

2.5 Results

The Cosumnes River Bridge design successfully achieved the goal of a three-span continuous structure that flows gracefully from one end to the other. The two intermediate piers were placed at the edge of the normal water level, and the ends of the bridge were located outside of the riparian zone to minimize permitting considerations. The continuity significantly reduced member forces and deflections in the center span but resulted in complicated connections at the supports. By carefully considering the erection sequence, these connections were designed to transfer all of the necessary forces, while remaining easily achievable during the erection process.

3. Lower Burnett Road Bridge – Buckley, Washington

3.1 Overview

The Lower Burnett Road Bridge is a three-span, timber arch bridge that replaces an early 20th century timber trestle on the Foothills Trail in Buckley, Washington. The bridge is located in the middle of a switchback where the old railroad grade gained 60.96m (200 ft) in a little more than 3.2 km (2 miles), with a horizontal radius of 198m (650'-0"). The bridge spans South Prairie Creek and Lower Burnett Road with a total span of 118.86m (389'-11½"). The structure is 5.49m (18'-0") wide and is designed to carry H15 vehicle loading in addition to the 4.07kPa (85 psf) pedestrian load.

3.2 Choice of Structure Type

Options considered for this bridge included a girder-type bridge with intermediate piers, a trestle system similar to the original train trestle and an under-arch system. The girder-style bridge presented several problems, including large longitudinal girders and the requirement for 11.58m (38 ft) tall intermediate piers capable of resisting both the gravity loads and significant lateral loads.

The trestle option would require numerous bents. This option allowed for the horizontal curvature to be built into the support structure. However, it was considered to be overly expensive and would include several foundations to be built in the flood plane. South Prairie Creek overflows its banks on a

regular basis, so fewer abutments in the flood plain reduced the exposure to scouring potential. Additionally, AASHTO aesthetic guidelines suggest limiting the number of piers.

The under-arch system was chosen as the most cost-effective and visually pleasing solution. The braced arches are capable of transferring both the gravity loads and lateral loads to the foundations. Using timber bents of varying heights on top of the arches and a curved glulam deck, the horizontal curvature and elevation rise was accommodated.

3.2.1 Secondary Superstructure Elements

The secondary framing system used a 17.14cm (6 ¾") longitudinal glulam deck and the horizontal curvature was easily accommodated by curving the deck panels to the 198m (650 ft) radius. The deck is supported by timber bents that are supported by the main arches.

3.3 Detailing and Fabrication Challenges and Solutions

Due to the slope of the bridge and the horizontal radius, each of the 28 bents was unique. The challenge of prefabricating the structure began with accurately detailing each of the timber members and welded steel connection assemblies. Detailing the bridge was done using solid modeling techniques where each glulam and steel element was drawn out as a solid member. In this way each timber and steel member were accurately depicted. The fabrication details for each glulam member and steel assembly were extracted from the model and exploded into the orthographic views needed by the glulam and steel fabrication facilities.

Figure 4. Preassembly of timber bents

Figure 5. Installation of timber bents

3.4 Results

The Lower Burnett Road Bridge is an extraordinary example of a bridge complementing its surroundings, as recommended in the AASHTO aesthetic guidelines. The beauty of the wood naturally blends in with the rural wooded location. The glulam arches and horizontal radius combine to provide a fluid link between the abutments.

Figure 6. Lower Burnett Road Bridge

4. Whistlestop Bridge – Portage, Alaska

4.1 Overview

This bridge was built as part of the long-term partnership between the USDA Forest Service and the Alaska Railroad Corp. to provide recreational opportunities in the remote back country of the Kenai Peninsula. In their request for a design/build proposal, the Forest Service stipulated that the new bridge must incorporate the feel of the early 20th century Alaska Railroad work camps.

Western Wood Structures, working as a subcontractor to Patrick Engineering, submitted a proposal to supply a camelback design, which emulates the railroad bridge downstream from the project site. With a span of 85.34m (280'-0"), this bridge is the longest clear-span, timber pedestrian bridge in North America.

4.2 Bridge Layout and Loading

The location of this bridge in a remote area of the Kenai Peninsula resulted in several design and construction challenges that needed to be considered. The loads in this remote site included a 9.57 kPa (200 psf) ground snow load, and 52m/s (120 mph), exposure C wind loading in addition to the 4.31kPa (90 psf) pedestrian live load.

4.3 Erection Coordination

The remote site, in addition to the capacity and reach limitations of the available equipment, dictated that the erection sequence be adjusted. The crane supplied by the contractor did not have the capacity and reach to set the center section in one unit as designed. Therefore the center section had to be built piece-by-piece, necessitating that the timber members and their connections be checked due to this change in erection procedures. For example, the 27.31cm x 64.77 cm (10 ¾" x 25 ½") bottom chord now behaved like a beam instead of a truss chord, and the connections to the end sections had to transfer beam reactions to the adjacent chord members, instead of axial tension forces typical for a truss bottom chord.

Figure 7. Piece-by-piece installation of the center truss section

A truss assembly sequence was developed where webs were installed and connected in such a way as to avoid overstressing the members and connections. Of particular concern was the notched shear capacity of the connections. When acting as a truss member the shear is relatively small, but when acting as a beam the notched shear at the connection is significant.

The weight of the end sections also exceeded the crane capacity. The contractor suggested removing several of the vertical web members and horizontal bracing members to reduce the weight of the lift. The members and connections were checked for erection stresses under this configuration.

Figure 8. Completed bridge before removal of shoring

The end of the short Alaskan building season arrived before the temporary shoring could be removed. The truss members and connections were checked again with the interior supports. Several of the web connections near the temporary supports were found to be significantly overstressed under full snow loading. Thus, the contractor was required to remove the deck for the winter, eliminating snow accumulation on the bridge.

4.4 Outcomes

The Whistlestop pedestrian bridge is now scheduled for completion in August 2013. This summer the contractor will remove the temporary interior supports and reinstall the decking. Through close coordination with the engineer, Patrick Engineering was able to evaluate the proposed erection sequences to build this truss in a difficult site without compromising the integrity of the structure.

5. Overpeck Park Bridges – Teaneck, New Jersey

5.1 Overview

The entrance to Overpeck Park in Bergen County, New Jersey, lies immediately adjacent to the interchange of the Interstate 80 and Interstate 95 freeways. The county, recognizing that thousands of commuters would see this entrance structure daily, chose identical glulam arch bridges for this entrance structure. Each bridge is a 42.67m (140'-0") tied arch bridge with a 9.14m (30'-0") roadway and a 3.05m (10'-0") walkway on one side only. These bridges are designed to carry two lanes of vehicle traffic. The design vehicle is the AASHTO HS20 vehicle with a 25 percent overload or a 400.32kN (90,000 lb) vehicle.

5.2 Bridge Style

The main structural elements on these bridges are three-hinged, tied glulam arches. Transverse glulam floor beams are suspended from the arches with steel hanger rods and hanger assemblies. A 22.25cm (8 ¾") longitudinal glulam deck forms the structural deck. Glulam transverse and diagonal braces form chevrons to provide lateral support to the arches.

5.3 Design Challenges

The significant challenge presented by the design of these bridges was the vehicle loading and the dead load of the asphalt wear surface. Historically, timber has not been considered for structures of this size and loading. For example, in addition to the two vehicles, the asphalt wear surface averaged 13.2cm (5.2") thick, which added more than 1236.6kN (278,000 lbs) of dead load to the bridge. The combination of span and loading resulted in extremely large arch members. These arches were fabricated with a 29.4m (96'-6") radius to provide a height of 9.6m (31'-6") at mid-span. The required clearance from the top of the asphalt to the bottom of the first cross brace was 4.42m (14'-6") resulting in a 6.68m (21'-11") unbraced length for the arches at the heel. These criteria resulted in 36.2cm x 152.4cm (14 ¼" x 60") arch members weighing nearly 3.65 kN/m (250 lbs/ft.)

The transverse floor beams were spaced at 3.28m (10'-9") on center and had a design span of 10.77m (35'-4"). Each floor beam was loaded with two 177.92kN (40,000 lbs) axles (four-wheel loads spaced 1.83m (6'-0") apart). The self weight of the beams, deck and asphalt added roughly 18.75 kN/m (1,285 plf). These members were 27.31cm x 121.92cm (10 ¾" x 48") glulam beams. The overall depth of the structure including the asphalt wear surface and longitudinal deck was roughly 1.625m (5'-4"). However, since the floor beams are parallel to the stream flow, they do not significantly obstruct the waterway during a flood event.

The 22.25cm (8 ¾") glulam longitudinal deck was laid out in three-span and four-span configuration. The continuous panels limited the deflection to 4.32mm (0.17"). Transverse stiffeners at the mid-span allow the deck panels to share the loads as a flat plate, and limit any differential deflections between deck panels.

5.4 Other considerations

The offset of the arches was just over 3.66m (12'-0"), making them too wide to fit in a 2.44m (8'-0") pressure-treating cylinder. Therefore moment splices were added to cut the arch members in half.

Figure 9. Workmen installing moment splice

The tension force in the splice was more than 596kN (134,000 lbs). The splices required two rows of ten 25.4mmϕ (1"ϕ) machine bolts with a 12.7mm (½") kerf plate and two 7.94mm (5/16") side plates. This configuration resulted in four shear planes on each bolt. The shear force at this connection was 204.61 kN (46,000 lbs) and required four rows of five 25.4mmϕ (1"ϕ) machine bolts in double shear.

Figure 10. Tension tie channels connected to heel assembly

The arches are tied between the heels to resist the thrust produced by the arches. Typically the thrust is resisted by the abutments. However, the section between the bridges was not capable of resisting this horizontal loading. Two C12x30 channels were used to resist the 1,512 kN (340,000 lbs) tension force. These channels are connected to the 13.335cmϕ (5 ¼"ϕ) pins at the arch heel assembly. In order for the tie to be effective, the arch heel assembly needed to be able to slide longitudinally at one end. Slotted holes for the anchor bolts in the bearing assembly allow for this movement.

5.5 Results

The Overpeck Park Bridges demonstrate that Modern Timber Bridges are capable of carrying today's highway loads for significant spans. The criterion for a dramatic park entrance was fulfilled in striking fashion as can be seen in Figure 11.

Figure 11. Completed Overpeck Park bridges

6. Discussion and Conclusions

The four case studies presented demonstrate the ability of Modern Timber Bridges to meet contemporary requirements for longer spans and heavier loads while fitting beautifully into any site. With the experience gained through solving the challenges presented by the completion of these structures, timber engineers can take the next step in pushing the envelope in timber bridge design.

7. References

[1] American Association of State Highway and Transportation Officials, *LRFDBridge Design Specifications,4th Edition 2007,* Washington, DC, p 3-17

[2] American Association of State Highway and Transportation Officials, *LRFDBridge Design Specifications,4th Edition 2007,* Washington, DC, p 2-16

[3] American Institute of Timber Construction, *Timber Construction Manual, Fourth Edition,* John Wiley and Sons, Inc., New York, 1994, p 6-331

[4] American Association of State Highway and Transportation Officials, *LRFDBridge Design Specifications,4th Edition 2007,* Washington, DC, p 2-16

Inspection of Louisiana Timber Bridges

Vijaya Gopu
Endowed Chair
UNO & LTRC
Baton Rouge, LA, USA
v.gopu@la.gov

Vijaya is currently the Associate Director at the Louisiana Transportation Research Center (LTRC) and the Schlieder Endowed Chair at the University of New Orleans

James Wacker
Research Engineer
Forest Products Laboratory Madison, WI, USA
jwacker@fs.fed.us

Jim earned his Civil Engineering degree from the University of Wisconsin. He is a registered professional engineer with more than 20 years' experience with timber bridge research.

Summary

This paper reports the result of a study undertaken to assess the condition of timber bridges in four distinctly different geographic regions in the state of Louisiana. This study was carried out as part of an overall national study spearheaded by the U.S. forest Products Laboratory to investigate the field performance of timber bridges. Louisiana, with the second highest number of timber bridges in the nation spread across the entire state, was an excellent location to assess the performance of these bridges under different environmental conditions. The bridges inspected were representative of the various types of timber bridges in the state and covered different exposure conditions. The inspections were conducted with the assistance of the Louisiana DOTD (Department of Transportation and Development) bridge inspection team. The inspection teams utilized advanced assessment tools such as the resistance micro-drill and acoustic devises to evaluate the condition of the suspect regions of the timber sub- and super-structure elements. The availability of the advanced inspection tools eliminated the bridge inspection teams from having to rely on the traditional tools such as the "pick and hammer" to guess the internal condition of the timber member and significantly improve the ability of the bridge inspectors to assign the proper load rating to existing timber bridges. Detailed results of the inspection conducted on a select group of bridges are presented and the benefits of utilizing the advanced assessment tools are highlighted. Lessons learned from the inspections are presented.

Keywords: Timber bridges, inspection, condition assessment, field performance

1. Introduction

Louisiana has the second largest number of timber bridges in the nation. The 2,068 timber bridges in the state account for 8.5% of all the timber bridges in the country. With nearly half of its timber bridges rated as structurally deficient and another 15% rated as functionally obsolete, the state has the second largest number of deficient timber bridges in the nation. Investigation of the field performance of the timber bridges is of particular interest to the state since the study has the potential to provide insight into construction methods/techniques that can improve the service life and functionality of the timber bridges. The inspection of a selected group of timber bridges in different regions of the state was conducted by the authors with the assistance of a bridge inspection team from the Louisiana Department of Transportation and Development, LaDOTD.

2. Selection of Timber Bridges for Inspection

LaDOTD provided a list of candidate timber bridges located in distinctly different geographic regions of the state. The bridges also represented different timber superstructures used in the state. US Forest Products Laboratory, which was spearheading the national study, made the final selection of the bridges to be investigated from the list of candidate bridges. Eleven bridges located in Districts 02, 08, 61 and 62 shown in Figure 1 were selected for the study. These four districts very effectively represent the different soil and environmental conditions that timber bridges are exposed to in the state.

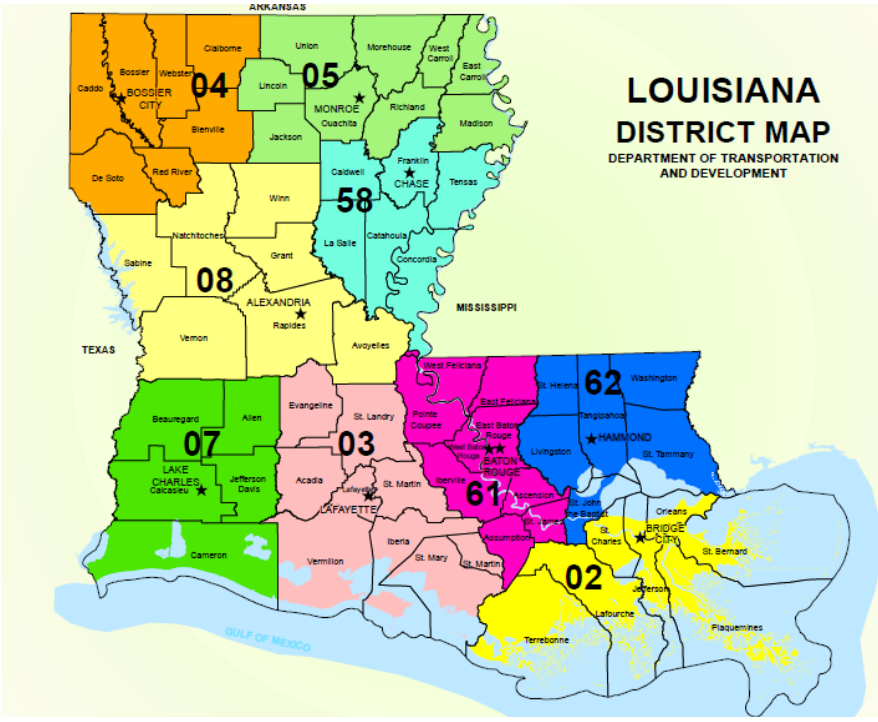


Fig. 1 Louisiana DOTD Districts

Specifically, the bridges inspected were located in LaFourche and Jefferson parishes in District 02, Winn and Natchitoches parishes in District 08, East Baton Rouge parish in District 61, and Tangipahoa parish in District 62. District 02 represents the coastal and marsh region of the state, District 08 represents the mid-section of the state that is outside the Mississippi flood plain, District 61 and 62 represent the low lying areas in the lower Mississippi delta plain. From the Wood Decay Hazard Map shown in Fig. 2 it is evident that all timber bridges in Louisiana fall in the Severe Deterioration Zone.

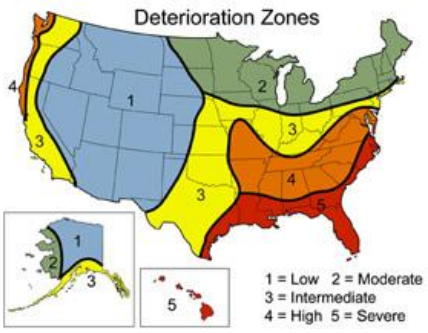


Fig. 2 Wood Decay Hazard Map (American Wood Protection Institute)

3. Bridges Inspected in Various LADOTD Districts

The bridges inspected in the four LaDOTD districts are listed in separate tables in the sequence in which they were inspected within a district. With the exception of the overlay and dimensions, the bridges within a district had similar construction features. The details of the bridges are presented in the tables to facilitate discussion of the bridges.

Table 1 Details of Bridges inspected in District 08

Bridge No.	Decking	Runners / Overlay	Stringers	Piling	Length	Max Span	Num. Spans	Built
08-1	Timber Plank	7" Asphalt	Sawn Lumber	Timber	39	10	4	1967
08-2	Timber Plank	2" Gravel	Sawn Lumber	Timber	170	19	10	1977
08-3	Timber Plank	4" Asphalt	Sawn Lumber	Timber	58	19	3	1941
08-4	Timber Plank	4" Asphalt	Sawn Lumber	Timber	77	19	4	1941

Table 2 - Details of Bridges inspected in District 62

Bridge No.	Decking	Runners / Overlay	Stringers	Piling	Length	Max Span	Num. Spans	Built
62-1	Timber Plank	2" Gravel	Sawn Lumber	Timber	31	15	2	1980
62-2	Timber Plank	5" Gravel	Sawn Lumber	Timber	154	19	8	1960
62-3	Timber Plank	5" Asphalt	Sawn Lumber	Timber	58	19	3	1968

Table 3 - Details of Bridges inspected in District 02

Bridge No.	Decking	Runners / Overlay	Stringers	Piling	Length	Max Span	Num. Spans	Built
02-1	Timber Plank	Runners	Sawn Lumber	Timber	57	15	4	1960
02-2	Timber Plank	Runners	Sawn Lumber	Timber	53	15	4	1980

Table 4 - Details of Bridges inspected in District 61

Bridge No.	Decking	Runners / Overlay	Stringers	Piling	Length	Max Span	Num. Spans	Built
61-2	Concrete	AC Ovrly	Sawn Lumber	Timber	56	19	3	1974
61-1	Concrete	AC Ovrly	Sawn Lumber	Timber	96	20	5	1960

4. Inspection Methods

All the bridges were inspected following a protocol recommended by the US Forest Products Laboratory and a 5-step inspection procedure. The 5-step procedure involved the following:

1. Labeling abutments, piers, girders, etc.
2. Conducting initial visual assessment with hammer sounding
3. Obtaining moisture content measurement in suspect & decay-prone areas
4. Establishing baseline NDE data by collecting stress-wave & resistance micro-drilling data from areas of suspected sound wood;
5. Investigating marked areas to measure the extent of internal deterioration by utilizing stress wave timer and resistance micro-drilling tool as needed.

Hi-resolution digital photographs of the bridge were taken to record the end views, side views, wearing surface, deck underside, and decayed or damaged areas of the bridge. Particular attention was paid to construction details and flaws. The impact of these details and flaws on the bridge durability/service life was noted. Photographs were obtained to record all aspects of these observations. Prior to visiting the bridge site, information about the bridge was collected from the bridge inspection division to facilitate compilation of information about the bridge. If the information did not contain the construction plans/drawings, then sketches were made on site to capture the necessary details of the bridge. The field sketches were converted to scale drawings in the office.

Figures 3 - 6 show a member of the bridge inspection team conducting various inspection tasks utilizing the traditional tools and the NDE equipment to assess the condition of the components of the sub- and super-structure. Figure 7 shows the labeling used to identify areas investigated in the bridge.

Fig. 3 Inspector using hammer to assess condition of pile from sound

Fig. 4 Determing moisture content of wood deck with moisture meter

Fig. 5 Resistance Microdrilling to assess condition of stringer

Fig. 6 Resistance Microdrilling to assess condition of pile suspected of being hollow

Fig. 7 Labeling of girder locations for reference

6. Bridge Examples

In this section the observations made in assessing the conditon of two of the bridges investigated in Louisiana are presented. The two bridges are located in central Louisiana in Winn Parish and LaDOTD District 08.

6.1 Bridge 08-4

This bridge is located on LA 126 and has four spans of 19 ft. each. The superstructure comprising of timber girders and timber deck is supported on a timber pile substructure. The timber deck is topped with asphalt paving. An average driver will not realize that he is crossing a timber bridge because of the asphalt paving. The bridge was built in 1941 and is heavily utilizied by logging trucks plying on the Highway 126. The following figures present various views of the bridge from above and underside.

.

Fig. 8 View of Approach to Bridge 08-4

Fig. 9 Side View of Bridge

Fig. 10 Bridge Abutment

Fig. 11 Stringers supporting timber deck

Fig. 12 View of deteriorated wood deck

Fig. 13 View of logging trucks traversing bridge

This timber bridge has been in service for over 70 years and appears to be in very good condition. Though the bridge is load posted for 40T for trucks, it was evident during the inspection that it was carrying significantly higher loads imposed by the logging trucks. The heavy creosote treatment used for the timber members contributed to a longer than expected service life for the structural elements.

6.2 Bridge 08-2

This bridge is located in Kisatchie National Forest on Carter Crossing Road. The bridge spans the Dudgemona River and has ten spans and aan overall length of 177 ft. The superstructure comprising of timber girders and timber deck is supported on a timber pile substructure. The timber deck is topped off with gravel that is periodically leveled by a grader. The bridge doesn't experience heavy traffic but offers the only crossing of the river in the forest. The bridge was built in 1977 and is traversed by light trucks.

The following figures present various views of the bridge from above and the underside and clearly show the significant damage caused to the decking primarily by the 41 and is heavily utilizied by logging trucks plying on the Highway 126. The following figures present various views of the bridge from above and underside and show the damage caused to the deck by the grader utilized to level the gravel topping on the deck. While the substructure and the stringers are in good condition, the deck is serious state of disrepair.

Fig. 14 View of Approach to Bridge 08-2

Fig. 15 Side View of Bridge

Fig. 16 Stringers supporting timber deck

Fig. 17 Deteriorated deck exposing stringers

Fig. 18 Loss of deck section spanning stringers

This major concern with this timber bridge is the loss of deck due to the grader. The inspection clearly indicated the need to replace the gravel topping with timber runners that can provider protection to the deck. The absence of adequate railing was of concern since the railing currently in place only serves the purpose of identifying the edge of the bridge and does provide any impact resistance. Extreme caution is necessary to cross the bridge in its current condition.

7. Challenges in Bridge Inspection

The inspection team encoutered a few challenges in conducting the study.

- Some of the timber bridges that were originally selected for inspection could not evaluated because of the high water level, currents and inadequate headroom to conduct an inspection from a boat. Substitutions were made to ensure the desired number of bridges were inspected.
- In long-span bridges it was extremely difficult, if not next to impossible, to use the resistance microdrill to assess the condition of the stringers and deck due to height of the pile bent. Climbing the braces provide access only to the pile cap and the strengers at the supports. Also, operating the microdrill at the top of the bent with one hand is nearly

impossible. Significant amount of equipment has to be mobilized to gain access to the underside of long-span bridges with tall bents.

- Bridges with low clearance under the stringers offered a challenge also since it was difficult to operate the NDE equipment in such small clearances.
- It was critical to have a three member team to conduct the investigation because of the extensive documentation of the bridge inspection and assessment.

8. Summary and Conclusions

The investigation of the selected timber bridges in Louisiana was conducted successfully with the support of the LaDOTD inspection team. Most of the bridges inspected have been in service for over 50 years and appear to have delivered the expected service. The following observations were made from the inspections and these can be used to improve on the construction methods used for timber bridges.

1. The use of asphalt paving over the timber deck has contributed to the decay and deterioration of the deck in a majority of the cases because of the trapping of moisture between the paving and the deck. Once the asphalt cracks -- which it does due to the flexibility of the timber deck -- moisture travels to the wood deck and gets trapped over the deck since there is no provision for moisture to escape.
2. It is critical that a flexible and reliable vapour barrier be placed between the timber deck and the asphalt paving to improve the service life of the deck.
3. The use of a water shedding groove in the timber deck board is important to avoid trapping of moisture.
4. The use of timber abutments, while an easy alternative in some cases, should be avoided if possible to improve the service life of the overall bridge. The abutments can become a weak link the bridge system.
5. Timber pile deterioration in the zone of wetting and drying was observed in several bridges. Inexpensive treatment of piles sections in this zone with inorganic resins will significantly reduce pile deterioration which is the single most important cause for DOTs staying away from timber bridge systems.
6. The lack of emphasis on keeping the timber deck dry and restricting access to moisture has been a major contributor to the deterioration of the deck in timber bridges.

9. References

[1] American Association of State Highway and Transportation Officials. 1983. Standard specifications for highway bridges. 13th ed. Washington, DC: American Association of State Highway and Transportation Officials.

[2] American Society of Civil Engineers. 1982. Evaluation, maintenance, and upgrading of wood structures. Freas, A., ed. New York: American Society of Civil Engineers. 428 p.

[3] American Society of Civil Engineers. 1975. Wood structures, a design guide and commentary. New York: American Society of Civil Engineers.

IN-DEPTH TIMBER BRIDGE INSPECTION AND LOAD RATING

James Scott Groenier, P.E.
Project Leader, Structures
USDA Forest Service,
Missoula Technology and
Development Center
Missoula, MT
jgroenier@fs.fed.us

David Strahl, P.E
Structures Engineer
USDA Forest Service,
Pacific-Northwest
Regional Office
Portland, OR
dstrahl@fs.fed.us

Summary

This paper focuses on field inspection and load rating of timber bridges. The U.S. Department of Agriculture, Forest Service is responsible for more timber bridges than any other U.S. Federal agency. The Forest Service has more than 7,300 timber road and trail bridges located on the National Forest transportation system. This paper discusses cursory and in-depth inspection of timber bridges and load rating timber bridges using two computer programs; Timber Bridge Analysis and Rating (TBAR) and MathCAD.

Keywords: inspection, NDT, timber bridge, moisture meter, resistograph, stress wave, TBAR, MathCAD

1. Introduction

The Forest Service has used timber components in the construction of many of its road bridges for more than 100 hundred years. There are approximately 6,200 road bridges currently in-service on the National Forest transportation network. Bridges constructed of timber make up over half these bridges. There are more than 3,450 timber bridges on Forest Service roads with one or more of the primary structural components constructed of timber (i.e., decks, girders, slabs). Forest Service timber bridges (Figure 1) have been built for more than 100 hundred years with most existing bridges being built 30 or more years ago. Due to the age of these structures, a good inspection and load rating process is required to maintain them and to ensure that they are safe for general public use.

Fig. 1 Typical Forest Service timber road bridge

2. Why Do We Inspect and Load Rate Bridges in The United States?

We inspect road bridges in the United States because it is the law, covered by the Code of Federal Regulations, Title 23, Part 650 (23 CFR 650) – Bridges, Structures, and Hydraulics. 23 CFR 650.311(a)(1) requires that routine inspections be conducted on each bridge at intervals not to exceed twenty-four months. In addition, 23 CFR 650.313(a) outlines the inspection procedures and 23 CFR 650.313(c) requires that each bridge be rated for a safe load-carrying capacity in accordance with American Association of State Highway and Transportation Official's *The Manual for Bridge Evaluation [1]*.

3. Bridge Inspection Types

Types of inspections can be classified as visual or in-depth inspections. Visual inspections are the most common method of inspection and use the human senses (vision, touch, hearing, and smell) and non-specialized equipment as the means for inspecting a structure. In-depth inspections typically use specific equipment that has been specially design for testing a certain type of material.

Visual inspections can be further broken down into two categories; cursory and hands-on. Cursory inspections involve looking at the bridge as a whole and identifying possible defects or problems. Examples of things that you would be looking for during a cursory inspection are sagging beams, water ponding, uneven surfaces, or any other things that do not look visually correct. The hands-on inspection involves getting within an arm's reach of the feature being inspected and looking for potential problems.

Typical tools used for hands-on inspection for timber members are a hammer, awl, level, and ruler. The hammer is used for sounding the timber beams and decks and listening for unusual sounds. Sounding works best for members less than 2 inches thick. Most timber bridges are pressure-treated and have a 1½ to 2-inch thick protective shell around the outside of the member, making it hard for the inspector to hear any difference. The awl is used to perform a pick test to find out if the wood fibers are starting to decay and also to probe the member to see how much decay is occurring on the outside of the member.

In-depth inspections can be either non-destructive testing (NDT) or destructive testing. This paper focuses only on non-destructive testing [2] while the inspector is trying to reduce any effect that the

testing will have on the timber member. There are three main types of specialized testing equipment we use for timber bridges: the moisture meter [3], the stress wave tester [4], and the resistograph drill [5].

Moisture meters are used to measure the percentage of water in the wood member. When the moisture content exceeds 25 percent, the chance of the wood rotting is greater. The stress wave tester is used to determine the velocity at which a stress wave travels through the wood. It can be used to determine the chance of decay being present in the wood. The resistograph drill is an electronic, high-resolution needle drill that is used to measure the resistance of the wood based on the power consumption of the drill.

4. Bridge Inspection Procedures

The procedure for inspecting a bridge should follow a consistent sequence. Following a constant procedure reduces the chances of overlooking a problem with the structure. A typical bridge inspection follows these basic steps:

1) Conduct a cursory inspection of the bridge looking for any sagging members, ponding water, uneven surfaces, and any other unusual observations.
2) Conduct a hands-on inspection of the bridge, making sure to investigate any problems that were observed during the cursory inspection.
   A hands-on inspection includes measuring the bridge components (beams, decks, sills, etc.) and beam spacing, determining wood species, looking for moist areas, decay and fruit bodies, and looking for defects, crushing areas, and hollow sections (Figure 2). Tools that are normally used for this is inspection are a hammer, awl, level, stringline, and tape measure.

Fig. 2 Hollow section caused by advanced decay

3) Conduct an in-depth inspection of any problems found during the hands-on inspection. The Forest Service uses a moisture meter, stress wave tester, and resistograph drill to conduct in-depth NDT. The in-depth process used includes:
   a. Looking for areas of moisture, crushing, or any other signs of distress and taking a moisture reading at these locations
   b. Using the stress wave tester to take readings along the entire length of the beam and marking high reading locations with chalk

c. Using the resistograph to drill the high stress wave reading locations to investigate them
d. Drawing up a diagram of all problem locations

A common problem with solid sawn timber beam bridges with plank decking and no asphalt wearing surface is that the decking has been replaced and the old nail holes never sealed up. This allows water to infiltrate into the top of the beam and causes the beam to decay from the inside out. One way to check for this is to use the resistograph and drill down through the decking and top of the beam to assess the top of the beam.

4) Collect data required for load rating:
   a. Total deck width and clear travel width
   b. Species and grade of beams and decking
   c. Beam type, size, and spacing
   d. Span length and bracing locations
   e. Deck type and size
   f. Moisture content of beams and decking
   g. Section loss of beams and decking

5. Load Rating

The Forest Service uses two main programs for load rating timber bridges: Timber Bridge Analysis and Rating Program (TBAR) [6] and Mathcad. TBAR (Figure 3) was developed for the Forest Service by HDR Engineering, Inc. It is a standalone program for load rating various types of timber bridges.

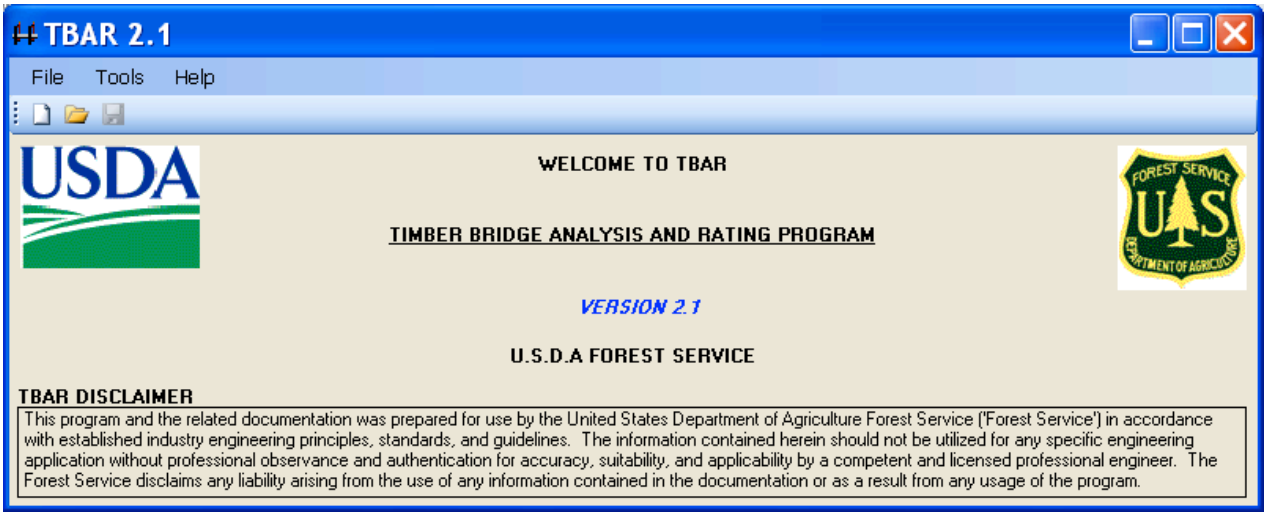


Fig. 3 Start-up screen for Timber Bridge Analysis and Rating program

The following bridge and deck types can be load rated in TBAR using load resistance factor design (LRFD) or allowable stress design (ASD):

Beam Type Bridges:

- Solid sawn
- Glued-laminated
- Native Log
- Steel girders with timber decks

Slab Type Bridges:

- Longitudinal glued-laminated deck
- Longitudinal nail-laminated deck
- Longitudinal spike- (or dowel-) laminated deck

Bridge deck types:

- Solid sawn planks
- Non-interconnected and interconnected glued-laminated panels
- Nail-laminated decks

TBAR works well for typical bridges that are in fair or good condition. For timber bridges that are in poor condition or were not constructed using a standard design, the Forest Service has developed numerous Mathcad worksheets (Figure 4) for these special circumstances.

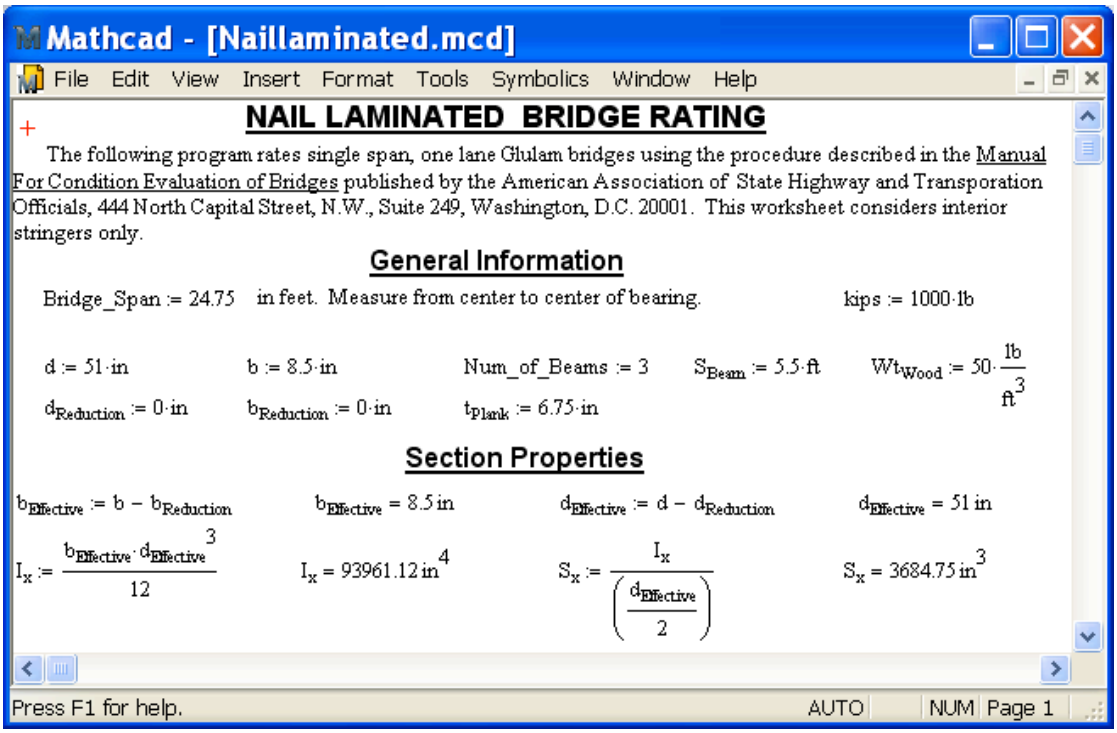


Fig. 4 Typical Mathcad worksheet for load rating

6. Conclusion

The Forest Service has been building timber bridges for more than 100 hundred years with most existing bridges being built 30 or more years ago. Due to the age of these structures, a good inspection and load rating process is required to maintain these structures and to ensure that they are safe for general public use. The Forest Service has accomplished this task by using the moisture meter, stress wave tester, and resistograph drill for in-depth inspections and Timber Bridge Analysis and Rating (TBAR) and Mathcad worksheets for load ratings.

7. References

[1] AASHTO. 2011. The Manual for Bridge Evaluation, American Association of State Highway and Transportation Officials, Washington, DC. 2011.

[2] James, William L. 1988. Electric Moisture Meters for Wood. FPL-GTR-6. Madison, WI: USDA Forest Service, Forest Products Laboratory.

[3] Ross, R.J.; Pellerin, R.F. 1994. Nondestructive testing for assessing wood members in structures: a review. FPL–GTR–70. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

[4] Brashaw, Brian K., Vatalaro, Robert J., Wacker, James P., Ross, Robert J. 2005. Condition Assessment of Timber Bridges 1. Evaluation of a Micro-Drilling Resistance Tool. FPL-GTR-160. Madison, WI: USDA Forest Service, Forest Products Laboratory.

[5] Brashaw, Brian K., Vatalaro, Robert J., Wacker, James P., Ross, Robert J. 2005. Condition Assessment of Timber Bridges 2. Evaluation of Several Stress-Wave Tools. FPL-GTR-160. Madison, WI: USDA Forest Service, Forest Products Laboratory.

[6] HDR Engineering, Inc. 2009. USDA Forest Service TBAR User Manual, Version 2.1.

Investigation of Glulam Girder Bridges Constructed Prior to 1970

James Scott Groenier, P.E.
Project Leader, Structures
USDA Forest Service,
Missoula Technology and Development Center
Missoula, Montana
jgroenier@fs.fed.us

Scott earned his BS degree from the University of Wisconsin and his MS degree from Montana State University, both in Civil Engineering. He has worked for the US Forest Service in several positions since 1992.

David Strahl, P.E.
Structures Engineer
USDA Forest Service,
Pacific-Northwest Regional Office
Portland, Oregon
dstrahl@fs.fed.us

David earned both his BS and ME degrees in Civil Engineering from Utah State University. He currently works on inspection and design of bridges for the Forest Service's Pacific Northwest Regional Office.

James P. Wacker, P.E.
Research Engineer
Forest Products Laboratory
USDA Forest Service
Madison, Wisconsin
jwacker@fs.fed.us

James focused his research work primarily on timber bridge inspection, durability, engineering, design and performance monitoring. He is the current chair of the ASCE Bridges Technical Committee.

Summary

The United States glulam industry began requiring high-quality, tension grade laminations in 1970 for the manufacture of beams designed primarily for bending applications. Glulam girders that were manufactured prior to that date, and remain in-service today, are currently subject to significant strength reductions during the bridge load rating process. This paper describes this issue as it pertains to an inventory of early glulam girder bridges, built primarily by the USDA Forest Service between the years 1950-70, on their roadway network. It describes a series of field bridge evaluations, a laboratory investigation of salvaged girders, and discusses the development of an alternative approach for load rating these unique timber bridges. The overall goal of these efforts was to preserve these early glulam girder bridges that are in danger of replacement even though they carried heavy logging trucks safely for decades, and in many cases, are still performing satisfactorily.

Keywords: tension grade, laminations, glulam, bridge, girder, load rating, strength reduction

1. Introduction

Historically, the U.S. Forest Service (FS) has used timber components in the construction of many of their highway bridges. There are approximately 4,100 bridges currently in-service on the National Forest transportation network that employs timber as primary bridge components. A large proportion of these bridges are comprised of glulam (glued laminated timber) materials that are manufactured using dimension lumber and structural "wet-use" adhesives. The current load rating process (AASHTO 2012) for glulam girder bridges contains significant changes that present new challenges to bridge engineers who must routinely assign safe load carrying capacities for bridges on public roadways.

A subset of these FS glulam girder bridges were manufactured prior to 1970, when the American

Institute of Timber Construction (AITC) first introduced a national standard for tension lamination quality. Without the specially-graded tension laminations, bending design values used for load rating are now required to be reduced by approximately 15% for beams 38 cm (15 in.) deep or less, and 25% for beams deeper than 38 cm (15 in.) (ASTM D3737). This translates into many of these structures having to be posted for reduced live load and slated for replacement, despite their satisfactory condition after 40-50 years in-service. The average replacement cost is estimated at more than $300,000 for structure, with several needing immediate replacement due to their vital service towards forest fire suppression efforts. As a result, further investigations were warranted before investing in significant bridge replacement costs. A field investigation of these bridges would verify their in-service performance and directly assess their load carrying capacity. A laboratory investigation of salvaged glulam beams would yield data on their ultimate bending strength values. The combined results would provide the basis for development of a new load rating strategy for assigning a safe and reliable load capacity to these bridges into the future.

A recent technical article by Powell 2004 spoke about the implications of this strength reduction for glulam beams in U.S. warehouse buildings constructed before 1970. A few years later the American Institute of Timber Construction released AITC Technical Note 26 (AITC 2007) for practicing engineers which provided more details about the strength reduction, but added some confusion for engineers. This reduction factor was initially thought to be duplicative with the glulam volume factor which was introduced beginning in 1991, and also presents a significant reduction for large glulam stringers (Moody and Others 1990). It was subsequently determined that both the volume factor and the pre-1970 strength reductions both apply to these early glulam stringers for load rating purposes.

The load rating of bridges in the United States is guided primarily by the American Association of State Highway and Transportation Officials (AASHTO). The Manual for Bridge Evaluation, AASHTO 2012 and AASHTO LRFD Bridge Design Specifications 2013, contain limited guidance for timber bridge ratings, but allow for diagnostic load testing to be performed as an alternative means of determining load capacity.

2. Objective

The primary objectives of this project were: 1) Conduct field investigations of remaining pre-1970 glulam girder bridges including live load testing; 2) Perform laboratory evaluations on glulam bridge girders salvaged from an Alaskan bridge; 3) Develop an alternative load rating strategy specifically for these older glulam bridges. The overall goal is to preserve these early glulam bridge structures that are being load restricted and slated for replacement, despite the fact they have been carrying logging trucks safely for decades and still performing satisfactorily in many cases.

3. Field Investigations

Two groups of glulam girder bridges were investigated in the states of Oregon and Washington during the 2009 and 2010 summer seasons. Static live load testing was conducted at all field bridges. Two different load cases were evaluated for each bridge. Load case 1 had the truck straddling the roadway centreline while its centroid was positioned at midspan. Load case 2 had the truck shifted laterally so that the wheel line was 0.6 m from the curb face. In all cases, data was collected before and after loading to detect residual movements. Supports were also checked to ensure there was no settlement under loading. Midspan deflections were collected with Celesco string potentiometers attached to the bottom side of the beams (Figure 1). Strain measurements were collected with transducers manufactured by Bridge Diagnostics Inc. Strain transducers were mounted on each girder at the midspan-bottom tension zone and at the intermediate support- topside tension zone. The data collection was automatically captured with a Campbell-Scientific CR-10 datalogger unit. In addition, suspended rulers and an optical level were used to collect midspan static deflections of the glulam girders (Figure 2)

In order to learn more about the quality of the tension laminations in these glulam girder bridges, an extensive field investigation was conducted at several field bridges. Core samples were removed to determine the wood species via microscopic analysis. Glue-joints and knot locations in the tension zones of the glulam girders was also recorded. Non-destructive testing was also performed to learn more about the internal integrity of the tension laminations at three bridges. Stress wave timing measurements were collected in two different orientations in the tension zone laminations using a

stress wave timer. The stress wave parallel-to-grain data was collected to correlate with stiffness values in the tension zone laminations. While the perpendicular-to-grain data were used to detect internal decay pockets. Resistance micro-drilling measurements were also taken at various locations in the tension laminations in order to gain a better understanding of their growth ring orientation and grade characteristics.

Figure 1. Live load testing data was collected via string potentiometers (deflection) and transducers (strain).

Figure 2. Optical level was also used to collect deflections from calibrated rules suspended from each girder.

4. Laboratory Evaluations

After the 15.2m (50 ft) long glulam girders were removed from service at the Jenny Creek bridge site near Kake, Alaska (Figure 3), they were transported to the Forest Products Laboratory in Madison, Wisconsin. Several laboratory investigations were conducted in order to reliably determine the residual capacity of these early glulam bridge girders. Visual defects were physically mapped, in addition to extensive scanning using stress wave timing device in a perpendicular to grain orientation (Figure 4). Static load testing was conducted (Figure 5) using 4-point bending arrangement in order to produce a uniform bending moment through the midspan section of the bridge girders. Testing was first completed to measure the modulus of elasticity of each beam. Then loading to ultimate failure was performed over a 10-minute load duration target period.

Figure 3. Pre-1970 glulam girders salvaged from site in Alaska.

Figure 4. NDE testing is performed in the laboratory to detect internal decay.

Figure 5. Static load testing was performed to determine member stiffness and ultimate load capacity of these 60-year-old salvaged glulam girders at the FPL Centennial Research Facility.

Testing results of four members showed that these pre-1970 manufactured glulam bridge girders had an average modulus of elasticity of 11,652 MPa (1.69 X 10^6 lb/in.^{2}) and failed at an average applied loading of 645 kN (145 Kips). These values are significantly higher than the original design stress level and suggest that the strength reduction factors may be overly conservative in some cases.

5. Modified Load Rating Approach

A modified load rating approach is currently being developed with several key components. A detailed on-site inspection will be a vital component of the new approach. Documentation of the type of finger joints utilized and the quality of the tension laminations will be very important. NDE tools will be employed to detect any internal decay or deterioration not obvious by visual assessment. It most likely will include an assessment of its live load performance under design loads. This general approach should allow the engineer to confidently assign a more accurate load rating than the existing bridge rating guides permit.

6. Summary and Conclusions

An extensive investigation was conducted in the field and laboratory settings to address a categorical strength reduction factor for glulam girders manufactured prior to 1970 in the US. These efforts demonstrated that a sampling of these bridges in the states of Oregon, Washington, and Alaska appear to be in satisfactory condition and capable to carry full design loads. A modified load rating approach is currently under development which could be applied to these unique structures reliably into the future.

7. References

[1] AASHTO. 2012. Manual for Bridge Evaluation, American Association of State Highway and Transportation Officials, Washington, DC. 2008.
[2] AASHTO. 2013. AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, Washington, DC.
[3] Powell, R. M. 2004. Allowable stresses for Glued Laminated Timbers. Structure, Vol. 11, No. 2. National Council of Structural Engineers Association.
[4] AITC. 2007. Technical Note 26 – Design values for structural glued laminated timber in existing structures. American Institute of Timber Construction.
[5] ASTM. 2008. Standard D3737 - Standard Practice for Establishing Allowable Properties for Structural Glued Laminated Timber (Glulam). ASTM International, West Conshohocken, PA.
[6] Bohannon, B.; Moody, R.C. 1969. Large glued laminated timber beams with two grades of tension laminations. FPL-GTR-113. Madison, WI: USDA Forest Service, Forest Products Laboratory.
[7] Marx, C.M.; Moody, R.C. 1981. Strength and Stiffness of Small Glued-Laminated Beams with Different Qualities of Tension Laminations. FPL-RP-381. Madison, WI: USDA Forest Service, Forest Products Laboratory.

8. Acknowledgements

This study was a cooperative effort between the following organizations in the U.S.D.A. Forest Service: Forest Products Laboratory, Missoula Technology and Development Center, and the Pacific-Northwest Region Engineering Team. Laboratory work was performed by staff of the FPL Engineering Mechanics and Remote Sensing Laboratory with assistance from Galen Reed (Northwest University Student) and Dr. Hudong Xu (Northeast Forestry University-China), and Dr. Junguo Zhang (Beijing Forestry University-China). William Salsig (retired FS bridge engineer) was responsible for planning and organizing the field investigations of the glulam girder bridges in the states of Oregon and Washington. The financial assistance of the Federal Highway Administration for surface transportation of salvaged glulam girders to the FPL is greatly appreciated.

Tretten Bridge – Timber and Steel in Harmony

John Are Hårstad-Evjen
Structural Engineer
Norconsult AS
Sandvika, Norway
jahe@norconult.no

Finished his M.Sc within structural engineering at the Norwegian University of Science and Technology in 2006. Has since worked with design of structures in road and railway projects.

Per Kristian Ekeberg
Senior Structural Engineer
Norconsult AS
Sandvika, Norway
yngve.aartun@plan.no

More than thirty years of experience as a structural engineer. Experience from design of large timber bridges during the last 20 years.

Trond Arne Stensby
Senior Structural Engineer
Norwegian Public Roads Administration
Hamar, Norway
trond.stensby@vegvesen.no

More than twenty years of experience as a structural engineer and is currently working as design manager for the E6 project.

Yngve Aartun
Architect
Plan Arkitekter
Trondheim, Norway
yngve.aartun@plan.no

Almost thirty years of experience as an architect, mainly within road projects. Has been involved in a number of timber bridge projects during the last 15 years.

Hilde Rannem Isaksen
Senior Structural Engineer
Norconsult AS
Sandvika, Norway
hri@norconsult.no

More than twenty years of experience as a structural engineer. In the last ten years working with timber bridges and maintenance of all type of bridges.

Bjørn Vik
Senior Structural Engineer
Norconsult AS
Sandvika, Norway
bv@norconsult.no

Over fifty years of experience as a structural engineer, mainly from design of structures in road and railway projects. Has been involved in the design of many timber bridges during the last 20 years.

Summary

The County's Hedmark and Oppland have a tradition of building timber bridges. Tretten Bridge is the last large bridge built suitable for traffic loads. The bridge is intended to have a 100 year life expectancy. This is achieved by creosote impregnation, constructive protection, detailing or a combination of these actions. As for the people at Tretten, a new bridge had long been anticipated and this bridge stands as a landmark not only to the locals but for everyone traveling in the area. The combination of timber and steel made it possible to give the truss a new and unconventional shape, which also improved the traffic situation considerably.

Keywords: Timber Bridge, slender shape, existing foundations, weathering steel.

1. Introduction

The village of Tretten is located in the valley Gudbrandsdalen about 30 kilometres north of Lillehammer, Norway. With the village centre and railway station located on each side of the river Gudbrandsdalslågen, the bridge built in 1894 was the only crossing point for the 900 inhabitants. In the 1980`s the single lane, 3 meters wide truss steel bridge was strengthened with tendons and widened 2 meters for pedestrian traffic. However, the increasing traffic loads, especially vehicles from the industrial area located on the western side of the river, made it necessary to build a new bridge.

2. The concept

As part of the main highway between Oslo and Trondheim, two of the largest cities in Norway, the planning of a 10 kilometres section between Tingberg and Tretten started in September 2003. Initially Tretten Bridge was a part of this development plan, but it was early decided to prepare a separate plan for the bridge only. Several meetings were arranged during the winter of 2003/2004 with the team consisting of architect, structural engineer and representatives from the Norwegian Public Road Administrations, NPRA. The team had to emphasized the aesthetics during the planning as bridge site was strongly exposed both from the highway and city centre.

During this phase several different alternatives were investigated. The Norwegian Water Resources and Energy Directorate, NVE, are restrictive when it comes to establishing new foundations in the river making a normal concrete bridge not an option because of the span length requirement. A combined steel girder and concrete bridge was possible but was excluded because of the aesthetics and the limitations in road geometry below and on the bridge. Remaining as the most obvious option was a truss bridge similar to the existing bridge. Having decided on the bridge system inspiration was gathered from other projects of this time.

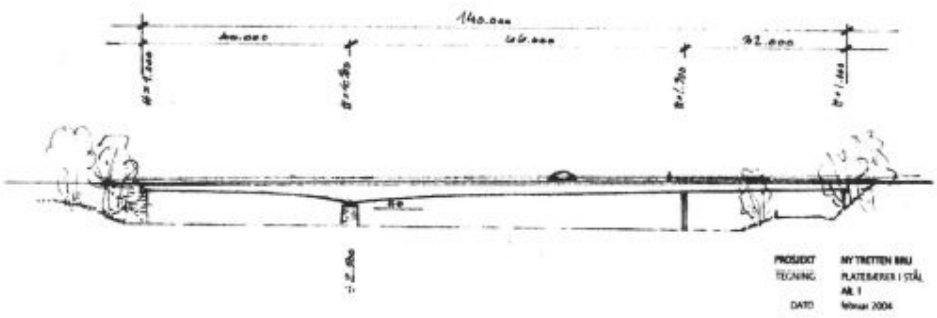


Figure 1 Steel girder and concrete alternative

In June 2003 the new Flisa Timber Bridge was opened for traffic. It was natural to continue the experiences gained during this rebuild. In many ways the two old bridges had a lot in common. They were built during the same period. They were both founded on stone foundations, that showed no signs of settling damages or other damages and were visually in great shape. The superstructure was steel trusses with a relative light timber deck. At Flisa the existing foundations were reused. A premise for reusing these foundations is avoiding an increase in weight of the superstructure and timber is a material often used to fulfil this requirement. Especially a stress laminated timber deck has turned out favourable with its light weight and high stiffness. Based on these experiences it was decided to build a new timber truss bridge on existing foundations at Tretten.

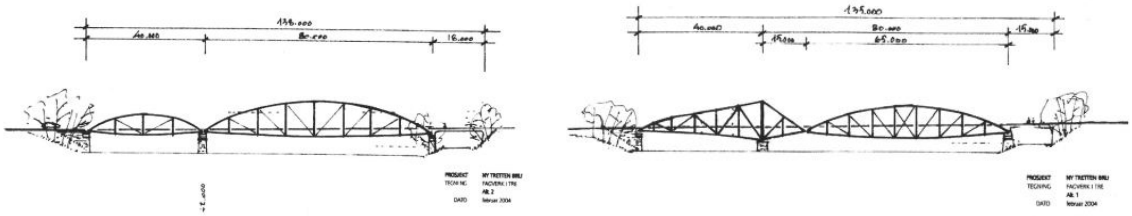


Figure 2 Timber truss alternatives

Traditional truss bridges, like the existing truss at Tretten with elevated wind trusses for each section, often give a tunnel sensation while driving through them. A continuous truss was expected to counteract this feeling, minimizing the need of wind truss. Also, the bridge deck was lifted up from beneath the truss and placed inside the truss. This lowered the height of the bridge without reducing the capacity of the truss.

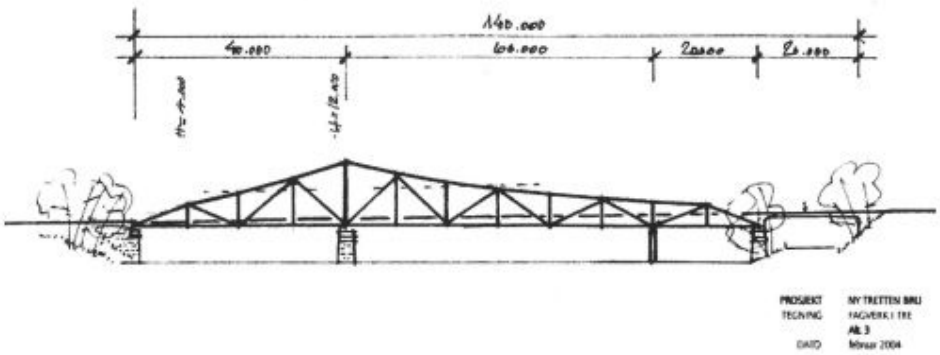


Figure 3 Continuous truss

The bridges also had some differences that influenced the shape of the superstructure. At Flisa all the existing foundations were reused and a similar truss system to the existing truss was designed. At Tretten only the foundations on the western side of the river could be reused. The abutment on the eastern side was partly damaged during a reconstruction in the 1980`s and it was early decided that a new foundation was needed. This resulted in an asymmetry in the foundations, old and "heavy" in the west and new and "light" in the east, and the architect wanted to reflect this asymmetry in the superstructure. The distinctive wing shape is strengthened by the asymmetry in foundations.

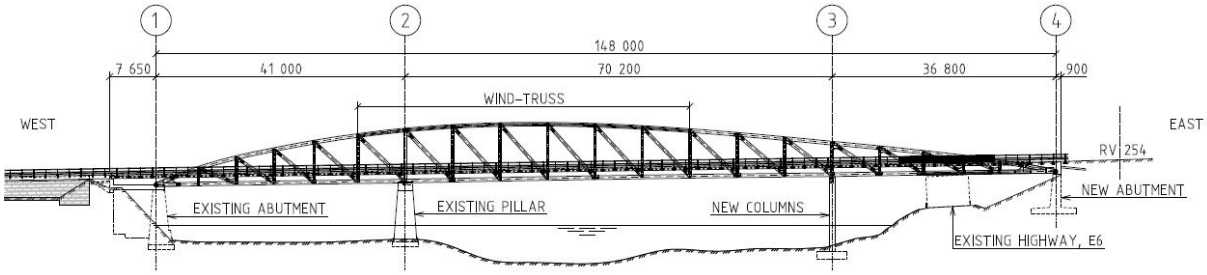


Figure 4 The final shape of the truss

The pointy end in the east was also favourable for the T-intersection at this side. The low height made it possible to start the widening of the carriageway somewhat inside the truss supports strongly improving the tracking for vehicles, especially larger trucks.

Also a big advantage with the truss shape was the lack of negative support forces at the ends. This simplified the detailing of the support as there was need to anchor the bridge.

3. Analysis

The Eurocode Design Code was introduced in November 2010 as the governing code for all new bridges in Norway. However, the design of the bridge started prior to this date and it was decided to continue using the Norwegian material codes, NS3470-1 [1], NS3472 and NS3473. The bridge is exposed to traffic loading defined in the Handbook HB185 issued by the Norwegian Public Roads Administrations, NPRA, [2]. Three concentrated axel loads, each of 210 kN, and a uniformly distributed load of 9 kN/m is applied to each lane of width 3.0 m. In addition, the pedestrian pathway is submitted to a uniformly distributed load of 4 kN/m. The traffic loads are applied both symmetric and asymmetric in the lateral direction on the 7.5 m wide driving area.

The bridge was analysed using the program RM-Bridge [3] and consists of the complete model of the superstructure, including the bridge deck and columns in axis 3. The foundations in axis 1, 2 and 4 were modelled as simple supports.

The predesign phase anticipated steel parts at the truss ends and at axes 2 and 4. However, with all cross beams located above the lower chord every connection between cross beam and vertical member had to be able to transfer moment forces. Common connection design using vertical members of wood or vertical steel hangers could not be used. Hence, all vertical members were carried out in steel.

4. Substructure

Minor modifications were done to the existing foundations in the west. The upper most stone layers were removed enabling space for a new concrete slab working as support for the new cross beams. Expansions of the bridge-width lead to cantilever cross beams used to support the new trusses. The concrete slab, cross beam and truss support in axis 1 is shown in figure 5.

Figure 5 Truss support in axis 1

Calculations showed that the existing foundation in axis 2 had sufficient capacity without further modifications. However, based on experiences gained at Flisa, steel piles under each bearing were installed to give extra capacity as a precaution.

The shape of the new columns in axis 3 was challenging for the architect. Reproducing the appearance of the old foundations was possible, but this construction method is out of date and not an optimal option. The solution was two slender columns diverging from the existing foundations and amplifying the asymmetry of the bridge.

The new abutment in the east was built to take into account for future expansion of E6. Also by widening the bridge deck with a cantilever concrete beam, no widening was done in the timber bridge deck.

Figure 6 Cantilever beam in axis 4

5. Superstructure

How did the shape of the truss influence the design of the superstructure?

All timber connections were done by embedded steel plates in sawn slots. The forces are transferred by dowels through the plates and timber. The timber deck consists of stress-laminated beams clamped together by pre-stressed high strength steel rods. These methods are thoroughly described in the article "Technical Concepts for Long Timber Bridges" presented at ICTB 2010 [4].

The bridge railing used was developed for timber decks solely and is used at most timber bridges built in Norway during recent years. The development was presented in the article "Development of a New Barrier System for Stress Laminated Road Bridge Decks" at ICTB 2010 [5].

Figur 7 Bridge railing at pedestrian side

5.1 Main truss

The main truss was carried out in glue laminated timber. The material properties of the timber are given in [1]. In general timber in strength class GL32c (GL = Glue Laminated, 32 = 32 MPa bending capacity, c = combined cross-section) was used.

The diagonal braces were individually designed with cross sections depending on the loads. Initially it was desired to use the same cross section for all diagonal braces, but the braces closest to the supports needed larger cross section. The difference in dimensions was barely noticeable on the new bridge.

The buckling capacity of the upper chord was expected to determine the dimensions because there was no wind truss over the lower points of the structure. However, timber has greater compression than tension strength and it turn out that the most critical area was above axis 3 where the chord has its maximum tension force. Also, substantial lateral moments occurred at this point as the only lateral stability in this area is given by the U-frame created by the verticals and cross beam. To reduce these moments the cross beams closest to axis 3 were made stiffer to give more lateral stability. Also the timber strength class changed to GL32h (h = homogenous) which has greater tension capacity compared to GL32c by about 15 %.

Figure 8 Crossbeam/vertical connection

At the eastern end the steel part was extended for the lower chord as the capacity of the timber was insufficient. Also the timber was changed to GL32h for the timber next to the steel.

5.2 Weathering steel

Weathering steel, also known under the trademark COR-TEN steel, was used in all visible steel parts. The colour of COR-TEN steel resembles the colour of creosote impregnated timber and this was an important reason for choosing COR-TEN steel in the main truss members.

The main advantage with using COR-Ten steel is the reduction in maintenance. As there is no need for surface treatment the construction period will be shortened and simplified. For conventional steel, surface-damages appearing during transport, storage and installation have to be repaired. These small damages are insignificant for COR-TEN steel as the protective patina will be restored on its own.

COR-TEN steel is more vulnerable to moisture and dirt compared to conventional steel. It is therefore important to emphasize detailing, especially avoiding areas where moisture and dirt might accumulate.

The selection in steel quality is somewhat limited for COR-TEN steel compared to regular steel. The best steel quality available is S355K2W, specified in NS-EN 10027-1 [6]. The Norwegian Process Code [7], issued by the NPRA, specifies the maximum thickness of the steel to avoid brittle fractures depending on the minimum temperature at the bridge site. With a minimum temperature of -40°C the maximum thickness is 35 mm. This had to be incorporated in the structure.

5.3 Lightning

The architect wanted to avoid the standard light pylons used to light up the road surface. These pylons would also interfere with the contour of the truss. Special armatures were developed exclusively for this project. These were fitted on the pedestrian side of the bridge and were designed to light up the road surface without blinding the drivers on the road below.

6. Fabrication and installation

October 2011 a new temporary bridge was constructed. This bridge was founded on embankments in the river. The embankments would also serve as construction site for the truss assembly and mobile crane arrangement. At wintertime the water flow in the river is less than during the rest of the year, and especially in the months of May and June when the spring floods arrives. As a consequence, the temporary embankments had to be removed before the middle of April. Early November the temporary bridge was finished and the demolishing of the old steel truss started.

All steel parts were prepared in Denmark before they were shipped to the Czech Republic and assembled. The finished steel parts were then shipped to Norway and the glue-lam factory Moelven AS at Moelv, 60 kilometres south of Tretten.

The size of the timber parts was limited to the size of the creosote impregnation tank, transportation and reasonable truss size during the lifting operation. Thus, the truss was divided into 6 sections giving a maximum timber part length of 23.4 m.

To secure the fitting of steel and timber members all parts were assembled on the floor in the factory in advance. The sections were coupled before they were disassembled and sent to the creosote tank. Finally the sections were reassembled at the bridge site and hoisted into position. The final truss section was installed in the end of April, shortly after the removal of the provisional bridge.

The last couple of months were used installing the parapet, membrane and asphalt, and finalizing other details. June 15th 2012 the new bridge opened, only 7 months after the old bridge was closed.

Figure 9 New bridge in the sunshine

7. Cost

Did the truss shape, with a relative high share of steel, influence the costs of the bridge? Will the price increase along with the amount of steel? As seen in the table below, this seems not to be the case. All prices are adjusted to 2013 values.

	Year	Length [m]	Width [m]	Area [m^2]	[NOK/m^2]
Evenstad	1996	180	6.5	1170	23 238
Flisa	2003	182	10,5	1911	26 297
Rena	2005	158	6,3	995	28 924
Tretten	2012	148	10.5	1554	26 810

Table 1 Compared prices

The difference between Evenstad in 1996 and Flisa in 2003 is thought to be from the increase in steel prices around the turn of the century. During the following years the prices have stabilized.

It is most natural to compare bridges at Flisa and Tretten as these have the most in common. As Tretten Bridge has more steel parts and also two new foundations it is somewhat surprising that the prices in reality are the same. The explanation might be in preparing the steel parts. The number of connections between steel and timber are considerably reduced as there is only one vertical steel member at each cross beam. The detailing and accuracy level at connections is an important cost factor and a reduction of these will compensate for the increase in steel.

The bridge at Rena is designed for heavy military vehicles and was, as might be expected, more expensive.

It is also worth mentioning that timber truss bridges are price competitive in comparison to steel truss bridges.

8. Discussion

The feedback on the truss shape and the amount of steel has varied. Some wondered why the bridge wasn't carried out in steel in its entirety. Others react on the unconventional truss shape with all diagonal braces pointing in the same direction.

As mentioned above the pre-design phase had several guidelines when determining the shape. The team was influenced by other projects at that time and Oppland County is a forestry county where timber has been an important building material. This combined with the benefits of the light deck made a timber bridge the best option.

The combination of timber and steel made it possible to shape the bridge as desired without any additional bracing at the axes, as done at Flisa. A structural engineer might think that the materials weren't utilized to its maximum, but the common person in the street wouldn't think about this.

And who are in majority?

9. References

[1] NS 3470-1, Prosjektering av trekonstruksjoner. 5. utgave juli 1999

[2] Statens vegvesen, Bruprosjktering. Håndbok 185. Oktober 2009

[3] Bentley, RM Bridge, Analysis user guide.

[4] Ekeberg P. K., "*Technical Concepts for Long Span Timber Bridges*", ICTB 2010

[5] LUND B. A. and PEZZUCCHI M. S., "*Development of a New Barrier System for Stress Laminated Road Bridge Decks*", ICTB 2010

[6] NS-EN 10027-1, Desination systems for steels. 2nd edition December 2005

[7] Statens vegvesen, Prosesskode 2, Standard beskrivelsestekster for bruer og kaier. November 2007

Condition Assessment of Iowa Timber Bridges Using Advanced Inspection Tools

Travis Hosteng
Research Engineer
Bridge Engineering Center
Iowa State University
Ames, IA United States
kickhos@iastate.edu

Travis earned his MSc degree from Iowa State University. He is the lead engineer for wood bridge projects at the BEC. He is also a member on the ASCE committee for timber bridges.

James Wacker
Research Engineer
Forest Products Laboratory
Madison, WI United States
jwacker@fs.fed.us

Jim earned his Civil Engineering degree from the University of Wisconsin. He is a registered professional engineer with more than 20 years' experience in timber bridge research.

Brent Phares
Director
Bridge Engineering Center
Iowa State University
Ames, IA United States
bphares@iastate.edu

Brent is the Director of Iowa State University's Bridge Engineering Center. He is regarded as an international expert in the field of advanced bridge evaluation technologies.

Summary

This paper reports the results of a study undertaken to assess the condition of timber bridges in the state of Iowa. This study was carried out as part of an overall national study spearheaded by the Forest Products Laboratory and FHWA to investigate the field performance of timber bridges throughout the United States. In the U.S., Iowa has the most timber bridge structures at more than 2,500. The inspection team utilized advanced NDE assessment tools such as the resistance micro-drill and other acoustic devices to evaluate the condition of the suspect areas of the timber sub- and superstructure elements. The three clusters of timber bridges were selected by the Forest Products Laboratory to ensure that they are representative of the various types of timber bridges in the state and matched other sets of bridges in other climatic regions of the U.S. Detailed results of the inspection conducted on these three bridge clusters in Iowa are presented. These results will provide a better understanding of the design, performance, and durability of timber bridges and their components in the state of Iowa.

Keywords: inspections; NDE, timber, bridge, field performance

1. Introduction

As many engineers begin to implement life-cycle cost analyses within the preliminary bridge design phase, there is a significant need for more reliable data on the expected service life of highway bridges. Many claims are being made about the expected longevity of concrete and steel bridges being 75 years or more, but few are based on actual performance data. Because engineers are least familiar with timber bridges, their expected longevity is typically conservatively estimated at 20-30

years. Additional research is needed on a national scale that provides more reliable data about the true longevity of timber bridges in the US and allows for more accurate life-cycle assessments.

In order to generate more quantitative and unbiased bridge performance data, the Federal Highway Administration (FHWA) recently launched a new initiative called the Long-Term Bridge Performance Program (LTBP). The LTBP is a national program and it includes detailed inspection, periodic evaluation and monitoring of approximate 200 bridges over a 20-year period. The LTBP program concentrates on "work-horse" highway bridges. This includes steel, reinforced and pre-stressed concrete bridges of stringer/multi-beam or girder, multiple box beam or girders and slab design types constituting 75% to 80% of the National Bridge Inventory (NBI). Under the LTBP program, a representative sample of bridges was evaluated in a cluster/reference bridge sampling method. Each reference bridge anchors a cluster of 5 or more bridges which are within a small geographical location (along a linear highway corridor or scattered about a geographic region approximately 30 miles wide) and subject to similar climate and traffic conditions. Evaluation of the reference bridges involves 1) detailed and arm-length visual inspection (VI); 2) advance nondestructive evaluation (NDE); 3) global testing (load testing, modal testing, and continuous monitoring); and, 4) destructive material sampling. The LTBP has developed an open, scalable, and extensive data management system called the "Bridge Portal". This database is capable of integrating LTBP data with other sources, such as NBI, PONTIS, weather, and traffic. Unfortunately, the LTBP currently does not include timber bridges in their current scope. A new research study aimed at generating more reliable bridge performance data and establishing a framework for long-term performance monitoring will be needed if timber bridges are to be incorporated into the LTBP in the future.

The following briefly discusses the three bridge clusters inspected in Iowa, an overview of the findings of those inspections, and discussion of the overall process.

2. Iowa Timber Bridge Inventory

2.1 Inventory Statistics

The state of Iowa has, according to the most recent NBIS listing, 2,454 timber bridges. Of those, approximately 189 are timber slab bridges, and the remaining are timber girder bridges. The timber slab bridges range in age from about 4-65 yrs, with the majority built after 1990; the inventory of timber girder bridges range in age from about 3 yrs to nearly 100 yrs old and the majority of those were built between 1940 and 1990.

2.2 Selected Clusters

Critical elements desired of the cluster bridges during the selection process included: a) bridge superstructure consists of timber as structural component, b) bridge located on public roadway, c) accessibility for safely conducting 'arm's length' bridge inspection, d) bridges within a cluster should be of same superstructure design type and preferably of similar age. Additionally, to simplify things logistically, once an age range of bridges was selected for a given bridge type, the list was further subdivided by county to make travel and inspection of the respective clusters more efficient. Three clusters of bridges were selected from the inventory of timber bridges in Iowa. Since timber girder bridges represented the largest percentage of timber bridges in Iowa, two of the three clusters were selected from that inventory, and the third cluster was selected from the timber slab inventory. A brief description of each cluster and its constituents is provided here, in addition, a sample bridge inspection from each cluster is also provided in Section 4 of this report.

2.2.1 Iowa Cluster Geography

The three Iowa clusters were located in Bremer, Blackhawk and Clarke Counties, which are highlighted in Fig. 1. Bremer and Blackhawk Counties are in the Northeast part of the state, and Clarke is in the South central part of the state.

Fig. 1. Iowa county locations for inspections.

2.2.2 Cluster 1 Details

The five timber bridges located in Bremer County, Iowa were all single span, timber girder bridges with transverse timber plank decks and were located on secondary gravel roads. All five of the bridges were built between 1994 and 1997 and the superstructure elements consisted of Douglas-fir timber treated with creosote. Each bridge consisted of between 25 - 35 timber girders measuring 102 mm (4 in.) x 406 mm (16 in.) and the timber planks were 102 mm (4 in.) x 305 mm (12 in.) which were nailed to the tops of the girders. All of the bridges had timber abutments consisting of timber piles, plank backwalls/wingwall and timber caps and timber guardrails. The ends of the girders were typically nailed to the timber plank backwall as well as toe nailed to the solid timber abutment caps. See Figs. 2 and 3 for a typical bridge layout.

Fig. 2 Timber girder bridge elevation view from Cluster 1 in Bremer County

Fig. 3 Timber girder bridge underside view from Cluster 1 in Bremer County

2.2.3 Cluster 2 Details

The five timber bridges located in Blackhawk County, Iowa were also all single span, timber girder bridges with transverse timber plank decks and were located on secondary gravel roads. One bridge, Blackhawk 3, was on an approximately 20 degree skew, all other bridges had no skew. All five of these bridges were built between 1983 and 1985 and the superstructure elements consisted of Douglas-fir timber treated with creosote. Each bridge consisted of 19 or 28 timber girders ranging in dimensions of 152 mm (6 in.) x 406 mm (16 in.) for the bridge with 19 girders and 102 mm (4 in.) x 406 mm (16 in.) for the four bridges with 28 girders. Timber planks on these bridges were typically 102 mm (4 in.) x 305 mm (12 in.) and were nailed to the tops of the girders. All of the bridges had timber abutments consisting of timber piles, plank backwalls/wingwall and timber caps. Four of the bridges had steel guardrails and the fifth, shown in Fig. 4, had timber rail posts with thrie beam rail. The ends of the girders were typically nailed to the timber plank backwall as well

as toe nailed to the solid timber abutment caps. See Fig. 5 for a typical bridge layout.

Fig. 4 Timber girder bridge elevation view from Cluster 2 in Blackhawk County

Fig. 5 Timber girder bridge underside view from Cluster 2 in Blackhawk County

2.2.4 Cluster 3 Details

The five timber bridges located in Clarke County, Iowa were all single span, longitudinal timber deck bridges with a single transverse timber stiffner beam located at midspan. All five of these bridges were built between 1982 and 1985 and the superstructure elements consisted of Douglas-fir timber treated with creosote. Each bridge consisted of nail-laminated timber plank sections made up of individual timbers approximately 76 mm (3 in.) to 102 mm (4 in.) x 357 mm (14 in.). The individual deck sections were then joined together with a longitudinal lap joint and galvanized dome head bolts. Gravel provided the wearing surface for four of the bridges located on secondary gravel roads; Clarke 1, which was located in a small town, was covered with an asphalt wearing surface. All of the bridges had timber abutments consisting of timber piles, plank backwalls/wingwall and timber caps, as well as timber guardrails and timber curbs with scuppers. See Figs. 6 and 7 for a typical bridge layout.

Fig. 6 Timber deck bridge elevation view from Cluster 3 in Clarke County

Fig. 7 Timber deck bridge underside view from Cluster 3 in Clarke County

2.3 Decay Hazard Zones

There are 5 decay hazard zones located throughout the United States, Zone 1 being the lowest risk, Zone 5 being the highest risk. The two clusters of bridges in Bremer and Blackhawk Counties are approximately on the boarder of Zones 2 and 3, whereas the bridge cluster in Clarke County is most likely a Zone 4 classification.

3. Inspection Methods

3.1 Pre-Inspections

Inspection of each bridge was a multi-phase process initiated in the office prior to field inspections. Once the bridges within each cluster had been selected, previous inspection records were searched for and downloaded from the Iowa DOT Structure Inventory and Inspection Management System (SIIMS). These inspection reports were carefully reviewed to obtain background knowledge regarding the history of the bridge, confirm dimensions, note any deficiencies, and gather as much preliminary information pertaining to the bridge as possible. Once the inspection reports had been combed through and the research team had confirmed that the subject bridges were quality candidates for full inspection, the county engineer for each cluster was contacted. Any additional information available from the county engineer was collected, and reviewed and permission to conduct a full inspection was requested (and graciously accepted by all entities). In addition, all counties involved were gracious enough to provide existing, or obtain new, photos of the bridges for use in the recon process and determination of access limitations.

3.2 Inspection Methodology

Inspection guidelines and recommendations along with details on specific information required to be collected during the inspection were outlined by the project advisory committee for all inspection regions. Below is a condensed summary of the tasks and information collected during all inspections:

- Bridge orientation, location
- Top side inspection (wearing surface, curb, guardrails, approaches, etc.)
- Label and photograph superstructure elements
- Collect additional required bridge photographs
- Conduct initial visual inspection (hammer sounding)
- Collect moisture content information
- Investigate areas of potential decay/damage with advance NDE equipment
    - Stress-wave timer
    - Resistograph
- Record all bridge dimensions, NDE evaluation locations, and areas of decay/damage
- Upload inspection data to spreadsheet/website database

Prior to investigating areas of potential deterioration, an area considered to be sound material was investigated to provide reference information. Each bridge was then initially, and thoroughly, visually inspected and hammer sounded as accessibility allowed. Areas of discoloration or exhibiting obvious signs of decay or damage were documented in the notes, and marked for further investigation using NDE tools including a moisture meter, stress wave timer and resistograph Areas highlighted for further investigation were first tested for their moisture content and then an investigation of the area/member was conducted with the stress wave timer. Those areas/members found to consist of critical amounts of damage and/or decay with the stress wave timer were then further investigated with the resistograph to provide detailed information regarding the extent of the deterioration.

4. Iowa Bridge Inspection Case Studies

This section briefly describes the inspection of one bridge in each of the three clusters in Iowa, namely the Bremer 2 Bridge, Blackhawk 5 Bridge, and the Clarke 3 Bridge.

4.1 Cluster 1 – Bremer 2 Bridge

As mentioned previously, the timber bridges inspected in Bremer County were all single span timber girder bridges, constructed in the mid 1990's. The Bremer 2 Bridge, hereafter referred to as Bremer 2, was located on secondary gravel road (160th St.) approximately 9.7 km (6 mi.) northwest of Waverly in Bremer County and provides access over a drainage ditch. The bridge is oriented east-west, and is approximately 1.2 km (0.75 mi.) west of Hwy 27. Bremer 2 is 6.4 m (21 ft.) from center-center of bearing and approximately 7.3 m (24 ft.) wide. The superstructure consists of 25 Douglas-fir timber girders which are 102 mm (4 in.) x 406 mm (16 in.) and Douglas-fir deck planks measuring 76 mm (3 in.) x 305 mm (12 in.), all of which were treated with creosote.

4.1.1 Visual Inspection Findings

Bremer 2 is situated in a shallow sag curve on 160th St. and has a gravel wearing surface, although there were small signs of asphalt on the curbs. The deck planks were found to be in good condition, all seated well on the girders, and had gaps between adjacent planks of approximately 3 mm (0.125 in.) to 13 mm (0.5 in.). The guardrails were constructed of untreated dimension lumber which had been painted and were connected to the bridge by simply anchoring the rail posts on top of deck planks that were cantilevered off the edge of the bridge, see Fig. 8. This guardrail detail is not vehicle-impact sufficient and the members are showing sign of deterioration.

Fig. 8 Bremer 2 guardrail detail

From below, Bremer 2 was generally in pretty good condition. The girders at the end-of-bridge (EOB) side were found to have slightly higher moisture content readings (14-15% vs 12-13%) towards the north side of the bridge. On the beginning-of-bridge (BOB) side of the bridge there were areas on the timber abutment cap that showed signs of moisture as well as piles of debris between girder ends. This indicates there may be gaps between the backwall and the bridge deck allowing debris and runoff from the approaches to fall through and collect on the top of the abutment cap. The abutment cap was originally covered with felt or tar paper, however over time this protective layer has become ripped, and is missing in various locations.

Bremer 2, like all the other Bremer bridges inspected, also had timber block diaphragms located at midspan between all girders. In addition, at midspan were steel tension rods (one above and below the girders) that ran the full width of the bridge. See Fig. 9 for photos of the timber diaphrams and steel tension rod details, respectively. Both the timber diaphragms and steel tension rods were in satisfactory condition on Bremer 2.

From a purely visual standpoint, aside from areas that appeared to have minor moisture problems, the only other noted area of concern was the tension side of girders G12, G13 and G14 which all had significant number of knots.

a. Exterior tension rod connection

b. Tension rod between girder and deck

Fig. 9 Steel tension rod on Bremer 2

4.1.2 NDE Findings

Hammer sounding and moisture content readings did not turn up any areas of significant concern that warranted further investigation with the stress wave timer or the resistance micro-drill. However several readings from both pieces of equipment were taken. Stress wave readings from girder G19 were approximately 58-60 micro-seconds, typical for sound timbers.

4.2 Cluster 2 – Blackhawk 5 Bridge

The timber bridges inspected in Blackhawk County were all single span timber girder bridges just like in Bremer County. However, these bridges were constructed in the early to mid 1980's. The Blackhawk 5 Bridge, hereafter referred to as Blackhawk 5, was located on Tama Rd., a secondary gravel road approximately 8 km (5 mi.) east of Reinbeck in Blackhawk County and provides access over a drainage ditch. The bridge is oriented north-south, and is approximately 0.5 km (0.33 mi.) east of Hicks Rd. Blackhawk 5 is 5.8 m (19 ft.) from center-center of bearing and approximately 7.3 m (24 ft.) wide. The superstructure consists of 28 Douglas-fir timber girders which are 89 mm (3.5in.) x 406 mm (16 in.) and Douglas-fir deck planks measuring 76 mm (3 in.) x 305 mm (12 in.), all of which were treated with creosote.

4.2.1 Visual Inspection Findings

Blackhawk 5 has a gravel wearing surface and steel guardrails composed of I-beam posts and thrie beam rails. Each rail post was connected to the bridge via a steel plate that sat on top of the first three girders under the deck panels along with bolts through the exterior girder, see Fig. 10. In addition, attached to the bottom of the guardrail posts were steel channel sections, flanges pointing down that extended the full width of the bridge from rail post to rail post. The channel was nailed to the bottom of all girders to provide lateral support for the girders. The deck planks were found to be in good condition, all seated well on the girders, and had gaps between adjacent planks of approximately 3 mm (0.125 in.) to 13 mm (0.5 in.).

Fig. 10 Bremer 2 guardrail detail

From below, Blackhawk 5 was generally in pretty good condition. Noteworthy items include: some girders were found to have settled under load approximately 3 to 6 mm (0.125 to 0.25 in.) at the EOB; shimming was present at EOB between a couple abutment piles and the abutment cap; G19 had a plywood shim placed between it and the deck planks over its entire length as well as section loss due to a large splinter at midspan that was approximately 3-3.4 m (10-11ft.) in length. Girder G25 had the most significant deterioration. Hammer sounding, in addition to the ease with which the moisture meter pins could be inserted into the member, quickly identified a problem area that began at approximately 0.7 m (2 ft.) from BOB and extended to just past midspan. This area was investigated extensively with the stress wave timer and resistograph, the results of which are summarized below.

4.2.2 NDE Findings

Stress wave and resistograph readings were conducted at numerous location on girder G25 of the Blackhawk 5 Bridge. A sample resistograph plot is presented in Figs. 11. The stress wave readings along the member, along with comparison of resistograph readings helped identify and bound the location of the deterioration in girder G25. Areas of the plot that flatline to zero relative resistance (y-axis) levels are indicative of advanced decay.

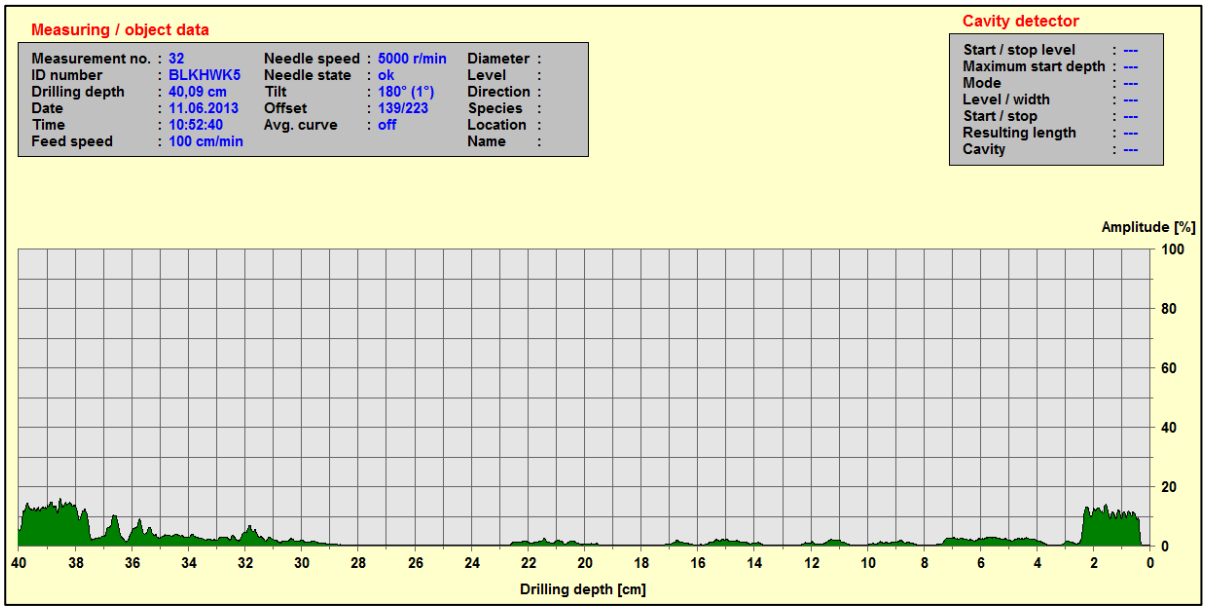


Fig. 11 Resistograph plot from girder G25, location 32, of Blackhawk 5

4.3 Cluster 3 – Clarke 3 Bridge

The timber bridges inspected in Clarke County were all single span timber deck bridges and were constructed in the early 1980's. The Clarke 3 Bridge, hereafter referred to as Clarke 3, was located on 250th Ave., a secondary gravel road approximately 4.8 km (3 mi.) northeast of Osceola in Clarke County and provides access over Otter Creek. The bridge is oriented north-south, and is approximately 1.6 km (1 mi.) south of Hwy 69. Clarke 3 is 9.5 m (31 ft.) from center-center of bearing and approximately 9.1 m (30 ft.) wide. The superstructure consists of nail laminated timber deck panels made up of Douglas-fir timbers each measuring 102 mm (4 in.) x 356 mm (14 in.). The nail laminated deck panels are then connected to each other with a lap joint and galvanized screws in predrilled holes. The bridge consisted of timber curbs with scuppers and timber guardrails, and all of the deck, rail and curb timbers were treated with creosote. Located at midspan of the bridge was a transverse stiffner beam that measured 152 mm (6 in.) x 305 mm (12 in.) and was attached to the deck with thru bolts.

4.3.1 Visual Inspection Findings

Clarke 3 has a gravel wearing surface that due to grading has all the scuppers partially to completely blocked. The most obvious signs of damage were at locations of physical connections, such as the guardrail post connections (see Fig. 12), and the longitudinal joint between deck panel sections where the lag screws have begun to push through the bottom of the deck and split the bottom of the member (see Fig. 13).

Fig. 12 Damage at connection detail between guardrail post and deck, Clarke 3

Fig. 13 Damage at longitudinal deck panel connection, Clarke 3

Other minor issues discovered in the visual inspection were that the EOB abutment cap was slightly rotated away from midspan and had slid back on the pile bearings slightly; in addition, the EOB abutment cap had checking and splitting on its interior face. Lastly, the transverse stiffener beam has signs of splitting/crushing at two of the interior thru bolt connection locations, but they were not significant at the time of this inspection.

4.3.2 NDE Findings

Overall moisture content readings were approximately 16-19% for both abutment caps, and the majority of the deck was in the range of 19-23%. These higher moisture contents in the deck are likely due to the fact that the gravel wearing surface is placed directly on top of the timber deck without any other separation or barrier.

Stress wave readings and resistograph readings at random locations revealed no areas of concern even around the areas were the deck was damaged/deteriorated due to the panel connection detail.

5. Challenges

For the three clusters of bridges in Iowa selected for inspection, very few challenges presented themselves. This is largely attributed to the research team conducting a thorough reconnaissance of potential bridge candidates prior to scheduling and conducting any inspections. This allowed for several candidate bridges, which had serious access issues, to be replaced with other bridges in the inventory which still met the needs of the project but provided better, and safer, access for the inspections.

Weather during the week most of the inspections were conducted was very cooperative and created no issues. It should be noted that only weeks prior to the time of the inspections, heavy spring rains had filled most of the streams, valleys and drainage areas that the subject bridges provide access over. Had the inspections been attempted 1-3 weeks earlier, access under most of the bridges would not have been possible due to high water.

No challenges in terms of unexpected wildlife presented themselves, aside from the one or two racoons found hiding between girder ends on an abutment cap, as well as the unavoidable mosquitos, ticks and barn swallows. However, preparedness in this regard (expect the unexpected) should not be overlooked.

6. Summary

In summary, the timber girder bridges in Bremer and Blackhawk Counties were typically rated between 6-7 (NBI Condition Rating) for the overall structural condition rating and, with the exception of two girders out of the five bridges inspected, all the bridges were found in good condition. The longitudinal timber deck bridges inspected in Clarke County were typically a 5-6 overall structural condition rating with most of the problem areas focused on the longitudinal nail-laminated panel joints and the abutments.

Initial visual inspection of the bridge components coupled with hammer sounding and moisture content readings provided adequate input on suspect areas of decay/deterioration for further investigation with the NDE tools (stress wave timer and resistance drill). Furthermore, the simplicity and output of direct, useful information from both pieces of equipment allows the inspector to quickly and accurately assess and document the location, extent, and bounds of any decayed/deteriorated areas.

7. Acknowledgements

The research and inspection team at Iowa State University would like to extend a special thanks to the following organizations and individuals for their contributions, assistance, and cooperation throughout the process of inspecting the three clusters of timber bridges in Iowa: USDA Forest Products Laboratory; Bremer County Engineers Office; Scott Meier, Bremer County Engineer; Blackhawk County Engineers Office; Catherine Nicholas, Blackhawk County Engineer; Clarke County Engineers Office; Thomas Andersen, Clarke County Engineer; Doug Wood, ISU CE Dept. Structures Lab Manager; and Tyler Muhlbauer, ISU undergraduate student.

8. References

[1] "Waverly, Iowa." 2013. Google Maps. http://maps.google.com. [June 25, 2013]

[2] "Waterloo, Iowa." 2013. Google Maps. http://maps.google.com. [June 25, 2013]

[3] "Osceola, Iowa." 2013. Google Maps. http://maps.google.com. [June 25, 2013]

Development of the Smart Timber Bridge

A Partnership between the Forest Products Laboratory and Iowa State University

Conceptual Smart Timber Bridge

- Glued laminated girders plus deck
- Measurement attributes
  - Structural adequacy
    - “Load” side via changes in load distribution, dynamic load allowance, and life usage.
    - Serviceability to monitor functionality and changes in stiffness (i.e., damage)
      - Delamination, vehicular collision, etc.
  - Deterioration
    - Moisture intrusion
    - UV-based degradation
    - Corrosion-related damage

Conceptual Smart Timber Bridge

- Sensors
  - Use off-the-shelf technology where applicable.
- Communication and reporting
  - Processed and communicated in near-real time.
  - Provide instant notification when important events occur.
  - Condition summarized in easy-to-understand reporting format.

Developmental Milestones

- Methodology for reliably attaching sensors to timber members.
- Reliable, long-term moisture sensor integration.
- Sensors to detect ferric ions.
- Sensors to detect degradation of wood lignin.
- Data processing techniques for determining structural adequacy parameters.

Developmental Milestones

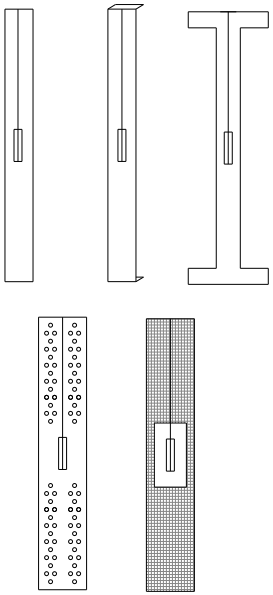
- Data processing techniques for determining changes in structural stiffness.
- Data processing techniques to determine vehicle characteristics
- Turnkey software.
- Demonstration

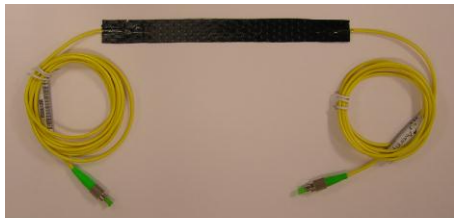
Development and Evaluation of Sensor Attachment Techniques

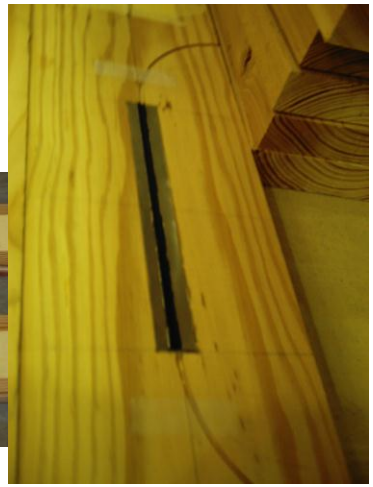
- Scope
  - Develop conceptual sensor packages
  - Small-scale testing
  - Selection of most promising techniques
  - Full-scale testing

Package Development

Specimens	*Adhesive*	*FBG Sensor Package*
Specimen 1	454 ™ Prism ®	CFPR and RS-SS
Specimen 2		CS-SS and IS-SS
Specimen 3		72H-SS and AS-SS
Specimen 4	426 ™ Prism ®	CFPR and RS-SS
Specimen 5		CS-SS and IS-SS
Specimen 6		72H-SS and AS-SS
Specimen 7	4212 ™ Prism ®	CFPR and RS-SS
Specimen 8		CS-SS and IS-SS
Specimen 9		72H-SS and AS-SS

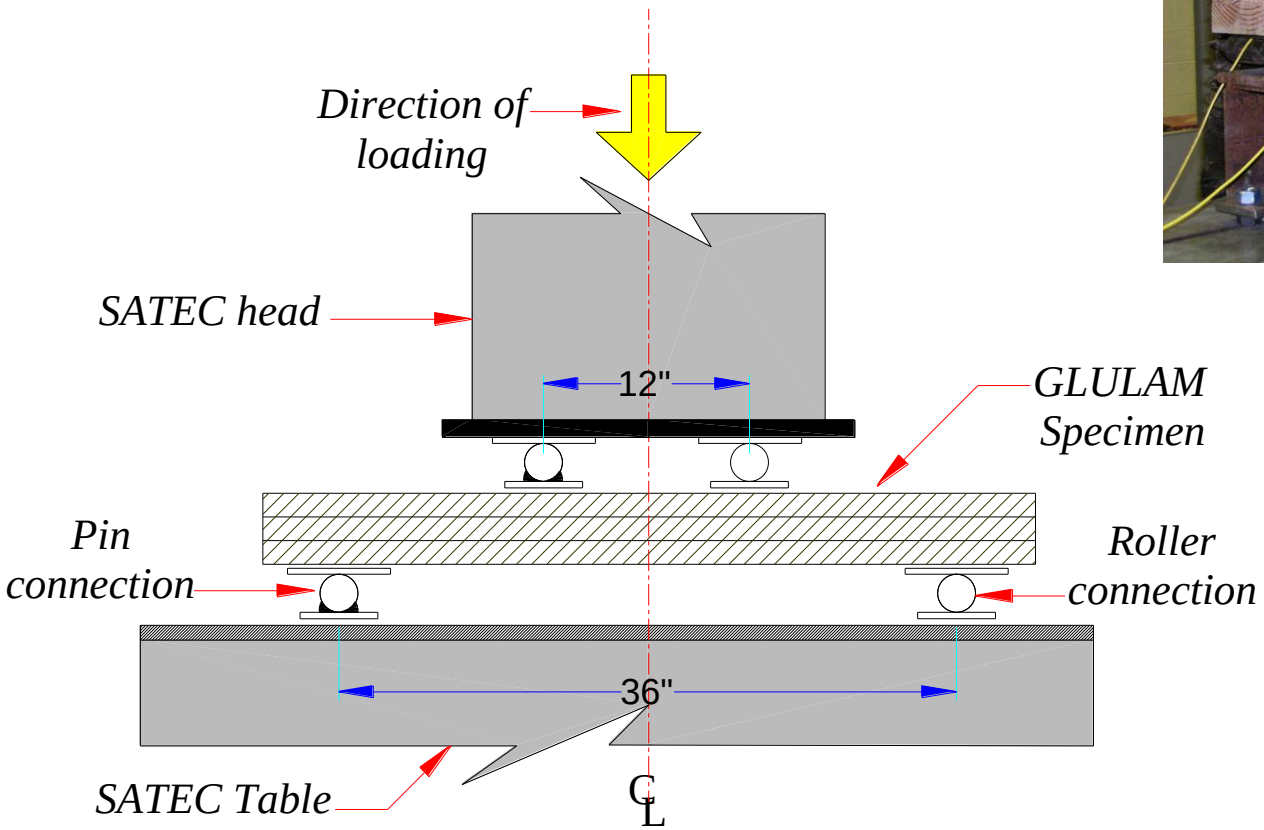


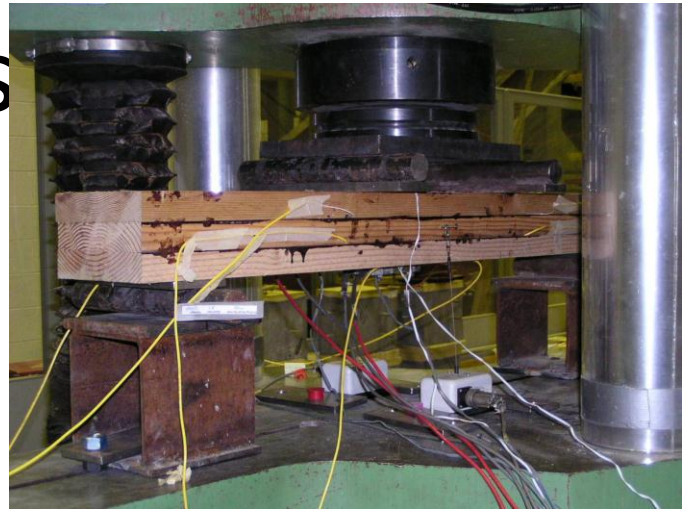


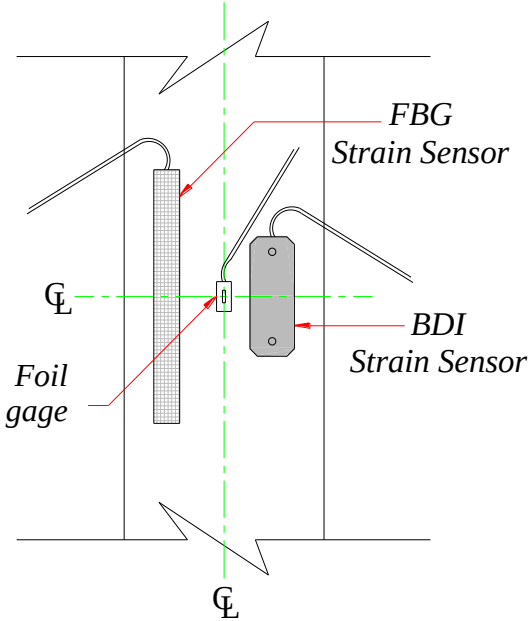


Small-scale Tes

2500 lbs



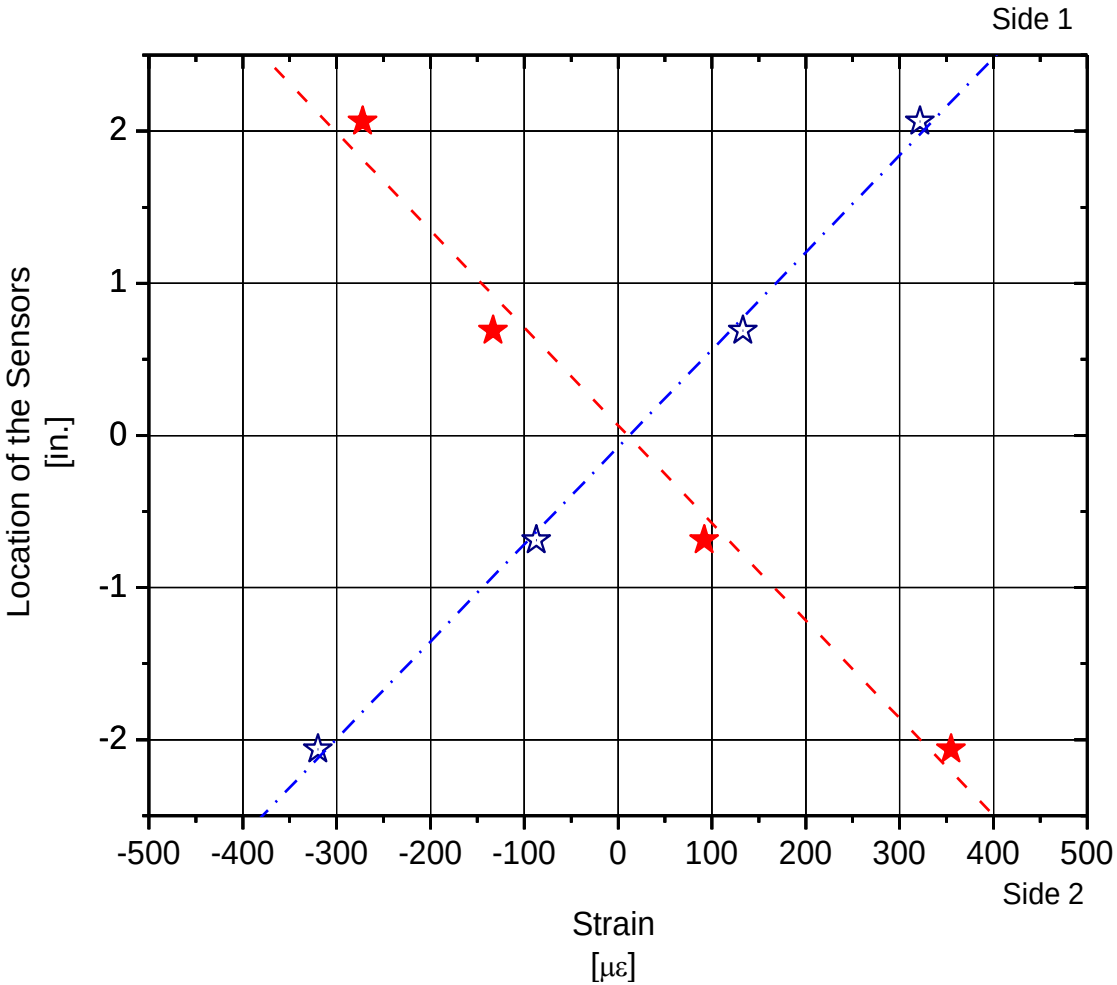


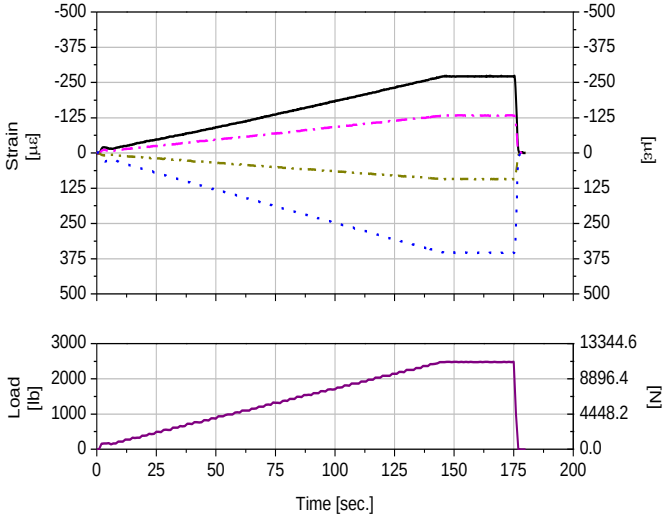


Test Program

- Bending
- Sustained load
- Accelerated load
- Psuedo cyclic load
- Heat and sustained load
- Cold and sustained load

Typical Bending Test

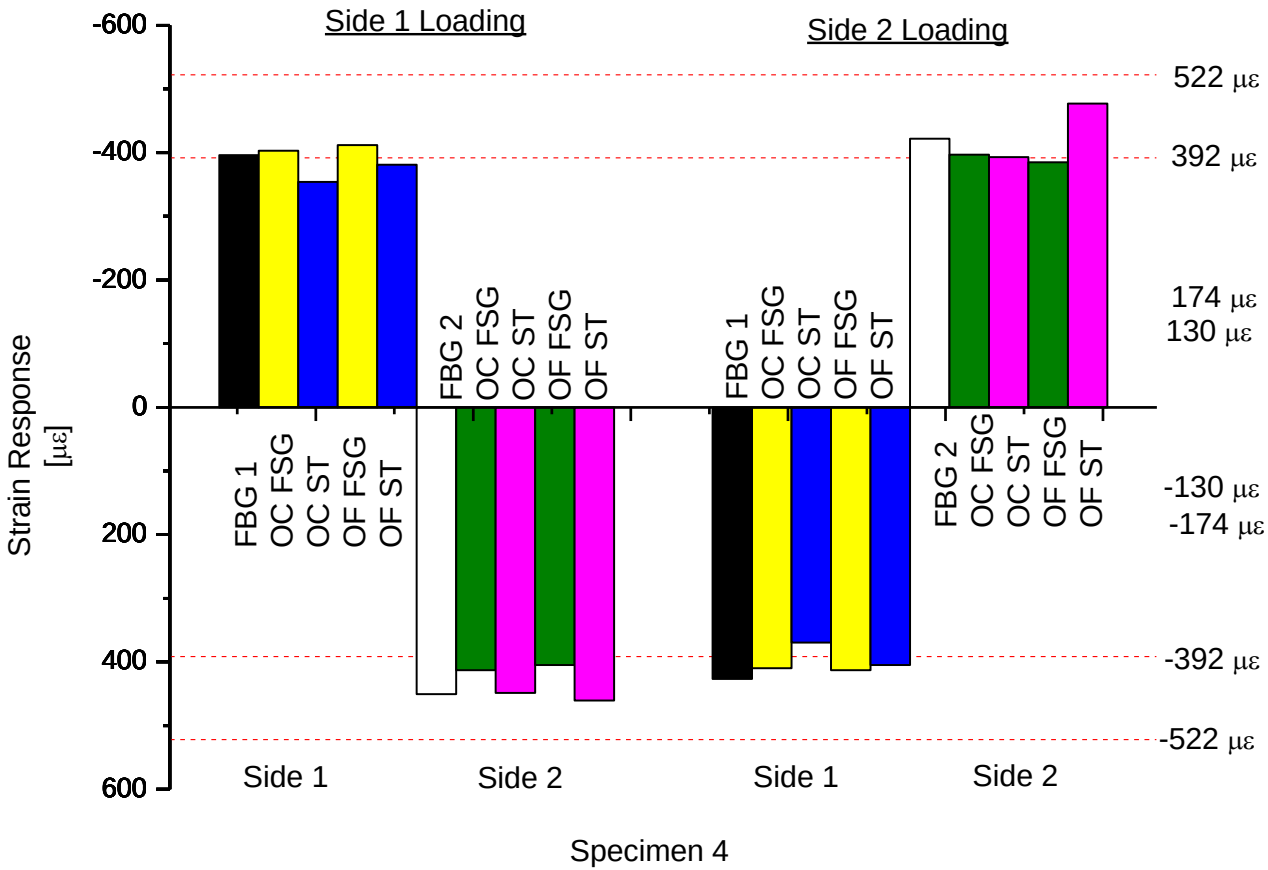




Specimen 1 - Adhesive Loctite 454

Package	Location [in.]	Side 1 [με]	Side 2 [με]
C-FRP	2.063	-272	322
C-FRP	0.688	-133	133
RS-SS	-0.688	93	-87
RS-SS	-2.063	355	-320

Sensor Comparisons



FBG 1 or 2: FBG Structural Package Type 1 or 2
OC FSG: On center Foil Strain Gage
OC ST: On center Strain Transducer
OF FSG: Off center Foil Strain Gage
OF ST: Off center Strain Transducer

---Theoretical External Upper and Lower Bound

General Conclusions

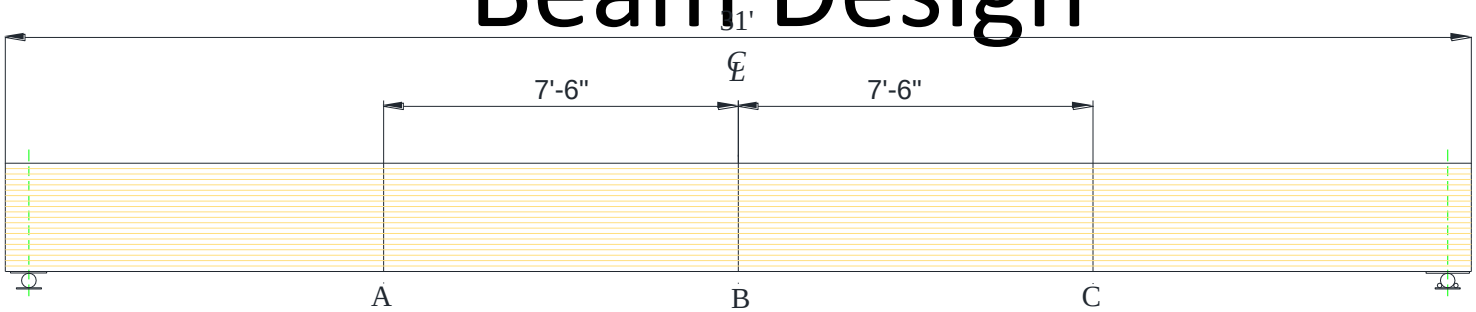
- Techniques for connecting sensors to timber can be achieved using proper adhesive and package design.
- Fiber optic sensor are not robust enough to withstand glulam fabrication processes.
  - Recommended to use traditional sensor types.

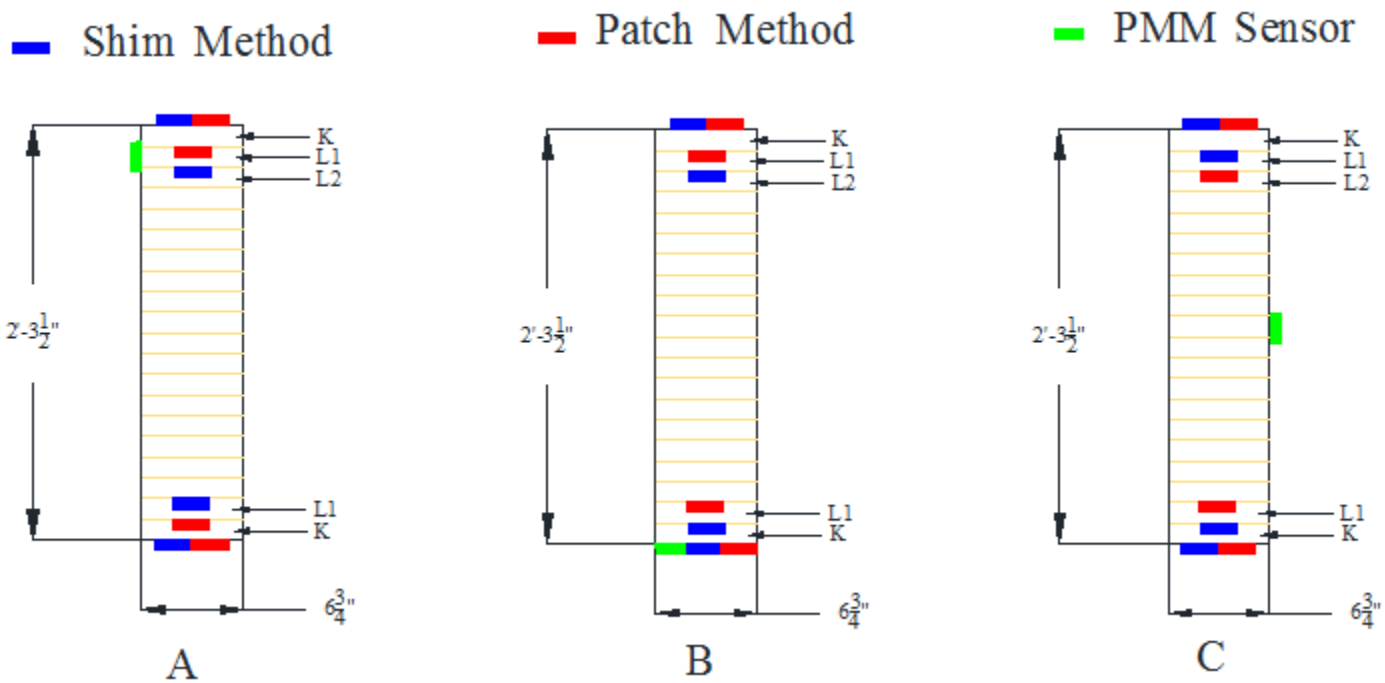
Full-scale testing

- More successfully integrate sensors into the fabrication process.
- Evaluate operability under repeated loadings.
- Also…investigate one additional attachment techniques – the patch method.

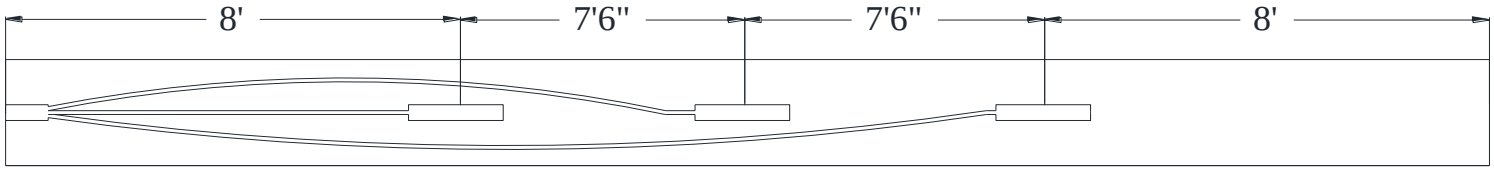
Shim and Patch

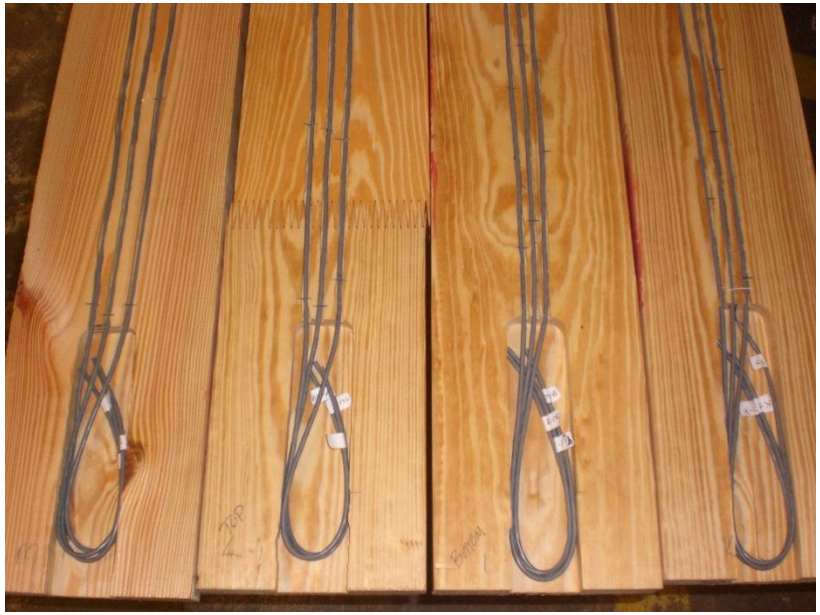
Beam Design


31'
℄
7'-6"
7'-6"
A
B
C


Shim Method
Patch Method
PMM Sensor
K
L1
L2
2'-3½"
L1
K
6¾"
A
B
C

Fabrication





Fabrication

a.) Aligning the laminates in the clamping aparatus

b.) Checking the gages with a volt meter

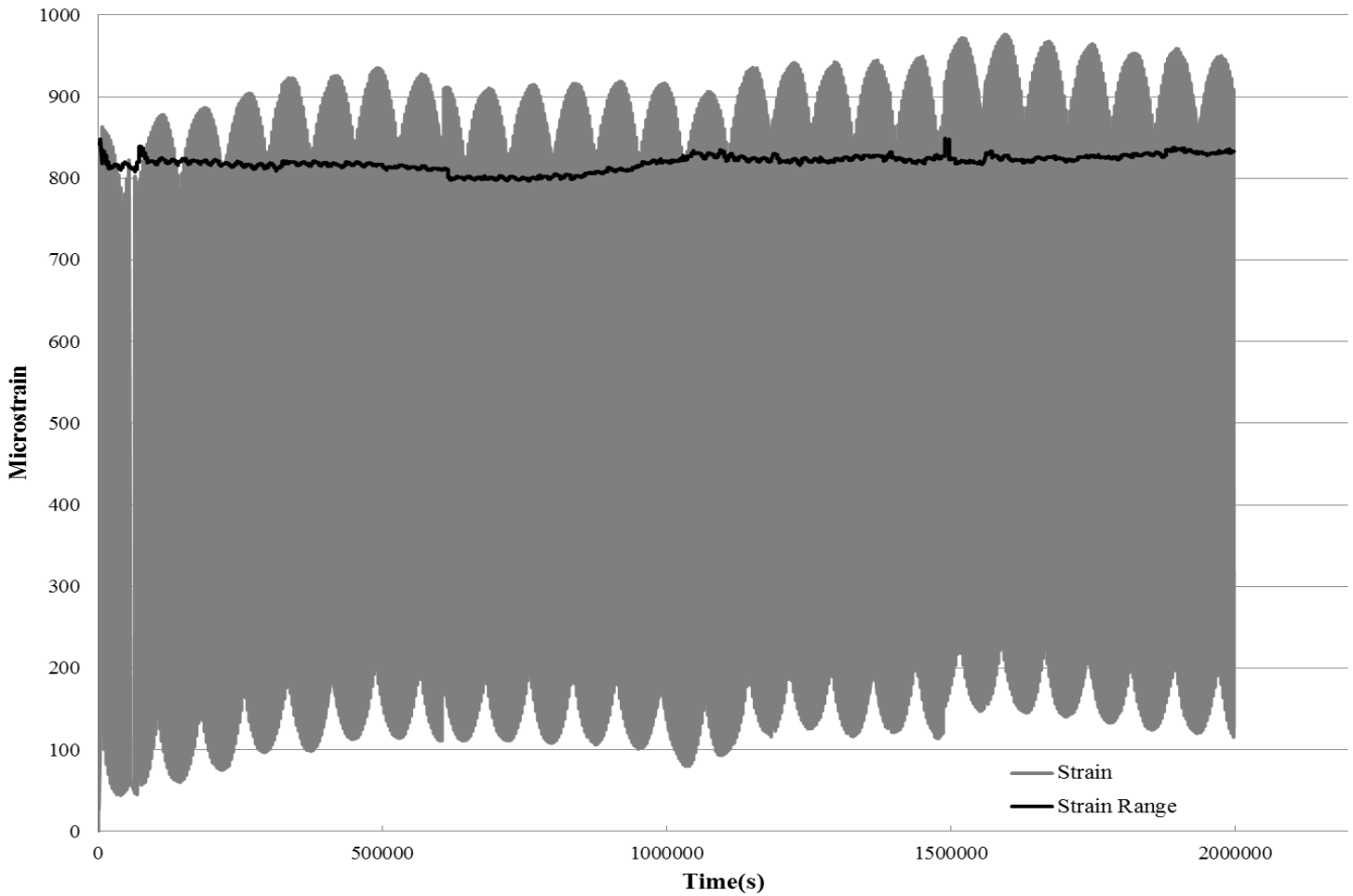
c.) Planing the beam

d.) Finished beam

Fatigue Testing

- 1,000,000 cycles at 0.5 Hz
- Two point loading method—24,000lb peak load

Typical results



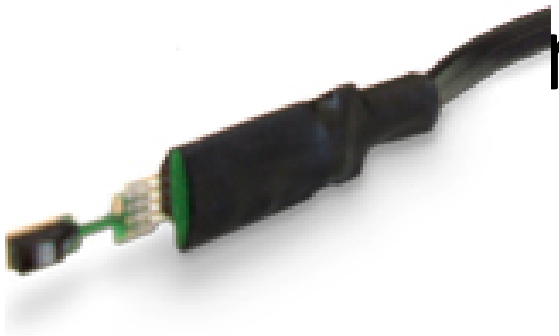
Results

	Sensor	Theoretical Peak Strain (microstrain)	Experimental Peak Strain (microstrain)	Percent Difference (%)
Shim Method	Top-A	705	512	27.4
	TL1-A	622	564	9.3
	K-A	686	685	0.1
	Bottom-A	705	819	16.1
	Top-B	987	811	17.9
	TL1-B	871	803	7.8
	BL1-B	808	769	4.8
	Bottom-B	987	867	12.2
	Top-C	705	638	9.5
	TL2-C	584	437	25.1
	BL1-C	611	604	1.1
	Bottom-C	705	645	8.5
Patch Method	Top-A	705	606	14.1
	Top-B	987	865	12.4
	Bottom-B	987	948	4.0
	Top-C	705	628	11.0
	Bottom-C	705	710	0.7

Moisture Sensor

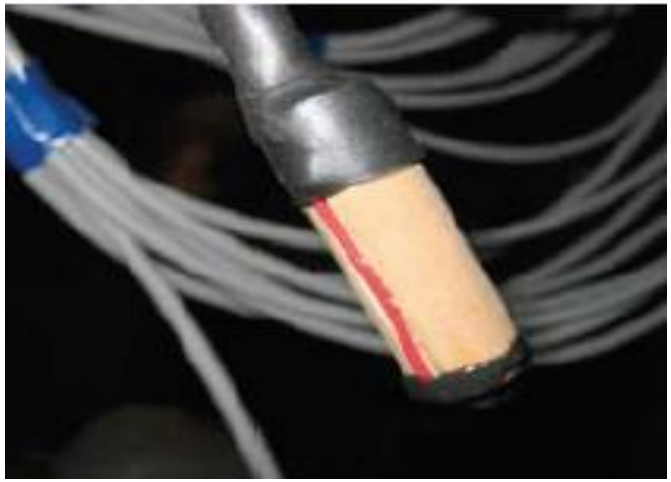
- Evaluate (and select?) a sensor for assessing changes in moisture content.
- Required to be able to be wired into off-the-shelf data acquisition systems.
- Reasonable accuracy in the range of 20% moisture content.

rs Cons



Relative humidity/Temperature

PMM Sensor

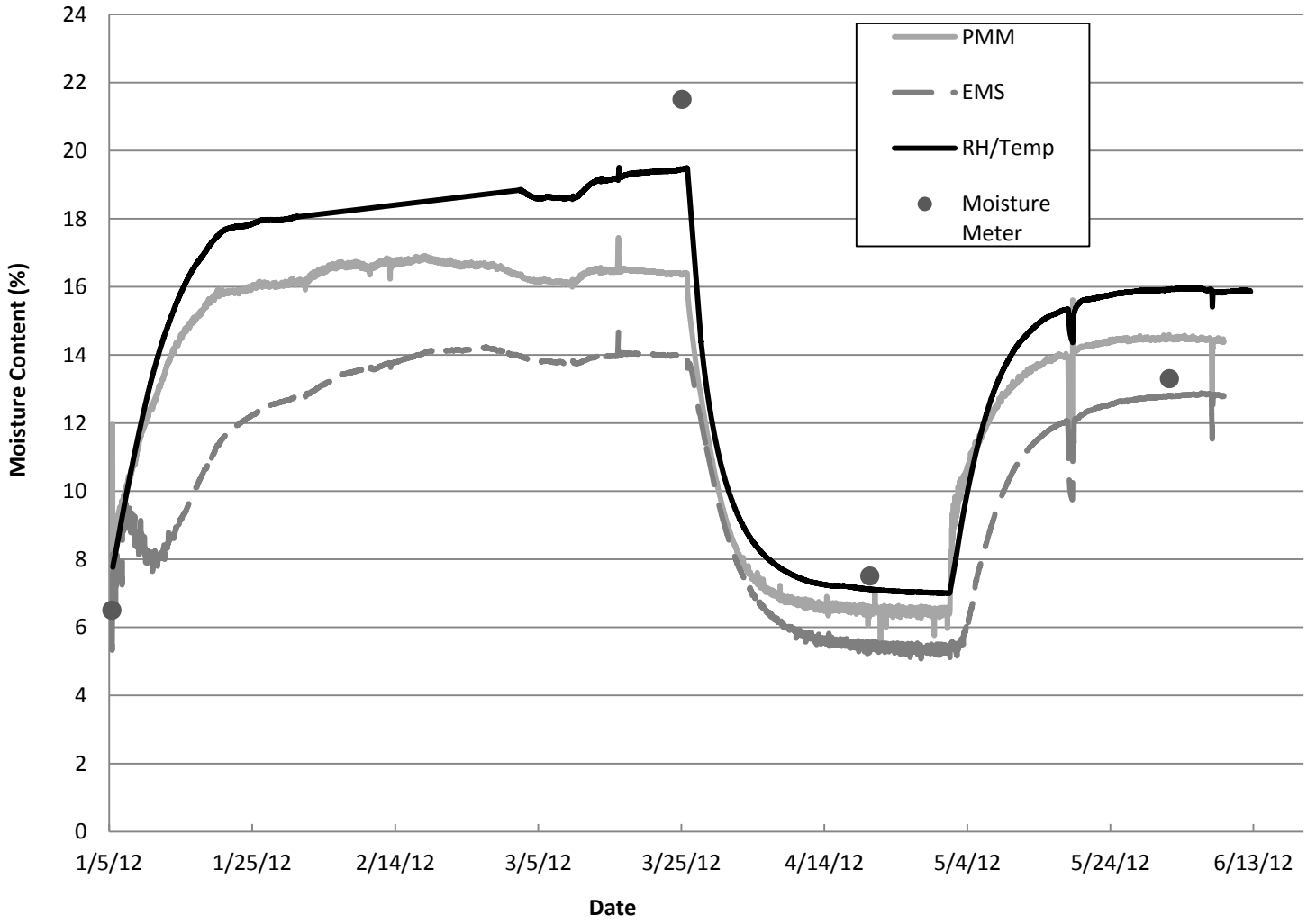


EM Sensor (EMS)

Testing

- Four 6”x6”x2” SYP specimens
- All three sensors installed
- Tested under various relative humidity and temperature conditions

Typical Results

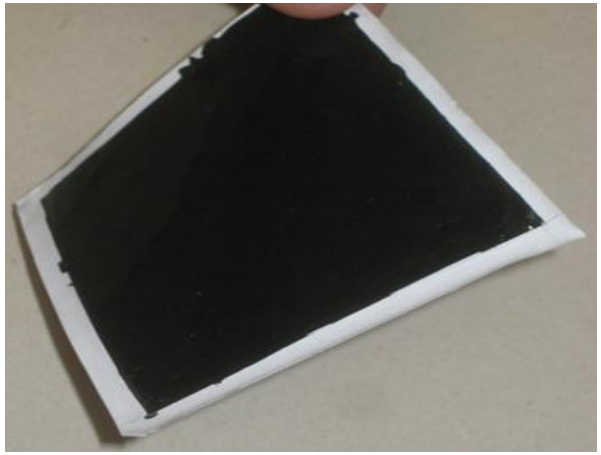


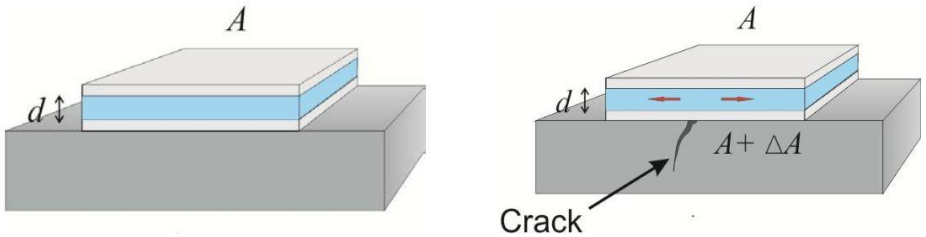
Conclusions

- PMM selected as most promising measurement approach
  - Successfully connected to traditional data loggers.
  - Simple installation procedure.
  - Responded well to varying conditions.
  - Reasonable accuracy.

Vehicle Characterization

- Evaluate the efficacy of a soft elastomeric capacitor
  - Large scale surface strain gauge
  - Based on capacity sensing.
  - Sensitivity depends upon permittivity of medium and thickness of elastomer.

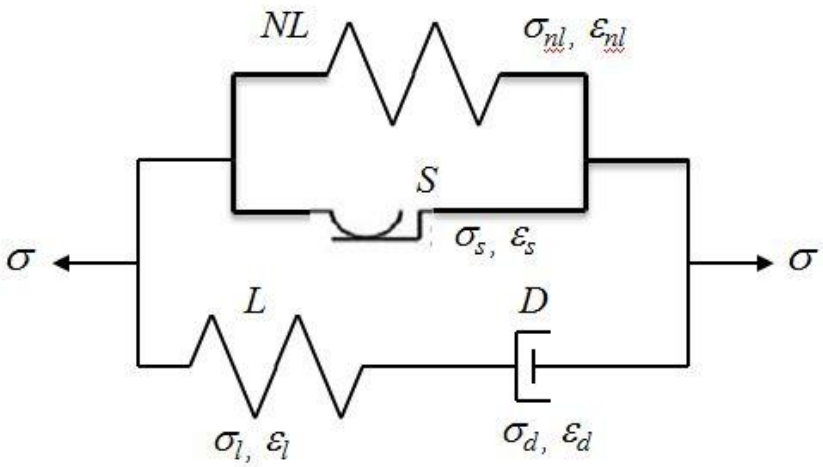




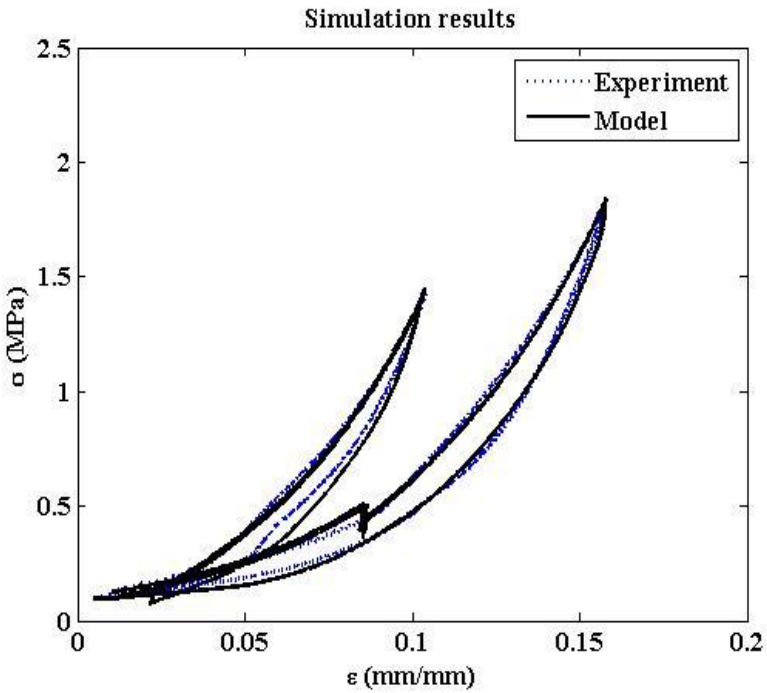
Capabilitites

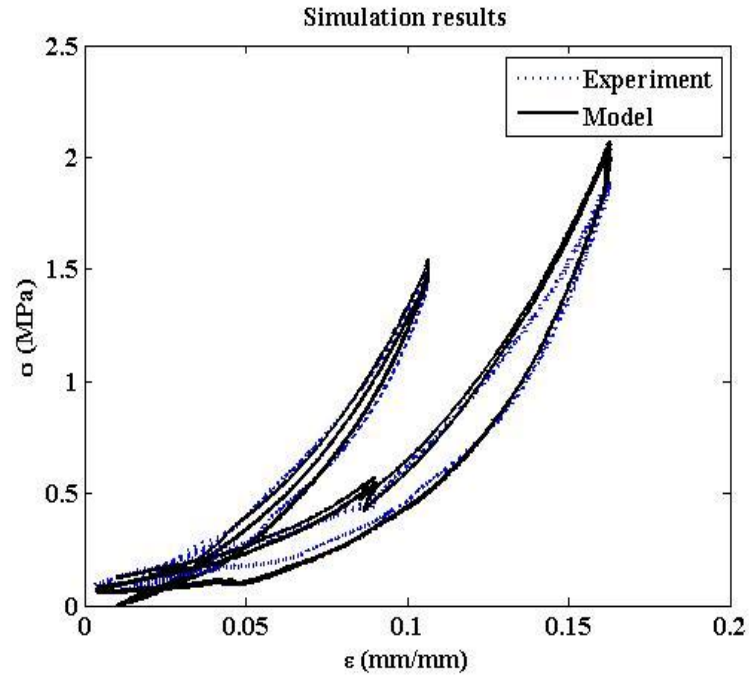
- Detect strain and surface cracks.
- Simplicity in shape – improved durability.
- Damage resistant.
- Possibilities of reconstructing deflection shapes.
- Comparable accuracy with traditional strain gauges.

Development of Rheology Model



Impact of Select Patch Characteristic

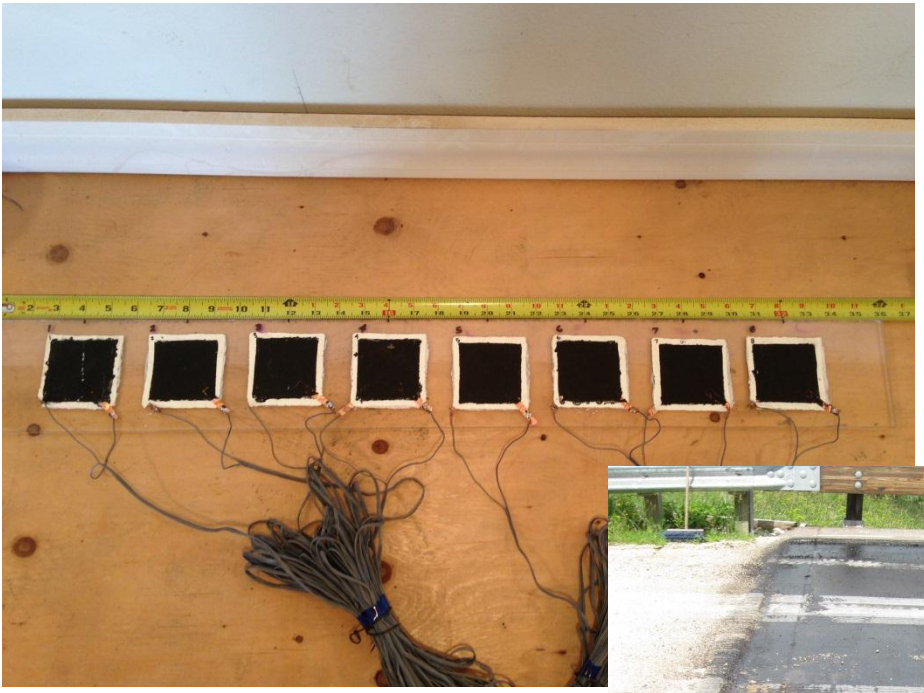




Conclusions

- Constitutive model developed.
- Effectively able to estimate observed stress with low errors.
- Developed proposed model ready for field trial.

Field Trial



Field Trial

Concluding Remarks

- Significant advances have been made in the development of a timber bridge that can report on its own condition and health.
- Using a combination of conventional and cutting-edge sensors.
- Combined with robust data processing techniques and a strong communication backbone.

Questions?

Thank You!

Live Load Testing of Historic Covered Timber Bridges

Travis Hosteng
Research Engineer
Bridge Engineering Center
Iowa State University
Ames, IA United States
kickhos@iastate.edu

Travis earned his MSc degree from Iowa State University. He is the lead engineer for wood bridge projects at the BEC. He is also a member on the ASCE committee for timber bridges.

James Wacker
Research Engineer
Forest Products Laboratory
Madison, WI United States
jwacker@fs.fed.us

Jim earned his Civil Engineering degree from the University of Wisconsin. He is a registered professional engineer with more than 20 years' experience in timber bridge research.

Brent Phares
Director
Bridge Engineering Center
Iowa State University
Ames, IA United States
bphares@iastate.edu

Brent is the Director of Iowa State University's Bridge Engineering Center. He is regarded as an international expert in the field of advanced bridge evaluation technologies.

Summary

The National Historic Covered Bridge Preservation Program (NHCBP), sponsored by the Federal Highway Administration (FHWA), is intended to preserve covered timber bridge structures nationwide. Today, less than 700 covered timber bridges still exist in the United States and of those many are closed to vehicular traffic. Furthermore, a large percentage of the remaining bridges open to traffic are restricted by load postings. Currently, no load rating standards exist for covered timber bridges which often results in overly conservative load ratings/postings. Rating engineers simply do not have adequate resources or background in timber design, especially in covered timber bridge design, for calculation of accurate load postings on covered timber bridges. To better understand the structural performance of covered timber bridges and ultimately develop improved criteria for load rating of these bridges, a series of field tests were performed on eleven historic covered timber bridges. Field live load testing was conducted on three Burr arch truss bridges in Indiana, four Howe truss bridges in Indiana, and four Queen post truss bridges in Vermont. This paper discusses the live load testing of the Cox Ford Burr arch Covered Bridge in Indiana. Included in the discussion is the procedure for installing displacement and strain sensors on the bridge, monitoring those sensors for global displacements and member strains at various cross-sections during passage of a known test vehicle, the results from the load test, and information on how these field tests were used to develop and calibrate an analytical model. The analytical model could then be used for determining more accurate bridge load ratings of the bridge. We believe this work will result in a viable technique for determining reliable load ratings for covered timber bridges using field tests and more accurate computational modeling techniques.

Keywords: Covered Bridges, Timber, Load Rating, Burr arch, Queenpost.

1. Introduction

The Federal Highway Administration (FHWA), in partnership with the USDA Forest Products Laboratory and the National Park Service (NPS), has sponsored a comprehensive national research program on Historic Covered Timber Bridges in the USA. The main purpose is to develop improved methods to preserve, rehabilitate, and restore the timber bridge trusses that were developed during the early 1800's, and in many cases are still in service today. The overall goal of the National Historic Covered Bridge Preservation Program (NHCBPP) is to preserve these iconic bridge structures for future generations. The study described herein is part of the NHCBPP and is aimed at establishing a procedure for safely and reliably load-rating historic covered bridges through physical testing. To accomplish this goal, live load testing was conducted at three Burr arch truss bridges in the state of Indiana, four Queenpost truss bridges in the state of Vermont, and four Howe truss bridge in Indiana. The field testing results have formed the basis for improved instrumentation, load testing, and load rating of similar historic covered bridges in the future. This paper provides an overview of the field testing methods, results, model calibration and a brief overview of how the calibrated analytical model may be used for load rating procedures for the Cox Ford Burr arch Bridge in Indiana.

2. Background

Often, given the age and complex behaviour of timber bridges and especially covered timber bridges, they are assigned relatively low ratings. It is also widely known that, when tested, most bridges are found to possess structural characteristics that supersede those assumed for the assigned ratings which were determined using prudent engineering judgment. In general, these behaviours result from additional, unaccounted-for stiffness and improved load distribution characteristics. Although testing procedures have been established for conventional bridges, no such procedures have been established for historic covered bridges. Given the historic nature and unusual geometric features of these structures, a procedure needed to be established detailing how to safely and reliably determine load ratings for historic covered timber bridges through physical testing. Furthermore, the developed testing and rating procedure needed to be simple and general in nature such that practicing engineers have the ability to quickly and accurately analyze and assign safe load limits to covered bridges in their inventory with basic, off the shelf analysis software.

3. Methodology

In order to develop testing and rating procedures for historic covered timber bridges that are both accurate and easily applicable by practicing engineers, physical load tests were conducted on three groups of covered bridges with typical truss configurations, namely Burr arch, Queenpost and Howe trusses. These three truss configurations were selected because they have a significant population of structures surviving today. The physical load tests provided invaluable structural performance data, in addition to modeling and calibration parameter information for the basic 2D analytical models of the trusses for calculating load rating.

4. Cox Ford Burr arch Covered Bridge - Indiana

During October of 2010, three Burr arch covered timber bridges, Zacke Cox, Cox Ford and Portland Mills Bridges, were evaluated and tested in the State of Indiana. These bridges were all located in Parke County, IN, which maintains over 30 historic covered bridges within their roadway network. These single lane bridge structures are all currently restricted to vehicle weights below current legal loads, but still provide vital transportation links to rural communities in the western part of the state. Approximately 93 covered bridges exist within Indiana. The following outlines the testing and modeling of the Cox Ford Bridge. The Cox Ford Bridge (County Bridge No. 227) is located on Cox Ford Road (unpaved) and carries traffic over the Sugar Creek just west of Turkey Run State Park. An end view photograph of the bridge is shown in Figure 1.

Fig. 1 Interior view of the Cox Ford Bridge showing the Burr arch truss configuration

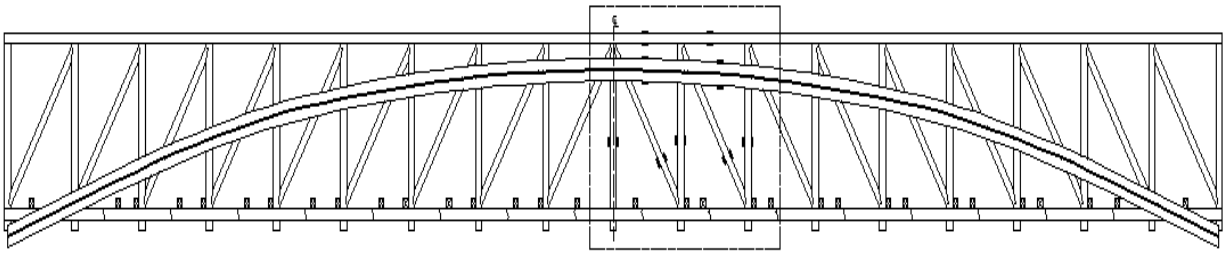
The Cox Ford Bridge was originally built in 1913 to replace an iron bridge and has been rehabilitated twice previously in 1975 and 1991. The structure is a one lane, single span, simply supported double Burr arch truss with a total length of 58.5 m (192 ft.). The bridge measures 52.9 m (173.5 ft.) from center-to-center of abutment bearing and the width of the bridge is 4.8 m (15.9 ft.) measured from out-to-out of the bridge deck. The total height from the bottom of the bottom chord to the top of the top chord was measured to be 5.4 m (17.8 ft.) and the average truss panel spacing is 3 m (9.75 ft.). The trusses consist of rectangular parallel chords sandwiched by double concentric arches, two member bottom chords with nine single-headed hook fishplate and iron shoe splice joints, single member upper chords, single member diagonals and single member verticals. The trusses and arches are interconnected with iron spikes/bolts at the vertical and diagonal members. Currently the structure is rated and posted for a 5 mton (5 ton) load limit. The following is a case study focusing on the field testing and analytical modeling results from the Cox Ford Bridge.

5. FIELD TESTING

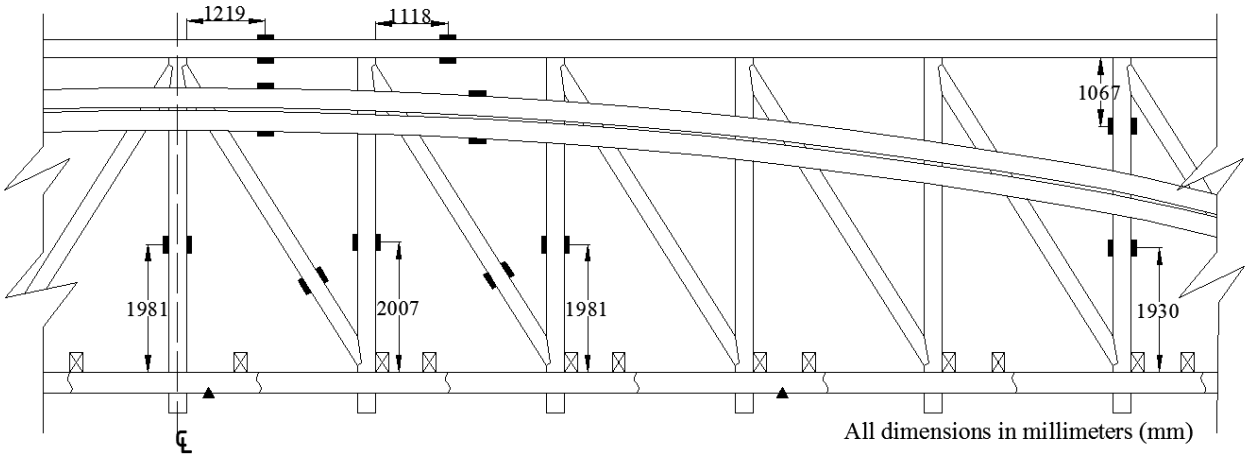
Testing of the Cox Ford Bridge involved installing displacement and strain transducers on the bridge at various cross sections and loading the structure with a vehicle of known weight.

Global displacements of the structure, specifically the trusses, were measured at the following locations on each truss (see Fig. 2 for the instrumentation plan used for the testing of the Cox Ford Bridge): midspan, and at the location of both bottom chord splices. These displacements were recorded with ratiometric displacement transducers mounted on tripods connected to the bridge via aircraft grade steel cable extensions and recorded with an Optim Megadac data acquisition system (DAS) along with a Dell laptop computer running TCS software.

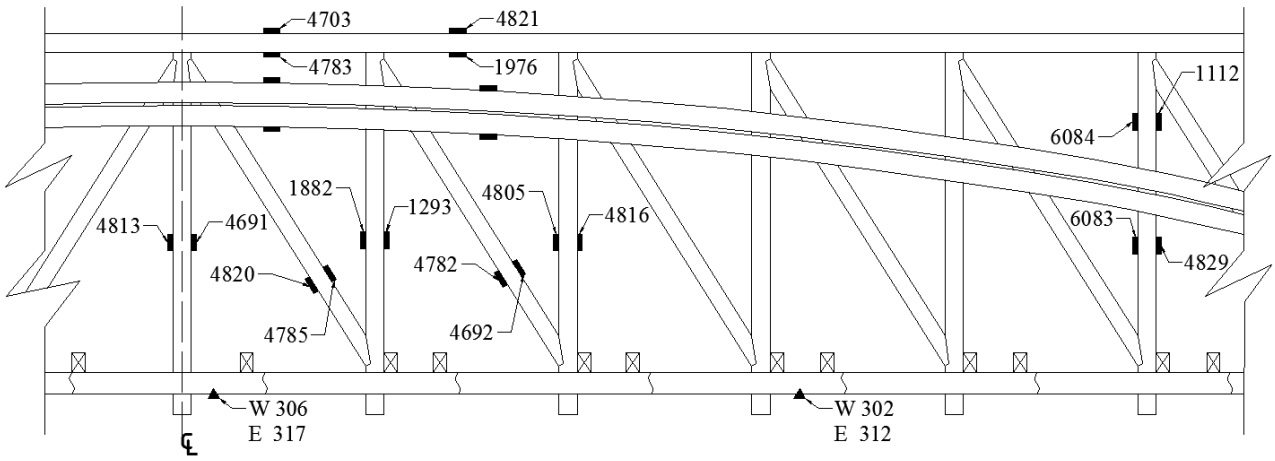
In addition to global displacements, member strains were recorded at various locations on the west truss using Bridge Diagnostics Inc. (BDI) DAS and BDI strain transducers. The strain transducers were attached to the timber members with hex-head screws and washers; in cases where aesthetics were an issue, smaller diameter drywall screws were used to minimize the holes left upon removal of the strain gage. Due to the limited number of gages available and time constraints, symmetry was assumed on the trusses and only one truss was instrumented for strain measurement (see Fig. 2 for locations of strain transducers).



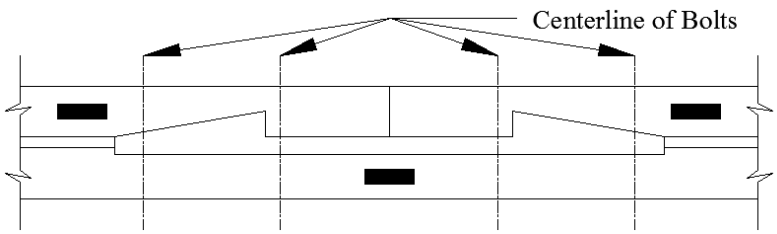
a. West truss instrumentation plan



b. Strain sensor locations



c. Strain sensor details



d. Typical chord joint strain sensor locations

Fig. 2 Strain sensor locations for the Cox Ford Bridge load testing

Loading for the Cox Ford Bridge was applied by driving two different trucks across the bridge: a full size dually pickup truck with a gross weight of 4,735 kg (10,440 lb.) (see Fig. 3) and a single axle dump truck with a gross weight of 8,528 kg (18,800 lbs.) (see Fig. 4).

Fig. 3 Small truck used for load testing the Cox Ford Bridge

Fig. 4 Large truck used for load testing the Cox Ford Bridge

Three load paths were used during the testing of the Cox Ford Bridge. Load Case (LC) 1 involved the small truck centered on the longitudinal centerline of the bridge; LC 2 had the center of the small truck offset 610 mm (2 ft.) west of the longitudinal centerline of the bridge; and LC 3 was the large truck centered on the longitudinal centerline of the bridge. Given the narrow width of the bridge between the interior guardrails, no other load cases were deemed necessary.

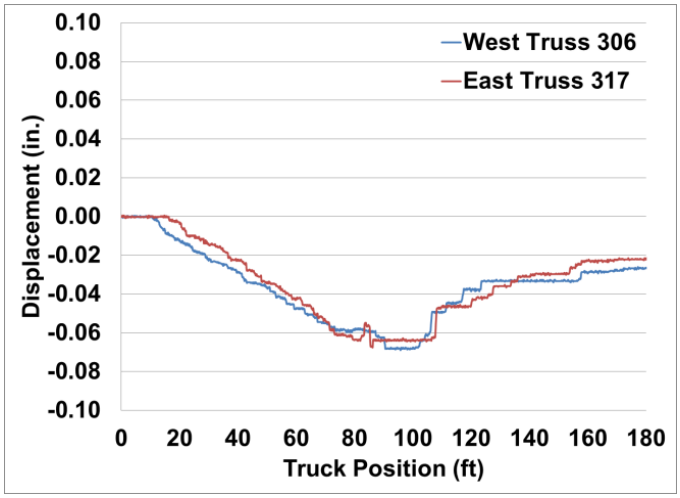
6. Field Test Results

As noted previously, the main purpose in field testing the Cox Ford Bridge was to obtain measureable structural performance data that could be used to modify and calibrate an analytical model of the bridge; that analytical model would subsequently be used to perform the load ratings. The following section briefly discusses some of the test results and the key components evaluated during the testing process. Refer back to Fig. 2 for the locations of displacement and strain transducers discussed below.

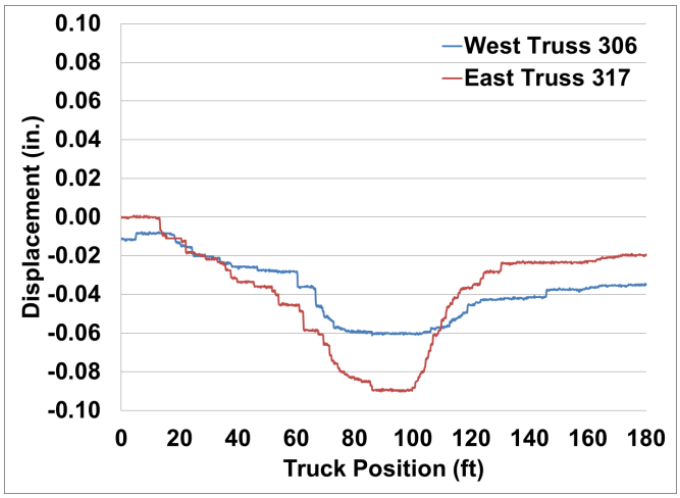
6.1 Global Displacements

Figure 5 illustrates the displacements measured at the midspan of both truss bottom chords. A notable characteristic evident in Fig. 5 is that the bottom chord displacements are similar in

behavior and magnitude. This indicates approximately equal load distribution transversely across the bridge deck to the trusses. Secondly, as the load is shifted away from centerline, LC 2, the magnitude of the deflection of the truss nearest the load becomes larger than the opposite truss, as expected. These transverse load distribution characteristics provide valuable information for the analytical model calibration process, and indicate that the structural response of the bridge is as expected. Furthermore, this means the field data are useful for the model calibration and load rating going forward.



a. LC1



b. LC2

Fig. 5 Global Displacements at Mid Span of the Bridge

6.2 Strains

Illustrated in Figs. 6 and 7 are the strain verses truck position plots for several cross sections of the Cox Ford Bridge. There are some key points of interest to be noted and carried over to the analytical model. First, since both strain gages on either side of the diagonal member essentially mirror one another in both direction and magnitude, the diagonal members may be classified as axial force members. Looking ahead, this gives clues as to the boundary conditions that may be needed in the modeling of the diagonals and verticals. Secondly, referring to the bottom chord splice strains, localized strain peaks are evident with the passage of each truck axle. This suggests that the bottom chord behaves more like a beam than an axial member when under loading, even though the truck/deck load is only transferred to the bottom chord as point loads at the locations of

the transverse spreader beams. Again, this is critical information needed for accurate model generation. One final point to make from the two figures is that the structure is behaving elastically, i.e. the strains return to approximately zero after passage of the load truck. As a side note, instrumentation near the abutments would have been beneficial; had that been an option, then this data would have provided additional guidance in setting the initial level of end restrain in the analytical model.

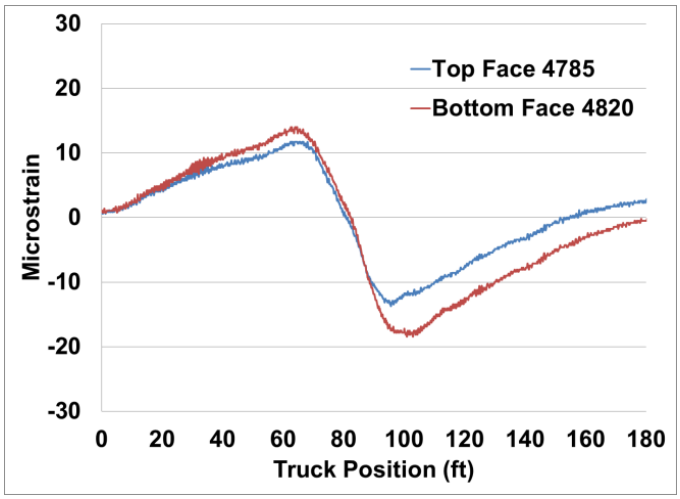


Fig. 6 West Truss Diagonal Between 9W and 10 W, LC1

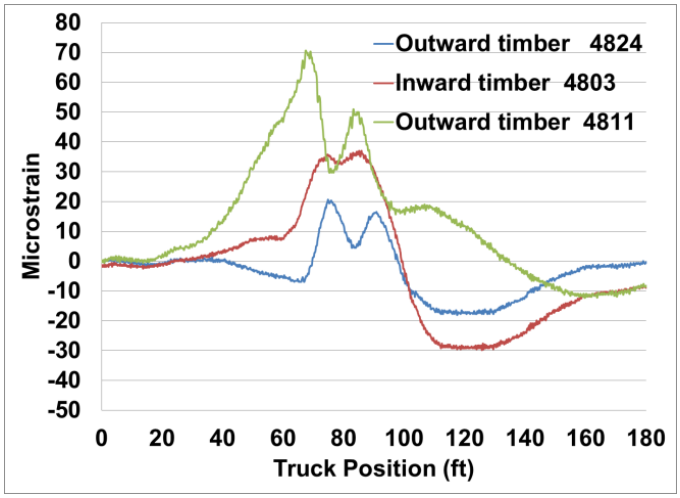


Fig. 7 East Truss Bottom of Bottom Chord, LC3

7. ANALYTICAL MODELING

7.1 Model Generation

Finite element models of historic covered timber bridges can be generated following the common linear-elastic modeling approach. Details on the modeling approach used for the Cox Ford Bridge are provided in the following sections. Information given below includes geometry definition, member properties, boundary conditions, end restraint decisions, and placement of live load. The finite element modeling software selected for this work was the program STAAD due to its familiarity and widespread use in the engineering community. Initial modeling of the structure began by creating a basic truss model of the structure, using truss elements for all members, pinned supports, and pinned connections between all members. The preliminary 2-D analytical model (Model 1) of the Cox Ford Bridge generated using STAAD is illustrated in Fig. 8.

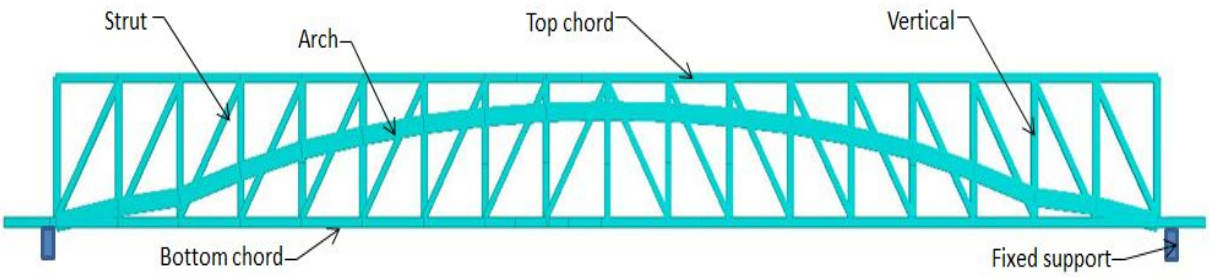


Fig. 8 STAAD model of the Cox Ford Bridge

Since the bridge was modeled in 2-D, and only one truss was modeled, transverse distribution was assumed to be 50 percent such that one wheel line went to each truss; this amount of transverse load distribution was verified (and discussed) earlier by the live load test results. The preliminary model was developed using two-dimensional beam elements for all primary structural members. Since as-built plans were not available, the model geometry was based upon field measurements taken during load testing. For convenience, the supports were defined to be at the end points of the bridge and initially assumed as pinned-pinned. Any built-in eccentricity between the diagonal and vertical members was assumed to be 152.4 mm (6 in.) (based on field inspection of joint details) to simplify the modeling process. The bottom chord was modeled as a monolithic, continuous member (i.e., neglecting the presence and location of member splice), and the connection between the top and bottom chord at the bearings was defined as rigid. All the primary load resisting components (bottom chord, top chord, diagonal and vertical) were defined as beam members whereas the arch members were defined to be compression members based on the knowledge of arch behaviour under loading and confirmed from the live load testing data. The arch members were connected to the truss by means of bolted connections to the verticals over its length and were initially assumed to be rigid. Because the model is two dimensional, geometric dimensions of the arch member were taken as the combined width and depth of the inner and outer arch rings. Furthermore, all members were assumed to be Southern Yellow Pine and the initial modulus of elasticity was assumed to be 0.333 kN/m^2 (1850 ksi) for all members of the truss in Model 1 [1]. After a model of the load truck used during testing was generated in the analysis software, calibration of the model was carried out by applying the truck loading scenarios to the model (in 30.5 mm (0.1 ft.) increments across the structure) along the same paths travelled during testing and comparing the analytical results with the results from the field testing.

7.2 Model Calibration

Initial results from running the analysis of the model and comparing the field and analytical strains on the pinned-pinned model were found to be less accurate than desired; therefore the bridge model end restraint conditions were modified to fixed-fixed in an attempt to bound the results. Strain comparisons from the fixed-fixed model indicated that the actual performance of the structure was somewhere between the pinned-pinned and fixed-fixed conditions, as expected. Subsequently, rather than incorporate complex joint fixity parameters and/or variable support restraints (springs) to improve the accuracy of the results, the decision was made to develop simpler methods of modifying the structure to obtain the best results while, for the most part, retaining the integrity of the structure. Again, the ultimate goal was to make the modeling process as straight-forward as possible.

Through trial and error (modifying member element types, member properties (E), member connection details, etc.) a 2-D model of the truss was developed that improved the accuracy of the results. The resulting STAAD model consists of fixed supports; pinned member connections; truss elements for the verticals, diagonals and upper chord; and beam elements for the bottom chord. These changes improved the correlation between the field test data and the analytical model results from approximately 40-50 percent to approximately 75-85 percent. Given the complexities inherent with timber as a material, along with the additional uncertainties input from the truss connection details and the presence of the framing/roof of the covered bridge, a percent error between the field

and model results of approximately 20 percent is acceptable.

A side note: although the wall sheathing and roof of these covered bridges are not intended to be structural elements, they contribute to the stiffness of the structure in some regard. It should be noted that these and other unknown factors contributing to the performance of the actual structure were not represented in this STAAD model. These unaccounted for features likely contribute to the discrepancy between the field results and the analytical modeling results presented herein. It is felt, however, that ignoring these other sources of stiffness is a conservative assumption. The handling of these bridge characteristics in future modeling and rating of covered timber bridges is left to the discretion of the engineer of record.

8. LOAD RATING

The final step in the process is performing a load rating for the subject bridges using the calibrated FE model developed for that structure from the live load testing results discussed previously. The basic procedure for performing the load rating in this manner after generating a FE model are as follows: 1) based on the findings and recommendations from the previous section, create a FE model of the structure; 2) calculate and apply dead loads for the structure; 3) apply live loads, rating vehicles and/or different legal trucks, across the 2-D model to obtain member forces (noting that things such as vehicle height and width in addition to weight may control if the vehicle can safely enter and cross the covered bridge); 4) determine appropriate impact factor and input into model; 5) calculate member capacities (for all critical members); 5) analyze the model and calculate load ratings (ratio of member capacities to member forces output from FE model). The following briefly defines/outlines each step.

The initial step is calculation and application of the dead loads of the structure. Included in the dead loads are the weights of all bridge components, consisting of the primary (e.g., chord members, inclined posts, and vertical posts) and secondary components (e.g., side planks, roof, etc.). The dead loads are generally assumed to be uniform loads and can generally be calculated from the sectional properties for each component as measured in the field or from as-built plans and an assumed density of timber. Once the dead loads are calculated, they are applied to the calibrated model and the user moves on to live loads.

According to the AASHTO Load and Resistance Factor Rating (LRFR) manuals (AASHTO 2011), the AASHTO HL-93 truck and the AASHTO design tandem load combined with the standard uniform lane load should be used as the live loads to determine load ratings. The HL-93 truck and design tandem load are to be considered individually (with the standard uniform lane load). In all cases the HL-93 truck and tandem load should be placed 610 mm (2 ft.) from each curb as well as located at all other critical transverse locations (AASHTO 2004; 2010).

Along with the live load, consideration of the impact factor to be used for the particular bridge must be established. According to the AASHTO LRFD Specification (AASHTO 2004; 2010), a 33 percent increase in live loads for typical bridges can be utilized for dynamic load allowance (IM). However, the IM of timber components used for rating purposes is to be reduced by one half (to a maximum of 16.5 percent) according to the AASHTO LRFR Manuals (AASHTO 2011).

Once all the dead and live loads have been determined and entered into the model, calculation of member capacities is performed to complete the ratings. Typical historic covered bridges consist of members in tension and/or compression and members in combined flexure and axial loading; thus, the axial and/or flexural capacities should be determined separately in accordance with the appropriate AASHTO LRFD Specifications (AASHTO 2004; 2010).

After the member capacities are computed, the loading is performed to obtain the member strains that result from the rating vehicles. These member strains are then compared to the member capacities to obtain the load rating. Load ratings for members not exposed to combinations of loading types (including tension/compression members, flexure members, members under shear) may be computed as outlined in the AASHTO LRFR Manuals (AASHTO 2011).

During analysis of the live load field test data from the covered timber bridges, strain responses of certain members indicated they act like beam-column members because they resist combined flexure and axial loads; for Burr arch trusses those members are the top chord, bottom chord, and

verticals. Therefore, load ratings reflecting the effects of the combination of flexure and axial loads should be considered for these members. Load ratings for the beam-column members can be estimated using design-based interaction equations provided by the AASHTO LRFD Specifications (AASHTO 2004; 2010).

It should be noted that the current AASHTO LRFD (2010) Specifications provide the interaction equation that has been modified to take into account the Euler buckling stress and gross cross-sectional area.

9. Future Work & Acknowledgements

Phase I of this work involved live load testing 11 covered timber bridges, divided into three groups of different truss configuration, namely Burr arch, Queen post and Howe trusses. The collected field test data along with a preliminary finite element analytical model of each bridge were used to calibrate each model to replicate the structural response of the bridges. These calibrated models could then be utilized to perform load ratings on each bridge for various vehicle configurations. Currently there are more than 3 truss types that make up the approximately 600-700 covered timber bridge still in service in the United States. Phase II will seek out an additional 2 or 3 common truss types of covered timber bridges for live load testing and analysis such that recommendations for modeling and load rating of these types of bridge through physical load testing may be added to those from Phase I.

This study is part of the Research, Technology and Education portion of the National Historic Covered Bridge Preservation (NHCBP) Program administered by the Federal Highway Administration. This study is conducted under a joint agreement between the Federal Highway Administration – Turner Fairbank Highway Research Center, and the Forest Service – Forest Products Laboratory.

10. References

[1] Ross R.J., White R.H., Pellerin R.F., Wang X, and Barshaw B. K., *Wood and Timber Condition Assessment Manual, 2010.*

[2] American Association of State Highway and Transportation Officials (AASHTO), *The Manual for Brigde Evaluation, AASHTO, Washington, D.C., 2011.*

[3] American Association of State Highway and Transportation Officials (AASHTO), *LRFD Bridge Design Specifications, AASHTO, Washington, D.C., 2010.*

A timber bridge across Lake Mjøsa in Norway

Svein Erik Jakobsen
Director
Aas-Jakobsen
Oslo, Norway
sej@aaj.no

M.Sc. from Norwegian Institute of Technology from 1984. Joined the company Aas-Jakobsen in 1985 and is currently Director of the Bridge Department.
.

Trond Arne Stensby
Project Manager
Norwegian Public Roads Administration
Hamar, Norway
trond.stensby@vegvesen.no

Widar Mikkelsen
Project Manager
Norwegian Public Roads Administration
Hamar, Norway
widar.mikkelsen@vegvesen.no

Yngve Årtun
Chief Architect
Plan Arkitekter AS
Trondheim, Norway
yngve@plan.no

M.Arch. from Norwegian Institute of Technology from 1983. Joined the company Plan Arkitekter as a partner in 1996. Specialist in bridge aesthetics and timber bridges.

Per Meaas
Project Manager
Aas-Jakobsen
Oslo, Norway
pme@aaj.no

M.Sc. from Norwegian Institute of Technology from 1974. Joined the company Aas-Jakobsen in 1976 and is Project Manager for several Major Infrastructure Projects.

Summary

Pre-studies are performed on how to cross Lake Mjøsa in Norway for a new 4-lane link on Highway E6 between the cities Hamar and Lillehammer. Preliminary design is also performed for timber bridges in several alternatives. The timber alternatives are truss bridges with two underlay timber trusses composite with a concrete bridge deck. Typical span width is 69m. Main spans are suggested to be cable stay spans with the same timber truss solution and span widths 120.75m. Total length of the bridge is approximately 1650m.

The conclusions from preliminary design in accordance to ref. [1] have been that a concrete extradosed alternative or a timber truss alternative with cable stayed main spans is to be preferred.

A research program was recently launched by the Norwegian Ministry of Transportation in order to investigate critical aspects involved in construction of such a large timber bridge. The research will focus on the large scale aspects, durability and the technical solution.

Keywords: Timber truss, world record, research program.

1. Introduction

1.1 1st Mjøsa Bridge

Fig. 1 Overview of 1st Mjøsa Bridge

The 1st Mjøsa Bridge was opened to traffic in 1985 and is a part of the main north-south highway in Norway, the E6. The bridge crosses the largest lake in Norway, Lake Mjøsa, and is in total 1311m long. It is a concrete box girder bridge with typical span widths of 69m. The bridge is founded on piles, some to rock, some as friction piles. At the bridge site, the lake is app. 30-40m deep, giving a considerable free span in water for each pile. The bridge has two lanes of traffic and a pavement for walking/bicycling. An overview picture og the bridge is shown in Fig. 1.

1.2 Feasibility study 2006

Since traffic is increasing on the E6, plans have been introduced to increase capacity across Lake Mjøsa. In 2006, Norwegian Public Roads Administration carried out a feasibility study in order to investigate if it was possible to cross Lake Mjøsa with a bridge in vicinity to the existing one. The feasibility study report was worked out by Aas-Jakobsen AS. The existing Mjøsa Bridge is situated on a narrow sill formed by ancient glaciers. The feasibility study concluded that it was possible to place yet another bridge at the same sill. The study also identified another possible alignment for the crossing further south, this however requiring deep sea foundations.

1.3 The timber bridge background

The Norwegian Public Roads Administration Region East is responsible for construction and operation of the possible 2nd crossing. This region has been the leading developer of timber bridges in Norway. Through a large number of timber bridge projects, the knowhow and expertise has grown over the years, making this region one of the leading timber bridge environments in the world. Since Mjøsa Bridge is situated in the very heart of this timber bridge society, it is only natural that there are strong interests to consider a timber bridge alternative for the new crossing. The fact that this will be the world's longest bridge of its kind, if carried out as a timber bridge, only strengthen this attitude. In 2010, a timber bridge seminar was held close to the bridge site in order to determine whether a timber bridge alternative was feasible for Mjøsa Bridge. Experts from the whole industry were invited, and the conclusion was positive. The seminar even introduced three different technical solutions as basis for further work.

1.4 Conceptual design 2012

In 2012, Norwegian Public Roads Administration launched a conceptual design phase involving evaluation of all consequences of realization of the project for the 2nd crossing of Lake Mjøsa. The project was awarded Aas-Jakobsen AS. One of the basic boundary conditions of the project was that at least one timber bridge alternative should be investigated.

The first part of the project was dedicated to evaluations of road alignments as defined in the

feasibility study. Along this work a broad spectre of bridge alternatives was evaluated for the different alignments. The recommendation from this part of the project was that even if the southern alignment has technical challenges from deep sea foundations, it is preferable to traffic and to public considerations.

Given the recommended alignment, it was concluded that both from technical aspects and from aesthetical aspects, the bridge should have increased mid span lengths compared to side span lengths. This lead to the conclusion that the bridge type should either be a concrete box girder type with extradosed main spans, or a timber underlay truss type with main spans supported by cable stays.

This paper further describes the latter.

2. Technical challenges

2.1 Technical description

The proposed new bridge has a total length of 1650m and span widths typically 69m. The middle four spans have span widths 120.75m. The bridge is founded on rock in two axes, on piles to rock on 13 axes and on friction piles on the remaining 8 axes. Deep sea foundations in the five main span axes are friction piles up to almost 110m length, with a 40m portion in soil and a water depth of 70m. The free spanning through water is restricted through a concrete caisson solution acting both as guides and as buckling constraints to the piles. The piles are steel piles with diameter 1220mm. A side overview of the bridge solution is given in Fig. 2.

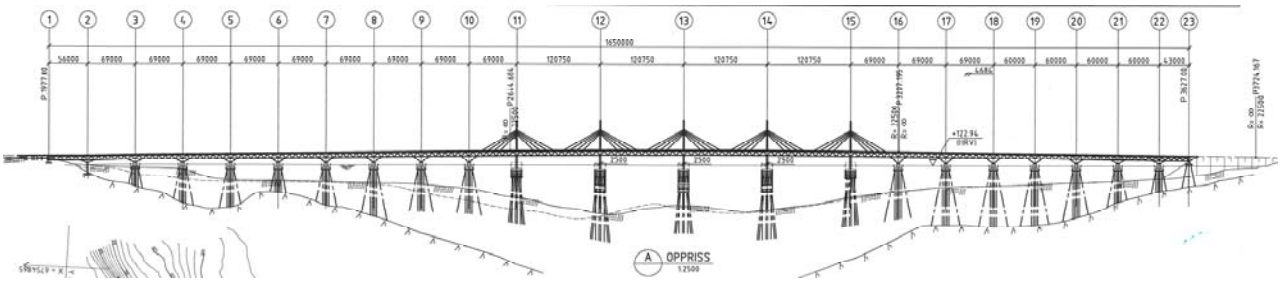


Fig. 2 Side overview, timber bridge

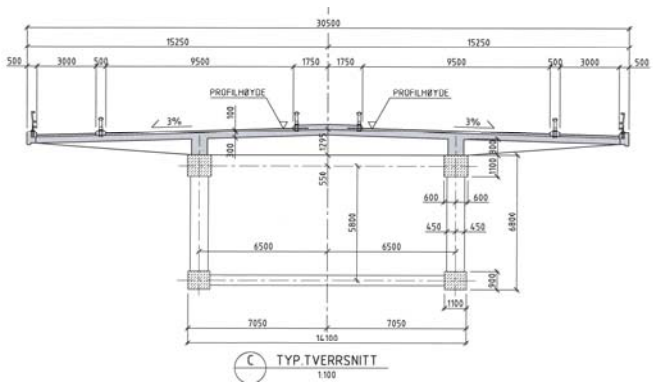


Fig. 3 Typical cross section

The bridge carries four lanes of traffic, with free space between railings of 2 x 9.5m, one pedestrian lane of 3m and one service lane on the opposite side of 3m, giving a symmetric layout og the bridge as can been seen on Fig. 3.

The superstructure is a two plane timber truss with a height of 6.8m proposed in composite action with a concrete bridge deck. The timber truss members have typical dimensions of 1100mm x 1100mm, 1100mm x 900mm and 900mm x 900mm, and with steel nodes.

Fig. 4 View beneath the bridge

Typical columns are V-shaped in side spans to reduce effective span widths and W-shaped in main span to transfer the large vertical forces to the foundations. This can be seen in Fig. 4.

A third longitudinal beam is required in main spans to transfer the horizontal compression force from the stays into the bridge deck.

The stay plane is placed in the centre, implying the introduction if single towers in between the traffic lanes. The towers are preferably wooden with steel anchor attachments.

The bridge deck is a concrete deck with typical thickness of 300mm supported by longitudinal beams above each truss plane and transverse cross beams every approximate 7m, situated above the truss nodes.

2.2 Structural behaviour

The overall length of the bridge requires careful considerations on global behaviour and force transfer of for instance creep, shrinkage and temperature effects. In the preliminary design performed, the bridge is proposed to be a composite bridge with regards to the timber truss and the concrete deck, and with full force transfer between these elements. The superstructure therefore will be a continuous beam from abutment to abutment. The bridge is fixed at the central pylons and has sliding bearings in side spans. This requires large expansion joints at the abutments.

The vertical forces in the main span are partly transferred through the stays to the central pylons, while the transverse forces are transferred through the bridge beam to the same pylons.

2.3 Aesthetics

The idea of the truss timber bridge was developed from the requirement of relatively large spans. This requirement was agreed to be vital both from aesthetical and economic reasons, to avoid costly foundations and to avoid the common barrier look that the long strait crossings with small spans often tend to have.

It was also agreed that the introduction of longer mid spans was beneficial to the overall looks of the bridge. These spans underline the bridge as an icon, and also emphasize the proposed ship channel areas. It was also a discussion of how many towers should be introduced. The common opinion within the project group was that a number of 3, 5, 7 etc. would be better than 2, 4, 6 etc. This is demonstrated in Fig. 5.

Fig. 5 Example of a 3-tower configuration

The final decision was to introduce 5 towers as this also minimizes the number of deep sea foundations.

2.4 Large scale challenges

The bridge will be the longest timber bridge in the world if chosen. Hence it has been vital to the project to identify what challenges this introduces. A list of topics has been outlined as follows:

- Composite behaviour with concrete (internal force transfer, different material characteristics)
- Deliverance capacity (manufacturing, transportation, assembly)
- QA (certificates, guarantees)
- Durability (large scale effects)

2.5 Durability

The bridge will be designed for a 100 year life. The project must however make sure this is feasible in reality for such a large scale project. For normal size bridges it is more relevant to do repair and rehabilitation on exposed parts of the structure, but with the size of the Mjøsa Bridge it is of vital importance to ensure durability through technical solutions, treatment and planning.

To place the timber trusses beneath the wide concrete deck as proposed will make timber less exposed to sun, temperature differences, rain, salt treatment, etc. The environment should therefore be the best to increase durability. In addition, the experience from the many neighbouring timber bridges will constantly improve the durability, giving the best state of the art design available as a reference.

3. Further development

3.1 Goals

The project will now enter the next phase doing extended preliminary design of the chosen concepts before entering a future detail design phase. As per now, the time schedule for further work is however uncertain due to the political situation in Norway, with a parliament election this autumn.

It is however the goal of the project team to design and construct this prestige bridge in near future, making it an iconic symbol for both the region and the country. To make it into the largest timber bridge ever built would in addition be a valuable achievement for the whole industry.

3.2 Research programme

The Norwegian Department of Transportation has recently launched a research programme to make the timber bridge alternative for the 2nd Mjøsa Bridge competitive to other solutions with regards to the technical challenges involved. The research programme will focus on the following aspects:

- Large scale effects on timber bridges
- Cable stay solutions on timber bridges
- Large spans on timber bridges
- Temperature effects on timber bridges
- Material specifications on timber
- Preservation of timber
- Composite behaviour between timber and concrete

4. Conclusions

The conclusion of the feasibility study and preliminary design of the 2nd Mjøsa Bridge is that a timber truss alternative is recommended as one of the alternative solutions. The bridge will have a total length of 1650m. A research programme is now being launched in order to bring this alternative up to level of other recommended alternatives. The program will focus on large scale aspects, durability and technical solution.

5. References

[1] Statens vegvesen: “Kommunedelplan. E6 Gardermoen-Biri. Parsell Moelv-Biri. Rinsaker og Gjøvik kommune. Konstruksjonsrapport”. B003 (Aas-Jakobsen), ver. 14.06.2013.

CHALLENGES OF THE SUPERSTRUCTURE “ÄLVSBACKA BRIDGE”

Peter Jacobsson
Martinsons Träbroar AB
Skellefteå, Sweden
peter.jacobsson@martinsons.se

Fredrik Eliasson
Martinsons Träbroar AB
Skellefteå, Sweden
fredrik.eliasson@martinsons.se

Thomas Darholm
Cowi Sweden AB
Gothenburg, Sweden
THDM@cowi.se

Summary

The main purpose of this conference paper is to show the opportunities for using timber in bridges and share our experience regarding some of the challenges related to this, in many ways, complicated superstructure in timber.

This paper presents two of the main challenges:

- The dynamic behaviour for this superstructure with a span of 130 m made of timber that has to fulfil the requirements according to the Swedish Bridge Code “Bro 2004”
- The erection of the superstructure which has been done with a method developed for this project and as far as we now never has been used before.

In the late summer 2011 this new cable stayed pedestrian bridge passing over the river of Skellefteå in Skellefteå city was taken into use, named “The Älvsbacka Bridge”.

The client of the bridge, the municipality of Skellefteå has a great interest in timber construction and had a vision that this bridge could be built in timber in a cost effective way. After an evaluation the municipality ordered the bridge into two separate contracts, one for the foundations in concrete from Skanska and one for the timber superstructure from Martinsons Träbroar AB.

Fig. 1: Views of the final result of “The Älvsbacka Bridge”

Some short facts:
Distance between pylons, span length: 130 m, total length 182 m.
Free width of decking: 4 m
Height of pylons: 24 m
Amount of timber: Approximately 200 ton, 400 m3
Amount of steel: Approximately 70 ton, 9 m3

Keywords: Cable stayed bridge, timber bridge, dynamic properties, assembling method

1. Introduction

Martinsons Träbroar has designed and produced timber bridges for the last 20 years. Around 40-50 bridges is produced each year. This actual pedestrian bridge, “The Älvsbacka Bridge”, is very unusual and unique among these bridges. The bridge is large, has a rather difficult type of design, and has surrounding conditions at constructions site like

- streaming river that made it difficult to be with a larger barge in the water
- lack of area for mobile cranes and trucks with bridge parts.

One very important issue like always when it comes to timber bridges was to achieve a performance that protect timber from the influence of climate and ensure durability for the superstructure for a long time.

The final design of the bridge is presented in the Fig. 2. The primary construction parts are built up from glue lam and steel. The primary glue lam parts are made of untreated spruce protected with coating and cladding panel. All steel parts are protected against corrosion with hot dip galvanization.

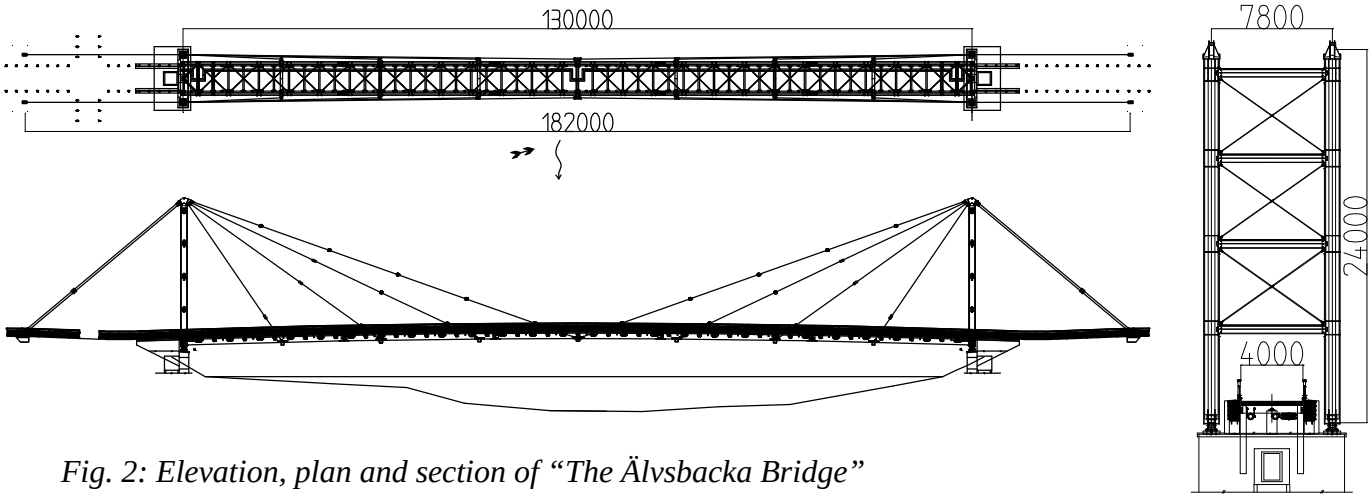


Fig. 2: Elevation, plan and section of “The Älvsbacka Bridge”

The assembling time of the bridge on site where approximately 5 weeks, 2 weeks to erect the pylons and launching the bridge deck and 3 weeks with complementation of joints and cladding, demounting temporary materials, adjustments and electrical and water tube installations.

A large amount of time has been invested in design work. A close collaboration between the consultant firm in bridge design, Cowi, and Martinsons has been established and resulted in solutions, general one and detailed one, that satisfied the manufacturer and client in relation to the initial conditions like a tight budget and time schedule. The prefabrication and the assembling on site have technically been carried out in a successful way when it has been done according to the plan from the design phase and without any larger deviations The assembly method is also unique and definitely something that we will continue to work with and develop further if we are into building such a bridge again.

2. Selection of global performance of the superstructure

Early in the planning process of this bridge the municipality of Skellefteå was in contact with a local architectural firm. This cooperation ended up with a suggested superstructure in timber with one pylon in the middle of the river shown in Fig. 3. One advantage with this design was that no parts of the bridge would trespass on the land area at the bridge ends. In both ends of the planned bridge there were sites with houses and not so much free area.

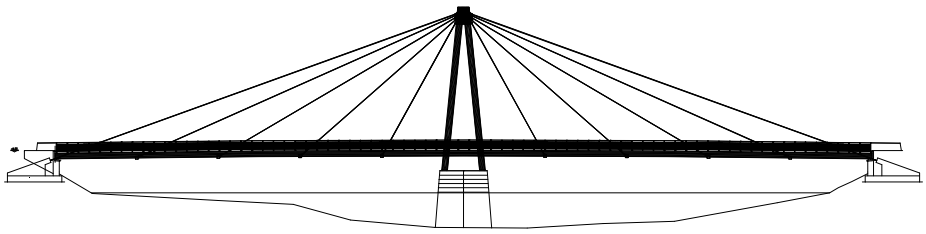


Fig. 3: Elevation of initially suggested solution

The suggested bridge performance also had an appealing esthetical value, a distinct landmark. However, after the first offers from different potential contractors it had a higher cost than what the municipality had estimated the maximum cost to and made the budget for. So now the question was gathered – what can be done to get down the cost? After a new procurement process with fewer requirements on the design and more focus on the possible maximum cost, new alternative solutions were developed and offered. One conclusion from the first alternative was that “we have to get up from the water to bring down the cost”. The original alternative with one foundation and pylon in

the middle of the streaming river was a great part of the total cost. So, - how can a bridge in timber without supports in the river be built we thought.
Martinsons in cooperation with Cowi suggested a cable stayed design with a pylon at each landside, as near the water as possible to get shortest possible span length and to achieve area for the anchoring bars on land. With a relatively steep slope of the anchor bars the solution seemed possible without trespass the close house sites too much. The great advantage with this solution was the price, at the same level for the originally suggested superstructure but with much less estimated cost for the foundation work.

However some more difficult challenges and questions rise with this alternative solution.

- One was the span length, especially transverse the bridge, that now increases from approximately 65 m to 130 m. Doubling the span increases the uncertainties regarding dynamic behaviour. This will also produce an increased static strain.
- Temporary supports and temporary road or bridge to the middle of river that could have been used to assemble the superstructure was now disappearing
- Will this new solution create the same landmark sign as the initial alternative?
- Will the anchor bars on each landside have an acceptable influence and not disturb the persons living closest to the bridge.

The dynamic behaviour of this solution seemed as such a great challenge that a pre study was done of Cowi and Martinsons. Conclusion from it was that the superstructure should have an acceptable dynamical behaviour.
Conclusion regarding the other challenges mentioned above where that they could be handled or were in an acceptable level.

After the municipality had evaluated different proposals of the superstructure they chose to contract Martinsons

3. The design process

Below are some of the most important challenges and difficult work during the design phase described, dynamic behaviour and method for assembling the superstructure on construction site.

3.1 Dynamic behaviour

A primary concern for the bridge was to verify that the dynamic behaviour would be acceptable. This means both comfort criteria for pedestrians on the bridge and effects due to the wind.

The complete bridge with main girders, pylons, cables and the supporting piles were modelled in a general all-purpose FEM – software.

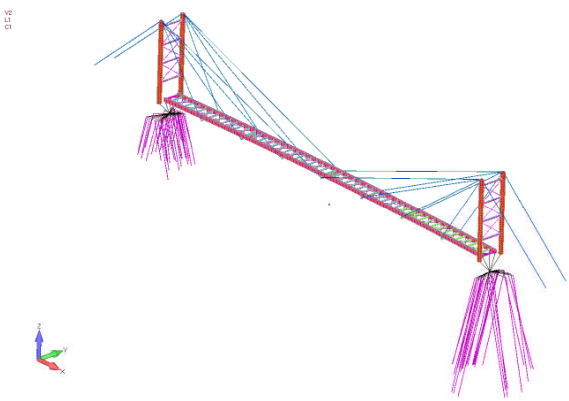


Fig. 4: FE-model

The dynamic analysis indicated that the bridge was relatively weak. The first horizontal mode had a natural frequency 0.6 Hz and the first vertical mode had frequency 1.4 Hz

Fig. 5: Horisontal mode, f = 0.6 Hz

Fig. 6: Vertical mode, f = 1.4 Hz

The Swedish Bridge code [1] includes a force model for walking pedestrians. The force depends on the size of the bridge and the natural frequency. The model gives the damping for a timber bridge (ξ = 0.6%) and an acceptance criteria for vertical acceleration $a_{rms} \leq 0.5$ m/s^2. The model does not give any directions regarding horizontal vibrations.
The Älvsbacka Bridge can be considered a horizontal beam with a 130 m span so horizontal vibrations must obviously be considered. In the literature there are recommendations regarding the horizontal force from pedestrians. It is approximately 10% of the vertical force and $a_{rms} \leq$ 0.2 m/s^2. These recommendations were used for the design.

Due to requirements from the municipality Skellefteå for installations on the bridge different mass cases was also included in the design.

The analysis indicated that the accelerations would be above the level accepted (~1 m/s^2) in the code for vertical accelerations, Fig. 7, while the horizontal was acceptable. But it was also deemed that the assumed damping was conservative so the complete bridge would probably meet the requirements. The option to install damping elements was also discussed with the municipality of Skellefteå and the bridge was prepared for such an installation.

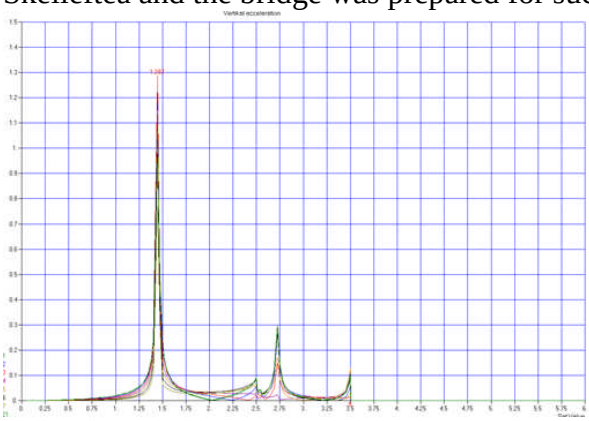


Fig. 7: Dynamic response plot: Acceleretion(f)

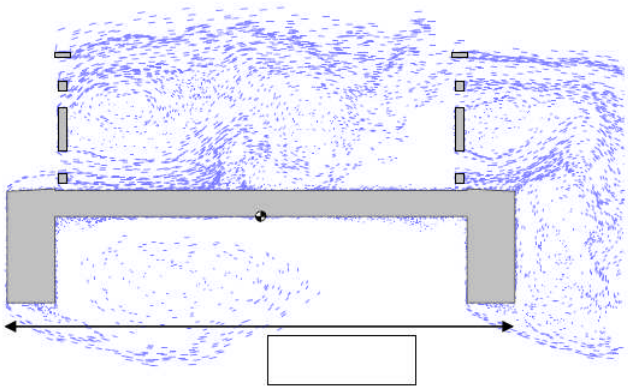


Fig. 8: Results from flow analysis for the bridge section

The second challenge related to dynamic was the wind. There are simplified methods that COWI and Martinsons have used in the past for smaller cable stayed bridges. But due to the size of this bridge these methods were not deemed as reliable.

For the Älvsbacka bridge the method was instead to use calculated natural frequencies and mode shapes from the FEM-model. This information combined with the mass and mass moment of inertia for the bridge was used by COWI Denmark, Major Bridges, to analyse the bridge for wind effects. The results were critical wind speeds for aerodynamic instability. Phenomena such as vortex

shedding, flutter, galloping, torsional divergence and buffeting were studied and the critical wind speeds were above the design wind speeds. Accelerations and force effects on the bridge due to wind effects were also calculated and deemed acceptable. Force effects were based on flow analysis, se Fig. 8. The FEM – model was also used in the static design for verification purposes.

3.2 Planning of the assembling method

One of the most difficult things with this bridge was the erection of the superstructure. The initial idea was to apply Martinsons method developed for shorter span cable-stayed bridges, using mobile cranes to lift out the pylons, tension bars and the deck pieces. In this case, a really large crane, and many of them, would have been needed. When studying this method closer we realised that the method would be complex and expensive.
So, new ideas came up. Could it be possible to first erect the pylons and all of the tension bars including transverse beams and after that stress up tension bars and then pull out the bridge deck with help of the ordinary transverse beams as supports? The answer was yes, the greatest effort seemed to be minimized the cost for mobile cranes and less demand of land area for all the cranes needed. A great planning work to find solutions for this method started and continued parallel with the design of the superstructure. The planning work ended up in a rather detailed description of the assembling process.

The method can roughly be described like:

- Two180 m long temporary cables from each anchor foundations were stressed up. The purpose of these cables where to pull out the bars and transverse beams.

- Delivery of prefabricated pylon columns and other parts belonging to the pylon and putting these parts together on construction site. Three days was planned for this activity.

- Erection of the pylons, anchor bars, the longest tension bars reaching to mid span and its related transverse steel beam. All these parts connected together before and during the erection, se Fig. 9. The bars then were to be stressed up with temporary weights in transverse beam for stabilization.

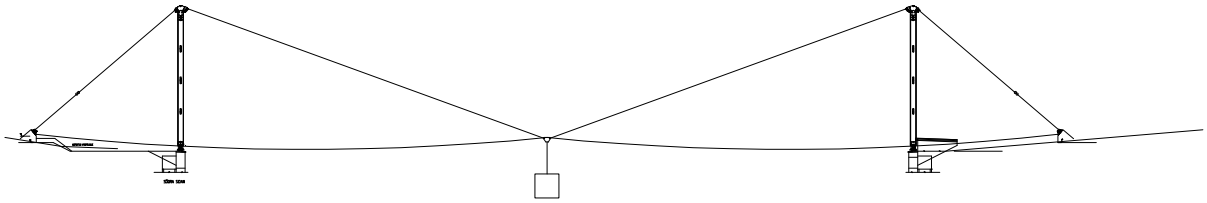


Fig. 9: Pylons, the longest bars and anchorage bars erected and stressed up with weights.

- Pulling out the other 12 bars and the six others transverse beams with help of the temporary cables, a winch and assistance from the barge.

- With all the bars in place, the system shall be stretched up with temporary horizontal bars between transverse beams and to pylon foundations.

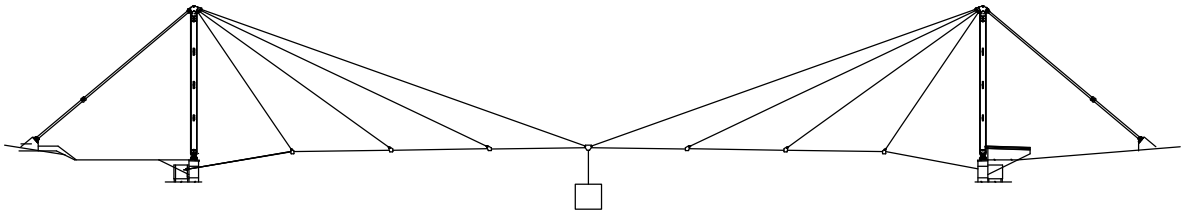


Fig. 10: All bars erected and the system stretched up.

- Then the 130 m long bridge deck, consisting of five parts each about 27 m long, was to be put in place by an incremental launching method from the south end. One important issue for this part of the assembly was the pull out force and to get it as low as possible. Tests of different treatments were conducted in the factory to find as low friction coefficient as possible. The maximum estimated pull out force based on the friction tests where 11 tons. Equipment to do this job has a capacity of 20 tons to allow for some margins. A tow truck was used for launching of the deck.

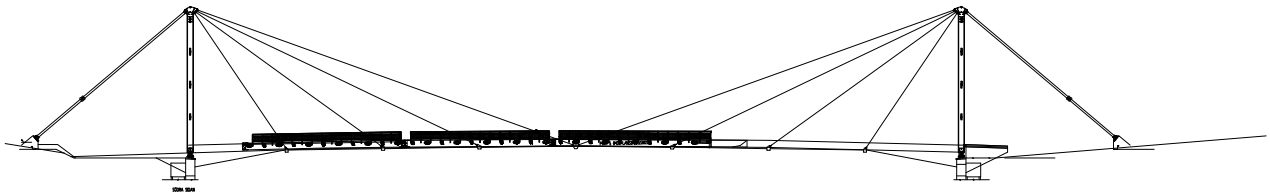


Fig. 11: Three connected bridge deck parts during launching and in position to connect the forth part.

- After launching the bridge deck, about three weeks of work was left with connections, installations, remaining cladding and decking.

4. Production in factory and assembling at the construction site.

An overview of the work in factory and at the construction site is described below.

4.1 The production in factory

One of the great possible advantages with timber bridges is the short assembling time on construction site due to the high degree of prefabrication. For this bridge there were high requirements of short time on site and the degree of prefabrication was developed from that perspective and from the cost effective perspective.

The pylons were delivered from the factory in parts.

- Pylon columns with steel members in top, bottom, connections for transverse beams and around 90% of protective cladding
- The transverse beams
- The stabilization bars

The 130 m long bridge deck where delivered in 5 parts. The decks had a high degree of prefabrication, around 90% of the cladding and the decking surface in place. Steel members at the end of each piece had been designed to minimize the time needed at construction site. All the 5 parts had been put together in factory when steel members with dowels where fastened into the primary beams. In this way it had been ensured that they would fit together on site. The “first” bridge deck was performed with a nose with a lifting device to simplify the passage of transverse beams.

A lot of steel bars are used in the superstructure. The main bars carrying vertical loads on the bridge deck, the stabilization bars in the pylons and the bars in the bridge deck distributing the horizontal transverse loads to the foundations. All the bars in the bridge deck were mounted in factory. All the other bars were put together as far as possible and adjusted to the right lengths in factory so the only part missing on constructions site was pins at bar ends.

4.2 Assembling on the construction site

In beginning of June 2011, assembling on the construction site started. During the erection process of the superstructure subcontractors were used to:

- provide mobile cranes taking care of the vertical lift forces
- provide tow truck taking care of the erection forces acting horizontal
- assist with a small barge with a small crane and scissor platforms.

After preparations on both sides with temporary supports, mobile crane surfaces, establishing the barge and stressing up the temporary wires the work with permanent parts could start. Initially the longest bars with the “mid transverse beam” were pulled out with help of the temporary wires. Work continued with putting together prefabricated parts of the pylon. See Fig: 12.

Fig 12: Long bars on their way out (left picture) and assembling of pylon.

After four days work it was time for one of the most critical moments in the erection process, to lift the pylons with the longest bar to its permanent vertical position. This was done with the equipment of five mobile cranes, three tow trucks and the barge.

Fig 13: Start lifting pylon (left picture) and pylons almost in final position (right picture)

When the pylon was vertical and the pins in the bottom had been mounted the final part was to place temporary weights in the transverse beam that stretches up the long bars and make the system stable.

The assembling work continued with pulling out the rest of the 12 bars and six transverse beams. Then the hole system with bars had to be stressed up which were done with horizontal temporary bars between transverse steel beams and stressed against the pylon foundations at ends. With this performance the system was ready to act like support when pulling out the five 27 m long bridge deck parts. The main equipment in this stage was two mobile cranes in the south end of the bridge and one tow truck in the north end pulling all the five parts from south end and heading north. The first deck part, the one ending up in the north end had an elongation of the permanent beams acting like noses that made it possible to pass the transverse beams in an effective way. The noses had also a lifting device that was an important function. This operation was successful and was performed during 3 days. One critical issue during this operation was weather conditions and especially the wind and how it easily can generate oscillations transverse the bridge. This risk had been considered and the bridge was secured with wires from the transverse beams to anchors in the bottom of the river up streams and down streams and at several places along the bridge.

Fig. 14: Bridge on its way out in left picture and almost in final position in right picture.

When all five pieces where in right length position the work with adjustment and permanent looking of connections took place during a couple of days parallel with the concluding work with removing temporary material, put on missing cladding and installation of the electricity equipment and the water tube.

Figure 15: Finishing construction work after pulling out all the bridge decks.

5. Conclusions

In august 2011 the bridge was consecrated and around 500 peoples walked from the south bridge end to the north. The bridge has become very popular among citizens and is frequently used. The client, municipality of Skellefteå, is also very satisfied and proud. The bridge has fulfilled the municipality’s expectations and also became the landmark they wanted. With this result we as a contractor off course also are satisfied

Conclusion regarding the assembling method is that it has worked out well and we will use it again if the opportunity comes up. Tough, there are always things that can be made better. Possibilities we see are

- that fewer mobile cranes is possible during the pylon erection.
- that tow truck worked very well, but for the several of the horizontal pulling forces, equipment with lower cost should be possible to find, develop.
- refined detail solutions to pull out parts on the temporary cables
- excluding some of the temporary parts and use permanent parts instead.

An attractive and challenging question for the future that we have asked ourselves is – can the method be developed further so that the bridge can be erected without working from a barge? The answer is to be searched in the future and if it results in a yes, probably an even more efficient and useful method is created.

6. References

[1] Swedish Bridge Code “Bro 2004”,Publ. 2004:56, ISSN 1401-9612

Carney Mill Road Bridge

2nd International Conference on Timber Bridges

Las Vegas, Nevada

2013 Sep 30 – Oct 02

Murray M. Johnson, P.Eng., PE

Buckland & Taylor Ltd.

BUCKLAND & TAYLOR | COWI

Existing Bridge

BUCKLAND & TAYLOR | COWI

Existing Bridge:
15 t Load Limit, Narrow for Pedestrians

BUCKLAND & TAYLOR | COWI

Existing Bridge:
Stream Banks OK, Abutments Poor!

BUCKLAND & TAYLOR | COWI

Existing Abutment

BUCKLAND & TAYLOR | COWI

Existing Bridge

- **25 m Span "Bailey" Bridge**
- **"Temporary" several decades earlier**
- **Load limited to 15 tonnes (approx. 20% of legal highway load)**
- **Steel deteriorating**
- **Poor abutments**
- **Pedestrians sharing narrow roadway with traffic**
- **Greater flood clearance desired**

- **Could be removed and not replaced, existing highway bridge nearby**
- **Replacement not in Owner's current budget, until timber bridge incentive program provided**

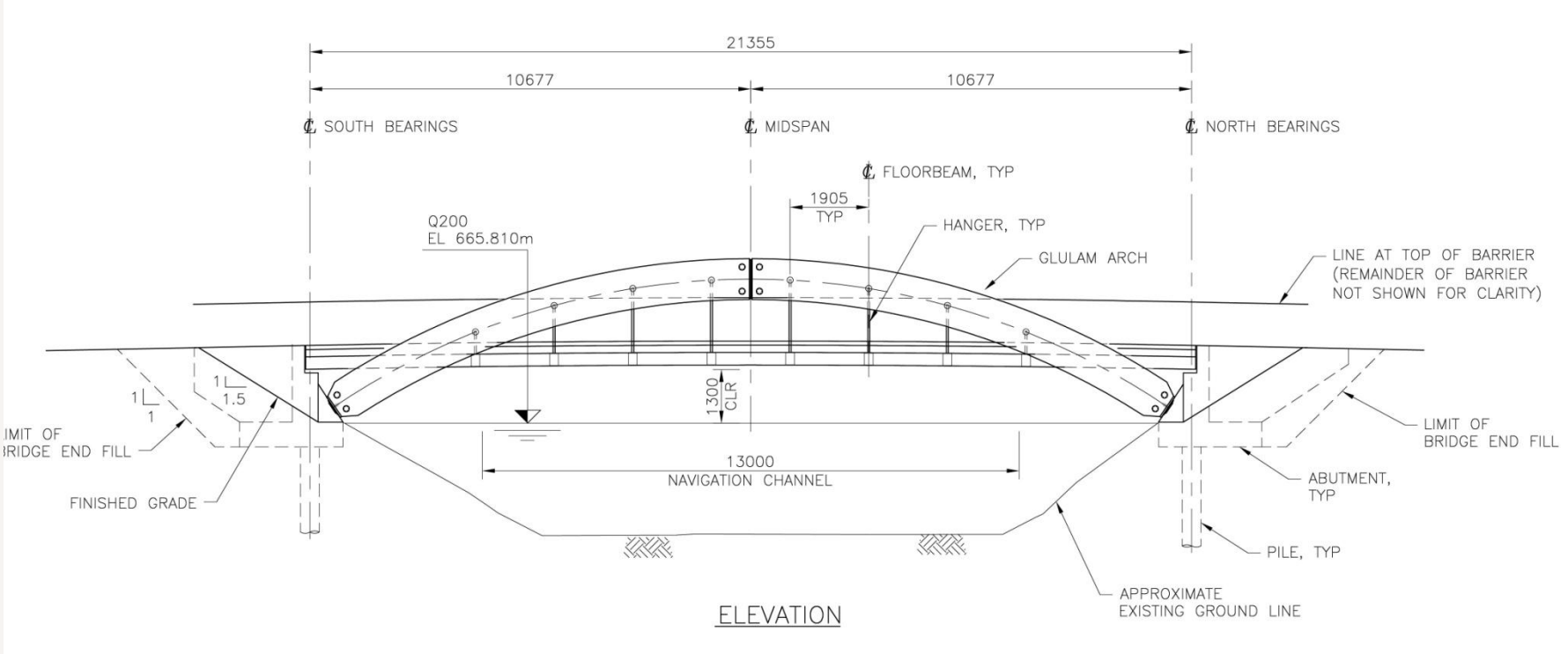
BUCKLAND & TAYLOR | COWI

New Bridge

- **Approx 20m minimum span required**
- **Permanent bridge**
- **No load limit (63,700 kg design vehicle)**
- **Timber structure desired**
- **One traffic lane, but wider to share with pedestrians**
- **Improved flood clearance to be provided**
- **Roadway not to be raised: existing streets and driveways**
- **Available depth for structure 0.6m**
- **Minimize disruption to stream**

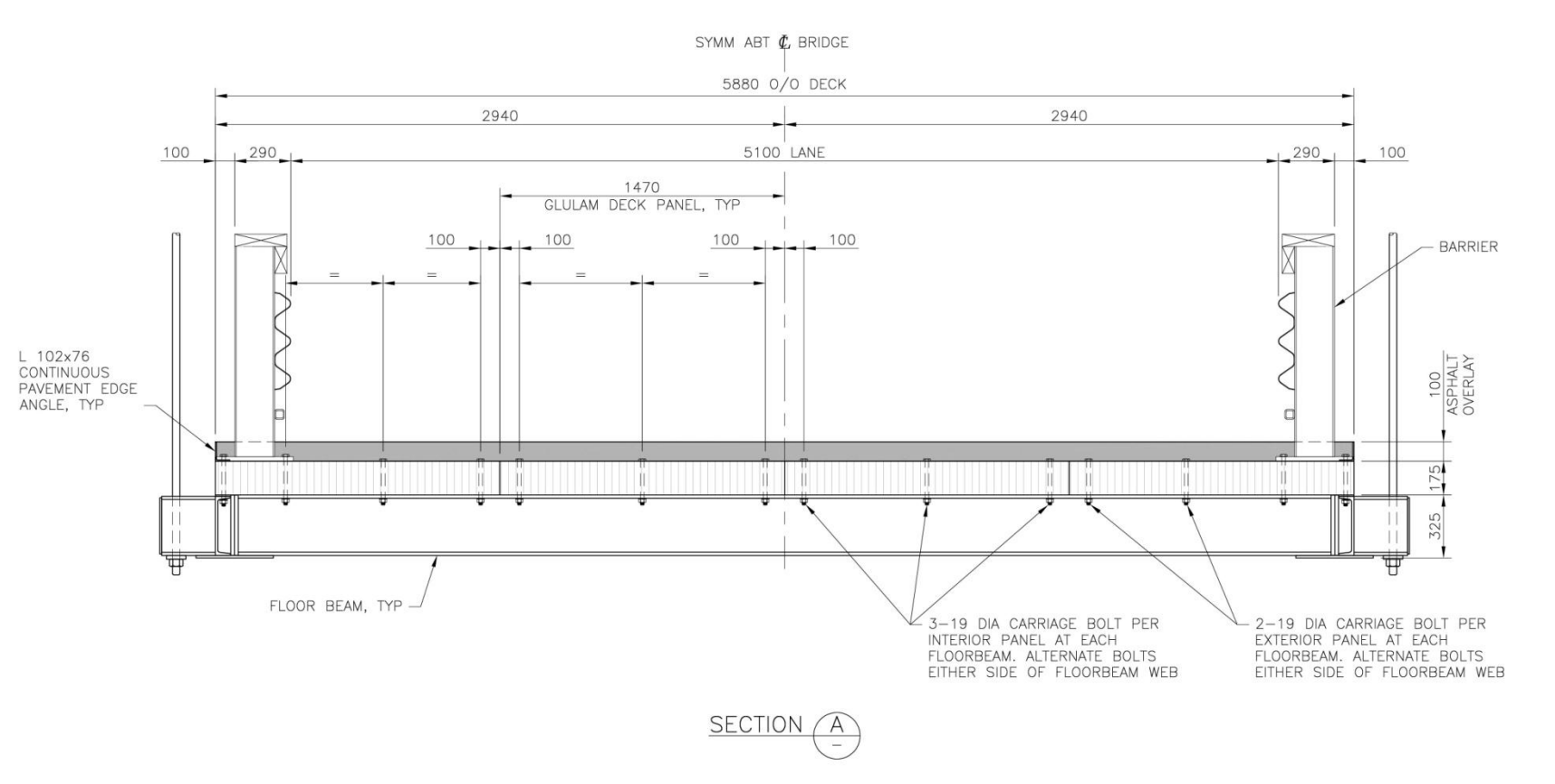
- **Solution: glulam timber arch with shallow deck structure**

New Bridge: Glulam Timber Arch



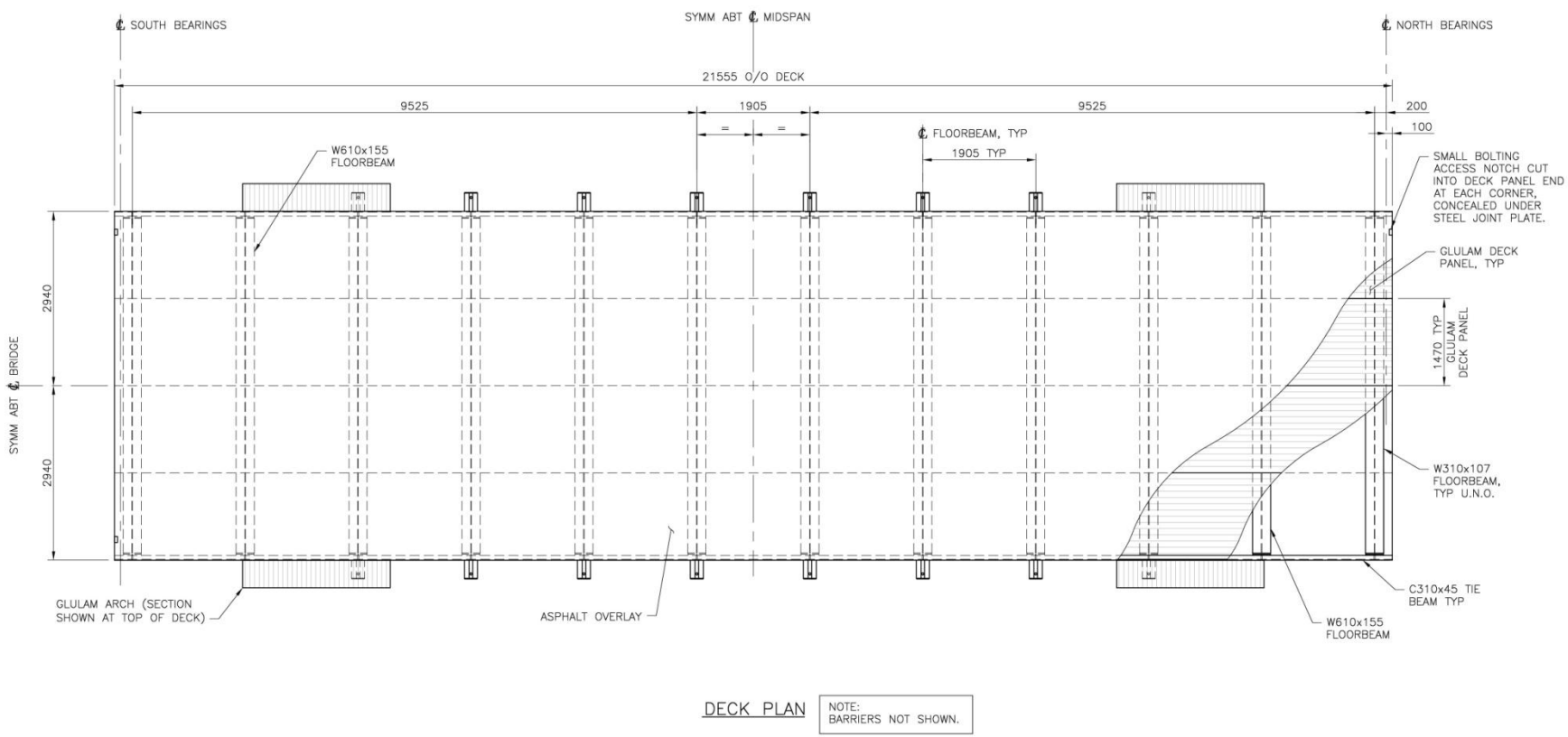
BUCKLAND & TAYLOR | COWI

Deck Cross Section



BUCKLAND & TAYLOR | COWI

Deck Plan



BUCKLAND & TAYLOR | COWI

Minimal Impact on Stream

BUCKLAND & TAYLOR | COWI

Abutments

BUCKLAND & TAYLOR | COWI

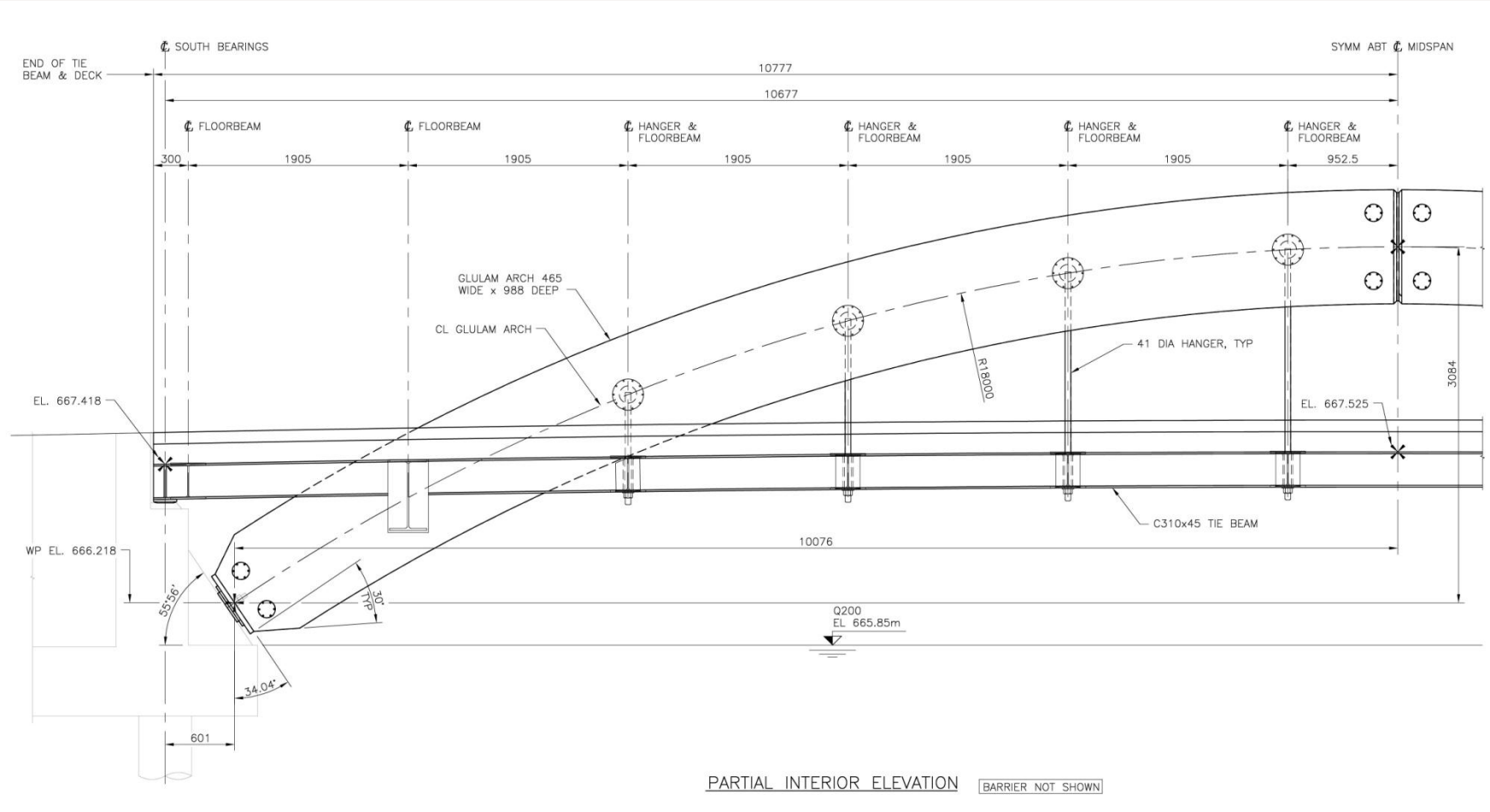
Single Truckload of Glulam Timber

BUCKLAND & TAYLOR | COWI

Galvanized Steel Deck Grid

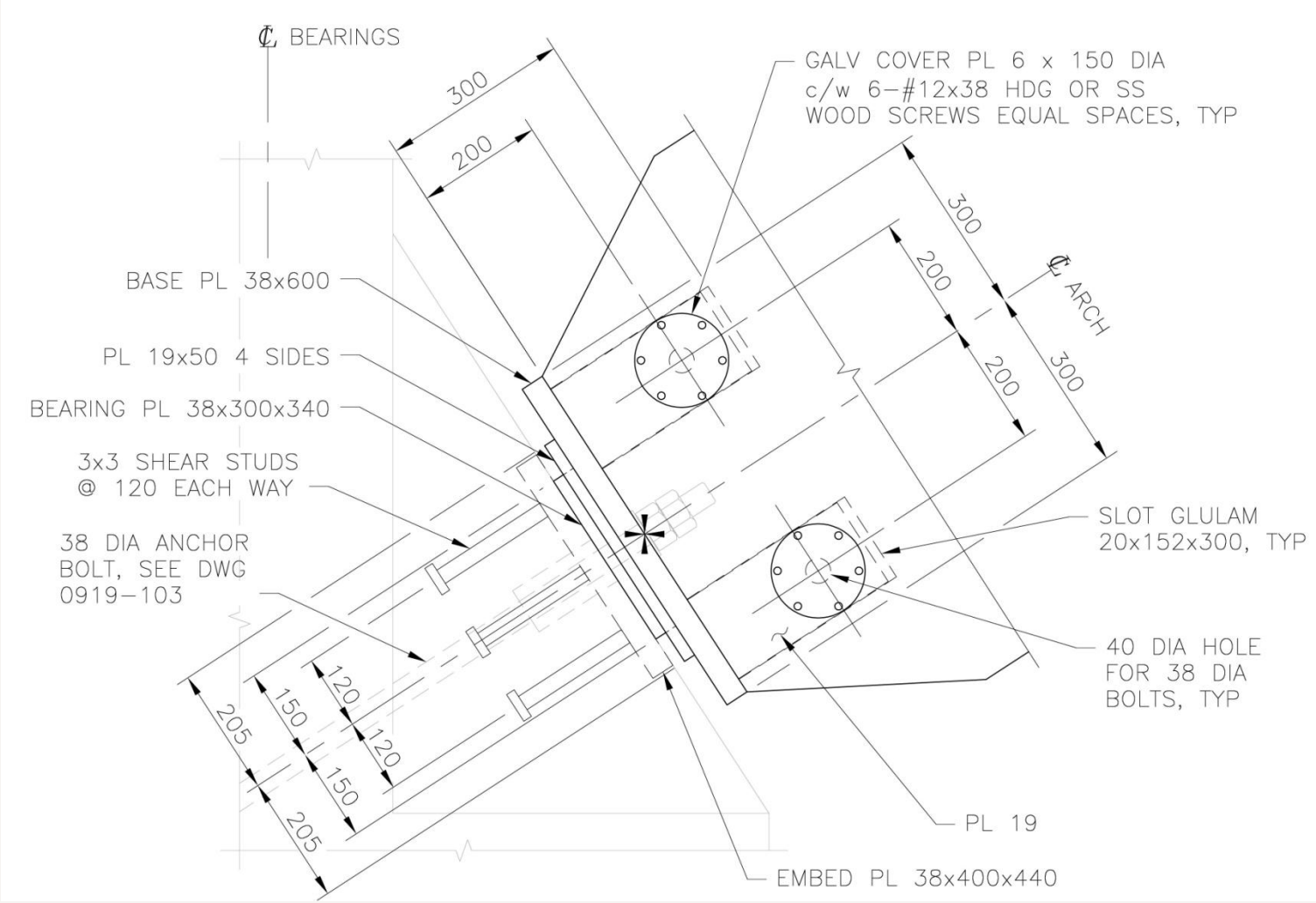
BUCKLAND & TAYLOR | COWI

Arch & Tie Beam



BUCKLAND & TAYLOR | COWI

Arch Bearing



BUCKLAND & TAYLOR | COWI

Arch Erection

BUCKLAND & TAYLOR | COWI

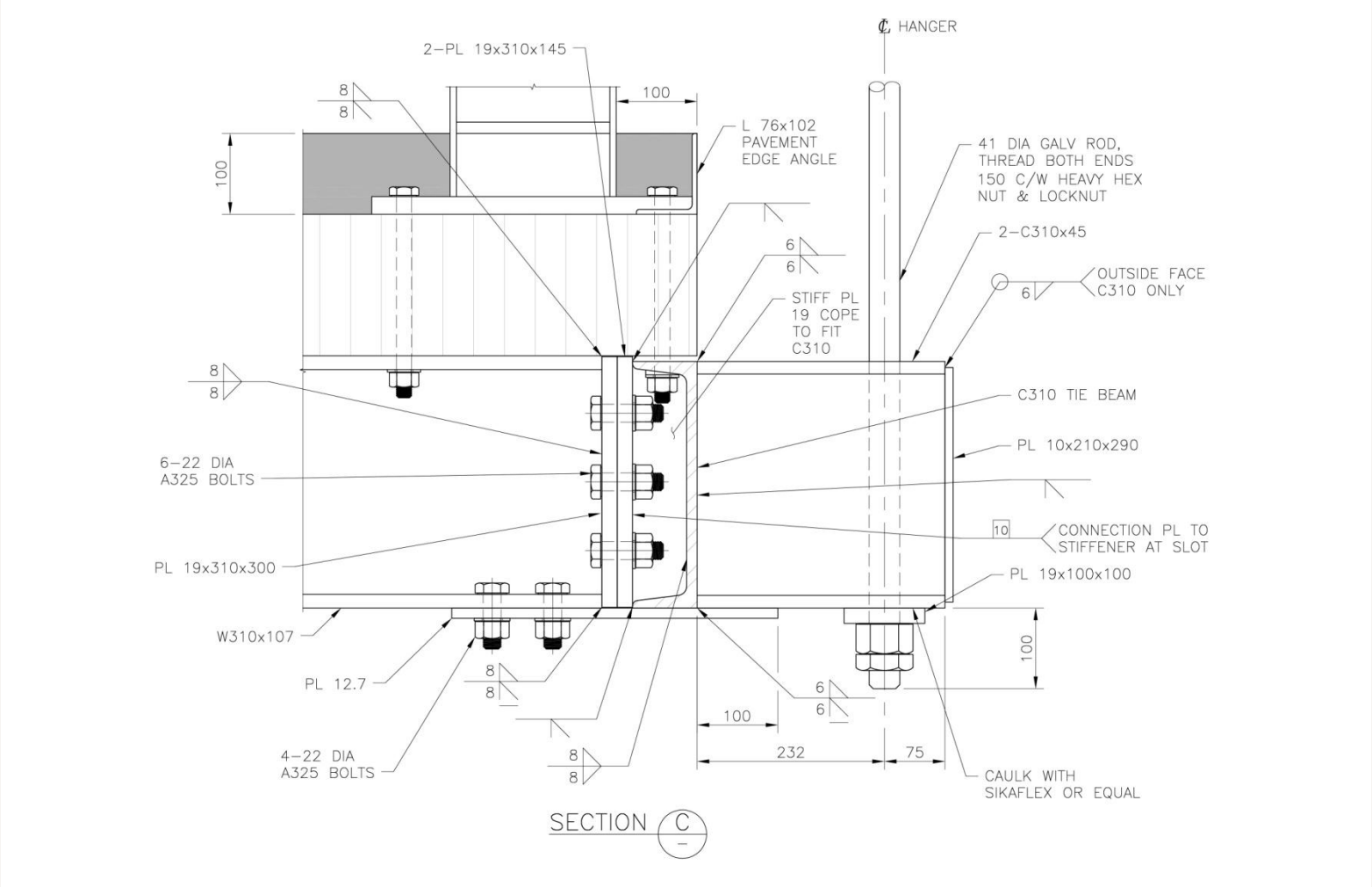
Arch Erection

BUCKLAND & TAYLOR | COWI

Arch End Details

BUCKLAND & TAYLOR | COWI

Hanger / Tie Beam / Deck Detail



BUCKLAND & TAYLOR | COWI

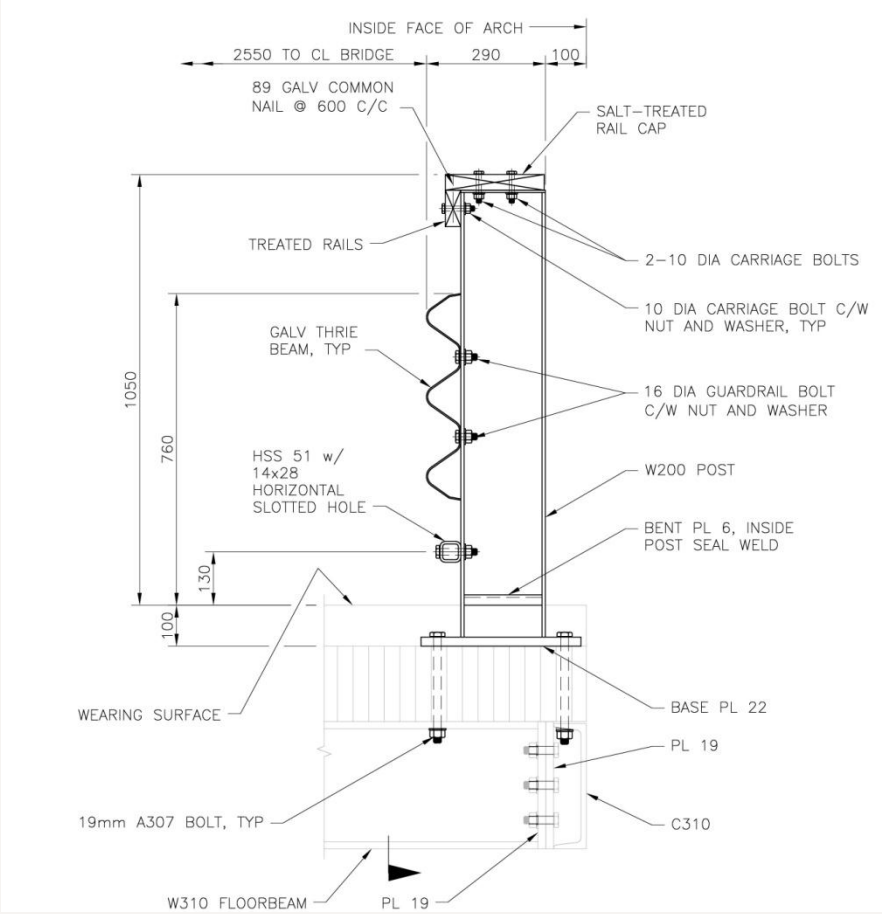
Deck Panels

BUCKLAND & TAYLOR | COWI

Crash-Tested Barrier Design

BUCKLAND & TAYLOR | COWI

Barrier Detail



BUCKLAND & TAYLOR | COWI

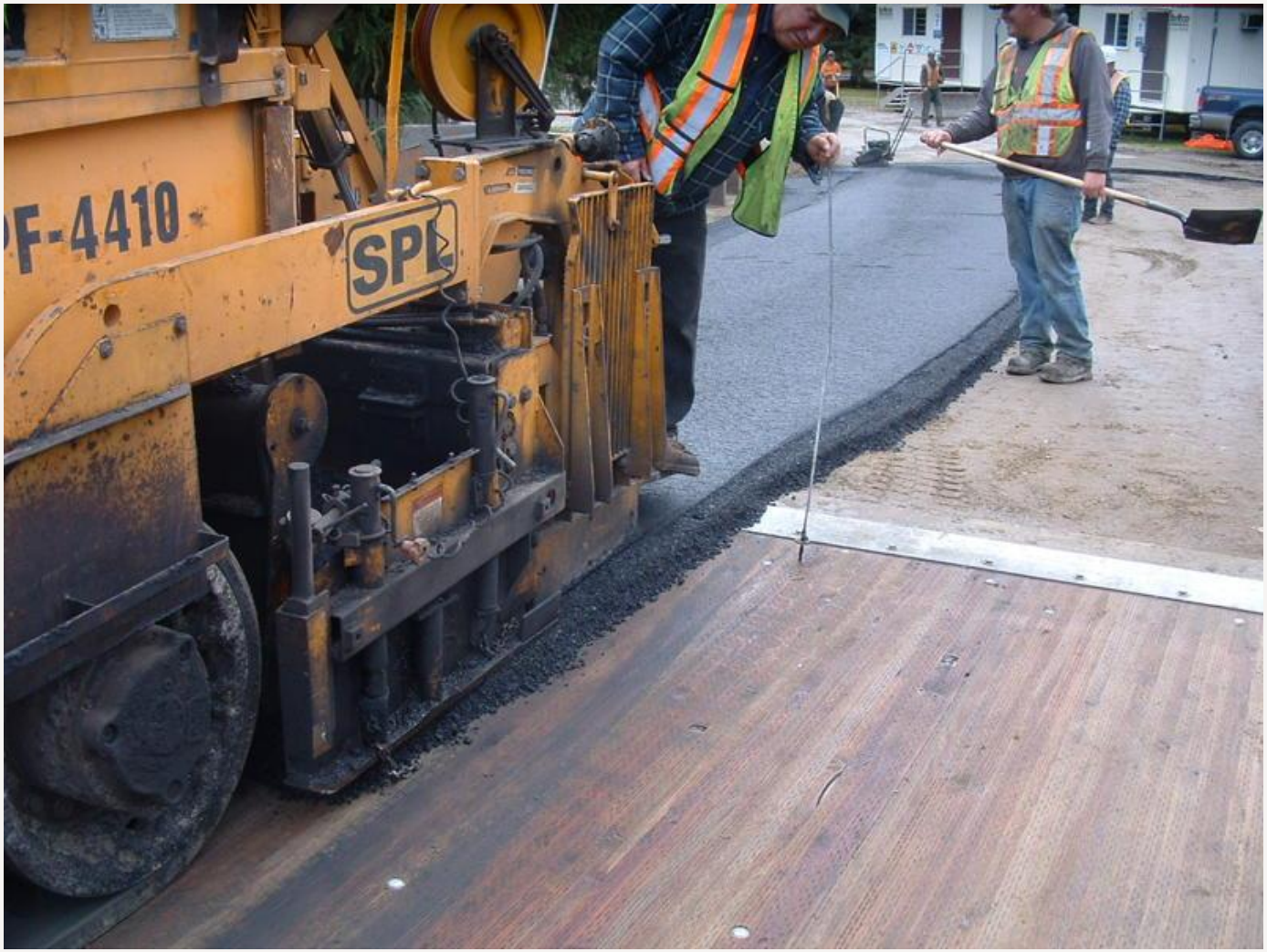
Barrier Post

BUCKLAND & TAYLOR | COWI

Complete Prior to Paving and Barrier

BUCKLAND & TAYLOR | COWI

Paving



BUCKLAND & TAYLOR | COWI

End Joint Sawcutting

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Thank You For Your Attention!
Questions ?

BUCKLAND & TAYLOR | COWI

Rehabilitation of St. Marys Wycliffe Bridge in British Columbia, Canada

2nd International Conference on Timber Bridges

Las Vegas, Nevada

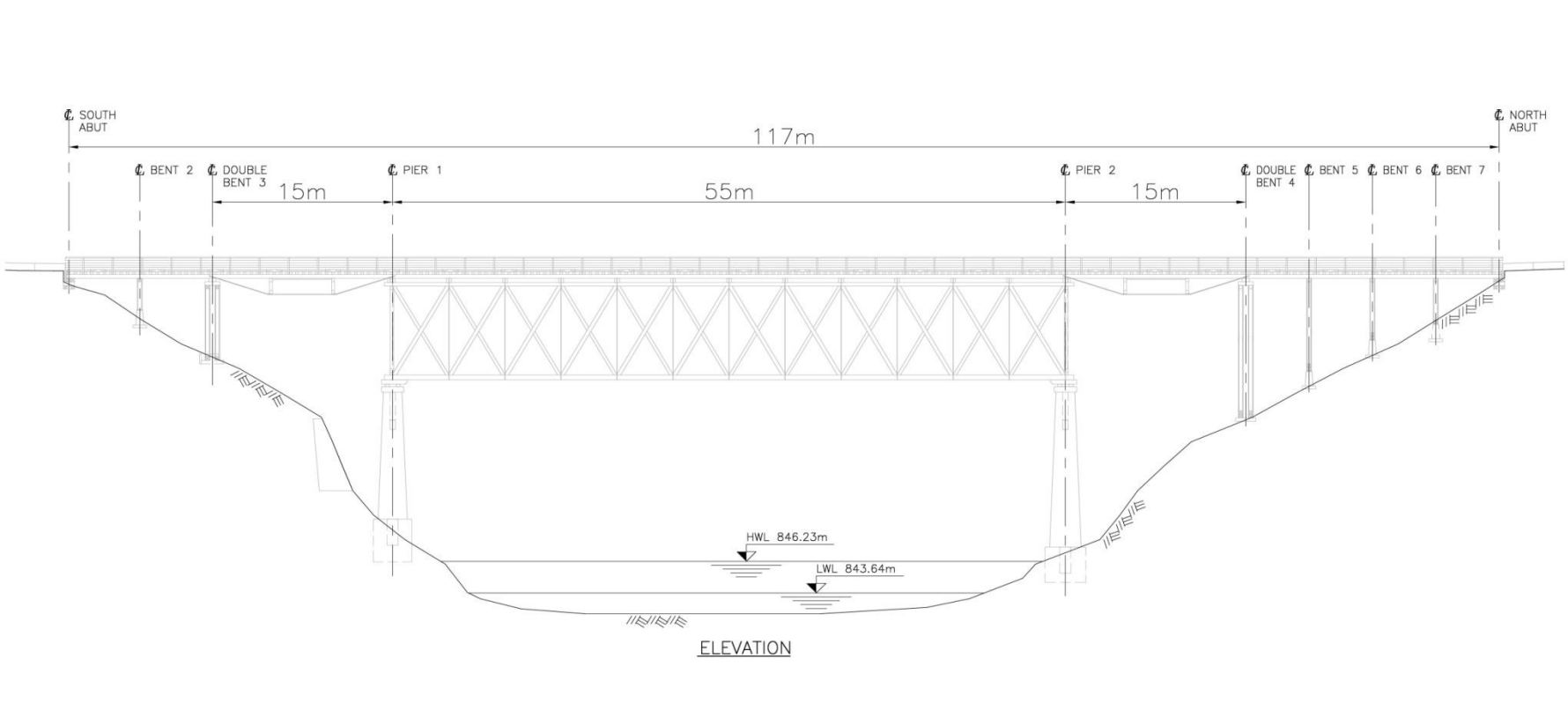
2013 Sep 30 – Oct 02

Murray M. Johnson, P.Eng., PE

Buckland & Taylor Ltd.

BUCKLAND & TAYLOR | COWI

Existing Bridge



BUCKLAND & TAYLOR | COWI

Existing Bridge

BUCKLAND & TAYLOR | COWI

Existing Bridge

BUCKLAND & TAYLOR | COWI

Howe Truss Detail

BUCKLAND & TAYLOR | COWI

Existing Bridge

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Deck Level Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Queenpost Truss Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Queenpost Truss Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Queenpost Truss Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Substructure Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Substructure Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Substructure Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: Substructure Deterioration

BUCKLAND & TAYLOR | COWI

Inspection: South Slope Movement

BUCKLAND & TAYLOR | COWI

Inspection: South Slope Movement

BUCKLAND & TAYLOR | COWI

Inspection: South Slope Movement

BUCKLAND & TAYLOR | COWI

Inspection: Howe Truss

BUCKLAND & TAYLOR | COWI

Inspection: Howe Truss

BUCKLAND & TAYLOR | COWI

Inspection: Main Piers

BUCKLAND & TAYLOR | COWI

Inspection: Main Piers

BUCKLAND & TAYLOR | COWI

Bridge Rehabilitation Design

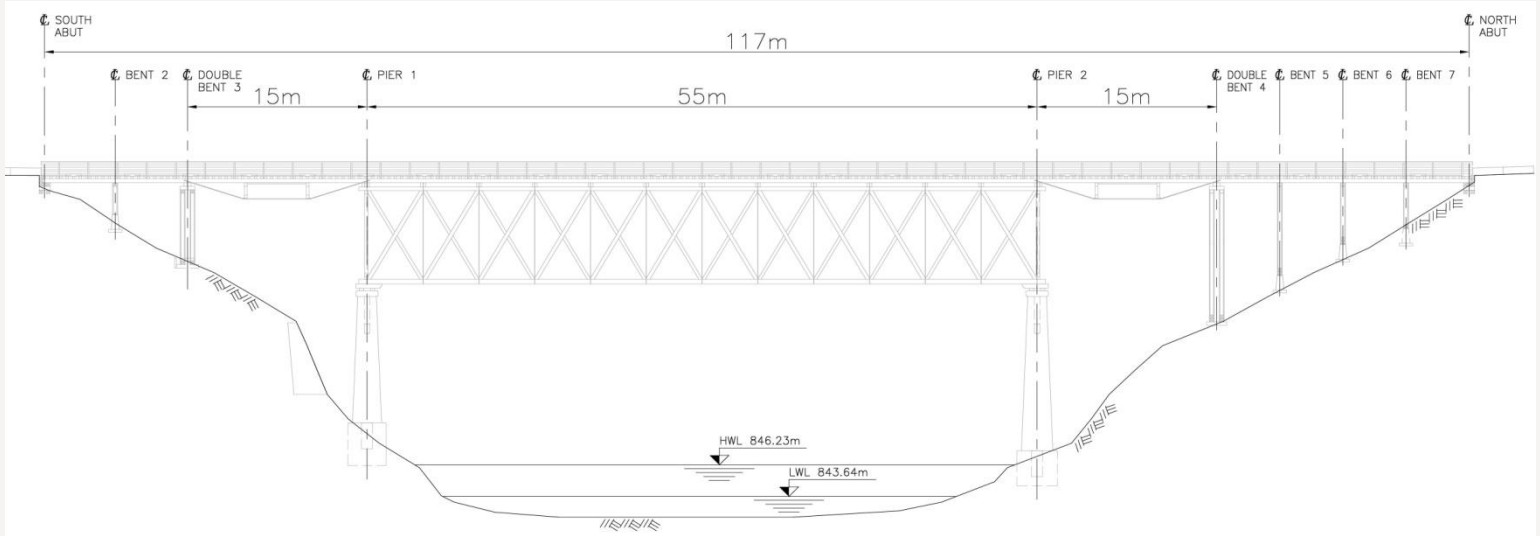
Required:

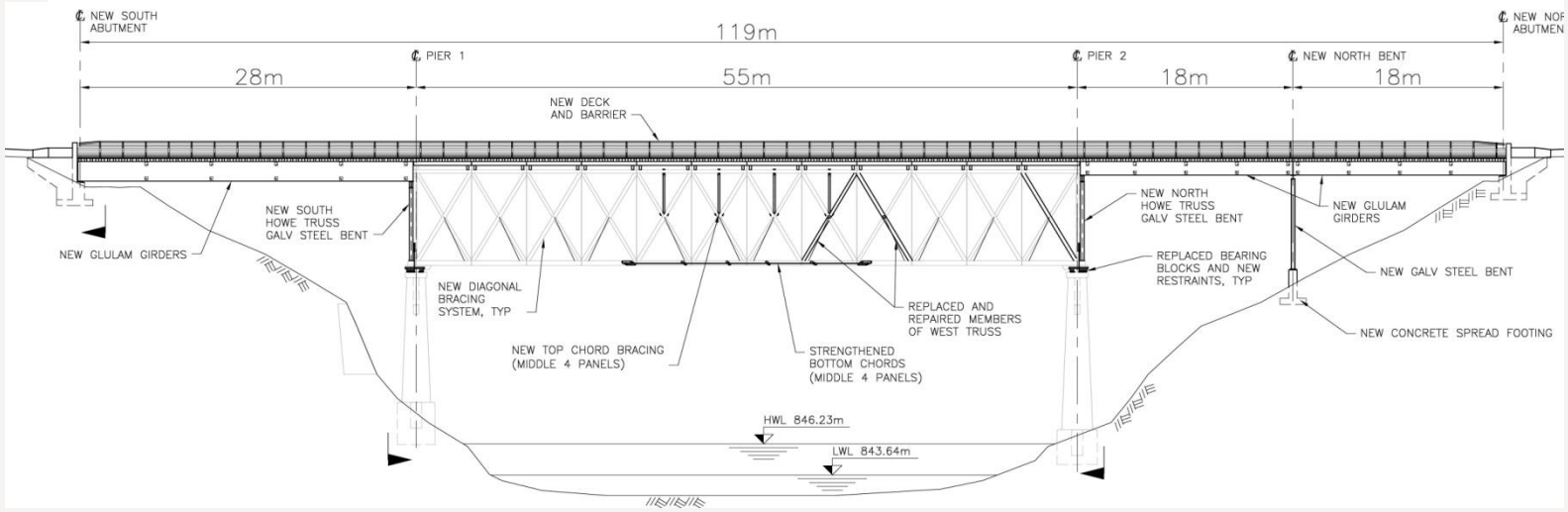
- **Double load limit to 36 tonnes**
- **Pedestrians and cyclists to share roadway**
- **Drastically reduce maintenance requirements**
- **Mitigate south slope movement problem**

Solution:

- **Remove entire deck and barrier system**
- **Keep main Howe truss span (55m), from floorbeams down**
- **Keep existing concrete piers**
- **Remove both queenpost spans and all trestle spans**
- **Build new concrete abutments and new north bent**
- **New 28 m and 18m/18m glulam girder spans**
- **Select truss strengthening**
- **Complete new deck system and crash-rated barriers, cyclist height**

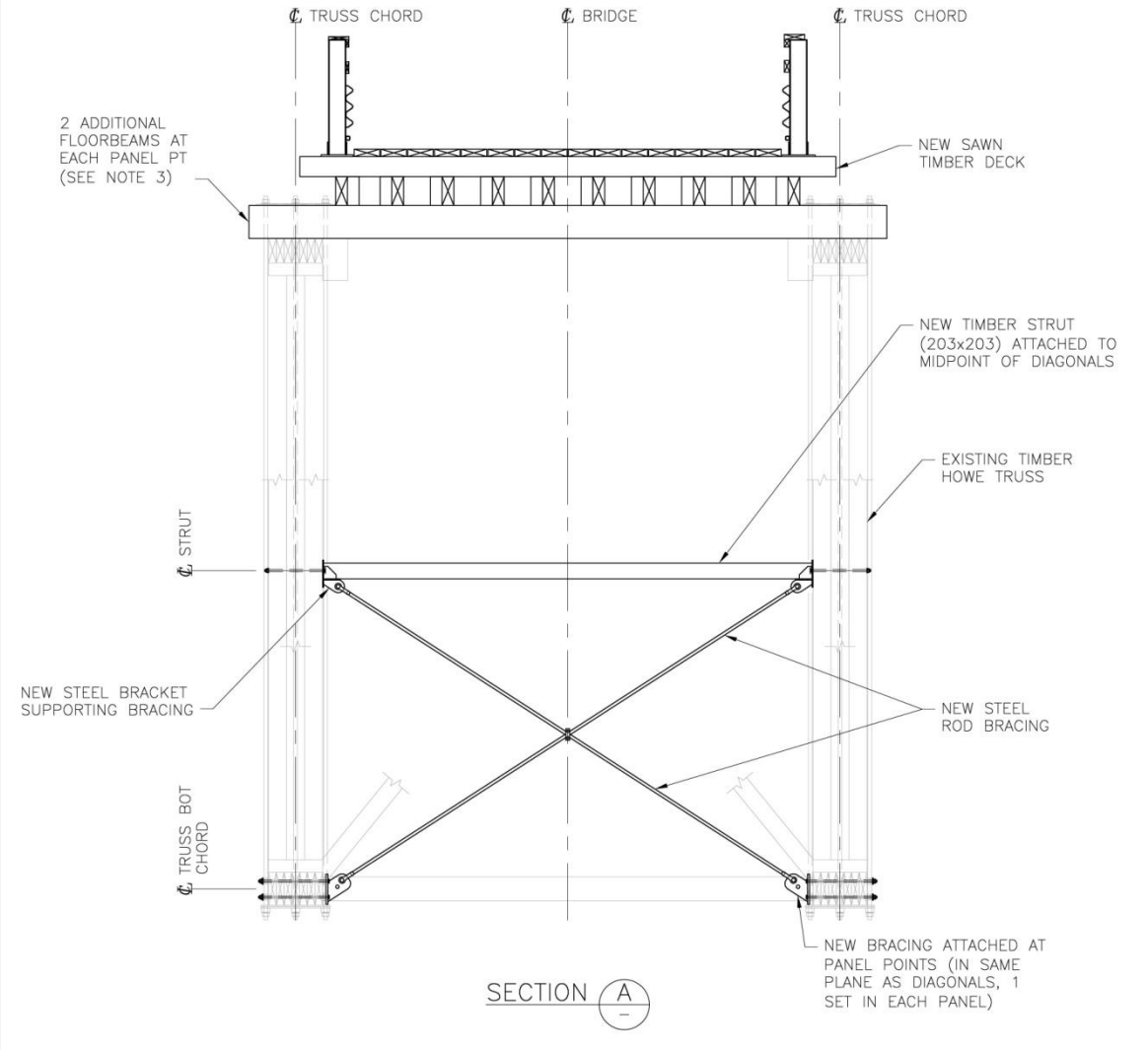
Bridge Rehabilitation Design





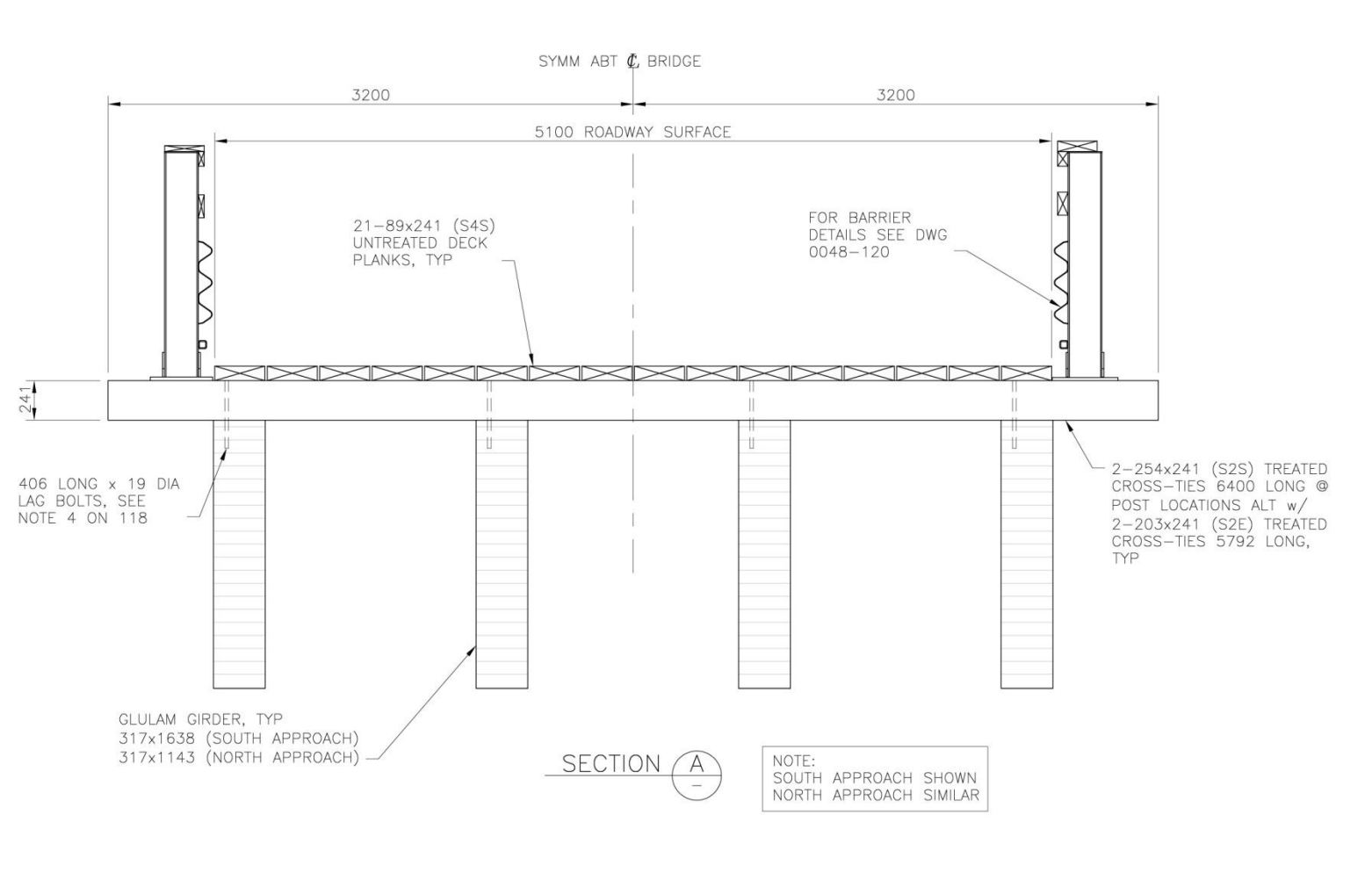
BUCKLAND & TAYLOR | COWI

New Cross-section: Howe Truss Span



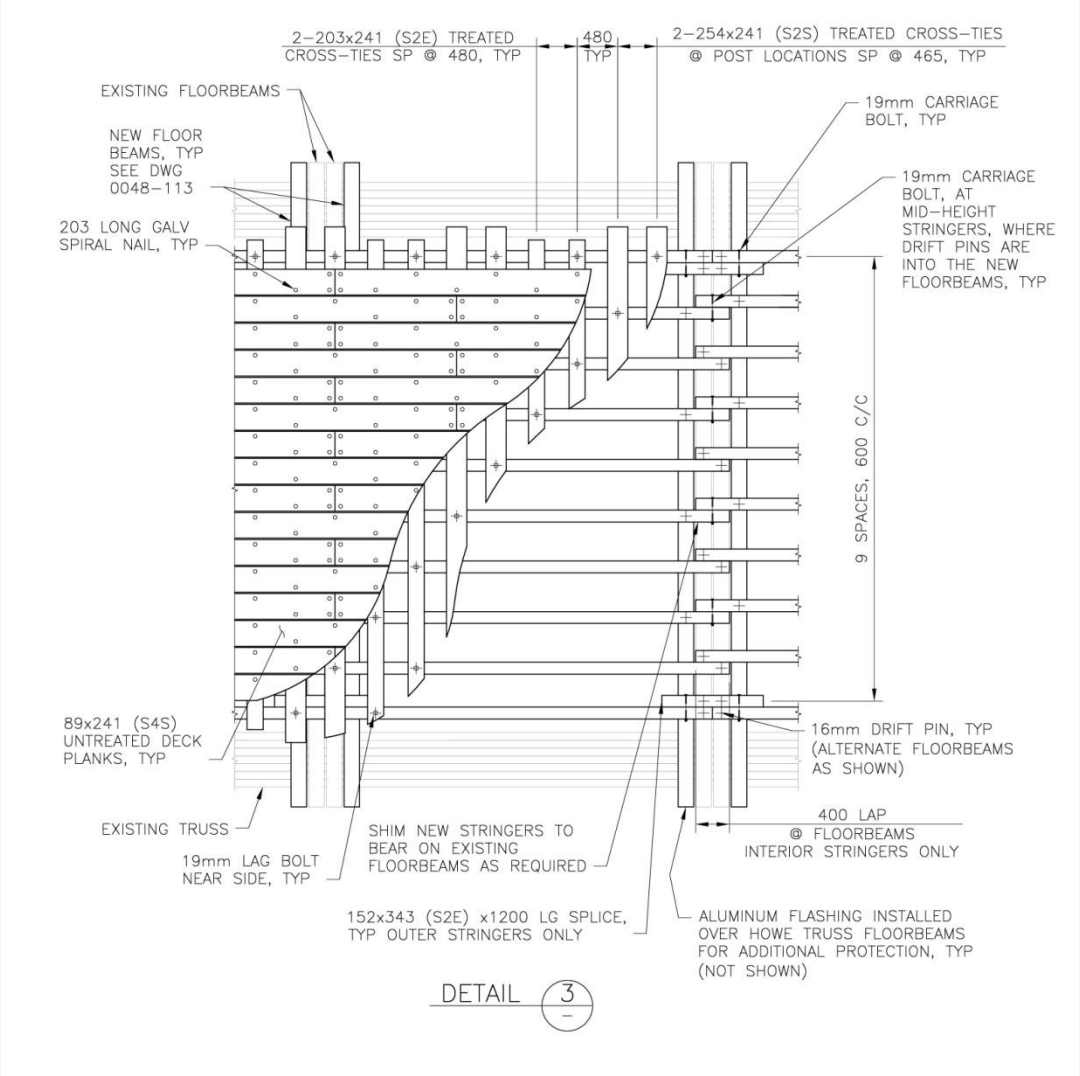
BUCKLAND & TAYLOR | COWI

New Cross-section: Glulam Girder Approach Spans



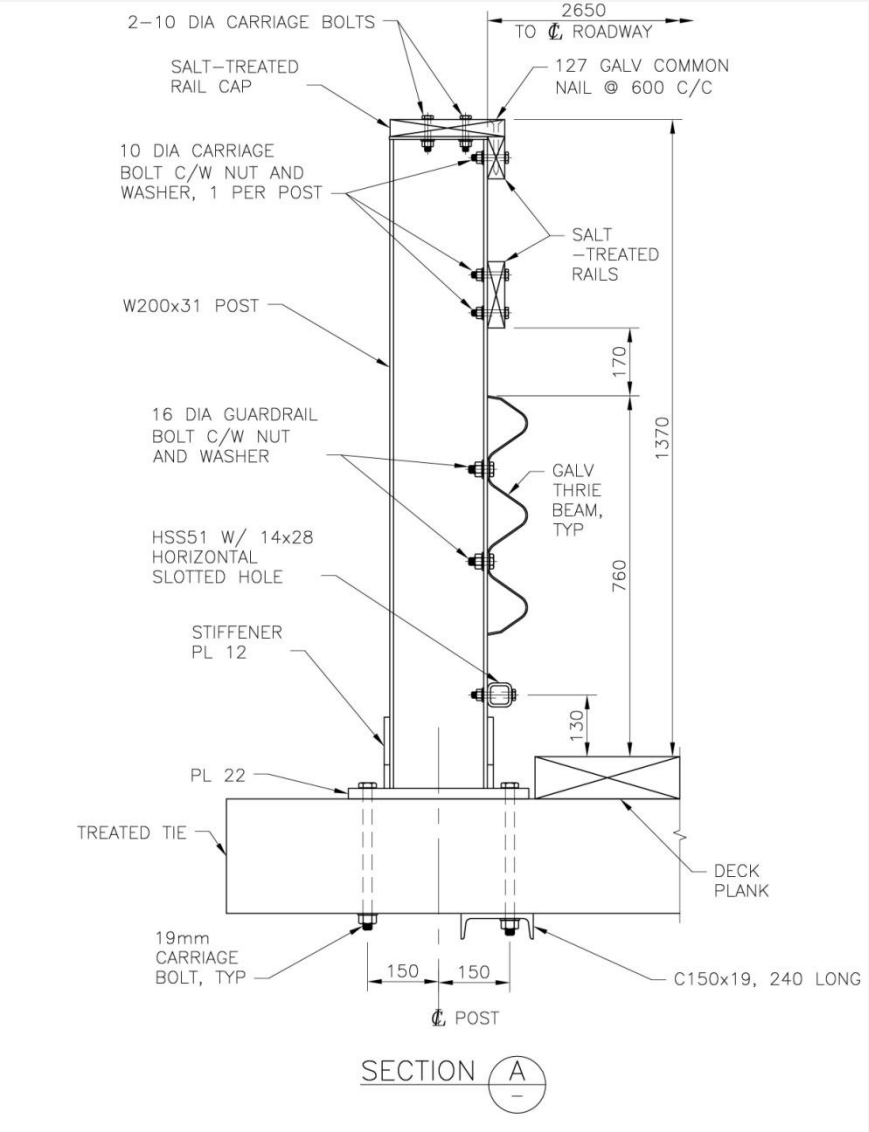
BUCKLAND & TAYLOR | COWI

New Deck System



BUCKLAND & TAYLOR | COWI

New Crash-rated Barrier (cyclist height)



BUCKLAND & TAYLOR | COWI

Construction

BUCKLAND & TAYLOR | COWI

Construction

BUCKLAND & TAYLOR | COWI

Supplemental Floorbeams

BUCKLAND & TAYLOR | COWI

Ready for Deck

BUCKLAND & TAYLOR | COWI

North Glulam Spans

BUCKLAND & TAYLOR | COWI

Construction

BUCKLAND & TAYLOR | COWI

Stringer Installation

BUCKLAND & TAYLOR | COWI

Decking

BUCKLAND & TAYLOR | COWI

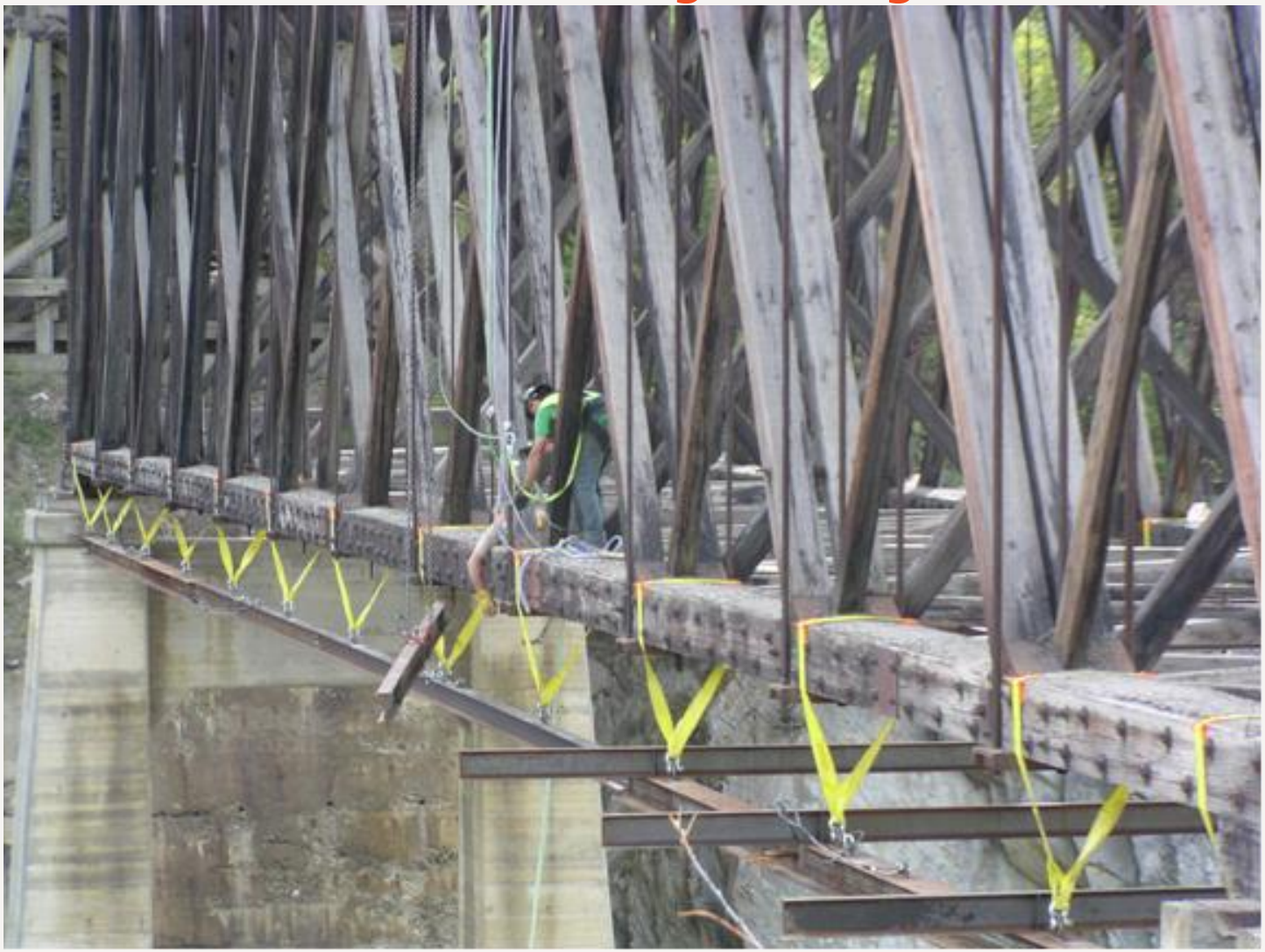
Glulam Girders

BUCKLAND & TAYLOR | COWI

Glulam Spans

BUCKLAND & TAYLOR | COWI

Truss Rehabilitation & Strengthening



BUCKLAND & TAYLOR | COWI

Truss Rehabilitation & Strengthening

BUCKLAND & TAYLOR | COWI

Truss Rehabilitation & Strengthening

BUCKLAND & TAYLOR | COWI

Truss Rehabilitation & Strengthening

BUCKLAND & TAYLOR | COWI

Truss Rehabilitation & Strengthening

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Completed Bridge: Barriers

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Completed Bridge

BUCKLAND & TAYLOR | COWI

Thank You For Your Attention
Questions?

BUCKLAND & TAYLOR | COWI

Inspection of Timber Bridges in the Southern US

P. David Jones, Assoc. Prof. Department of Forest Products, MSU Starkvile, MS, USA
Joseph Dahlen, Asst. Prof. WSFNR, UGA Athens, GA, USA
Rubin Shmulsky, Dept. Head,Department of Forest Products, MSU Starkvile, MS, USA

Summary

Nineteen timber bridges were selected in Alabama, Mississippi, and Tennessee for inspection for the field performance of timber bridges study. All bridges selected had both super and sub structures manufactured out of Southern pine, treated with various preservatives and installed at different points in time. Alabama had four bridges in the study followed by Tennessee with seven bridges and Mississippi with a total of eight bridges. All of the bridges inspected in this study were built using a sawn lumber stringer system.

Keywords: Mississippi, Alabama, Tennessee, Timber, Wood, Bridges.

1. Introduction

1.1 Problem

Timber bridges are widely used in the Southern US, but there use is often related to the county engineer's preference for construction material and cost. Many of the engineers do not know the life cycle of the bridges and may specify bridges manufactured of other materials in place of wood because of a lack of information. The purpose of this study was to examine the condition of bridges in service, and there likely longevity.

In an effort to research timber bridge populations and information pertaining to these bridges, an excel macro was programmed to label and sort through the National Bridge Inventory (NBI) data. The program is similar to what is used on the www.nationalbridges.com website, but differs by including additional information such as average daily traffic, deck type, superstructure and deck rating, bridge width, number of lanes, etc. Having more of the NBI data available in one location has proven useful in narrowing down bridge candidates.
From the NBI data, nine potential clusters throughout Mississippi, Alabama, Georgia, South Carolina, North Carolina, and Tennessee have been identified and are shown in Table 1. These clusters account for both American Wood Protection Association hazard zones 4 and 5 as well as bridges in rural and urban environments.

The clusters were primarily being targeted because of the relatively high number of timber bridges located in a single county or neighbouring counties in their respective states.

Table 1: Potential Bridge Clusters

Potential Cluster	State	Counties	Timber Bridges in County or Counties	Median Year Built	Median Superstructure Rating	Median Bridge Rating	Median Average Daily Traffic	AWPA Zone
1	Mississippi	Hinds	28	1992	7	39	650	5
2	Mississippi	Attala, Choctaw, Montgomery, Oktibbeha, Webster, Winston	109	1967	5	28	50	4
3	Mississippi	Lafayette	68	1992	7	46	55	4
4	Alabama	Pickens, Tuscaloosa	105	1980	7	50	50	4
5	Tennessee	Crockett, Gibson, Henderson, Madison	136	1970	6	53	55	4
6	Georgia	Colquitt, Mitchell, Thomas	78	1980	5	26	140	5
7	Georgia	Bryan, Liberty	78	1993	7	95	500	5
8	South Carolina	Laurens, Newberry	36	1982	6	30	126	4
9	North Carolina	Cherokee, Clay, Graham, Haywood, Jackson, Macon, Swain, Transylvania	275	1961	7	52	220	4

1.2 Bridge selection

The number of clusters was reduced from above down to three clusters. One of the clusters was Pickens County, Alabama, a second cluster was chosen in Lafayette County, Mississippi and the third cluster was located within the two counties of Crockett and Madison, Tennessee. All of the bridges selected are of a sawn lumber stringer construction and had varying deck materials and road surface materials. The bridges in this study were all found in AWPA decay zone 4. A total of nineteen bridges were inspected using non-destructive techniques.

2. Inspections

2.1 Methods used

All bridges were examined ocularly first to identify obvious issues or deficiencies. In many cases this was the method used to find mechanical damage and seepage through the deck onto the stringers below. After the initial inspection the bridge was labelled using sidewalk chalk to identify stream flow, the beginning and end of the bridge, and to number and label the parts of the bridge. Once this was complete, the use of a moisture meter, microsecond timer, and resitograph along with a hammer were used to closely inspect the integrity of the wood.

2.2 Examples

2.2.1 Alabama

All four of the bridges in Alabama were of similar construction, utilizing creosote treated stringers, caps, and pilings. Figure 1 shows and example of the bridge construction in Pickens county Alabama. The bridges were constructed between 1986 and 1997.

Figure 1. Alabama bridge 14268

2.2.2 Mississippi

All eight bridges inspected in Mississippi were treated with CCA and had similar construction. Figure 2 is an example of the timber bridges found in Lafayette county Mississippi. The bridges ranged in age from 22 years in service to only 16 years in service.

Figure 2 Mississippi bridge 77

2.2.3 Tennessee bridges

The seven bridges examined in Tennessee were of a similar construction of those in Mississippi. Chromated Copper Arsenate treated Southern pine sawn stringer construction, with similarly treated piles and caps. The decks were varied in that some were concrete poured in place and others were lumber based. The bridges in Tennessee that were inspected were built between 1960 and 1974 with improvements made over that time.

3. Discussion, Conclusions and Acknowledgements

3.1 Discussion and Conclusions

The purpose of inspecting bridges in the Southern US was to collect data for the Field Performance of Timber Bridges: A National Study was to examine how timber bridges have held up over time. The timber bridges inspected in Alabama, Mississippi and Tennessee were all of similar sawn stringer construction, spanning both active streams and relief streams that help reduce flooding. All of the bridges that were inspected had been in place for more than two decades and were still ranked well using the NBI guidelines indicating that timber bridges hold up better than previously considered.

3.2 Acknowledgements

The authors would like to thank the USDA US Forest Service Forest Products Laboratory of funding of the project and key support when questions needed answers. We would like to thank the following people:

Crocket County, Tennessee, Mr. Milton Legions

Madison County, Tennessee, Mr. Bart Walls

Lafayette County, Mississippi, Mr. Larry Britt

Pickens County, Alabama, Mr. Daniel Hallman

Their help in selecting bridges and reviewing files was crucial in starting this project. Finally we would like to thank Mr. Brian Lindsey for helping perform the inspections.

ontribution to Structural Details on Timber Bridges

Otto Kleppe
Senior Principle Engineer
Norwegian Public Roads Administration
Oslo, Norway
otto.kleppe@vegvesen.no

Otto Kleppe, born 1943, received his civil engineering degree from Østfold University College, Norway. He has worked with bridge design since 1970 and with timber bridges for the last 23 years.

Hauke Kepp
Principle Engineer
Norwegian Public Roads Administration
Oslo, Norway
hauke.kepp@vegvesen.no

Hauke Kepp completed his M.Sc. degree in structural engineering at the Norwegian University of Science and Technology in 2009. He has since worked at the bridge and construction department of the Norwegian Public Roads Administration.

Tormod Dyken
Senior Principle Engineer
Norwegian Public Roads Administration
Oslo, Norway
tormod.dyken@vegvesen.no

Tormod Dyken completed his diploma in civil engineering at Graz University of Technology, Austria in 1972. He has worked with design and engineering of concrete offshore structures for 20 years and with timber bridges at the Norwegian Public Roads Administration for the past 15 years.

Summary

The paper makes a contribution to details on timber bridges based on the experience gathered by the Norwegian Public Roads Administration (NPRA). The design errors which are included in this paper are selected because they are important with respect to durability. Most of what is stated in this paper may appear to be obvious. However, the problematic details are still used in the design of timber bridges and are therefore not obvious for all designers.

Construction of modern timber bridges in Norway started in 1995. Some were constructed in the 1960s and 70s, but using a different approach. We therefore have nearly 20 years' operating experience with modern timber bridges. The experience so far indicates that some of these bridges will not last as long as intended; however, some will most likely be in operation for more than 100 years. The difference lies in the detailing – in most cases related to design to keep water away. We are convinced that in the future we will find further damage that will influence the service life of these bridges.

The most serious problems are leakage of membranes, members placed on top of the other in stagnant water, cracks in exposed timber surfaces where the cracks collect water, and corrosion on tendon bars. This can cause a substantial reduction in service life compared to the expected 100 years.

Keywords: Details, timber bridges, deterioration, decay, moisture content, protection of timber, service life issues.

Fig. 1 Evenstad Bridge, constructed in 1996, five spans, dual carriageway, total length 180 m

1. Introduction

Construction of modern timber bridges in Norway started in 1995. We therefore have nearly 20 years' operating experience – not very long compared to the intended service life, but long enough to ascertain what will definitely not work. During the 19th century, a large number of timber bridges were constructed in Norway. None of these are in service today. The last one was probably demolished in the 1960s, when a new era of CCA-preserved timber bridges began. Most of the CCA-preserved timber bridges from the 1960s are still in use; however, some have needed comprehensive maintenance because of poor detailing and most of them have acquired a shabby appearance.

Today the Norwegian Public Roads Administration (NPRA) has approximately 60 timber bridges on the national road network and 70 on the county road network. In addition there are some on the municipal road network and some few private timber bridges. In total we have nearly 200 timber bridges, both road and pedestrian.

The design service life for bridges in Norway is 100 years. This requirement also applies to timber bridges. We know from moisture measurements that the moisture content in general is around 12 –17% for protected members and that this is fairly independent of both location and surroundings. This level of moisture content is well below that where fungus may develop.

The main bridge types used in Norway are arch bridges and some truss bridges. For smaller bridges both slab and girder types are used. Stress-laminated deck with creosote-impregnated lamellas, waterproof membrane and asphalt wearing course is widely used. Both glulam and sawn timber are used as lamellas.

The cross-sectional size of members of the main load-bearing structure mostly requires block gluing of glulam elements. In such cases the glulam is built up using copper-treated lamellas. In addition the members are normally treated with creosote and generally have metal cladding on top, often copper sheeting. If the members are too large to be accommodated in the creosote treatment tank, the lateral faces are provided with structural protection, frequently with a timber louver boards.

1.1 Monitoring of five timber bridges

1.1.1 Moisture content

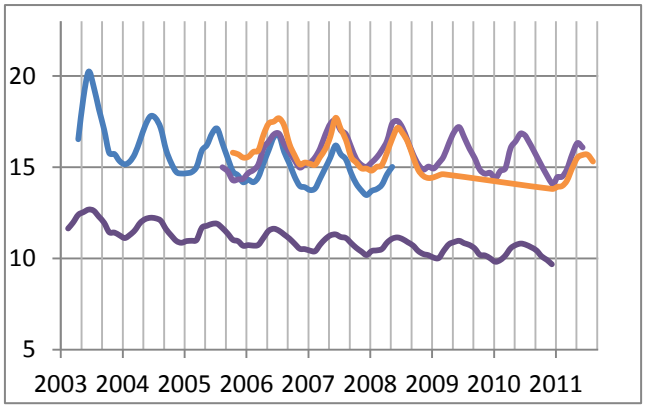


Fig. 2 Moisture content from monitoring of four bridges [1] plus in-house adaptation

Error! Reference source not found. shows he measured moisture content of four different bridges. With the exception of one, the bridges experience very similar moisture levels. However all the measurements show that the moisture content continues to decline after erection, and the lowest graph represents Evenstad Bridge which is also the oldest of the monitored bridges.

The intention of measuring is to document the durability of timber bridges and to understand their response to different climates. In addition, it is important that the moisture level is not rising.

1.1.2 Stress reduction on tendons over time

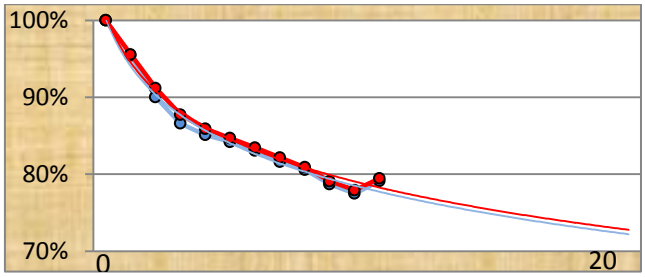


Fig. 3 Yearly average tension forces in steel bars over ten years at Evenstad Bridge [1].

To the left a graph represents the yearly average tension force in three neighbouring bars. The intention of measuring the tension force is to find the point of time where the bars have to be re-stressed. According to the design rules, a loss of bar force of 50% for sawn timber and 40% for glulam beams must be allowed for. The collected data show that 20 years between re-stressing should be sufficient.

1.2 Expected service life for timber bridges

Timber structures in Norway have lasted for more than 900 years when properly maintained. Creosote-treated electricity pylons with no other protection than the treatment or maintenance last for more than 60 years. The timber bridges have structural protection and in combination with creosote impregnation we expect them to be in service for a very long time.

1.3 Maintenance on timber bridges

In general the maintenance on newer timber bridges has been very little to date. Where problems occur they are mainly design issues and some details will need reconsideration. As the bridges are assembled on site, it will likewise be possible to disassemble the bridges and change parts if necessary.

1.4 Appearance of deterioration (decay) of structural timber

We have recognised that some bridges are deteriorating far too fast. This is nearly always due to lack of attention to precipitation and water flow. Timber will not endure moisture content above 22% for a long time.

Reduced service life may be due to various causes, such as poor detailing, construction errors, failure to repair damage and poor maintenance in general. Even if a bridge is designed to be virtually maintenance free, some maintenance will always be required. In this paper we wish to highlight the causes of reduced service life related to poor detail design, as this is the basic precondition for a long service life.

During recent years we have carried out a number of special inspections of our timber bridges with particular focus on durability and design errors. As the same observations have been made on various parts of the structure, we have tried to group the findings according to the

structural effect:

- Precipitation running off from one member to one below
- Stagnant water which is absorbed into the timber sections
- Dirt or debris on the construction section keeping the moisture content high
- Horizontal member without structural protection
- Ionian water dripping onto metal with lower ionian potential
- Movement of dowels
- Corrosion on the pre-stressing bars (tendons)

2. Precipitation running off from one member to the one below

Problem description

Fig. 4 The rainfall follows the steel parts to the timber parts.

Precipitation collected by a bridge part, and emitted in a concentrated form on the bridge construction, occurs more frequently than expected. An example is the parapet posts fixed to the side face of the bridge deck – see figure to the left. Rainwater gathers on the post and is directed to the side face of the timber deck causing a concentration of moisture in the timber. This solution is very rarely used today.

Fig. 5 Parapet post which do not lead water to the deck

Instead of a plate direct bolted to the side face, shown in the previous image, we have a bolt connection between the post and an anchoring plate which is direct connected to side face of the deck – see fig. 5. The two upper bolts is approximately 1 meter and skewed into a steel plate slotted in between to lamellas in the deck. The two lower bolts are very short. The drip nose on the deck allows the water to drip outside of the all anchoring plates.

Fig. 6 A V-shaped column with stays.

In the figure to the left a wet top face of a V-shaped column is shown. Water follows the top face of the inclined members and is concentrated at their joint. From there the water runs down the side and is sucked up by the end grain.

The increased moisture at the end grain is in general probably one of the more serious problems we have with timber bridges. It occurs at several locations on a several timber bridge.

Consequences for service life and maintenance action

Heightened moisture content due to a concentration of water from leakages is a serious problem. The seriousness depends on the precipitation and could at worst lead to a substantially reduced service life. Maintenance action will be to tighten the leakage, direct it to a location where it will not harm, or to change deteriorated members.

Possible alternative, improved design

The parapet post should have a drip nose facing away from the bridge deck. The leakage between the edge beams could be avoided by mitre-cutting the beams such that the cut faces upwards. The columns should have a cladding that conducts water away from the lower sections.

3. Stagnant water which is absorbed into adjacent timber sections

Problem description

Fig. 7 Pole sleeve at support.

Sometimes adjacent structural members retard the drying of wood because proper circulation of air is prevented. This is the case with all exterior steel plates and sleeves. Even if there are drainage hole in the sleeves, the moisture will remain there for a long time and will eventually cause decay to the timber. Fig. 7 shows a typical example.

The steel plate on the outer side of the structure will in addition to the original intention prevent the wood from drying out. The bolts and the pressure against the wood cannot prevent moisture from penetrating behind the plate. In this particular case the moisture comes from behind the plate. In this case separate washers would most likely be a better solution.

Fig. 8 Exterior steel plate collects moisture.

Fig. 9 Water on a steel beam has over time decayed the timber beam.

The timber bridge to the left has an open deck consisting of planks with small openings resting on beams. The underlying beams are therefore not protected and rot has developed at the cross-girders. The bridge was constructed in 1991 and is CCA-impregnated.

Also where massive bridge decks are used, this can be an issue as water stays on the cross-beam creating a pool of water that causes localised wetting of the deck.

Fig. 10 Open abutment shelf which collects water, dirt and gives high moisture to the sill

Problems at the abutment are by no means only related to timber bridges. However, many of the problems at the abutment of timber bridges are often related to design solutions that are well known from concrete bridges and are applied to timber bridges without giving much consideration to the difference in material behaviour. When dealing with different types of materials, such as at the abutment, where timber and concrete will interact, special care has to be taken. Often standard solutions for concrete bridges do not constitute viable solutions.

In general the use of expansion joints is not really desirable on timber bridges because they normally represent a weak point in the structure. They give rise to additional cost, and also require special attention with respect to maintenance. In addition, they have to be specially created for timber-concrete connections.

Fig. 11 The timber deck are in direct contact with the concrete

Casting concrete directly against the timber deck may look like a good solution at the outset, but over time the timber creeps and moves and cracks are created in the intersection between the two materials. Water will be trapped and remain for a long period before drying.

Consequences for service life and maintenance action

Standing water may be difficult to detect in cracks and around sections that are not easy visible. Also if the bridge is inspected in dry weather, design faults may not be discovered.

Improving alternative design

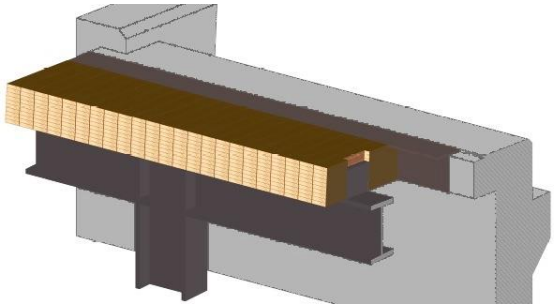


Fig. 12 Possible improved design

For the abutment a new design approach will be tested. The abutment differs from previous designs by having considerable free space below the intersection between concrete and timber in order to prevent standing water and dirt from gathering close to the end wood. In addition the joint between timber and concrete is easily inspectable from below.

4. Horizontal member without structural protection

Problem description

Fig. 13 Fungi is growing on a horizontal member. The photo is taken in 2009.

Horizontal members without any structural protection soon experience fungus attack in the cracks that face upwards. Cracks must be expected in members that are not creosote-impregnated. Upward-facing cracks will collect water that will remain there for some time and dry up very slowly. Fungus spur is always in the timber and starts growing when conditions are amenable to this.

It is not possible to completely avoid this through other means than structural protection. One can postpone fungus growth by using chemicals, but 100 years of service life is unlikely by only chemicals.

Consequences for service life and maintenance action

Fungus attacks in cracks grow from the inside and are difficult to reveal before the attacks are severe.

Improving alternative design

Fig. 14 The Leonardo da Vinci bridge also had fungus attack before it received structural protection to its upper face of the arcs. Constructed in 2005

All timber bridge should have structural protection on the upper side of the sections.

5. Dirt or debris on the construction section keeping the moisture content high

Problem description

Fig. 15 Dirt on the cross-beam

A concrete cross-beam with dirt from the road gathering next to the deck. The dirt will retain moisture. The cross-beam also follows the transversal slope of the deck and road. This will conduct water to the deck. In spite of the deck is lifted up from the beam by a list the water will be transfer to the deck.

Fig. 16 A clean and better solution as above

The cross-beams to the left will most likely experience the same problem as the above-mentioned cross-beam, but in less degree. An even better design for this cross-beam would be to incline the beam before the edge of the deck see fig 17. In this way dirt and water in general would run away from the deck when washed by rain.

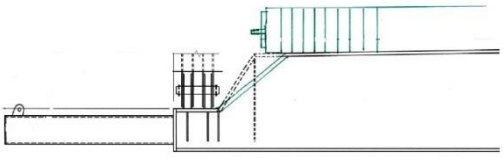


Fig. 17 Possible improved solution

6. Ions dripping onto metal with lower potential

Problem description

Fig. 18 Metals of different relative nobility staying in contact with each other may cause galvanic corrosion.

Using metals with different electronic potential that are in contact with one another can result in reduced service life. When ions transported in water drip from a metal with higher electronic potential onto another metal with a lower potential a bi-metallic corrosion is created. This rapidly tears into the zinc layer of galvanized steel and causes a loss of protection of galvanized members.

We have observed this problem when water runs off the cladding and onto cross-members or joints.

Consequences for service life and maintenance action

The metal with the lowest electronic potential will corrode and part of it will disappear. The maintenance action will be to replace the metal that has disappeared and to conduct the dripping water to another location where it does not corrode the underlying part.

Improving the section or introducing an alternative design

The design has to include drainage from an upper section to a suitable location.

7. Moving of dowels

Problem description

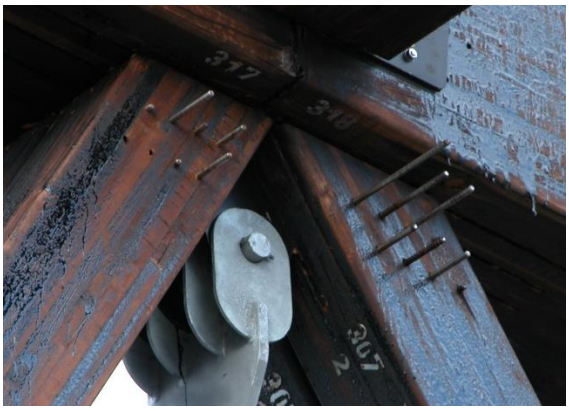


Fig. 19 Dowels dislodging from the structure

In some cases the horizontal dowels have moved out of position. This has happened on horizontal dowels with alternating load direction. The dowel is not quite horizontal, and when the load on the members alternates between compression and tension they can dislodge. The problem is observed on some bridges and only on joints which alternate load direction.

Consequences for service life and maintenance action

This movement of dowels may be very serious if they are not pushed back in time. This can reduce the service life of some bridges when it exceeds a certain point. In any case it will increase the maintenance cost. If damage has occurred to the bridge because the dowels have moved too far, the cost of repair may be substantial and the bridge will probably need to be closed for a period.

In the worst case, dowels that have become displaced can result in bridge collapse and this is

therefore more of a safety issue than a service life issue.

Improving alternative design

Fig. 20 Dowels fastened with nuts

The improvement is to prevent the dowel from moving out of position by using nuts and bolts instead of a simple dowel – the image to the left. Inserting a screw or nail next to the dowel to prevent it from moving has been shown to be insufficient. The solution has been to use a nut at the end of the dowel as shown in figure 19.

8. Corrosion on tendon bars in the deck

Problem description

Fig. 21 Corroded tendon bar on a 17-year-old bridge

Significant corrosion on tendon bars at the anchoring plates and to some extent inwards has recently been detected. The bars are situated in pre-drilled holes in the creosote-treated deck and originally they were sprayed with zinc. This bridge was only 17 years old.

The nut was placed where the iron piece is shown and the anchored plate was to the left of the iron piece.

It is not known how comprehensive this problem is, but it probably concerns bridges with a stress-laminated deck and may be without drip nose. This bridge has no drip nose. We suppose that a drip nose will improve the problem considerably. Depending on how extensive this problem is it will give the bridge owner an increase in maintenance costs.

Consequences for service life and maintenance action

At least one of the bars have serious attack. This will substantially reduce the service life of the tendon bar. However, it is not currently known how long this corrosion can remain before we have to change the tendon bars, since this depends on the stress and corrosion level of the bar. We have to take action before it breaks, since it could in some situation be dangerous for the environment if it breaks and falls from the construction. The maintenance action will be to replace the bar.

Improving the part or introducing an alternative design

There are several ways to improve this structural detail, consisting of three main alternatives: preventing the water from touching the nut, preventing the bar from corroding, e.g. by wrapping it, or using a non-corrosive tendon bar e.g. a fibreglass bar.

9. Discussion and conclusions

This paper shows some of the main findings from inspection of timber bridges. Some problems of a common and less serious nature have been omitted. In addition to this discussion, some discussion has been included when presenting the problems above.

In some cases the consequences of what we thought was a minor problem were more serious, or we failed to realize the seriousness of the problem. The more serious issues were that the water in some cases flows from one section to another or penetrated the end grain, and water lying on one horizontal member with a timber beam above it. The corrosion on the tendon bar seems to be very serious as well, however, for the time being we do not know how comprehensive this problem is.

Most of the problems were related to water that either followed a wrong path (bad workmanship) or was due to bad design. We found several places where humidity on the surface was increased substantially because of e.g. water situated on a shelf/beam or only dirt remaining close to a timber member. Detailing the water flow is very important. For other problems, such as the moving of dowels and corrosion on tendon bars, new rules need to be written. Another finding was that all part of the timber bridge should be inspectable with visual inspection.

However, with some maintenance work, all or the very most of the timber bridges in Norway should last for the anticipated time.

10. References

[1] Horn H., Monitoring five timber bridges in Norway – result 2012, Norsk Treteknisk Institutt, Oslo, Norway, 2013

Vibration response of long cable-stayed timber footbridge – case study

Robert Kliger
Chalmers Univ. of Technology
Div. of Structural Engineering
SE-412 98 Gothenburg
Sweden
Robert.kliger@chalmers.se

Ph.D. Professor of Steel and Timber Structures. His research focuses on the structural behavior of built-up wood-based panels with semi-rigid connections, wood properties, timber products, strengthening of structures, in-situ assessment of timber structures and timber bridges.

Tomas Svensson
COWI AB
Box 12076, SE- 402 41Gothenburg
Sweden
tosv@cowi.se

Technical Director and Project Manager at Major Bridges and Marine Structures.
Civil engineer with more than 30 years' experience of structural engineering within the fields of buildings, industry and bridges.

Hanna Jansson
COWI AB,
Box 12076, SE-402 41Gothenburg
Sweden
hajn@cowi.se

MSc in Civil Engineering, Consultant and Coordinator at Major Bridges.
Civil engineer specializing in structural engineering within the field of bridges.

Isak Svensson
COWI AB
Box 12076, SE-402 41Gothenburg
Sweden
issv@cowi.se

MSc in Civil Engineering, Consultant and Coordinator at Major Bridges.
Civil engineer specializing in structural engineering within the field of bridges.

Summary

Timber footbridges with a span larger than 30 meters are sensitive to vibrations and dynamic factors must therefore be taken into considerations in the design. A case study of a cable-stayed footbridge, *Älvsbacka Bridge,* with a free span of 130 meters, is presented. The dynamic design of the bridge is compared, based on the old Swedish code, BRO 2004 and the Eurocode 5. Simplified force models presented in BRO 2004 and the ISO 10137 international standard are compared. Vertical and lateral accelerations are measured on the bridge with accelerometers attached to the bridge deck. According to the simplified force model, the vertical acceleration limit was met for a group of twenty pedestrians, which was regarded as a reasonable design situation. From the measured accelerations, the damping factor was calculated as 1.2% of critical damping, which is twice the value used in design.

Keywords: pedestrian bridge, dynamic response, force models, damping

1. Introduction

The cable-stayed timber footbridge, *Älvsbacka Bridge,* was completed during the summer of 2011 in the center of the City of Skellefteå in the north of Sweden. The bridge has a free span of 130

meters, making it the largest cable-stayed timber bridge in Scandinavia to date. The client was the Municipality of Skellefteå, the contractor was Martinsons Träbroar AB and the structural engineering consultant was COWI AB. *Älvsbacka Bridge* was designed according to the Swedish Road Administration Bridge Code BRO 2004 [1], but since 2009 the Eurocode 5 [2] has been applied in all designs. The effects on the dynamic design of the transition between two codes, i.e. Eurocode to BRO 2004, are studied and the findings are related to useful experience for future projects.

Timber footbridges with a span greater than 30 meters are sensitive to vibrations and dynamic considerations are therefore very important in the design [3]. One source of vibrations is pedestrian traffic and a special loading situation is caused by a group of pedestrians walking at the same tempo on the bridge. This loading situation was one of the greatest challenges when designing *Älvsbacka Bridge,* together with predicting how the dynamic behavior of the modeled bridge would correlate to the real response of the bridge. The general design rules in Eurocode 5 when it comes to vibrations in timber footbridges state that a footbridge should be designed in such a way that the loads on the bridge do not result in uncomfortable vibrations for the users [2]. According to the part which applies to timber bridges, Eurocode 5, Part 2, [4] the vibration serviceability criteria include a limit for the first resonance frequency induced by pedestrians and a limit for an acceptable level of vertical acceleration. The acceleration limit is strongly dependent on the damping ratio of the structure. During the design process, the damping ratio from BRO 2004 was twice the value proposed in the Eurocode [4] and, as a result, the Eurocode value was used, resulting in lower accelerations.
The aim of this paper is to report whether the assumption of the damping ratio from the Eurocode was more reasonable than that from BRO 2004 and, using measurements in situ, to ascertain whether the acceleration limits were met. The results presented in this paper focus on the vertical accelerations, as the maximum measured lateral acceleration was 0.14 m/s^2 and this value complied with all the design requirements.

2. Guidelines and codes – force models and comfort criteria

There are various guidelines to reduce the risk of pedestrians experiencing discomfort or feeling unsafe due to vibrations when crossing a footbridge. The design guidelines recommended by various countries set limits based on so-called comfort criteria and assume that within these limits most people do not feel any disturbance as a result of the vibrations in the bridge. The most common factors that are set as limit values are either the natural frequency or the acceleration. The acceleration is usually expressed in terms of maximum permitted acceleration in the bridge deck. There are many guidelines and codes in which various force models and the relevant comfort criteria are proposed. However, in this paper, only the relevant two codes are discussed and compared with regulations proposed by the international ISO 10137 standard [5].

BRO 2004 was published by the Swedish Road Administration and, until 2009, it was the Swedish standard for bridge design. This code was replaced by the Eurocode, but, for projects procured before 2009, it is still allowed to use BRO 2004 as the design code. The structural design of *Älvsbacka Bridge* was procured before 2009 and, as result, BRO 2004 was the valid design code for this project. The force model from BRO 2004 for pedestrian loading on footbridges is a harmonic vertical concentrated force which should be placed at the most critical point of the bridge deck. When designing according to BRO 2004, a damping factor of 0.6% of critical damping should be used. The first vertical natural frequency of the bridge deck should be higher than 3.5 Hz. If the first natural frequency is below 3.5 Hz, a maximum limit value for the vertical acceleration of the bridge deck is 0.7 m/s^2. No requirements or recommendations for lateral vibrations were given in BRO 2004.

In Eurocode 5, a simplified method for calculating vertical vibrations in timber bridges is given. The method applies to simply supported or truss bridges but not to other bridge types. Instead, it is stated that other methods may be specified in the national annex or for each individual project. In Eurocode 5, the damping factor for structures with mechanical joints is set at 1.5%, which can be used if no other value is stated in the national annex [2]. A check of the maximum peak acceleration in all directions must be made if the first natural frequency of the bridge deck is less than 5 Hz for vertical vibrations or less than 2.5 Hz for lateral and torsional vibrations. The maximum permissible accelerations according to Eurocode are 0.7, 02 and 0.4 m/s^2, depending on the direction of acceleration and load; i.e. for vertical acceleration, lateral acceleration for normal use and exceptional crowd conditions respectively. These accelerations should be met by a considerable margin due to large-scale uncertainty in the calculation of the response. The resulting vertical and horizontal acceleration is calculated differently depending on whether it is caused by one person or a group of people crossing the bridge and depending on the natural frequency, mass of the bridge and damping ratio. In the Swedish national annex [6], no force model for the pedestrian force is given. Instead, it is the client who is expected to choose the appropriate force model for the dynamic force, in the same way as for the comfort criteria.

In the international ISO 10137 standard, the force from a pedestrian is described by the static weight of the pedestrian and the corresponding periodic dynamic contribution. Both the vertical and lateral components of the dynamic pedestrian force are represented with the same equation [5]. The difference between vertical and lateral direction is made using the numerical coefficient. There are different values for the two coefficients for one moving pedestrian. The action from a group of people crossing the bridge is expressed by multiplying a coordination factor by the total pedestrian force. The coordination factor aims to describe the number of people in the group that tend to walk at the same tempo. The vibrations of footbridges are based on different design situations that occur during the design life of the bridge.

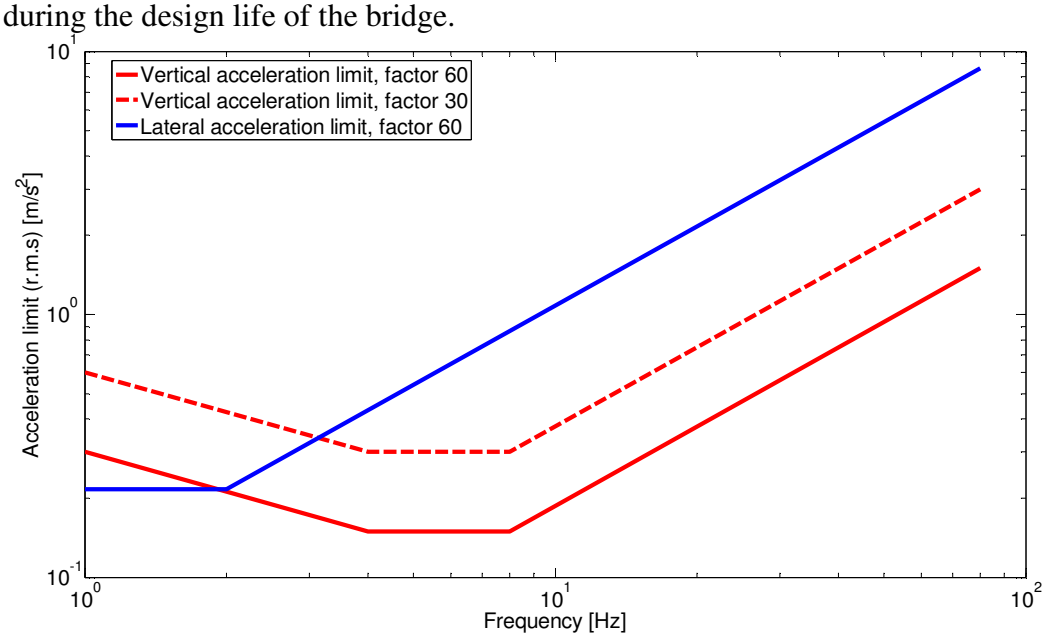


Fig.1. Acceleration limits for vertical and lateral accelerations with corresponding multiplication factors for footbridges according to ISO 10137. Factor 30 applies to the first design situation when one person is standing still on the bridge and factor 60 to other design situations.

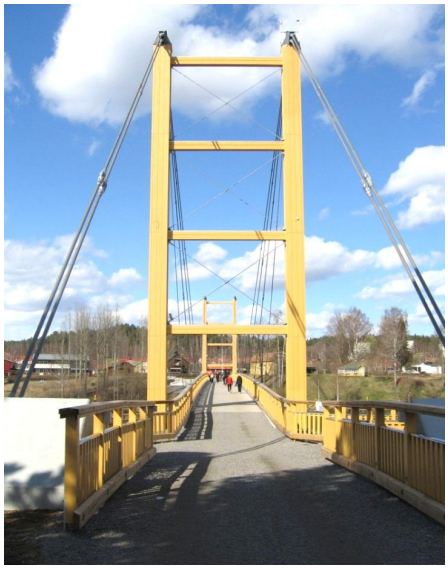
The second design situation "A group of eight to fifteen pedestrians crossing the bridge. The number of people depends on the size of the bridge deck" is designed to symbolize the regular use of the bridge. The comfort criteria given in ISO 10137 state a maximum limit for the acceleration in

the vertical and lateral direction respectively. The limits for each direction depend on a specific base curve multiplied by a certain factor. The base curves for vertical and lateral accelerations are shown in Fig. 1. [5].

3. *Älvsbacka Bridge*

Älvsbacka Bridge is a symmetrical cable-stayed bridge with one span of 130 meters and two pylons, shown in Fig2. The bridge deck is slightly curved where the mid-point is about one meter above the end points. The height of the pylons is 23 meters and one pylon consists of two glulam columns with four glulam cross-beams and steel cross-bracing to stabilize the pylon. The main beam consists of three glulam beams glued together, creating one beam with the dimensions of 645 x 1,125 mm. The main beams, see Fig. 2C, are supported by steel cross-beams every 16.25 meters to which the cables are connected at each side. Glulam cross-beams connect the two main beams with a distance of 2.5 meters. On top of the glulam cross-beams, purlins are placed to carry the deck of wooden planks. To stabilize the bridge deck, steel cross-bracing is used. The main beams of the deck and the cross-beams and columns of the pylons are covered with yellow painted panels for weather protection.

A)



B)

C)

Fig. 2 A) Overview of the bridge, B) The south pylon of Älvsbacka Bridge with its anchorage cables and concrete foundations, C) The main beams are supported by steel cross-beams to which the cables are attached.

4. Design FE models of the bridge

The dynamic design of *Älvsbacka Bridge* was carried out by COWI in Gothenburg in cooperation with the Division for Aerodynamics from COWI Denmark. The finite element software called NEiNastran with the pre- and postprocessor Femap was used for the dynamic analyses of *Älvsbacka Bridge*. This original design analysis included both the impact of wind and the corresponding aerodynamic instability and the dynamic effects of pedestrian traffic. In this case study, the FE model was made using Brigade/Plus software to analyze the pedestrian-induced vibrations using linear analysis and referred to as a simple model. Input data in both the original and simple models were taken from BRO 2004, see Table 1.

Once the simple linear model was verified against the original design by COWI, the same model was run once more with new input data provided by the Eurocode. The difference in terms of results between the original COWI design model and the simple linear model was very small, i.e. the deflection due to the uniformly distributed pedestrian load of 4 kN/m^2 was 166 mm and 164 mm respectively and the first vertical natural frequency was 1.448 Hz and 1.441 Hz respectively.

Table 1 Input data according to BRO 2004 and Eurocode 5.

		Young's modulus [GPa]	Poisson's ratio	Density [kg/m^3]	Shear modulus [GPa]
BRO 2004	Deck	4.846	-	454	0.323
	Glulam	13	0.4	600	4.643
EC 5	Deck	5.923	-	259	0.372
	Glulam	13.7	0.4	430	4.893

5. Acceleration measurements at *Älvsbacka Bridge*

To verify the dynamic performance of *Älvsbacka Bridge,* three different tests were conducted on the bridge. The tests were controlled walking, controlled jumping and heel impact. Four ICP accelerometers of model number 393B12 were used, together with a portable measuring device. Two of these accelerometers were mounted at the center of the bridge span and two others at a quarter of the bridge span, see Fig. 3. At each position, one accelerometer measures the accelerations in the vertical direction and the other measures them in the lateral direction. Frequencies for the controlled walking and jumping tests were chosen according to theoretical natural frequencies calculated using the simple linear model. The natural frequencies calculated with the two input data, see Table 1, differ slightly from one another. This was the reason why two similar frequencies were chosen to excite a specific mode in the controlled walking and jumping tests.

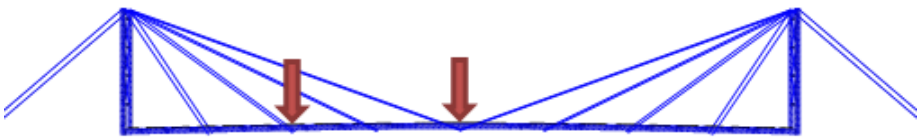


Fig. 3 Structural model of Älvsbacka Bridge

In the controlled walking test, a group of eight people walked over the bridge with the same pacing rate. The natural frequency of the lateral mode is half the walking frequency, due to the fact that only each alternate step gives rise to a lateral force in each direction.

The jumping test aims to simulate the performance of an electrodynamic shaker. The group of eight people jumped at a certain frequency to create a periodic load that acted on one point of the bridge. To excite the first vertical mode, the jumping group was positioned in the center of the bridge and, to excite the second vertical mode, the group jumped at a quarter of the bridge span. To excite the lateral mode, ice skating jumps were performed at the center of the bridge. The frequency for this type of jumps is twice the expected natural frequency since only every second jump gives rise to a lateral force in each direction. The expected natural frequency was around 0.7 Hz and, as a result, the jumping frequency was chosen as 1.4 Hz.

The heel impact test was performed to induce a simple impact load on the bridge. The heel impact test was performed on the bridge in the midspan and at a quarter of the span. The same group stood

on their toes close together and fell back on their heels at the same time. This procedure for both the jumping and impact tests was repeated twice with a 30-second interval during which all the test participants stood still. The purpose of the tests was to estimate a damping factor.

6. Results

6.1 Force amplitudes for dynamic pedestrian forces

The force amplitudes for the different force models are presented in Table 2. The static weight of a pedestrian is included in the force amplitude in the ISO 10137 model, which in this study is assumed to be 80 kg. The sum of the first three α_n-factors that are used in the simplified ISO 10137 force model is 0.3.

No force model related to lateral vibrations was given in BRO 2004. Instead, 10% of the vertical force is assumed to act as a lateral force in the calculations. The different loads are all harmonic periodic forces and the resulting accelerations were calculated over a frequency sweep from 0 to 5 Hz. One of the recommended design situations in ISO 10137 is a stream of pedestrians significantly larger than fifteen pedestrians. In the simplified force model, a group of fifty pedestrians was chosen.

Table 2 Force amplitudes for the dynamic pedestrian forces applied in BRO 2004 and ISO 10137. These values are used to calculate accelerations in the bridge deck.

Code	Vertical force amplitude [N]	Lateral force amplitude [N]
BRO 2004	1082	108
ISO 10137, 1 runner	1520	160
ISO 10137, 1 pedestrian	240	80
ISO 10137, 10 pedestrians	760	250
ISO 10137, 50 pedestrians	1700	565

6.2 Performed measurements at the bridge

6.2.1 Controlled walking test

Measured vertical accelerations from one of the controlled walking tests are shown in Fig. 4. The measured accelerations never exceeded 0.5 m/s^2. The data are measured at a quarter of the span with eight people walking on the bridge with a walking frequency of 1.9 Hz. The accelerations were the highest when the group passed a quarter of the bridge span, but, when the group passed the midspan, the accelerations were smaller. One of the intentions of the controlled walking tests was to excite the bridge in the lateral direction. The maximum lateral acceleration measured in the midspan from a test with 1.5 Hz as the walking frequency was 0.037 m/s^2. On both test occasions, the cables of the bridge started to sway from the applied forces in the controlled walking tests, but the oscillations decayed as soon as the group had left the bridge.

6.2.2 Jumping test

The largest measured vertical acceleration from the jumping tests was 1.66 m/s^2, as shown in Fig. 5. After 30 seconds, the group stopped jumping and stood still as the accelerations decreased. The results from the jumping tests were used to calculate a damping factor of the bridge. One of the fitted curves using MATLAB with decaying accelerations is shown in Fig. 6. Taking the results from all the jumping tests into account, the approximate damping factor from the jumping tests is

estimated to be 0.6%. Even during this test, the cables started to oscillate and these oscillations continued for about one minute after the group had stopped jumping.

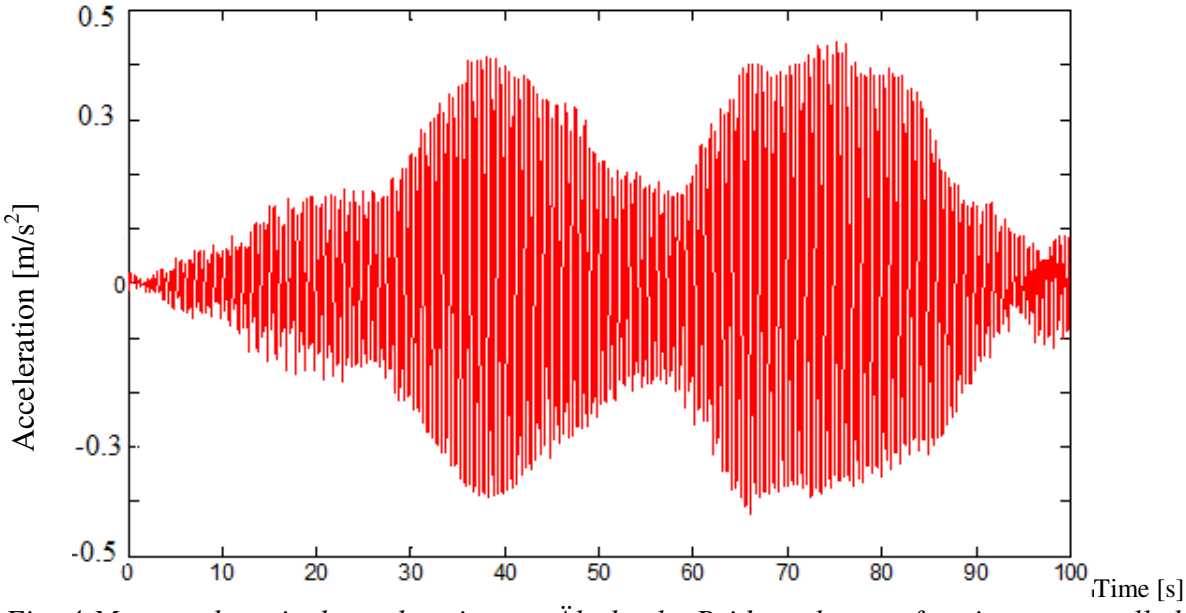


Fig. 4 Measured vertical accelerations at Älvsbacka Bridge when performing a controlled walking test with a walking frequency of 1.9 Hz.

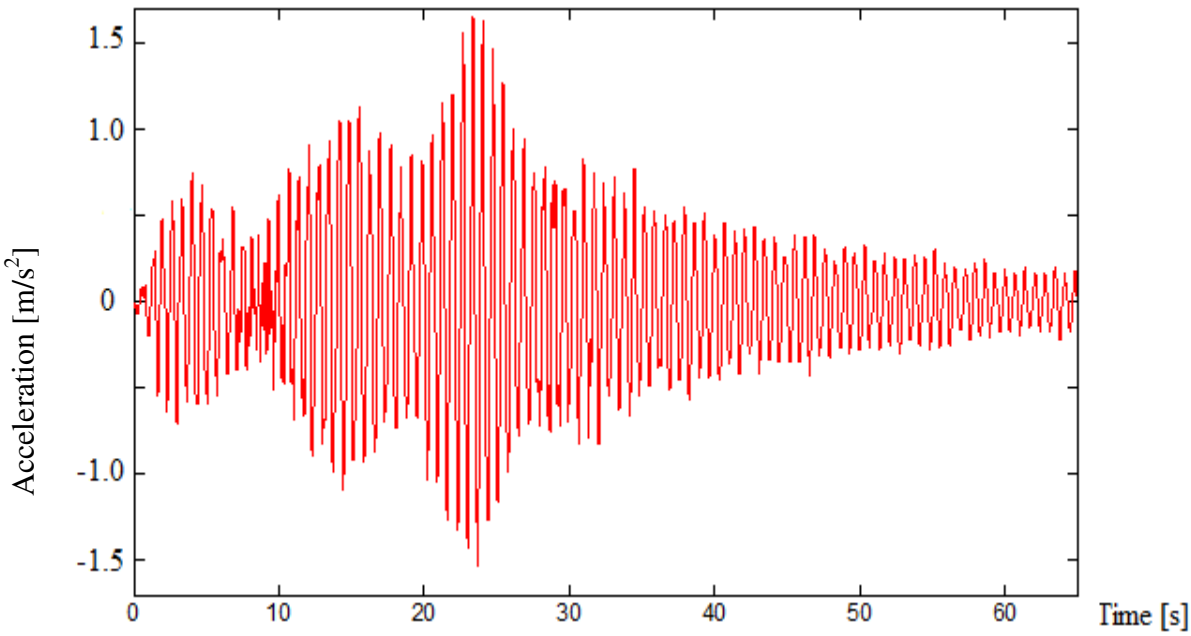


Fig. 5 Measured vertical accelerations on Älvsbacka Bridge at midspan when performing a controlled jumping test with a jumping frequency of 1.4 Hz.

6.2.3 Heel impact test

Two types of test were performed: one in which the load was applied at midspan and the accelerations were measured at a quarter of the span and one in which the load was applied in a quarter of the span and the resulting accelerations were measured in the center of the bridge. The maximum vertical acceleration, measured both in midspan and in a quarter of the span, was

approximately 0.1 m/s^2. From all the heel impact tests, the damping factor is calculated at approximately 1.2%, which is twice the damping calculated from the jumping tests.

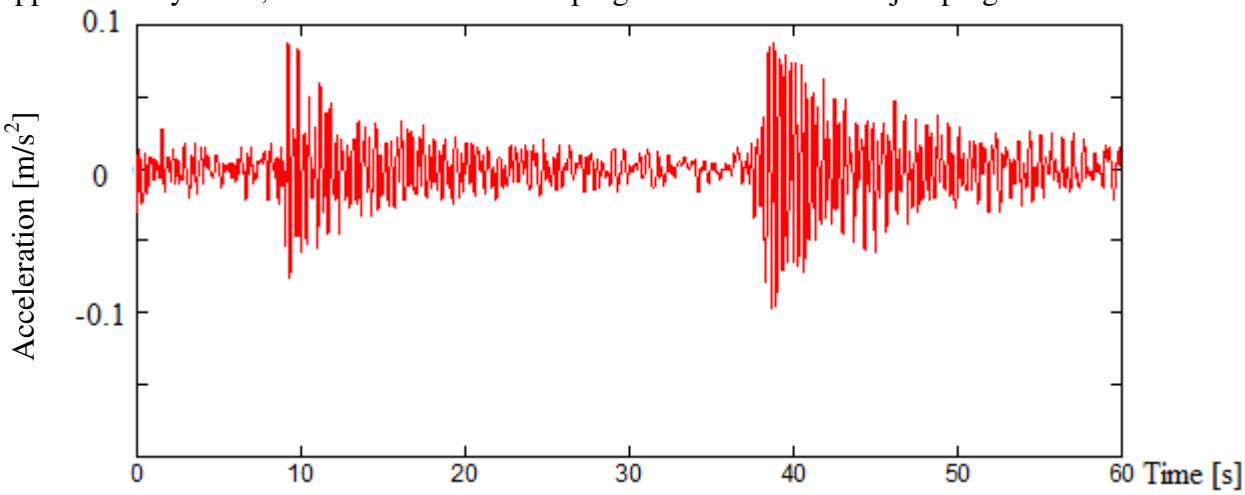
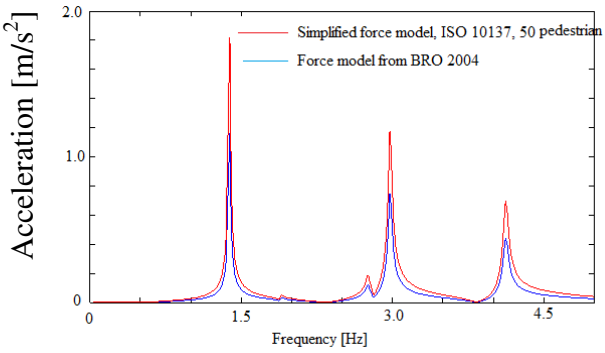


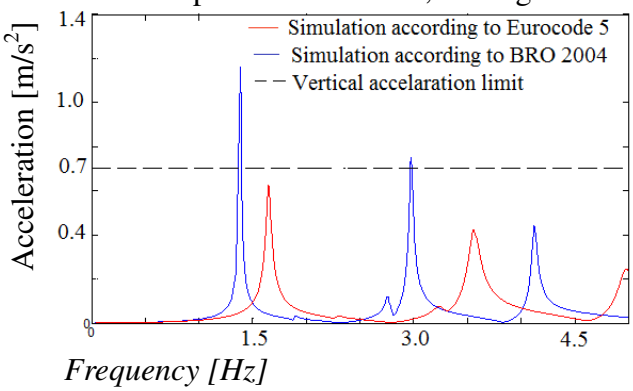
Fig. 6 Measured vertical accelerations at a quarter of the span as a result of one heel impact test at Älvsbacka Bridge when applying the load at midspan.

6.3 Comparison between the force model in BRO 2004 and ISO 10137 and simulation of the response

Since no force model is given in Eurocode for this type of bridge, the comparisons were made between the force model in BRO 2004 and the simplified force model from ISO 10137. The accelerations measured at *Älvsbacka Bridge* were compared with the results of the simulations of the tests using the simple model in Brigade/Plus. The worst-case scenario was pedestrian streams significantly larger than fifteen people, which explains why in this comparison a group of fifty pedestrians was chosen for the calculations. The force amplitude from the simplified ISO 10137 model and BRO 2004 was 1,700 N and 1,082 N respectively, cf. Table 2. As a result, higher acceleration was obtained by applying the force model from simplified ISO 10137, see Fig. 7A.



A)Response due to the force models in BRO 2004 and ISO 10137 for a group of fifty pedestrians



B)Response due to the force models in ISO 10137 for a group of twenty pedestrians and comparison to the acceleration limit

Fig. 7 Vertical acceleration response

In order to compare the effect of the data presented in BRO 2004 and Eurocode 5, the same force was applied in the simple model, i.e. the force model from BRO 2004 with a force magnitude of 1,082 N. The results are shown in Fig. 7B, where data according to BRO 2004 result in higher vertical accelerations and lower natural frequencies than the same model according to the data from Eurocode 5. The reason for this is the difference in the material parameters. The density of timber

and glulam given in BRO 2004 is probably too high, but it is assumed to include non-structural mass, while the values given in Eurocode 5 are too low and all the extra mass has to be considered separately, cf. Table 1. The force model given in BRO 2004 only depends on the size of the bridge and cannot be varied. The force model in ISO 10137 depends on the number of pedestrians walking on the bridge at the same time. To comply with the vertical acceleration limit, twenty pedestrians can be used in the simplified ISO 10137 force model, which is a more relevant number of people walking on *Älvsbacka Bridge*. By applying this force model, the simulation using data from the Eurocode 5 meets the requirement for vertical accelerations of 0.7 m/s^2 which applies in both BRO2004 and Eurocode 5, see Fig. 7B.

6.4 Simulations of the tests in Brigade/Plus

The measured and simulated acceleration shows fairly good agreement, especially when the energy of the system is dissipating. As an example, the simulated and measured accelerations from the heel impact test are shown in Fig. 9. The comparison is shown for the first ten seconds after one impact. From the measured values from the heel impact tests, the damping factor was calculated to be 1.2%, which is between the value of 0.6% given in BRO 2004 and the value of 1.5% given in Eurocode 5. The measured accelerations were also transformed into the frequency domain with Fast Fourier Transformation, FFT, to obtain the natural frequencies from different tests, see Table 3. In the simulation of the heel impact test, a damping factor of 1.2% was used, the same as that calculated from the measured heel impact test.

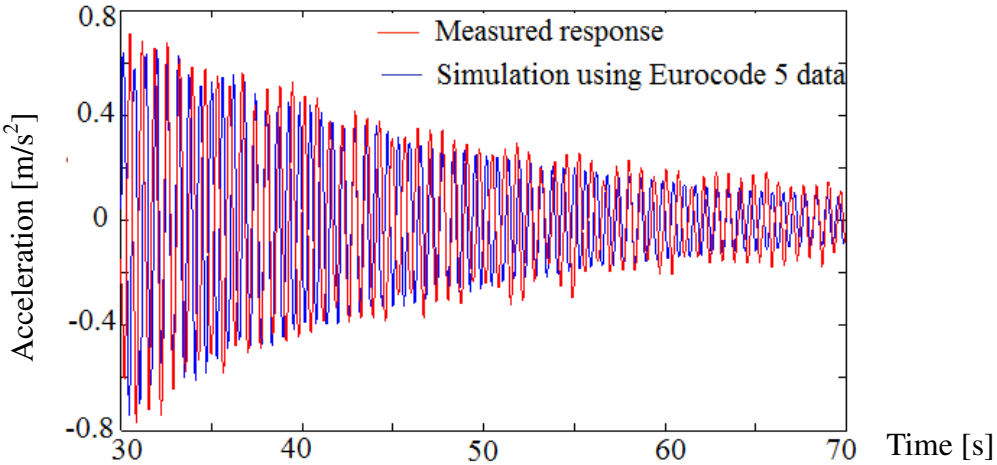


Fig. 9 Comparison of measured response of Älvsbacka Bridge and simulations using the simple model according to Eurocode after the group has stopped jumping.

The first vertical natural frequency obtained from the FFT curve from one of the jumping tests was 1.45 Hz. The second natural frequency was 2.01 Hz, which was close to the theoretical second vertical natural frequency. The curve was based on accelerations measured in a quarter of the span from a controlled walking test with a walking frequency of 1.9 Hz. These measured frequencies were compared with the calculated frequencies, see Table 3.

Table 3 Comparison of the natural frequencies measured on Älvsbacka Bridge with frequencies calculated using the simple model and data according to BRO 2004 and Eurocode 5.

	Measured frequencies [Hz]	Calculated values according to BRO 2004 [Hz]	Calculated values according to Eurocode [Hz]
First vertical	1.45	1.39	1.65
Second vertical	2.01	1.90	2.29

7. Discussion and conclusions

One of the purposes of measuring accelerations at *Älvsbacka Bridge* was to investigate whether the measured accelerations exceed the limits or if they comply with the design limits. The measurements showed that the vertical acceleration limit of 0.7 m/s^2 in Eurocode 0 was exceeded during the jumping test, where the maximum measured vertical acceleration is 1.6 m/s^2. This value might seem high, but the jumping tests are considered to be extreme loading situations with large forces pulsating with the frequency close to the natural frequency of the bridge. The results from the controlled walking tests, which were considered to symbolize a possible loading situation, never exceeded 0.5 m/s^2 and the vertical accelerations from the continuous measurements never reached more than 0.2 m/s^2. These values show that *Älvsbacka Bridge* fulfils the design requirements relating to vertical vibrations presented in both Eurocode 0 and BRO 2004. From the measured vertical accelerations at *Älvsbacka Bridge,* two damping factors were estimated based on the jumping tests and the heel impact tests. The jumping tests resulted in a damping factor of 0.6% and the heel impact tests in a damping factor of 1.2%. One possible explanation of the different damping factors is the amount of energy imposed on the bridge by the two tests. The jumping tests introduced a larger amount of energy into the bridge than the heel impact test. The large amount of energy from the jumping tests was absorbed by the cables which began to sway and the energy was kept in the bridge during a longer period of time which explains the lower damping factor. The results of the simulations showed that the damping factor had the greatest influence on the accelerations.

According to Eurocode 5, a damping factor of 1.5% can be used for structures with mechanical joints. This value is closer to the measured value from the heel impact test. The main differences between the codes that affected the natural frequencies were the densities of the timber and the glulam.

8. Acknowledgements

This paper is based on the work done as a master's thesis by two of the co-authors, i.e. Hanna Jansson and Isak Svensson [7], and supervised by Tomas Svensson and Robert Kliger. The authors are grateful to Martinsons Träbroar AB, who organized the testing of the bridge, and COWI AB, who supported and sponsored this work.

9. References

[1] *VV Publ 2004:56 BRO 2004 del 1 - Terms and conditions (in Swedish)*, Vägverket, Borlänge.

[2] *SS-EN 1995-2:2004 - Eurocode 5: Design of timber structures - Part2: Bridges*, 1st ed., Swedish Standards Institute, Stockholm.

[3] Pousette, A. "Cable stayed timber bridges" Stockholm : Trätek, 2001. Serie, Rapport / Trätek, 0112042 .

[4] *SS-EN 1995-2:2004 - Eurocode 5: Design of timber structures - Part2: Bridges*, 1st ed., Swedish Standards Institute, Stockholm

[5] *ISO 10137:2007 Bases for design of structures – Serviceability of buildings and walkways against vibration,* 1st ed., International Organization for Standardization, Geneva.

[6] *TRVFS 2011:12 Transport Administration Regulations amending the Road Adminstration regulations (VVFS 2004:43) on the application of European calculation standards (In Swedish:Trafikverketsföreskrifter om ändring i Vägverkets föreskrifter (VVFS 2004:43) om tillämpningen av europeiska beräkningsstandarder),* Trafikverket, Borlänge.

[7] Jansson, H., Svensson, I. "Vibrations in timber bridges due to pedestrian induced forces – A case study of Älvsbackabron" Master's Thesis 2012:96. Chalmers University of Technology, Göteborg, Sweden 2012.

Wood Preservative Solutions for Creative

And

Sustainable Bridge Design and Construction

Ted LaDoux
Executive Director
Western Wood Preservers Institute (WWPI)
Vancouver, WA - USA
Ted@WWPInstitute.org

Education: 1971 Forest Technology Degree - Central Oregon College; 1978 Forest Engineering Institute Certificate – Oregon State University
Career: 1971 to current: USDA Forest Service; NW Independent Forest Manufacturers Association; Evergreen Partnership; WWPI; and BOD Impact Washington.

Summary

Wood is a remarkable fibrous material that has served the needs of mankind for centuries by being an essential source of fuel, shelter, chemicals, paper, and a host of other goods. Because of its many attributes and aesthetic benefits wood continues to be the preferred building material in a wide variety of construction applications providing a natural long life and when chemically preserved can be extended even longer. The use of wood for maintenance and construction of pedestrian bridges and highway and railway bridges over aquatic and wetland environments has long been the material of choice and when a timber bridge project is properly planned and the environmental conditions of a project site are properly evaluated the environmental risks can be minimized and managed.

Keywords: wood preservatives; decay; durability; aquatic environment; risk assessment models; best management practices; renewable resource; quality assurance

1. Introduction

Why should you use preservative wood? It is a known fact that wood degrades via non-living or living agents, or both at the same time when left unprotected from these agents resulting in a significantly shorter life expectancy. This will continue to be a significant concern when using wood and for this very concern preserved wood plays a key role in protecting wood from degradation while significantly extending the life expectancy.

Some important factors to keep in mind about wood in its natural state: *A number of non-living agents can cause degradation of wood such as: mechanical damage from handling or in-service use; weathering caused by exposure to ultraviolet light in sunlight that breaks down the lignin near the wood surface causing the wood to erode away by wind or water overtime; prolonged exposure to elevated levels of heat that break down individual wood polymers; exposure to chemicals such as strong acids and bases; and repeated wetting and drying in salt water causing surface damage to the wood as the cells absorb so much salt they literally burst.*

There are also a variety of living agents or biological agents that can degrade wood, but in all cases these organisms need four basic requirements: adequate moisture, oxygen, adequate temperature, and a food source. Some common wood destroying organisms include bacteria, fungi, insects, marine borers, and some vertebrates [1] - such as woodpeckers.

For proponents of timber bridge construction, the use of preserved wood provides an environmental and cost effective solution for protecting wood from the degradation of non-living and living agents that will significantly extend the durability and service life of a wood structure. Knowing the environmental benefits of using preserved wood is also an important factor to consider when selecting it as a construction material for a project. In a series of quantitative evaluations conducted by the organization Treated Wood Council of the environmental impacts associated with the national production, use, and disposition of various preserved wood products compared to alternative materials, they found that in 7 out of 8 key environmental impact indicators preserved wood required less total energy and less fossil fuel, had lower overall environmental impacts, and when reused for energy recovery in permitted facilities with appropriate emission controls there would be further reduction of greenhouse gas levels in the atmosphere. The evaluation was conducted using Life Cycle Assessment (LCA) methodologies and followed ISO 14044 standards, as well as being peer-reviewed. Summary information and links to the full LCA's can found by going to www.wwpinstitute.org.

Further, under the Federal Insecticide, Fungicide, and Rodenticide Act (FIFRA), wood preservatives containing a pesticide are fully regulated by the U.S. Environmental Protection Agency (EPA) as they are required to go through a very rigorous registration and re-registration process. The EPA considers wood preservative systems as antimicrobial pesticides and requires that the pesticides must be supported with thorough scientific review and analysis as well as show they can be used without causing undue adverse effects to human health or the environment. It is also important to be aware that under federal law preserved wood products are not considered to be a pesticide and therefore not regulated by FIFRA.

2. Five Steps to Appropriate Use of Preservative Wood

Because most types of timber bridges, such as pedestrian, auto or railway, cross some form of water body or drainage they pose varying degrees of environmental risk or required protection when constructed or maintained. For this reason it is important to have some guidance in understanding some of the science behind wood preservative systems and how to select and manage the use of preserved wood to ensure the desired performance required of a bridge project is achieved while minimizing the potential risk for any adverse environmental impacts. The process needs to begin at project conception and all the steps through installation and maintenance.

The following five basic steps are recommended when planning use of preserved wood in aquatic and wetland environments (fresh and salt water):

1. Selecting the Proper Preservative
2. Environmental Considerations and Evaluations
3. Specifying Best Management Practices
4. Requiring Quality Assurance and Certification

5. Following Basic Handling, Installation and Maintenance Guidelines

2.1 Selecting the Proper Preservative

To make sure the appropriate preserved wood product is selected it is important to fully understand how to identify and specify the appropriate wood preservative system based on the desired species and existing environment on a project site. Some good resources available that provide helpful information are the US Forest Products Lab Wood Handbook (FPL–GTR–190 – 2010) on wood preservation found at www.fpl.fs.fed.us/documnts /fpl_gtr190.pdf; Western Wood Preservers Institute guidance documents on use of preservatives in aquatic and wetland environments at www.wwpinstitute.org/aquatics.html#guidance and WWPI APP; and American Wood Protection Association (AWPA) book of standards – Use Category System Standard U1, Sections 3, 4 & 5 found at www.awpa.com or WWPI APP.

While the AWPA book of standards identifies around 27 different wood preservative systems, only seven are commonly used to preserve material designated for use in aquatic and wetland environments either in and/or over fresh and salt water. There are a few other preservative systems available, but they will not be addressed in this paper as they are not commonly used in the western regions of the United States. In addition, there are a few other proprietary formulations available that are often selected for aesthetic purposes, also not discussed.

The seven commonly available preservatives for use in aquatic and wetland environments can be broken down into two general categories – Waterborne and Oil-type systems:

2.1.1 Waterborne Preservative Systems

Waterborne systems are considered inorganic preservatives and are characterized by the fact water is the primary carrier of the preservative chemical. In these systems the chemicals are precipitated into the wood substrate and become attached to the wood cells, minimizing migration once the chemical is stabilized or fixed to the wood cells. In general all waterborne preservatives perform basically the same. They also leave a dry and paintable surface. The primary environmental concern with these preservatives is the potential environmental effect the loss of copper from each of these preservatives will have on the specific project environment when placed into service. For this reason it is critical to conduct a screening level assessment for each project site.

The four main waterborne systems or groups used in aquatic and wetland environments are:

- **Chromated Copper Arsenate (CCA)** – Since 2004 CCA has only been available for use in preserving commercial and/or industrial type wood products. While CCA preserved wood products are readily produced throughout the U.S., use near, in, or over bodies of water are largely discouraged or prohibited in many western states by permitting agencies, even though it has been demonstrated the environmental risks are minimal. This is primarily a result of perceived concerns about the toxicity of arsenic in this preservative. In addition, because coastal Douglas fir is usually the preferred wood species for many commercial and industrial applications CCA is not recommend for treating this species and other hard to treat species. The opposite is true for many other parts of the U.S., like in the southeastern states, where the preferred species is the easier to treat southern yellow pine and where CCA preserved products are considered environmentally friendly. Also, CCA is the only preservative system that has testing

technology (Chromotropic Acid Test) that can determine whether fixation of the preservative has been achieved in the wood cells.

- **Ammoniacal Copper Zinc Chromate (ACZA)** – Under the trade name Chemonite®, ACZA is an ideal preservative to use for hard to treat species like the popularly used coastal Douglas fir. Because of this quality and its environmental record ACZA is normally the preservative of choice to treat coastal Douglas-fir and other western species products for such uses as piling, bulkhead and bracing that will be immersed or come into contact with fresh or salt water. Although in some local areas in the west you may find restricted use because of the perceived environmental risk associated with short term migration of the preservative from the wood or requirements for additional mitigating measures, such as wrapping or coating, to help stop or minimize the loss of preservative from the product. ACZA is also commonly used in a variety of other above water applications.

- **Alkaline Copper Quat (ACQ) and Copper Azole (CA) -** These preservatives are widely used throughout the U.S. in a variety of residential, commercial and certain agricultural applications and often thought of as "general use" preservatives. Both ACQ and CA preserved wood products perform basically the same with some minor product application differences. They are both commonly used to preserve lumber and timbers used above and in fresh water and subject to some brackish or saltwater splash. The exception is that ACQ preserved round and sawn wood piling can also be used for land and in freshwater applications. As with ACZA, there is an environmental concern by some permitting agencies over the perceived environmental effects the loss of copper from these preservatives may have on the specific project environments. However, with some exceptions, products preserved with ACQ and CA are generally viewed favorably for general use in or above freshwater or near saltwater applications.

2.1.2 Oil type Preservative Systems

Oil type preservatives are organic preservatives characterized by the fact they are 100 percent active (Creosote) or dissolved in an oil-based solvent. These mixtures fill or coat the wood cells walls during treatment.

<u>The three primary oil-type systems used in aquatic and wetland environments are:</u>

- **Creosote -** This is a coal tar-based wood preservative and when used as a preservative it can only be manufactured by the distillation of tar obtained from coal. It typically has some odor and is not paintable. Primary use is the treatment of industrial products such as railway ties, utility poles and cross-arms, piling and timbers for bridges and other transportation structures. Creosote preserved wood can be used in a variety of applications requiring in-ground contact, or in and/or over fresh and salt water. Creosote has a long history of being a very effective preservative and it is not uncommon to find marine piling and bridge structures today ranging in age from 50 - 90 years old still in good serviceable condition. Acceptance and use of creosote preserved material varies by region. For example, in Alaska and the southeastern states it is widely used for preserving a variety product, such as marine piling, dock structures, bulkheads, utility poles, and bridges.

Creosote is extensively used to treat railway ties used by our nation's railroads, which represents approximately 95% of the creosote use today. In most of the western states, other than the railways, use is typically restricted to replacement of existing structures for maintenance purposes, while in the states of New York and New Jersey all aquatic uses are prohibited. Creosote is not recommended for use in residential, industrial or commercial interiors except for laminated beams or building components that are in ground contact and where there may be frequent or prolonged contact with bare skin.

- **Pentachlorophenol (PCP)** – This preservative in a solid state is dissolved in petroleum oil either in diesel, or fuel oil grades and light hydrocarbon solvents. PCP is diluted to approximately 5% to 10% in oil in order to be used in a preservative solution. Use of PCP is popular for preserving wood utility poles and cross-arms, and solid wood and laminated timbers that are used in construction of buildings and bridges. PCP preserved material in aquatic applications are restricted to above water structures in saltwater and in or above freshwater. Like creosote, PCP is not recommended for use in residential, industrial, or commercial interiors except for laminated beams or building components that are in ground contact and where there may be exposure to frequent or prolonged contact with bare skin. PCP in light hydrocarbon solvents leaves a more natural appearance, and may be specified where stain finish is desired.

- **Copper Napthenate (CuN)** – This preservative is different than the other copper-based preservatives in that the copper is reacted with naphthenic acid, a hydrocarbon by-product of crude oil processing. The CuN concentrate is diluted with fuel oil at treating plants to make the preservative solution. Unlike the other oil-type preservatives CuN is not a restricted pesticide. When CuN is applied it is initially a light green color that diminishes over time due to weathering and often has an odor. There are odor neutralizers available that can be applied should odor be an issue. After thorough drying CuN preserved wood can be painted or stained, but a stain-blocking primer or second topcoat is recommended for finishing minimizing the CuN treatment's discoloration of the finish. CuN is used to preserve a variety of products for industrial projects such as foot and auto bridges, as well as fence rails and posts, guardrail posts, railroad ties, utility poles, piling and outdoor recreational structures. Other than being restricted from use in brackish or salt water applications CuN can be used to preserve a variety product materials for use near saltwater or in and above ground for freshwater applications.

In addition to the above referenced informational resources, many other factors will likely come into play when selecting the appropriate preservative system. Managers will likely weigh the economics, type of project, availability of wood species, aesthetics, environmental concerns, and the permitting or approval process itself. These decisions will be influenced in part or whole by the permitting authority, existing laws, personal preference, organizational policy, professional knowledge, and environmental conditions.

2.2 Environmental Considerations and Evaluation

In designing a project the characteristics of various preserved wood products should be taken into consideration in relation to the purpose of the project and the environmental conditions at the project site. Products used in a heavy industrial application, like a bridge used for motor vehicles,

will likely be different from those used in a public structure, such as a foot bridge or boardwalk. Similarly, the use of a moderate amount of preserved wood in a fast flowing river or stream is likely to pose a minimal risk; whereas, the use of large amounts of preserved wood in somewhat stagnant water may pose greater risks.

Nearly any material used in aquatic environments will introduce some degree of chemical and have an environmental effect if present in large enough concentrations. When specifically using the previously described wood preservatives, a certain amount of preservative will migrate from all these preservatives, but typically for only a short period of time, and enter the water column or sediment adjacent to the project area. For this reason it is important to be able to evaluate the level of potential risk on a site specific basis to properly manage the risks. There are project situations where the use of preserved wood may be of significant environmental concern such as previously contaminated waters or very slow moving waters with no natural flushing. However, based on scientific studies and field results 95% of projects being constructed today in some type of aquatic environment should not be significantly impacted from use of preserved wood when the risks are identified and managed.

To help biologists and project proponents of preservative wood peer-reviewed risk assessment models recognized by NOAA-Fisheries as being useful in evaluating the potential environmental effects due to the conservative assumptions used in developing them are readily available to assist in determining the potential risks associated with a proposed project. A detailed discussion of the models and supporting information preservative wood can be used safely in aquatic environments when the risks are evaluated can be found in NOAA-Fisheries 2009 guide for treated wood titled - The *Use of Treated Wood Products in Aquatic Environments: Guidelines to West Coast NOAA Fisheries staff for Endangered Species Act and Essential Fish Habitat Consultations in the Alaska, Northwest and Southwest Regions*.

In addition to the risk assessment models, a companion Level One Screening Assessment tool based on the science used to develop the more robust risk assessment models has also been created to further assist in making evaluations of the environmental risks. This simplified assessment tool utilizing tables and some basic project site conditions was designed to easily make some preliminary predictions on whether a more extensive risk assessment should be undertaken, or support a conclusion there would be no significant environmental effect from using preserved wood on a project.

The risk assessment models are based on research knowledge of preservative loss rates from properly preserved wood and when coupled with site-specific project environmental data, such as water current speeds, and background levels of metals and organics in the sediment it allows users the ability to predict the environmental response to any project design when preservative wood is used in and/or over an aquatic environment, including the use of multiple wood preservatives. For those interested in a detailed discussion on the science and model assumptions used they can be found in a book published by the Forest Products Society in 2011 titled *Managing Treated Wood in Aquatic Environments.* The NOAA-Fisheries guide, risk assessment models and screening level assessment tool can all be found at www.wwpinstitute.org under the aquatic section.

Other than legacy facility sites preserved wood has a long history of safe use in aquatic environments with no published report describing any significant loss of biological integrity associated with its proper use when, again, the risks are first evaluated and the proper preservative is selected.

2.3 Specifying the Best Management Practices

Another key element available for managing risk whenever preservative wood products are planned for an aquatic environment is the specification of the Western Wood Preservers Institute's (WWPI) *Best Management Practices for Use of Preservative Wood in Aquatic and Wetland Environments (BMPs).*

The BMPs are additional wood preserving guidelines for all individual or groups of preservative systems used to preserve wood designated for use in aquatic or wetland environments. The established guidelines are intended to further minimize the amount of potential chemical migration or movement from preserved wood material during the wood preserving process.

Specification of the BMPs gives specifiers another valuable environmental protection tool to use to assure preserved material used on a project site has been preserved with the minimal level of preservative needed for protection that meets AWPA standards while reducing the amount potentially available for migration or movement in to the environment.

Along with the additional processing requirements, BMPs are separate from and in addition to the AWPA standards. There is a shared responsibility between the Specifier and Treater to assure the level of preservative system application selected will meet the goal of minimizing the migration of the preservative into the environment.

The full details on what and how BMPs work can be found by going to www.wwpinstitute.org under the aquatic section.

2.4 Providing Quality Assurance and Certification

One of the benefits of specifying wood material be preserved to the BMPs is that it will not only assure the material meets AWPA standards, but also that it meets the BMP guidelines as third-party independent inspection procedures and certification are in place to provide the needed quality assurance required by the BMPs.

CheckMark Symbol

To assure products meet the AWPA standards, it is important that the presence of a quality checkmark be present on all structural product labels or a letter of certification should labeling not be present. The presence of the **CheckMark** logo is a quick and simple way of identifying whether the product material purchased has been inspected by an approved American Lumber Standard Committee (ALSC) third-party inspection agency authorized to assure compliance with AWPA standards.

BMP Certification Mark

Additionally, to assure material has been preserved in accordance with the BMP guidelines certification should also be verified by an authorized ALSC third-party inspection agency by letter of certification or the presence of the WWPI BMP Certification Mark on the product or unit. Details on the quality assurance inspection procedures and requirements are incorporated as a separate chapter in the BMP document, which can be found under aquatics at www.wwpinstitute.org.

It is strongly recommended for the specifying agency and/or contractor and the selected supplier to review the project specifications and material requirements to assure the proper material will be produce to the desired standard and specification for the project, along with an understanding of the required quality assurance and certification. It is also advised, if practicable or customary for the wood preserving company to be directly contacted to discuss the required specifications, including the environmental concerns for the project. Past experience has shown that when a preservative product has not met the expectations of the purchaser it has typically been the result of a breakdown in communications.

2.5 Appropriate Handling, Installation and Maintenance

One of the most critical times in the life of a project using preserved wood, in terms of environmental impacts, is during and immediately following construction. While use of a U.S. EPA registered preservative treated to AWPA standards, along with specification of the BMPs will help assure minimal environmental impacts there are several other actions that can be taken to further ensure the project is constructed and maintained in an environmentally safe manner during installation or maintenance of the structure.

Some suggested additional actions are as follows:

- To degree possible framing, sawing, cutting and drilling should be specified to be done prior to preserving the wood.
- Products should be inspected when it arrives on project site.
- Use containment measures when working over water to catch and collect cutting, shavings and sawdust where necessary. Where practical conduct additional fabrication work away from water and provide for collection of waste.
- All field cuts and drill holes created on project site should be field treated. Available treatments include Copper Napthenate, Outlast Q8, and Hollow Hear CB.
- Removal of old preserved wood structures for maintenance purposes or demolition can be either be recycled for reuse, if suitable, or by federal and most all state laws can be disposed as non hazardous or exempt hazardous waste in approved landfills.
- Routine inspection and timely maintenance is critical to extending the service life of a preserved wood structure.

For further perspectives on using preserved wood in aquatic or wetland environments it is suggested you read *Guide for Minimizing the Effect of Preservative-Treated Wood on Sensitive Environments* published by the USDA Forest Service Products Laboratory.

3. Conclusion

For over a century preserved wood has played an essential role in the economic prosperity and quality of life in North America. From the rail way ties that carry our trains; to the poles that carry communications and power; to bridges that cross our rivers and valleys carrying vehicles and foot traffic; to industrial and commercial structures serving businesses and communities; and to scenic and recreational structures enjoyed by millions of visitors the use of preservative wood has been the preferred, time-proven, cost effective material of choice.

Since the awakening of environmental awareness by society in the second half of the twentieth century there have been numerous environmental laws adopted and the implementation of regulatory policies, in some cases unwritten polices, that restricted the use of some construction practices and materials in aquatic environments, like preserved wood. As result, this awakening also brought about greater scrutiny over the use of preserved wood products in aquatic and wetland environments. Because of this emerging concern the wood preserving industry undertook action to better understand the environmental effects of wood preservative systems on aquatic and wetland environments to better determine the proper applications and assure they could be environmentally safe to use. For the past two decades an effort has been in progress to conduct research and studies in partnership with various governmental agencies, universities and the wood preserving industry to better understand the environmental performance and potential effects of using preserved wood in aquatic and wetland environments.

All the information presented in this paper, and more found at www.wwpinstitute.org, represents the collective result of two decades of environmental research, case studies, and technical analysis conducted on the environmental performance of preserved wood in aquatic and wetland environments. It also represents the most authoritative and comprehensive science available critical to developing needed risk assessment tools for conservatively predicting environmental effects, as well as useful management tools to further minimize environmental risks associated with use of preserved wood in aquatic and wetland environments. Access to the science and management tools is readily available, as well as training, to assist biologists and managers better understand the science, evaluate the potential environmental effects, and manage the risks in order to make informed management decisions.

While there are no federal laws prohibiting use of preservative wood, and only a few states with limited restrictions, there is a general bias against the use of any type of preservative wood material among some regulatory agencies and individual biologist responsible for enforcing the provisions of the Magnuson-Stevens Act – Essential Fish Habitat and the Endangered Species Act due to perceived detrimental environmental effects. However, the vast majority of empirical science is contrary to this viewpoint and clearly supports the use of preserved wood in most situations. What is critical to know is that when the appropriate preservative system is selected, the potential environmental effects evaluated on a site specific basis, and the WWPI BMPs are specified the potential risks will be minimal and manageable for the environmentally safe use of preserved wood products in the majority of projects where use is proposed.

4. References

[1]. Morrell J, Brooks K, Davis C, *Managing Treated Wood in Aquatic Environments*, Forest Products Society 2011, pp. 2-3

Development of a Slab-on-Girder Wood-Concrete Composite Highway Bridge

Andrew Lehan, M.A.Sc., EIT
Structural Designer
Delcan
Markham, Canada
a.lehan@delcan.com

2009 B.A.Sc. (Hons.) Civ. Eng., Univ. of Waterloo

2012 M.A.Sc. Civ. Eng., Univ. of Toronto

2012- Structural Designer at Delcan

David Moses, PhD, PEng, PE, LEED®AP
Principal & Owner
Moses Structural Engineers
Toronto, Canada
dmoses@mosesstructures.com

1993 B.Sc. Civ. Eng., Queen's Univ.

1995 M.Sc. Civ. Eng., Queen's Univ.

2000 Ph.D. Civ. Eng., UBC

2009- Principal at Moses Structural Engineers

Summary

This paper examines the development of a slab-on-girder wood-concrete composite highway bridge superstructure. The proposed bridge makes use of a slender ultra-high performance fibre-reinforced concrete (UHPFRC) deck made partially-composite in longitudinal bending with glued-laminated wood girders. Longitudinal external unbonded post-tensioning is utilized to increase span capabilities. Prefabrication using double-T modules minimizes the need for cast-in-place concrete on-site. Durability is realized through the highly impermeable UHPFRC deck slab that protects the girders from moisture. Results show that the system can span up to 30 m while achieving span-to-depth ratios equivalent to or better than competing slab-on-girder bridges.

Keywords: wood-concrete composites, timber bridges, short span bridges, external post-tensioning

1. Introduction

1.1 Scope

Research was recently completed at the University of Toronto to develop a single-span wood-concrete composite slab-on-girder bridge superstructure for use on secondary highways, by making use of glued-laminated (glulam) girders, an ultra-high performance fibre-reinforced concrete (UHPFRC) deck slab, and external unbonded prestressing [1]. This superstructure is referred to as the "proposed concept".

This paper focuses exclusively on superstructure design, for spans ranging from 10 m to 30 m. Fatigue is not considered, nor are lateral loads due to wind and seismic loading.

The design of the proposed concept is based on CAN/CSA-S6-06 Canadian Highway Bridge Design Code (CHBDC). Since the proposed concept is not explicitly recognized by the CHBDC, it was necessary to utilize other design standards, research papers, and engineering principles to complete the design. This paper presents some of the relevant design information that would be required in a design standard for the proposed concept, and introduces potential future research concepts for wood-concrete composite bridges.

1.2 Wood-Concrete Composites

Wood-concrete composites (WCCs) are a structural configuration wherein a concrete slab is made composite in longitudinal bending with either wood girders or a wood deck beneath. Composite action is achieved by means of a mechanical shear connector. The key benefits of WCCs are

improved strength and stiffness. Figure 1 illustrates the qualitative interlayer slip, longitudinal deflections, and elastic strain distributions for non-composite, partially-composite, and fully-composite WCC systems under the same midspan load.

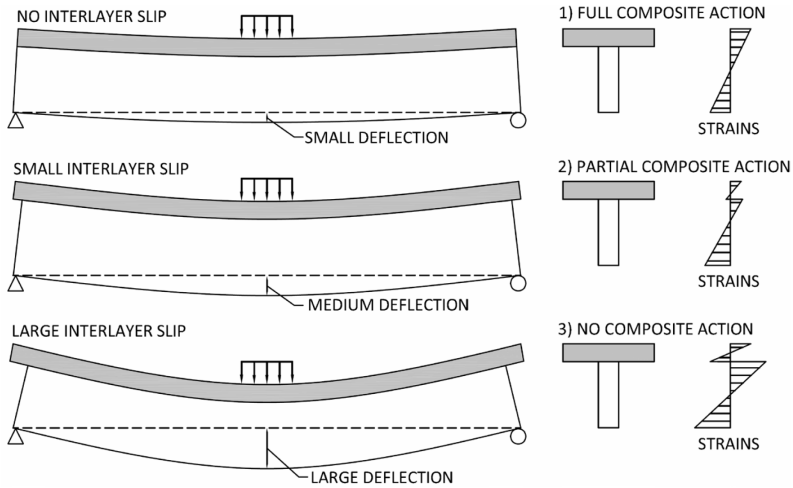


Figure 1 Varying degrees of composite action in a wood-concrete composite system (adapted from [2])

Large interlayer slip and longitudinal deflections develop in a wood-concrete system with no composite action because both the wood and concrete components undergo bending independently. The interlayer slip and deflections decrease when partial composite action is introduced. Furthermore, partial composite action reduces the strains in both materials, with the concrete tending towards compressive strain and the wood tending towards tensile strain under positive moment. There is no interlayer slip in a fully-composite system. The deflections are also smaller relative to partially-composite systems, and especially non-composite systems. In a fully-composite system, the concrete experiences predominantly or entirely compressive strains, while the wood experiences predominantly or entirely tensile strains. Due to flexibility of the shear connectors, all WCCs known by these authors, including the proposed concept, undergo partial-composite action.

WCCs combine wood and concrete to capitalize on the structural attributes of each material [3]. Concrete performs best in compression, and has relatively low tensile strength. Concrete is well-suited at the top of a WCC section, where it can be designed to resist only compressive stresses under partial or full composite action. The wood girders provide a very lightweight form of tensile resistance when compared to an equivalent all-concrete section [4].

1.3 The Need to Develop a Wood-Concrete Composite Bridge

In Ontario, non-composite slab-on-wood-girder bridges have been historically limited to spans of 15 m, beyond which the wood girders became too large for practical purposes [5]. The guidance offered by the CHBDC for WCCs is restricted to a specific slab-on-deck WCC system that is limited to spans up to 8 m [6]. As a result, WCC systems are rarely utilized in Ontario, or any other part of Canada.

Forestry is the second-largest industry sector in Ontario, with over 26 million of Ontario's 71 million hectares of forested land being sustainably harvested in commercial logging ventures [7]. However, wood is an underutilized material in Ontario bridge design. Only approximately 250 of the 2800 provincially-owned bridges are made primarily of wood [5]. With the Ministry of Transportation of Ontario having been a driver of wood bridge innovation in Ontario [5] [8] [9], it is seemingly unlikely that the municipal bridge inventory contains a higher percentage of wood bridges than does the provincial bridge inventory. The need for a wood bridge system in Ontario that is cost-competitive with conventional solutions is apparent given the scarcity of wood bridges, in spite of a sizeable forestry sector.

An examination of civil engineering undergraduate curricula in Canadian universities indicates that most schools offer no more than one course on the design of wood structures. Many schools do not even offer such a course. At best, students might be taught wood design over a few weeks in a general structural design class. Engineers who are familiar and comfortable with wood design have typically studied the subject at the graduate level, or learned it informally on the job. This educational limitation has left young engineers lacking in technical knowledge and ability to design wood structures. The proposed bridge design concept in this paper presents an opportunity to incorporate wood more commonly in infrastructure design.

2. The Proposed Concept

2.1 The Structural System

The proposed concept is a two-lane slab-on-girder bridge consisting of a 110 mm-thick UHPFRC slab and eight 365 mm-wide glulam girders. The cross-sectional dimensions have been standardized for all spans of the proposed concept. Only the glulam girder depths vary with span. Figure 2 illustrates a plan, longitudinal section, and cross-sections of the proposed concept for a 30 m span. The layouts would be similar for spans ranging from 10 m to 25 m.

UHPFRC was chosen rather than conventional concrete because its high tensile strength precluded the need for passive reinforcement in the deck slab, except in the cantilevers where reinforcing steel is necessary to resist barrier impact loads. The use of UHPFRC also allows for a slender slab that is much lighter than traditional deck slabs, thereby improving the live load capacity of the system. UHPFRC has very low permeability and high freeze-thaw resistance that enhances system durability. Although expensive, it is anticipated that the premium paid for the UHPFRC will be offset by these benefits over the life of the bridge.

Each glulam girder is prestressed by external unbonded longitudinal tendons located on each side of each girder. The tendons are deviated along a polygonal profile at the span fifth-points by U-shaped steel saddle deviators. The inclined tendons reduce longitudinal shear demand, flexural demand, and deflections of the glulam girders. The tendons are anchored inside full-width UHPFRC end buttresses that are poured monolithically with the deck slab. The buttresses house the prestressing anchorages and protect the ends of the glulam. The buttresses absorb the high forces within the anchorage zone, and distribute the anchorage forces between the slab and girders. The buttresses can be constructed with a shear key to transmit lateral loads from the superstructure to the substructure.

The proposed concept comprises two interior and two exterior full-span prefabricated double-T modules. Each module consists of two glulam girders made partially-composite with a UHPFRC deck slab by longitudinal shear connectors. The modules are match-cast so that their deck fasciae are geometrically compatible when abutted. Transverse post-tensioning tendons within the diaphragms and end buttresses are used to connect the modules after they have been erected.

Each module has several 200 mm-wide UHPFRC diaphragms that are cast monolithically with the deck slab. Notches are cut into the compression zones of the glulam girders to achieve continuous full-width diaphragms. No wood in tension is to be removed. The diaphragms improve live load distribution between girders and house the transverse post-tensioning that connects adjacent modules. Sufficient live load distribution occurs when the 15-30 m spans have diaphragms at their span tenth-points, and when the 10 m span has diaphragms at its fifth-points.

2.2 Fabrication & Construction

Prefabrication of the double-T modules affords many benefits. Prefabrication improves quality control by providing the best curing conditions for the UHPFRC, and results in a higher level of durability [10]. Prefabrication also significantly reduces the on-site duration of construction, thus reducing traffic impacts associated with any construction closures. The prismatic shape of the girders and slab allows for straightforward manufacture of the modules. The modules are light and small enough to fit on conventional flatbed trucks for transportation to site. Stability during erection is provided by the use of two-girders per module, with the UHPFRC slab and diaphragms stabilizing the glulam girders against buckling.

Prefabrication begins with the manufacture of the glulam girders. Two narrow longitudinal slots are cut along the entire top face of each girder for the longitudinal shear connectors. The girders are then pressure-treated with a wood preservative, if necessary. Two longitudinal shear connectors are glued into the slots. Two girders are placed in a casting bed for construction of the UHPFRC slab, diaphragms, and end buttresses. All longitudinal and transverse post-tensioning hardware is placed before casting. The technique of match-casting is utilized to preclude the need for on-site cast-in-place concrete closure joints.

The modules are erected on-site by a mobile crane. The transverse tendons are stressed to connect the modules together. The longitudinal tendons are then stressed. The curbs, steel PL-2 barrier, and asphalt wearing surface can be constructed after all stressing operations. Since the curb is cast on

site, conventional high-early strength concrete could be used as the curb material rather than UHPFRC. A life-cycle cost analysis would be required to determine the best alternative for the curb material.

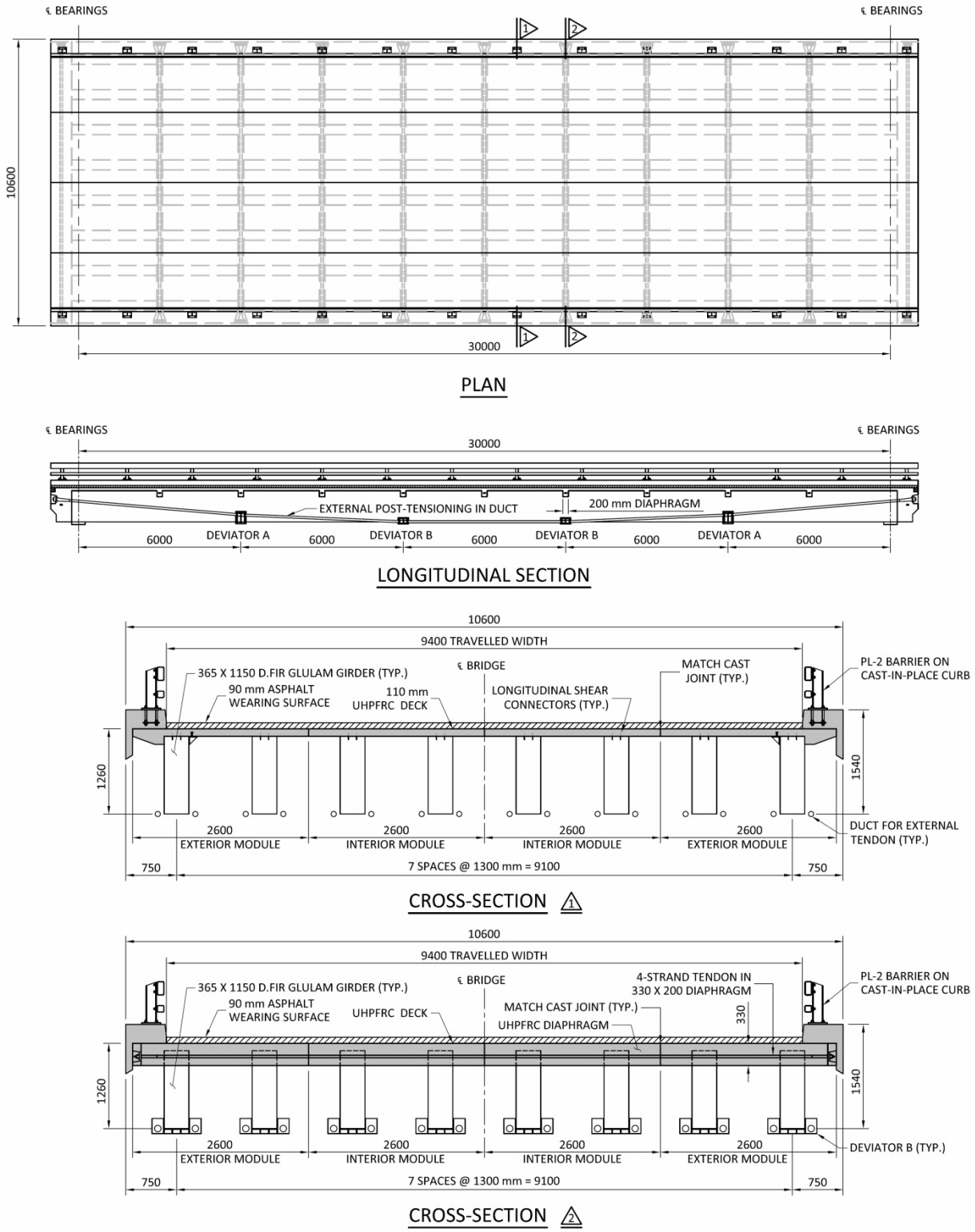


Figure 2 Plan, longitudinal section, and cross-sections for a 30 m span

2.3 Materials

The glulam girders are assumed to be Douglas-fir 24f-EX bending stress grade commonly manufactured in Canada.

The assumed properties for the UHPFRC are based on a mix design developed and tested at the University of Toronto [11] [12]. The peak compressive stress is 120 MPa, elastic modulus is 42065 MPa, cracking stress is 7 MPa, peak tensile stress is 14.6 MPa, long-term creep coefficient is 0.8, and total shrinkage strain is 552 microstrain after 50 days.

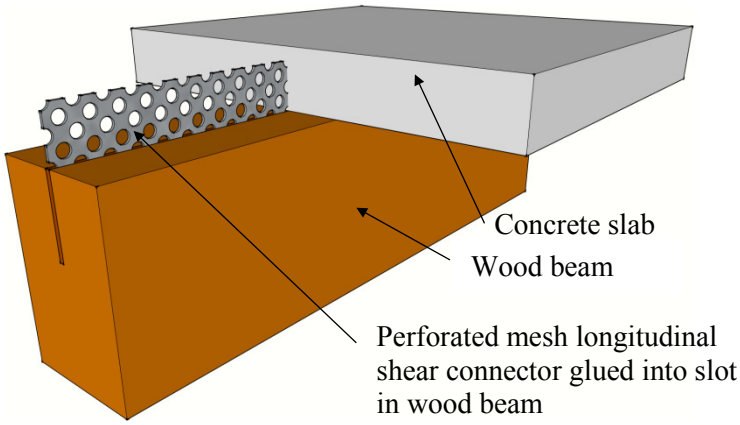


Figure 3 Perforated mild steel mesh longitudinal shear connector (adapted from [13])

Mild steel perforated mesh longitudinal shear connectors are assumed, per Bathon and Clouston [13], as shown in Figure 3. The 100 mm-deep connectors are embedded 50 mm into the glulam girders, and 50 mm into the UHPFRC slab, over the full length of the girders, with local discontinuities at diaphragm locations. This particular connector was chosen because it has well-developed, experimentally validated mechanical properties that are superior to most other shear connectors presented in academic literature, particularly with respect to stiffness and ductility. A different connector could be utilized, but the system behaviour would require re-evaluation.

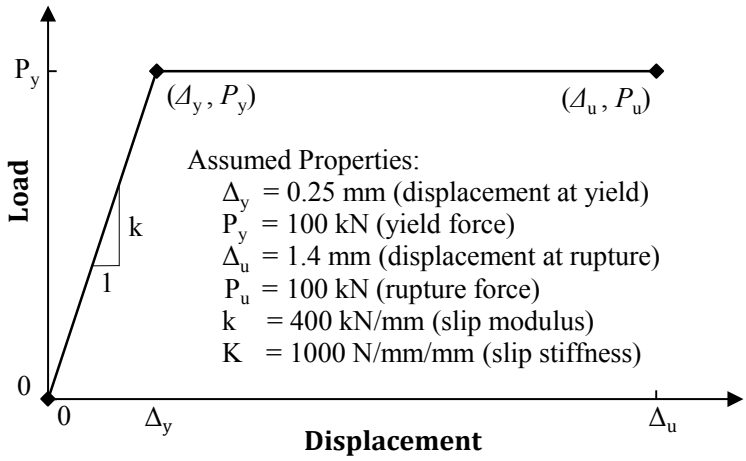


Figure 4 Idealized load-displacement curve for assumed longitudinal shear connector

The proposed shear connector was developed at the University of Applied Sciences, Wiesbaden, Germany [14] [15]. Figure 4 illustrates the idealized load-displacement curve of the connector. The behaviour is linear-elastic until yield, and perfectly-plastic until rupture. The observed plastic region is important because it indicates that the connector will allow slip between the wood and concrete as it approaches its own capacity, without reducing the shear force transferred between the wood and concrete components. This slip will result in system deflections that should alert users of the impending system failure brought on by the impending connector rupture.

2.4 Durability

Bridge durability can be enhanced by the use of WCCs. Controlling the ingress of moisture into wood is the most important aspect of preventing wood decay [16]. A WCC slab-on-girder bridge provides protection of the wood girders by means of the concrete deck slab. The deck can greatly increase the durability of the wood girders by preventing contact with moisture that originates on the deck surface [14]. Due to composite action and prestressing, the deck slab can be kept under compression, thereby precluding tensile cracks that could expose the wood girders to moisture from above [4]. Using UHPFRC for the deck slab is more effective than a deck slab made of conventional concrete because the material has very low permeability and is extremely resistant to freeze-thaw cycles [11]. For these reasons, it is anticipated that the moisture content of the glulam can be kept below its fibre saturation point, making wood preservation unnecessary for prevention of decay by moulds or fungi (note that preservatives may be desirable for preventative purposes and/or as a deterrent to insect attack). The prestressing tendons will be protected in greased ducts to

reduce the risk of corrosion.

3. Analysis of the Proposed Concept

3.1 The γ-Method

The CHBDC does not address partially-composite slab-on-girder WCCs. Consequently, the system was analysed as single T-girders by use of the "γ-method" ("gamma-method") that is described in Annex B of Eurocode 5: Design of Timber Structures [17]. The effective slab width was determined to be the centre-to-centre glulam girder spacing, implying that the entire slab participates in resisting loads acting on the longitudinal system. Transverse live load distribution was determined by a grillage analysis. The key assumptions of the γ-method are as follows:

- the wood, concrete, and longitudinal shear connectors behave linearly-elastically;
- the wood and concrete are connected by a mechanical fastener with constant slip stiffness K;
- the beam is simply-supported over a length L and transversely loaded to produce a shear force $V = V(x)$ and bending moment $M = M(x)$ that varies sinusoidally or parabolically;
- plane sections do not remain plane for the cross-section as a whole due to inter-component slip, but plane sections remain plane in each component.

The γ-method estimates the effective bending stiffness, as well as the normal stresses due to combined bending moments and axial forces within the system. The following explanation of the γ-method is best understood in conjunction with an examination of Figure 5.

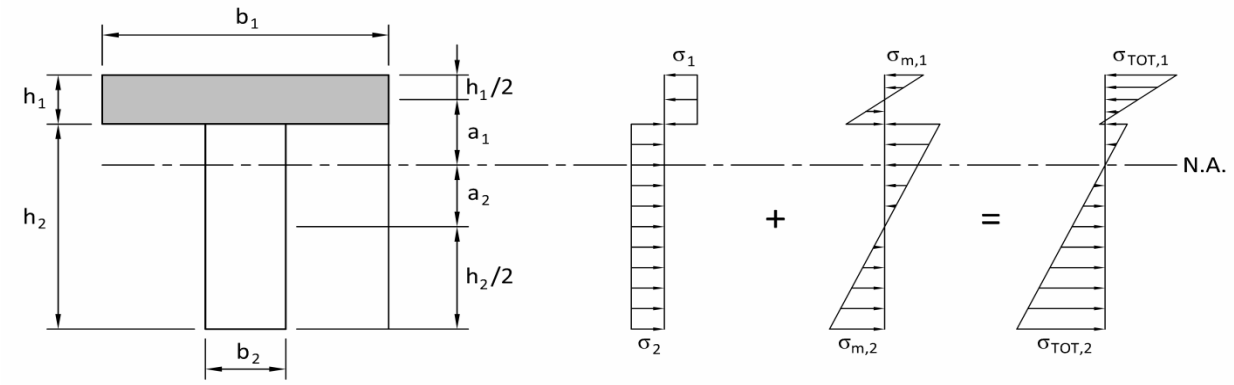


Figure 5 Cross-sectional geometry and stresses according to the γ-method

The effective bending stiffness $(EI)_{eff}$ of a simply supported WCC beam of length$L$, and constant slip stiffness K, can be calculated using equations 1 to 7,

$$(EI)_{eff} = \sum_{i=1}^{2} (E_i I_i + \gamma_i E_i A_i a_i^2) \tag{1}$$

$$A_i = b_i h_i \tag{2}$$

$$I_i = \frac{b_i h_i^3}{12} \tag{3}$$

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1}{K L^2}} \tag{4}$$

$$\gamma_2 = 1 \tag{5}$$

$$a_1 = \frac{h_1 + h_2}{2} - a_2 \tag{6}$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2(\gamma_1 E_1 A_1 + \gamma_2 E_2 A_2)} \tag{7}$$

where E is the modulus of elasticity, I is the moment of inertia, A is the cross-sectional area, a is the distance between the centroid of a component and the partially-composite neutral axis, γ is the shear coefficient, b is the component width, and h is the component depth. Subscripts 1 and 2 refer to the concrete and wood components, respectively. The maximum axial stress σ_i and maximum bending stress $\sigma_{m,i}$ for a given internal bending moment M, are calculated according to equations 8 and 9.

$$\sigma_i = \mp \frac{\gamma_i E_i a_i}{(EI)_{eff}} M \quad (- \text{ for } i = 1, + \text{ for } i = 2) \tag{8}$$

$$\sigma_{m,i} = \mp \frac{0.5 E_i h_i}{(EI)_{eff}} M \quad (- \text{ for the top fibres}, + \text{ for the bottom fibres}) \tag{9}$$

3.2 Effect of Prestressing

The tensile force in the unbonded tendons is a function of their elongation. It is proposed to ignore the additional strain in the tendons brought about by overall system deformations. Neglecting this additional strain is conservative and greatly simplifies the calculation of the prestressing force. The simplification is made by considering prestressed glulam analogous to prestressed concrete in an uncracked state. Before cracking, the latter is known to experience negligible additional strains in the tendons [18]. Noteworthy additional strains occur only after cracking. Since flexural cracking of concrete is analogous to the flexural rupture of glulam, it is posited that the additional strain in the tendons of the proposed concept will be negligible, even at the ultimate limit state. With flexural rupture of the glulam being taken as system failure, the additional strain in the tendons incurred beyond such rupture need not be calculated. Therefore, the prestressing force can be treated as a series of static loads applied at the anchorages and deviators.

The total loss of prestress, at and after transfer, is assumed to be 20% of the ultimate tendon stress. This assumption is based on a review of prestressed wood literature [19] [20].

4. Performance of the Proposed Concept

A slab-on-girder system using standardized girders from the Canadian Precast Prestressed Concrete Institute (CPCI) is the most common bridge superstructure in many parts of Canada for spans up to 45 m. Figure 6 illustrates a typical cross-section of this superstructure. It consists of parallel, prestressed, precast, concrete I-girders made composite with a cast-in-place concrete deck.

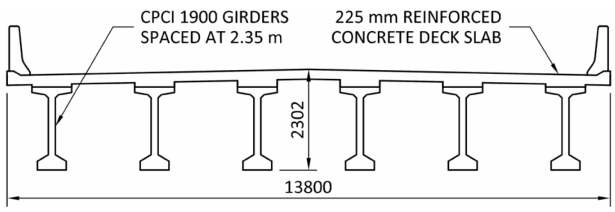


Figure 6 *Cross-section of a typical slab-on-CPCI-girder bridge*

The current popularity of the CPCI girder system stems from its large degree of standardization, resulting in relatively straightforward design and construction techniques. The precast girders provide an obvious construction time savings; although, this savings is somewhat undermined by the cumbersome process of casting the deck slab. This non-prestressed

Table 1 *Single-span CPCI girder bridges used for comparative purposes*

Span	Deck Width	# of Girders	Girder Type
16.6 m	13.5 m	7.0	CPCI 900
22.0 m	14.2 m	7.0	CPCI 1200
31.5 m	20.5 m	8.0	CPCI 1600

slab tends to crack in service, consequently compromising its durability. These cracks allow for the penetration of de-icing chemicals that can corrode the reinforcing steel. Nonetheless, the CPCI girder system remains popular in many parts of Canada.

Given its popularity and given that it is a slab-on-girder system, the CPCI girder system is a suitable benchmark for comparison with the proposed concept. Specifically, comparisons can be made with regard to superstructure weight, volume of cast-in-place concrete, and slenderness. The three single-span CPCI girder bridges presented in Table 1 have been chosen for comparison to the proposed concept. Three is a small sample; however, given the large degree of standardization among CPCI girder bridges, it is expected that the variance in geometrical and structural properties is rather small between any two CPCI girder bridges of similar span and width.

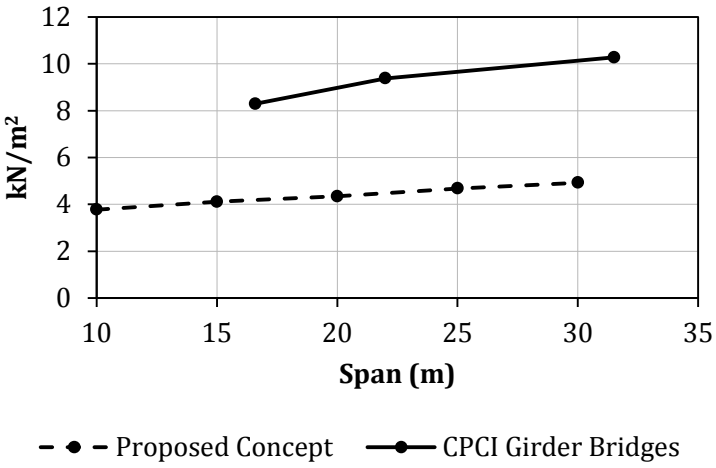


Figure 7 Superstructure weight per unit deck area

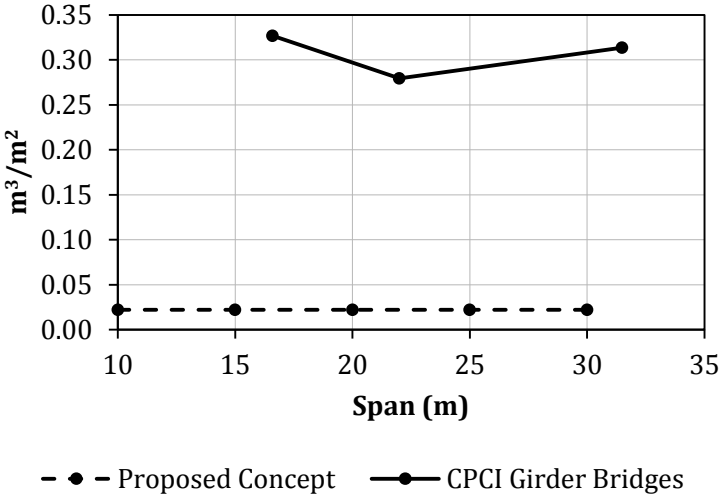


Figure 8 Volume of cast-in-place concrete per unit deck area

Before making comparisons to the CPCI girder system, recall that the proposed concept is valid for spans ranging from 10 m to 30 m. Also, recall that the proposed concept has been designed to carry two lanes of traffic loaded by the CL-625-ONT design truck that is the standard design vehicle for highway bridge design in Ontario.

A comparison of superstructure weights in Figure 7 shows that the proposed concept is about half as heavy as the CPCI girder system, suggesting that the latter is carrying a substantial amount of weight in its structural elements that does not enhance the structural capacity of the system. This weight disparity is also notable because it suggests that proposed concept will not require as large or as expensive foundations when compared to the CPCI girder system. A similar argument can made with regard to transportation of all constituent elements and/or materials to site. Finally, the significantly lesser weight of the proposed concept means that it will experience lesser inertial forces in a seismic event, which could result in savings throughout the superstructure and foundations.

Precasting concrete elements reduces the duration of on-site construction. Since both systems incorporate prefabricated elements, which require very little time for on-site erection, the volume of cast-in-place concrete is a useful metric in estimating which system will likely require a longer duration of on-site construction. Figure 8 illustrates such a comparison, where it is apparent that the CPCI girder system requires much more cast-in-place concrete than does the proposed concept.

The implication in Figure 8 is that the proposed concept will not require as much on-site labour or time to construct as does a CPCI girder bridge. To be fair, the proposed concept requires post-tensioning operations that the CPCI girder system does not require. Even so, the post-tensioning operations should not take more than a few days, ultimately having little effect on the duration of

on-site construction.

The volume of cast-in-place concrete is also a useful metric in evaluating durability because the controlled environment of a precasting facility affords the appropriate conditions for optimal curing. Conversely, cast-in-place concrete offers less quality control, as the curing of the concrete is largely at the mercy of on-site environmental conditions. For these reasons, precast concrete is often assumed to be more durable than cast-in-place concrete. In view of that, the proposed concept is likely more durable than the CPCI girder system, given its lower volume of cast-in-place concrete.

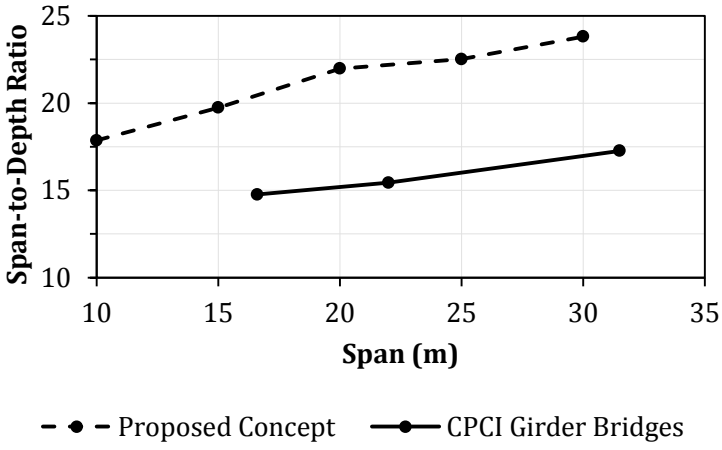


Figure 9 Comparative slenderness

Slender bridges tend to be lighter, more aesthetically pleasing, and more efficient in bending than less slender bridges [18]. Slenderness is commonly measured by the span-to-depth ratio, where the depth excludes all elements that do not explicitly contribute to longitudinal system strength (i.e. barrier walls). Figure 9 shows that the proposed concept is much more slender than the CPCI girder system. This observation is not surprising given the superstructure weight disparity shown in Figure 7. The implication is that the CPCI girder bridges are less efficient in bending than the proposed concept, which could also signify inferior economic performance.

Another implication is that the approaches for the CPCI girder system need to be higher than those for the proposed concept. Thus, the all concrete CPCI girder system requires additional fill material for the approaches and an increase in the height of any structures that retain approach fill.

5. Conclusions

The proposed concept is a feasible concept that can be included in the sub-set of spanning solutions for Canadian bridge engineers. It uses glued-laminated timber to capitalize on the sizeable forestry industry in Ontario that is underutilized in bridge construction. The following remarks can be made about its structural performance relative to the commonly constructed CPCI girder superstructure:

- The proposed concept is significantly lighter, likely affording it advantages with regard to foundation design, seismic design, and transportation of constituent elements to site;
- The proposed concept utilizes far less cast-in-place concrete, suggesting that it should be more durable and likely capable of being constructed more quickly;
- The proposed concept is much more slender, implying greater structural efficiency and requiring shallower approach fill and associated retaining structures.

6. Acknowledgements

This paper was borne from research conducted at University of Toronto by the first author under the supervision of Professor Paul Gauvreau. Research input was provided by Moses Structural Engineers. Funding was generously afforded by Moses Structural Engineers, FPInnovations, the Natural Science and Engineering Research Council of Canada, and the University of Toronto. The authors would like to extend their gratitude to both Delcan and Moses Structural Engineers bringing this paper to fruition.

7. References

[1] Lehan, A.R., "Development of a Slab-on-Girder Wood-Concrete Composite Highway Bridge", M.A.Sc. thesis, Dept. Civ. Eng., Univ. Toronto, Toronto, Canada, 2012.

[2] Lukaszewska, E., "Development of Prefabricated Timber-Concrete Composite Floors", Doctoral dissertation, Dept. Civ. Min. & Env. Eng., Luleå Univ. of Tech., Sweden, 2009.

[3] Clouston, P & Schreyer, A., "Design and Use of Wood-Concrete Composites", *Practice Periodical on Structural Design and Construction*, Vol. 13, No 4, 2008, pp. 167-174.

[4] Clouston, P., Civjan, S., & Bathon, L., "Shear Behaviour of a Continuous Metal Connector for a Wood-Concrete Composite System", *Forest Products Journal*, Vol. 54, No 6, 2004, pp. 76-84.

[5] Krisciunas, R., Gratton, J., Weiss, G., & Scalzo, P., "Renaissance of Wood Bridges in Ontario", 8th Int. Conf. on Short & Medium Span Bridges, Niagara Falls, Canada, 2010.

[6] Taylor, R.J. & Keenan, F.J., *Wood Highway Bridges Design Manual*, Canadian Wood Council, Ottawa, 1992, pp. 16.

[7] Ontario Ministry of Northern Development, Mines, and Forestry. (2012, May). 2010-2011 Annual Report. Government of Ontario. Available: http://www.mndm.gov.on.ca/en/about-ministry/annual-report/annual-report-2010-2011

[8] Taylor, R.J., "Field Applications of Prestressed Laminated Wood Deck Bridges", *Canadian Journal of Civil Engineering*, Vol. 15, No. 3, 1988, pp. 477-485.

[9] Krisciunas, R., "Two Composite Structural Lumber Bridges in Northwestern Ontario, Canada", *Structural Engineering International*, Vol. 10, No. 3, 2000, pp. 158-160.

[10] Visscher, B.T., "Innovative Pre-cast Cantilever Constructed Bridge Concept", M.A.Sc. thesis, Dept. Civ. Eng., Univ. Toronto, Toronto, Canada, 2008.

[11] Habel, K., Charron, J.P., Braike, S., Hooton, R.D., Gauvreau, P., & Massicotte, B., "Ultra-High Performance Fibre Reinforced Concrete Mix Design in Central Canada", *Canadian Journal of Civil Engineering*, Vol. 35, No. 2, 2008, pp. 217-224.

[12] Salonga, J.A., "Innovative Systems for Arch Bridges using Ultra High-Performance Fibre-Reinforced Concrete", Doctoral dissertation, Dept. Civ. Eng., Univ. Toronto, Toronto, Canada, 2010.

[13] Bathon, L. & Clouston, P., "Experimental and Numerical Results on Semi-Prestressed Wood-Concrete Composite Floor Systems for Long Span Applications", The 6th World Conference on Timber Engineering, Lathi, Finland, 2004.

[14] Clouston, P, Bathon, L., & Schreyer, A., "Shear and Bending Performance of a Novel Wood-Concrete Composite System", ASCE *Journal of Structural Engineering*, Vol. 131, No. 9, 2005, pp. 1404-1412.

[15] Bathon, L. & Bletz, O., "Development of an Innovative Wood-Concrete Composite System", SECON International Conference on Bridges, Dubrovnik, Croatia, 2006.

[16] Morris, P., *Understanding Biodegradation of Wood in Structures*, Forintek Canada Corp., Vancouver, 1998, pp. 14-15.

[17] ENV 1995-1-1:2004 (E) (Eurocode 5): *Design of Timber Structures, Part1-1 General Rules and Rules for Buildings*, 2004.

[18] Menn, C., *Prestressed Concrete Bridges* (translated and edited by Paul Gauvreau), Birkhäuser, Basel, Switzerland, 1990, pp. 139-147.

[19] Davies, M. & Fragiacomo, M., "Long-Term Behaviour of LVL Members Prestressed with Unbonded Tendons", *New Zealand Timber Design Journal*, Vol. 16, No. 3, 2008, pp. 13-20.

[20] Palermo, A., Pampanin, S., Carradine, D., Buchanan, A.H., Dal Lago, B., Dibenedetto, C., Giorgini, S., & Ronca, P., "Enhanced Performance of Longitudinally Post-Tensioned Long-Span LVL Beams", 11th World Conf. on Timber Eng., Riva del Garda, Trentino, Italy, 2010.

On development of network arch bridges in timber

Kjell A. MALO
Professor, Dr. ing
NTNU, Dep. of Struct. Eng.
Trondheim, Norway
kjell.malo@ntnu.no

Anna OSTRYCHARCZYK
MSc, Ph.D. Candidate
NTNU, Dep. of Struct. Eng.
Trondheim, Norway

Runa BARLI
MSc Student
NTNU, Dep. of Struct. Eng.
Trondheim, Norway

Idun HAKVÅG
MSc Student
NTNU, Dep. of Struct. Eng.
Trondheim, Norway

Kjell Malo is currently working on research topics like methods for increased strength and stiffness of connections for timber structures, and vibrations in timber structures.

Anna Ostrycharczyk is working in the field of timber constructions. Current research topic is vibration in timber structures.

Runa Barli and Idun Hakvåg have started their professional career in consulting companies in the field of structural engineering.

Summary

Inclined hangers have been used in bridges for nearly hundred years. The network arch bridge was developed in the early sixties based on steel arches, concrete decks and tension ties. The most important feature is the use of inclined and crossing hangers, which typically cross each other twice or more. The use of inclined hangers dramatically lowers the moment in the arch and makes the bridges much stiffer. By introducing this concept for timber bridges very interesting perspectives are opened, as it becomes easier to obtain the necessary stiffness and also simplified connections for the hangers may be introduced. The paper describes current work on a network arch bridge where both the deck and arch are based on timber. The chosen concept for this study is represented by a scaled experimental bridge model in the laboratory, which is used in parallel to numerical models.

Keywords: timber bridge, arch, inclined hangers, network arch, stress laminated, timber deck, experimental, numerical, scaled model

1. Introduction

1.1 Arch bridges

Many of the arch bridges built in Norway in the recent years have the arch above the deck and use vertical hangers to transfer the deck loading to the arches. Bridges having monolithic cross-section in the arches, are typically based on the three hinge statically determined lay-out. In cases with longer spans the arches are made by the use of truss work which enhances the bending capacity of the arch without much increase in material use. These design concepts are well expressed at Tynset bridge (70 m span), see Fig. 1. The connections used in the truss work consist usually of slotted-in steel plates combined with a plurality of steel dowels. For the truss work type of arches it may be more convenient to apply the two hinge arch concepts.

The size of a prefabricated bridge part depends on the curvature and length due to limitations in chemical treatment facilities as well as transport. Typically, parts exceeding the length of 30-35 m cause problems. For the truss-work type of arches this size handling problem may be overcome by

using the truss connections also as mounting connections. This can readily be done as the connections in a truss are exposed solely to axial forces. However, the splicing of arches having massive cross sections is much harder since these have to be spliced also for quite large moments.

Fig. 1 Tynset bridge, Norway (photo: K. Bell)

In bridge lay-out with vertical hangers the moment actions in the arch will easily become large due to traffic loads, introducing concentrated high forces as point loads in the arch. Furthermore, by introducing the loads into the arch by few vertical hangers, the combination of shear stress and tension normal to grain might become challenging, see more about this e.g. in [1].

1.2 Sideway stability

Usually the arches are made so slender that there is a need for sideway support to increase the stability. Having arches unsupported along the span necessitates clamping at the support in the transverse direction. In accordance to St. Venant's principle, the effect of the clamping will diminish as the arches are made longer and more slender. For long span bridges the height of the arches might be sufficient to design a frame action for sideway forces. The rigid corner of the moment carrying frame are then formed by the wind bracing at the top of the arches, connecting two arches by e.g. a K-shaped truss, carrying all forces to the supports. This has been used for Tynset bridge, see Fig. 1. I this case the hangers are vertical and hinged at both ends transferring only vertical forces and hence, no horizontal forces from the arches are transferred to the deck.

For smaller spans a simpler design has been utilized where the transverse actions have been transferred to the timber deck. This is achieved through the use of transverse U-shaped steel frames. Here, the hangers are replaced by rigid beams, increasing the necessary transverse stability properties of the arches. The pre-stressed timber decks have high stiffness and acts as a horizontal beam, transferring the transverse actions to the supports, see Fig. 1.

1.3 Durability issues

The durability requirements have mainly been met by use of chemical treatments like CU and creosote impregnation, in combination with cover of details and connections. This is the approach for durability used on Tynset bridge, see Fig. 1. It is expected that in the coming years more restrictions will be put forward on the use of environmental unfriendly chemicals and for long durability of timber bridges this can easily lead to increased costs. Considering Fig. 1, it is quite obvious that "clean" designs like the two small side span of the Tynset bridge are much easier to cover in some way than the complicated 3-D truss of the main span. An example of cover used on a simple bow-string bridge is Fretheim bridge, Norway, see Fig. 2. Here, copper cladding is used on the top faces, while the side faces is cover by ventilated venetian blinds .

Fig. 2 Fretheim bridge, Norway (Photo by SWECO, [2])

The basic principle to obtain durable timber structures is simple, it is about to keep the water out of the wooden material in order to maintain a moisture content below 18 to 20 % (in the Scandinavian climate). Susceptible points for entrance of water are everywhere, where water can be collected in liquid form like on horizontal upward surfaces, in cracks on horizontal and vertical surfaces, around details like fish-plates, screw discs, bearing shoes and in connection to drilled holes and cut-outs which are not pointing downwards. It is also to be kept in mind that water is very rapidly transported into the wooden material if end-grains are brought in contact with liquid water. More on recent findings related to durability can be found in [3] and [4].

Some countries like Switzerland and USA have several fully covered bridges showing good performance for centuries. However, this approach has mainly been used for pedestrian bridges and it is more questionable for road bridges. Heavy goods vehicles will on a rainy day transport considerable spray of water into the bridges and the cover may prevent effective ventilation and drying up. The result may be high levels of moisture content in the wood. Furthermore, the wind exposed area of fully covered bridges might be considerable, leading to strong requirements for sideway stiffness.

2. Massive Arch bridges

2.1 Inclined hangers

Many durable timber bridge designs exist, e.g. where the load carrying structures are located beneath a water tight deck and the end support details are designed so that they do not retain dirt and water. However, most building sites necessitate that the load-carrying structure is located above the deck. For traffic loading both the truss and the arch are effective structures in this case. The arch is preferred here since this structure can be more easily adopted to design for long durability.

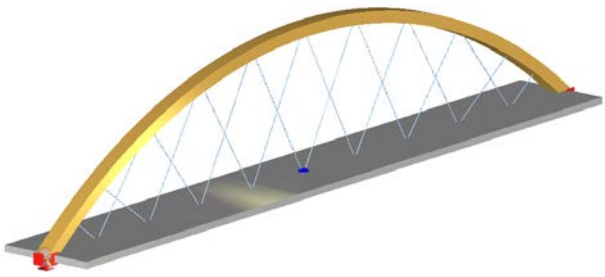


Fig. 3 Network arch, after [1]

For bridges exposed to traffic loading, the design case is often heavy loading located in a skew position. Using an arch with vertical hangers, this skew and relatively local loading will be transferred to the arch through the hangers as point loads. The result is large moment action in the arch. For the truss work arch this moment action is dealt with by a considerable increase of the internal moment arm, but it is also a possibility to reduce the moment action. O. F. Nielsen [5], nearly hundred years ago, appears to be a pioneer introducing inclined hangers in a V-configuration for bowstring arches transforming the moment action into a more clean compressive arch bearing. This concept was further developed by P. Tveit [6] in the late fifties, letting the inclined hangers cross each other more than once, denoting it as “network arch bridge”. A summary of his design considerations is available from [7]. The effect of inclined hangers and network hanger design on bowstring arch bridges is clearly

demonstrated in [1], showing that the moment action in the arch might be reduced to roughly one quarter and a give reduced vertical displacement of the deck to nearly one sixth.

2.2 Stability of network arch

The in-plane stability of a network arch is dependent on the stiffness of the arch member as well as the deck and the distance between the hangers. In general it is forced to buckle in an inward fashion, leading to good in-plane stability.

For the out-of-plane stability, the inclined hangers do not make much difference compared to vertical hangers. For a two hinge network arch sideway clamped at the supports, the two lowermost buckling modes are vizualized in Fig. 4. In fact, assuming that the load can be imposed on the arch in some way, the instability modes do not change much from an arch without hangers or a three hinged arch with the same geometry. For the three hinged arch the critical buckling loads will be only slightly smaller. Usually, the out-of-plane stability is assured by coupling two arches together with a bracing in-between. However, in the present work a different approach will be explored.

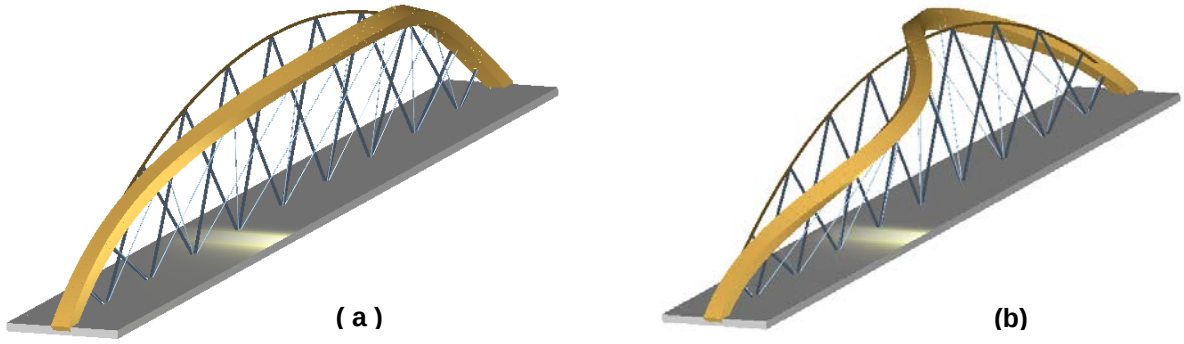


Fig. 4. The two lowermost buckling modes for an arch, after [1]

Consider Fig. 3, and assume that each of the hangers consists of two parallel rods. If the fasting points are displaced in the transverse direction in the deck for the two rods, the layout will be very similar to the familiar layout of the spokes in a bicycle wheel, see a visualization in Fig. 5.

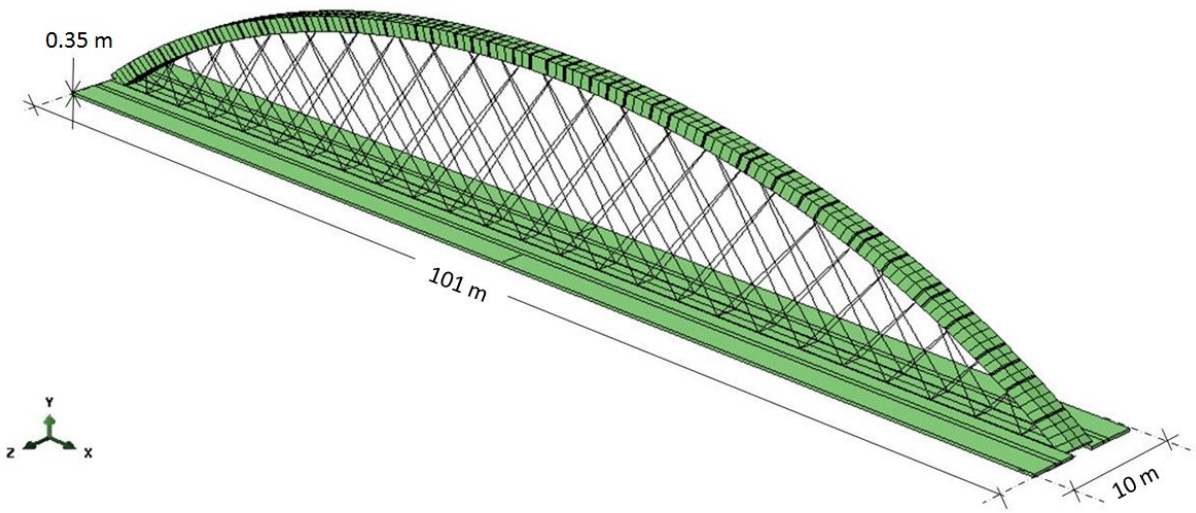


Fig. 5 Network arch with double hangers in spoked wheel configuration.

A simple model to demonstrate the possible effect on stability from this spoked wheel configuration can be made as follows. Consider Fig. 6 a), showing a situation with hangers only in the plane of the arch. If we assume that the arch tend to displace and rotate about the line between the support points, this rotation will be like a rigid body rotation and no lateral forces are activated to resist the

rotation. However, in the spoked wheel configuration, see Fig. 6 b, a lateral displacement or rotation about the support line will lead to an elongation of one of the hangers and thereby forces will develop to resist the lateral displacement.

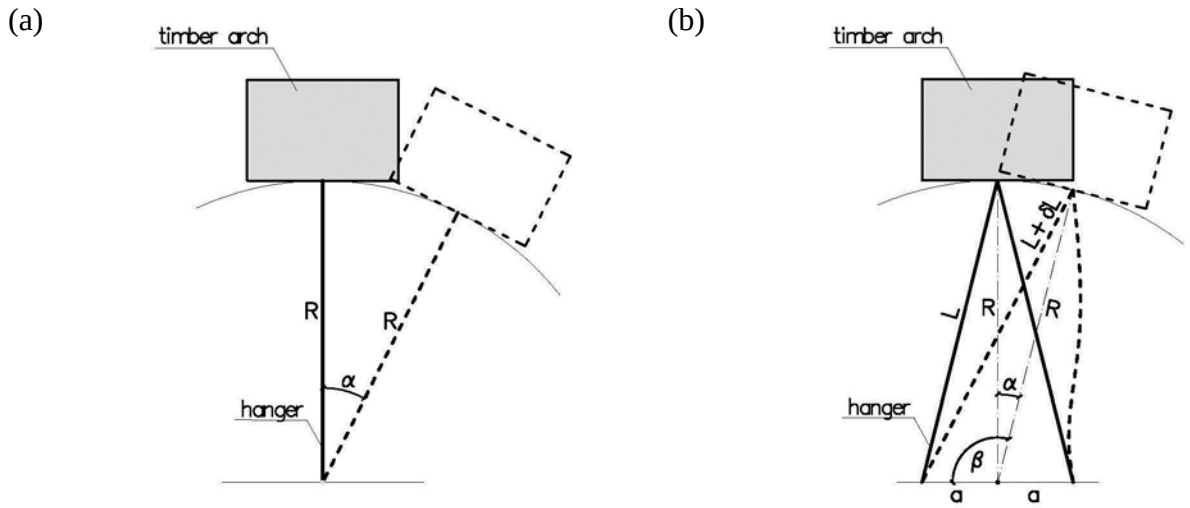


Fig. 6 Lateral stiffness from spoked wheel configuration.

A simple plane model reveals the governing parameters, consider Fig. 6 b. Assuming hangers of length L and a rigid body rotation α, the length of the tensile hanger in the displaced position is obtained by solving Eq. (1)

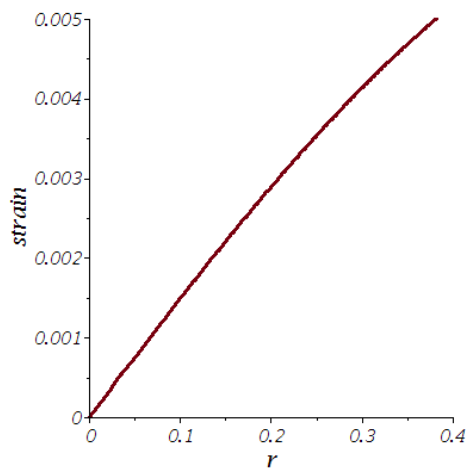
$$\left(L+\delta L\right)^2 = a^2 + R^2 + 2aR\sin(\alpha) \tag{1}$$

Here, the elongation is δL, a is the distance between the displaced fastening points of the hangers relative to the centreline of the arch, and R is the distance to the fastening point in the arch from the line of rotation. The strain in the tension hanger becomes:

$$\varepsilon = \sqrt{1+\frac{2aR}{a^2+R^2}\sin(\alpha)} - 1 \tag{2}$$

By introducing the geometric ratio $r = a/R$ and assuming small angles of α, a good approximation of the strain is

$$\varepsilon = \frac{r}{r^2+1}\alpha \tag{3}$$



A reasonable value of the rotation is about $\alpha = 0.015$ and Eq. (3) is plotted for this value in Fig. 7. Even for small values of $r = a/R$ there is considerable amount of strain in the tensile hangers with the spoked configuration. The actual values of a and R are of course dependent on the chosen design, but it is quite clear that arches having smaller curvature are beneficial from this point of view. The practical range of interest for the geometric ratio $r = a/R$ is may be up to the value of 0.4. Typically this value may be obtained close to the supports, but it becomes considerable less at mid-span. Therefore the sideway support effect will vary along the span in the same manner.

Fig. 7 Strain in hanger.

3. Concept study of long network arch bridge with spoked hangers

The major part of this study has been performed through the Master Thesis' of MSc Runa Barli and MSc Idun Hakvåg [8]. Only parts of the study will be presented here.

3.1 Conceptual design

3.1.1 Structural models

The objectives of this study were to explore if the network arch concept using inclined hangers can be combined with a light-weight deck and to study the effects of the hangers in a spoked configuration i.e. a configuration where the hangers are inclined both in the plane of the arches as well as in the transverse direction. The approach in this study has been to use a full 3-D numerical model [9] for the bridge and in order to catch the real behaviour of materials and connections, a scaled experimental model has also been built. The experimental model has been made using geometric scaling 1:10. A few exceptions have been necessary due to limitations in availability of materials, and here similar stress level has been used as the governing condition. It is believed that this has negligible influence on the overall behaviour of the bridge. Finally, to bridge the gap between the scaled experimental model and the full scale model, an accurate numerical model of the experimental layout has also been created.

The design configurations have been made on the full-scale numerical model applying the European timber structure regulations [10] and [11] and the regulations for traffic loads [12] and wind [13].

3.1.2 Design requirements

The requirements for the conceptual design approach were the following; free span of 100 m, two lines of road traffic leading to width of deck of 10 m, glulam circular arch(es), inclined network hangers in spoked configuration, tension tie, no wind truss between arches and a timber stress laminated deck. Design lay-out which is known to cause large problems with respect to moisture trapping and durability, should be omitted [3].

3.2 Design considerations

3.2.1 Inclination of hangers in the plane of the arch

The objective of the use of inclined hangers or network arch design is to reduce the moment in the arch, letting it carry the loads by compressive stresses. This lead to a great reduction in the amount of material needed for the arch. The side-effect which might occur from the use of inclined hangers is that some of the hangers may become relaxed. This can lead to buckled hangers and a visual or practical problem. But also a stability problem might occur, since the in-plane buckling length is dependent on the distance between the hangers and relaxed hangers are ineffective in this regard.

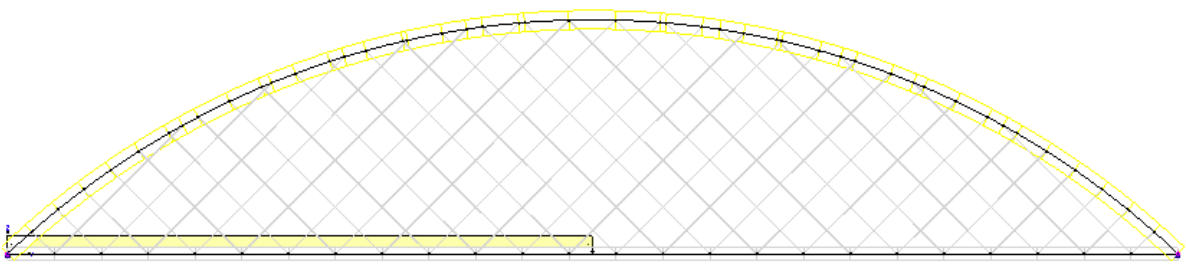


Fig. 8 Hanger layout with constant horizontal spacing and angle

The most straight-forward approach is to use a hanger layout with constant horizontal spacing and angle, see Fig. 8. From numerical simulations in the range of angles between 45 to 80 degrees (relative to the deck), it seems like the best choice is 45 degrees since this will lead to few relaxed hangers for skew loading. Imagine that the hangers occur in pairs from the fastening point at the deck and that they approximately have the same force, the resulting force in the pair of hangers will be vertical for all fastening points.

The idea behind the next lay-out is visualized in Fig. 9, to the left. To maximize the effect of arch bearing, it is best to impose the loading along radial rays with constant angle between adjacent rays. However, using only single rods along the rays when only one or a few of the rays carry loads, leads to large moment action in the arch and possible instability due to relaxed hangers. To spread out the force, pairs of hangers are introduced. I this lay-out the resultant of the forces in the pairs of hangers follow the direction of the radial rays.

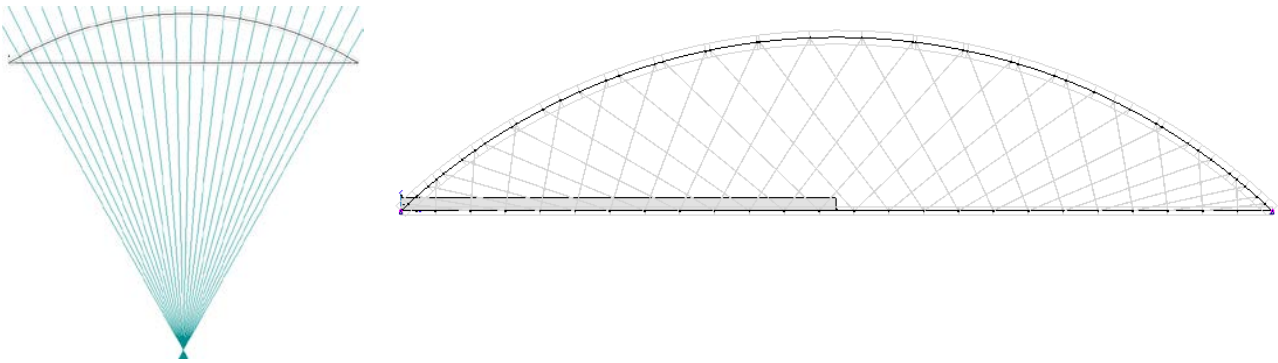


Fig. 9 Hanger layout with radial resultants of pair of hangers

Ideally the spacing along arc length should be kept constant, and this will lead to varying spacing of the fastening points in the deck. This might be a good choice if the deck has sufficient stiffness for the largest distance between the fastening points.

A concept which is close to the ideal situation is also gained by choosing equidistant fastening of the pair of hangers along the deck. In this case the fastening points along the arch will vary and be determined by the spreading angle of the pairs of hangers. This design concept is depicted to the right of Fig. 9. The angle between the intersecting pairs of hangers in this case is close to 90 degrees. Numerical simulation on this lay-out does not show any relaxed hangers, and this lay-out was chosen for further evaluations.

3.2.2 Stability

The height (depth) and width of the cross-sections together with the rise to span ratio and the out-of-the-plane support condition of the arch strongly influence the stability properties. The in-plane stability of a network arch is high and it may be reasonable to let the width of the cross-section be larger than the height in order to increase the out-of-plane stability. Furthermore, although the compressive forces in the arch will increase together with a slight increase in moment actions, it will be beneficial for the sideway stability to use an arch with quite low rise to span ratio. Suitable range for the rise (bow height) for arch bridges is about 0.1 to 0.25 of the span. In this case it was found that 0.14 is a reasonable good choice.

Moreover, to benefit from clamped conditions in transverse direction, It was chosen to let the width of the cross-section be 1.5 times the height, leading to an increase of the sideway stability properties of about 50%.

3.2.3 Transverse inclination of hangers

As observed from Eq. (3) as well as from Fig. 7, the transversal support from the transversely inclined hangers is almost linear with the ratio $r = a/R$. Therefore, the transverse distance between the fastening point of hangers ($2a$) should be made large and the rise of the arch small in order to improve sideway stability.

The area determined by the vertical projection of arches onto the deck cannot be used for traffic purposes since the entrance will be blocked by the arches. Therefore, this area can be used for inclined hangers and eventually guard railings. Without exceeding this area the transverse distance between the fastening point of spoked hangers ($2a$) is limited to the width of the cross-section.

3.3 Design for full scale 100 m bridge

The chosen design is not optimized, it is meant for conceptual study. A key point is the sideway support, and it is here chosen a relatively thick timber deck and weak transverse beams in order to perform further studies of the requirements for sideway stability.

The design for the full scale bridge leads to the following geometry and materials; The distance between the supports is 100 m and the rise of the circular arch is set to 14 m. The two hinge arches are in glulam, Scandinavian quality CEL40c (close to European GL 32c). The cross-section of the arches has width equal 1.8 m and height 1.2 m, constant along the span. A stress-laminated timber deck is used with width of 10 m and thickness of 1 m. Under the timber deck transverse steel beams, IPE 400, are used with spacing of 4 m. All hangers occur in pairs both in-plane and in the transverse direction, and are fixed to the top face of transverse beams and to the and the bottom face of the arches, see visualization from the numerical model in Fig. 10. All hangers are steel rods with diameter equal to 40 mm and they are fastened by axial load carrying screws to the wood pointing in the same direction as each pair of hangers

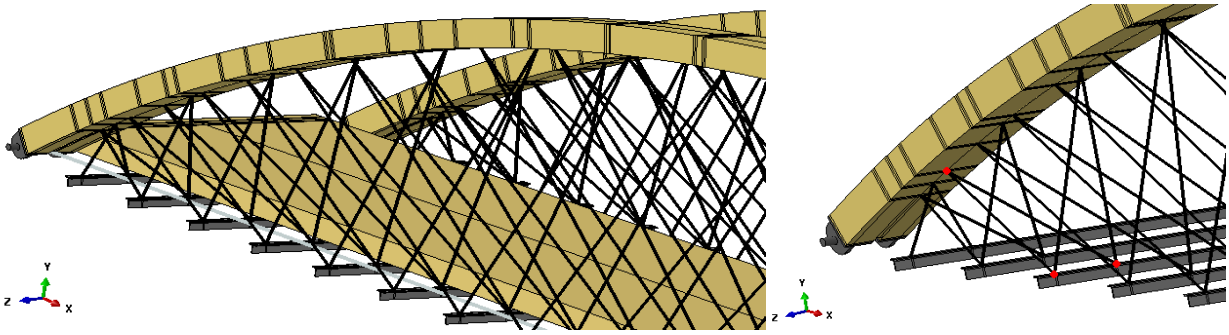


Fig. 10 Fastening of hangers to the transvers beams (numerical model)

The design was mainly performed by numerical non-linear 3-D simulations using Abaquas [9]. The investigations with respect to the stability were made by first imposing the dead weight and wind loading in the transverse direction. The deformation of the arches due to wind loading is then used as out-of-plane deformations for the consecutive stability analyses increasing the dead and live load until instability occurred. The analyses showed that the stability with respect to the load carrying capacity was good, the design load level could be increased by a factor of about 11 before instability occurred (exclusive load combination factors).

4. Scaled laboratory model

4.1 General

The 1:10 scale experimental model of the chosen bridge design is shown in Fig. 11 in finished condition. The experimental model will be further used as basis for future studies where different design options will be investigated. The arches, the timber deck, the support hinges for the arches and the tension ties will be kept as here, but different options for hangers and sideway support will be investigated. In the present study most emphasis has been put on stiffness and dynamic properties in order to spot the governing parts for possible improvements. The load capacity and safety of the bridge appears to be good, and therefore the main focus has been on the properties in the serviceability condition.

4.2 Arch support

The support of the bridge is collected on one square hollow section (HUP 120-120-10) beam in each end. The deck rests on a small square hollow section (HUP 50-50-5) and is fixed for sideway movements by adjustable steel support plates, visible on Fig. 11and Fig. 12. The arches are mounted in the supports by installation of the pins.

Fig. 11 Experimental model in scale 1:10

Ideally, the arches should have been fully clamped with respect to sideway stability, but this is of course not achievable in reality. There will be some clearance around the pin in the connection and some flexibility in the fixing of the plane support plates against the mating compression surface of the wooden arches. However, it has been previously observed quite rigid behaviour using inclined threaded rods [14], so for this prototype self-tapping screws have been used experimentally model the behaviour of connections using threaded bars.

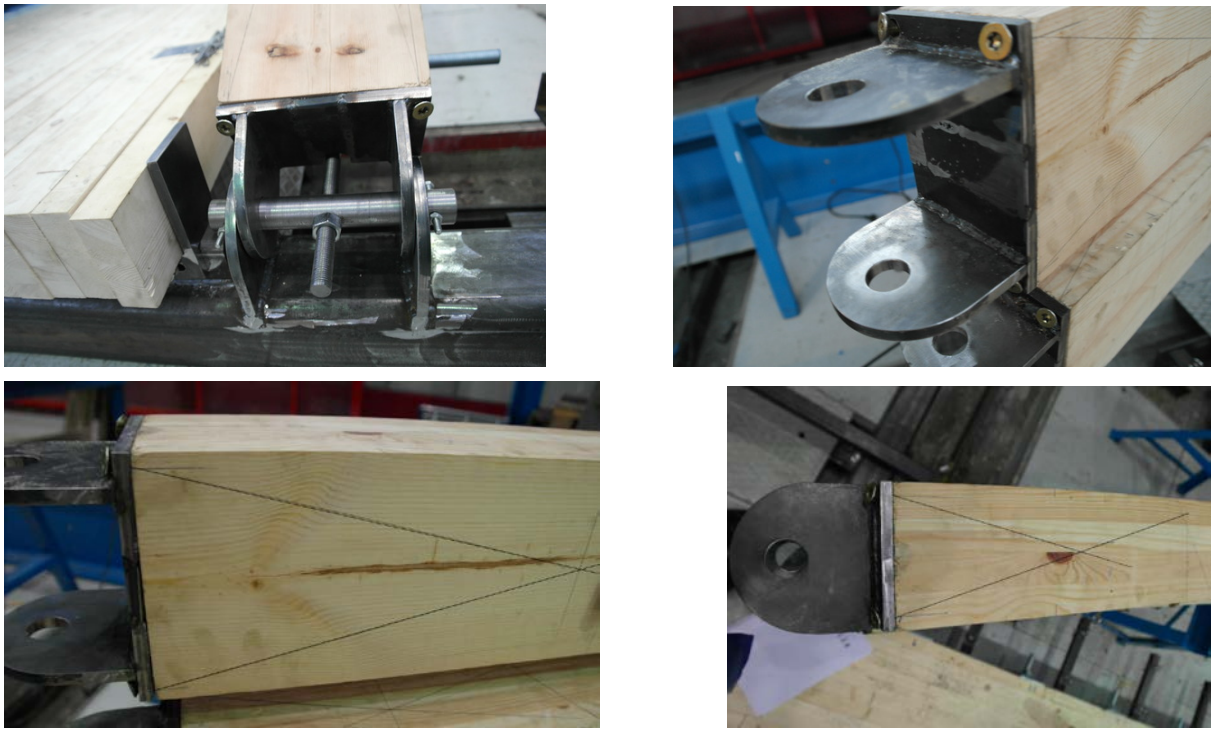


Fig. 12 Support conditions; hinged in the plane of the arch, transversely rigid

Four screws has been used, one in each corner of the supporting steel plate. The inclination of the screws is roughly 15 degrees in one plane and 8 degrees in the other. The inclination of the screws at the support detail is visualized by the drawn lines on the wood surface. Tensile rigidity is gained by the axial stiffness of the screws. The shear forces in the connection are addressed by the transverse force component of the inclined screws.

4.3 Hangers

A very simple fastening technique has been used to fix the hangers to the wooden arch. The hangers are mounted by threads in a small steel piece which in turn are fastened to the wooden arch by just using axial load carrying screws installed in the direction of pair of hangers; see the details on Fig. 13. The fastening of the hangers to the transverse beam is made using eye bolts and turnbuckles. The transverse beams are fastened on the bottom face of the deck by self-tapping screws.

Fig. 13 Fastening of hangers to the wooden arch.

5. Structural behaviour of bridge

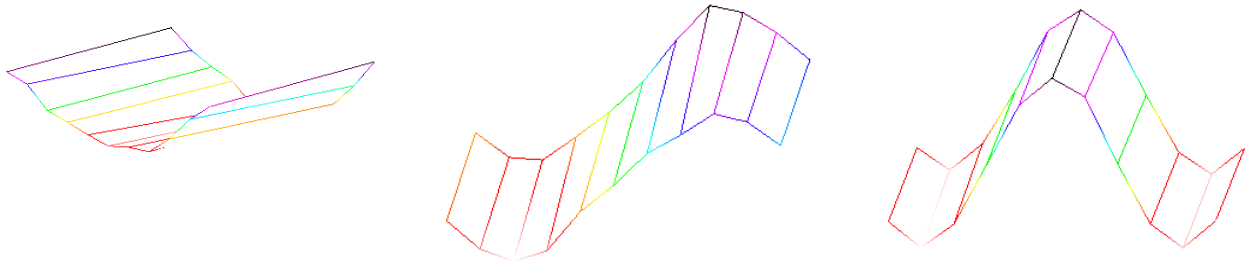
The load bearing capacity and safety of the bridge appears to be sufficient, and it is deformations and vibrations which become the most challenging subjects. Bridges exceeding span length sixty to eighty meters tend to get so low fundamental frequencies that wind and traffic loading may induce dynamical effects. The interesting parameters for evaluation of the bridges than become stiffness and mass distribution, eigenfrequencies and vibrational modes, acceleration levels and damping characteristics.

5.1 Scaled models of the timber deck

The amount of wood material in the timber deck is roughly twice of that in the arches. The deck is an important part of the bridge and has a great influence on the structural behaviour of the bridge. The stress-laminated deck height in the scaled model is 98 mm and the deck was laid out on a horizontal floor and pre-stressed to nominal stress of 1.0 MPa. The evaluation here was done prioer to mounting of the transverse beams. After pre stressing, the deck was put on two simple beam supports and the deformation due to the self-weight (measured to 560 kg) was compared to the simple beam formula. The measured deformation was only 4.4 % larger than the theoretical result for a beam having a massive cross-section.

The dynamic behaviour of the deck alone was also characterized by experiments and numerical simulations. The experimental results were obtained by using an impact hammer and the roving hammer test (RHT) protocol. The input to the numerical simulation was made on the basis of measured stiffness properties of the timber deck material and on the measured total weight of the deck including the pre-stressing bars. The measured mode shapes is plotted in Fig. 14, while the results obtained from the numerical simulation is given in Fig. 15. The two figures should not be

directly compared as the experimental results are based on the experimental grid which does not fully cover the experimental model. However, as it may be observed, there is a high degree of similarity for the three lowermost modes. Only the lowest vibrational mode was measured in the transverse horizontal direction. The measured and numerically obtained results are given in Table 1. Note also that the damping can be characterized by these types of experiments.



(a) (b) (c)

Fig. 14 Measured vibrational modes in vertical direction, experimental model of timber deck.

(a) (b) (c)

Fig. 15 Numerically obtained vibrational modes in vertical direction of timber deck

Table 1 Measured damping, modes and frequencies compared to numerically obtained frequencies

Mode	**Measured damping [%]**	**Measured frequency [Hz]**	**Numerical frequency [Hz]**	**Deviation, frequency [%]**
Vertical 1	4.2	3.0	3.2	5.8
Vertical 2	0.63	8.0	7.5	6.3
Vertical 3	0.95	17.5	16.9	3.3
Horizontal 1	2.4	17.8	18.2	2.2

From the above results it can be concluded that the stress-laminated deck behaves like a massive wooden block (with additional mass from the steel bars) with respect to dynamic properties, given that the pre-stressing is sufficient.

5.2 Scaled models of the bridge

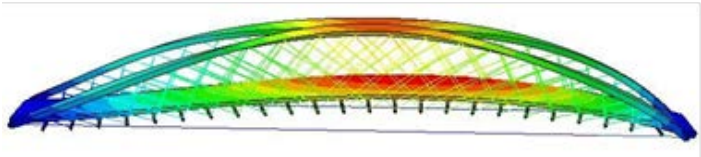
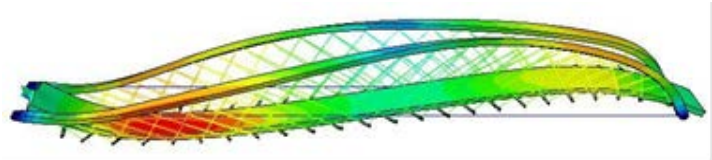
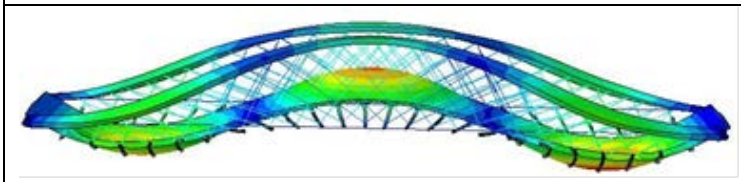
5.2.1 Vertical vibrations of the deck

After the deck was mounted and the hangers attached to the deck and arches, similar experiments as performed on the deck were also done here. In the impact hammer experiments only the deck were impacted using the same grid as for the deck alone. For the results presented in Table 2 the deck was impacted in vertical direction and only vertical deformations were registered. The evaluations of the measurements are based on an FFT transformation of the load and response, leading to frequency response functions. It should also be added that this measurement method is dependent on a certain minimum response and in this case the response corresponding to the lowest mode (mode 1) was almost absent. In the frequency range corresponding to the lowest vertical mode of the deck, the frequency response function was almost flat and hence no corresponding frequency could be extracted. Nevertheless, from the numerical models a mode 1 frequency could be determined, but more information is needed to evaluate the sensitivity of the structure to mode 1.

Axial vibrations are independent of scaling, while vibrations due to bending will be scaled with a factor inverse proportional to the geometric scaling factor. If the measured frequency in a 1:10 scaled model equals 1.0, only 1/10 of that frequency will occur in the full scale model for the bending case. As observed from the two rightmost columns of Table 2, the modes which to a large extent are dependent on bending of the deck are quite close to this. It should also be noted that the current full scale numerical model has nominal material data, while the scaled numerical model is based on measured material data from the experimental model. The influence from connections on scaling of model results is very uncertain, but considering the quite close resemblance between the experimental and the numerical scaled model, it seems like this effect is small here.

Based on these results it is concluded that no problems with vertical vibrations for this structure is expected as the fundamental frequency is quite high.

Table 2 Vibrational modes with vertical vibrations of the deck

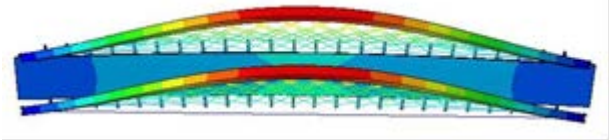
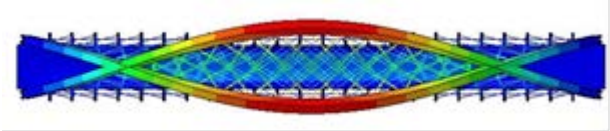
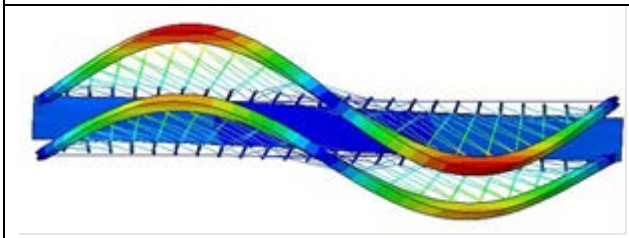
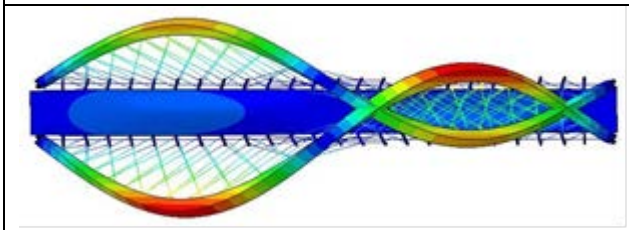
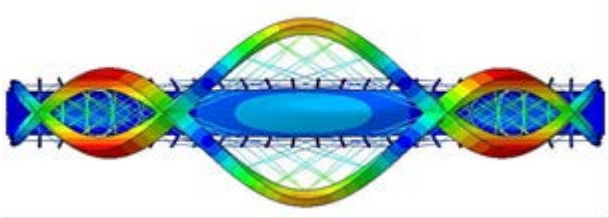
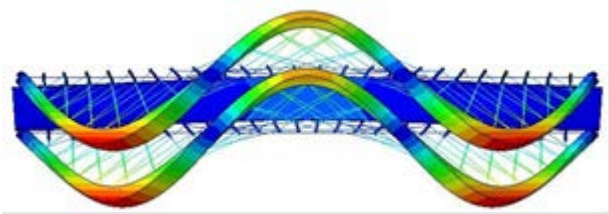
Mode shapes with deck vibrating in vertical direction	Mode	Measured experimental model 1:10 Frequency [Hz]	Numerical model scale 1:10 Frequency [Hz]	Numerical full scale(1:1) Frequency [Hz]
	1	none	26.5	2.95
	2	28.5	24.7	2.27
	3	43.5	42.2	3.99

5.2.2 Horizontal vibrations

A special evaluation of the deck was performed in order to evaluate the effect from the transverse beams, the hangers and their attachments, as well as the end support. Here, the deck was impacted on the side face in horizontal direction and the response was collected solely from the deck in the horizontal directions as well. Similar numerical simulations were also performed. The results are given as mode 1 in Table 3. More mass has been added and as expected the fundamental frequency for the horizontal mode of the deck has been reduced from about 18 to 16, and excellent agreement between the numerical scaled model and the test is achieved. From the full scale numerical simulation it is observed that the frequency decrease less for the higher modes than expected from bending response. Maybe some effects from the hangers come into play here.

Further evaluations of the mode shapes and eigenfrequencies have been based on numerical simulations. The results are visualized and quantified in Table 3. Although six modes of vibration, greatly exaggerated on the plots, are shown in Table 3, the difference between mode a and b is just dependent whether the arches are in-phase or out-of-phase relative to each other. All these three modes show a local bending vibration of the arches in a sideway manner, but almost no horizontal movements of the deck. It is therefore concluded that this vibration will not cause any problematic accelerations of the deck or the live load. However, as the frequencies are so low that wind-induced loading might give dynamic effects and this should be investigated further.

Table 3 Vibrational modes mainly in horizontal direction

Mode shapes with vibrations mainly in horizontal direction	Mode no	Measured experimental model 1:10 Frequency [Hz]	Numerical model scale 1:10 Frequency [Hz]	Numerical full scale(1:1) Frequency [Hz]
Horizontal deck impact	1	15.9 – 16.4	15.9	1.98
	1a		8.66	.809
	1b		9.29	0.814
	2a		16.83	1.787
	2b		16.86	1.802
	3a		32.18	3.66
	3b		32.59	3.67

6. Conclusive remarks and acknowledgements

The network arch bridge is a very efficient bridge concept which may enhance the competitiveness of timber bridges. A network arch bridge becomes very stiff in the plane of the arches. It is also possible to use this concept to build long bridges without the need for truss-work for wind forces or stability, by using hangers in a spoked configuration, i.e. hangers which are inclined both in the plane of the arches and in the transvers direction. The paper has presented some work on a prototype of this concept with promising results. More work on the concept is underway.

This work has been made possible by a project grant received from The Research Council of Norway (208052) and financial and technical support from The Association of Norwegian Glulam Producers, Skogtiltaksfondet and Norwegian Public Road Authorities. These contributions are gratefully acknowledged.

7. References

[1] Bell K. “Structural systems for glulam arch bridges”, *Proceedings of the International Conference on Timber Bridges (ITCB2010)*, Lillehammer, Norway, 2010, pp. 49–66.

[2] SWECO NORGE AS, “Brosjyre om trebruer (In Norwegian: Brochure about timber bridges)”, available from *http://www.sweco.no/no/Norway/Markedsomraader/Infrastruktur/Bruer/*.

[3] Kleppe O. “Durability of Norwegian timber bridges”, *Proceedings of the International Conference on Timber Bridges (ITCB2010)*, Lillehammer, Norway, 2010, pp. 157–168.

[4] Pousette A. and Sandberg K. “Outdoor tests on beams and columns”, *Proceedings of the International Conference on Timber Bridges (ITCB2010)*, Lillehammer, Norway, 2010, pp. 169–178.

[5] Nielsen, O. F. “Foranderlige Systemer med anvendelse på buer med skraatstillede Hængestenger.” (“Discontinuous systems used on arches with inclined hangers”, in Danish.) 121 pages. *Gad Copenhagen. Ph.D. thesis*, 1929.

[6] Tveit, P. “Bogebruer med skrå krysstilte hengestenger.” (“Arch bridges with inclined intersecting hangers,” in Norwegian.) *Ph.D. thesis presented at the Tech. Univ. of Norway* 1959.

[7] Tveit, P. (1987). ”Considerations for Design of Network Arches.” *J. Struct. Eng.*, 113(10), 2189–2207, 1987.

[8] Barli R. and Hakvåg I., *Conceptual Study of Long Arched Timber Bridges. Experimental and Numerical Modelling*, Master thesis, NTNU, Trondheim, 2013.

[9] Simulia. Abaqus 6.11 Documentation. 2012.

[10] NS-EN 1995-2. Eurocode 5: Design of timber Structures. Part 2: Bridges. Standard Norge, 2005.

[11] NS-EN 1995-1-1. Eurocode 5: Design of timber Structures. Part 1-1: General. Common rules and rules for buildings. Standard Norge, 2005.

[12] NS-EN 1991-2 Eurocode 1: Actions on structures - Part 1-2: Action on structures Traffic load on bridges. CEN Standard Norge, 2003.

[13] NS-EN 1991-2 Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions. CEN Standard Norge, 2003.

[14] Malo K. A., “On Connections for Timber Bridges”, *Proceedings of the International Conference on Timber Bridges (ITCB2010)*, Lillehammer, Norway, 2010, pp. 297–312.

ID 63 - New timber bridges- Inventive design by Block-Gluing

Frank MIEBACH
Engineer (Dipl.-Ing. FH)
Ingenieurbüro Miebach
D-Lohmar, Germany
info@ib-miebach.de

CEO Schaffitzel + Miebach,
Faszination Brücken GmbH
D-Lohmar, Germany

1996-2000 timber engineers studies Rosenheim, Germany
2000-2005 Project manager timber bridges Schaffitzel GmbH
2005 Founding Ingenieurbüro Miebach in Cologne, Germany
2009 CEO of Schaffitzel + Miebach GmbH,
Since 2009 executive committee member of quality association timber bridges, Bonn Germany.

Summary

The technology of gluing in wood construction has evolved considerably in recent years. This has been shown especially in timber bridges that Glulam is now the main building material. A further development is the so-called block gluing, which provides a good basis for supporting structures. Numerous bridges, especially in central Europe, appeal by unique design and monolithic and solid construction. These structures base on two main developments: block lamination of glulam and the composite of timber and concrete to one structural system.

Keywords: Block-Gluing, Glulam, Timber-Concrete-Composite.

Introduction

1.1 History in Europe after industrialization

The history of bridge construction has always been connected to the technical state of the art. It started with applications of natural material as stones and wood. During the industrialization construction materials changed to steel and afterwards to concrete and high developed wood constructions got displaced by concrete or steel structures. Due to a mind change to ecological matters there is a rediscovery of wooden structures in the last decade. And the characteristics of wood –light but strong, easy to process – push this natural material into focus of architectural interests.

1.2 Retarding in evolution

Timber constructions have their own industrialization, but with some retardation. In 1906 a German Otto Hetzer patented the gluing of wooden beams. The modern glue-lamination was born. Glulam is a structural timber product consisting of number of layers glued together to a certain cross section. Additionally it is possible to block laminate a number of glulam beams together to achieve bigger cross sections.

This invention provides the possibility to produce wooden beams with huge dimension in a constant quality: length of 50m and more, width and heights of 3m and more. The gluing process even offers the possibility to produce curved elements or even twisted elements. This allows thinking and constructing in free forms.

Since 40 – 50 years glulam is used in halls – not just for the roof but also for the posts – and reaches a respectable market share by now. Wooden bridges are on the rise since almost 15 – 20 years.

There is a diversity of reasons for using of timber in bridges. For example, prefabricated large-sized parts are easy to handle and assemble because of their lower weight - a well-known advantage for economical constructions.

The durability of the construction often is discussed seriously because latest planning often does not consider historic knowledge as constructive wooden protection.

The development of gluing technology including block lamination is a type of the retarded timber industrialization.

2. Block laminated structures

Block lamination generates massive timber cross section to achieve loadbearing structures for large spans with a thin side view. Block lamination comprises three main steps: limitless gluing of lamellas through finger jointing, gluing layers of lamellas to a glulam beam and finally gluing numbers of beams to a massive block.

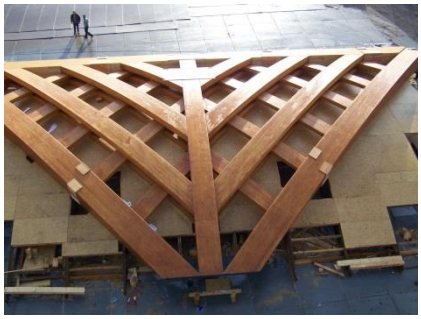
A certain form like a curve is created in step 2 “lamination of layers”. The curve is always vertical to the longitudinal axis because the layers only are flexible according to the thickness. Therefore smaller radiuses require thinner lamellas.

According to the required form a block laminated beam may have a curvature in top view or in side view. The difference in production is

a) Horizontal block lamination (curvature in side view)

or b) vertical block lamination (curvature in top view).

Even twisted forms are possible. Twisted beams are curved around two axes. The first curvature is generated in step 2 “lamination of layers” as usual. Afterward a horizontal band saw divides the curved beams into thin layers. These thin layers are sufficient flexible to be curved again in a second direction during block lamination.



A part from curved gluing, through block lamination stepped beams are feasible both in cross section and in longitudinal section. There for glulam beams with different height or length are block-laminated to one block beam.

3. Timber-concrete-composite bridges

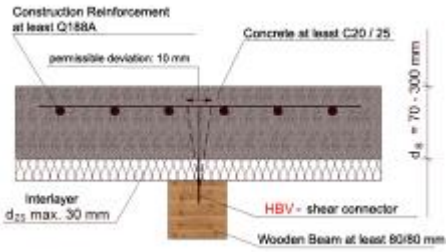
The increasing role of ecology and sustainability of building materials more and more influences construction industry and leads to a rethink on site of the builders. The symbiosis between wood and concrete has the best conditions to meet all resulting claims. Constructions of massive wood beams statically connected to a concrete slab on the upper side ensuring optimum utilization of the material specifications of both materials. In this case the wooden cross-section is considered to take tensile forces and the concrete slab takes the pressure forces. Special connectors ensure the interaction of timber beams and concrete slab to get a more effective loading capacity and serviceability.

Advantages in comparison to conventional timber bridges:
- higher load capacity with lower height of construction
- good structural wood protection through cantilevered concrete slab on the top side
- optimal load spreading of point loads by the concrete slab
- better cross bracing
- use of proven details in connections to the concrete

Advantages in comparison to conventional concrete bridges:
- lower weight of the superstructure and thus more efficient structure
- fast and efficient installation with high degree of prefabrication without extensive formwork
- cost savings in foundation and the abutment
- improved energy balance and eco-balance, sustainability through CO^2 reduction

3.1 HBV-shear connector

The HBV-shear connector by TiComTec (Haibach - GER) is an expanded metal part that is glued perpendicularly into the wooden structure. The dimension and number of connectors is determined to the static needs. The concrete slab usually has a thickness of at least 20 cm and has several functions: road decking, carrier plate for the dispersal of transverse loads and constructional wood protection.



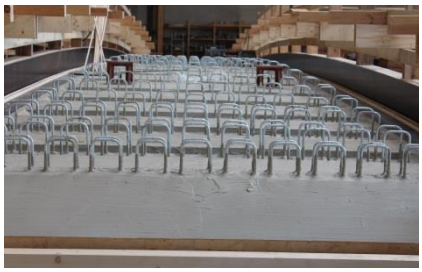
© Ticomtec

3.2 Head bolts

As known from steel-concrete bond structure in this alternative the connecting parts between wood and concrete are steel bars with welded head bolts. The bars fit exactly into milled kerfs and are fixed with screws. To achieve an efficient utilization, the axial distances of the dowel bars correspond to the traffic load and transverse force caused by the traffic load.

3.3 Kerfs and Glued in reinforcement bars

The shear connection in this type of timber concrete composite is implemented by a combination of kerfs and glued-in steel bars. The kerfs are milled in transverse direction and transfer shear forces (from longitudinal direction) into compressive forces to the vertical sides of the kerfs. To avoid a lifting of the concrete additional fasteners or connectors are considered (e.g. glued in reinforcement bars).



4. Conclusion

A changed awareness on ecological matters is the proper basis for timber bridges. Timber is the one and only material that saves and stores CO2 permanently. And the technical possibilities are still growing. For example, researches on modern glues and gluing methods show in near future glues will be more efficient (temperature-resistant and the level of bonding pressure during the gluing process will be halved from 0,4 N/mm2 to 0,2 N/mm2). Against this background the combination of block lamination and timber-concrete-composite is the beginning of the latest development: The comeback of timber bridges as adequate road bridges.

Reference list:

1) Dipl.-Ing. Antje Simon, Weimar 2008
„Analyse zum Trag- und Verformungsverhalten von Straßenbrücken in Holz-Beton-Verbundbauweise“;

2) Matthias Gerold, Bruderverlag 2001
„Holzbrücken am Weg“

3) Dipl.-Ing. Frank Miebach, in Brückenbau 03/2012
„Holzbrückenbau 3.0 als Weiterentwicklung“

4) Dipl.-Ing. Richard J. Dietrich, Verlag Georg D.W. Callwey GmbH & Co. 1998
„Faszination Brücken“

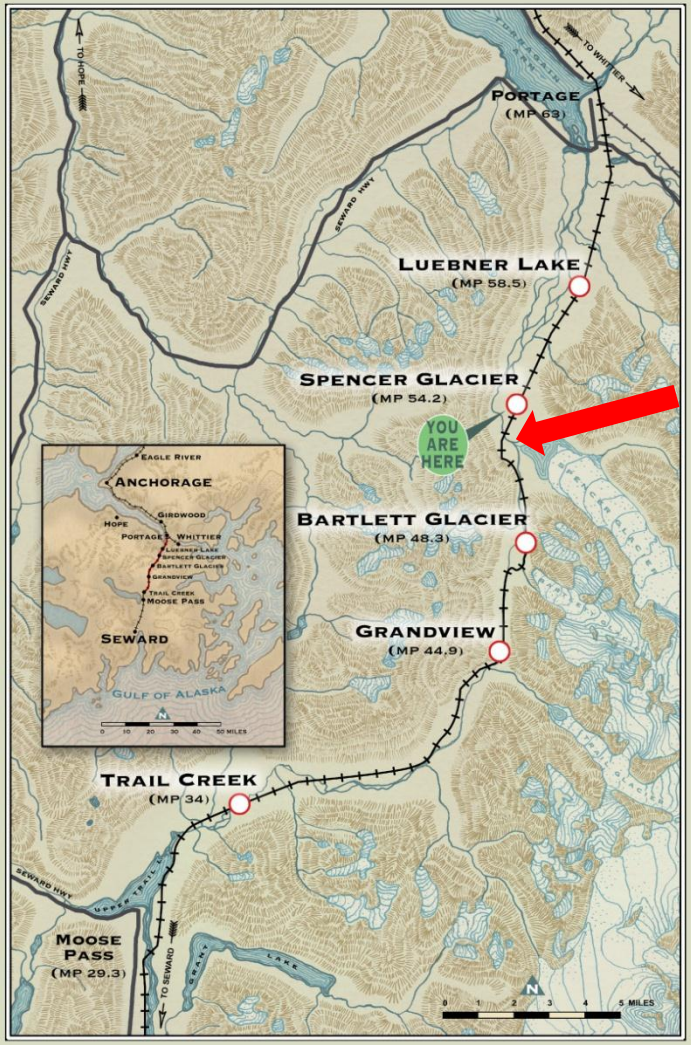
OVERVIEW OF THE PLACER RIVER TIMBER BRIDGE PROJECT IN ALASKA

Eleanor Oman, PE
Rod Dell'Andrea, PE
USDA Forest Service

07/26/2012

All Aboard!

Chugach National Forest Whistle Stops



The Forest Service and the Alaska Railroad invite you to explore the Alaska backcountry. As the trail system and five planned whistle stop stations are developed, you will have the opportunity to explore the glaciated terrain in a day hike, or strike out into the Kenai Mountains for a multi-day adventure. The following is a list of some of the activities planned for each whistle stop.

Luebner Lake
(planned):

Follow a boardwalk trail to Luebner Lake for outstanding wildlife viewing opportunities. Begin your multi-day trips here at the northern end of the Glacier Discovery Trail.

Spencer Glacier

Experience the awe-inspiring power of glaciers on the landscape. Spend a few hours walking out to view the glacier and icebergs. Take a longer time and hike to the base of Spencer Glacier. Or embark on a backpacking trip into the heart of the Kenai Mountains.

Bartlett Glacier
(planned):

View the northern reaches of the Placer River Valley, whose beauty disguises the difficult engineering feat needed to put a railroad through the area. Hike the trail to Bartlett Glacier, the shortest hike to 'blue ice' along the Whistle Stop route.

Grandview
(planned)

Enjoy the rich history and majestic views of this railroad outpost while hiking the Grandview Loop Trail. Trail Glacier is a four-mile hike, while Spencer Lake can be reached by following the Glacier Discovery Trail.

Trail Creek
(planned)

This whistle stop connects with the Iditarod National Historic Trail. This trail was once the main transportation route used by early natives and settlers not only on the Kenai Peninsula but all the way to Nome.

09/06/2012

Spencer Glacier
Whistle Stop

Plan Ahead!

Leave yourself plenty of time to make it back to this whistle stop to meet your train.

Distance from Whistle Stop

Group Campsite	1.1 miles
Glacier Viewing Area	1.3 miles
Glacier Overlook (End of Trail)	3.1 miles

LEGEND

- Spencer Glacier and Bench Trails
- Alaska Railroad
- Dispersed Camping (open)
- Spencer Developed Recreation Area (camping in designated sites only)
- Tent Pads for Dispersed Camping (No fee, no reservation required)
- Group Campsite (Fee site, Reservations required)
- Restrooms
- Wheelchair Accessable
- Scenic Viewpoint
- Boat Launch

SCALE IN MILES

0 .25 .5

Look Around

You are standing on glaciated terrain. Less than 100 years ago, this very spot was underneath Spencer Glacier. As the glacier melted and retreated over the past century, it deposited these gravels. The Placer River worked the gravels, forming the broad outwash plain in front of the glacier and lake.

07/26/2012

WATCH YOUR ST
PLEASE USE HANDR
TRUCK CUTOUT
08/15/2011

Thomas
10/25/2011

08/16/2012

10/25/2012

08.30.2011

09.06.2011

09. 19. 2011

09/29/2011

09.30.2011

DEERE
07/24/2012

08/15/2012

S175
08/24/2012

10/24/2012

10/24/2012

General Contractor:

- Patrick Albin Carlson, a JV
- Project Manager Neil Hunt

Sub Contractors:

- Goode Construction
- Johnson Construction

Superstructure Design

- Paul Gilham, Western Wood Structures

Substructure Design

- R&M Engineering

Thank you

The Bartonsville Covered Bridge Replacement

Phillip Pierce, P.E.
Associate, Sr. Prin. Engr.
CHA Consulting, Inc.
Albany, NY
ppierce@chacompanies.com

BSCE, ME, F ASCE;
40+ years experience;
Bridge engineering – all types – movable, fixed, pedestrian; all aspects;
Private and public sector employment;
Focus on historical projects;
Nationally recognized covered bridge expert.

Summary

The destruction of the 140 year old Bartonsville Covered Bridge in the Town of Rockingham, Vermont by flood waters of Hurricane Irene not only forced a lengthy detour, it tore at the heartstrings of the local community and covered bridge lovers everywhere. A youtube video of its destruction instantly went viral and much attention by the press followed developments during its replacement. A replacement bridge was needed as quickly as possible, but it needed to honor and generally mimic the destroyed bridge. Abutment replacement commenced immediately with design support by a local consultant. CHA was invited, and quickly underway, to prepare bid documents for a new covered bridge, built for another century of service. At 51.2m (168 ft) between end joints, the new superstructure is the longest single-span covered bridge supported by Town lattice trusses in the world and was opened to acclaim on January 26, 2013. This presentation/paper discusses the design and construction of the replacement covered bridge using traditional materials and details and how it incorporated attention to the Secretary's Standards for Historic Preservation in the design and detailing.

Keywords: timber, covered, bridge.

The historic Bartonsville Covered Bridge, built in 1870 and one of the longest single span covered bridges in the United States, was destroyed by the flooding caused by Hurricane Irene in August 2011, when the west abutment was scoured away from under the bridge – see Figures 1 and 2.

Figure 1 - The Bartonsville Covered Bridge just prior to its destruction (photo provided by Susan Hammond)

A 12.9km (8-mile) one-way detour made prompt replacement of the bridge imperative. However, the fact that it was an historic timber covered bridge was an issue. The local residents wanted a similar bridge to replace it, but recognized that added cost would be a factor.

The replacement effort had to commence immediately, starting with the construction of new abutments to accommodate whatever superstructure was ultimately approved. A local consulting firm was authorized to start a geotechnical exploration in short order. That same firm also began designing the abutments based on initial estimated loads for several types of superstructures. A timber covered bridge would obviously be the heaviest and require the longest supporting length.

Figure 2 - Beyond salvage (photo provided by Susan Hammond)

Based on a past relationship involving an investigation and refined analysis of the destroyed covered bridge, the Town of Rockingham, VT, who owned the bridge, invited CHA to provide design services for the replacement superstructure. We were under contract within a month.

Because the weight of a timber covered bridge is substantially more than more modern bridges, and because the design of the abutments needed to proceed quickly, it was very important that a realistic estimate of weight of the timber bridge be prepared in advance of its design. CHA's strong background with covered bridge work enabled development of such an estimate, which turned out to be very close to the final.

Additionally, while developing the geometry and location of the abutments was the responsibility of the other consultant, a professional informal teaming effort between the two firms led CHA to offer key counsel on the abutment design. Limitations on positioning the abutments farther back from the stream (due to immediate proximity of an active railroad line), coupled with the unusually long bearing seat needed for a timber covered bridge, led to adoption of a rather unique concrete ledge designed to cantilever past the face of the abutment stem. This concrete ledge, acting in concert with traditional timber bolster beams, enabled the rear of the timber superstructure to be much closer to the front face of the abutment than would normally be possible, thereby providing the necessary hydraulic opening for flood waters.

The contract for construction of the abutments was let and contractor was on site by the first of December 2011 and the temporary bridge was open to traffic by March 12, 2012. The cost of abutments and temporary bridge installation was about $850,000. The temporary bridge was rented directly by the Town at a cost of $2000/week.

The first task for CHA was to prepare construction cost estimates for a "vanilla" bridge for comparison against a timber covered bridge using traditional materials and details. The initial estimates demonstrated that a traditional timber covered bridge would cost only slightly more (approximately 10%) than a more modern/traditional bridge and the community was prepared to pay the difference.

The replacement bridge utilizes the same Town lattice configuration of trusses - so named after Ithiel Town, whom patented the configuration in 1820. The single level of top chord elements, as well as trim details of portals and windows, closely simulates the earlier bridge; these were used to address historic preservation principles. Further, the unusual double intersection top lateral system configuration was retained. Glue-laminated floorbeams were intentionally used as substitutes for solid sawn floorbeams used in the original covered bridge to provide extra capacity and long service life, in keeping with provisions of the Secretary of the Interior's – Standards for the Treatment of Historic Properties when use of different materials (albeit still wood) are required. Unfortunately, the remains of the destroyed timber superstructure were twisted and broken to such an extent that they were of no practical use in the replacement structure.

Figure 3 - Replacement timber bridge being built on a work platform bridge. The temporary vehicular bridge is at left in this photo. They were racing to beat bad weather, but looking good.

The new bridge was made 5.2m (17 feet) longer than the original structure to provide an increase in its hydraulic opening. The new superstructure (51.2m [168 ft] at deck level and 54.3m [178 ft] overall) will become the longest single-span covered bridge in the United States (if not the world) supported by Town lattice trusses. Figure 3 provides and image of the skeleton of the bridge showing construction of the Town lattice trusses with partial completion of overhead bracing and partial installation of floor beams. The timber structure is being erected on a work platform bridge alongside the temporary bridge, which sit atop the replacement permanent abutments.

In keeping with common practice in Vermont for bridges with timber decks, the bridge had been posted with a 7,260kg (16,000 pound) vehicular weight limit. The town intends to maintain the same load posting on the replacement bridge. As to the "design" weight vehicle, it was decided to use a 27.2t (30-ton) two axle vehicle (aka H30 per national bridge design codes), compared with 18.1t (20 ton) or even lighter vehicles often used for such structures. The heavier design vehicle will provide reserve capacity for unauthorized overweight vehicles and extend the life of the structure, as typical everyday stresses will be significantly lower than design allowables.

The sizing of the elements of the highly indeterminate trusses and verification of stresses in the solid sawn members and wood trunnels represented significant challenges. These were overcome by developing a first order three-dimensional finite element model of the six vertical planes of truss components.

The community desired retaining the original bridge opening to deter use by oversize vehicles. However, the extra length and desire for ample reserve capacity for potential overweight vehicles led to accepting a truss height increase of 0.6m (two feet). That extra vertical height remains hidden behind the portal entrance and is not noticeable to lay people traveling through the bridge.

The increase in truss height and element sizes led to a corresponding increase in the spacing of the lattice from 1220mm (4'-0") of the destroyed bridge to 1370mm (4'-6")for the replacement bridge. Primary bottom chord and top chord elements were increased from 75x305's to 100x355's (3x12's to 4x14's). The upper bottom chord (aka secondary bottom chord) was increased from 75x305's to 100x355's (3x12's to 4x12's). The maximum length of chord sticks was increased from 4.9m to 12.3m (16'-0" to 40'-6") to provide better load distribution among the chord elements. All lattice elements were made 75x355 (3x14) [original 75x305 (3x12)] to accommodate connections with the primary bottom chords using four 50mm (2") diameter traditional oak trunnels. Three 50mm (2") diameter trunnels were used in all other truss element connections.

The design originally called for Select Structural Southern Pine for primary truss elements to take advantage of its slightly higher bending strength than Douglas Fir. However, concern over the dimensional stability of such large pine elements during their drying led us to ultimately specify Douglas Fir.

Many original covered bridges supported by Town lattice trusses used floorbeams spaced to match the lattice spacing, and were "threaded" through the lattice openings to allow support by both the inner and outer pairs of lower bottom chord elements. In this case, the size of the truss elements prevented that option. The floorbeams therefore are only supported by the inner pair of chords.

Accordingly, the bending stresses were higher than usual in the chord, as were the shear stress in the trunnels.

Since almost all floors have been replaced in historic covered bridges, we felt that it was tolerable to include some improvements in the floor system. Based on the floor design details of the nearby Hall Covered Bridge, drainage/ventilation gaps were provided along the curbs, but with fills above the floorbeams and bottom lateral system to prevent drainage from falling directly atop those critical elements.

Extra ventilation was also provided around the truss elements by spacing timbers to avoid direct contact of the siding and the truss chords. Additionally, shiplap siding was used to protect against rainfall penetration.

The trusses are supported on glulam bolster beams made from pretreated lumber to promote complete preservative penetration. Floorbeams were made in a similar fashion. The bolster beams are supported by glulam bearing blocks atop raised concrete strip pedestals.

As noted above, the top lateral system is comprised of double intersecting elements which are connected to the tie beams with mortise and tenon connections held tight by opposing wedges. Kneebraces are bolted through lattice intersections and are connected to the tie beams in notches with bolts. The bottom lateral system is comprised of single intersection timbers and held tight with transverse tie rods.

Metal roofing was used on the destroyed bridge. It was also specified for the replacement bridge, as opposed to wood shingles (preferred by many purists inasmuch as wood shingles were typical on original covered bridges), in large part due to the designer's strong belief that metal sheds snow loads much faster, thereby lessening long term loading on the bridge.

Wood preservative treatment was specified for the 100mm (4") thick Douglas fir deck planks and the timber curbs. It was not specified for the truss elements for a number of reasons, including:

- Douglas fir elements must be incised to provide uniform retention of preservative treatment. US design codes specify a 15 percent reduction in strength due to the incising – a critical loss of strength.
- Preservative treatment would have required an additional month to obtain materials. This was deemed too long given the urgency of the project.
- Preservative treatment would have added an extra $40,000 to the cost of the project. The designer was not convinced that it was a necessary or cost effective investment – many historic covered bridges have survived for more than 150 years without preservative treatment.

Figure 4 - Bridge being moved sideways into position atop the permanent abutments (with a bit of good-natured help from the owner of the construction company).

Figure 5 – A happy crowd turnout for the ribbon cutting.

Discussion and Conclusions

Design and construction of a timber bridge based on traditional materials and connections, with an anticipated service life of well beyond 100 years demonstrates that new timber covered bridges can handle modern loads and still resemble historic structures. And for bridges at sites such as this, where single lane service continues as it has for more than 140 years, this type of construction represents excellent cost/benefit value to the community.

Replacement of the destroyed bridge with one so similar to it, continues Vermont's long tradition of staunchly supporting its precious population of covered bridges. A more "modern" bridge of steel was possible and would have cost slightly less on a first-cost basis, but there was no hesitation of the community to commit to the slight increase in cost in order to obtain a timber replacement. This bridge received media attention across the world throughout the process of its replacement and the community continues to derive financial benefit via the thousands of tourists who roam Vermont's countryside each year visited these structures.

It is interesting to note that the contractor, who won separate bids, built the abutments and installed a temporary Mabey pony truss superstructure. Then the permanent replacement timber covered bridge was built atop a work platform bridge, placed alongside the temporary bridge – thus three bridges to yield one. Construction of the $1.2M superstructure was completed with ribbon cutting on January 26, 2013.

Acknowledgements:

Owner - The Town of Rockingham, VT – what a wonderful client

Substructure Designer - Eckman Engineering, LLC – a cooperative effort between two engineering firms with no contractual relationship

Contractor - Cold River Bridges, LLC – a contractor who put their heart and soul into this bridge

The community of Lower Bartonsville – we would not have had a project without them

Glulam beams and columns after 5 years exposure to outdoor climate

Anna POUSETTE
Researcher
SP Technical Research Institute of Sweden/ Wood Technology
Skellefteå, Sweden
anna.pousette@sp.se

Anna Pousette, M.Sc., Lic.Tech, has been working with timber bridges since 1993, with design and construction and also with inspections and inspection methods for timber bridges and other outdoor structures such as facades and decking..

Karin SANDBERG
Researcher
SP Technical Research Institute of Sweden/ Wood Technology
Skellefteå, Sweden
karin.sandberg@sp.se

Karin Sandberg, PhD, has been working with outdoor constructions such as windows, facades and decking, measurements of water absorption and desorption with CT scanning.

Summary

A field test started in 2007 to increase the knowledge about cracks and moisture in wooden structures in outdoor climate. The test included wooden beams and columns in two test fields in Sweden. Measurements and tests were made every summer in 2008-2012. In the current compilation of test results the main focus was to answer questions about the relationship between cracks and moisture contents and the influence of different surface treatments. The intention was to give directions for the assessment of the risk of cracks, rot or reduced bearing capacity. This paper includes parts of the collected data, and presents moisture contents and cracks in long beams and columns. The data give an overview of the development of cracks and moisture during 5 years. Surface treatment, wood material and impregnation give different amount of cracks. There are also significant differences between the north and south sides for some beam types.

Keywords: timber bridge, timber beam, glulam beam, checks, cracks, durability, moisture content, surface treatment, field test

1. Introduction

1.1 Service life of exterior wood

Long life and low life cycle costs are increasingly in demand in planning and construction. The ability of a structure to resist degradation by climate is therefore essential for any outdoor structures. It depends on the type of material and design but also on the maintenance. The durability of wooden structures depends on the wood material, impregnation, surface treatment, moisture content, temperature, and mechanical impact.

Extending the life of existing structures can be economical, but only if it is more profitable to repair and maintain than to demolish and rebuild. Much research has been done on sustainability of exterior wood and glue-laminated timber [1] but it is not easy to determine the rate of degradation and the remaining life for various structures and environments. Timber bridges can be constructed in many ways [2]. Structural parts can be provided with claddings to protect against the impact of the climate, but sometimes the designer wants to avoid claddings because of costs or aesthetics. The exposed wood can then be protected with impregnation and surface treatment. Cracks are often

detected during inspections of these impregnated and painted beams and columns [3], [4].

1.2 Cracks in exterior wooden parts

Glulam beams often get cracks on the surface, and the cracks usually occur at or near the glue lines between the lamellas. Some small cracks in the wood can be considered a natural part of the wood, as wood is hygroscopic and has the ability to take in and give off moisture. Below the fibre saturation point, about 30% moisture content, wood shrinks when it is losing moisture and swells when moisture is gained. These moisture movements are different in different directions. For Swedish spruce and pine the movements are tangentially twice radially, which in turn is more than ten times greater than in the longitudinal direction.

Varying humidity due to annual fluctuations will affect outdoor wood. At low humidity the outside of the wood will dry out faster than the inside, and tensions will occur in the wood as shrinkage of the outer part is prevented by the moist interior. Consequently, cracks occur if these stresses exceed the strength of the wood. Surface cracks that are small can later be closed if the surface gets wet and they will be difficult to detect. But they still exist and can contribute to cracks in the paint.

Cracks in the laminations of glulam beams usually occur parallel to the fibers and along the fibers around knots or at growth disturbances [5]. Strength grading of structural timber limits the widths and lengths of cracks according to EN 14081 [6]. In the manufacture of glulam according to EN 386 [7] the gluing and delamination is controlled. Cracks from drying should not be mixed up with delamination.

Glulam industry in the U.S. has published guidelines for the evaluation of cracks in beams and columns [8], [9]. If beams have end-cracks that go through the beam but are short, they have little effect on the strength. Cracks on beam sides that are deep and long may affect the shear strength, and cracks parallel to the fibers should be observed [10]. The greatest risk of cracks is at the ends of beams where the shear stresses are greatest at the supports with a maximum value in the neutral layer. But the shear stresses are generally not often critical in beams. For columns, the cracks affect the compressive strength if they go through the wood and along the entire column.

1.3 Objectives

The aim of this project is to increase knowledge about cracks and degradation of wood for various factors, thereby providing a basis for lifetime assessments of timber structures in outdoor environment and recommendations for design and inspection of structures made of wood such as wooden bridges, wooden balconies, and wooden columns. This paper presents a compilation of some test results, with the main focus to answer questions about the development of cracks, relationship between cracks and moisture contents and the influence of different surface treatments.

2. Field test with glulam beams and columns

2.1 Beams and columns

This project included studies of beams and columns made of glulam and wood. They had different surface treatments, and factors like south or north side, height above ground and climate were considered. The field test and test objects were described in a Swedish report [11] and in [12].

2.2 Long beams

A total of 60 beams were included in the test. Out of these beams were 20 long beams, and results for these are presented in this paper. There were five beams of each type, see Table 1. The beam size was 140 mm x 450 mm, and the beam length was 9 meters. The length in proportion to the height of the glulam beams was chosen according to EN 408 [13] for tests of modulus of elasticity. Most beams were made from pressure treated lamellas but also one beam type made of spruce was included. The top and end surfaces of the beams were covered with a sheet metal.

Surface treatments were chosen similar to what is used for many timber bridges in Sweden. The wood oil was solvent-based oil, based on a specially treated linseed oil. The paint system gave a thin film that showed the wood structure. It was an alkyd-based stain, fortified with linseed oil. The stain was waterborne and environmentally friendly. It gave coverage, but still showed the grain of the wood. The binder was water-soluble alkyd oil and the stain created a semi-glossy paint film. Pre-treatment was made with an impregnating oil primer used for exterior wood. It was waterborne

and based on modified linseed oil.

Table 1. Beams 140 mm x 450 mm with length 9 meters, in test field Bygdsiljum

Beam type	Wood	Surface treatment	Colour
B1	Glulam, pine, pressure treated	Wood oil	-
B2	Glulam, pine, pressure treated	Paint system	White
B3	Glulam, spruce	Paint system	Red
B4	Glulam, pine, pressure treated	Paint system	Red

2.3 Columns

A total of 52 wooden columns were included in the test, solid wood, glulam, hollow glulam columns so called Comwood and Quattrolit. The 40 columns reported in this paper are described in Table 3. Five columns of each type were tested. All columns were covered with sheet metal or wood on top. Surface treatments were of same types as for the beams.

Table 3. Columns, length two 2 meter, in test field Bygdsiljum

Column type	Support	Wood	Dimension (mm^2)	Surface treatment	Colour
S1	Beam	Glulam, pine, pressure treated	90 x 135	Wood oil	-
S2	Beam	Glulam, spruce	90 x 135	Paint system	White
S3-1	Beam	Glulam, spruce	90 x 135	Paint system	Red
S4	Beam	Glulam, spruce (hollow)	90 x 135	Paint system	Red
S5	Foundation	Comwood, spruce	Diam. 400	Paint system	Red
S6	Foundation	Quattrolit, spruce	110 x 110	Paint system	Red
S7	Foundation	Solid wood, spruce	100 x 100	Paint system	Red
S3-2	Foundation	Glulam, spruce	90 x 135	Paint system	Red

2.4 Test fields

Beams and columns were placed on the test field in Bygdsiljum in Skellefteå in northern Sweden (Lat 64°20′57″N, Long 20°30′14″E). Some beams and columns were also placed in Borås (Lat 57°43′15″N, Long 12°56′24″E). The climate in Borås is warmer and wetter than in Bygdsiljum.

Beams were placed with one side to the sunny and warm south, and the other to the shaded and cooler north. The beams in Bygdsiljum were placed on supporting frames at different heights with the lowest beam approximately 1 m above the ground surface, see Fig. 1. All beams in Borås were placed 1 m above ground level, see Fig. 2. The columns were screwed to glulam beams that were 1 m above the ground, alternatively fastened to a concrete foundation, see Fig. 2.

Fig. 1 Beams, nine meters long, in Bygdsiljum

Fig. 2 Columns and short beams, in Borås

2.5 Measurements

Beams and columns were inspected and documented every summer during five years. The measurement methods were non-destructive. Visual inspections included cracks and other defects. Width, length and depth of the cracks were measured manually with a steel tape, ruler, precision feeler gauge and a crack depth gauge. Cracks with width at least 0.4 mm were measured, small cracks (0.2 to 0.4 mm) were only counted, and cracks less than 0.2 mm were not documented. Locations of the cracks were indicated by x and y coordinates of the starting point. The width was measured on the widest point of the crack. The length was measured as the straight distance between the start and end point of the crack, even if the crack could be slightly bent. This means that the length might be a bit underestimated but the width a bit overestimated. In this paper the amount of cracks is presented as crack area on the surface, i.e. length multiplied by width.

Measurements of moisture contents in long beams and columns were performed manually with fixed pins in the wood and a resistive moisture meter. The pins were placed in the middle of the beams on the south side at the bottom, centre and top, see Fig. 3. There were also pins at the bottom at one end of the beam on the south side and at the other end on the north side. Columns had pins at the bottom and the top on the south side, see Fig. 4.

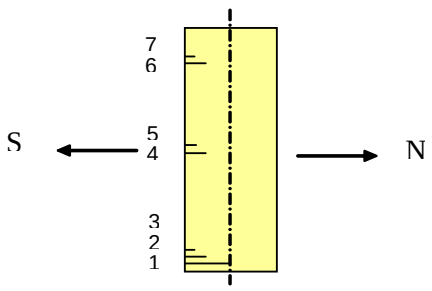


Fig. 3 Moisture contents in beams, pins at depths 10 mm, 35 mm and 70 mm

Fig. 4 Moisture contents in columns, pins at depths 10 mm and 45 mm.

3. Results beams

3.1 Cracks in long beams

Total crack areas during 2007-2012 for each beam type are presented in fig. 5-8, for south and north side. The figures show the average of measured cracks on the five beams of each type. But some beams were not measured every year so the average is sometimes from fewer beams. Beam types B1 and B4 had most cracks and maximum amount on the south side especially beam type B4. Also the other red painted beam type, B3, had many cracks on the south side, but fewer on the north side. The white painted beam type, B2, had little crack area both on the south and north side.

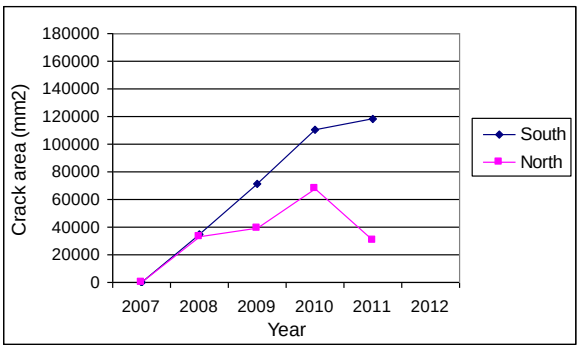


Fig. 5 Crack area (mm^2) in beam type B1, on south and north side, average of five beams (year 2011 only one beam measured)

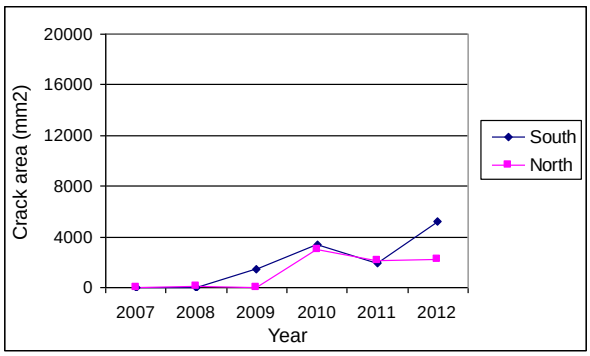


Fig. 6 Crack area (mm^2) in beam type B2, on south and north side, average of five beams

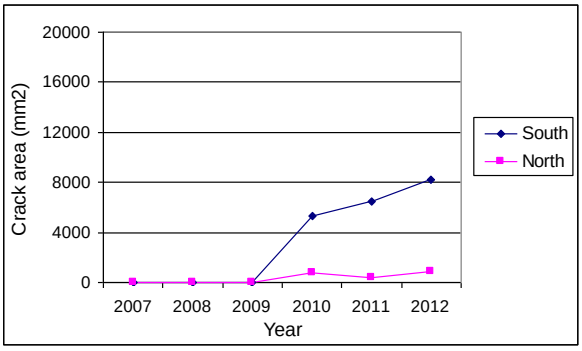


Fig. 7 Crack area (mm^2) in beam type B3, on south and north side, average of five beams

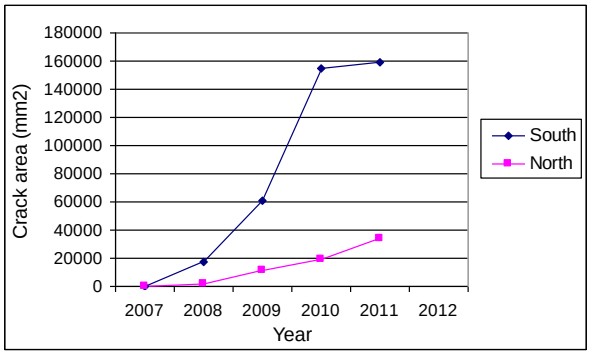


Fig. 8 Crack area (mm^2) in beam type B4, on south and north side, average of five beams

3.2 Moisture contents in long beams

Fig. 9 shows the average moisture contents in beam types B1-B4 during 2007-2012. The values for 2011 are sometimes from fewer beams because several beams had no measurements for 2011. The average values were calculated from the values of all pins 1-7 in the middle of the beams. At the start in 2007 there were some high moisture contents at some pins and variations in moisture were large in all beams. The summers in 2010 and 2011 were dry, but the summer in 2012 was rainier. The beam type B3 (red spruce) had the lowest moisture contents all years.

In Fig. 10 the moisture contents are shown for each pin 1-7 in the middle of beams. These are average values for all years. The pins 3, 5, and 7 had in general the lowest moisture contents, which means the surface was dryer than the inner of the beams. There was little difference between the moisture contents at pins 1 and 2, which means the moisture contents at 35 mm depth and at 70 mm depth were almost the same. The moisture contents were quite similar at top and bottom of the beams.

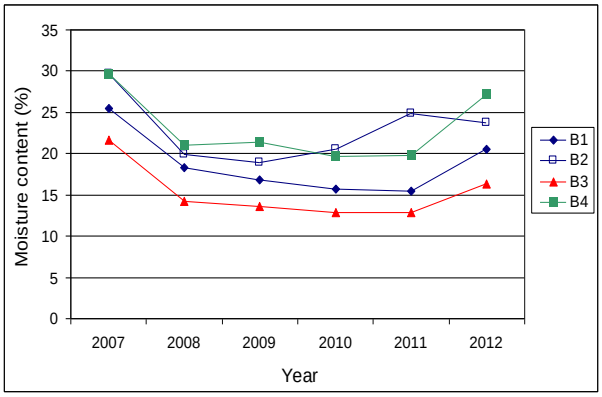


Fig. 9 Moisture contents in beam types B1-B4, average of all pins 1-7 in middle of beam.

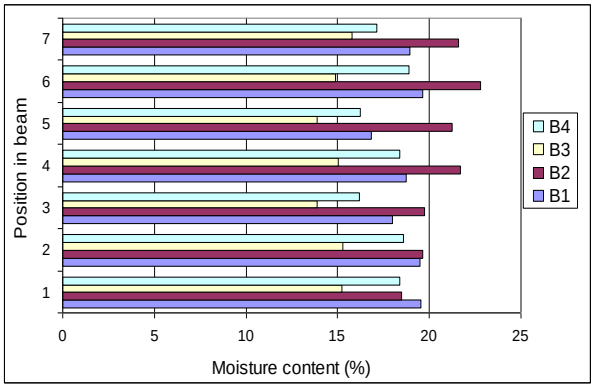


Fig. 10 Average moisture contents at pins 1-7 in middle of beams, in beam types B1-B4.

4. Results columns

4.1 Cracks in columns

Fig. 11 displays the average amount of crack area for the different sides of the columns. The crack area was calculated in the same way as for the beams. Cracks in columns were not measured every year. Fig. 11 shows the maximum measured crack area for each column type, which can be for different years 2009, 2010 or 2012 (no measurements in 2011). The oiled glulam columns had most cracks, except on the north side. Also the Comwood columns had quite a lot of cracks. All the other columns had very little cracks.

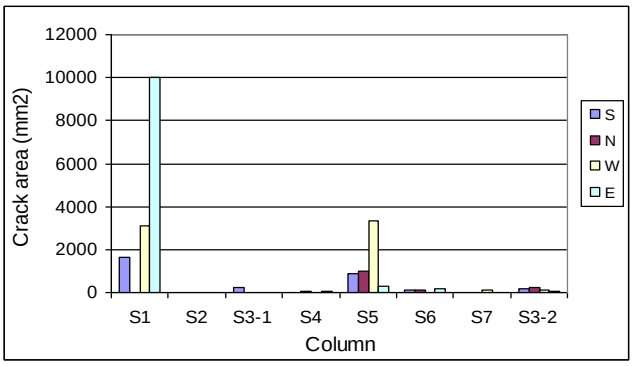


Fig. 11 Crack area (mm^2) for the different sides south (S), north (N), west (W) and East (E)

4.2 Moisture contents in columns

The columns were quite dry when they were placed in the test field, about 12-15 %, see Fig. 12. The moisture contents of the Comwood columns (S5) were not measured. No measurements were made during 2011 and 2012, and until 2010 the moisture contents had not changed very much.

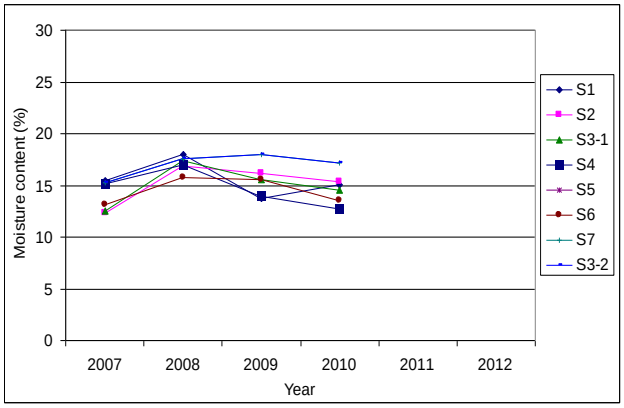


Fig. 12 Moisture contents in the different types of columns

5. Discussion and conclusions

Data presented in this paper give an overview of the development of cracks and moisture contents in glulam beams and columns. There are a lot of collected data, and this paper presents only some of the results from the field test. Test results can be further studied concerning details, such as in which areas the beams mostly crack and the influence of different factors.

After 5 years the crack area in beam type B1 (impregnated, oiled) was about 25 times larger than in B2 (impregnated, white painted) on the south side and about 15 times larger on the north side. Beam type 4 (impregnated, red painted) had crack area in the same order as in type B1. This indicates that the surface treatment made a difference, especially the colour. In the sunshine a light colour will not get as hot as a darker colour, and thus for the white beams the difference in the moisture contents between the surface and the inner of the beams is little.

Beam type B4 (impregnated, red painted) had about 20 times more cracks on the surface than beam type B3 (spruce, painted red) after 5 years. This shows that the wood material is also important and not just the colour. It is the sapwood in pine that is impregnated. Impregnated pine wood that cracked very much probably had more sapwood than spruce. The amount of sapwood and heartwood depends on how the laminations are sawn from the log. This will also influence the

annual ring orientation in the laminations and the beam surface, which could affect the amount of cracks.

The amount of manually measured cracks reported in this paper are presented as crack area. This could also be used if photography and image processing is used as a method to determine the crack amount on the beam surface. For example the oiled beams had after some years very many cracks which was not practical to measure manually and therefore other more effective methods were also investigated, but they are not presented in this paper.

Beams with most cracks had a crack area of about 3-4 % of the total area of the beam surface. After 5 years these relatively small cracks should not significantly reduce the strength of the beams. The maximum measured crack depth was 85 mm in one beam B3, but most cracks of all beams were not deeper than 50 mm, see Fig. 13. The average depths were about 8-12 mm, see Fig. 13. The maximum measured width was 12 mm, but most cracks in all beams were not wider than 2 mm. It should also be taken into account that the depths and widths were only documented with the maximum values, which means all crack areas were overestimated.

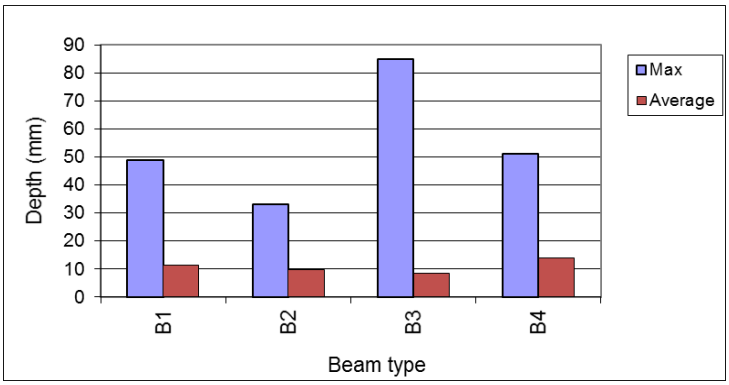


Fig. 13 Maximum and average crack depths (mm) for beam types.

It is difficult to answer the question if cracks give higher moisture contents in the wood. Measured moisture contents were measured in fixed points, which were not always where the cracks occurred. The measured moisture contents therefore mostly gave a general representation of the moisture conditions in the different beams and columns. After 5 years, moisture contents had not increased compared with the first year. During the first year the beams were dried out, as they were quite wet from lying outside in the rain before installation on the test field. The pins were placed in boxes for protection. Some cover boxes were not tight and water penetrated into the wood at the pins. This gave high moisture contents which were not dried out for some beams. After the first year the moisture contents decreased further. During the summer of 2012 the moisture contents were slightly higher again. Probably this was because of the rainy weather.

One side of an oiled beam B1 was filmed during a few days, and when it rained the wood swelled quite quickly and the cracks were closed. This means that not much rain water could enter the beam. When the rain stopped the cracks opened up again and any moisture could dry out. A painted beam has a protective film against the rain and the wood can not react and swell as quickly as an oiled beam. Further studies are needed on this.

For the columns, the moisture contents had not varied much during 5 years and there were few cracks. Probably one reason compared with the beams was the smaller cross sections. The stresses from moisture differences were thus smaller. The oiled columns had more cracks than the other beam types and this indicates that the surface treatment with white or red paint protected against the climate. It would also be interesting to study beams and columns with larger dimensions to see how much the crack development depends on size.

6. Acknowledgements

The authors would like to express their thanks to EU Structural Funds together with TräCentrum Norr, Luleå University of Technology and SP Technical Research Institute of Sweden for providing

funding for this project.

7. References

[1] Pitzner B., Rambøl A., Lind P., Evaluation of glulam beams after 6 years exposure to outdoor climate. B, Treteknisk, Report 57, Oslo, September 2004, ISSN 0333-2020

[2] Ritter M.A., 1990, Timber Bridges – Design, Construction, Inspection and Maintenance, Washington DC

[3] Pousette A ., Jacobsson P., Gustafsson M., Fjellström P.-A., 2002, Inspektion av träbroar, Trätek Rapport P 0211039 (in Swedish)

[4] Pousette A., Fjellström P.-A., 2004, Broinspektion – Träbroar, SP Rapport 2004:41 (in Swedish)

[5] APA, 2001, Checking in glued laminated timber, APA Technical Note EWS R465E, November 2001

[6] EN 14081 - Part 1-4, Timber structures - Strength graded structural timber with rectangular cross section, 2009-2012

[7] EN 386, Glued laminated timber. Performance requirements and minimum production requirements, 2001

[8] APA, 1999, Evaluation of check size in glued laminated timber beams, APA Technical, Note EWS R475C, June 1999

[9] AITC, 2004, (American Institute of Timber Construction) Technical Note 18, Evaluation of checking in glued laminated timbers, March 2004

[10] APA, 2006, Understanding Checking in Glulam, www.apawood.org, 2006-06-28

[11] Pousette A., Sandberg K., 2007, Träbalkar och trästolpar i utomhusförsök – planering och utplacering, SP Rapport 2007:35 (in Swedish)

[12] Pousette A., Sandberg K., 2010, Outdoor tests of timber beams and columns, International Conference on Timber Bridges (ITCB 2010), Lillehammer, Norway, September 12 -15, 2010

[13] EN 408 - Timber structures — Structural timber and glued laminated timber — Determination of some physical and mechanical properties, 2010

From Damage Maps to Condition Inventories:

A proven concept for documentation of results of inspection of timber bridges and other timber structures

Frank Rinn
President
Rinntech Inc.
Heidleberg, Germany
frank.rinn@rinntech.com

Frank invented the Resistograph™ drilling technique in 1988 and has been refining the tool over the years and has been inspecting numerous timber structures.

Summary

Since 1988 we develop technical devices and corresponding application concepts to inspect trees and timber. The following statements are not the result of theoretical scientific work but show results of practical work of our company, inspecting timber in buildings and bridges in Germany and several other countries.

Keywords: inspection, quasi-non-destructive, drilling, relative density, decay detection

1. Introduction

In Europe, several thousand timber bridges have to be inspected on a regular base. Commonly, results of such inspections had been shown in black and white inventory sketches, showing decayed parts shaded and marked with a number, referring to a list of damage descriptions of each position. This way, reports about timber bridges and other timber structures every now and then ended up being some hundred pages long but still left significant space for question marks: the structural engineer, for example, had no idea about the areas of the structure that were not marked. Have these parts or sections not been inspected (for whatever reason, such as being inaccessible) or are they intact?

One consequence of this situation usually was (and still is in many projects) that engineers assumed more damages being present in the structures than documented and thus, as a precaution, they planned additional repair and reinforcements without knowing if these works are really required.

Another consequence was that carpenters during repair work often found additional decay or they realized that parts were replaced or repaired that did not need to – but afterwards it could not be proven anymore if this additional work was really required because the replaced parts are usually not stored.

Finally, most timber structure and timber bridge repair projects were much more expensive than planned and required.

Based on the success of the application of resistance drilling for inspecting timber starting 1986, we then developed a concept how to document inspection results that provides more precision and reliability but is, at the same time, more easy to understand for both engineers and carpenters.

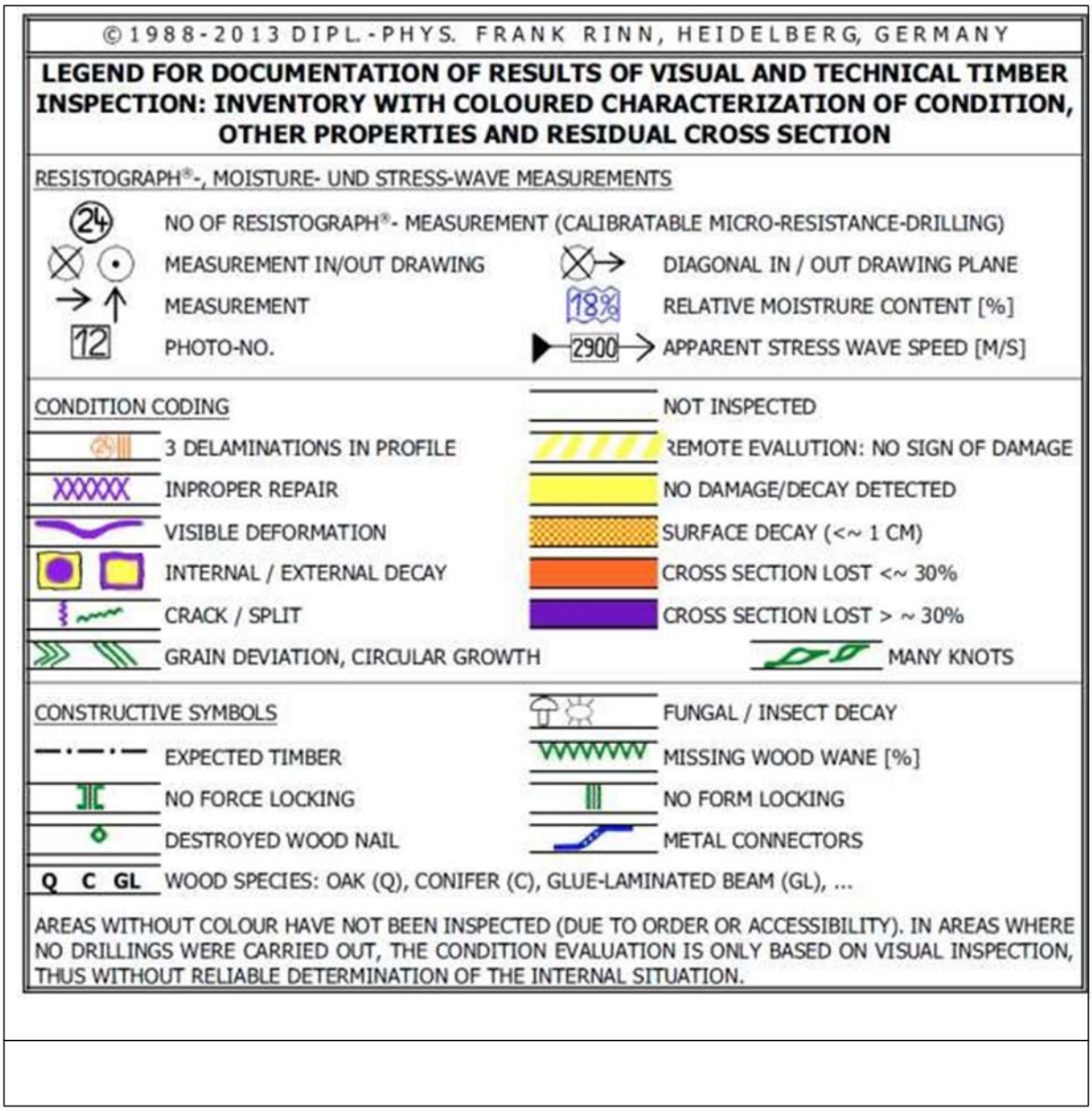


Fig. 1. Legend of colored inventory sketches showing the condition of timber in three major colors and describing additional signs for specific symptoms identified at a beam or structure.

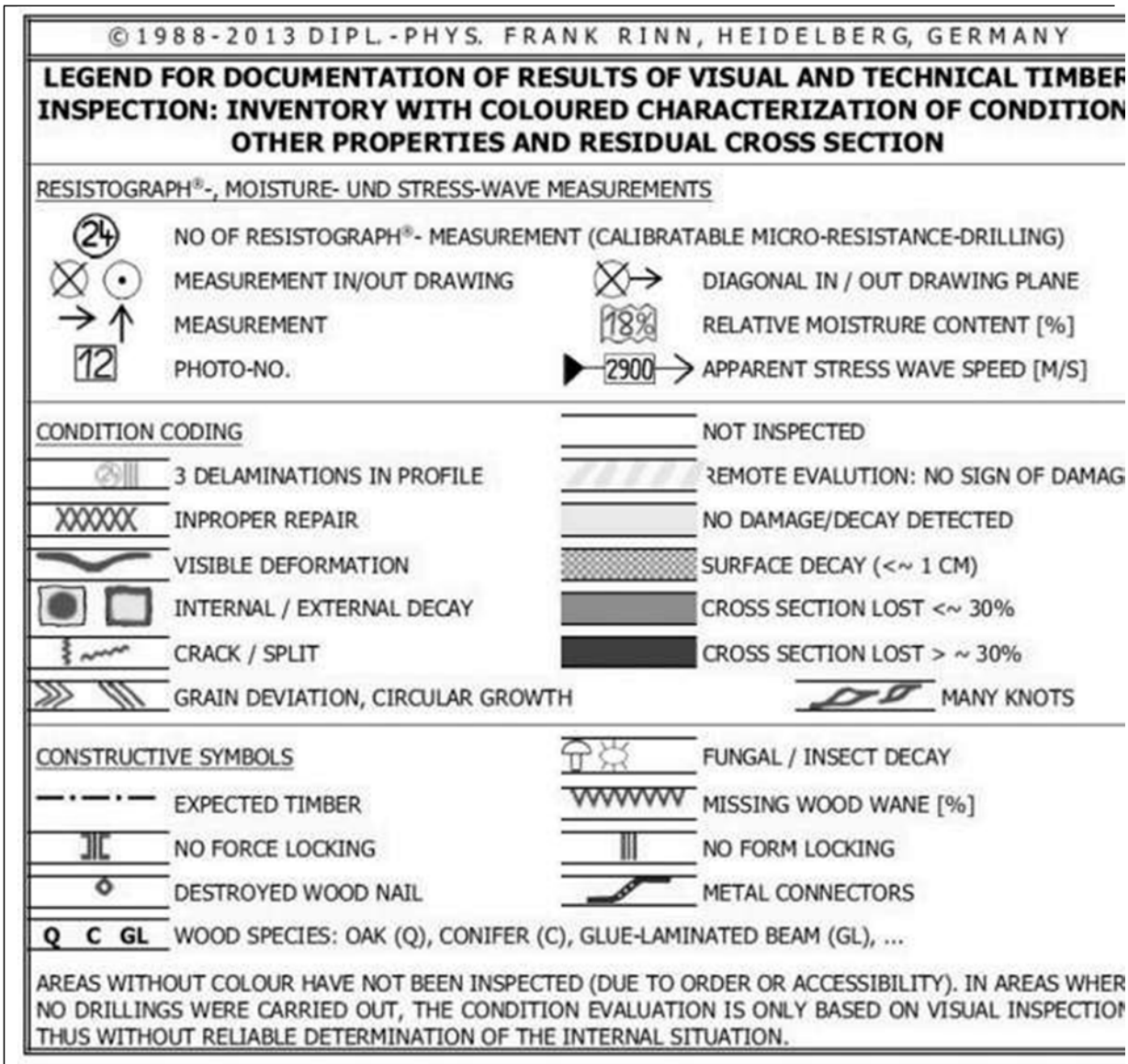


Fig. 2. Black and white copy of the condition legend still providing three major condition markers reliably differentiated by different grey scales.

2. Documentation concept

The first step forward coming from black and white sketches of timber structures with shadings for marking decay was to use colors. But, in order to make the drawings as easy as possible to read, the number of main colors had to be as small as possible, at most three or four.

At the time we developed our concept (late 1980ies / early 1990ies), color copies were still quite expensive, especially if printing in larger than standard letter sizes. The colors thus had to be selected in a way that allows black and white copies still providing the major information about decay and condition (Fig. 1). Consequently, we selected red (extensively decayed), orange (mean decay), and yellow (intact) as the major colors – because they can be differentiated easily on the first view and because black and white copies still show the three

colors reliably in differentiated types of grey (Fig.2).

The traffic-light color scheme, green for intact), yellow for partially decayed, and red for strongly decayed parts, was no option because of several reasons: in a black and white copy, green was commonly darker than red, leading to a wrong impression about the condition of the corresponding parts. In addition, structural engineers in Germany commonly used green for marking structurally relevant, local aspects and symptoms, such as cracks.

The biggest step forward was introducing a color for marking parts of timber that were inspected (either visually, by tapping and/or resistance drilling) and where found to be intact and sound. This means, if a beam was tested in whatever kind and no sign of decay was found, this beam is marked with a certain color.

For the first time, this way it was possible to distinguish between the sections of a timber structure that were not inspected (no color) and the parts that were inspected without finding damages (yellow). This may sound as a tiny little aspect but changed a lot because from then on later planning and working steps did know what parts of the structure they can rely on without doubting whether these parts had been checked or not (because there was no decay marked).

Another big step forward was combining as many parts of the usually many individual sketches of a structure as possible into one single overview drawing: this reduced the total number of sketches representing the condition of a structure often from 10 to 1 or 2 – making it much easier for engineers and architects as well as for carpenters getting an overall impression about the condition of the bridge or structure as a whole. In addition, the overview given by a single sketch with a color coded condition inventory allows the identification of connections between sources and reasons of different spots or areas of decay. That means, these overview inventories provide a base for a much deeper understanding of the structure as a whole instead of only working locally on repair of individual parts.

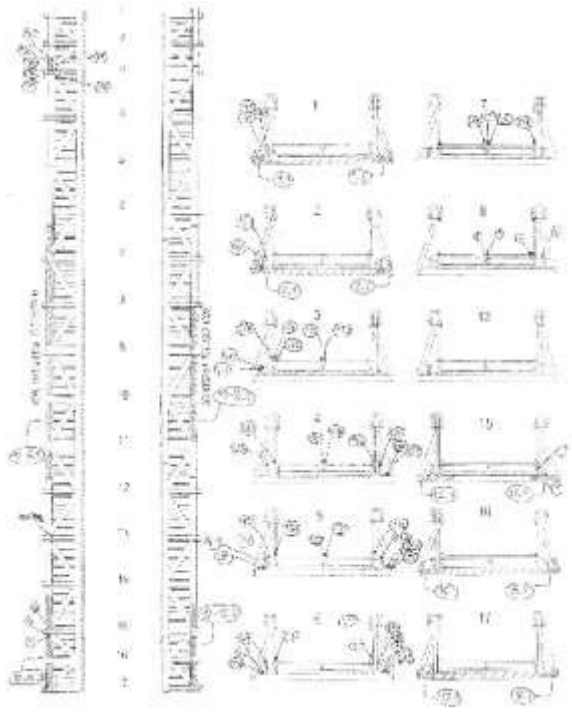
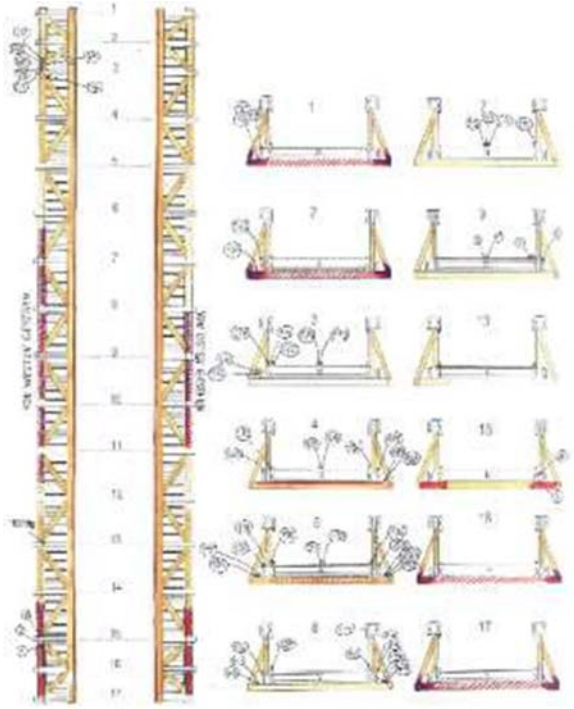
	
Conventional black and white damage map of a timber bridge. Originally it was common to mark decayed parts with a certain kind of shading and a label that refers to the text list position of the corresponding description of the found damage. Such a drawing consisted usually of 18 individual sketches of each axis and was accompanied by many pages of text within the report.	Colored version of an inventory map showing wood condition in different colors. The colors do not only reveal where decay was found but furthermore show what parts of the structure were found and proven to be intact. Because colors allow the reader to much easier identify damaged areas, such a combined sketch replaces many conventional drawings.

Fig. 3. Typical timber bridge to be inspected because of decay (although made by tropical hardwoods).

3. Practical working steps

Commonly we prepare the basic drawings of structures before the technical inspection starts. Such structural sketches have to show all relevant timber parts that belong to at least one plane of the structure or are connected with this plane. While doing that, we try to avoid showing different beams in one sketch that in reality overlay each other and represent different planes – because it is impossible to show correct colors if these beams have different conditions and thus would have to be characterized by different colors overlaying each other.

Usually, the sketches are prepared in a larger size and scale for enabling the inspector on site to put in all relevant information while inspecting - as one of our major goals was to avoid text notes but reveal all relevant aspects in the sketch. And, all evaluations should be done on the spot without having to go back to office and again work on profile analysis and come to a conclusion that, for example, additional assessments are required. This is time consuming and inefficient. Our goal was to always come to a final conclusion about the condition of timber on site while inspecting because only on site at the structure you can just tap or drill another time at another spot in order to confirm unclear results or suspicious symptoms. The highest (cost and time) efficiency we always achieved when the inspection came to a final conclusion on the site and when all relevant results were documented in the color coded inventory map on site. This drawing has then only to be reproduced in the office and surrounded by a short text note.

The reproduction of the colored on-site drawing is usually done by a reduction factor of 4. These squeezed sketches then represent the most significant part of the report. In addition, the report usually contains some illustrating pictures and a short text summary with recommendations. Even the recommendations for repair work can be partially included in the color coded sketch because lines may be implemented indicating where and how damaged beams should be cut and/or replaced.

All this fits to the traditional German saying: “A good drawing is the language of a good engineer”.

Fig. 4. Overview sketch with color coded condition inventory showing the complete facade of a historical building and all rafter of the roof structure.

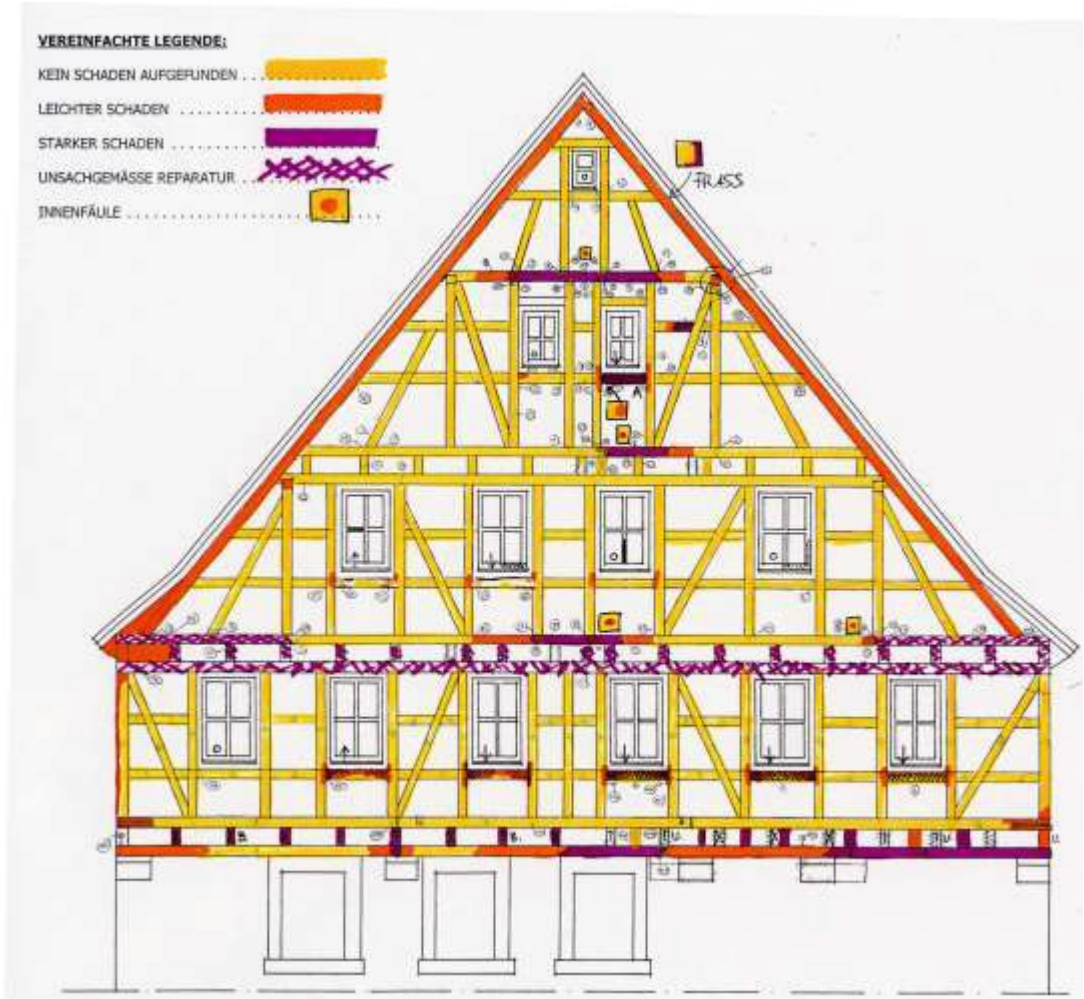


Fig. 5: Overview sketch with a simplified color coded condition inventory. This inspection was carried out by one person on one day including the drawing of the inventory what is usually done on site.

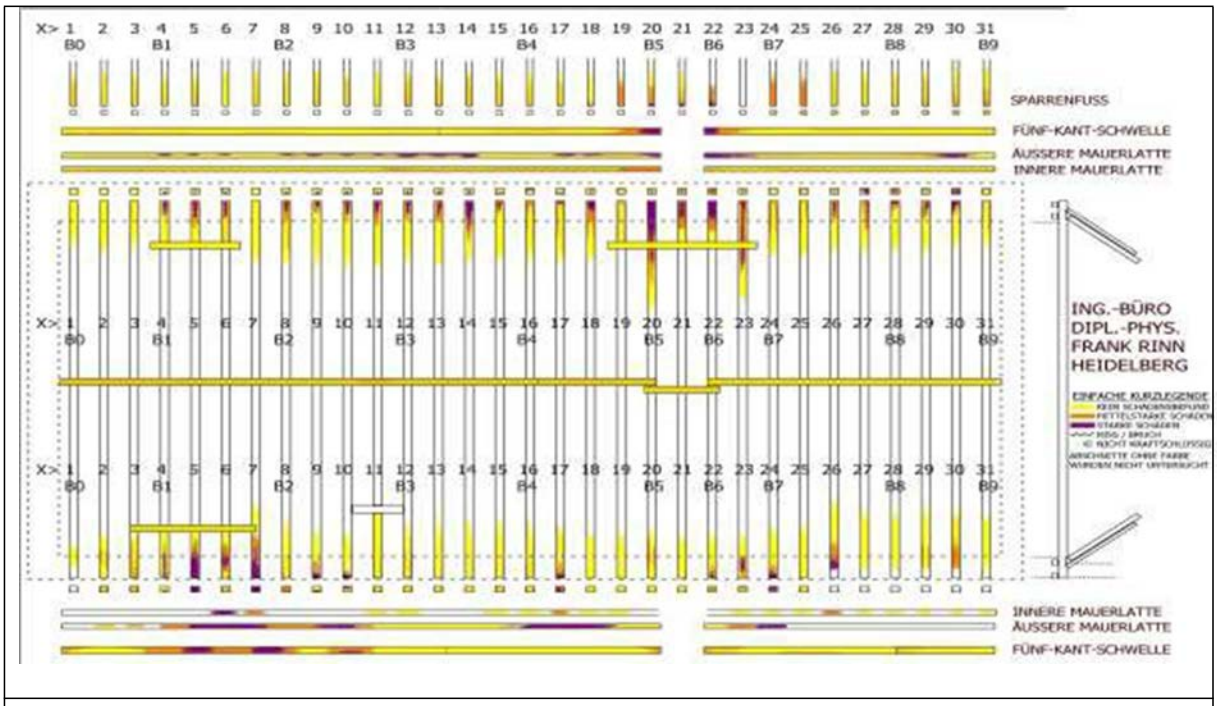


Fig. 6. Ceiling beams of a historic church plus underneath wall beams and the foot parts of looming rafters. This one sketch replaced several dozen sketches of all axes of conventional documentations

4. Consequences

Practical application of this concept in several hundred restoration projects of very different size scales proved its suitability and led to a significant increase of planning safety and furthermore to dramatically reduced total costs.

5. References and further reading

Dackermann, U., K. Crews, B. Kasal, J. Li, M. Riggio, F. Rinn, and T. Tannert. 2013. In situ assessment of structural timber using stress-wave measurements. Materials and Structures. June 2013. DOI
10.1617/s11527-013-0095-4.

Fischer, H.-B., Rinn, F. 1996: Bestandsplan mit farbiger Zustandskartierung der Holzkonstruktion. Bauen mit Holz 11 (1996): 852-858.

Rinn, F. 1988: A new method for measuring tree-ring density parameters. Physics diploma thesis, Institute for Environmental Physics, Heidelberg University, 85pp.

Rinn, F. 1990. Device for material testing, especially wood inspection by drill resistance measurements. German Patent 4122494.

Rinn, F. 1993: Gucken, Klopfen, Bohren. Zerstörungsfreie Bohrwiderstandsmessung als Teil der ingenieurtechnischen Holzuntersuchung. Bausubstanz, 5 (1993): S. 49 - 52.

Rinn, F. 1993: Catalogue of relative density profiles of trees, poles and timber derived from RESISTOGRAPH micro-drillings. Proc. 9th int. meeting non-destructive testing, Madison 1993.

Rinn, F. 1994. Resistographic visualization of tree ring density varia-tions. International Conference Tree Rings and Environment. Tucson, AZ, 1994. Printed in: Radiocarbon 1996, pp. 871-878.

Rinn, F. 1994: One minute pole inspection with RESISTOGRAPH micro drillings. Proc. Int. Conf. on wood poles and piles. Ft. Collins, Colora-do, USA, March 1994.

Rinn, F. 1994: Resistographic inspection of building timber. Proc. Pacific Timber Engineering Conference. Gold Coast, Australia, July 1994.

Rinn, Frank (2006): Konzept für Zustandsanalysen von Holzkonstruk-tionen. bauen mit holz 10/2006. S. 26-33.

Rinn, F. 2012: Basics of micro-resistance drilling for timber inspection. Holztechnologie 53(2012)3. - S. 24 - 29

Tannert, T., R. W. Anthony, B. Kasal, M. Kloiber, M. Piazza, M. Riggio, F. Rinn, R. Widmann, and N. Yamaguchi. 2013. In situ assessment of structural timber using semi-destructive techniques. DOI 10.1617/s11527-013-0095-4.

Applicability of Cross Laminated Timber in Bridges

Lauri SALOKANGAS

Robin GARNIER

Las Vegas, October 1st 2013

CLT as a construction material

- Buildings
  - Floors
  - Roofs
  - Walls

Finnish Nature Center Haltia, FInland

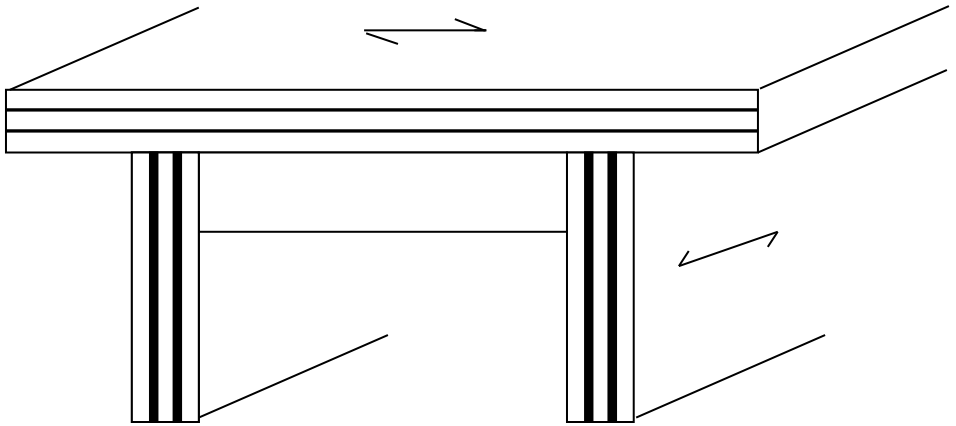
Klagenfurt, Austria

- Bridge
  - Decks

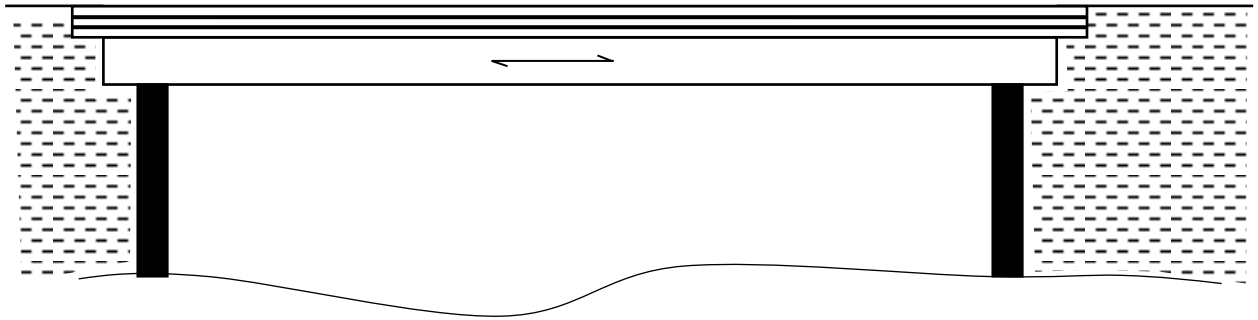
Hundorp Bridge, Norway

Applicability of CLT in bridges

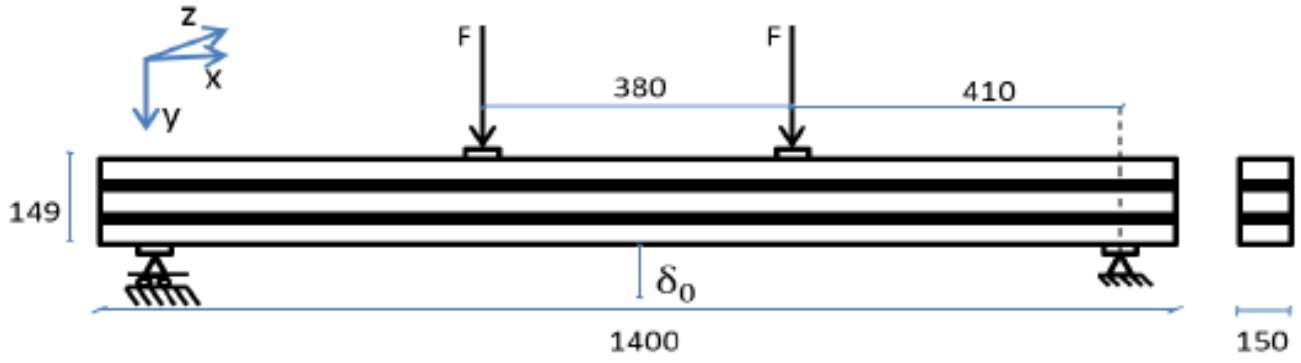
- Small span bridges
  - In theory all parts in CLT
  - Differently oriented



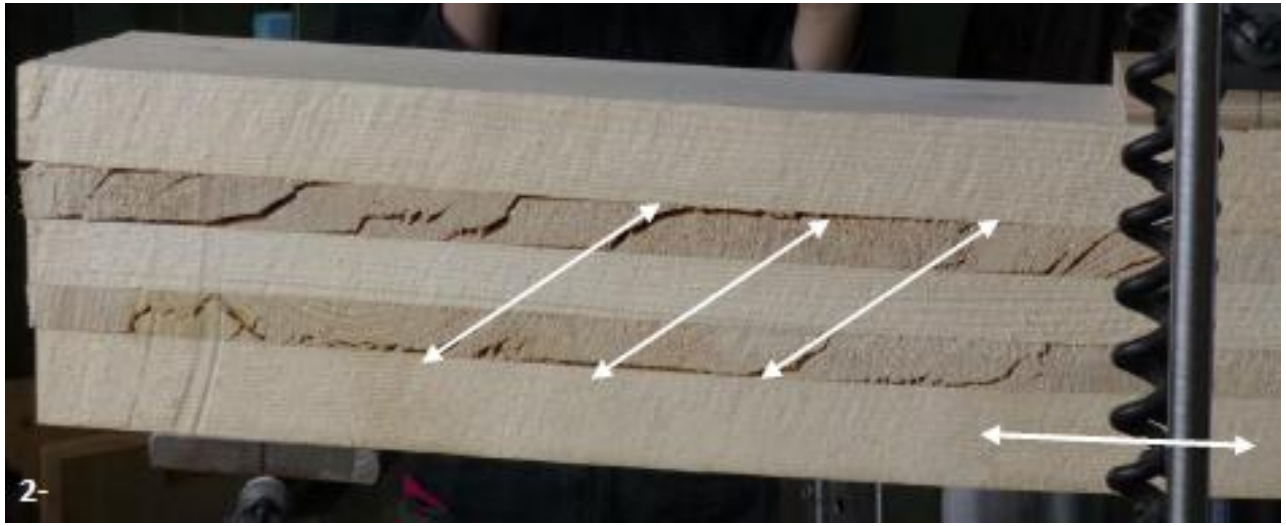
- Long span bridges
  - Deck
  - CLT long beams too weak against bending



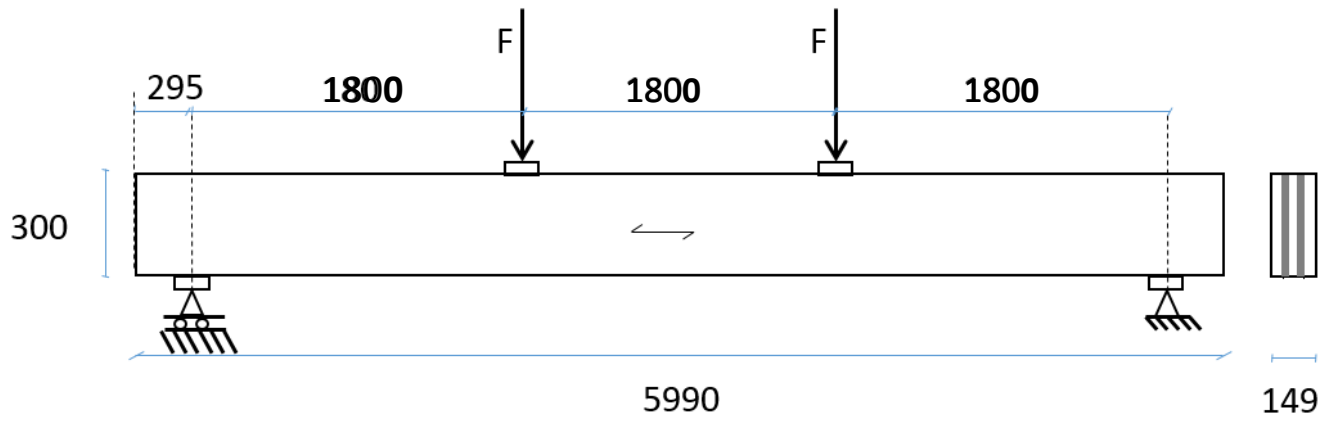
Experiments : Small scale test



- Deck representation
  - Rolling shear failure (50kN)



Experiments : Full scale CLT-beam test

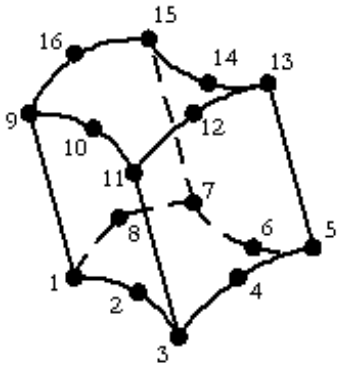


- Main girder representation
  - Bending failure (72kN)

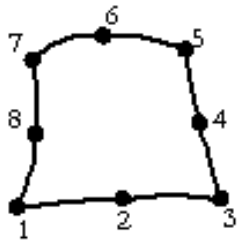
Numerical Analysis

- LUSAS finite element model
  - 3D Solid composite model
    - 16 noded solid elements (HX16L)
  - 2D Thick shell composite model
    - 8 noded thick shell elements (QTS8)

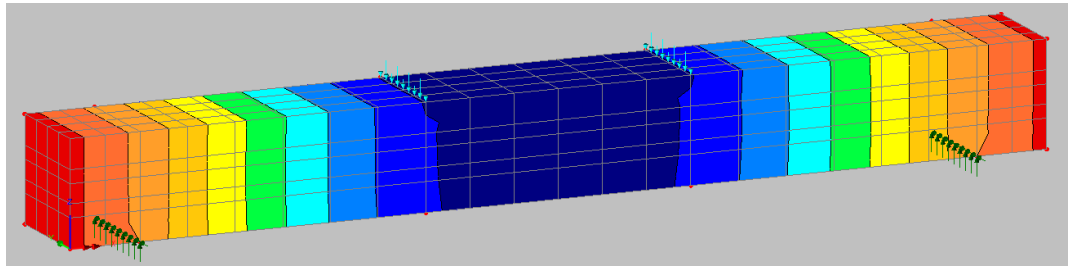
HX16L



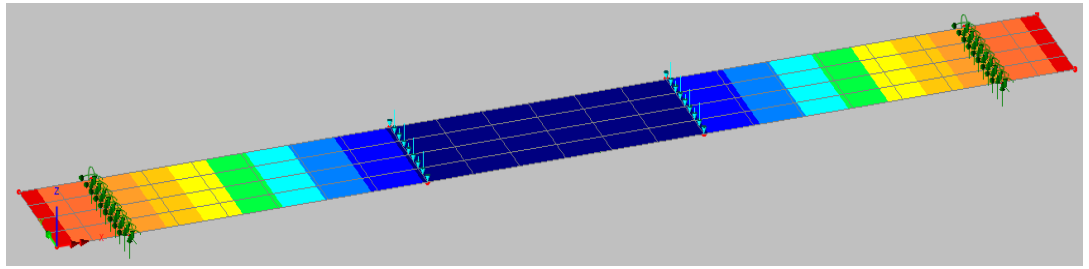
QTS8



Numerical Analysis : Small scale test



3D Solid composite model



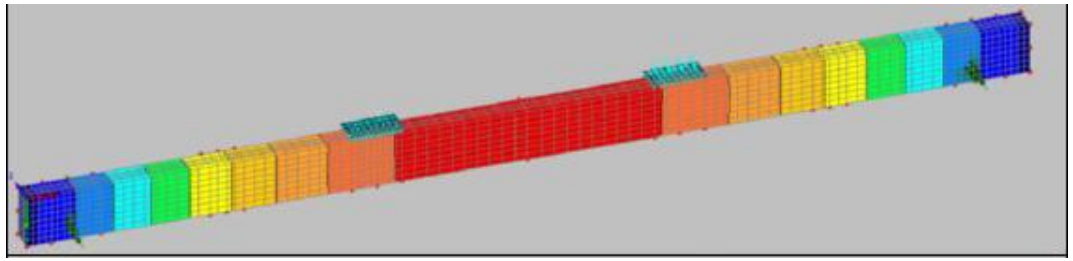
2D Thick shell composite model

Identical deflection

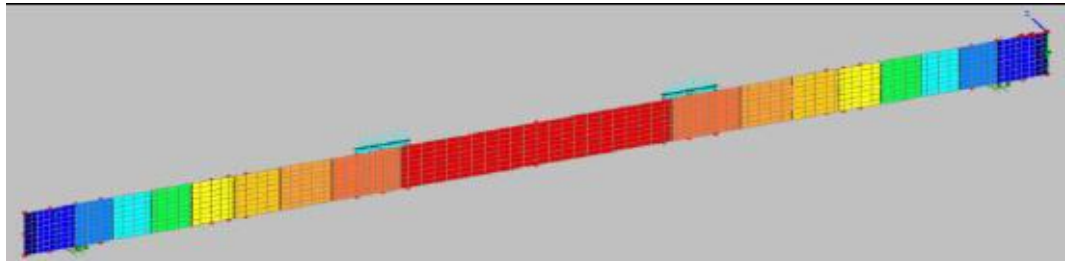
Load [kN]	Solid model	Thick shell	Experiment
20	3mm	5mm	3mm
35	5mm	9mm	5mm
50	7mm	13mm	7mm

3D model needed even for preliminary studies on CLT decks

Numerical Analysis : Full scale beam test



3D Solid composite model



2D Thick shell composite model

Identical deflection

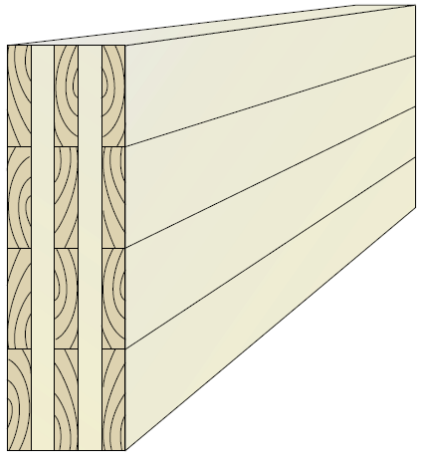
Load [kN]	Solid model	Thick shell	Experiment
20	17mm	17mm	19mm
35	42mm	42mm	46mm
50	59mm	59mm	65mm

Accuracy of 90%

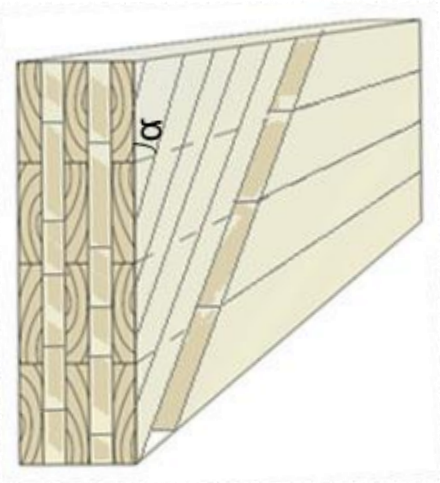
Simple 2D composite model seems accurate enough for preliminary studies on CLT beams

From CLT beams to DLT[1] beams

- Cross Laminated Timber beam
  - 90° angle orientation difference
  - Low bending stiffness if too long



- Diagonal Laminated Timber[1] beam
  - α angle orientation difference
  - Optimizable angle
    - ➢ Depending on the configuration of the bridge



[1] designation used in Bejtka I., *Cross (CLT) and diagonal (DLT) laminated timber as innovative material for beam elements*. KIT Scientific Publishing, Karlsruhe, Germany. 2011. 134 p.

Conclusion

- Cross Laminated Timber is being used for decks
- Small span all-CLT bridges possible

  Preliminary studies of CLT beams easily obtainable
- Longer span CLT bridges possible with Diagonal Laminated Timber beams
- DLT beams analysis and improvement needed

  Preliminary studies easily obtainable

Thanks for your attention!

The Durability of Wood-Concrete Composite Bridges

Lauri SALOKANGAS
Prof. (act.)
Aalto University,
School of Engineering
Espoo, Finland
lauri.salokangas@aalto.fi

Born in 1950 in Helsinki, Finland. 1981 Dipl. Eng. at TKK, Finland. 76-79 Eng. des. office Pontek Ky, Espoo, 1979-2008 Assistant and Laboratory Manager of Bridge Engineering at TKK. 2008-2013 Acting Professor of Bridge Engineering at TKK and Infrastructures and their Structural Design at Aalto University.

Aarne JUTILA
Prof. em., Dr Tech., Dr h.c.(Budapest)
Extraplan Oy, Tapiola,
Espoo, Finland
aarne.jutila@extraplan.fi

Born in 1940 in Helsinki, Finland. 1966 Dipl. Eng. at TKK, Finland. 66-67 ETH Zurich. 67-69, 70-72, 74-76 Kjessler & Mannerstråle (KM), Stockholm and Tapiola. 69-70 QUB, Belfast, N. Ireland. 72-74 TVH, Bridge Dep. 76-84 Sito Oy, Siltatekniikka Oy and TKK. 1984-2008 Professor of Bridge Engineering at TKK. 2008- Managing Dir. of Extraplan Oy.

Summary

In wood-concrete composite bridge decks glued-in are bars are simple connectors to join the two materials. For the design of the bars a simple truss model is introduced. Durability of the wood-concrete connections was tested by two long-term test series. The purpose of the tests was to find out, if outdoor condition or ageing affects the behaviour of the connections. First of the tests series consisted of pull-out tests of glued-in bars. Prepared specimens were divided into two control groups. One group of specimens was newly tested and the other group after ten years outdoor storing. In the second test fatigue loading was applied to full-size composite beam in three time periods. The beam was first newly loaded, then after two years and finally after 10-years outdoor storing. Connections with compression bars turned out to be stable and non-sensitive to ageing. Remarks of the current condition of two wood-concrete composite bridges are given.

Keywords: composite construction, wood-concrete connectors, long-term testing, durability.

1. Introduction

Many wood-concrete composite bridges have been built in Finland during the last two decades. The oldest one, *Kruununmylly Bridge*, was built already in 1993. In the longest timber bridge in Finland, the *Vihantasalmi Bridge*, a composite deck structure is used in all the five spans. [1]

The connectors can be formed simply using reinforcing steel glued-in bars. The behaviour of the connectors and a simple truss analysis to design the bars is explained in Chapter 2.

Results of long-term pull-out tests of glued-in bars and the fatigue bending test of a full-size composite beam are reported in Chapters 3 and 4. The purpose of the long-term tests was to study experimentally the influence of outdoor condition and ageing on the behaviour of the joint used in composite beams.

In Chapter 5 some remarks are given on the current condition after inspection of the two bridges mentioned above.

2. Behaviour and design of connectors

The principle of a composite beam in bending is shown in *Fig. 1*. Without any connectors slip occurs between the two materials as they both work separately and rotate around their own neutral axis. Putting connectors between the two materials the slip is prevented and the beam acts as a composite structure. Consequently, the stiffness of the whole structure increases considerably and deflection decreases.

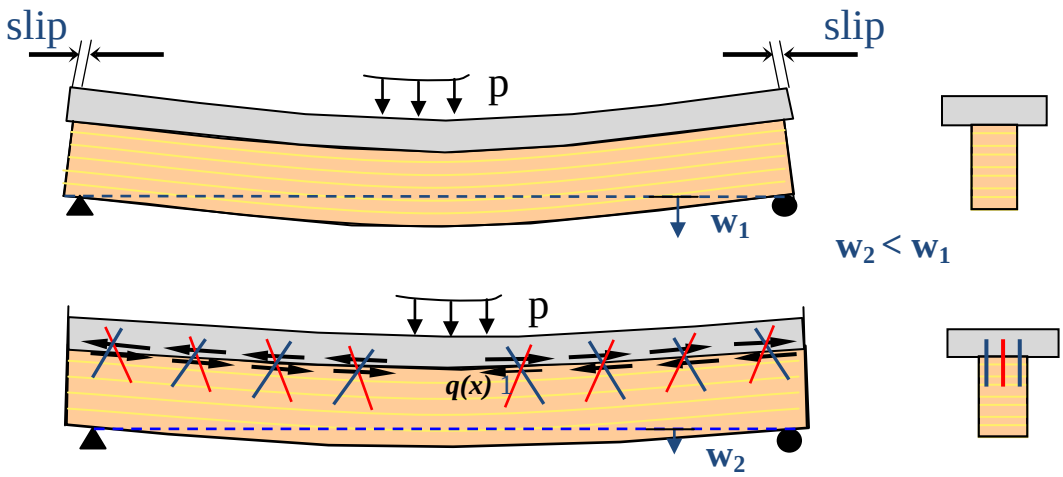


Fig. 1. The principle of joining a timber beam and a concrete slab using glued-in steel bars.

In the design of the connectors the distribution and magnitude of the horizontal shear flow *q(x)* is needed *(Fig. 1)*. This is dependent on the stiffness of the materials, flexibility of the connectors and distribution of external loading. Simple formulas and methods to determine the horizontal shear are explained in references [1, 2].

The connection between wood and concrete can easily be constructed simply by using tension and compression bars embedded in wood and anchored in concrete. If the friction, dowel action and notches between the two materials are not considered, the capacity of the joint is only dependent on the pull-out or push-in capacities of the glued-in bars.

The shear force *Q* to be transferred by one connector unit is then

$$Q = q(x) \cdot s \tag{1}$$

where *q(x)* is the shear flow and *s* is the spacing between two adjacent connectors. The tension and compression forces in bars may be determined simply by using a truss model and applying the force diagram as shown on the right hand side of *Fig. 2*.

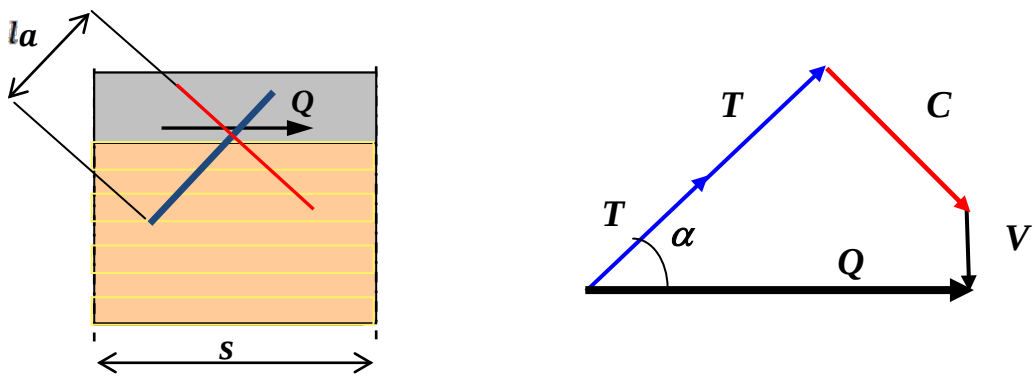


Fig. 2. Force equilibrium in a joint, where two tension bars and one compression bar is used.

The strength of the connectors is dependent either on the shear capacity of the wood or the bonding capacity of the glue layer within the anchoring length l_a in *Fig. 2*. If the shear capacity of wood is governing the strength, the capacity formula in *Eurocode 1995-1-1, Annex A, A.2.2.1*, may be used.

In this case the ultimate capacity of the bars in inclined direction can be defined as

$$T_c = \pi d_a l_a f_{v\alpha k} \tag{2}$$

where d_a is the diameter of the embedded bore hole in wood. The characteristic ultimate shear strength of wood can be expressed as

$$f_{v\alpha k} = k_\alpha \left(\frac{\rho_k}{\rho_o}\right)^{3/2} \left(\frac{d_o}{d_a}\right)^{1/5} \cdot f_{vo} \tag{3}$$

where

$$k_\alpha = \frac{1}{\sin^2\alpha + \frac{3}{2}\cos^2\alpha} \tag{4}$$

is a dimensionless coefficient depending on the angle α between the bar and the grains, ρ_k is the characteristic density of wood and ρ_o = 500 kg/m^3, d_o = 10 mm and f_{vo} = 8,465 MN/m^2 are constants.

The ultimate shear strength of the real joint is usually much higher than predicted by the truss model, if friction, dowel action or notches are taken into account.

When the capacity of the joint depends on the shear strength of the glue, the ultimate strength should be verified by testing. Usually the anchoring length and adhesive are selected so that this will not be the case.

3. Long-term pull-out tests of the glued-in bars

Altogether 24 specimens were prepared and divided into two control groups. The first group of 12 specimens was tested after three weeks from gluing. Remaining group, other 12 specimens, was tested after ten years outdoor storing to find out the long-term environmental effect on the behaviour of the connection. [3]

The specimens were square glulam sections with side length 300 mm. One corner was cut so that the hole for steel bar could be drilled in 45 degrees angle to grains. The steel bars were threaded M12 plain rods with the outer diameter of 11,7 mm.

Two types of adhesives were used; 12 pieces were glued using epoxy (E-specimens) and 12 pieces using polyurethane (PU-specimens), respectively. The diameter of the bore holes in wood was slightly larger than the diameter of bar being 11,8 mm for E-specimen and 16,2 mm for PU-specimen, respectively. The glued anchoring length was 130 mm in each specimen.

The loading procedure followed the ISO standard [4].The relation between the load and displacement was measured using load arrangement shown in *Fig. 3*.

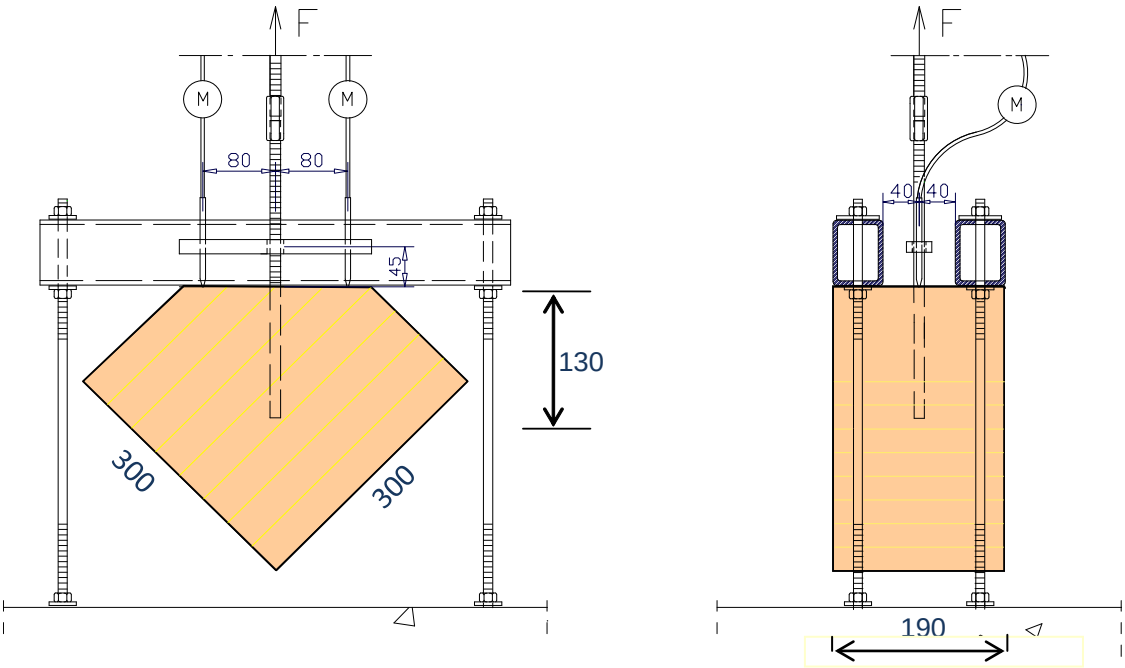


Fig. 3. Test arrangement of the pull-out test of the specimens. Measures in millimeters.

The force-displacement relations obtained in pull-out tests for 10 years old epoxy specimens are presented graphically in *Fig. 4.*

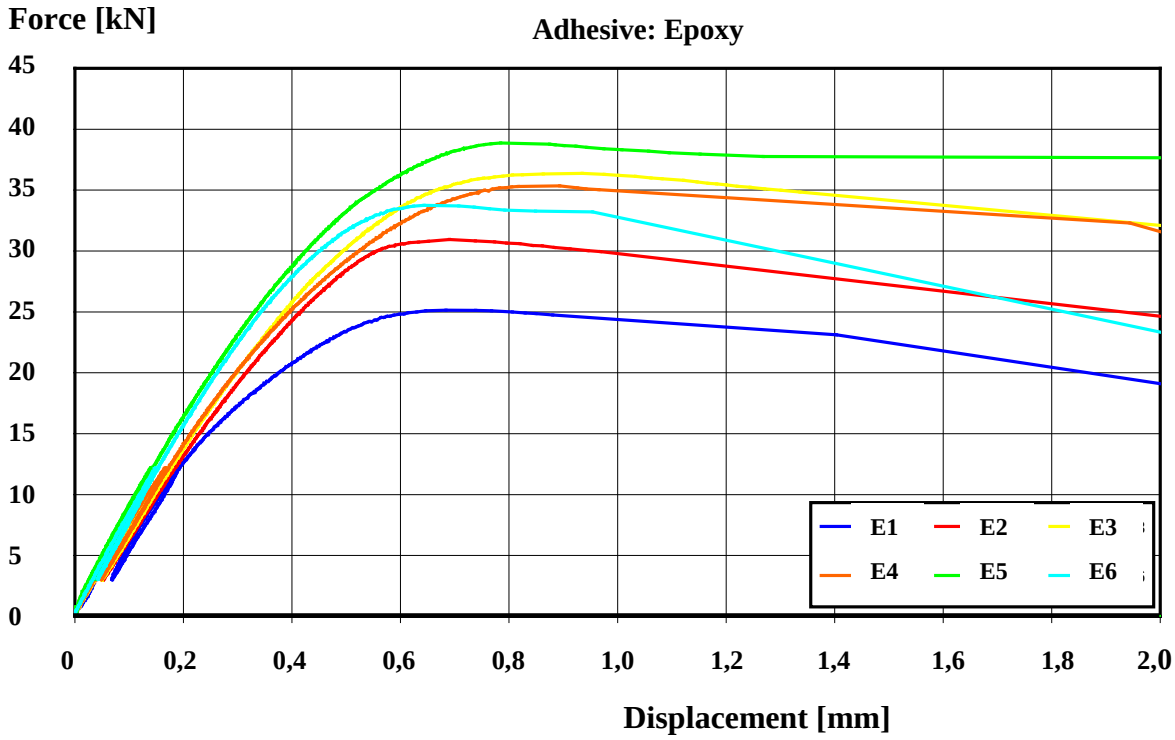


Fig. 4. Load-displacement curves of six E-specimens tested after ten years outdoor condition.

The glued-in connections were ductile as can be seen from the curves in *Fig. 4*. The ultimate load obtained varied usually between 30 – 38 kN, but for one E-specimen it was only 25 kN. By closer inspection it turned out that in this specimen 30 mm of the desired glue length was without adhesive, and that explaines the obtained lower test values (blue curve in *Fig. 4*).

The newly fabricated specimens failed as contact shear failure between wood and adhesive. After the tests the surface of the adhesive of E-specimens was noticed to be smooth, but the surface of the PU-specimens was porous indicating that moisture of wood may have reacted with the PU-adhesive.

Generally the failure modes of aged specimens were very similar to those of the failures in tests of the control group of the just fabricated specimens.

The results of pull-out tests are presented in *Table 1*, where the mean values of the measured ultimate load F_u, elastic stiffness k_s and moisture content of different groups are shown. In determination of the elastic stiffness of the joint, the method given in ISO standard was followed [4].

The test results show that adhesive quality affects both the ultimate strength and the elastic stiffness of the connection. The ultimate load capacity by using epoxy as adhesive produced more than 1,2 times as high pull-out capacity as what was achieved using polyurethane in gluing.

The same trend can be seen when the values of elastic stiffness are compared. By using epoxy the stiffness of the connection is approximately three times as high as the stiffness obtained by using polyurethane.

The test results presented in *Table 1* show that the ultimate strength and elastic stiffness increase due to aging of the joint. The increase of mean ultimate pull-out force was 12 % for PU-specimens and 26 % for E-specimens. Also the increase of the mean stiffness of the aged specimens was more than 13 % compared to newly tested specimens.

Table 1. Mean values for ultimate load F_u and stiffness k_s of different groups of specimens.

Adhesive:	Testing age	Group	F_u [kN]	k_s [kN/mm]	Moisture %
Epoxy	Three weeks	E-0	26,16	63,91	15,2
	10 years	E-10	33,05	72,13	18,1
	Ratio	E-10/E-0	1,26	1,13	1,19
Polyurethane	Three weeks	PU-0	21,42	20,79	15,4
	10 years	PU-10	23,93	28,70	16,7
	Ratio	PU-10/PU-0	1,12	1,38	1,09

4. Long-term fatigue tests of a composite beam

The deck of the *Vihantasalmi Bridge* is composed of glued-laminated timber girders and concrete slab. Reinforcing steel bars and notches are used as connectors (*Fig. 5).* Many laboratory tests were made during the design phase of the bridge. [5, 6]

To investigate the fatigue behaviour and the influence of the long-term outdoor condition, two equal full-size beams were prepared (*Fig. 6*). The beams were tested for fatigue loading in three periods.

The first fatigue test was applied to newly fabricated beams and the second after two years outdoor condition. The third and last test was applied to one beam only after it was stored outside ten years period. [7]

Fig. 5. The connectors and notches of the timber girders in the deck of the Vihantasalmi Bridge.

Fig. 6. Test arrangement and the wood-concrete composite test beam.

The beam selected for long-term fatigue testing experiment was equipped with two different connector arrangements. Half of the beam was equipped with tension and compression bars, the other half with tension bars only. The structure of the beam is shown in *Fig. 7*. Additionally, wedge-shaped notches on the upper surface of the timber beam were machined on both sides to improve the shear capacity between wood and concrete (*Figs 5-7*).

The beams were loaded by a sinusoidal pulsating load with a frequency of 1 Hz and constant amplitude generated by two hydraulic jacks. During the first two tests series the maximum load *P* in one jack was 200 kN, which corresponded to the design load of one beam of the bridge deck. During the last 100 000 cycles the test was carried out by raising the maximum load *P* up to 300 kN.

The minimum load was 10 kN in all fatigue tests.

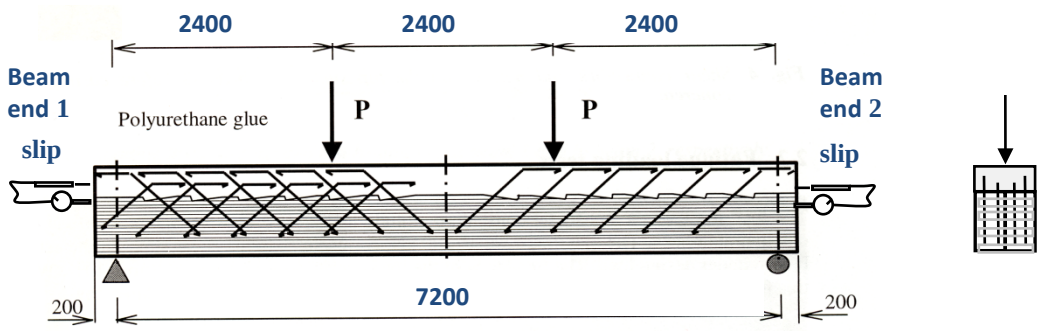


Fig. 7. Test arrangement in fatigue tests. Slips were measured at both ends of the beam.

The third fatigue test was applied to the beam, which had been kept outdoors without shelter approximately 10 years. The maximum load *P* in the last fatigue test was increased up to 330 kN.

Complete loading history of the three fatigue tests is shown in *Fig. 8*.

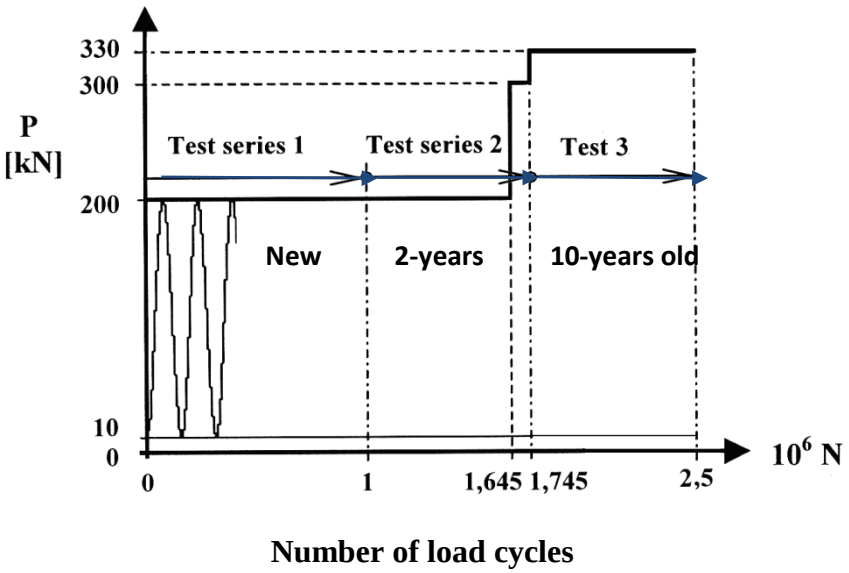


Fig. 8. The loading history of the three fatigue test series of the composite test beam.

The fatigue test was completed when the total number of load cycles *N* reached 2,5 million. No visible damage could be observed. The change of the stiffness due to cyclic loading was controlled by measuring the force-slip relation subjecting the beam under static test after regular number of load cycles. Two test results after 1,8 and 2,5 million load cycles are shown in *Fig. 9*.

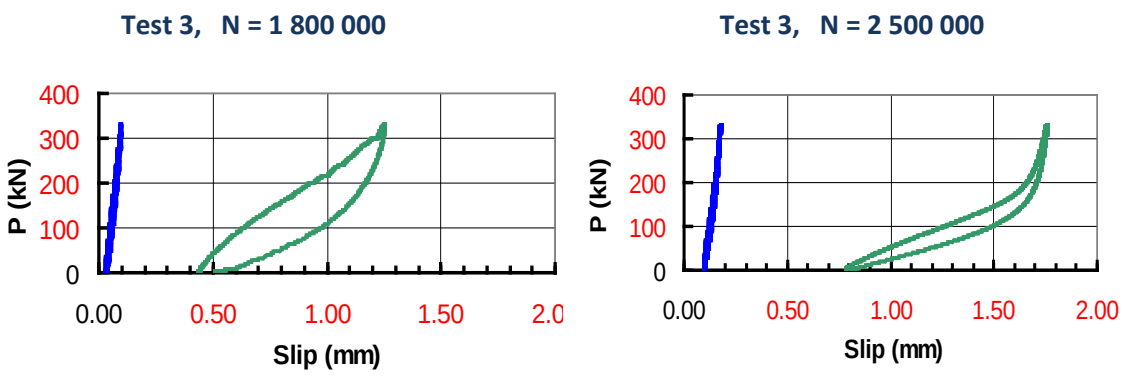


Fig. 9. Relation between applied load and slip after 1,8 and 2,5 million load cycles, respectively. The blue curve describes the measured slip at the end with compression bars (beam end 1) and the green curve at the end without compression bars (beam end 2).

At the end of the beam in which the compression bars were used, the joint was stiff and behaved linearly elastic. The permanent slip was very small even after 2,5 million load cycles.

At the other end of the beam, where compression bars were not used, the maximum slip increased constantly with increasing number of load cycles. Similarly, the permanent slip grew gradually and the connection behaved nonlinearly. Plain notches are not able to carry the compression force until the displacement in the joint is large enough.

The measured slips at both ends of the beam after different number of load cycles are shown in *Table 2*.

Table 2. Maximum slips measured during the three fatigue tests.

Fatigue test	Testing age	Max. Load [kN]	Number of load cycles [N]	Slip at beam end 1 (compression bars) [mm]	Slip at beam end 2 (without compr. bars) [mm]
Test 1	0 years	200 200	1000 1 000 000	0,066*) 0,09*)	0,33*) 0,40*
Test 2	2 years	300	1 745 000	0,09**)	0,50**)
Test 3	10 years	330 330	1 745 001 2 500 000	0,09 0,19	1,19 1,80

*) Slip value was scaled using load factor 330/200 = 1,65
**) Slip value was scaled using load factor 330/300 = 1,10

The test results show that ten years outdoor condition did not cause any changes to the stiffness at beam end 1, where compression bars were used, and the absolute value of slip was very small.

At beam end 2, where the compression bars were missing, the slip was significantly greater than at beam end 1. Moreover, in the beginning of the third test (N = 1 745 001) the measured slip 1,19 mm was 2,4 times as big as it was 8 years earlier (0,50 mm). After 755 000 further load cycles the slip grew up to the value of 1,8 mm. On the basis of the test results, both ageing and increasing number of load cycles affected the slip growth in the joint at beam end 2.

It can be concluded that compression bars are essential to keep the slip on low level and to preserve the stiffness in a long run.

After the last fatigue test was finished, the wooden part of the test beam was cut and opened. Inner wood was inspected and found to be in perfect condition (*Fig. 10*).

The bottom end of the glued-in bar in wood was also uncovered and examined. No damage due to fatigue loading could be observed and no signs of slip could be found. The surface of the glue was smooth (*Fig. 11*).

Fig. 10. Cross-section of the test beam after ten years outdoor condition and fatigue tests.

Fig. 11. The bottom end of the glued-in bar in wood after ten years outdoor condition and fatigue tests.

5. Remarks of the current condition of two composite timber bridges

Kruununmylly Bridge is the oldest wood-concrete composite bridge built in the Nordic countries (*Fig. 12*). The beams of the deck are glued-laminated timber beams with salt impregnation.

Fig. 12. Kruununmylly Bridge after completion in 1993.

The connectors are glued-in reinforcing bars in two inclined directions.

The bridge has been in service for vehicle traffic without any special problems for twenty years.

The connectors are glued-in reinforcing bars in two inclined directions, similarly as shown in *Fig. 5.*

The span of the bridge is 8 m and the effective width is 12 m.

The current outlook of the bridge has somewhat suffered during years. Bushes have grown wildly hiding partly one side of the deck. Additionally, the surface of the concrete edge beams and the wooden railings need refurbishment (*Fig. 13*).

The glulam beams were inspected and found to be in perfect condition *(Fig. 14).* Cracks or other damages in the wooden beams were not discovered. The moisture content of several beams was measured and found to be on normal level in a range from14,2 % to 18,2 %.

Fig. 13. Kruununmylly Bridge in 2013.

Fig. 14. Outer surface of the glulam beam.

Vihantasalmi Bridge is the largest timber bridge in Finland with its three 42 m long main spans.

Fig. 15. One span of the *Vihantasalmi Bridge in 2013.*

Due to its unique structure the bridge has been under continuous survey after its inauguration in 1999.

Some remarks about the current condition may be noted after 14 years service.

The zinc coating of the railing poles has caused runs on the surface of the utmost beam causing visible harm to the aesthetics of the side view (*Fig. 15*).

The wooden rail on the deck is worn out and needs to be replaced (*Fig. 16*).

The southern end of the bridge needs clearing of bushes (*Fig. 16*).

Fig. 16. Worn top guide of the railing. On the background wildly growing bushes are seen.

Fig. 17. Water pocket and crack at the end of the inclined load-carrying member.

Two inclined load-carrying members of the superstructure were inspected by removing the protecting wooden cover on the beams. Water pockets were found at the ends of the members and the beam ends were practically wet. A long crack in the direction of grains was noticed (*Fig. 17*).

At the height of 500 mm from the deck surface, the moisture content of the inclined members was on a normal level varying between 17,2 % and 18,5 %.

High moisture contents were reported earlier at the same joint, but at the ends of the wooden beams under the concrete deck [1]. The problem of the connection is that the ends of wooden members are on all sides confined by steel walls. The situation is even worse at the top part of the connection, where concrete cover encloses the joint.

The problematic joints need to be inspected and repair actions planned.

6. Conclusions

The joint made of glued-in bars is simple and offers an effective way to connect a wooden beam with a concrete slab. The behaviour of the connection is easy to understand, design and analyse.

The results of long-term pull-out tests showed that glued-in bars are ductile and stable connectors. Ten years outdoor condition did not reduce the strength or stiffness of the connection. Furthermore, the test results of old specimens indicated a little higher strength and stiffness values than obtained for newly fabricated specimens.

Epoxy is recommended to be preferred as adhesive, when strong and stiff connectors are needed. However, gluing procedure of the bars is sensitive to mistakes, especially when PU-adhesive is used. Additionally, the work has to be carried out with special care.

The fatigue tests showed that the connectors are ductile. No fatigue damage could be observed even after 2,5 million load cycles. When the compression bars are used in the connection, the joint behaves practically linearly. It became evident during fatigue tests that compression bars are essential to guarantee the long-term strength and stiffness properties. It was also pointed out that the ageing and environmental exposure did not influence negatively on the behaviour of the joint.

Wood-concrete composite deck is most suitable to be used in small or medium-span simple road bridges. This was confirmed during the inspection of the beams of the oldest bridge of this type, Glulam beams are in perfect condition after 20 years period of service.

In small wood-concrete composite bridges the moisture content of the wooden beams stays on a low level. In more complicated bridges there are a lot of details and connections, which require special actions to avoid excessive moisture content.

In any case, timber bridges must be designed and maintained so that the wooden parts can be kept dry during the service life of the bridge. Consequently, 100 years service life can be reached.

Acknowledgements

The authors want to express their gratitude to Ms Eeva Heiskanen, Dipl. Eng., and Mr Jarmo Tommola, Tech. Lic., for preparing the long-term tests and reports, and to Mr Risto Mäkipuro, Dipl. Eng., for the valuable information and comments related to the bridges presented.

References

[1] Jutila, A. & Salokangas, L., *Wood-Concrete Composite Bridges - Finnish Speciality in the Nordic Countries*. International Conference on Timber Bridges, Lillehammer, Norway 12-15 September 2010. pp. 383-392.

[2] Mäkipuro, R. & Jutila, A., *Analysing Methods of a Composite Girder Containing Wood*. Nordic Timber Bridge Project. Nordic Timber Council. 1999. 21 p.

[3] Tommola J. & Heiskanen E., *Effect of outdoor conditioning to glued-in bars*. Test Report. Helsinki University of Technology, Department of Structural Engineering and Building Technology. 2008. 6 p. (Unpublished).

[4] ISO 6891 - 1983 Timber structures. Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics. Brussels: European Standard, 1991. 4 p. (EN; 26891).

[5] Rantakokko, T. & Salokangas, L., *Design of the Vihantasalmi Bridge, Finland.* Structural Engineering International (SEI) Vol 10, 3/2000. pp. 150-152.

[6] Jutila, A. & Salokangas, L., *Research on and Development of Wooden Bridges in Finland.* Structural Engineering International (SEI) Vol 10, 3/2000. pp. 182-185.

[7] Jutila, A., Salokangas L. & Heiskanen, E., *Effect of long-term outdoor conditioning to fatigue strength of wood-concrete composite girder.* Follow-up of the Vihantasalmi Bridge. Test Report. Helsinki University of Technology, Laboratory of Bridge Engineering 2008. 9 p. (Unpublished).

Short-term monitoring of a cable-stayed timber footbridge

Erhan SARAÇOĞLU
Researcher
SP Technical Research Institute of Sweden/Wood Technology
Skellefteå, Sweden
Erhan.saracoglu@sp.se

Erhan Saraçoğlu, M.Sc., has been working on structural health monitoring, structural mechanics and Finite-element modelling of timber constructions since 2011.

Anders GUSTAFSSON
Researcher
SP Technical Research Institute of Sweden/Wood Technology
Skellefteå, Sweden
Anders.Gustafsson@sp.se

Anders Gustafsson M. Sc., has been working with cross laminated wooden elements since 1996, with design and construction and also other wooden constructions as safety barriers in new post and beam systems.

P. Anders FJELLSTRÖM
Inspector
SP Technical Research Institute of Sweden/Wood Technology
Skellefteå, Sweden
Per-Anders.Fjellstrom@sp.se

Per-Anders Fjellström has been working with full scale tests, health monitoring, inspection test methods and inspections manuals since 1994.

Summary

Älvsbacka cable-stayed timber bridge was built in 2010, over Skellefteå River in Sweden. Sensors were integrated at strategic locations of the bridge during the construction process. This paper explores the bridges' synchronized structural health monitoring system that consists of GNNS, force transducers, a weather station and wireless accelerometers. The initial results from short-term monitoring are reported and correlation between deformation of the bridge and effect of temperature changes is emphasized. It is planned to perform long-term monitoring for complete understanding of structural behaviour of the bridge.

Keywords: structural health monitoring, GPS/GNSS, cable-stayed bridge, timber bridge, short-term monitoring, climate effect

1. Introduction

Älvsbacka cable-stayed timber bridge was built in summer 2010, over Skellefteå River in Sweden. This timber footbridge connects the two neighbouring areas of Anderstorp and Älvsbacka. The bridge was constructed as part of the "Sense Smart City" European Union project under which Luleå University of Technology and Skellefteå Municipality are cooperating. The overall aim of this project was to reduce the cost of maintenance work of timber bridges by implementing a continuous measurement system on the structure over the long term. Furthermore, the feedback

from monitoring result ensures that the design of the next generation of timber bridges will be safer and more durable.

Currently Älvsbacka cable-stayed timber bridge is the longest cable-stayed timber bridge in Scandinavia, with a span of 130 meters. This unique timber bridge provides exclusive opportunities for researchers to investigate and understand the behaviour of long-span timber structures.

The sensors for monitoring of the bridge were integrated during construction. This procedure provided an efficient solution for collecting data from strategic locations of the timber bridge. Non-destructive evaluation has been implemented at the bridge since March 2011 and the information that is collected from the sensors is being saved for current and future analyses.

In this paper the monitoring system of Älvsbacka bridge is presented and the initial results from short-term monitoring are reported. The main scope of this paper is to explore and investigate the correlations between changes of climate and bridge movements/deformations.

2. The timber bridge specifications

This cable-stayed footbridge was constructed in summer 2010 and has a span of about 130 meters. The bridge deck is held by 48mm and 64mm diameter steel cables which are connected to four wooden pylons and each pylon is anchored to concrete anchor blocks by two parallel steel cables with a diameter of 80 mm. The height of the pylons is 23 m and they have an identical cross-section of 900x900 mm^2 and are manufactured from untreated European white wood.

The main beams are manufactured from glulam with a cross-section of 645x1100 mm^2 and the distance between the main beams is 4.8 m. The bridge deck consists of a 45 mm open plank deck on longitudinal beams and cross beams which carry the deck. The camber of the bridge deck is 1m.

Fig. 1 Cable-stayed Timber Bridge of Skellefteå River

The bridge was designed for a uniformly distributed load of 4 kN/m^2 or alternatively 40 and 20 kN of a maintenance vehicle (e.g. forklift or sky lift). According to Eurocode 5, the bridge is designed for a the maximum deflection of L/400 where L is the total span of the bridge [1], [2].

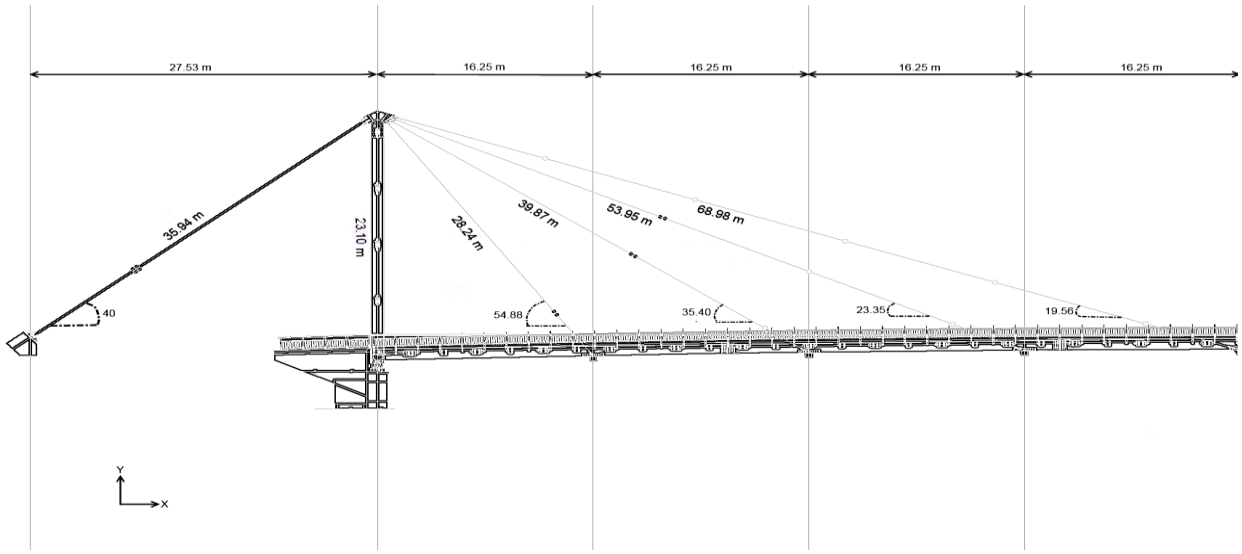


Fig. 2 Elevation of symmetrical half of the bridge

3. Structural Health Monitoring (SHM)

Monitoring technologies are now well-established and they have been used in many complex infrastructures all around the world. Nevertheless, deployments are still mostly limited to very large-scale or severely deteriorated structures or are implemented for research activities [3]. This means that SHM is still a high-cost option for operators. On the other hand, the data that are provided by the SHM systems give crucial information about the structural integrity, characteristic behaviours, fatigue behaviours, etc. of structures that are observed in short and long-term. Maintenance planning and action is mostly performed by using the live-data or post-processed data from the infrastructures. Thereby, the cost of maintenance may be optimized and necessary reinforcements and modifications can be done before they reach a risky level.

A good overview of the state of the art of monitoring technologies and the implementations and practical applications of SHM systems is given in the literature [4], [5]. However, most of today's implementations of SHM are performed on concrete infrastructures. In the literature, there are few applications of SHM on wooden structures. Nevertheless, owing to their increasing of complexity and scale, these timber structures demand advanced monitoring techniques.

3.1 Monitoring system of Älvsbacka timber bridge

The SHM system for Älvsbacka timber bridge was designed for both long-term and short-term monitoring. The monitoring system consists of advanced monitoring sensors to observe and understand the long-span bridge's behaviour. The sensors were mounted only on the southern half of the bridge due to its symmetrical geometry. Therefore, resolution of the data is increased and a more detailed view of the behaviour of that part of the bridge is obtained. Table 1 shows the monitoring system details and specifications.

Table 1 Monitoring system details and specifications

Sensors/Equipment	Quantity	Specification/ Brand	Phenomenon	Sampling Interval	Resolution	Accuracy
GNSS	3	Leica GMX 901/AS10	Displacements	1 s.	0.2 mm rms	±20 mm
Force Transducer	5	HBM	Cable tensions	1 s.	0.01 kN	±0.10%
Weather Station	1	Vaisala WXT 520	Climate changes	2 s. – 5 s.	0.1	*
Wireless Accelerometer	18	Mulle v6.2	Vibrations	0.02 s.	3.9 mg/LSB	±1%
Data Logger	1	Delphin Top Message	Data acquisition for cable tensions	1 s.	21 bit	-
Computer	2	Amb. Ctrl. Atom 1,66 Ghz	General data acquisition	-	-	-

* ±0.3 m/s wind speed, ±3° wind direction, ±3°C temperature, ±3 %RH relative humidity

In this project, the aim is to create a synchronized monitoring system with different types of sensor. Therefore, the correlations between data from the sensors are performed easily without introducing any time shift.

3.2 Sensors and deployment

To optimize the health monitoring costs it was assumed that the bridge behaves homogeneously, and that the structural characteristics can be mirrored or identification of the system can be obtained from one half of the span of the bridge. Sensor locations and the main unit are illustrated in Figure 3.

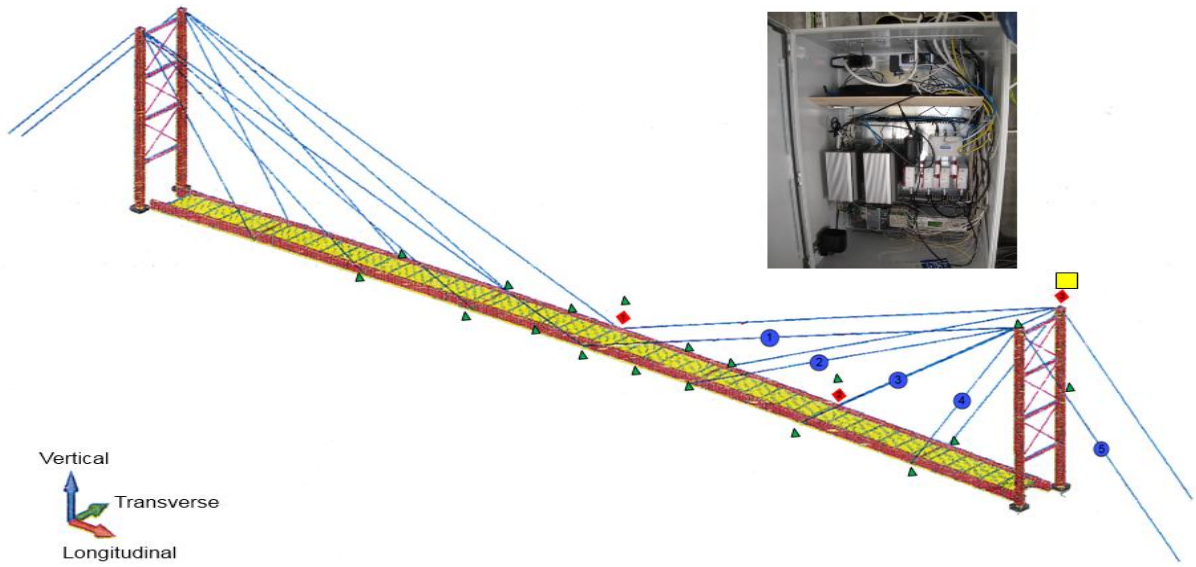


Fig. 3 Sensor locations of Älvsbacka Timber Bridge and backend unit. Red diamonds are GNSS antennas, green triangles are wireless accelerometers, blue circles are force transducers and yellow square is weather station.

3.2.1 Global Navigation Satellite System (GNSS)

Three Leica GMX 901 GNSS receivers and AS10 type antennas were mounted on the bridge. One is on the mid-span, one is on the quarter-span and the last one is on the south-east pylon. The antenna mounted on the south-east pylon is used as a reference antenna for the antennas (rover antennas) mounted on the eastern side of the deck. Rover antennas provide relative 3D coordinate changes based on the reference antenna. The sampling interval of all the GNSS receivers is 1 s and they are connected to a backend unit. The navigation information is being recorded in NMEA (National Marine Electronics Association) GGQ files [6].

3.2.2 Force Transducers

Five custom-made strain gauge based force transducers were integrated in the stay cable on the western side of the bridge. These force transducers use Wheatstone bridge resistance elements that provide better signal control in harsh environmental conditions, and are connected to a Delphin Top Message data logger with a sampling interval of 1 s. The data logger is then connected to one computer to store the tension data.

3.2.3 Weather Station

Climate conditions around the bridge area are measured by a Vaisala WXT520 type weather station [7]. The weather station was mounted at the top of the south-east pylon. Four different climate parameters are being measured by the weather station, namely temperature, humidity, average wind speed, and average wind direction. The sampling frequency of the weather station is between 0.3 Hz and 0.5 Hz.

3.2.4 Wireless Accelerometers

On the bridge 18 wireless micro-electro-mechanical system (MEMS) accelerometers have been used to identify the bridge's dynamic characteristics. The locations of these accelerometers are illustrated in Fig. 3. The accelerometers' boards are designed to use battery power; however, due to harsh climate conditions, wired electricity is in fact used to supply the power to the accelerometers. The results from the accelerometers are out of the scope of this paper, the emphasis of which is on the short-term deformation of the bridge and climate changes.

3.3 Backend unit and synchronization

The backend unit consists of data acquisition systems, receivers, a data logger, power supply and heating unit. All these systems are in a protective box located under the southern part of the bridge. The heating unit governs the inside temperature of the protection box,as can be seen in Fig. 3, and protects the sensitive equipment against harsh winter conditions. Two of the three data acquisition systems (Amb. Ctrl. Atom 1,66 Ghz) are connected to the weather station, force transducers and GNSS receivers output. The third one is connected to the wireless accelerometers to collect vibration data.

Different types of sensor have been deployed to create a comprehensive SHM system for Älvsbacka timber bridge. Since the sensors are different, in order to shorten the post-processing time and perform better correlation between sensor responses, time synchronization plays a significant role. Synchronization between the different sensors are established by using one data acquisition unit as a Network Time Protocol (NTP) server. This unit is connected to the GNSS receivers which receive the time stamps from the satellite, and this time stamp then constitutes the basis for the other sensors' time stamps. Thereby the data interval of all the sensors' output is equal. Synchronization provides is highly convenient during the post-processing of data when instant correlation between measurements is needed.

Remote access to data acquisition systems has been established by an internet connection to of the construction area. This feature makes, inspection of the sensors' health easier because there is now no need to phsically go to the constructional area. Another advantage of remote access is the ability to observe the bridge's structural condition from a distance provided the user/owner has an internet connection.

4. Short-term monitoring results

In this section, short-term monitoring results of Älvsbacka timber bridge are presented. The emphasis is on correlation between climate condition changes and the bridge's total deformation.

4.1 GNSS measurements

Älvsbacka timber bridge has been observed using three GNSS system since March 2012. The results provide a verification of the structural design in the short-term. Generally, deformations in a timber structure are a good indicator of its structural condition. The short-term measurements provide the bridge's daily or instant total deformation pattern, which is mostly influenced by temperature changes.

GNSS measurement data are saved in NMEA (National Marine Electronics Association) GGQ files with the file extension of "*rtl*". Each file consists of 3D navigation data, coordinates quality, the number of satellites the antenna sees, time stamps, etc. The length of each measurement file is 25 hours, which consists of 90,000 lines of navigation data. These files are then post-processed using MATLAB commercial software.

It is assumed that, if the antenna mounted on the top of the south-east pylon is accepted as a reference antenna, due to the robustness of the foundation of the bridge's pylons, deformation measurements in vertical direction will less error than in the longitudinal and horizontal directions.

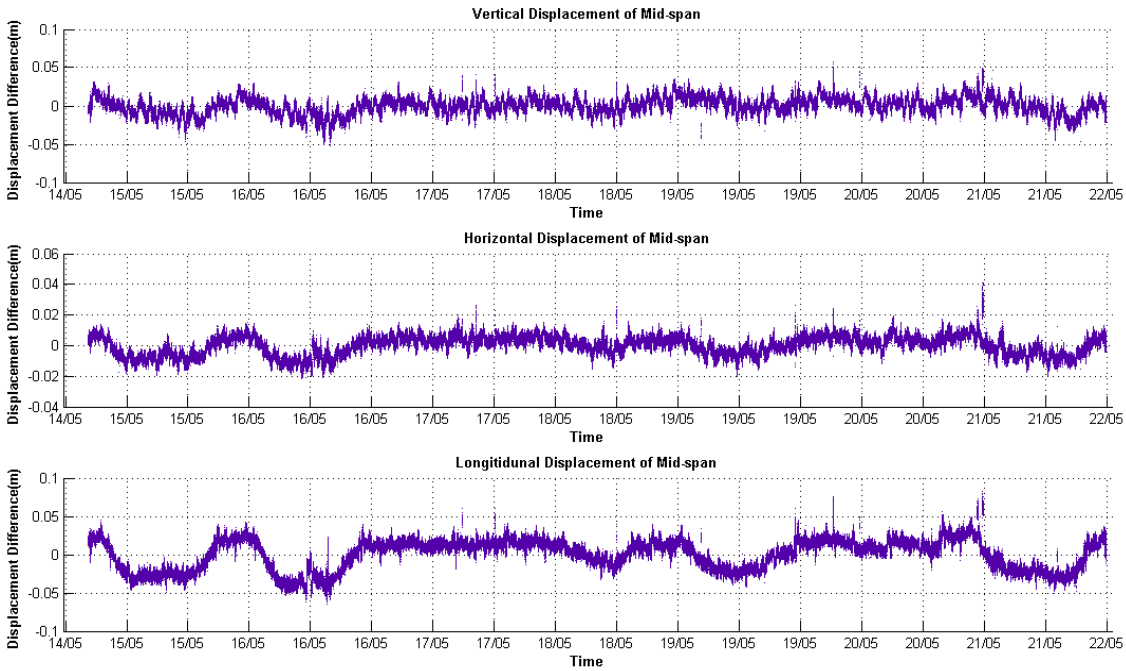


Fig. 4 Vertical, horizontal, and longitudinal relative movements of the mid-span of the bridge.

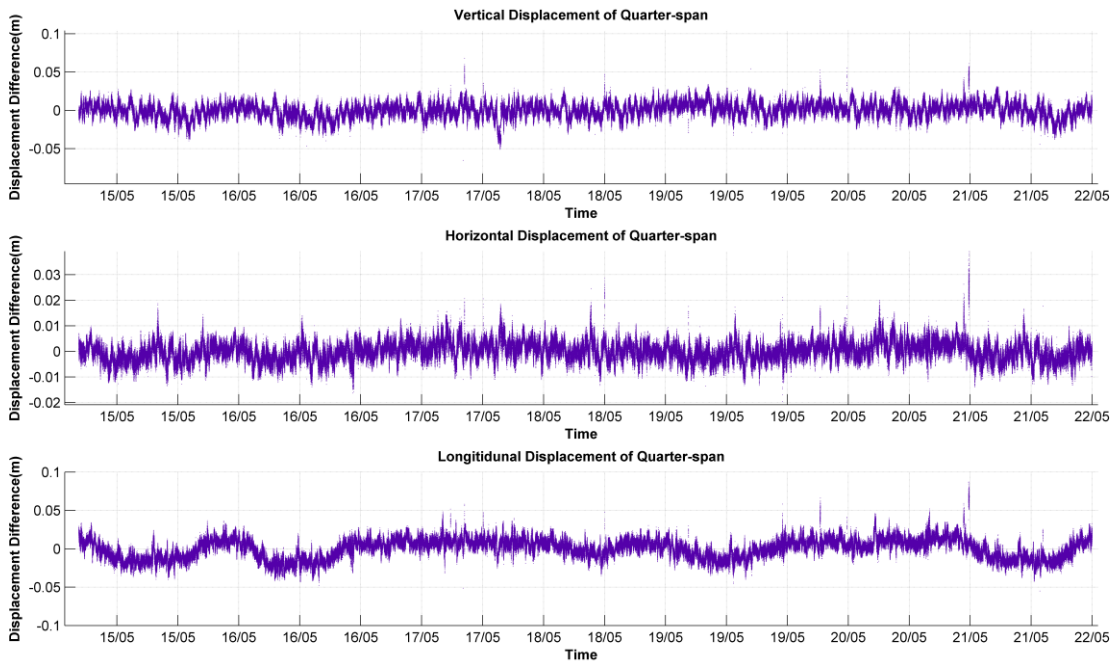


Fig. 5 Vertical, horizontal, and longitudinal relative movements of the quarter-span of the bridge

The GNSS measurements show the relative movements of the mid-span and the quarter-span of the bridge based on the reference antenna position. The great interest is in the relative vertical movements of the mid-span and the quarter-span of the bridge, which can be seen in Figure 4 and Figure 5. The magnitudes of the vertical and longitudinal relative movement were in the interval of ±0.05 m during the monitoring period. However, rapid fluctuations were observed in the vertical movements, while the longitudinal movements had slower fluctuations. Furthermore, Figure 4 and Figure 5 show that there were not abnormalities observed that could amplify the deformation of the bridge during the monitoring period. The horizontal movements of the mid-span and the quarter-span of the bridge were in the interval of ±0.02 m.

4.2 Weather station measurements

The weather station mounted on the top of the south-east pylon has records temperature, average wind speed, average wind direction and relative humidity.

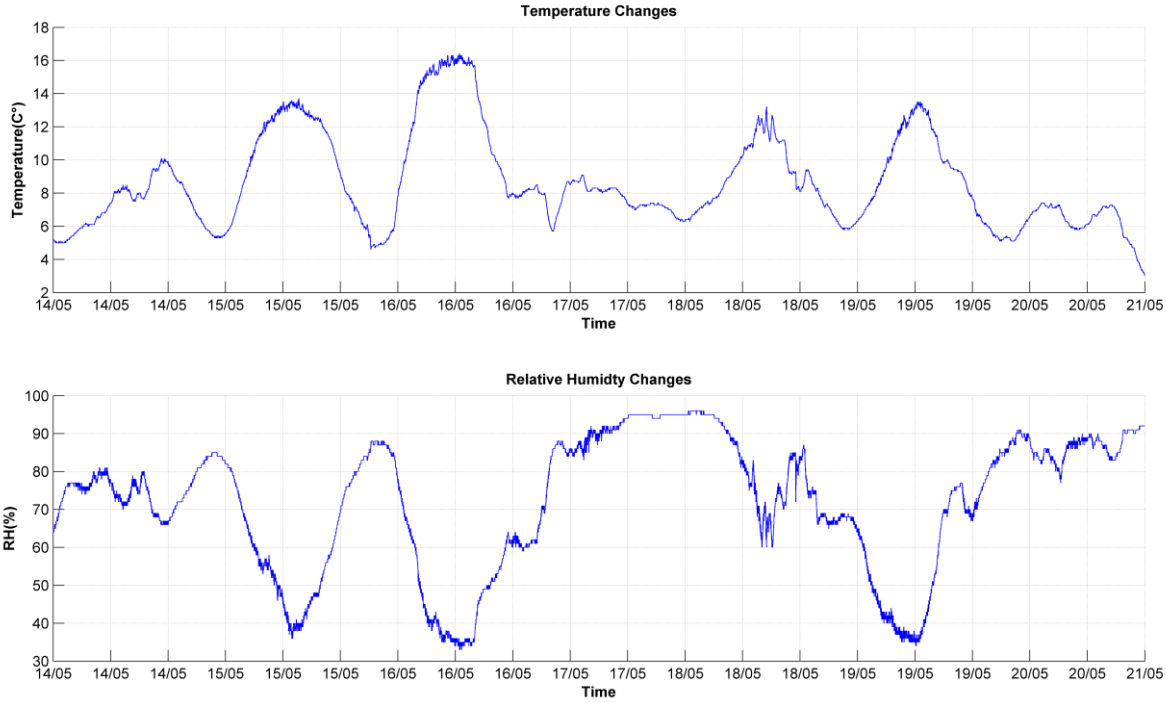


Fig. 6 Temperature and humidity changes during monitoring time

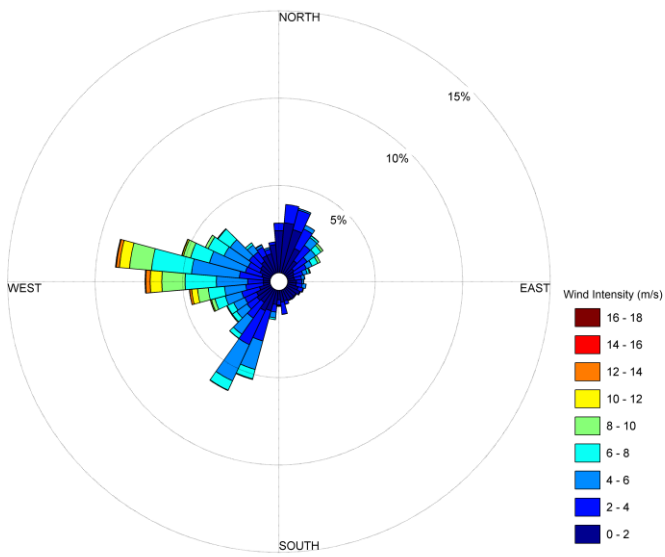


Fig. 7 Wind rose plot that shows wind direction and wind speed

Figure 6 and Figure 7 show the weather conditions measured by the Vaisala WXT 520 weather station during the monitoring period. The temperature range between 16° C and 3° C. Relative humidity varies between 95% and 35%. It should be noted that temperature and humidity measurements illustrate the air conditions around the bridge area, which can in fact differ from the temperature and humidity of the wooden structural elements themselves.

Figure 7 illustrates the wind speed and the wind directions. Wind intensity reached around 16-18 m/s and the wind direction was mostly from the western side of the *bridge*. Wind direction and wind speed measurements can be correlated with the bridge's horizontal movements.

4.3 Cable tension measurements

Custom-made force transducers recorded the stay-cable tensions during the monitoring period. Since these pre-tensioned cables are manufactured from steel and have a high thermal expansion coefficient, they have a dominant influence on the vertical movements of the bridge. In [8], it is reported that a 20° C temperature changes can cause a 1.5 mm axial deformation of a 15 m wooden pole. This means that the thermal expansion of the pylons has a negligible effect on the vertical movements of the bridge.

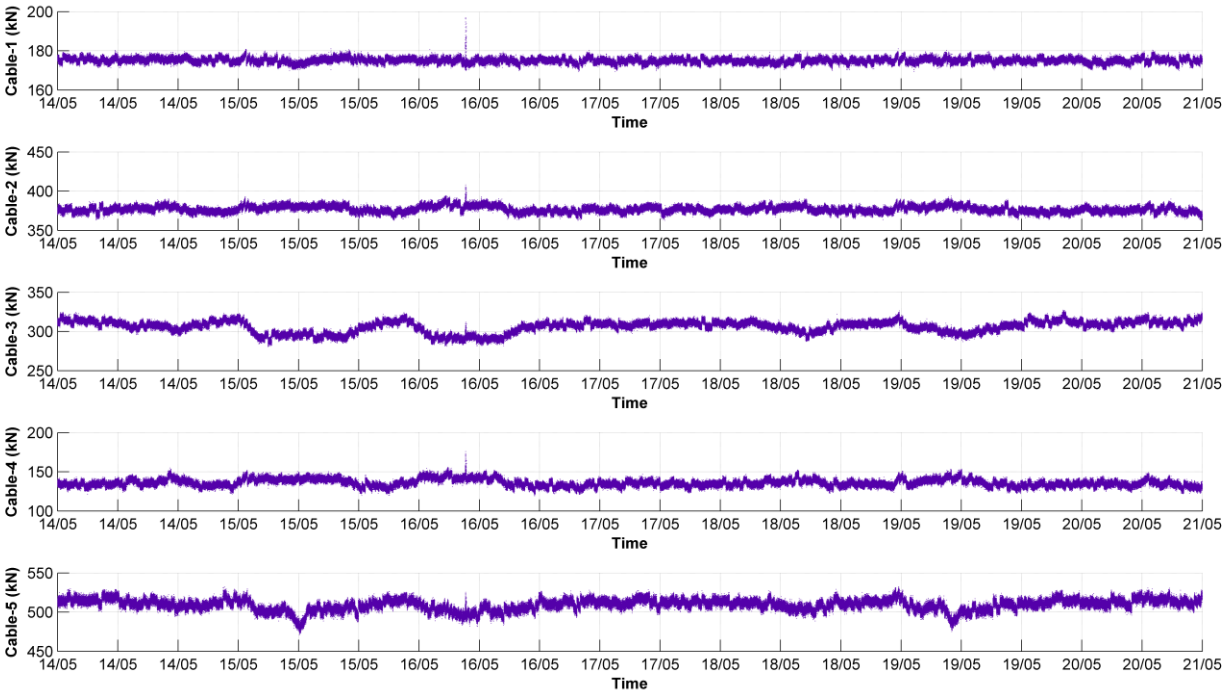


Fig. 8 Tension changes of pre-tensioned stay-cable during monitoring period

Figure 8 shows the change of tensions in the steel stay-cables. Thec cable numbers have the same numebering as illustrated in Figure 3. During the short-term monitoring period not any abnormal changes were observed in the stay-cables except for one instantaneous fluctuation which was observed on May 16 2012, which can be attributed to the restoring forces of the bearings being suddenly released. For the design conditions, these sudden releases are within the limits of the standards. However, these sudden reactions might cause secondary problems, such as restraints in the expansion joints [5].

4.4 Correlations

By using a synchronized monitoring system with different sensors, it is possible to perform short-term and instantaneous correlations between sensors that provide information about the response of the structure to changes in climate. If any abnormal changes occur during the monitoring time, these correlations deliver information on the reason for the abnormalitiesi indicating weather they are due to climate conditions or not. Furthermore, by monitoring the bridge in the short and long time, the structural design verification can be done. In this section, the one-day correlations between temperature changes and bridge vertical movements, and also between wind speed and bridge horizontal movements are presented. However, long-term monitoring provides a more holistic approach to the effect of climate on bridge movements.

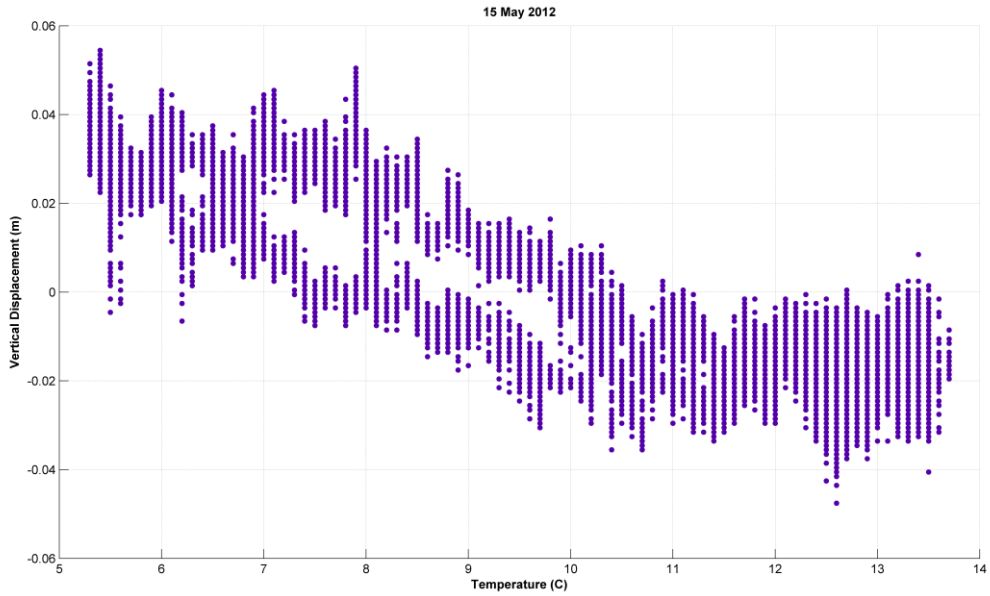


Fig. 9 Temperature changes influence on relative vertical movement of the mid-span bridge in a day.

Temperature changes have a clear effect on the stay-cables, as can be seen in Figure 9. Increase of temperature causes expansion of the steel cables so the mid-span of the bridge has negative displacement and vice versa. However, to investigate the permanent deformation of the bridge it is necessary to analyse long-term monitoring results. From Figure 9, it can be seen that about 5° C temperature change causes 0.05 m deformation of the mid-span of the bridge.

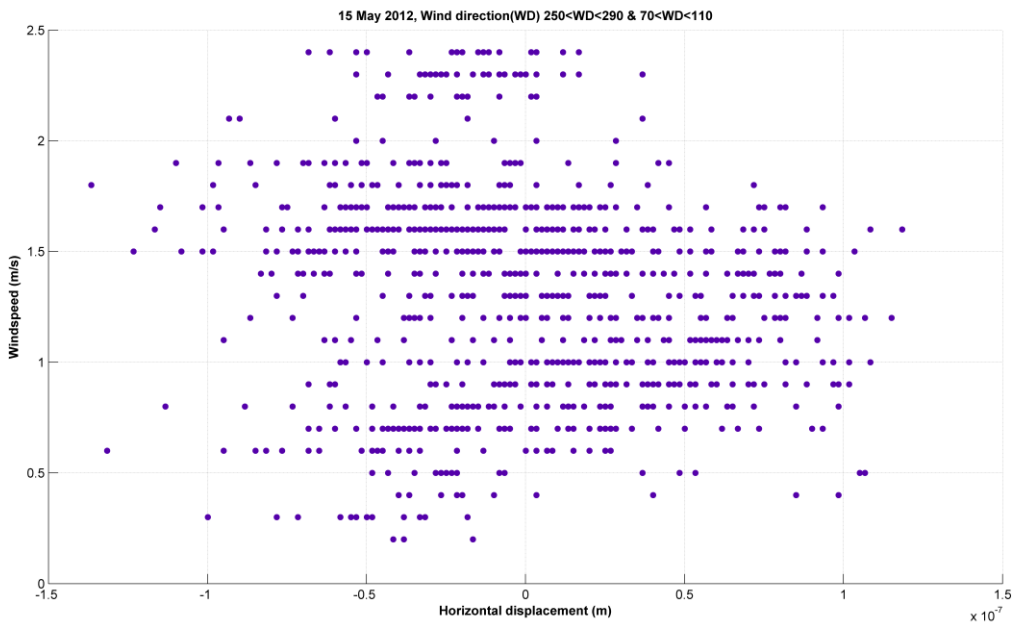


Fig. 10 Horizontal movements that is triggered by horizontal wind speed.

The weather station records the wind direction in terms of degrees, such that 0° is North, 90° is East, 180° is South and 270° degree is West. To investigate the horizontal wind speed effect on the horizontal movements of the bridge, wind directions between 250°-290° and 70°-110° were filtered out assuming that the filtered angles reflect the horizontal wind speed. Figure 10 shows the influence of the filtered wind speed on the horizontal movements of the bridge. Nevertheless, on 15 May, while the wind speed reached more than 10 m/s, the maximum horizontal wind speed reached only 2.5 m/s. The horizontal wind speed has a negligible effect on the horizontal movements of the bridge. Furthermore, the horizontal wind speed can only create a risk of torsional

oscillations when it exceeds 15.4 m/s, and also the actual mass moment of inertia of the bridge should be a certain value to trigger torsional oscillations. In order to investigate torsional oscillations induced by horizontal wind speed, it is necessary to perform long-term monitoring and investigate those days on which is the horizontal wind speed exceeds 15.4 m/s.

5. Conclusions

This paper presents the experimental short-term monitoring results of Älvsbacka cable-stayed timber footbridge. A monitoring system that consists of different types of sensors is introduced and the synchronization between the sensors is explained.

Analysis done by MATLAB commercial software shows that temperature changes have the dominant effect on the bridge's movement due to the steel cable's thermal expansion. However, to obtain the permenant vertical deformation of the bridge span, it is necessary to monitor the bridge over a longer period. Furthermore, based on the Eurocode standard, the design deflection criteria of L/400 is verified in the short-term monitoring period.

Horizontal wind force has a negligible effect on the bridge's horizontal movements. However, the wind speed measurements were recorded at the top of the south-east pylon which has a height of 23 m. Therefore, to increase the measurement accuracy of the effect of wind force, it is necessary to measure the wind from both the east and west sides of the bridge.

The short-term monitoring results show that no abnormalities occurred during the monitoring period. Furthermore, the measurement results are a good indicator of the bridge's structural condition.

Älvsbacka timber bridge is still being monitored to understand its behaviour and the effects of the climate in the long-term. Investigation of the bridge's movements in very cold climate conditions will provide the valuable information on the structural integrity of the bridge that can be feed-back for the next generation of timber bridges. It is planned to present the long-term monitoring results in future studies.

6. References

[1] Gustafsson A., Pousette A., and Björngrim N., "Health monitoring of timber bridges", International Conference on Timber Bridges (ICTB), Lillehammer, Norway, 2010.

[2] Björngrim N., Gustafsson A., Pousette A., and Hagman O., "Health monitoring of a cable-stayed timber footbridge", International Conference on Structural Health Monitoring of Timber Structures, Lisbon, Portugal, 2011.

[3] Feltrin G., Bischoff R., Meyer J., and Saukh O., "Wireless sensor networks: A new tool for structural monitoring", Empa, Swiss Federal Laboratories for Materials Science and Technology, Duebendos, Switzerland, 2011.

[4] Boller C., Chang F., and Fujino Y, "*Encyclopaedia of Structural Health Monitoring*", Wiley, Chichester, UK.

[5] Wenzel H., "*Health Monitoring of Bridges*", Wiley, Vienna, Austria, 2009.

[6] Leica Geosystems AG.*"User guide, Leica GNNS Spider v4.0, Reference stations*", Heerbrug, Switzerland 2010.

[7] Vaisala Oyj, "*User's guide, Vaisala weather transmitter, WXT520*",Helsinki, Finland, 2010.

[8] Pousette A., "*Cable Stayed Timber Bridges*", Nordic timber bridge project, Trätek Swedish Institute for Wood Technology Research, 1999, pp. 16

Influence of concrete cracking on wood concrete composite bridges

Frank Schanack
Prof. Dr. Ing.
Universidad Austral de Chile
Valdivia, Chile
frank.schanack@uach.cl

Frank Schanack studied Civil Engineering at TU-Dresden, Germany and received his doctorate degree at Universidad Cantabria in Spain.
Since 2008, he is professor at Universidad Austral de Chile where he is currently Director of the Civil Engineering School.

Summary

In certain cases the concrete slab of wood concrete composite beams can crack; deformations will increase and the internal forces will be redistributed. At present, the design recommendations available vary from ignoring the effect to reducing stiffness by 60%.

In order to characterize more correctly the influence of the concrete cracking on the force distribution in wood concrete composite beams a research project was conducted involving theoretical analysis and laboratory tests. First, a parameterized nonlinear FEM model was elaborated and calibrated by real test data available from the literature. Second, shear tests were conducted with test specimens that simulate the concrete cracks by a small separation between the concrete and the timber. The results indicate that the concrete cracking does not only reduce the stiffness of the concrete slab itself but also leads to a significant stiffness reduction of the shear connectors embedded in it.

Keywords: wood concrete composite, concrete cracking, timber bridge, Chile.

1. Introduction

Timber beams for composite bridges are an economical and ecologically sustainable alternative to the steel or pre-stressed concrete beams usually used in Chile. Over the last two decades, wood concrete composite bridges have been built in many countries and design recommendations on the assessment of the connection system have been incorporated into the design standards. However, when designing the first Chilean wood concrete composite bridge, it was found that the design recommendations for considering the effect of concrete cracking on the composite beam vary from ignoring the effect [1] to reducing stiffness by 60% [2]. In order to characterize more correctly the influence of the concrete cracking a research project was conducted, whose methodology and results are presented in this paper.

2. Bridge construction in Chile

2.1 General Overview

Historically, bridge construction in Chile was not very different from bridge construction in other parts of the world. Railway bridges were made of steel trusses and concrete arches, suspensions systems and concrete girders were used for road bridges. However, during the last three decades, the variety of structures for new bridges has been limited principally to simply supported beams made of:

- timber beams and a timber deck for provisional bridges with spans of up to 10 m
- steel beams and a timber deck for provisional bridges with spans of up to 30 m

- prefabricated RC beams and a concrete deck for bridges with spans of up to 20 m
- prefabricated PC beams and a concrete deck for bridges with spans of up to 30 m
- steel beams and a composite concrete deck for bridges with spans of up to 40 m

The large part of the new bridges being built in Chile are made of prefabricated beams (RC, PC or steel) with a composite concrete deck. Therefore, the introduction of wood concrete composite bridges, where the prefabricated beams are made of glued laminated timber, does not suppose a radical change in the construction technology and is interesting for the Chilean bridge authorities.

2.2 Timber bridges

The construction of timber bridges in Chile is standardized by the Ministry of Public Works (Ministerio de Obras Públicas), which publishes a Manual for road and bridge construction [3]. According to this manual, timber bridges are provisional bridges with a wooden deck and concrete abutments and one single traffic lane. The typical construction system for span lengths from 2.0 to 10.0 m is given in figures 1 and 2. The cross section of the longitudinal beams varies from 11”x11” to 21”x21”, depending on the span length. The wood species used are Chilean oak (Nothofagus obliqua) or Chilean beech (Nothofagus dombeyi), which are especially durable species. Furthermore, all wooden parts are protected by three layers of hot creosote. In the humid south of Chile (average annual precipitation of about 2500 mm), this type of bridge deck has a service time of only about 6 years.

In practice, about 40% of all Chilean bridges have an all timber deck and another 25% have a wooden deck on steel girders. That means that nearly two thirds of all Chilean bridges have wooden decks. Consequently, timber plays a decisive role in bridge construction in Chile and it is necessary to devise for innovative construction systems that increase the service life of these timber bridges.

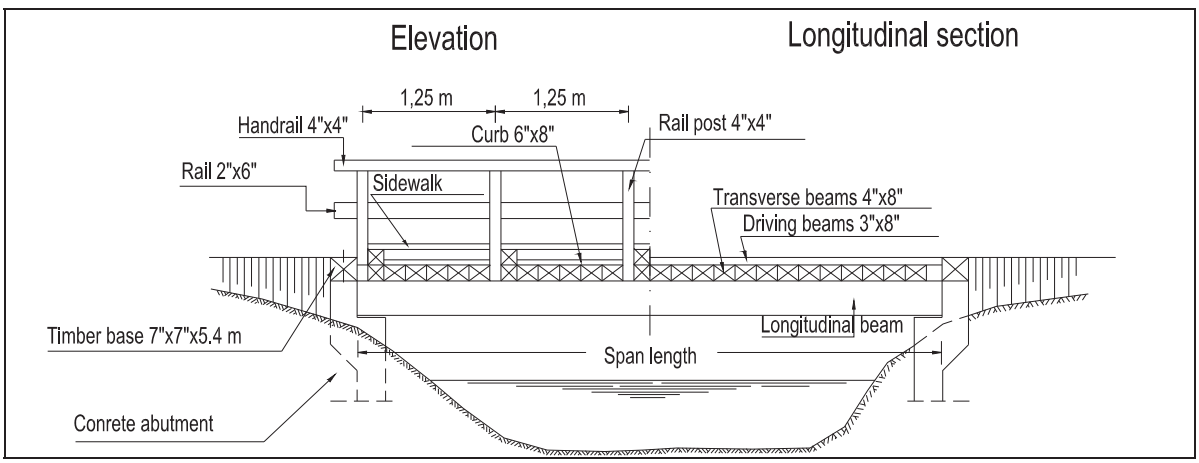


Fig. 1 Elevation and longitudinal section of a standard Chilean timber bridge [3]

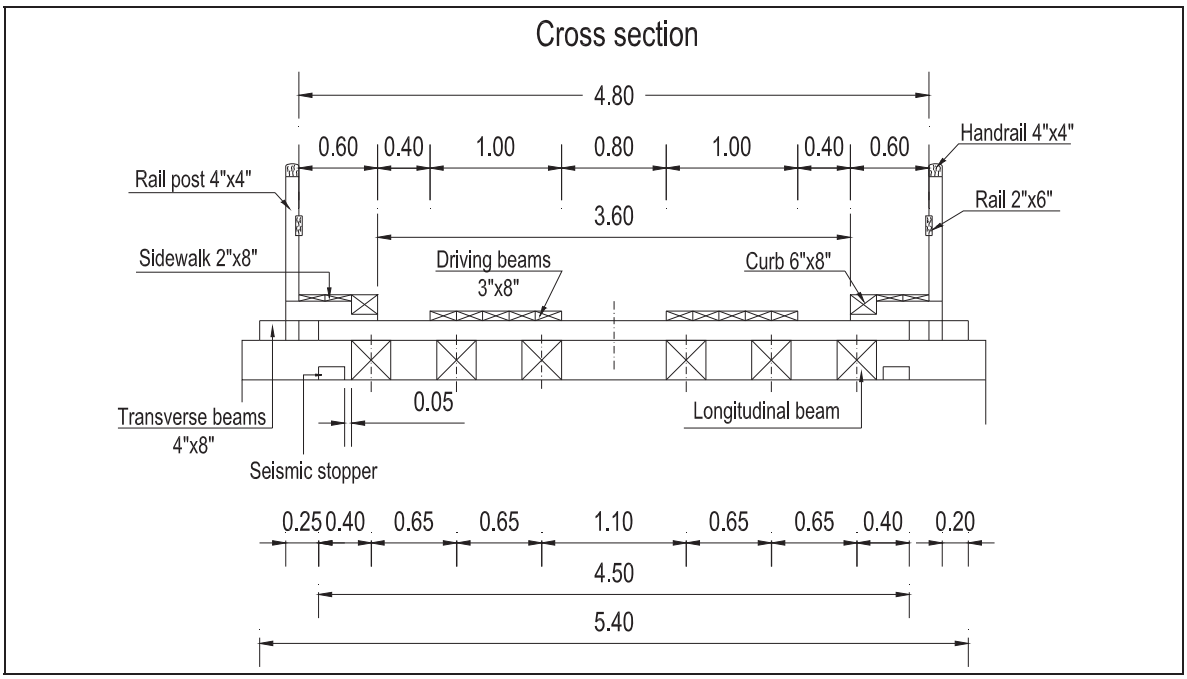


Fig. 2 Cross section of a standard Chilean timber bridge [3]

2.3 Ancahual Bridge

The first Chilean wood concrete composite bridge is going to be the Ancahual Bridge (Fig. 3), located in the state 'Región de la Araucania', close to the town of Loncoche. It is a simply supported girder bridge for two lanes of traffic with a span length of 14.38 m. It was designed with a 18 cm deep concrete slab of and 9 longitudinal glue laminated pine timber (Pinus radiata D. Don) beams. The timber beams are 14.0 m long, with a width of 0.25 m and 1.40 m in depth (Fig. 4).

The shear connector used between the concrete slab and the timber beams are ½"x6" wood screws. The main reason for the decision to use this type of connector was the requirement of the bridge authority to use existing design standards. The head of the ½"x6" wood screws is similar to stud connectors and can be designed as such, while the connection of wood screws to the timber beam is a standard design detail. There are two rows of wood screws on each beam; the longitudinal separation between the connectors is 10 cm. In order to increase the slip stiffness of the connection, the total number of wood screws is considerably higher than necessary to achieve the adequate ultimate load capacity.

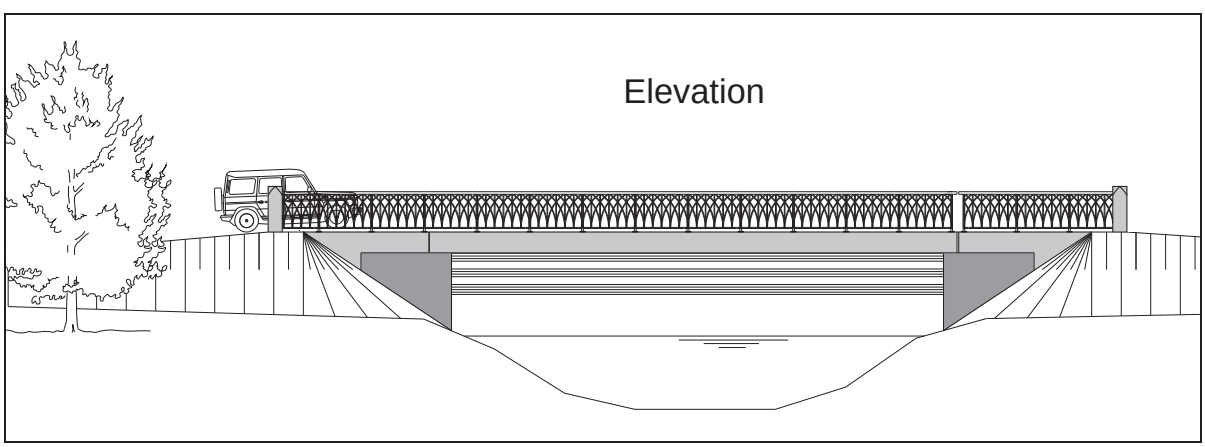


Fig. 3 Elevation of Ancahual Bridge

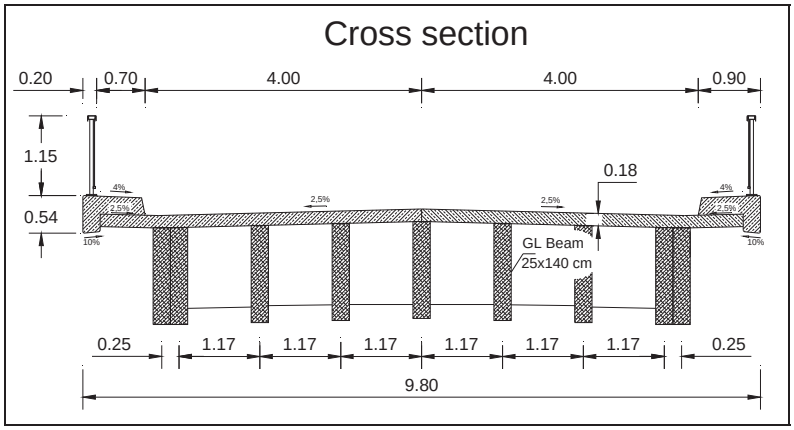


Fig. 4 Cross section of Ancahual Bridge

The Ancahual Bridge is planned to substitute an existing timber bridge at the same location, though currently only 5 m length. During the design process, the bridge length was gradually increased due to different requirements of the bridge authority. Also, the bridge width was increased to accommodate two lanes of traffic. This process provided the opportunity to study different geometries and depth ratios of decks and beams.

For certain geometry or stiffness combinations it was found that traction is produced at the bottom of the concrete slab. In such cases the concrete slab can crack; deformations increase and the internal forces are redistributed. For example, the deformation was found to increase by up to 30% and the bending moment in the timber beams was found to increase by up to 20%. As mentioned above, current design recommendations available from literature vary from ignoring the effect to reducing stiffness by 60%. In the Ancahual Bridge, the influence of concrete cracking was determined for each case by nonlinear finite element calculations, which may not always be cost-effective in the design of simply supported beam bridges. Consequently, a research project conducted at Universidad Austral de Chile aimed at obtaining more generally valid design recommendations.

3. Methodology

3.1 General approach

The research project is dived into three parts: first, existing laboratory tests of wood concrete composite beams, available in literature, are simulated in FEM calculations. The calculation model is preliminarily calibrated by means of the existing results. Second, laboratory tests of shear specimens are conducted in order to check the assumptions made in the preliminary calibrations. In a third and future step, the calculation model will be calibrated by conducting bending tests and a parametric study of a great variety of different geometries and stiffness ratios of wood concrete composite beams.

3.2 Simulation of existing laboratory tests

After a review of the existing publications of laboratory tests of wood concrete composite beams the following three studies were selected for the FEM simulation: Van der Linden 1999 [4], Yeoh 2010 [5], Klingenberg 2012 [6] and Ávila 2012 [7].

Van der Linden conducted 153 bending and shear tests with three types of connection. The first consist of pairs of wood screws, disposed at opposite angles of 45° to the vertical. The second type are steel plates nailed to the timber beam, and the third has notches in the timber beam, filled with concrete and anchored with vertical reinforcement bars. Figure 5 shows the original drawings of Van der Linden.

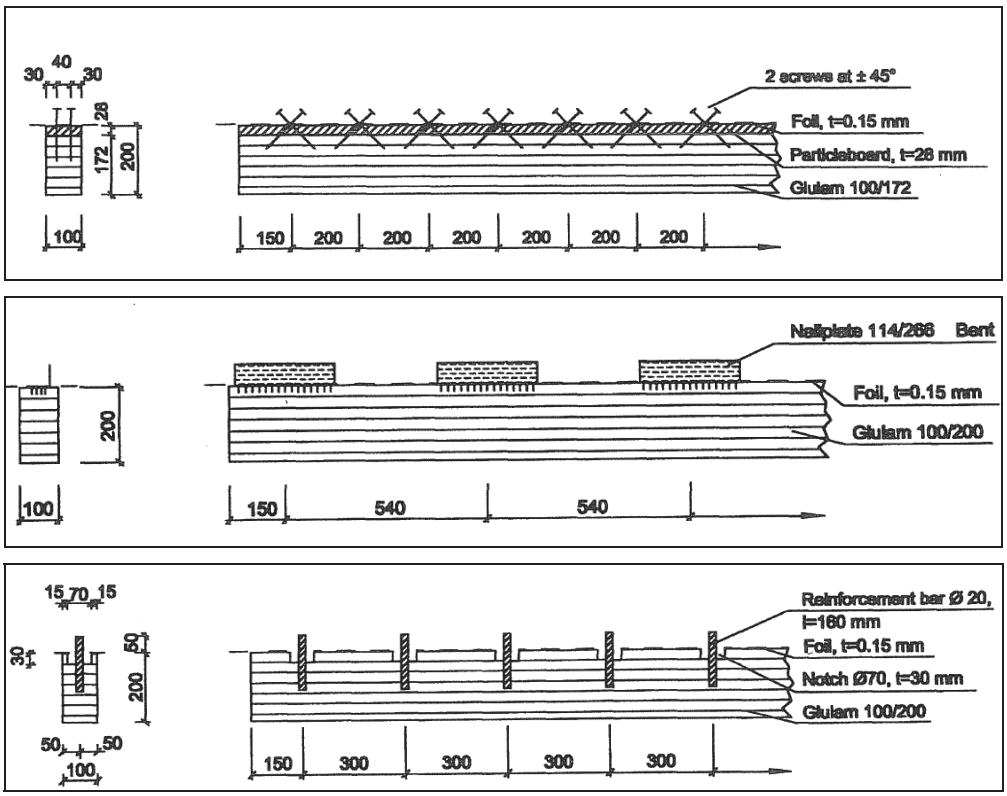


Fig. 5 Connection types used in the laboratory tests of Van der Linden [4]

In the work of Yeoh, the laboratory tests of 30 specimens are described. Different types of connectors were used, that include different notch forms with or without anchor bars and also vertical nail plates. Finally, the laboratory test conducted by Klingenberg and Ávila use ½”x6”wood screws for all tests.

In order to calibrate the finite element model and obtain simulated results as close to the laboratory test results as possible, a parameterized FEM model was elaborated. The parameters incorporated are listed in table 1. The numerical model is based on finite beam elements (Fig. 6). The timber beam and the concrete slab are modelled by 2 node Timoshenko beam elements with the corresponding mechanical properties. The actual reinforcement was assigned to the cross section of the concrete slab. The computer program used for the analysis was SOFiSTiK FEA under academic license.

The vertical contact between the concrete slab and the timber beam was modelled by a kinematic coupling of the vertical displacements of their nodes. The shear connection was modelled by discrete non-linear horizontal springs at the horizontal joint. The springs were connected rigidly to the centres of gravity of the concrete slab and timber beam. The non-linear stiffness behaviour of the springs was one of the parameters varied in order to obtain correlation with the documented results.

Table 1 Variable parameters of the FEM calculation model

Concrete slab	Timber beam	General
Depth	Depth	Time dependent load-displacement behaviour of the connection
Modulus of elasticity	Modulus of elasticity	Distribution of reinforcement
Creep	Creep	Type of loading and load history
Shrinkage	Shrinkage	Length and width
	Swelling	

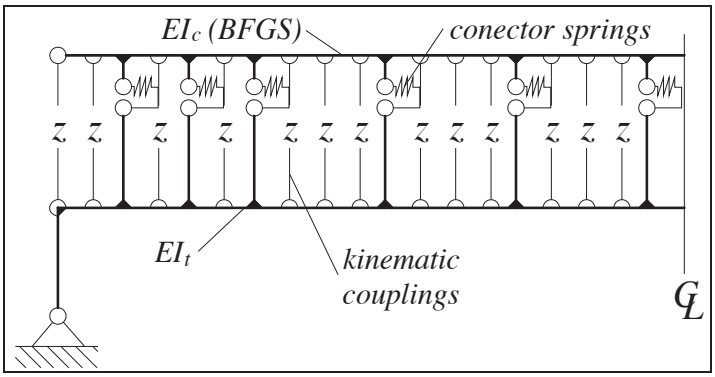


Fig. 6 Schematic view of the numerical analysis model

The non-linear iteration in order to find the force equilibrium between bending moment and curvature of the concrete slab was performed by BFGS (Broyden Fletcher Goldfarb−Shanno) procedure with line search, a generally accepted method for deformation determination in reinforced concrete structures. The stress-strain function of the concrete is adopted from the ACI318-08 [8], which is the valid standard in Chile.

3.3 Laboratory tests

The laboratory tests were designed as shear tests. In the shear test specimens, the cracked concrete was simulated by a reduction of the embedded length of the connectors in the concrete. This was achieved by a separation between the concrete and the timber of 0 mm, 2 mm, 4 mm and 6 mm. The shear test specimens, 3 of each type, have overall dimensions of length / width / depth of 0.6 m / 0.15 m / 0.3 m (Fig. 7).

Consequently, a total of 12 tests were conducted. In all test specimens ½”x6” wood screws at 10 cm spacing were used as shear connectors.

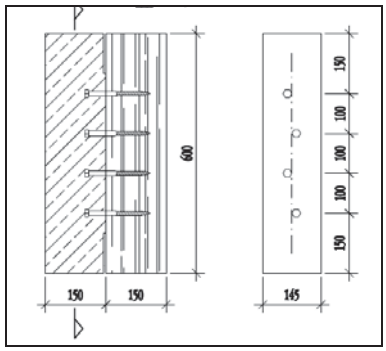


Fig. 7 Test configuration of shear tests

4. Results

4.1 Calibration of the FEM model

In a first step, the results documented in the literature were compared with those of the non-calibrate numerical model, which means, a numerical model was established with nonlinear material and connector properties in compliance with usual design recommendations. As an example, figure 8 shows this comparison for the data of Klingenberg and Van der Linden. As can be seen, in some cases a relatively high correlation was achieved, as in the case of Klingenberg. There, both curves are close together, but the FEM shows a greater stiffness in the elastic range and a less stiffness beyond this range. However, in the case of Van der Linden, there is low correlation between the initial MEF and the test results. There, the FEM model still shows elastic behavior, when the test results had already entered into the plastic range.

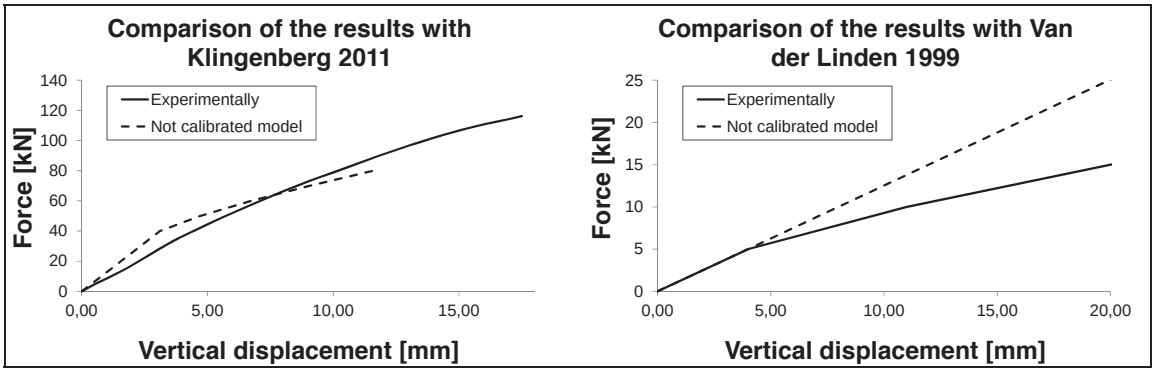


Fig. 8 Comparison of the results from the not calibrated model

The calibration process involved all of the parameters presented in table 1, but the most influential factors in the improving the simulation test correlation were the tensile strength of the concrete, the tension stiffening magnitude, as well as the stiffness and strength of the shear connectors. The decrease of any of these parameters led to an improvement in terms of model calibration.

The stiffness and the strength of the shear connector had the greatest influence. The corresponding theoretic values used for the calculation model were obtained from shear tests, while the examined cases were bending tests. There seems to be a significant difference in the connector behaviour between bending and shear tests. The difference might be due to the fact, that in bending specimens the connectors are embedded in the tensile zone of the concrete. The clamping of the connector to the concrete is less effective, especially if the concrete is cracked.

When applying these calibration concepts to the FEM model, the correlation between simulation and test results improved greatly. Figures 9 to 11 show the results of the calibration process.

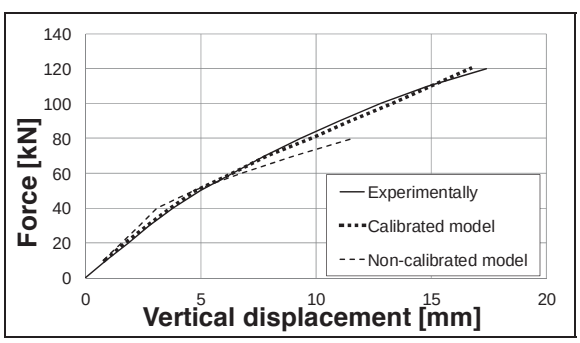


Fig. 9 Calibrated with Klingenberg 2011

The main change in order to obtain a better correlation with the results of Klingenberg was the reduction of the connector stiffness and strength by 50%. Furthermore, for the wood screw connector, the bilinear law proposed [2] is used.

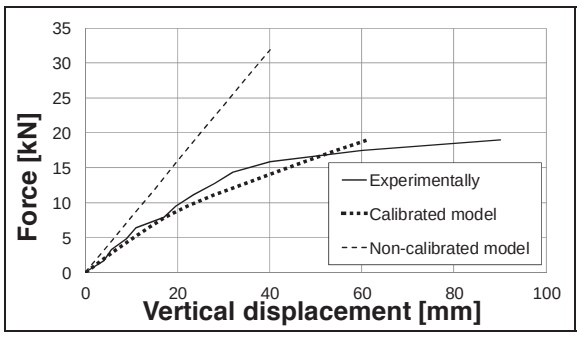


Fig. 10 Calibrated with Van der Linden 1999

Figure 10 shows the comparison of the non-calibrated and calibrated FEM model with the test results of oppositely inclined wood screws. The correlation between the simulation and the laboratory test is improved, when the connector stiffness is reduced by 50%. Also, the numerical model allows for concrete cracking. For loads bigger than about 20 kN the model diverges from the test result. In fact, the test beams seem to reach their ultimate capacity at this load level.

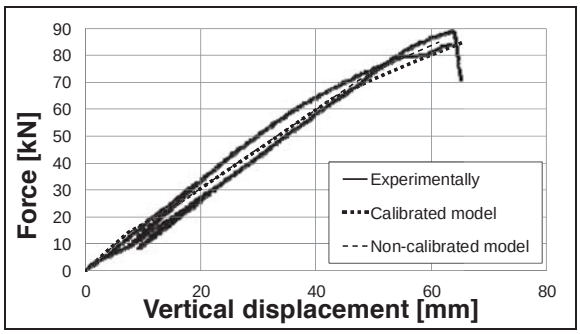


Fig. 11 Calibrated with Yeoh 2001

When the numerical model allows for concrete cracking, the results are very close to the test results of Yeoh (Fig. 11). Due to the notched shear connectors, there is no significant plastic behaviour before rupture. In the elastic range, the theoretic model has a high correlation with the laboratory tests. The use of the calibrated model, with 50% reduced connector stiffness, does not lead to an important improvement, nor does it lead to a poorer correlation.

4.2 Laboratory test results

As mentioned above, the shear test specimens have different separations between the concrete and the timber in order to simulate the effect of loss of embedding length of the shear connectors in case of concrete cracking. There is a supposed relation between the mean crack depth and the separation used in the tests, ruled by a factor of 2 to 5.

The relatively small separations used in the test, 2mm, 4 mm and 6 mm, cause a very significant stiffness loss in the shear connectors. On average, the elastic stiffness is reduced by 63%, 67% and 77% respectively (Fig 12). Considering these results, by conservative assumption cracks 10 mm deep could cause a stiffness reduction of 50% in connectors.

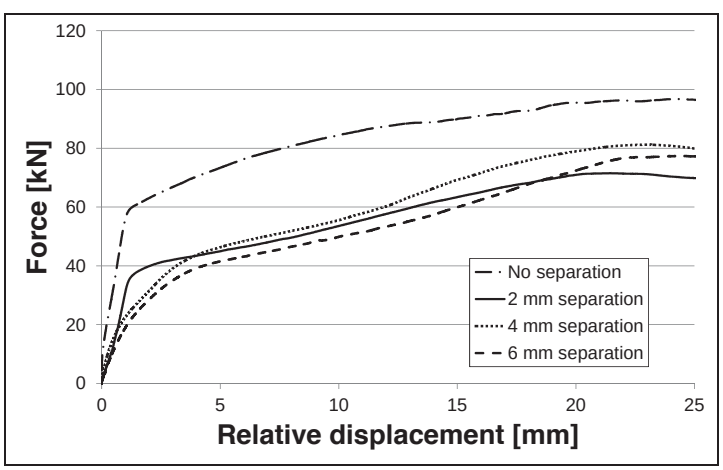


Fig. 12 Results from shear test with different separations between concrete and timber

5. Discussion

During the design of the first Chilean wood concrete composite bridge, it was found that certain timber concrete depth ratios and geometric proportions can cause traction on the bottom side of the concrete slab. However, the existing design recommendations do not give consistent advice. Therefore, this research project on the influence of concrete cracking on wood composite beams was conducted.

The theoretical part of the research aimed to numerically simulate real laboratory tests. The numerical model was calibrated by the results of laboratory tests found in the literature. Surprisingly, the consideration of the mere stiffness reduction of the concrete slab due to cracking has very little effect on the improvement of the correlation between simulated and real test results. For an effective calibration it was necessary to reduce additionally the stiffness of the shear connection by about 50%. This means that a concrete crack might weaken the embedding of the shear connectors in the concrete over the depth of the crack. This behaviour is likely similar to what occurs when there is a small separation between the concrete and the timber.

Indeed, in laboratory tests at the UACH of shear specimens with 2 mm, 4 mm and 6 mm separations it was found that the connection stiffness was reduced by up to 77%. These separations are very small and should have a similar effect as small cracks with depths between 10 to 15 mm.

6. Conclusions

The results from the FEM simulations and of the shear tests indicate that the influence of the concrete cracking is not limited to a stiffness reduction of the concrete slab. In order to calibrate the numerical model it was also necessary to reduce the stiffness of shear connectors. No other parameter was found to significantly improve the correlation between simulations and real tests. Laboratory tests validate that small concrete cracks can cause significant connector stiffness reductions.

In a further research step, laboratory tests of bending specimens should be conducted, with geometric proportions that are especially susceptible to concrete cracking in order to observe if the behaviour of the shear connector changes, and if there are bending cracks at the bottom surface of the concrete slab.

7. Acknowledgements

This research project was financed by CONICYT, grant n° 11110310 and sponsored by the Direction of Research and Development of the Universidad Austral de Chile.

8. References

[1] Deutsches Institut für Normung DIN *Entwurf, Berechnung und Bemessung von Holzbauwerken - Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau DIN 1052*, 2004, Berlin

[2] Comite Europeen de Normalisation CEN, *Eurocode 5: Design of timber structures - Part 1-1: General rules and rules for buildings*, 2006, Brussels

[3] Ministerio de Obras Públicas (MOP), “Manual de Carreteras”, Vol. 4, 4.603.001 and 4.603.003, 2012, pp. 322-324.

[4] van der Linden M. L. R., *Timber-Concrete Composite Floor System*, Technische Universiteit Delft, 1999.

[5] Yeoh D., *Behaviour and Design of Timber-Concrete Composite Floor System*, University of Canterbury, 2010.

[6] Ávila L., *Cálculo de un puente de vigas mixtas madera- hormigón basado en parámetros de diseño experimentales*, Universidad Austral de Chile, 2012.

[7] Klingenberg T., *Determinación experimental de Parámetros de Diseño para Puentes de Vigas Mixtas Madera-Hormigón*, Universidad Austral de Chile, 2012.

[8] American Concrete Institute ACI, *Building Code Requirements for Structural Concrete (ACI 318-08)*, 2008, MI, USA

Cable-stayed timber footbridge with two towers

I	**Carlito CALIL JUNIOR** **Prof. Dr.** **Laboratory of Woods and Wood Structure, School of Engineering of São Carlos, São Paulo University** **São Carlos, SP, Brazil** **calil@sc.usp.br**		Civil Engineering, Masters in Structural Engineering and PhD in Industrial Engineering from the Polytechnic University of Catalunya. Professor, Department of Structural Engineering operates mainly in Timber Structures, Wooden Bridges.
II	**Francisco Antonio ROCCO LAHR** **Prof. Dr.** **Laboratory of Woods and Wood Structure, School of Engineering of São Carlos, São Paulo University** **São Carlos, SP, Brazil** **frocco@sc.usp.br**		Civil Engineer, Masters in Structural Engineering and PhD in Structural Engineering from the São Paulo University. Professor, Department of Structural Engineering, operates mainly in Timber Structures, Wooden Bridges and Wood Derivatives.
III	**Antonio Alves DIAS** **Prof. Dr.** **Laboratory of Woods and Wood Structure, School of Engineering of São Carlos, São Paulo University** **São Carlos, SP, Brazil** **dias@sc.usp.br**		Civil Engineer, Masters in Structural Engineering and PhD in Structural Engineering from the São Paulo University. Professor, Department of Structural Engineering, operates mainly in Timber Structures, Wooden Bridges.
IV	**Pedro Gutemberg de Alcântara SEGUNDINHO** **Prof. Dr.** **Federal University of Espirito Santo, Center of Agricultural Sciences** **Alegre, ES, Brazil** **p_gutemberg2001@yahoo.com.br**		Civil Engineer, Masters in Structural Engineering and PhD in Structural Engineering from the São Paulo University. Professor, Department of Forestry and Wood Sciences, operates mainly in Timber Structures, Wooden Bridges.

Summary

This paper presents design aspects of cable-stayed timber footbridge with two towers of 6,10 m, 45 m long, located in Brotas, São Paulo, Brazil, built by the Municipality of this city, whose project was prepared by members of the Wood and Timber Structures Laboratory, Department of Structural Engineering, São Carlos Engineering School, São Paulo University. The structure of the cable-stayed timber footbridge consists of foundation and support made of concrete; principal beams in *Eucalyptus citriodora* CCA-treated in an autoclave poles; deck made of *Mezilaurus itauba* pieces, with the painting process against biological demand. Brazilian Standard Code to timber structures design NBR7190 presents all the guidelines for design of this structure. The cable-stayed timber footbridge shows the feasibility of using wood reforestation, particularly the species used in the main parts, as an excellent structural material, which has low energy consumption, optimum ratio strength/weight and durability when treated.

Keywords: Timber footbridge, stayed footbridge, planted forests, *Eucalyptus citriodora*.

1. Introduction

The Parque dos Saltos is the postcard of Brotas, municipality of São Paulo, Brazil, and reveals important environmental and tourism. Located in the urban area of the city, is crossed by a river that forms falls and rapids, where events are held periodically bound to practice adventure sports such as rafting and canoeing championships. The area is also of historical and architectural importance as it houses a building of a former power plant, currently disabled.

The crossing by pedestrians over the river enables hereby guyed wooden walkway, a second alternative walk this recreation area. The first alternative was possible river crossing through a metal suspension footbridge built next to power plant also for the workers to access the service site. Before disabling the generation of electricity and the subsequent transformation of the region into a public viewing area, there was a need to open up paths to allow access to various parts of the park. In this context the walkway was built, and the city opted for wood construction.

This paper presents aspects of the design and implementation of the cable-stayed footbridge wood Parque dos Saltos, comprising two towers of 6.10 m high, 45 m long, two side spans of 10 m. The footbridge was built with oversight of the Department of Planning of the City of Brotas, whose project was prepared by members of the Laboratory of Wood and Timber Structures, Department of Structural Engineering, School of Engineering of São Carlos, University of São Paulo.

The connections of this footbridge follow the trend of using appropriate elements of the industrialization process for timber structures. The wood used in the superstructure of *Eucalyptus citriodora* from planted forests and underwent CCA preservative treatment process for vacuum-pressure autoclave to ensure greater longevity insect repelling the attack. The wood *Mezifaurus itauba*, used on the board in the form of sawn pieces, was obtained from certified forest area and went through the process of painting in order to have a longer life.

2. Materials and Methods

The structure of the cable-stayed footbridge designed in wood Laboratory of Wood and Timber Structures consists of foundation and basis for support made of reinforced concrete, the pieces from *Eucalyptus citriodora* CCA treated in autoclave: two main towers with two masts each 36 cm in diameter and 6.10 m in height; locking the mast 30 cm in diameter and 2.6 m in length; stringers diameter of 30 cm with 10 me 7.5 m in length; beams 30 cm in diameter and 3.4 m in length; support the railing with 12 cm diameter and 2 m in length; banister railing about 12 cm in diameter and 5 m in length; closing the umbrella body with 8 cm diameter and 2 m long. The tray was made of sawn *Mezifaurus itauba* gauge 4 x 16 x 280 cm receiving painting process. The Dywidag steel bar 15 mm in diameter. Metal parts and special connections: plates, threaded rods, bolts, nails, steel bars galvanized and painted with fire.

3. Design

3.1 Geometrical characteristics of the footbridge

The structure of the footbridge shown in Figures 1-4 was designed as stayed structural system with two towers, two masts per tower. The structure of the central footbridge with stretch span of 25 m was designed with double stringers of 7.5 m and 10 m in length attached to tower. The side spans with span of 10 m were also designed with double stringers 10 m in length. Four out of every mast stays, two for each side, and you are one of the external is docked in the cradle supporting the footbridge to avoid tipping the tower to the inside of the river. The foundation was executed in reinforced concrete, whose masts were set: the foundation blocks.

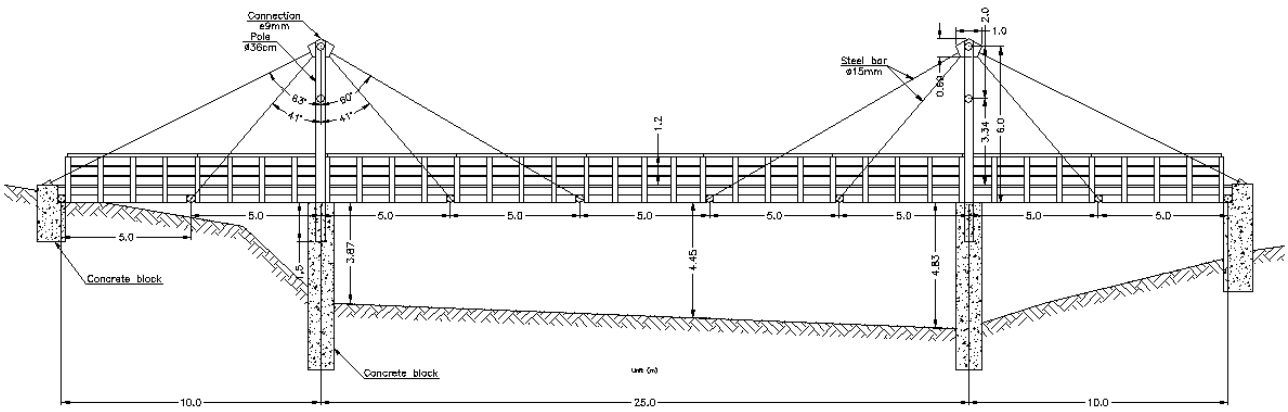


Fig. 1 Lateral view of the footbridge.

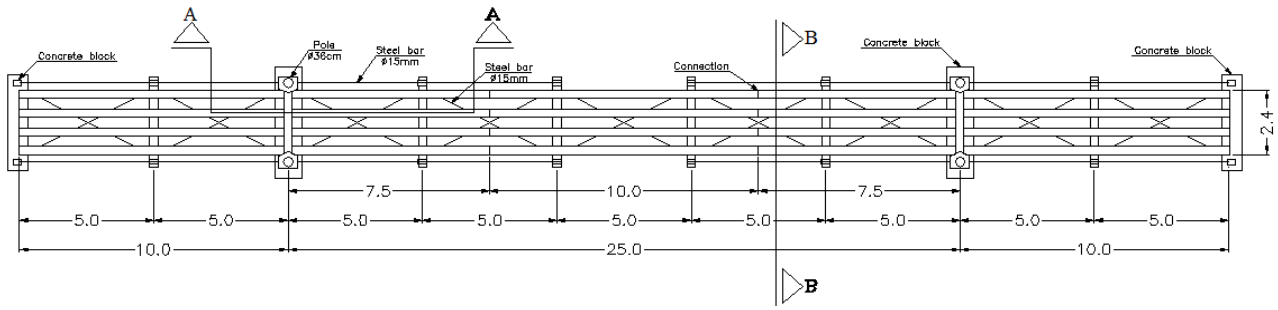


Fig. 2 Bottom view of the footbridge.

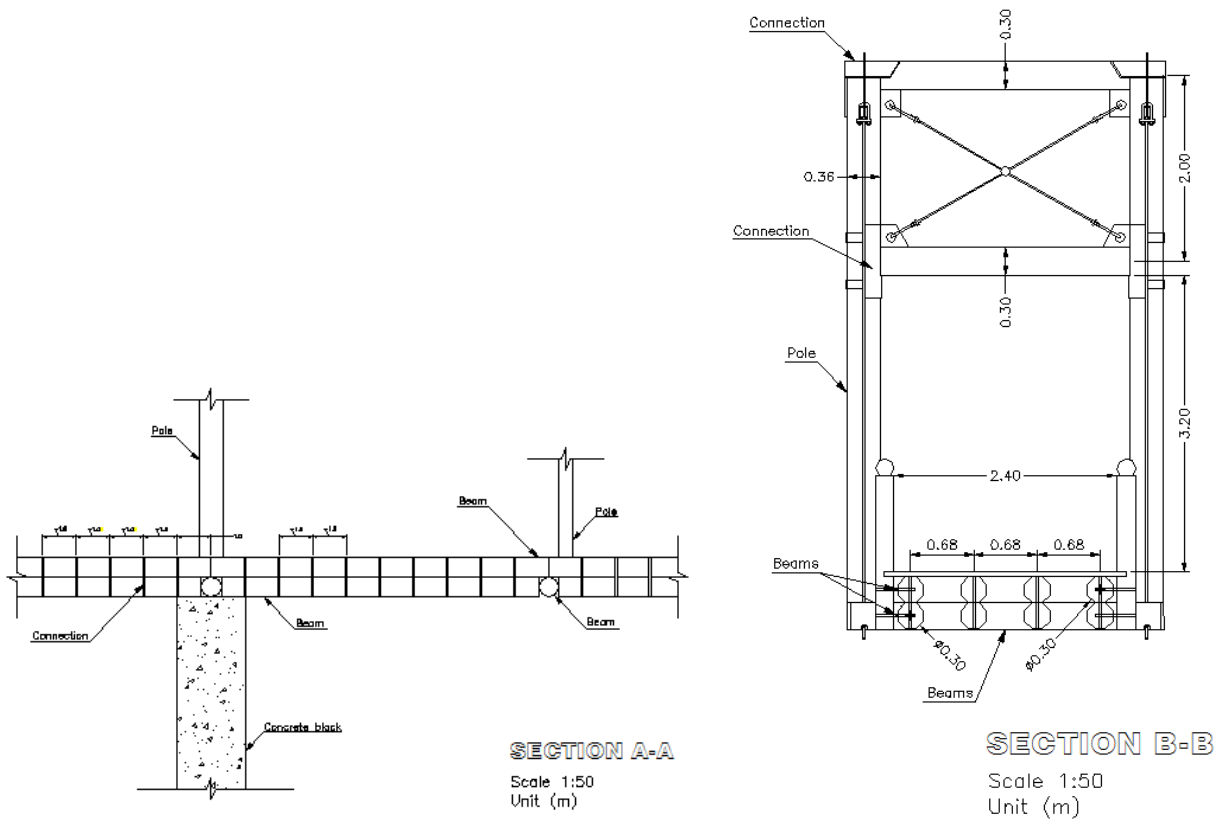


Fig. 3 Sections of the footbridge.

The connecting pieces, especially of the art in mast are designed in 3D graphical environment through the program AutoCAD®, in order to visualize how these parts would connection after finished, as shown in Figure 4.

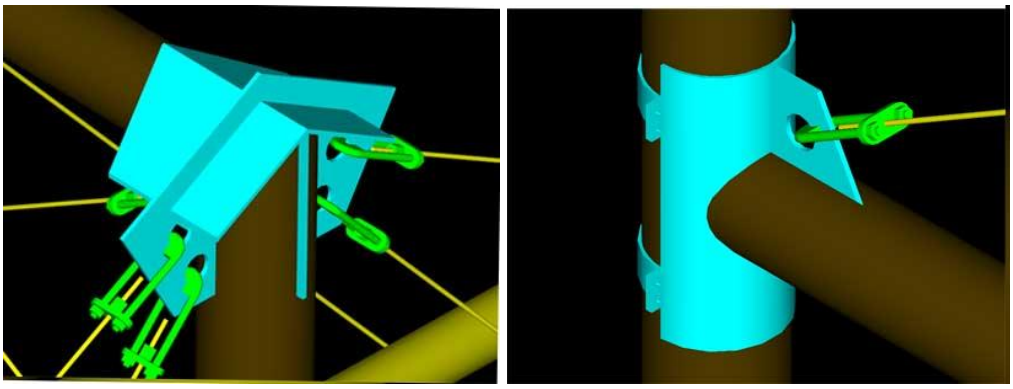


Fig. 4 Design of the footbridge links on the poles.

3.2 Actions

The building process contemplated situations loading in security checks, through the combinations given in standard Brazilian Actions and Security in Structures [1].

3.3 Structural analysis and design

The survey of efforts used in the preparation design were obtained through structural analysis program SAP2000® [2], based on finite element method. The secondary structural elements and were sized following the guidelines of the Brazilian Standard Design Timber Structures [3]. In Figure 5, for example, the removal of the efforts used in the design of the foundation.

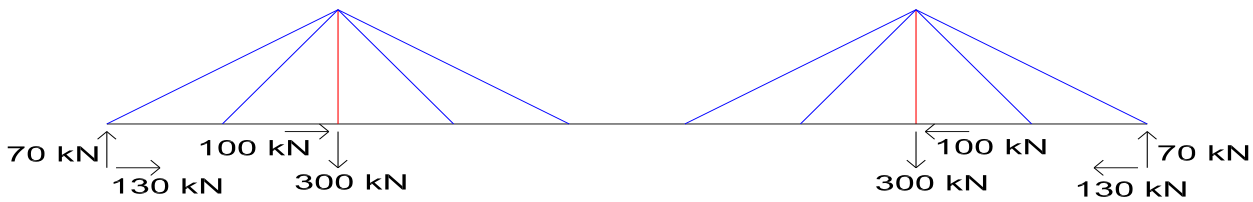


Fig. 5 Survey efforts used in the design of the foundation.

3.4 Footbridge constructed

In Figures 6-11 has photos of timber footbridge after construction. The execution of the footbridge was planned to allow the elements were made at ground level and positioned from the towers. After locking the two towers were set the stringers, the transversinas and stay. The horizontal rigidity of the footbridge is secured by bracing system of steel bars "X" set in the upper parts of the double stringers by means of bolts galvanized steel.

Fig. 6 Longitudinal view of the footbridge Parque dos Saltos, Brotas, SP.

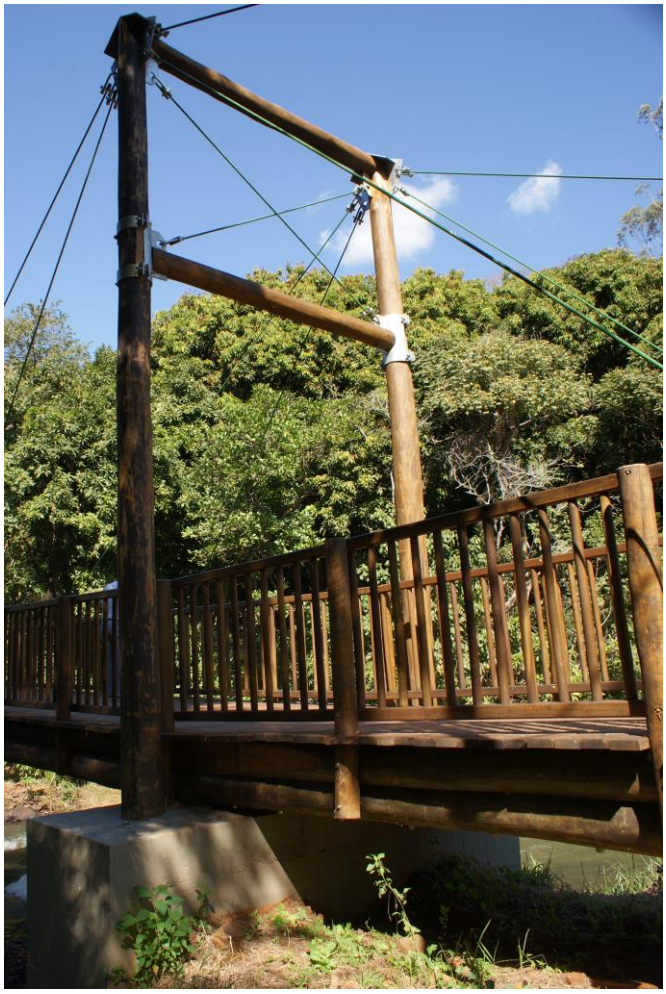


Fig. 7 Foundation block and a tower poles composing of the footbridge.

Fig. 8 Connects the poles of the footbridge.

Fig. 9 Connections are stayed the transversinas stringers and double pieces.

Fig. 10 Anchoring of you are in the cradle of external support.

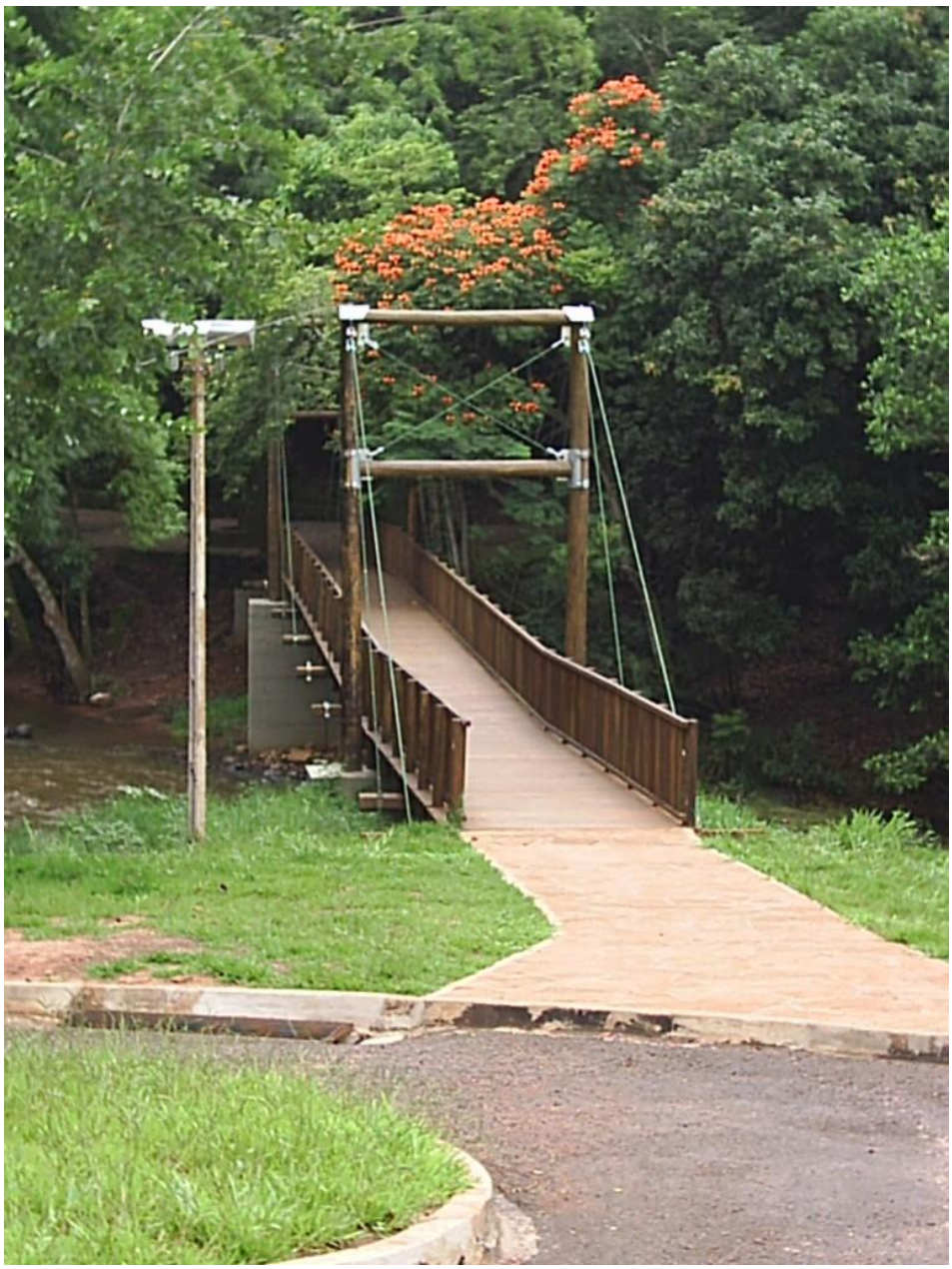


Fig. 11 Top view of the footbridge Parque dos Saltos, Brotas, SP.

4. Conclusions

The cable-stayed footbridge wood shows the feasibility of using wood from reforestation, renewable, particularly *Eucalyptus citriodora* as being an excellent structural material, which features low power consumption, great relationship weight / resistance and durability when treated.

The Brazilian Standard Design Timber Structures [3] presents all the concepts for the design and dimensioning of timber structures.

The structure was adequate to the environment, because the material used in the construction does not contrast with the proposal to the use of this public work. Undoubtedly, the wooden structures could be used in these projects.

5. References

[1] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS (1984). NBR 8681 – Ações e Segurança nas Estruturas, Rio de Janeiro.

[2] SAP2000 (2005). Integrated Finite Element Analysis and Design of Structures. Computers and Structures, Inc., Berkeley, California, USA.

[3] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS (1997). NBR 7190 – Projeto de Estruturas de Madeira, Rio de Janeiro.

Steien Network Arch Bridge

Johannes Veie
Senior Principle Engineer
Norwegian Public Roads Administration
Hamar, Norway
johannes.veie@vegvesen.no

M.sc. in structural engineering from NTH Trondheim 1996. Worked with steel and timber bridge planning since 2000 at the Norwegian Public Roads Administration.

Rune B Abrahamsen
Structural engineer Sweco Norge Lillehammer, Norway **rune@sweco.no**

M.sc. in structural engineering from NTH Trondheim 1995. Worked for Sweco since 1998. Group leader for bridges in Sweco.
Involved in the design of 60 timber bridges since 1996

Summary

The Steien bridge is situated in the county of Hedmark, Norway. A new network arch bridge will replace an old steel beam road bridge as well as a pedestrian bridge. The new bridge will have a single span length of 88 meters and arches that rise 15 meters. The bridge is located at Highway 3, which is one of the country's main roads. The bridge crosses Norway's largest river Glomma and will be a new landmark for the village Alvdal.

The new bridge will consist of laminated timber truss arches, network cable system and a reinforced prestressed concrete slab that also works as the tie for the arches. The network cable system allows for a significant reduction in bending moments in deck and arch, which leads to a structure of great slenderness and transparency. The new Eurocodes give the basis for the design, leading to comprehensive increase of load effects compared to earlier design codes. The structure will be assembled over the river, starting with casting of the reinforced prestressed slab and to be completed with the assembly of the timber arches and the wind truss. The construction on site is planned to start in the autumn of 2014 and finished within the year of 2015.

Keywords: Timber network arch bridge, concrete slab.

1. Introduction

Steien bridge is situated in Alvdal municipality. Alvdal is located in Hedmark County, 300 km north of the capital Oslo. The bridge is located on highway 3, which is one of the main roads connecting the east and mid-Norway. Daily traffic is about four thousand vehicles. The bridge, ref. figure 1, crosses Norway's largest river Glomma and will be a new landmark for the village Alvdal.

The new bridge replaces the old Steien road bridge built in 1953 as well as a parallel pedestrian bridge built in 1983. The bridge will be designed with traffic loads according to EN-1991-2 and with a 100-year design-life.

Fig. 1 The existing Steien bridge

The network arch bridge will be built with a record breaking span reaching 88 meters. The slab is made of reinforced prestressed concrete that also works as the tie for the arches. The slab carries the tension load which is in balance with the arch compression force. The vertical loads working on the slab are distributed to the arch through a total of 68 cables in the network cable system. One of the main benefits of using this type of bridge in this specific location is addressed to the slenderness of the deck structure. A normal beam bridge would have raised the road level considerably to overcome a 200 years flooding. The adjusted level would correspond to the height of the girder. The bridge carries two traffic lanes and two 3 meter wide pedestrian walkways.

2. Structural performance of the network arch bridge

A key feature of the network bridge is that it almost does not experience bending moments in the arch. This is the case as long as the majority of the hangers are in tension. The buckling strength in the arch is high, so is the stiffness regarding vertical deflection. The network system with inclined hangers allows for an efficient structural response which leads to very homogeneous hanger dimensioning. The cross section for the hangers is the same along the bridge. Figures 2 and 3 show the results from a comparison between the network arch concept for Steien bridge and a traditional 3-hinged arch bridge regarding moment distribution and vertical deflection due to asymmetric loading combined with self-weight of the structure.

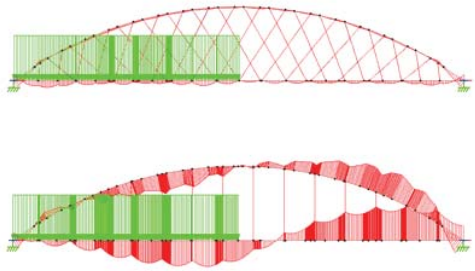


Fig. 2: Bending moment

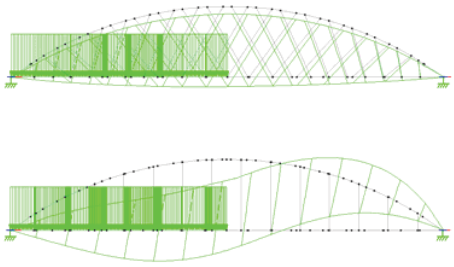


Fig. 3: Vertical deflection

For those who are interested in the general performance of network bridges, reference is made to the work done by dr. ing. Docent emeritus Per Tveit, found at http://home.uia.no/pert [1]. Reference is also made to work done by professor Kolbein Bell presented at IABSE conference in Lahti in Finland in 2001[2] addressing a design for a 70 meter span for a timber network arch bridge and another work presented at the ICTB2010 [3] conference in Lillehammer, Norway, which includes a proposal for a timber network arch bridge spanning 100 meter using a concrete slab.

After 50 years of development Tveit has come up with some suggestions to design the optimal network bridge:
The deck should be made out of concrete to give the necessary dead load to reduce the risk of relaxing hangers. The tie should be imbedded in the deck in the form of prestressing cables. This will decrease the tendency for the concrete to crack and thus increase the durability.
The arch should be made as part of a circle. This will give a more evenly distributed bending moment in the cords and will make the production easier.
The upper node of the hangers should be placed equidistantly along the arch and the hangers should not be merged in the nodal points. This gives a better support for the in plane buckling and reduces the bending moments.

3. Design for the new Steien bridge

Preliminary studies for a replacement of the existing Steien bridge showed that a network arch bridge is competitive with other relevant alternatives. Two other network arch bridges are built in Norway the last four years. Brandangersundet bridge with a main span of 220 meters and Aasnes bridge [4] with a main span of 111 meters. Before this, two network arch bridges were built in 1963 and 1964, both of them designed by dr. ing. Docent emeritus Per Tveit. All of them are made in steel and concrete. Brandangersundet bridge, the most slender arch bridge in the world, was of course a given example to compare against in terms of design. Brandangersundet bridge has the same type of slab working as a tie for the arch.
An experience from Brandangersundet bridge project was that although it was generally substantial savings in steel quantity, it was on the other hand, high installation costs. Although the bridge sites are not directly comparable, we chose a design philosophy that does not utilize the materials to their maximum in the operational phase, but rather provides some flexibility in the construction phase to reduce installation costs. Compared to arch bridges with vertical hangers the slenderness of the arches and tie is still extreme accounting for the use of a concrete slab instead of a much lighter stress laminated glulam deck.

Fig. 4: Steien bridge

Fig. 5: Steien bridge from the driver's view

Fig. 6: Steien bridge, 3D illustration

The new bridge showed in figures 4 to 6, will have a single span length of 88 meters and arches that rise 15 meters. The bridge carries two traffic lanes with a total width of 8 meters and two 3 meter wide pedestrian walkways. The total width of the slab is 19.8 meter.
The bridge is defined by arches made of rectangular glulam sections, slab made of prestressed concrete, wind bracings made of timber in combination with compression steel bars, and crossing hangers made of steel rods. The arches are inclined 7 degrees inwards.
The slab is made of in situ casted concrete with prestressing. The slab is casted in one stage. The hangers will be made of steel rods according to the ASDO tension rod system or likewise. The rods made of S550 steel are 48 millimeters in diameter and are delivered with adjustable fittings. The breaking load capacity is 795 kN.

3.1 Details

Figures 7, 8 and 9 show some typical details for the bridge

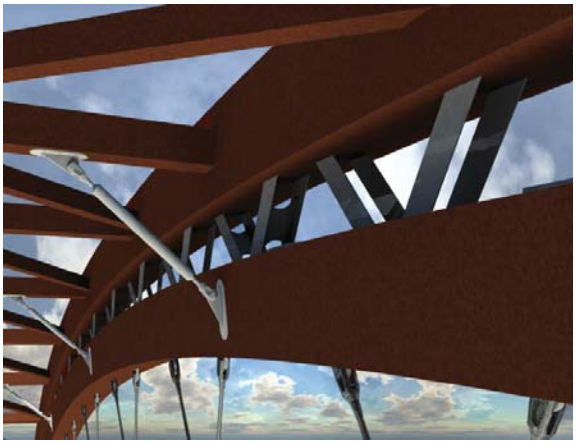


Fig. 7: Details of the truss arch

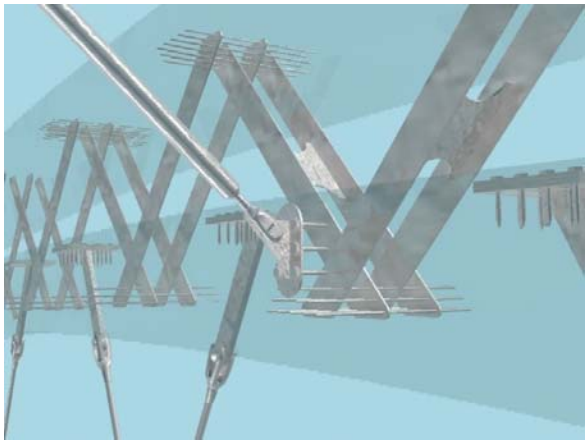


Fig. 8: Transparent view through the arch.

Fig. 9: Details of the hanger and slab connection

At the upper part the hangers are connected to steel plates that are going through the lower arch and are welded to a top steel plate that transfer the axial load from the hanger directly into the arch through compression perpendicular to the grain. In addition screws are used between the top plate and the arch in order to transfer shear loads between the steel and glulam. The diagonals in the truss arches are made of 10 mm thick stainless steel plates that are connected to the glulam with stainless D12 mm dowels. These steel plates have the same geometry throughout the arch structure.

At the lower part the hangers are anchored to the slab through a bolted connection as shown on figure 9.

4. Structural Analysis

The global analysis is performed with STAAD.pro. With the chosen cable layout it was found that there was an acceptable distribution of load distribution in the hangers without the need of individual stressing. In theory, if the overall geometry was in a perfect shape, the hangers could be produced with an initial unloaded length L0, which after installation did not have any need for adjustment. Since there are tolerances both for cable lengths and the arch assembly, there is a need to compensate for these deviations. For the ultimate limit state calculations the utilization of tension only elements in STAAD makes it possible to do a check of the impact of hangers going into compression. Some critical load combinations were selected for more comprehensive checking by non-linear analysis. For these calculations the initial geometry was given the shape of the first buckling modes, with the initial geometrical imperfection scaled to 0,0025L (but never less than 100 mm which is 3 times accumulated construction tolerances) in order to take residual stresses and geometrical imperfections into consideration. L is the length between the inflection points of the relevant buckling shape of the arch. In general for the code checking of the glulam sections a buckling length extracted from the first out of plane buckling mode was considered.

4.1 Traffic loading

The load combination cases that have been used are defined in NS-EN 1990.
Regarding global behavior due to traffic loading the only consideration has been taken to load model 1 according to NS-EN 1991-2, ref. fig. 10. Load model 1 comprises of two parts: one point-load part to simulate a heavy vehicle and one distributed part to simulate busy traffic.

Position	**Double axel load (BL)**	**Equal Distributed load**
	Axel load $\alpha_{Qi} \cdot Q_{ik}$ [kN]	**$\alpha_{qi} \cdot q_{ik}$ [kN/m^2]**
Loading area nr 1	1,0 · 300=300	0,6 · 9= 5,4
Loading area nr 2	1,0 · 200=200	1,0 · 2,5 = 2,5

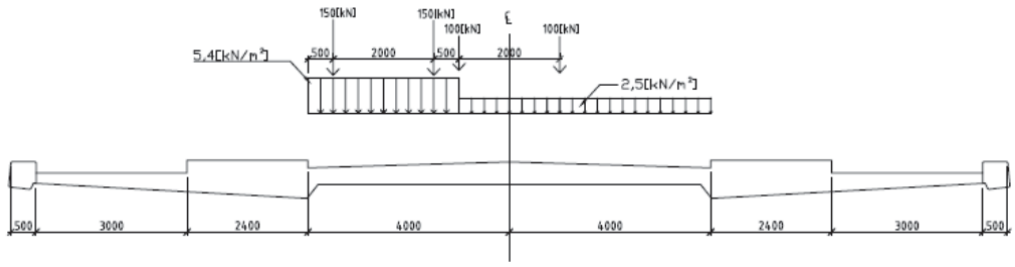


Fig. 10: Loadmodel 1 according t. NS-EN 1991-2:2003

Equally distributed load on the pedestrian carriageways are
q=5,0 kN/m^2 without simultaneous traffic loading
q=2,5 kN/m^2 with simultaneous traffic loading

4.2 Buckling analysis

The first buckling mode is shown on figure 11 and 12 for the case with self-weight. This mode gives basis for the non-linear analysis and also the basis for calculating the buckling factor, B_F. Three other buckling modes are also considered.

The buckling factor B_F (determined through an eigenvalue analysis) times the entire load combination (including the dead load) yields the (1st) buckling load.

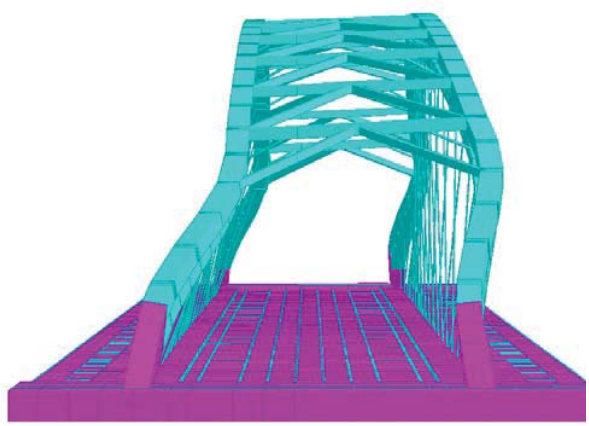


Fig. 11: Buckling mode 1

Introducing the classical formula;

$$P_{cr} = \pi^2 \frac{EI}{L_k^2}$$

for the Euler load we can estimate the buckling length to be:

Out of plane, $L_k = \sqrt{\frac{\pi^2 EI}{N_{max} B_F}} = 9.6$ meter

E = 11100N/mm2
I = 3.071E10 mm4
B_F = 7.993
Nmax = 4576 kN

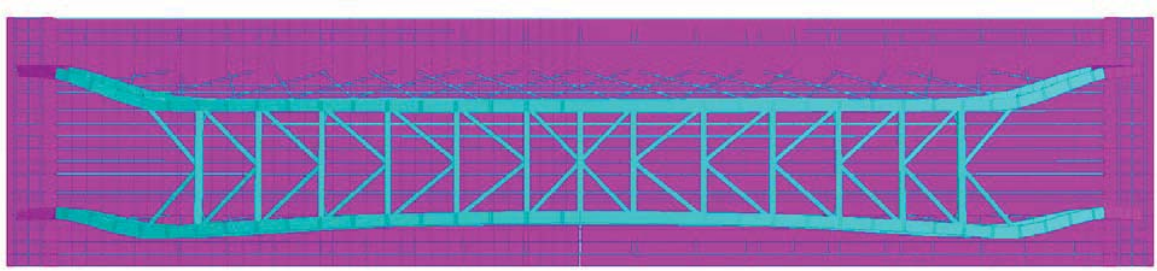


Fig.12: Buckling mode 1

Since the cross section for the glulam arches does not vary, the area around the wind portal will govern the design of the arches. This means that it is conservative to use L_k , derived from buckling mode 1, for all checking. This is also verified by the nonlinear analysis as described above.

4.3 Utilization

The wide slab generating high weight and combined with a reduced yield strength factor, k_{mod}, for the permanent load condition, the self-weight is governing for the design of the arches using normal concrete in the slab. In order to make the arches as slender as possible it is decided to use light weight aggregate in the concrete. This leads to that the utilization of the structure in the ULS condition with varying loads, corresponding to short time loading class and higher yield strength factor, is in the same magnitude as self-weight alone and permanent loading class.

5. Materials

The bridge consists of approximately 850 m^3 of concrete and 200 m^3 timber.
The cross section of one arch is designed with two massive 850 mm wide and 600 mm tall rectangular profiles forming a truss arch through steel diagonals.
The cross section of the slab is designed with a prestressed concrete slab with thickness varying from 800 to 250 mm.
The diagonals are designed as stainless steel profiles 200 x 10 mm connected to the timber with stainless steel dowels.
The cross sections of the wind truss are composed of square 400x400 and 300x300 glulam profiles and steel bars.
The inclined hangers are designed with D48 steel rods, S550 quality.

6. Durability

The Norwegian bridge design standards, issued by the Norwegian Public Roads Administration, require a service life of 100 years for all permanent bridges. Based on 20 years' experience on modern timber bridges we know were the possible weak points are. In general these are found in connection to stress laminated timber decks. For a typical timber bridge weight is critical, so stress laminated decks are normally preferred. In the case with network arches, the opposite is normally the issue. Because of this, it was natural to use concrete.
The protection of the arches is based on a combination of chemical treatment, metal cladding of exposed parts and good detailing. The wood treatment of the glulam members are carried out in a two-step process. First the individual lamellas are given a Cu treatment to a degree that corresponds to what is required for members in soil contact. The second step, a creosote treatment, is carried out after the glulam member is completely finished, including drilled holes and sawn slots. This allows for an optimum creosote penetration into the wood.

One millimeter thick copper sheeting will be used as metal cladding. Copper cladding of the top surface of the upper chord is quite easy, while covering the lower chord of the truss arch is not straightforward. The object is to cover horizontal surfaces as well as preventing water from running down the diagonals and into the slotted joints. In addition to wood treatment the slots will be sealed locally around the steel plates with a bitumen or epoxy layer. Horizontal plates will be connected to the lower part covering the slots for additional protection.

7. Fabrication and assembly

The brigde is planned to be assembled on site. A temporary bridge will handle the traffic after the demolition of the existing bridge. The abutments and slab are completed before the arch is mounted. The arch is planned delivered in 4 pre-assembled parts. The joints connecting the parts will not transfer bending moment. After the final assembly of the bridge the framework used for casting the deck is lowered and removed. Only a minor need for adjustments of the hangers should be necessary.
A significant challenge is how to ensure that the hangers are actually getting the defined tensile load and thus achieves the intended geometrical shape of the bridge. The behavior of the system is quite non-linear in terms of the effect of tensioning or de-tensioning of the hangers. Due to the system's non-linear behavior, a method involving tensioning of the cables in several stages would cause quite time-consuming analysis and complex tensioning procedures. A preferred solution is to produce the hangers in their final theoretical lengths in unstressed condition, and install them by jacking the slab up to a level where they could be installed in slack condition and then lower the slab again. Based on measurements to detect geometrical imperfections, the hangers are corrected before lowering the slab. The hangers are also checked after lowering to adjust any hangers if they are out of tolerances.

The tender for the construction works is due to be issued in the spring of 2014. The construction on site is planned to start in the autumn of 2014 and finished within the year of 2015.

8. References

[1] Tveit, P., 2011, „The Network Arch. Bits of Manuscript in February 2011 after Lectures in 50 Countries", http://home.uia.no/pert/index.php/The_Network_Arch

[2] Bell K. & Karlsrud E., 2001, „Large Glulam Arch Bridges - A Feasibility Study", IABSE2001, pp. 31-36.

[3] Bell K., 2010, „Structural systems for glulam arch bridge", ICTB2010, pp. 49-66.

[4] Veie J. & Gausen M., 2012,:"Åsnes network arch bridge; Nordic Steel Construction Conference 2012, Oslo, Norway

Opportunities for Timber Bridges in North America

Marshall Leslie, M. Leslie Inc., Canada
Jasmine Wang, Ph.D., P.Eng., Canadian Wood Council

Canadian Wood Council / Conseil canadien du bois

wood WORKS! Project of the Canadian Wood Council

Outline

- Timber Bridges in North America
- Opportunities for Timber Bridges
- Codes and Standards
- Conclusions and Discussions

Canadian Wood Council / Conseil canadien du bois

wood WORKS! Project of the Canadian Wood Council

Bridges in North America

– 70,000 in Canada and 710,000 in the U.S.

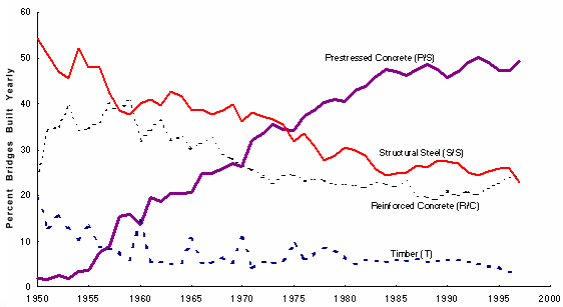


Figure 1 – U.S New Bridge Material Comparison, 1950-2000

Canadian Wood Council / Conseil canadien du bois — woodWORKS! Project of the Canadian Wood Council

Timber Bridges in the U.S.

- Total number of U.S. bridges is 712,313
- Of which the number of timber bridges is 24,599 (3.4%)
- And the number of timber decked bridges is 45,985 (6.4%)

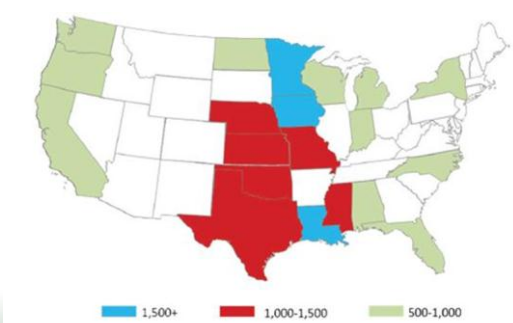


Figure 2 - U.S. States With More than 500 Timber Bridges, December 2010

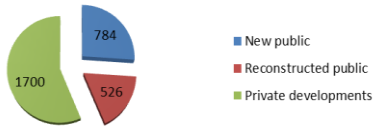
Source: US Federal Highway Administration's National Bridge Inventory

Canadian Wood Council / Conseil canadien du bois — woodWORKS! Project of the Canadian Wood Council

Table 1 – Top 10 U.S. State Timber Bridges, New & Reconstructed, 2001-2011

	New	Reconstructed	Total
Iowa	183	67	250
Texas	103	23	126
Indiana	83	14	97
New York	63	31	94
Florida	58	23	81
West Virginia	76	1	77
Michigan	48	27	75
North Carolina	70	5	75
Ohio	48	21	69
Louisiana	55	9	64

Figure - U.S. Timber Bridge Market Segments, 2001-2011



Canadian Wood Council Conseil canadien du bois

woodWORKS! Project of the Canadian Wood Council

Table 2 - U.S. Timber Bridge Structural Evaluation

	Number	Percent
Bridge closed	1,098	4.5
Basically intolerable requiring high priority of replacement	4,930	20.0
Basically intolerable requiring high priority of corrective action	1,870	7.6
Meets minimum tolerable limits to be left in place as is	4,573	18.6
Somewhat better than minimum adequacy to tolerate being left in place as is	4,220	17.2
Equal to present minimum criteria	3,840	15.6
Better than present minimum criteria	2,272	9.2
Equal to present desirable criteria	822	3.3
Superior to present desirable criteria	71	.3
Total	23,696	96.3
Missing	903	3.7
TOTAL TIMBER BRIDGES	24,599	100.0

Canadian Wood Council Conseil canadien du bois

woodWORKS! Project of the Canadian Wood Council

The Profile of New Timber Bridges

	Number	Percent
State Highway Agency	24	3.1
County Highway Agency	584	74.5
Town or Township Highway Agency	38	4.8
City or Municipal Highway Agency	45	5.7
State Park, Forest, or Reservation Agency	18	2.3
Other State Agencies	2	.3
Private (other than railroad)	1	.1
Railroad	1	.1
Bureau of Fish and Wildlife	1	.1
U.S. Forest Service	46	5.9
National Park Service	2	.3
Army	17	2.2
Total	779	99.4
Missing	5	.6
Total bridges	784	100.0

Table 3 – U.S. New Timber Bridge Ownership & Maintenance, 2001-2011

	Number	Percent
One	182	23.2
Two	596	76.0
Three or more	2	.3
Total	780	99.5
Missing	4	.5
Total bridges	784	100.0

Table 4 – U.S. New Timber Bridge Number of Lanes, 2001-2011

Canadian Wood Council / Conseil canadien du bois

woodWORKS! Project of the Canadian Wood Council

The Profile of New Timber Bridges

	Number	Percent
Other or Unknown	252	32.1
M 9 / H 10	44	5.6
M 13.5 / H 15	41	5.2
MS 13.5 / HS 15	2	.3
M 18 / H 20	45	5.7
MS 18 / HS 20	306	39.0
MS 18+Mod / HS 20+Mod	20	2.6
MS 22.5 / HS 25	67	8.5
Total	777	99.1
Missing	7	.9
Total Bridges	784	100.0

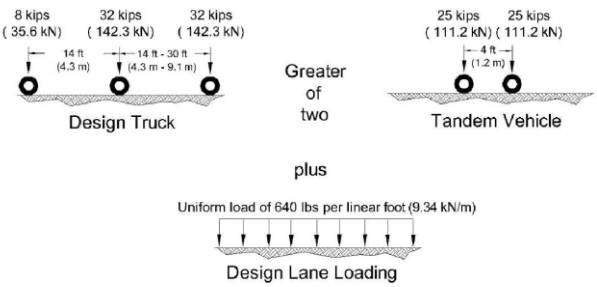
Table 5 – U.S. New Timber Bridge Design Load, 2001-2011

	Number	Percent
Slab	311	39.7
Stringer/Multi-beam or girder	434	55.4
Truss - Thru	18	2.3
Other	21	2.7
Total bridges	784	100.0

Table 6 – U.S. New Timber Bridge Type of Design/Construction, 2001-2011

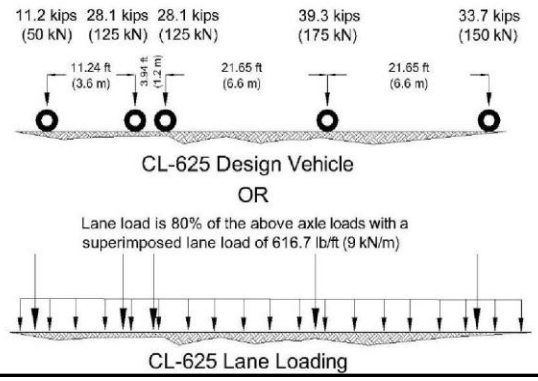
Canadian Wood Council / Conseil canadien du bois

woodWORKS! Project of the Canadian Wood Council



Truck/lane loading configuration in USA

Truck/lane loading configuration in Canada



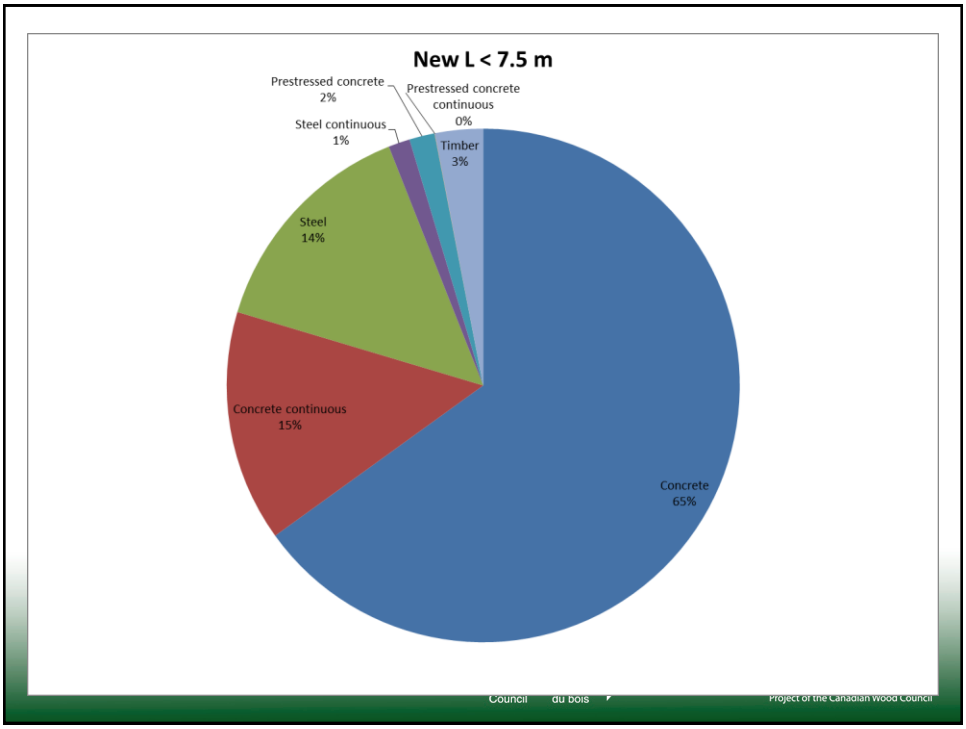
Source: Groenier & Wacker, 2008

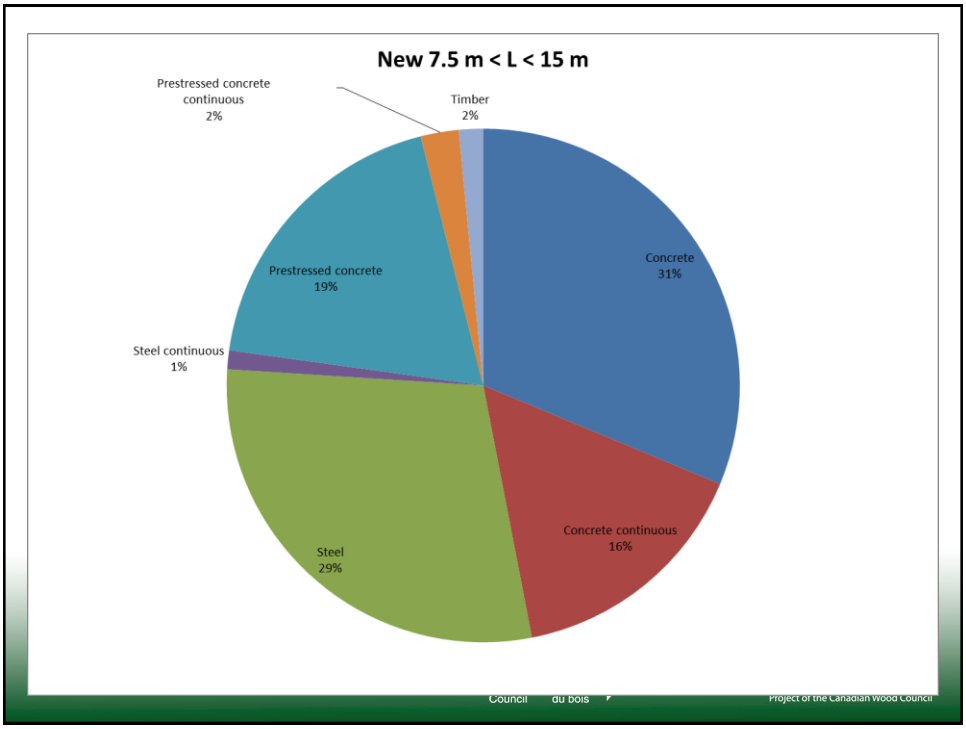
Table 7 – New U.S. County Bridges, Design Load MS18/HS20, Maximum Span Length & Material, 2001-2011

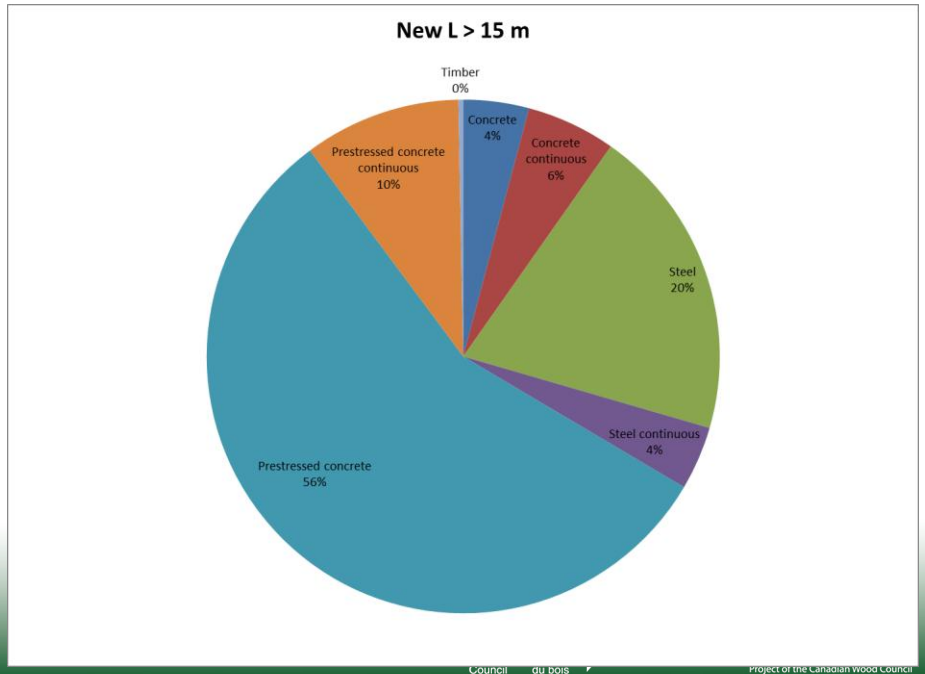
		Type of Material						
		Concrete	Concrete continuous	Steel	Steel continuous	Prestress-ed concrete	Prestressed concrete continuous	Timber
Length of maxim-um span	Less than 7.5	3211	719	710	67	79	1	149
	7.5 to less than 15	1825	915	1695	69	1102	140	89
	15 and greater	191	260	912	185	2602	457	14
Total bridges		5227	1894	3317	321	3783	598	252

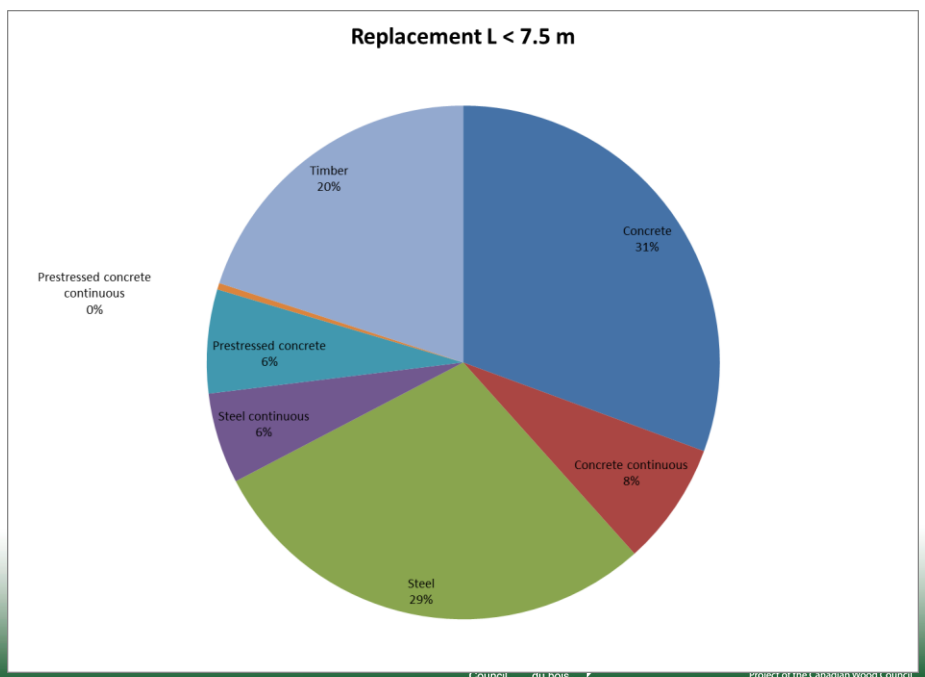
Canadian Wood Council Conseil canadien du bois

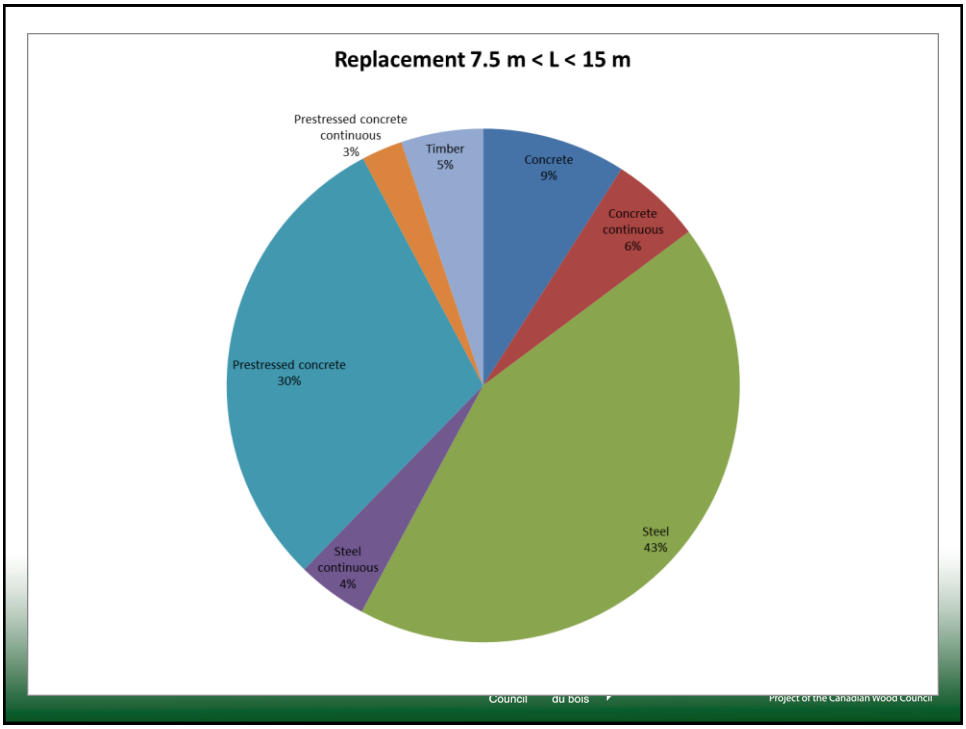
woodWORKS! Project of the Canadian Wood Council


New L < 7.5 m
Prestressed concrete 2%
Prestressed concrete continuous 0%
Steel continuous 1%
Timber 3%
Steel 14%
Concrete continuous 15%
Concrete 65%
Council du bois
Project of the Canadian Wood Council


New 7.5 m < L < 15 m
Prestressed concrete continuous 2%
Timber 2%
Concrete 31%
Prestressed concrete 19%
Steel continuous 1%
Concrete continuous 16%
Steel 29%
Council du bois
Project of the Canadian Wood Council


New L > 15 m
Timber
0%
Concrete
4%
Concrete
continuous
6%
Steel
20%
Steel continuous
4%
Prestressed concrete
56%
Prestressed concrete
continuous
10%


Replacement L < 7.5 m
Timber
20%
Concrete
31%
Prestressed concrete
continuous
0%
Prestressed concrete
6%
Steel continuous
6%
Concrete continuous
8%
Steel
29%


Replacement 7.5 m < L < 15 m
Prestressed concrete continuous 3%
Timber 5%
Concrete 9%
Concrete continuous 6%
Prestressed concrete 30%
Steel 43%
Steel continuous 4%
Council du bois
Project of the Canadian Wood Council

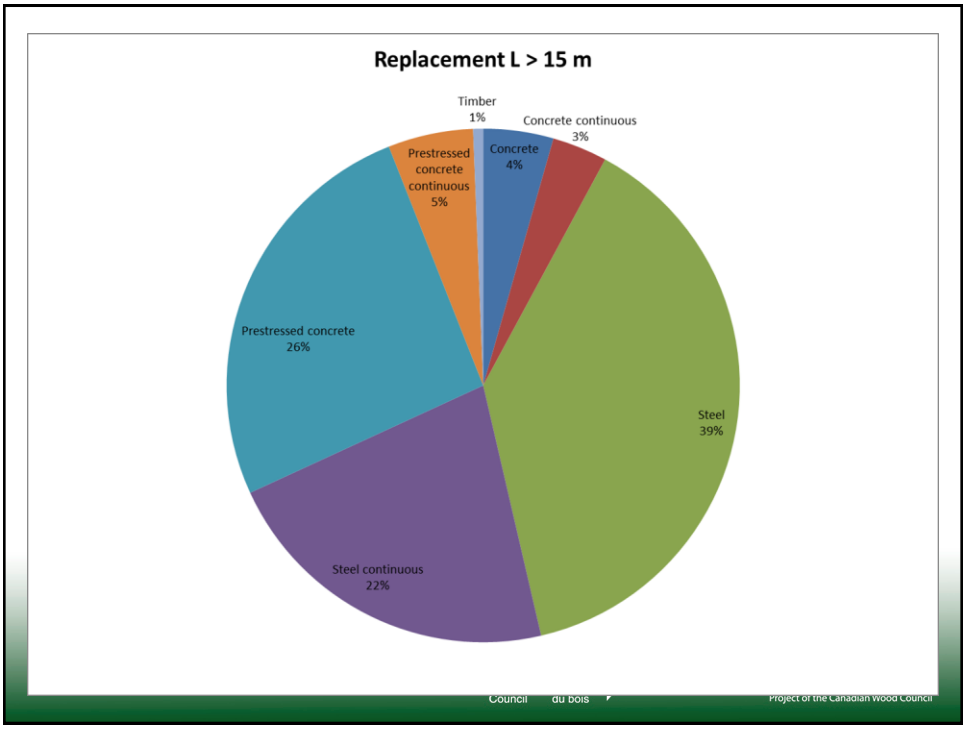

Replacement L > 15 m
Timber 1%
Concrete continuous 3%
Prestressed concrete continuous 5%
Concrete 4%
Prestressed concrete 26%
Steel 39%
Steel continuous 22%
Council du bois
Project of the Canadian Wood Council

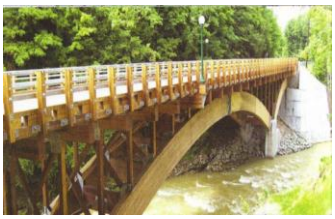
Table 8 – U.S. Timber Deck Total, by Type of Bridge Material

	Number	Percent
Other	18	.0
Concrete	124	.3
Concrete continuous	33	.1
Steel	22,710	49.4
Steel continuous	1178	2.6
Prestressed concrete	120	.3
Prestressed concrete continuous	9	.0
Wood or timber	21,660	47.1
Masonry	1	.0
Aluminum, Wrought Iron or Cast Iron	132	.3
Total bridges	45,985	100.0

Canadian Wood Council Conseil canadien du bois woodWORKS! Project of the Canadian Wood Council

Opportunities for Timber Bridges

- A few facts
  - Maximum length of span
    - 60% of new timber bridges span < 7.5 m
    - 50% of replacement timber bridges span < 7.5 m
    - Bridges with longer spans are the target for engineered wood products
  - Roughly twice as many timber decks than bridges
    - Demand for timber deck materials
- Largest existing current market opportunity in the US
  - County bridges, HS20 design load, a maximum span over 7.5 m



Top: Alton Sylor Memorial Bridge, US, 52 m (170 ft)
Bottom: Hiroshima Airport Bridge, Japan, 145 m (476 ft)

Canadian Wood Council Conseil canadien du bois woodWORKS! Project of the Canadian Wood Council

Timber Bridges in Canada

Kinsol Trestle

Canadian Wood Council / Conseil canadien du bois — wood WORKS! Project of the Canadian Wood Council

Timber Bridges in Canada

- Past
  - Wood was a prominent structural material used in bridge construction up until the 1950s.
  - Due to abundance of forests

- Present
  - Fell out of use in favour of steel and concrete
    - In Ontario, 250 wood bridges out of Province's inventory of 3000 bridges
  - Why wood fell out of favour?
    - Spans associated with conventional wood bridges were relatively limited
    - Wood can decay; treatments can be both a bonus and a penalty.
    - Lack of wood design knowledge

Canadian Wood Council / Conseil canadien du bois — wood WORKS! Project of the Canadian Wood Council

Timber Bridges in Canada

- Today and future
  - New technologies
    - High quality engineered wood products
    - Composite configuration: Carbon and glass fibre reinforced polymers (FRP), high performance concrete and epoxies
    - Major improvement in treatment methods
  - Resurgence of wood as a modern and yet sustainable bridge building material
  - Aesthetic and cultural considerations

Canadian Wood Council / Conseil canadien du bois — woodWORKS! Project of the Canadian Wood Council

Engineered Wood Products

- Laminated veneer lumber (LVL) & Parallel strand lumber (PSL)
- Minimize the presence of inherent defects
- More effective preservative penetration
- Waterborne treatments are permitted for PSL by CSA S6

Canadian Wood Council / Conseil canadien du bois — woodWORKS! Project of the Canadian Wood Council

PSL IN BRIDGES

Why PSL?

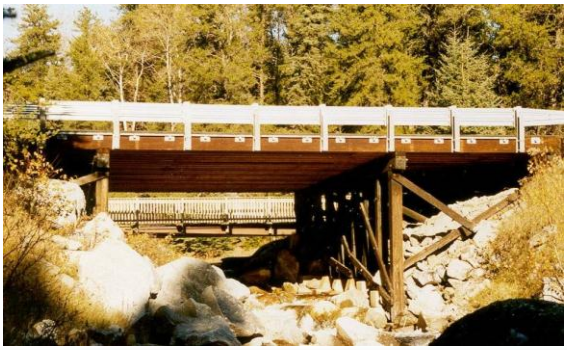
Domestic Product

Local Labour

Durable/'Palatable'

Contemporary Material

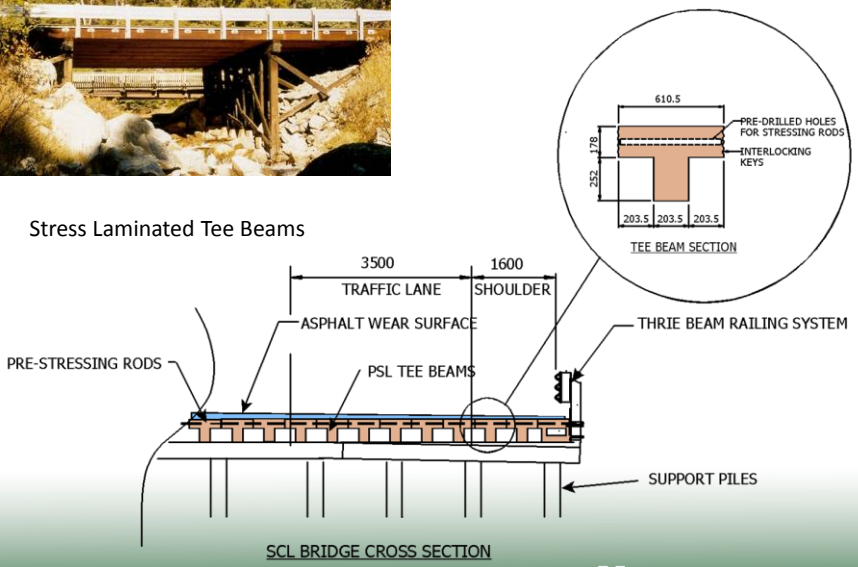
Sustainable Material



Canadian Wood Council | Conseil canadien du bois | wood WORKS! Project of the Canadian Wood Council

PSL IN BRIDGES

Stress Laminated Tee Beams



Source: Krisciunas, R., MTO

Canadian Wood Council | Conseil canadien du bois | wood WORKS! Project of the Canadian Wood Council

Rushing River Bridge 1996

Stress Laminated Tee Beams;
Penta Treated

Source: Krisciunas, R., MTO

Parallam in perfect condition >10 yrs later

Composite Concrete/Wood/Carbon Configuration

- Traditional composite wood/concrete bridge configuration
- Spans typically in the order of 6 m
- Many were built in 1950's and continue in operation today

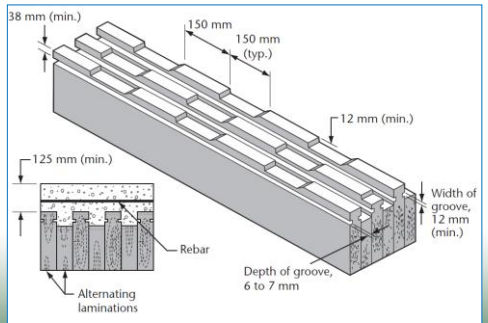


Figure 9.3
Details of wood-concrete interface

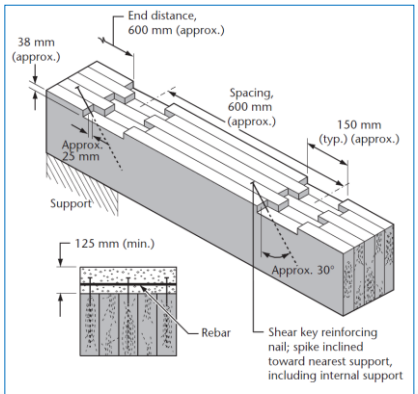
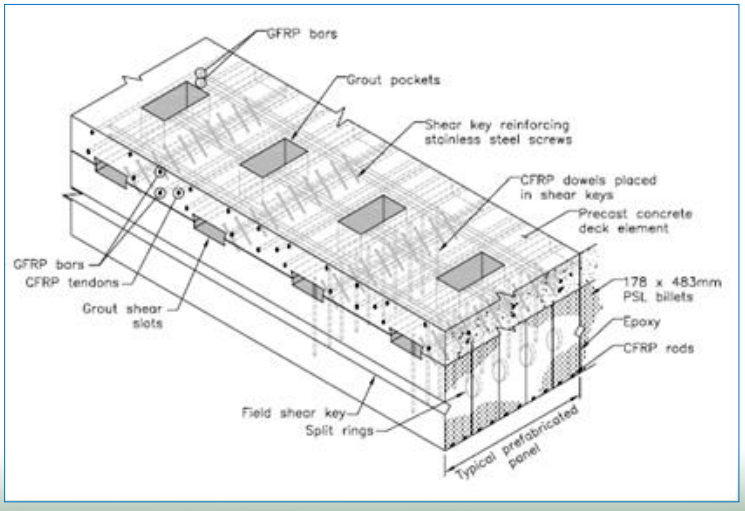


Figure 9.4
Alternative details of wood-concrete interface

Canadian Wood Council / Conseil canadien du bois

wood WORKS! Project of the Canadian Wood Council

Composite Concrete/Wood/Carbon Configuration



CFRP reinforced PSL/precast concrete configuration

Source: Krisciunas et al, 2010

Canadian Wood Council / Conseil canadien du bois — wood WORKS! Project of the Canadian Wood Council

Composite Concrete/Wood/Carbon Configuration

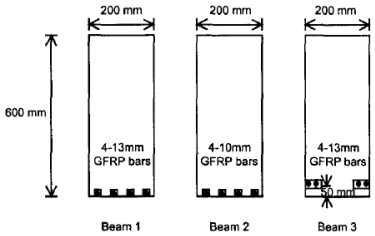


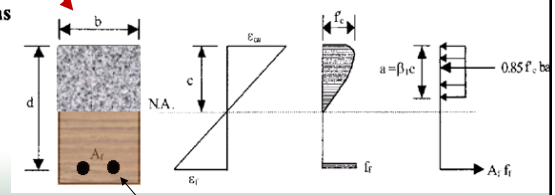
Figure 8 Full-scale specimen cross-sections

Combine High Performance Materials

Engineered Wood (PSL)

Precast Concrete

Carbon Fibre Reinforcing



MTO Concept

Wood Bridge Strengthening

Svekova et al, University of Manitoba

~2000

Canadian Wood Council / Conseil canadien du bois — wood WORKS! Project of the Canadian Wood Council

Mather Cr Bridge

Source: Krisciunas et al, 2010

Eramosa-Garafraxa Townline Road Replacement Bridge
Location: near Fergus, Ontario

Span: 11.8m
Width: 10m; two lanes
Skew: 18 degrees

The provided bridge resulted in a structure 50mm thinner than the originally-proposed precast concrete structure and approximately 20% cheaper in price.

Source: Zurell, C., Blackwell

Minto Bridge West Deck Replacement
Location: Ottawa, Ontario

Total length: 52.2 m
Width: 5.8 m; one design lane

Source: Zurell, C., Blackwell

Codes and Standards

- Canada: Canadian Highway Bridge Design Code (CAN/CSA-S6)
  - One of the first to adopt limit states design
  - References CSA Standard O86 *Engineering Design in Wood*
  - References CSA Standard O80 Series for preservative treatment as well as AWPA Standards
  - Addresses sawn lumber, structural composite lumber (LVL & PSL) & Glulam
  - Section on Fibre-reinforced structures (Section 16)
- US: AASHTO-LRFD Design Specification
  - Recently adopted limit states design
  - References NDS & AITC 117
  - Follows AASHTO M133 for preservative treatment and references AWPA Standards
  - Only addresses sawn lumber & Glulam
- Comparison (Groenier, J. and Wacker, J. (2008) Designing Timber Bridge Superstructures: A Comparison of US and Canadian Bridges Codes. Structures Congress 2008.)

Canadian Wood Council / Conseil canadien du bois
woodWORKS! Project of the Canadian Wood Council

Conclusions & Discussions

- A long history of wood bridges in North America
- Challenges and opportunities
- Resurgence of wood as a modern and yet sustainable bridge building material
- Suggested actions
  - Educating the design community
  - Development of design tools (publications & software)
  - Adoption of new products & technology
  - Development of market strategy
  - Awareness of sustainability of bridges

Canadian Wood Council Conseil canadien du bois

wood WORKS! Project of the Canadian Wood Council

Acknowledgements

- The Canadian Wood Council
- Mr. Tom Williamson, Timber Engineering LLC, U.S.
- Mr. Raymond Krisciunas, Ontario Ministry of Transportation (MTO), Canada
- Dr. Cory Zurell, Blackwell, Canada
- Mr. Crawford Dewar, Guardian Bridge Rapid Construction Inc., Canada

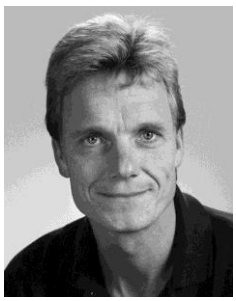
Canadian Wood Council Conseil canadien du bois

wood WORKS! Project of the Canadian Wood Council

Canadian Wood Council
Conseil canadien du bois
wood WORKS!
Project of the Canadian Wood Council

Combination of timber, CFRP and GFRP for the design and construction of a bowstring arch bridge

Robert WIDMANN
Empa
Dübendorf, Switzerland
robert.widmann@empa.ch



Civil Engineer
Research Associate at Empa
Swiss Federal Laboratories for Materials Science and Technology
Structural Engineering Research Laboratory.
Field of work:
Timber engineering.

Urs MEIER
Empa
Dübendorf, Switzerland
urs.meier@empa.ch

Summary

The replacement of an existing pedestrian bridge at the Swiss Federal Laboratories for Materials Testing and Research Empa, Switzerland gave the opportunity to head for a novel and experimental bridge design. It was decided to build a bowstring arch bridge. A remarkable feature of the design is the exclusive use of non-metallic construction materials, namely structural glulam timber, CFRP and GFRP. The span of the bridge is 12.0 m and the cross section of the timber bridge deck is 3.0 m by 0.16 m. The wooden deck is prestressed laterally and longitudinally (the bow) by CFRP pin-loaded straps and CFRP loops. The lateral prestressing aims at enhancing dimensional stability of the deck while the longitudinal CFRP loops act as a bowstring of the arch and thus shape and stiffen the construction. Load tests confirmed the good performance of the bridge.

Keywords: CFRP, GFRP, glulam, pin-loaded strap elements, monitoring.

1. Introduction

A bowstring arch bridge was built and installed on the areal of Empa, Swiss Federal Laboratories for Materials Science and Technology in Switzerland (Fig. 1). The bridge represented a novel design regarding two properties, its design and the materials that were used for its construction. A remarkable feature of the design is the exclusive use of non-metallic construction materials, namely structural glulam timber, carbon-fiber-reinforced plastics CFRP and glass-fiber-reinforced plastics GFRP. The span of the bridge is 12.0 m and the cross section of the timber bridge deck is width by height = 3.0 m by 0.16 m. The timber deck is prestressed laterally and longitudinally (the bow) with CFRP loops. The lateral prestressing aims at enhancing dimensional stability of the deck while the longitudinal CFRP loops act as bowstring of the arch and thus shape and stiffen the construction. The bridge serves also as a demonstrator for the efficient use of the pin-loaded CFRP straps that formed the bowstring. Since its installation in March 2007 the bridge has been continuously monitored. Several specific sensors are used to record deformations, bowstring tension, temperature and humidity. The monitoring system of this bridge was presented at ICTB 2010 in detail [1]. This paper will mainly focus on the design and the construction of the bridge.

Fig. 1: Pedestrian bridge at EMPA Duebendorf, Switzerland

2. Design of the bridge

2.1 Structure

The design principal of the bridge can be seen in Fig. 1. Based on an idea of U. Meier the detailed design of the bridge was developed by Dr. Deuring + Oehninger AG, an engineering company in Winterthur, Switzerland that has a lot of experience with the design of structures that contain fiber reinforced plastics. The structure consists of a bow-shaped girder, a bottom chord and struts acting as spacers between the beam and the bowstring. The camber of the originally straight beam results from prestressing the chord. As the camber of the bow (or arch) is relatively shallow the struts are needed for geometrical reasons. On the other hand this creates a truss-like behavior with the beam getting bending and compression loads, the chord tension loads and the struts compression loads. Advantages of this design are the slender appearance of the structure, its high stiffness, its light weight and the possibility to prefabricate the whole bridge.

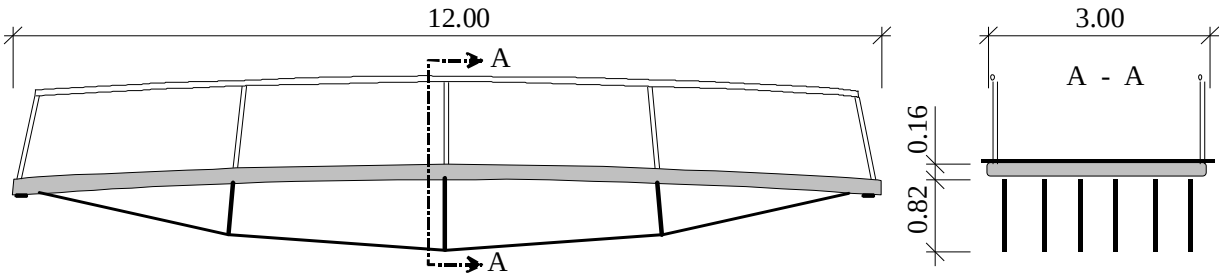


Fig. 2 Overview of the bridges design and main dimensions

3. Materials

3.1 Overview

Three kinds of materials were used for the construction of the bridge: timber, CFRP and GFRP (Table 1).

The bridge beam was made of glulam from Swiss grown Norway spruce (*Picea abies*) with the outer lamellas of the glulam elements made of oak (*Quercus robur*). The oak lamellas should prevent major indentations resulting from the lateral prestressing of the deck.

CFRP was used for the lateral and the longitudinal prestressing of the bridge deck. The bowstrings were composed of non-laminated pin loaded thermoplastic CFRP strap elements and the lateral prestressing was realized with loops from the same kind of material. It was the first time that such tendons and loops were applied in a timber bridge construction.

All other parts were made of GFRP, namely the bridge decking which consists of a plate with a thickness of 8mm, all struts, the handrails and the profiles that were used for anchoring the CFRP

straps.

Table 1: Basic mechanical properties of the applied materials.

Material	E GPa	ρ kg/m^3	Strength MPa
Glulam GL24h (EN 1194 [2])	11.5	455 at u = 12% (measured)	$f_{m,g,k} = 24$ $f_{c,0,k} = 24$
CFRP	150	1500	$f_t = 2000$
GFRP	44.5	2000	$f_c = 900$

3.2 CFRP loops and pin-loaded CFRP straps

Continuous, thermoplastic CFRP tapes of about 0.12 mm thickness were the base elements for the CFRP loops and straps. Compared to CFRP lamellas they are very flexible and can be bent with radii of as small as 20 mm to 25 mm. Thus, the tapes can be wrapped around an object in one or numerous layers. The straps and loops were produced by Carbo-Link AG [3] which is a Swiss high-tech company specialized in design, development and production of structural elements with carbon fibers.

Laboratory experiments and analytical modeling [3] have shown that there are severe stress concentrations in the region where the strap and the pin meet in the case of laminated pin loaded tendons (Fig. 3a). As a result of the pin-loading, the tensile resistance of the strap is limited to about 50 % of the material's unidirectional strength. An alternative option to reduce the undesirable stress concentrations, overcome manufacturing difficulties and to reduce cost is the use of a non-laminated strap. The concept which has been patented [4 and 5] is shown in (Fig. 3b). The CFRP strap comprises a number of unidirectional reinforced layers, formed from a single, continuous, thermoplastic tape. The tape is wound around the two pins and only the end of the outermost layer is fusion bonded to the neighboring layer to form a closed loop.

a) b)

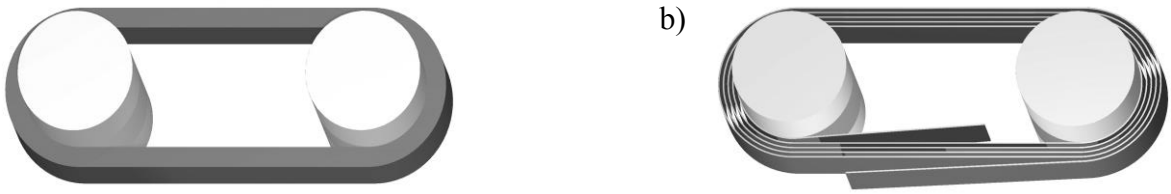


Fig. 3: Principal design of pin loaded strap elements (loops), a) laminated CFRP, b) not laminated CFRP tapes.

The non-laminated strap element enables the individual layers to move relative to each other, which allows an equalization of forces in the layers when the strap is tensioned. The stress concentrations are reduced since the new structural form is more compliant compared to a laminated equivalent. Control of the initial tensioning process reduces interlaminar shear stresses so that a fairly uniform strain distribution in all layers can be achieved. The approach allows greater flexibility in terms of the geometry of the tendon, and it can be manufactured on site. Moreover, the concept is going to be less expensive because there is no consolidation process required.

4. Assembly and installation

4.1 Overview and lateral prestressing

The bridge was assembled in a hall on the EMPA site. The main assembly steps consisted of the lateral and longitudinal prestressing of the deck, the mounting of the decks cover and hand-railing and the installation of the relevant components of the monitoring system. In order to apply the lateral prestressing, the glulam bridge deck consisted of two pieces each of which being 12.0 m long and having a width of 1.42 m which is a bit less than half the total decks target width. Both pieces were laid side by side, the CFRP loops were slipped over, arranged in a pattern with a mean spacing of about 0.30 m and the two pieces were pushed apart against the resistance of the CFRP loops with the help of inserted timber wedges (Fig. 4 Fig. 5 and Fig. 6).

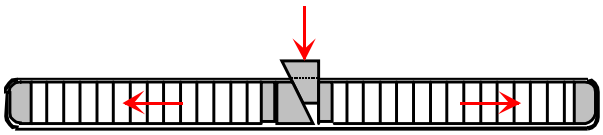


Fig. 4: Principle of the application of the lateral prestressing of the glulam slab with the help of wedges. The prestressing had to be conducted iteratively over the length of the slab and the wedges were glued-in at the final stage. The grey shaded parts ideally are made of hardwood in order to minimize the effect of possible indentations in the wood. In our case only the two outer lamellas were made of oak.

Fig. 5: Detail view of wedge, glulam and CFRP-loops (right)

Fig. 6: Left: Sliding on and distribution of the CFRP loops on the glulam deck.
Right: Upper side of the deck before the installation of the sensors for the monitoring system and the GFRP decking. The lateral tension elements can be seen as well as wooden spacers on the plate's upper side. These spacers guarantee a minimum ventilation of the deck and protect the CFRP elements from mechanical damage. Between the spacers and the GFRP decking impact sound insulation made of plastic foam was applied. An alternative to the wooden spacers for the GFRP decking could be the application strips or mats made of rubber gravel which would also provide a protection for the CFRP straps and serve as a impact sound insulation at the same time .Along the centreline of the plate the wooden wedges and spacers that were used to apply the lateral prestressing of the plate can be seen. The method to provide the force for lateral prestressing by the insertion of wedges proved to be easy and efficient even if the desired maximum force could not be applied eventually.

After reaching a predefined deck width the slots between the two deck parts were filled with glued-in timber blocks that serve as spacers. However, the originally predefined deck width could not be reached with this procedure as it was not possible to apply the necessary force with the insertion of the wedges. An alternative procedure for future applications could be the use of devices that apply a controlled force over the whole length of the plate, e.g. a series of a large number of hydraulic presses. However, such a procedure is more difficult to realize as the limited space between the two parts of the deck require special solutions.

4.2 Longitudinal prestressing

For the longitudinal prestressing the bridge deck was supported at the two outer quarter-points of its length (Fig. 7a). Then the GFRP struts were assembled (Fig. 7b) and the plate was loaded with concrete blocks at both cantilever ends (Fig. 7c). This loading (32.0 kN per side) resulted in a bending stress of around 15.0 MPa and a total camber of about 145 mm which was needed for the assembly of the CFRP straps. After putting these members into place and anchoring the loops at both ends of the bridge deck (Fig. 8) the prestressing was finally realized by the removal of the concrete blocks. The relaxation of the decks deflection is prevented by the truss system which results in the desired bowstring arch-shape of the deck (Fig. 7d).

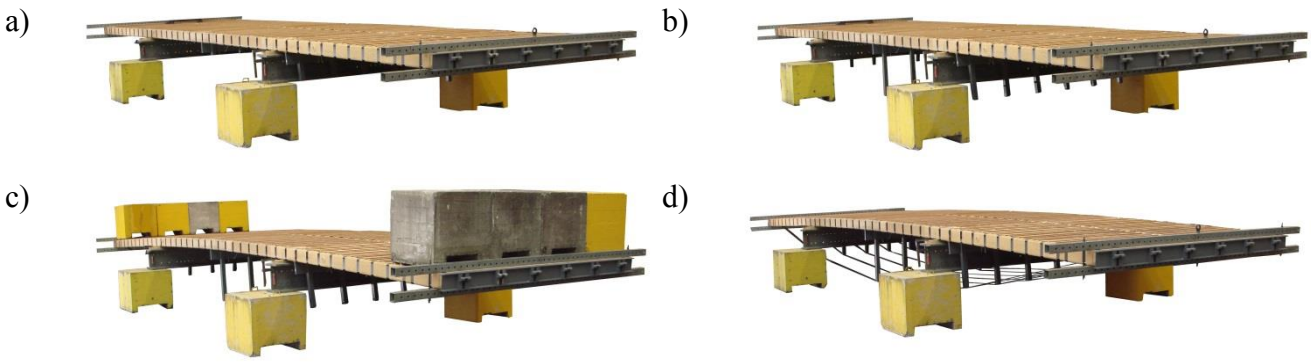


Fig. 7: Tensioning of the bow.

Fig. 8: Anchoring of the longitudinal tension loops at the plate's front end using a GFRP bolt with a diameter of 4 cm. This diameter is towards the lower end of possible bending radii for the used CFRP elements that can be realized in practice without damaging the CFRP structure. The fixation of the bolt as well as the profile that covers the front end of the plate and serves to distribute the loads originating from the loops into the slab are made of GFRP.

5. Load tests and design verification

Before the bridge was set into its final position a series of load tests was executed. The loading was realized with the same concrete cubes that served already for the tensioning of the bow as shown in Fig. 9. The aim of the tests was to gather data about the bending stiffness of the System as well as to

determine the tension force within the CFRP straps. The loads were applied at midspan and consisted of one to three cubes that were arranged symmetrically and asymmetrically in reference to the longitudinal axis of the bridge slab. Two gauges were used to measure the total deflection at midspan on either side of the slab.

For the analysis shown here only symmetrical loadings of the slab are considered which results in loads between approximately 8.5 kN and 25.6 kN.

The tension force of the straps was measured "acoustically" with the determination of the first natural frequency after the strap was excited to swing with the use of a stick. The tension strength per strap could then be calculated on base of the equation for the determination of the fundamental frequency of a vibrating string like follows:

$$F_t = 4 \cdot l^2 \cdot f^2 \cdot \rho \cdot A \qquad (1)$$

with:

F_t = tension force in CFRP strap

l = length of the CFRP strap section (3000 mm)

f = first natural frequency of CFRP strap

ρ = density of the CFRP (1500 kg/m^3)

A = nominal cross section of the CFRP strap (120 mm^2)

Fig. 9: Set-up for measurement of deflections and the tension force in the straps.

Fundamental frequencies between minimum 54 Hz and maximum 70 Hz were measured depending on the loading conditions.

The so determined tension force within the CFRP straps varied due to geometrical deviations up to maximum 20% from strap to strap. This is a consequence of assembly tolerances that directly affect the tensioning of the straps. The applied method of tensioning does not permit to adjust the tension forces for each single strap individually.

Table 2: Results of load tests compared to static calculations.

Step	Load	Mean deflection	Mean tension force	Accumulated tension force	Tension from static calculation	Deflection from static calculation
	kN	mm	kN	kN	kN	mm
0	0	0	21.0	126	149	0
1	8.5	6.54	23.8	143		
2	17.2	14.1	26.6	160		
3	25.6	20.8	29.4	176	208	18.7
Design load	4/m^2				405	83.1

The results of one of several loading cycles are shown in Table 2. It can be seen that mean deflection and mean/accumulated tension force in the straps correlate well. The accumulated tension force represents the total tension force of the six straps that is needed to provide the bow shape of the slab under the different loadings. However, it can also be seen that the accumulated tension force over the 6 struts does not well fit the tension force that has been found in the static calculation. The calculation was made with the consideration of second order effects. The reason for the discrepancy could not be verified in detail. Some possible contributions to this fact can be variations in material properties as well as in the measurement of the frequency. As the strap is not made of one solid part but of several layers, this might have an influence on the fundamental frequency of the string.

For the verification of the performance of the system the bending stiffness *EI* was calculated with the data from step 2 and step 3 in Table 2. The bending stiffness was determined to be $EI_{bow} = 4.54 \cdot 10^{13}$ Nmm^2. Assuming a possible bending MOE of the glulam elements of $E_{0,mean} = 11.6$ GPa to 14.7 GPa (Gl24h to Gl36h according to EN 1194) the bending stiffness of the slab itself without tensioning could be estimated to range between minimal $EI_{slab} = 1.19 \cdot 10^{13}$ Nmm^2 and maximal $EI_{slab} = 1.51 \cdot 10^{13}$ Nmm^2. With this it can be shown that the bending stiffness of the slab increased between 3 and almost 4 times due to prestressing it into a bow shape. This confirms the initial statement regarding the expected advantages of the construction

6. Conclusions

A pedestrian bridge made exclusively of glulam structural timber, CFRP and GFRP had been designed and constructed at the Empa site in Duebendorf, Switzerland. The advanced materials glulam structural timber, pin loaded CFRP straps and GFRP elements were combined in a new bridge design, a bowstring arch bridge. It turned out and could be shown, that the required tensioning of the bow as well as the lateral prestressing of the bridge slab could be realized as planned. A potential for an improvement of some details and procedures could be identified and it is proposed to consider this for future constructions.

Load tests confirmed a superior stiffness of the system.

Considering all the advantages of the described bowstring arch bridge like lightweight structure, high stiffness, prevention of corrosion problems, easy installation, good value and an expected long service life, a good market potential for such structures can be expected.

Acknowledgement

The authors would like to thank all partners that were involved in the realization of the project, namely Dr. Deuring + Oehninger AG, Carbo-Link AG, the staff of Empa laboratories 302 – Applied Wood Materials, 303 - Structural Engineering, 304 - Mechanical Systems Engineering and 405 - Electronics/Metrology/Reliability for their contributions to the assembly, installation and monitoring of the bridge. Many thanks also to Bafa for the support of this project.

References

[1] Broennimann, R., R. Widmann, and U. Meier. IDesign, construction and monitoring of a bowstring arch bridge made exclusively of timber, CFRP and GFRP. In: International Conference on Timber Bridges 2010. 2010. Lillehammer, Norway.

[2] www.deuringoehninger.ch

[3] EN 1194 (1999): Timber structures - Glued laminated timber - Strength classes and determination of characteristic values, CEN European Committee for Standardization.

[4] http://www.carbo-link.com/http://www.carbo-link.com/

[5] Winistoerfer A., "Development of non-laminated advanced composite straps for civil engineering applications". Ph.D. Thesis, University of Warwick, 1999.

[6] Meier, U and Winistörfer, A.: Multilayer Traction Element in the Form of a Loop, European Patent 0 815 329, January 7, 1998.

[7] Meier U. and Winistoerfer A., "Multilayer traction ele-ment in the form of a loop", US-Patent 6'209'279 B1, 2001.

Inspection of Timber Bridges in the Pacific West (ID-150)

Tom Williamson, P.E.
Managing Partner
Timber Engineering LLC
Vancouver, WA USA
Lola Coombe, P.E.
Civil Engineering Technician
Rogue River-Siskiyou National Forest – USDA Forest Service
Gold Beach, OR USA
David Strahl, P.E
Structures Engineer
Pacific Northwest Regional Office – USDA Forest Service
Portland, OR USA

Summary

Thirty timber bridges were inspected in the Pacific West region. Twenty four were sawn timber bridges and six were glulam. For the sawn timber bridges, all of the timber components were pressure treated DF with the exception of one bridge which was not treated and one bridge that was treated with copper napthenate. For the sawn timber bridges, 21 had multiple girders and the remaining 3 had longitudinal slab decks. The sawn timber bridges ranged in age from 31 years to 78 years and were determined to be meeting performance expectations. The glulam bridge components were all DF and were pressure treated with penta in oil. Four of the glulam bridges were multiple girder bridges with either a timber or concrete deck and the other 2 were longitudinal glulam deck bridges. The glulam bridges have been in service from 21 years to 46 years and were determined to be performing at a good or very good level.

Key Words: field inspection, timber bridges, sawn timber, glulam, NBI ratings

Introduction

The Pacific West region was identified as an integral part of the *Field Performance of Timber Bridges: a National Study* project. The Pacific West region consists of the states of California, Oregon and Washington. According to the NBI, each of these states has an existing inventory of over 550 timber bridges. A study of the NBI was undertaken by the team to determine which counties (two per state) should be included in the study. Variables evaluated included the number of bridges in a county, the type of construction, the age of construction and environmental considerations.

As a result of this evaluation, the following counties were identified for inclusion in the study:

California: Del Norte and Siskiyou counties
Oregon: Klamath and Washington counties
Washington: Grays Harbor and Yakima counties

Each of these counties is located in either zone 3 (intermediate) or zone 4 (high) according to the AWPA Wood Decay Hazard Map. The team identified possible bridges for inclusion in the

study for each county and then met with county officials to review these choices. Following this and based on input from the counties, the team selected the bridges to be included and provided this information to the management team for approval. The end result is that 30 bridges in total were included for the Pacific West region.

The primary tools used for inspection were a visual assessment of the bridge, hammer sounding of the timber members and the measurement of moisture content. When areas of high moisture content and/or possible decay were identified, a resistance drill was used. All of the timbers were DF. Preservative treatments varied and included creosote, penta in oil and copper napthenate.

Grays Harbor, WA

Four bridges were inspected in Grays Harbor County, WA. Grays Harbor is in Western WA and borders the Pacific Ocean with an approximate latitude of 47.0 N and longitude of 123.4 W. It is in the Olympic Mountain range and has a relatively high annual rainfall. The 4 bridges were all multiple girder sawn timber bridges and 3 had nail laminated timber decks while the fourth had a cast in place concrete deck. Details regarding the bridges are summarized in Table 1.

Table 1 Grays Harbor County, WA Bridges

Bridge ID	Superstructure Type	Year Built	# spans	Span Lengths, m	ADT	Width (m)/ # lanes	Wearing surface	NBI Rating		Repair comments
								Girders	Deck	
Garrard Creek	Sawn timber girders and timber deck	1970	3	5.6	225	7.7/2	Asphalt	7	6	No repairs to superstructure
Middle Branch Wildcat Creek #2	Sawn timber girders and timber deck	1951	3	5.2	600	7.4/2	Asphalt	6	6	No repairs to superstructure
Middle Branch Wildcat Creek #3	Sawn timber girders and timber deck	1951	2	5.3	650	6.5/2	Asphalt	7	6	No repairs to superstructure
Sylvia Creek	Sawn timber girders and concrete deck	1935	3	6.8	4250	16.3/2	Asphalt	7	6	No repairs to superstructure

The girders were all creosote treated DF. The range in age of these bridges is from 43 years to 78 years and all had an NBI rating of 6 or higher for the girders with 3 having a rating of 7. The three nail laminated decks all had an NBI rating of 6. According to county records, no major repairs had been made to any of the superstructure elements.

The Sylvia Creek Bridge is the oldest bridge inspected in the Pacific West region and is noteworthy in that it was originally constructed to carry 4 lanes of traffic. However, subsequent road construction in the county resulted in it only carrying 2 lanes with an ADT of over 4000 which was the highest for any bridge inspected in the region. This is shown in Figure 1. The bridge is posted for legal loads due to a badly deteriorated concrete guardrail system shown in Figure 2 which is well off of the travel paths and not planned for repair.

Figure 1 Sylvia Creek Approach

Figure 2 Sylvia Creek Guardrail

With 3 spans, the bridge has 60 individual girders and there was no significant decay observed in any of these despite being in service for 78 years. The concrete deck is 6-1/2” in thickness and is generally in good condition. There were some full width lateral cracks that reflect through the asphalt and alligator cracking was typical on the underside of the deck. There were some repaired sections of deck and areas of water staining on girders from deck leakage.

Overall, these 4 bridges had a total of 169 sawn timber girders with no signs of significant decay in any of them. The major types of distress noted in the girders were some areas of high moisture content and minor checking. The major types of distress noted in the nail lam decks were areas of high moisture content, several decayed lams and an instance of fungal body growth. Some decay was also observed in the curbs. There was rutting of the asphalt in the upstream lane of Wildcat Creek #2 with water staining and high moisture content in the deck lams and girder below.

Yakima County, WA

Seven bridges were inspected in Yakima County, WA. Yakima County is in Southern WA and is on the Eastern edge of the Cascade Mountain Range. It has an average elevation of 1100 ft, and has a relatively low annual rainfall but with a relatively high annual snowfall. It has an approximate latitude of 46.0 N and longitude of 120.5 W. The 7 bridges were all multiple girder sawn timber bridges with transverse sawn timber decks. Details regarding the bridges are summarized in Table 2.

Table 2 Yakima County, WA Bridges

Bridge ID	Superstructure Type	Year Built	# spans	Span Lengths, M	ADT	Width (m) # lanes	Wearing surface	NBI Rating		Repair comments
								Girders	Deck	
508	Sawn timber girders and timber deck	1965	2	5.9	10	7.4/2	Chip and seal with gravel	Ext. 5 Int. 6	5	No repairs to superstructure
509	Sawn timber girders and timber deck	1965	1	7.1	60	7.1/2	Chip and seal with gravel	7	6	No repairs to superstructure

521	Sawn timber girders and timber deck	1959	3	5.6	30	7.1/2	Chip and seal with gravel	7	6	No repairs to superstructure
724	Sawn timber girders and timber deck	1959	1	6.8	105	7.1/2	Chip and seal with gravel	5	4	No repairs to superstructure
738	Sawn timber girders and timber deck	1938	2	4.4	1605	7.7/2	Asphalt	7	6	No repairs to superstructure
739*	Sawn timber girders and timber deck	1939	4*	5.6	1605	7.7/2	Asphalt	6	6	One girder repaired with a helper beam
740	Sawn timber girders and timber deck	1939	2	5.9	3375	7.7/2	Asphalt	6	6	No repairs to superstructure

* It is noted that the inspection team was not able to hammer sound the girders in the center spans of this bridge but they all appeared to be visually sound with no signs of decay.

The girders on 6 of the bridges were creosote treated DF. Bridge 724 is an anomaly since it is the only bridge inspected in the Pacific West region that used untreated DF girders and deck. Four of the bridges were on low volume roads with gravel surfaces over a very thin (less than 25 mm) asphalt layer which was the remnant of previous chip and seal applications. The other 3 had asphalt wearing surfaces and were on relatively high volume farm to market roads with ADTs of over 1600. The range in age of these bridges is from 48 years to 74 years. The NBI ratings for the girders ranged from 5 to 7 and the ratings for the decks ranged from 4 to 6. According to county records, no major repairs had been made except for the addition of a helper beam for bridge 739.

Since bridge 724 used untreated DF and has been in service for 54 years, it had relatively low NBI ratings as would be expected. The deck had numerous wide gaps (severe delamination) allowing moisture to readily penetrate to the girders below as shown in Figure 3. The result was high moisture content and moderate decay in several of the girders. Figure 4 shows significant checking in an untreated exterior girder.

Figure 3 Deck cracks in bridge 724

Figure 4 Untreated girder in bridge 724

For the remaining 3 bridges with the old chip and seal surface, the NBI ratings for the decks was primarily influenced by separation of the deck lams and relatively high moisture contents as would be expected for this type of surface. The deck on bridges 508 and 509 also had decay in

the laminations near the bridge ends. The girders for these bridges were in relatively good condition with the exception of some areas of decay in the upper 6” of the exterior girders in bridge 508.

For the three bridges with the asphalt wearing surface, there were no signs of decay in the girders but shear cracks appeared to be developing in several girders in bridges 739 and 740 leading to a lower NBI rating. The asphalt wearing surface on all 3 bridges is badly cracked.

Washington County, OR

Seven bridges were inspected in Washington County, OR. Washington County is in Northwestern OR and is in the Willamette Valley. It is about 60 miles east of the Pacific Ocean and has a relatively high annual rainfall with over 160 days of precipitation annually. It has an approximate latitude of 45.3 N and longitude of 122.5 W. Five of the 7 bridges were multiple girder sawn timber bridges with transverse sawn timber decks. The remaining two bridges were longitudinal glulam deck systems. Details regarding the bridges are summarized in Table 3.

Table 3 Washington County, OR Bridges

Bridge ID	Superstructure Type	Year Built	# spans	Span Lengths, M	ADT	Width (m) # lanes	Wearing surface	NBI Rating		Repair comments
								Girders	Deck	
1228	Sawn timber girders and timber deck	1975	3	5.3	1010	5.9/2	Asphalt	6	7	No repairs to superstructure
1232	Sawn timber girders and timber deck	1959	4	4.7	280	5.9/2	Asphalt	6	6	One outside girder replaced / several deck boards replaced
1293	Sawn timber girders and timber deck	1960	2	4.7	55	6.5/2	Asphalt	6	6	One outside girder replaced
1298	Sawn timber girders and timber deck	1960	3	4.7	1130	5.9/2	Asphalt	7	7	One outside girder replaced
1302	Sawn timber girders and timber deck	1960	5	4.7	1130	5.9/2	Asphalt	Ext. 4 Int. 6	6	No repairs to superstructure
1358	Longitudinal glulam deck	1992	3	7.4	1520	7.7/2	Asphalt		8	No repairs to superstructure
1360	Longitudinal glulam deck	1992	1	7.4	390	8.3/2	Asphalt		8	No repairs to superstructure

The timbers on the 5 multiple girder bridges were all creosote treated. These 5 bridges ranged in age from 38 years to 54 years. The decks were all plank deck systems in relatively good condition with NBI ratings of 6 and 7. The girders had NBI ratings from 4 to 7. The low rating of 4 for the exterior girders on bridge 1302 was due to the fact that 2 outside girders are badly decayed and need replacement as shown in Figure 5. Washington County has an excellent maintenance program and has replaced several girders and deck boards during the last decade as noted. With the exception of bridge 1293, which was recently resurfaced, the asphalt surfaces on the other 4 girder bridges have significant cracking and are scheduled for maintenance.

The two longitudinal glulam bridges were treated with penta in oil. While they have only been in service for 21 years, they are in excellent condition with moisture contents at 16% or below as shown in Figure 6. There is some asphalt cracking at the bridge ends and over intermediate piers on bridge 1358. The county is very pleased with the performance of these bridges.

Figure 5 Decay in G14 of bridge 1302

Figure 6 Glulam deck on bridge 1360

Klamath County, OR

Five bridges were inspected in Klamath County, OR. Klamath County is located in South Central OR and is on the border with California. It is on the Eastern edge of the Cascade Mountain Range and has an average elevation of 4100 ft. It has a relatively low annual rainfall but a relatively high annual snowfall. It has an approximate latitude of 42.0 N and longitude of 121.0 W. Two of the 5 bridges were multiple girder sawn timber bridges with transverse sawn timber decks. The remaining three bridges have longitudinal sawn timber slab decks. Details regarding the bridges are summarized in Table 4.

Table 4 Klamath County, OR Bridges

Bridge ID	Superstructure Type	Year Built	# spans	Span Lengths, m	ADT	Width (m) # lanes	Wearing surface	NBI Rating		Repair comments
								Girders	Deck	
Anderson Rd.	Sawn timber girders and timber deck	1970	2	5.5	70	8.3/2	Asphalt	7	7	No repairs to superstructure
Hill Rd.	Longitudinal sawn timber slab deck	1965	1	6.8	270	8.3/2	Asphalt		7	No repairs to superstructure
Homedale Rd.	Longitudinal sawn timber slab deck	1963	2	4.7	315	8.3/2	Asphalt		7	No repairs to superstructure

Old Midland Rd.	Longitudinal sawn timber slab deck	1971	1	7.4	375	9.5/2	Asphalt		7	No repairs to superstructure
Summers Lane	Sawn timber girders and timber deck	1982	1	7.4	100	9.5/2	Asphalt	7	7	No repairs to superstructure

All of these bridges were in good condition with NBI ratings of 7 for the girders and decks. Figure 7 shows the typical condition of the slab deck bridges. The timbers were all creosote treated with the exception of the girders and deck on the Summers Lane Bridge which appeared to be treated with copper napthenate as indicated by Figure 8. The bridges ranged in age from 31 years to 50 years. The asphalt wearing surfaces were all in relatively good condition with transverse cracks near the bridge ends being typical.

Figure 7 Longitudinal slab deck on Old Midland Rd. bridge

Figure 8 Sawn timber stringers on Summers Lane Bridge

Del Norte County, CA

Four bridges were inspected in Del Norte County, CA. Del Norte County is located in the extreme NW corner of the state and borders on Oregon and the Pacific Ocean. It is in the Cascade Mountain Range and has a high annual rainfall and a relatively high annual snowfall. It has an approximate latitude of 42.0 N and longitude of 123.7 W. All of the bridges were multiple girder glulam bridges and are located on a Forest Service road with low traffic volumes. One of the bridges has a transverse sawn timber deck with steel running plates and the other 3 have concrete decks. Details regarding the bridges are summarized in Table 5.

Table 5 Del Norte, CA Bridges

Bridge ID	Superstructure Type	Year Built	# spans	Span Lengths, m	ADT	Width (m) # lanes	Wearing surface	NBI Rating		Repair comments
								Girders	Deck	
Knopki	Glulam girders and timber deck	1967	1	13.9	50	4.7/1	Steel running plates	7	8	No repairs to superstructure

Middle Fork Bridge #2	Glulam girders and concrete deck	1973	3	7.4/23.7/7.4	50	4.1/1	Concrete	7	7	No repairs to superstructure
Middle Fork Bridge #1	Glulam girders and concrete deck	1972	2	20.1	50	4.4/1	Concrete	7	7	No repairs to superstructure
Griffin Creek	Glulam girders and concrete deck	1978	2	13.0/21.3	50	5.7/1	Concrete	4	8	No repairs to superstructure

All of these bridges were in good condition with NBI ratings of 7 for the girders and 7 and 8 for the decks with the exception of the Griffin Creek Bridge. The girders for the Griffin Creek Bridge received an NBI rating of 4 due to the downstream 72 ft. span girder having a horizontal shear crack extending approximately 1/3rd of the span from the EOB abutment. Figure 9 shows the crack which is visible through the width of the girder and which resulted in a horizontal offset of more than 1 inch at the end. The remaining girders in the bridge were in good condition and would have an NBI rating of 7 similar to the other 3 bridges.

The glulam girders were all treated with penta in oil and the bridges ranged in age from 35 years to 46 years. The concrete decks were in good condition but two of them had some water ponding on them. The Knopki Creek Bridge was the only bridge inspected that had steel running plates as shown in Figure 10. Both the running plates and the transverse timber deck were in good condition after 46 years of in-service use.

Figure 9 Horizontal shear crack in Griffin Creek bridge

Figure 10 Steel running plates and transverse timber deck on Knopki Creek Bridge

Siskiyou County, CA

Three bridges were inspected in Siskiyou County, CA. Siskiyou is located in the NW part of the state and borders on Oregon and Del Norte County. It is in the Cascade Mountain Range and has a medium annual rainfall and a relatively high annual snowfall. It has an approximate latitude of 41.5 N and longitude of 122.0 W. All of the bridges were multiple girder sawn timber bridges with different deck framing systems. Details regarding the bridges are summarized in Table 6.

Table 6 Siskiyou County, CA bridges

Bridge ID	Superstructure Type	Year Built	# spans	Span Lengths, m	ADT	Width (m) # lanes	Wearing surface	NBI Rating		Repair comments
								Girders	Deck	
Red Bridge	Sawn timber girders and timber deck	1959	2	5.3	45	5.3/1	Timber running planks	6	6	No repairs to superstructure
Shovel Creek Bridge	Sawn timber girders and timber deck	1977	1	8.3	40	7.4/2	Timber deck	Ext. 5 Int. 7	5	No repairs to superstructure
Fishhook Bridge	Sawn timber girders and timber deck	1960	1	7.4	225	8.6/2	Asphalt	7	7	No repairs to superstructure

The bridges in this county ranged in age from 36 years to 54 years. One of the bridges has a transverse sawn timber deck, one has sawn timber stringers with sawn timber running planks and one has transverse sawn timber deck planks. Two of the bridges are on low volume roads and the third, the Fishhook Bridge, carries a relatively high volume of RV traffic as it is on an access road to a popular resort area. This bridge was determined to be in good condition with an NBI rating of 7 for the girders and deck

The Shovel Creek Bridge has sawn timber deck planks as the wearing surface and due to the traffic pattern, significant grooves exist in the two wheel paths as shown by Figure 11. This was the newest bridge inspected in the county and had the lowest NBI rating suggesting that using transverse deck panels functioning as the structural deck and the wearing surface is not conducive to longevity. The Red Bridge which has been in service for 54 years was unique in that it was the only bridge inspected that uses timber running planks as shown in Figure 12. It was considered to be in good condition considering the age of the bridge although one girder had previously been repaired using steel straps.

Figure 11 Wheel path depressions in timber deck on Shovel Creek Bridge

Figure 12 Timber running planks on Red Bridge

Conclusions

Of the 30 bridges inspected in the Pacific West region, 24 were sawn timber bridges and 6 were glulam. For the sawn timber bridges all of the timber components were pressure treated DF with the exception of one bridge in Yakima County which was not treated and one bridge in Klamath County which had been treated with copper napthenate. The glulam bridge components were all DF and were pressure treated with penta in oil.

For the sawn timber bridges, 21 had multiple girders and the remaining 3 had longitudinal slab decks. All of the slab deck bridges have been in service for over 40 years and all had an NBI rating of 7 and were considered to be performing well with no noticeable signs of distress or decay. For the multiple girder bridges, 18 had NBI ratings of 6 or 7 for the girders and deck. The remaining 3 had lower ratings due to observed decay in some deck laminations and decay in one or more girders in each bridge. This included the untreated bridge. A total of 742 individual girders were inspected for the 21 multiple girder sawn timber bridges. Of these, only 11 girders or 1.5% had significant decay and needed replacement.

Four of the glulam bridges were multiple girder bridges with either a timber or concrete deck and the other 2 were longitudinal glulam deck bridges. While they have only been in service for 21 years, the longitudinal glulam deck bridges received the highest NBI ratings for the bridges inspected in this region with a rating of 8. The multiple girder glulam bridges have all been in service for over 40 years and received an NBI rating of 7 with the exception of one bridge which received a lower rating due to a horizontal shear failure in one girder. The inspection team could not detect any discernible reason for the shear failure and determined that, in general, the glulam bridges are performing well.

Overall, the inspection team felt the in-service performance of the bridges in the Pacific West met or exceeded expectations.

Two Bridges Built Using Black Locust Wood

Ryan Woodward
Bridge Engineer
HNTB Corporation
New York, NY USA
rwoodward@hntb.com

Ted Zoli
Chief Bridge Engineer
HNTB Corporation
New York, NY USA
tzoli@hntb.com

Summary

Two pedestrian bridges were recently built in New York using Black Locust wood, an unusual choice for a structural material despite its high strength, rot resistance, and local availability. The bridges explore the structural abilities of Black Locust, representing two very different bridge types. Squibb Bridge (an underslung suspension bridge) is built from short logs used in the round, with tubular steel connections. The TA Footbridge at Vassar College (a Town lattice truss) borrows from historic covered bridge technology, incorporating wooden peg connections. As Black Locust is naturally rot resistant, a cover is not necessary.

Limited information is available regarding the mechanical properties of Black Locust, and the species is unaddressed by the design codes and grading agencies. Load testing was performed using full-scale members and small clears in order to substantiate average values reported in the literature. Data is presented compatible with ASTM standards and NDS practices.

Guidance is provided for visual inspection, with attention to unsound knots, slope of grain, and Locust Borers. An inexpensive method for acoustic testing was developed for measuring modulus of elasticity and detecting voids, and could be further developed for timber grading at any mill.

Keywords: Black Locust, Town lattice truss, underslung suspension bridge.

1. Introduction

Although it is not considered a commercially important wood species in the United States, Black Locust (*Robinia pseudoacacia*) has some potential as a more sustainable alternative to tropical hardwoods. Namely, it grows locally in the northeastern U.S. (reduced transportation distance), it is naturally rot resistant (requires no harmful chemical treatment), and it is a nitrogen fixing plant.

Black Locust has some unique properties: it grows in knotty, twisty, small diameters, and it should be cultivated young, before it is attacked by locust boring insects. An efficient design can harness these characteristics. Black Locust is typically procured green. Careful detailing can be employed so that the material can season in place, eliminating the time and expense of kiln drying for some applications.

Black Locust timber was chosen because it is a durable, attractive, and naturally rot resistant wood, superior to pressure-treated lumber while non-toxic to the environment. This makes it a locally grown alternative to chemically treated lumber, endangered tropical woods, and decay prone woods. The fast-growth timber is used in small diameters, and has a smaller carbon footprint than steel or concrete.

2. Squibb Bridge

Squibb Bridge forms a functional link launching near the Brooklyn Heights Promenade, crossing

Fig. 1 Squibb Bridge: a) main spans assembled off alignment and erected with a single crane.

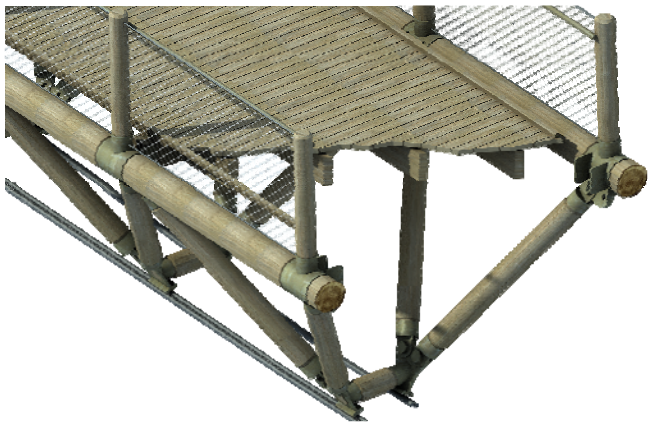


Fig. 2 Squibb Bridge: exploded view.

over Furman Street, and descending to the new Brooklyn Bridge Park along the East River. A design goal was to develop a bridge that is modest and that feels like it is part of the park, as opposed to a bridge in a park. Black Locust seemed to be an ideal choice – its tendency to darken, warp, and check as it seasons in place would create the sense of a weathered trail bridge.

The concept is an underslung suspension bridge composed of Black Locust timbers, and supported from below by a draped steel cable with props. The slender deck is just 25cm (10-inches) deep, with main spans up to 36.7m (120'-6”). To form continuous structural members the logs were turned on a lathe, and joined by steel pipe connections. The 150mm diameter (6”Ø) diagonal logs terminate in steel pipe fabrications, which are attached to the edge beams by bolted connections.

Bolted connections were designed to initially function as four bar linkages during fit-up, allowing the diagonal elements to be rotated into position and adjusted as required. Connection plates could be fabricated with a single bolt hole, the members could be assembled and adjusted as required, and the remaining bolt holes drilled to fit after completing the assembly.

After dry fitting the bridge spans, the timber-to-pipe connections were injected with a two part epoxy resin. The epoxy forms an integral structural connection by way of its bond strength, its compressive strength, and its confinement as it fills the annulus between steel and wood. Injection yielded mixed results, and is not easily inspected by the naked eye. By comparison, coating the ends of the logs with epoxy (buttering) prior to slipping them into the pipe could more reliably reduce the likelihood of forming air pockets within the connections, but would eliminate the adjustability of joints during fit-up.

Given the form of the bridge and the light weight of the wood, the main spans are quite flexible in the vertical and torsional modes, although measured vibrations are within the comfort criteria outlined by Sétra [17]. Horizontal vibrations are negligible.

3. TA Footbridge

The TA Footbridge at Vassar College in Poughkeepsie, NY is a Town lattice truss with Burr arch. It spans 16.4m (53'-8”) over the Casper Kill Creek, connecting student apartments to the campus. The new structure was built to replace an existing bridge that had deteriorated and become unsuitable for pedestrian traffic. Existing abutments were reused with some modification in width to accommodate a roughly 2.4m (8 ft) wide path suitable for combined pedestrian and bicycle traffic.

The previous bridge was composed of two unpainted steel W10x45 beams with timber decking and railing mounted on top. The steel framing was perched a few feet above the creek, and had undergone severe section loss due to advanced corrosion of the unpainted steel exacerbated by seasonal use of de-icing salt. The old bridge developed a distinct sway that disturbed pedestrians, and an inspector recommended the permanent closure of the deficient span. Considering the history of corrosion issues, a solution using black locust members and connections seemed appropriate.

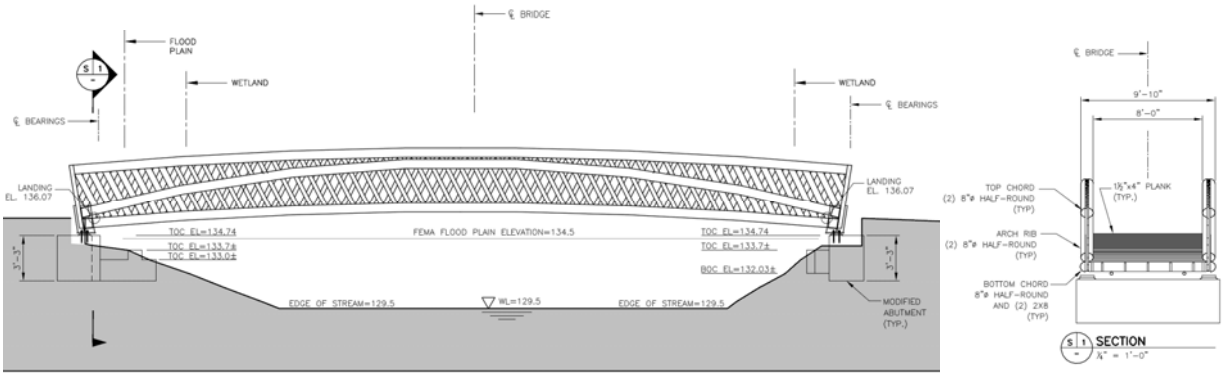


Fig. 3 TA Footbridge: Elevation & Section

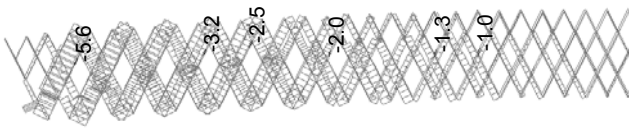


Fig. 4 TA Footbridge: F.E. model – Axial force in lattice members (kips) AASHTO STR-I combo

Fig. 5 TA Footbridge: F.E. model – Axial force in chords & arch rib (kips) AASHTO STR-I combo

The bridge adapts methodologies developed for covered bridges, however with the use of black locust, no cover is required (the cover serves to protect untreated timber from rot). The characteristics of a traditional Town lattice truss were appealing for a number of reasons. The lattice truss addresses a major shortcoming of other truss types, namely the lack of redundant load paths. Whereas typical truss bridges are highly non-redundant (i.e. all members are generally fracture critical), the lattice truss has a tremendous capacity to redistribute loads in the event of accidental loss of a member, and the lattice members serve as the web of a beam that is comprised of truss chords. Moreover, the TA Footbridge is detailed explicitly such that all members are replaceable in place. Town was successful in his original concept partly because of its extreme simplicity and economy, and in this case the bridge was built for about \$1,819/m^2 (\$169 / S.F.).

The principal advantage of a lattice truss is the use of small members and convenient framing, where the top and bottom chords are split in half and connected to either side of the lattice work. The chords need not be continuous, as long as the splices are staggered. The historical use of timber pegs (treenails) probably originated with ship building, was used in construction of Shaker furniture, and was adopted by Town again for its simplicity.

The burr arch serves as a redundant system whereby arch behavior is utilized to take best advantage of wood's high compressive strength. The bottom chord has a slight arch to mobilize compression behavior to the extent possible.

The top chord and arch members are comprised of 200mm (8") rounds cut in half and fastened on either side of the 50mm x 100mm (2"x4") lattice members. The bottom chord is comprised of a 200mm (8") round, cut in half, on the outside face and twin 50mm x 200mm (2"x8") members on the inside to support the 50mm x 100mm (2"x4") decking. The remaining framing for the deck are 100mm x 200mm (4"x8") joists, supported by joist hangers and span transversely with secondary 100mm x 150mm (4"x6") joists framing longitudinally.

Fig. 7 TA Footbridge: truss layout, assembly, and delivery (photos by Tim Leiching).

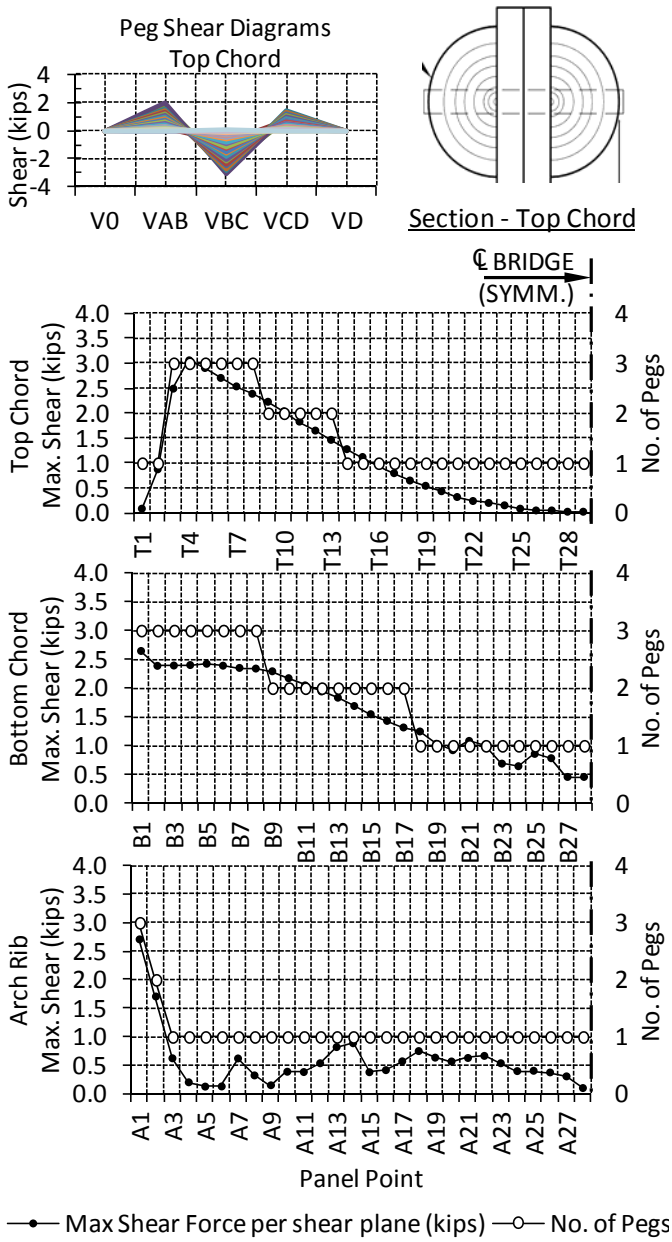


Figure 6 TA Bridge: maximum shear force per peg shear plane and no. of pegs at each panel point.

The use of timber peg connections presented some design challenges. Although treenail construction is ancient, it is unaddressed by the NDS and AASHTO design codes. Some load tests were performed in 1968 with full size joints with 50mmØ (2"Ø) White Oak trunnels for construction of the Union Street Bridge. The trunnels reportedly had ultimate strength of 106kN (24,000lbs) loaded in double shear [11]. Recent Timber Framers Guild guidelines provide comprehensive guidance for the design of mortise and tenon joints [18, 14], and with some modification was suitable to our purposes.

A useful scheme for truss fabrication is outlined by Graton [11]. The trusses were assembled in the shop by laying out the timber members on a level area. Chords were laid out with their natural sweep oriented in the direction of the bridge camber. The chords, diagonals, and arch ribs were clamped together, but within days the timber had moved significantly as the wood began to dry. In lieu of clamps, a single decking screw was temporarily installed in a pre-drilled hole at each peg location along the top and bottom chords. Over-fastening was an effective means of controlling the movement of the green timber.

With the trusses firmly clamped together, the holes for the timber pegs could be drilled. It was unclear what size holes to drill for the 31.8mm (1¼") diameter pegs, given the movement of the wood with changing moisture content. Metalworking bits were used such that hole diameters could be adjusted as needed in 0.4mm (1/64") increments. Pegs were driven using a 7kg (16 lb) hammer and a steel driving cap sized to prevent over-driving the pegs beyond flush. Based on Graton's description, we determined that if the holes are properly sized to match the pegs, each hammer blow should drive the peg about 12.5mm to 25mm.

The bridge halves were then raised into the vertical position after securing the bolted and peg connections, and the joists were installed using off-the-shelf face mount joist hangers. The completed span was shipped to the site on the back of a truck and erected using a single crane. Decking planks were installed on site, reducing the shipping and lifting weight of the span, and providing access to install conduit from above.
While we did not have the ability to control moisture content of the bridge members (wood was delivered green), the pegs were kiln dried – a combination that can be favorable in service, as the seasoning of the bridge members will cause the holes to shrink and tightly grip the pegs over time.

On subsequent inspection, the bridge is stiff and robust. Checking has begun for the exposed top ends of the lacing members. Small patches of mold were observed in three locations where a small skin of sapwood remains on the members. In time, the sapwood will naturally fall off, leaving only the heartwood, which is not hospitable to mold growth. Rather than waiting, the sapwood can be scraped or sanded down to heartwood.

4. Some Characteristics of Black Locust

Black Locust has been used by others for ship building, fire wood, and animal fodder. It is native to the U.S., and since the 1600's has been cultivated in Eastern Europe, France, Korea, China, and India [12]. It forms 23% of forested area and 19% of annual timber output in Hungary [16]. Significant differences have been reported within the species, and in particular Shipmast Locust is known to grow extensively in the Hudson Valley and on Long Island in New York [15]. Black Locust is attacked by Locust Borers (*Megacyllene robiniae*) in the U.S., but not Europe.

Fig. 8 Locust Borer (Megacyllene robiniae). [13].

The *Wood Handbook* indicates average values for strength, density, and modulus, which appear to be based on testing performed during the 1920's. Since then, the character of timber stock in the U.S. has changed substantially [9]. Recently, some demonstration projects have used small diameter Black Locust logs [1, 8], though there does not appear that any rigorous testing program has been performed to establish the statistical strength characteristics of the species. There seems to be little precedent in terms of its structural use in the U.S.

Black Locust has been cultivated extensively in Hungary, where it is used for glulam beams, residential housing, tool-sheds, fence components and panelling. One could presume there must be a trove of information on the mechanical properties available in the Hungarian language.

4.1 Experience of Others

With the idea that practical experience can be at least as valuable as research reports, we reached out to local professionals with experience using Black Locust – a timber mill, a trail bridge builder, and a wood specialist. Their works have included boardwalks, small trail bridges, fences, park furniture, and sculptures. They provided some useful guidelines for using Black Locust.

Black locust is typically procured in green condition, as the trees are generally cut down on demand. Lead time varies significantly depending on accessibility of the tree stands in the given seasonal conditions (snowfall, mud, etc.). The wood is typically used in its green condition, and allowed to season on place. In its green state, it is considerably weaker than the reference values at 12% MC (see Table 1), thus structures built using green wood are weaker at the beginning of their service life, and get stronger over time. In the absence of better test data, it has been suggested to use the published design values for White Oak.

Table 1 Ratios of dry to green clear wood properties as reported by ASTM D2555 [5]

Modulus of Rupture	Modulus of Elasticity	Compression Parallel to Grain, Crushing Strength
1.40	1.11	1.50

Seasoning can cause the wood to warp, twist, check, and split. It can take up to three years to season in place, and in that time the movement of the wood can be significant enough that it has been described as behaving like it is still alive. Conventional wisdom is that overfastening is the key to managing twisting and warping, and our experience so far has corroborated this advise. To reduce checking, the ends of members can be treated with an aqueous wax sealer (such as Anchor Seal). Some checking and splitting is unavoidable if the wood is allowed to season in place, and some kiln dried decking planks have reportedly warped and checked considerably despite efforts to control it.

Inspection of the source wood can be difficult. Locust Borer larvae burrow inside the logs and can hollow them out without leaving a sign other than a small entry hole, which is easily concealed by the bark. The wood species is not addressed by any American grading agencies, and for this reason at least one local mill would not supply wood for the projects knowing that it would be used for bridge construction.

5. Establishing strength characteristics

A robust testing program seemed like a sensible approach to establishing design values for the project-specific wood sources. For each of the two bridges, a testing program was implemented to substantiate key design and procurement decisions. No attempt was made to perform a statistically

robust program to characterize the mechanical properties of the entire species, so the data should be examined critically and carefully before contemplating its use for a different project. Indeed, the testing uncovered significant differences in the strength of wood used for the two projects. Wood was tested in its green state to establish conservative lower bound mechanical characteristics, consistent with the condition of the bridges at beginning of service life.

5.1 Testing Regime

The bridges were constructed independently by two different contractors. For Squibb Bridge (Block A), the contractor procured and milled most of the wood in his own shop. Logs were reportedly cut from trees near the New York – Pennsylvania border. For the TA Footbridge (Block B), wood was prepared by a mill, and the source location was not reported.

Initially, a few small clear specimens were cut from a log selected to be representative of the logs at the mill (Block 0), and were tested at the materials testing lab at New York City College of Technology, CUNY. Testing flexure and compression parallel to the grain demonstrated the general strength characteristics of the species, and helped to qualify the material that the timber mill could produce. While the Block 0 specimens tested below the published average values of Black Locust, they were superior to White Oak.

Table 2 Compression Test Data: In Block 0 a few small clears were tested to qualify the wood produced by the mill. Block A is selected to be representative of the timber used for Squibb Bridge. Block B is selected to be representative of TA Footbridge.

Compression Parallel to Grain		n	Ultimate Compressive Strength			Modulus of Elasticity		Moisture Content		Density		Specific Gravity	
			Mean (ksi)	COV (%)	PTL	Mean (ksi)	COV (%)	MC (%)	COV (%)	Mean (pcf)	COV (%)	Mean	COV (%)
FPL White Oak	Green	20	3,560	18	2,321	1,250	22	Grn				0.60	10
	12%	20	7,440	18	4,851	1,780	22	12				0.68	10
FPL Black Locust	Green	20	6,800	18	4,434	1,850	22	Grn		58		0.66	10
	12%	20	10,180	18	6,638	2,050	22	12		48		0.55	10
Block 0	Small Clears	2	6,582	2.6	6,037	2,011	14.6						
Block A	Small Clears	45	5,219	12.2	4,051	1,770	16.2	24.8	17.0	47.9	5.4	0.61	4.1
	150mmØ x 1.83m logs (6"Ø x72")	10	4,107	18.4	2,518	1,765	21.3	32.3	25.9	60.0	16.5	0.73	5.4
Block B	Small Clears	41	7,156	20.8	4,424	1,327	49.5						

A second round of testing (Block A) involved a large batch of small clears tested in flexure and compression, as well as several full size members loaded to failure [4]. We performed compression testing on 10 logs approximately 150mmØ x 1.83m long (6”Øx72”) diameter with natural taper and curvature. Flexure testing was performed on five rough sawn beams 50mm x 200 mm 2.49m long (2”x8”x98”) and on five beams 100mm x 267mm 2.49m long (4”x10 ½”x98”).

Block B consisted of 41 small clears loaded in compression parallel to the grain. The specimens were cut from samples taken from the stack of wood in the shop, selected to be representative of the wood used on the bridge. Test data are presented in Tables 2 and 3.

Table 3 Flexure Test Data: See caption for Table 2.

Flexure		n	Modulus of Rupture			Shear Parallel to Grain			Modulus of Elasticity		Density		Specific Gravity		Moisture Content	
			Mean (psi)	COV (%)	PTL	Mean (psi)	COV (%)	PTL	Mean (ksi)	COV (%)	Mean (pcf)	COV (%)	Mean	COV (%)	(%)	COV (%)
FPL White Oak	Green	20	8,300	16	5,733	1,250	14	912	1,250	22			0.60	10	Grn	
	12%	20	15,200	16	10,499	2,000	14	1,459	1,780	22			0.68	10	12	
FPL Black Locust	Green	20	13,800	16	9,532	1,760	14	1,284	1,850	22	58		0.66	10	Grn	
	12%	20	19,400	16	13,400	2,480	14	1,809	2,050	22	48		0.55	10	12	
Block A	Small Clears	45	11,249	14	8,380				1,542	14.9	47.3	6.4	0.61	6.8	24.6	18.2
	50mm x 200mm x 2.49m	5	10,174	8.7	7,993	650	8.9	507	1,705	4.6	48.7	4.9	0.65	9.5	32.1	6.4
	100mm x 267mm x 2.49m	5	7,552	33.6	1,300	638	33.5	111	1,625	7.9	56.2	3.8	0.68	5.9	36.5	3.8

5.2 Yellow Black Locust, an unnatural selection, and clearly unclear clears

The two blocks of tests were intended only to characterize the material used for their respective projects, but it is tempting to wonder whether a meaningful comparison could be made between the data sets for small clears loaded in compression parallel to the grain (Table 2). In terms of statistical rigor, the body of data could seem almost like a disaster. Samples were not randomly selected, nor were they selected to be representative of the species [7]. Block B specimens were rough sawn, and were not planed down to the exact dimensions normally required [3]. Though clear specimens are typically required to be free, or practically free, of all blemishes or defects, several of the test specimens exhibited checking, splitting, knots sound and unsound, discoloration, sapwood, mold spots, and Locust Borer holes.

From the context in which the test blocks were envisioned, these characteristics are suitable. The selection was intended be representative only of the wood used in construction of the two bridges.

Despite the sampling anomalies, some interesting information can be inferred by comparing the two

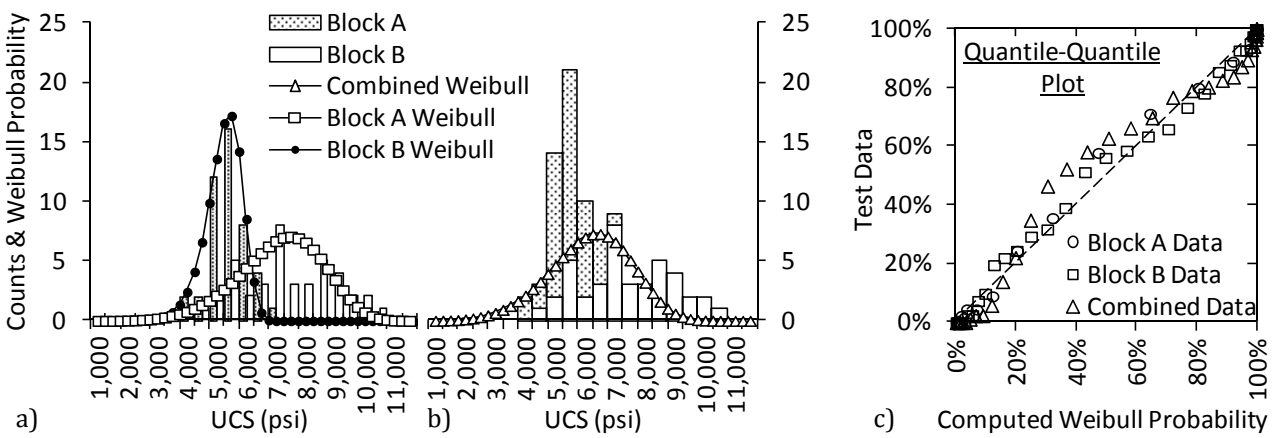


Fig. 9 Test data for small clears loaded in compression parallel to the grain: a) Block A and Block B data, with Weibull pdfs, b) combined data with Weibull pdf, c) Quantile-quantile plot illustrates a relatively poor fit for the combined data.

blocks of data. Compared to Block B, Block A exhibited a smaller mean strength and lower COV. Lower 5% one-sided confidence limits were computed at 75% confidence in accordance with ASTM D2915 [6], also referred to as the parametric tolerance limit (PTL). Despite having fewer samples and higher COV (broader spread of observed UCS), Block B has a higher PTL, and thus a higher design strength could have been used for TA Footbridge as compared to Squibb Bridge. Though the ASTM standard uses a normal distribution for computing the PTL, Weibull distribution is generally considered a better fit, and is used as the basis for calibrating Eurocode design values. A Weibull probability distribution function was fitted to each data set, shown in Fig. 9.

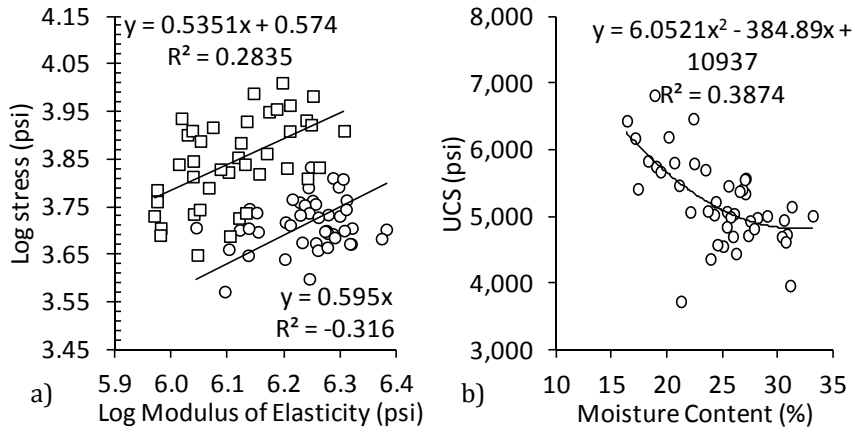


Fig. 10 UCS Relationships: a) ○Block A and □Block B MOE vs. UCS, b) Block A MC vs. UCS.

Block A specimens were tested at an accredited laboratory in accordance with ASTM D143 [3]. Moisture content and specific gravity were not measured for Block B specimens, since the samples were taken directly from the stack of wood used on the bridge, and the wood in service would never be greener than the samples. In both test blocks, the wood was considered green, as the specimens were freshly sawn from recently felled logs.

The character of the samples visibly differed. Block A was reddish brown heartwood, whereas Block B was bright yellow heartwood characteristic of Shipmast Locust, also called Yellow Locust. The production timber associated with Block B is generally straighter grained, with fewer and smaller unsound knots, also consistent with Shipmast Locust.

6. Visual inspection, acoustic testing

A major challenge was establishing quality control standards for an unusual wood species, particularly for small diameter logs. Existing log grading guidelines that we are aware of [20] penalize small diameter logs for their modest yield at the mill, and could not be used directly for a project that specifically intends to use small logs. For rail bridges, AREMA (7-1.2.1) permits use of hardwoods complying with the requirements of the Northeastern Lumber Manufacturers Association. NELMA does not address Black Locust. Several state DOTs allow wood to be used based solely on the Engineer's visual inspection. For awhile, it seemed we were out on a limb.

6.1 Visual Inspection

Visual inspection is the primary means of ensuring a quality product. ASTM D25 offers guidance for inspection of round timber piles, and was an excellent basis for a visual inspection program that could be tailored to Locust's peculiarities.

Unsound Knots (or rotten knots) are essentially a spongy void, and can severly limit the load carrying capacity of a member. In the Block B tests, two specimens with soft knots buckled under considerably lower loads than the clears, whereas specimens with sound knots were virtually indistinguishable from the clears.

Sound knots were limited to 20mmØ (¾”Ø), 13 per meter (4/foot). In practice, this was not achievable for Squibb Bridge, and unsound knots were scraped out and repaired with epoxy. The *Timber Bridges Manual* offers guidance on epoxy repair and coring of timber members.

Slope of Grain weakens timber, and Hankinson's formula is deeply embedded in the NDS code and standard practices for assessment of historical timber structures. Given the twistiness of Black Locust, the strength of rough sawn and round turned members could be affected by excessive slope of grain, and for Squibb Bridge at least one edge beam was rejected on this basis. Slope of grain was limited to 1:6.

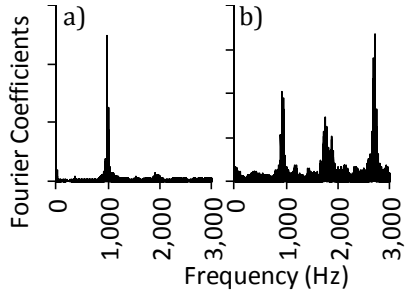


Fig. 11 Stress Wave Frequency: a) Typical log specimen with one frequency response, b) Rejected log specimen with three frequency responses as stress wave partially reflects off of large internal imperfections.

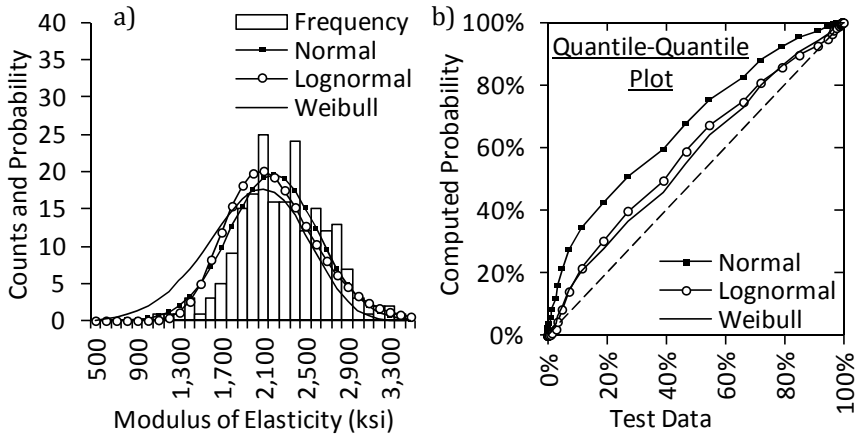


Fig. 12 Acoustic test data for logs: a) Block 0 data with Weibull pdfs, mean 2,264ksi, COV 20.3%, b) Quantile-quantile plot suggests Weibull distribution is suitable for the combined data.

6.2 Acoustic Testing

The relationship between Elastic Modulus and stress wave velocity has been known at least since St. Venant's experiments [19], and is well established in the literature [10]. When a specimen is struck with a hammer at one end, a stress wave travels along the axis of the specimen at a velocity that is related to the stiffness and the density of the material. The stress wave can be registered as a vibration and as an audible percussive sound. When the stress wave (sound wave) reaches the opposite end of the specimen, it is reflected off of the face, and returns to the struck end, and is reflected back and forth for several cycles. The MOE can be computed as:

$$\text{MOE} = \rho \cdot V^2 \tag{1}$$

A similar approach is used for pile integrity testing to reveal internal voids or weaknesses otherwise uninspectable. This ability could be particularly useful for detecting voids carved out by Locust Borers, and could be adapted to identify and even locate the position of defects in logs.

where:
MOE = modulus of elasticity
ρ = density
V = velocity of stress wave traversing axis of specimen $= 2 \cdot L \cdot f$
f = frequency of stress wave reflecting from end to end

While some commercial products are available for measuring the stress wave velocity, they were beyond the reach of this project. A homemade approach was taken to develop an acoustic testing methodology capable of achieving high precision results as inexpensively as possible, using readily available equipment.

The setup consisted of:

- A laptop computer
- Inexpensive ($5) headphones (JVC model HA-FX8-B)
- Adhesive tape, measuring tape, and a strong scale

Computers typically process sound at a frequency of 44,100Hz (sampling rate), and when plugged into a microphone jack typical headphones can capture soundwaves at pitches up to 22,050Hz with sensitivity of 101dB / 1mW (thus calibration frequencies of up to 8,200Hz for steel specimens were well within the Nyquist frequency of the system). Accuracy of the setup was compared to a professional musician's electronic tuning device. Measured frequencies were in agreement ±1Hz. Software was written to record the soundwave, process it using a Fast Fourier Transform, and detect peak frequencies (f). Length, diameter, and weight were measured the hard way (L and ρ).

Concurrent with Block 0 mechanical testing, acoustic testing was performed on 207 logs at the mill, with the aim of capturing elastic modulus data representative of the population of logs used for Squibb Bridge. Test data are presented in Fig. 12, alongside the computed Weibull distribution. Voids were successfully detected by the software as having more than one strong frequency peak. UCS relationships from testing of small clears could be used to relate acoustic elastic modulus to the strength, and adjusted for moisture content (Fig. 10).

7. Discussion, Conclusions and Acknowledgements

The research comprises contributions of several individuals. Block A testing was performed under contract at the AEWC, University of Maine by Russell Edgar and Jon Hill. Block B testing was done at New York City College of Technology, CUNY with assistance from Dario Feliciangeli.

8. References

[1] Al-Khattat, I., "Light Prestressed Segmented Arch (LPSA) Bridges: A Demonstration of Sustainable Engineering", *Structural Engineering International*, V. 18, N. 1, 2008, pp. 62-66.

[2] ASTM Standard D25, "Standard Specification for Round Timber Piles", ASTM International, West Conshohocken, PA, 1999 (2005).

[3] ASTM Standard D143, "Standard Test Methods for Small Clear Specimens of Timber", ASTM International, West Conshohocken, PA, 2009.

[4] ASTM Standard D198, "Standard Test Methods of Static Tests of Lumber in Structural Sizes", ASTM International, West Conshohocken, PA, 2009.

[5] ASTM Standard D2555, "Standard Practice for Establishing Clear Wood Strength Values", ASTM International, West Conshohocken, PA, 2006 (2011).

[6] ASTM Standard D2915, "Standard Practice For Sampling and Data Analysis for Structural Wood and Wood-Based Products", ASTM International, West Conshohocken, PA, 2010.

[7] ASTM Standard D5536, "Standard Practice for Sampling Forest Trees for Determination of Clear Wood Properties", ASTM International, West Conshohocken, PA, 1994 (2010).

[8] Elliott, J., "The Triakonta Structural System: From Toy to Tower," *Proceedings of the International Convention of Society of Wood Science and Technology and United Nations Economic Commission for Europe – Timber Committee*, Oct. 11-14, 2010, Geneva.

[9] Forest Products Laboratory, *Wood Handbook – Wood as an Engineering Material*, General Technical Report FPL-GTR-190, U.S. Department of Agriculture, Madison, WI, 2010.

[10] Graff, K. F., *Wave Motion in Elastic Solids*, Oxford University Press, London, 1975.

[11] Graton, M. S., *The Last of the Covered Bridge Builders*, Clifford-Nicol, Plymouth, NH, 1978.

[12] Hanover, J. W., "Black Locust: An Historical and Future Perspective," *Proceedings: International Conference on Black Locust: Biology, Culture, & Utilization*, Dept. of Forestry, Michigan State University, 1992.

[13] Hough, Cody, "A Locust Borer (*Megacyllene robiniae*)." Photo, *Wikipedia.org*, 9 December 2008, retrieved 21 July 2013. <http://upload.wikimedia.org/wikipedia/commons/c/c6/M_robiniae.jpg>

[14] Miller, J. F., Schmidt, R. J., and Bulleit, W. M., "New Yield Model for Wood Dowel Connections", *ASCE Journal of Struct. Engineering*, Vol. 136, No. 10, 2010, pp. 1255-1261.

[15] Peattie, D. C., *A Natural History of North American Trees*, Houghton Mifflin Company, New York, 1948 (2007).

[16] Rédei, K., Osváth-Bujtás, Z., and Veperdi, I., "Black Locust (Robinia pseudoacacia L.) Improvement in Hungary: a Review", *Acta Silv. Lign. Hung.*, Vol. 4, 2008, pp. 127-132.

[17] Sétra, *Assessment of Vibrational Behaviour of Footbridges under Pedestrian Loading*, Sétra/Association Français de Génie Civil, Paris, 2006.

[18] Timber Frame Engineering Council, *Standard for Design of Timber Frame Structures and Commentary*, TFEC 1-2010, Becket, MA, 2010.

[19] Timoshenko, S. P., *History of Strength of Materials*, McGraw-Hill, New York, 1953.

[20] Wengert, E. M. and Meyer, D. A., *Guide for Grading Hardwood Logs*, Forestry Facts No. 74, University of Wisconsin-Madison, Department of Forestry, 1994.

Field Load Testing of the First Vehicular Timber Bridge in Korea

Ji-Woon Yi
Ph.D. Student
Department of Civil & Environmental Engineering
Seoul National University
Seoul, Korea
jwyi@sel.snu.ac.kr

Wonsuk Park
Chief Researcher
Korea Bridge Design & Engineering Research Center
Seoul National University
Seoul, Korea
wpark@sel.snu.ac.kr

Chul-Young Kim
Professor
Department of Civil & Environmental Engineering
Myongji University
Yongin, Korea
cykim@mju.ac.kr

Sang-Joon Lee
Research Scientist
Korea Forest Research Institute
Seoul, Korea
lsjoon@forest.go.kr

Hyun-Moo Koh
Professor
Department of Civil & Environmental Engineering
Seoul National University
Seoul, Korea
hmkoh@snu.ac.kr

Summary

The first timber bridge for vehicles in Korea, a single-span, arched truss bridge 30m in length, was constructed in 2012. The bridge was designed and constructed as a part of national research projects for the development large timber structures funded by the Korea Forest Research Institute. This paper briefly introduces the development process of the bridge including design specifications, member tests, full-scale loading tests and finite element analysis. It then presents the results of the first in-situ truck load tests for the bridge. The tests are performed as an initiation of multiple-year monitoring project for performance assessment. Both static and dynamic responses such as deflection, mode frequency, and strains of major members are measured and compared to the analysis results of the design step.

Keywords: Vehicular timber bridge, Field loading test, Laboratory test, Finite element analysis.

1. Introduction

Timber's strength, light weight, and eco-friendly properties furnish features that are desirable for bridge construction, and they also present a natural and aesthetically pleasing appearance, particularly in natural surroundings. Due to these superior characteristics of timber as a bridge material, they have been used as main members of vehicular bridges in Europe and United States.

In Korea, however, timber has not been used as main member of vehicular bridges although the use of timber as a main structural member has been increasing for other structures such as pedestrian bridges and buildings. In an effort to apply timber to bridge construction, the Korean Forest Research Institute initiated a research project for the design and construction of vehicular timber bridge and the establishment of design specifications. As an output, a vehicular timber bridge was built successfully for the first time. This paper summarizes the research activities for the vehicular timber bridge in Korea, and the results of performance evaluation tests in the laboratory are briefly introduced. Finally field load tests and the results were presented.

2. Researches and development for vehicular timber bridge in Korea

In 2009, the Korean Forest Research Institute launched a research project for the design of a timber bridge. It started as the development of a bridge deck floor 30m in length. In this design, glue laminated timber (glulam) was used because it can be manufactured in a wide range of shapes, which is appropriate for the construction of a bridge deck floor with a relatively long span. In this research, the pre-stressed glulam deck floor system was designed based on the design guideline by USDA [1]. The required level of pre-stressing and design assumptions such as symmetric behaviour and non-separation at laminated faces were identified and verified.

Following the development of the pre-stressed glulam deck system, research on girder bridges was initiated in 2010. Girder bridges with two lanes and 25, 30 and 35m in length were developed. A sawn lumber deck floor system was adopted for the girder bridge, which was designed with a series of rectangular panels and was supported by 5 main girders. A design and analysis program for the girder bridge was developed and the design procedure was established. The structural behaviours of the bridge design were verified numerically through detailed 3D finite element analyses.

Based on the previous research, comprehensive research for the first vehicular timber bridge in Korea was started in 2011. The research project included the whole process including design, manufacturing, laboratory test, field test, and the construction of the bridge. Among the various bridge types, an arched truss structure was selected for a 30m single-span bridge due to the advantages of structural efficiency and aesthetic value (Fig. 1).

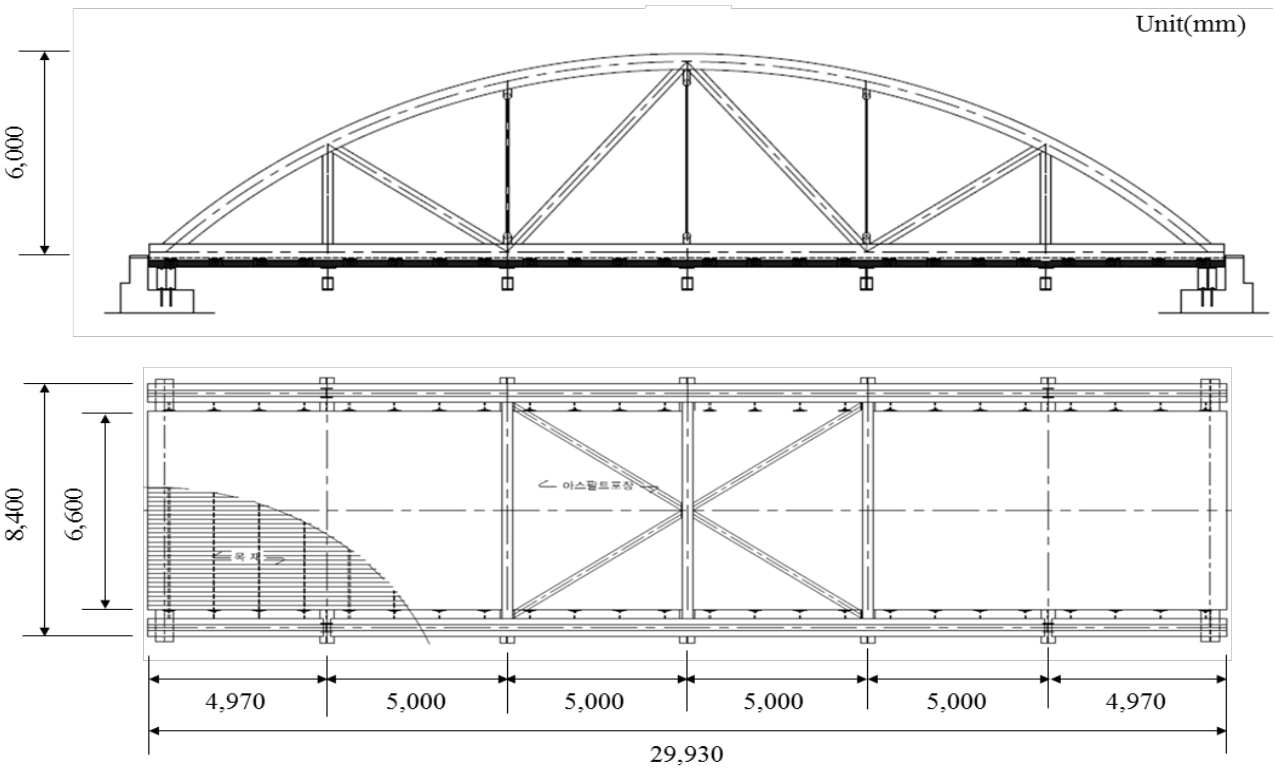


Fig. 1 Longitudinal section and floor plan of bridge designed in 2011 (Named Hanareum bridge later)

The main members of the arched truss were made of glulam, and dowel-type joints are used to connect the main members. The vertical members of truss, cross beams, pre-stressing elements, dowel plates and components to assemble were made of steel while the other main members are all made of wood. For the glulam deck system, stressed-laminated system, which was researched in previous studies, was chosen, due to the advantages of prefabrication, installation, and maintenance. A total of 5 cross beams were laid beneath the deck at the intervals of 5m to support the deck floor and to transfer the load from the floor to the truss members. An additional two cross beams were also installed to seat the bridge on the abutments. A number of codes, specifications and guidelines [2-6] were used to design the truss, joints and other detailed parts of the bridge and the allowable stress design method was applied. For the material of the main glulam members, pitch pine was chosen because it is one of the most common species of trees in Korea, would be predicted to provide cost-effectiveness for the bridge. Korean Industrial Standard provides the mechanical properties of symmetric laminated pitch pine timber as shown in Table 1, classified according to grade of strength. Among them, 10S-30B grade materials were selected for the bridge.

Table. 1 Mechanical properties of symmetric laminated pitch pine timber(KS F 3021)

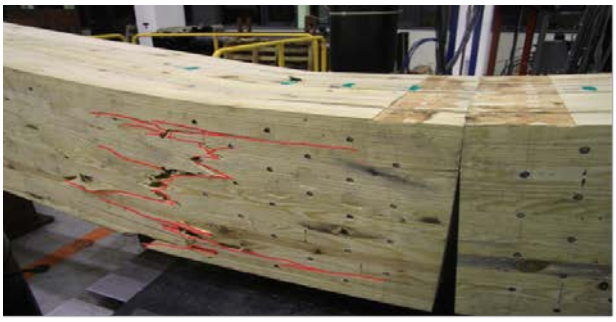
Grade of strength	Allowable Stress(MPa)				Modulus of elasticity(MPa)	
	Flexural		Axial		x-x axis	y-y axis
	x-x axis	y-y axis	Tensile	Compressive		
12S-37B	12.0	8.0	8.0	10.0	11,000	10,000
12S-33B	11.0	7.5	7.0	8.0	10,000	9,000
10S-30B	10.0	7.0	6.5	7.5	9,000	8,000
9S-27B	9.0	6.0	6.0	7.0	8,000	7,000

Using the above research, an arched truss timber bridge for vehicle was completely manufactured and tested in the laboratory before being constructed at the field. The next chapter presents the laboratory tests and corresponding results.

3. Full scale performance evaluation test in laboratory

In order to ensure the performance of members, joints and the entire bridge system, a series of full scale test was performed at Hybrid Structural Test Centre of Myongji University in 2011. The tests for glulam members consist of the bending, tensile and compression strength test, and static loading and fatigue loading tests were performed for the arch truss bridge [7].

In tests of the dowel-type joint connections, one of the most complex parts in timber structure design, structural performance and failure behaviour were verified by both a finite element analysis and experiments. Full scale test specimens for diagonal members and lower chord were manufactured to have same details as the connections of the bridge so as to assess whether the strength satisfies required level.



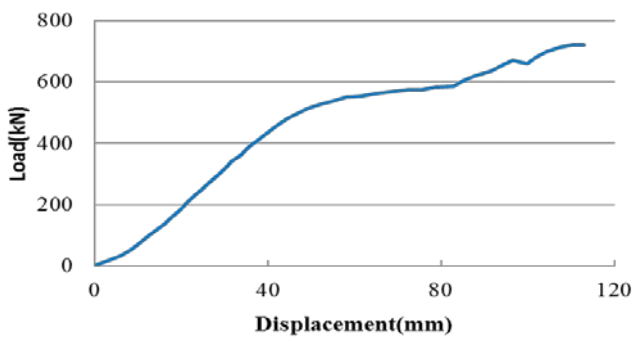


Fig 2. Flexural strength tests on the lower chord

As a result, the maximum stresses of members were expected to be from 80% to 95% of the allowable axial stress. In flexural tests on the lower chord, it was found that the strength of the

connected joints were sufficiently high such that maximum tensile stress of the connection is higher than the non-joint part of the member by 78%, due to the superior flexural strength of the steel dowel plate. Overall, the average axial strength, flexural strength and ductility were also at satisfactory levels.

The entire bridge span including arched truss, deck plates and cross beams was assembled in the structural test laboratory for static and dynamic loading tests (Fig. 3). The main objective of this full scale loading test was to verify whether the assembled bridge system behaves as designed. More specifically, the linearity for load-deflection relationship and the symmetry of deformed shape were investigated through a 384kN static loading case which is 90% of the design truck load. Test results showed that the resulting member forces were quite small compared with design values especially for timber, and the symmetric condition of whole bridge was well satisfied to meet well with the difference of deflection less than1mm. The maximum deflection of truss, cross beams and deck floor were also found to be smaller than design values.

Fig. 3 Static load testing in laboratory

In fatigue tests, cyclic loadings up to 50kN were applied. A total of 80,000 cycles were loaded throughout the test. It was found that there was no problem on the integrity of joints and members other than the slight increase in the center deflection was observed after the test. This deflection seemed to be caused by the relaxation of pre-stressing or the separation of joint connection occurred during the cyclic loadings.

4. Field load testing

4.1 Construction of the bridge

After the laboratory testing, the bridge was disassembled for transport and reassembled at the construction site. The construction field was located at a forest road of the Micheongol recreational forest, an eastern mountainous region near Yangyang city, Korea. After the installation of a fence and asphalt pavement, the construction of the entire bridge was completed by December 2012, and has been open to the traffic since the official opening ceremony that was held in April 19, 2013.

This bridge, given the official name of 'Hanareum bridge', is 30m in length, 8.4m in width and has a maximum load capacity of 110t. As the first vehicular timber bridge in Korea, Hanareum bridge is expected to serve as the groundwork for the construction of many more timber bridges in the future and establishment of Korean design specifications for timber bridges.

In order for that to occur, it is important to assess the present performance of bridge and to apprehend the long-term behaviour like loosen pre-stressing, deterioration etc. Furthermore, it is required to examine whether the bridge performs as it was intended. Therefore, field testing including static and dynamic load testing was performed May 10, 2013

Fig. 4 Hanareum bridge

4.2 Configuration of field load testing

The bridge was designed as a two-lane bridge with each lane loaded with a 43.2 ton truck. The loading tests were performed with two gravel loaded 27 ton trucks and corresponding calibration was conducted and the test results were compared to laboratory tests and design values. In order to assess the performance of the bridge and to obtain initial data for investigating its long-term behaviour such as loosen pre-stressing and deterioration, a series of static and dynamic load tests were performed in May, 2013.

The resulting deflections and strains at a series of locations were measured. One of main interests was the integrity of pre-stressed glulam deck. It was assumed that the laminated floor deck would retain its integrity without the separation of the glulam timbers if the deck was pre-stressed properly. In order to verify this assumption of integrity, a total of 13 LVDT's were installed beneath the deck floor to get a detailed shape of deflections in the lateral direction (Fig. 5-(a)). In addition, three more LVDT's were installed across the center cross beam for measuring the mid-span deflection of the bridge (Fig. 5-(b)).

(a) Center cross beam *(b) Deck floor*

Fig. 5 LVDT sensor attachment locations

Strains at important locations were also measured. The locations were identified through finite element analysis, which resulted in a total of 7 locations with the largest strain; 2 of upper chord and 2 of lower chord and 3 of deck floor (Fig. 6). Two accelerometers for identifying natural frequencies were installed at the outer sides of bottom deck floor (Fig. 7).

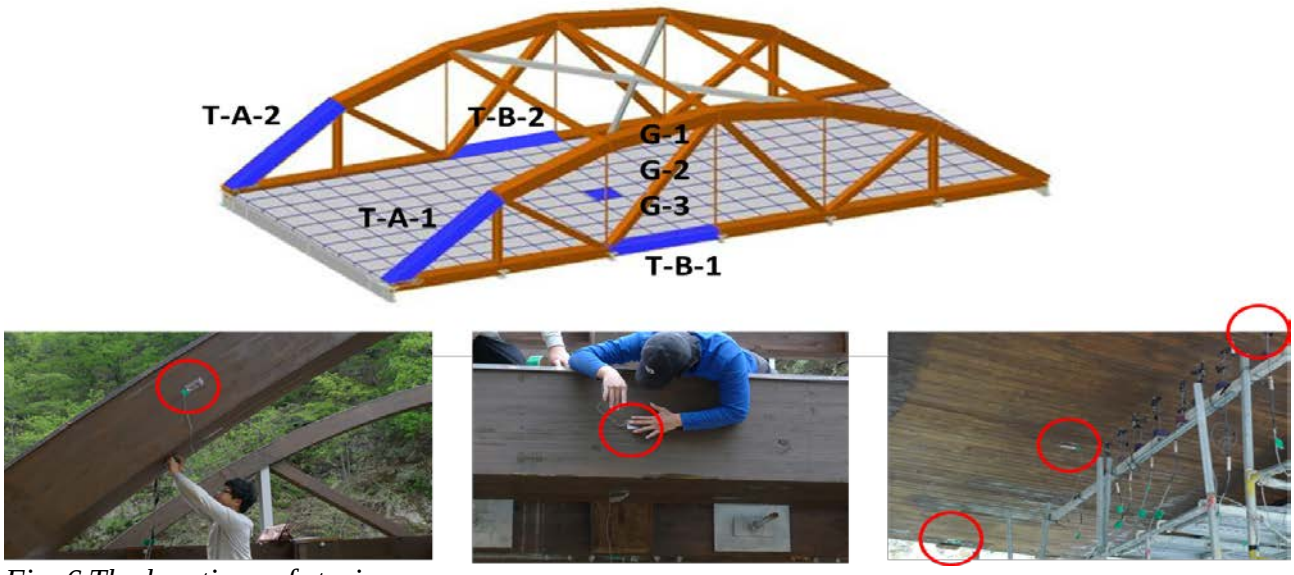


Fig. 6 The locations of strain gauges

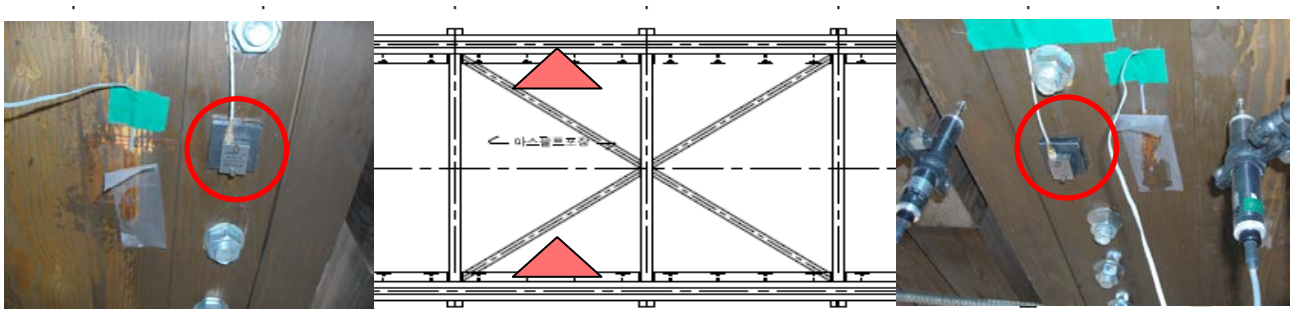


Fig. 7 The locations of accelerometers

In the static loading test, 6 load cases (LC1 - LC6) were set by positioning the truck alone, side by side, and in rows. And in the dynamic loading test, 6 load cases (LC7 - LC12) were set also by letting truck cross the bridge at speeds of 5km/h, 10km/h and 20km/h through each lane (Fig. 8, 9).

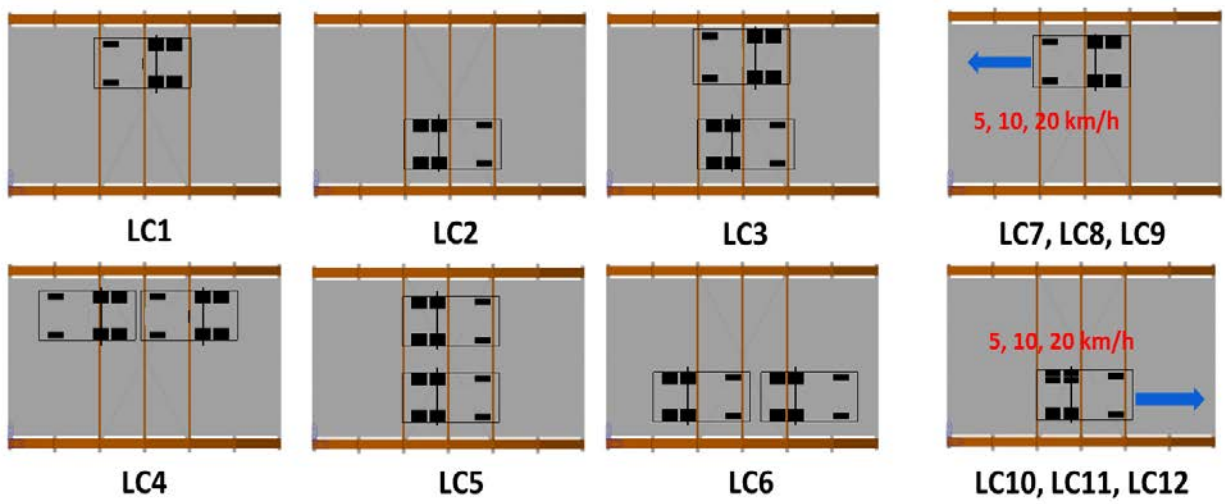
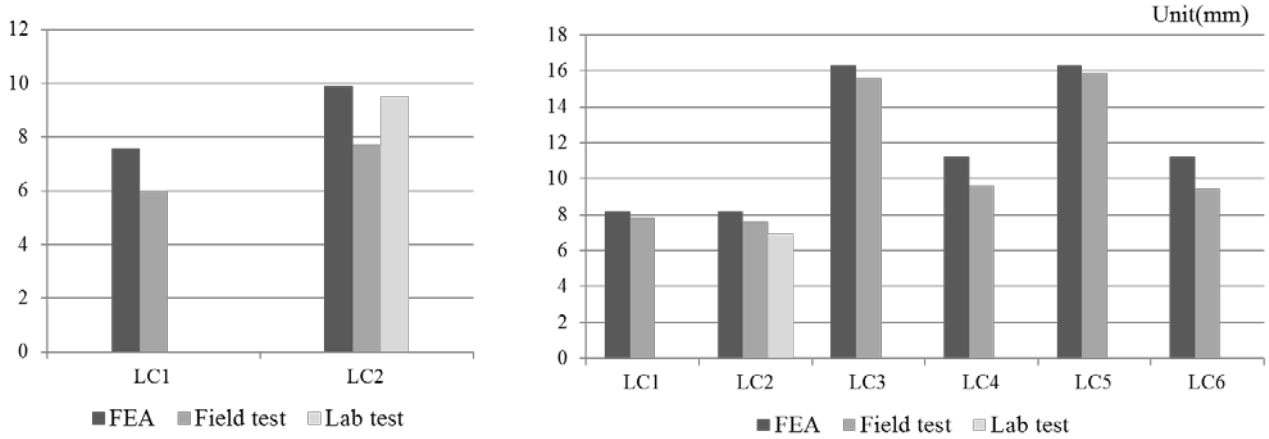


Fig. 8 Load cases for static(LC1-LC6) and dynamic(LC7-LC12) loading test

Fig. 9 Truck loading (left : static test, right : dynamic test)

4.3 Results and Discussions

The results of the static loading tests are plotted in Fig. 10. The displacements of the deck floor and the cross beam at the mid span are compared with their corresponding analysis responses. For a single lane load case, LC2, the results from the laboratory tests were also compared.



(a) Deck Floor *(b) Center cross beam*

Fig. 10 Maximum vertical displacement for static loading cases

As shown in Fig. 10, in general, good consistency was observed between the finite element analysis results and those of the field tests. Larger deflections in the finite element analysis are noted for all the cases, which can be explained by differences in the finite element model and the bridge structure. For example, the connections between timbers were modelled as hinge joints while the bridge has strong connections with dowel plates and bolts which can transfer flexural moment. Furthermore, asphalt pavement and fence, which can strengthen the stiffness of bridge, were not included in the finite element model. This overall stiffness increase is also observed in the comparison of deck floor deflections between laboratory and field test results of LC2.

The purpose of LC3 is to verify the linear behaviour by comparing it with superposition of LC1 and LC2. As shown in Table 2, the load-deflection relationship is found to be linear, and the maximum difference between LC3 and LC1+LC2 is 0.3mm.

Table. 2 Maximum deflections of cross beam for LC1, LC2 and LC3 (mm)

	LC1	LC2	LC1+LC2	LC3
Left	4.3	7.1	11.4	11.1
Middle	7.8	7.7	15.5	15.4
Right	7.3	4.5	11.8	11.9

The symmetry of right and left lanes is investigated by comparing the deflections of the center cross beam for single lane loading case pairs LC1-LC2 and LC4-LC6, which are single truck and double truck loading cases, respectively. A double lane load case of LC5 was also investigated and the test results are shown in Fig. 11, The results indicate that the symmetry of the deflection responses are well maintained, which can be considered to be an indication of the integrity and good quality of the construction of the bridge.

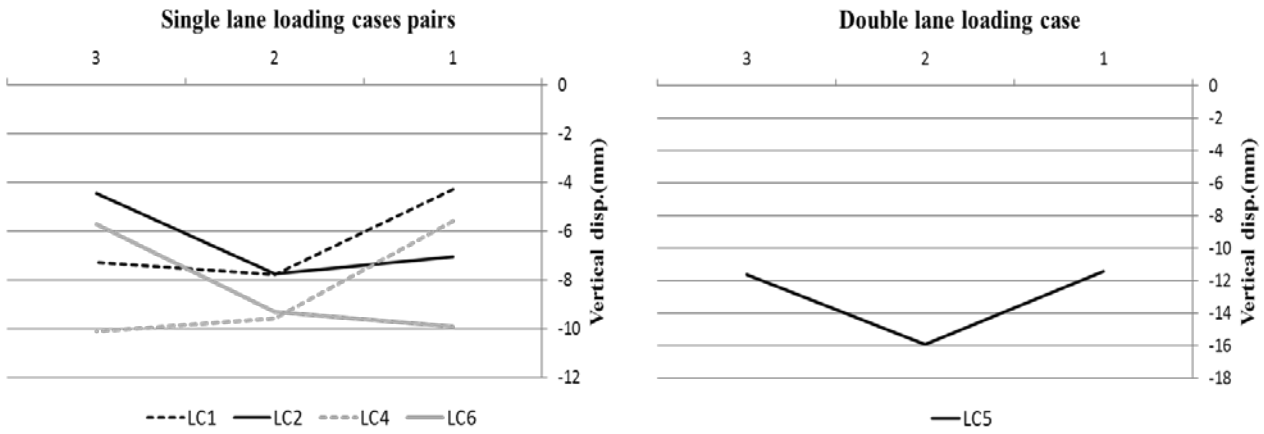


Fig. 11 Deflections of center cross beam for static loading cases

The natural frequencies of the bridge deck were identified from acceleration responses in dynamic loading tests. The first and second mode frequencies were 5.16Hz and 9.06Hz respectively (Fig. 12). Those results will be verified through a refined finite element analysis in further research.

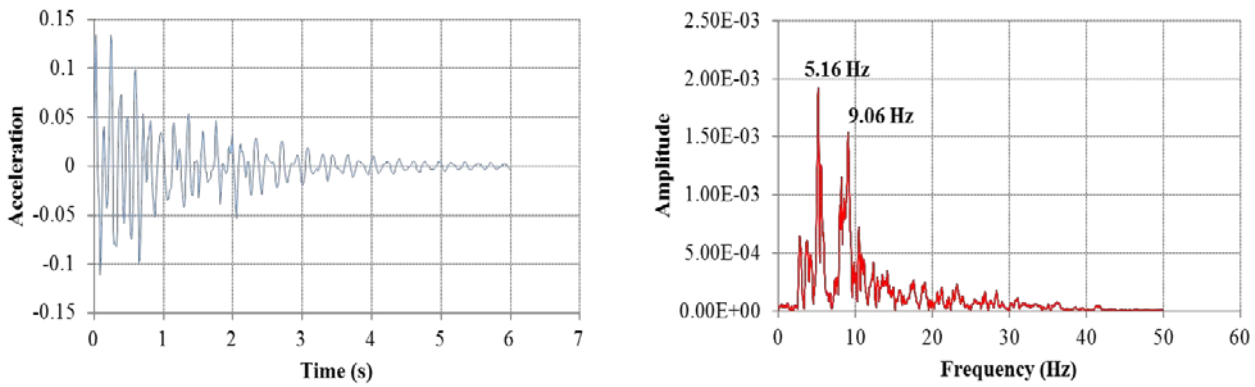


Fig. 12 Acceleration response and its frequency spectrum

In the dynamic loading test, the impact factors were also investigated. The impact factor, defined as the ratio of additional dynamic displacement to corresponding static displacement, depends on the condition of road surface, driving speed, structural characteristics, ratio of live load to dead load, and nature of the response, etc. In this study, for the different speeds of trucks, the maximum vertical displacements of the center cross beam at the crown were measured to calculate the impact factors. The results are depicted in Fig. 13. The largest impact factor was evaluated as 0.198 for 20km/h of the 1st lane. These dynamic test results will be analysed through a further analysis using refined 3D finite element model in a future project.

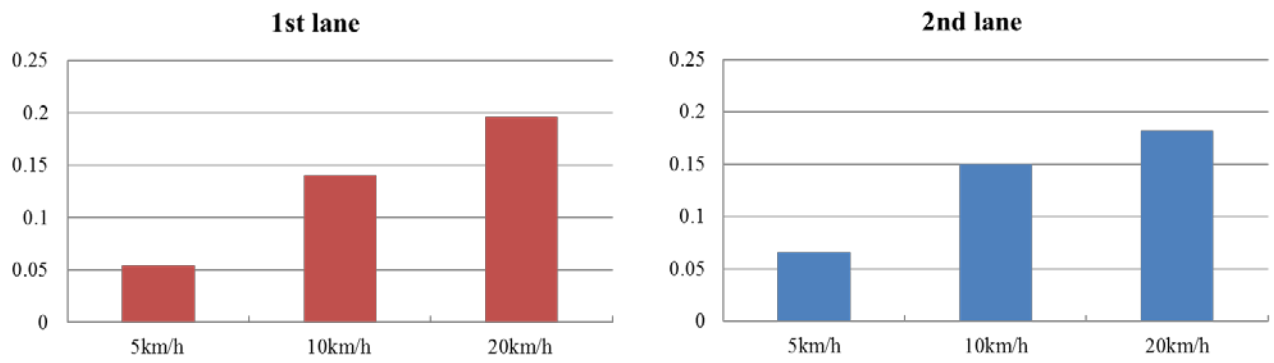


Fig. 13 Impact factors for dynamic load cases

5. Concluding remarks

Recent research and development activities for the construction of the first vehicular timber bridge in Korea are presented. The results of laboratory and field tests indicate that the manufacturing and fabrication quality of timber members is excellent, and that the bridge can perform as designed.

For the further research, a refined 3D finite element model which reflects the current status of the bridge will be established based on the results of the field tests. To investigate the long term behaviour of the bridge, periodic load testing is planned and the finite element model will be updated based on the corresponding results. The updated model will be used to assess current performance and to establish an appropriate maintenance plan for the bridge.

It would be expected that the successful construction of the Hanareum bridge will lead to the development of more vehicular timber bridges in Korea in the future. The development of new design specifications for timber bridges in Korea is also in the planning stages.

Acknowledgements

This research was supported and funded by Korea Forest Research Institute (0547-20130002).

References

[1] Michael A. Ritter, *Timber Bridges - Design, Construction, Inspection, and Maintenance*, Washington, DC., U.S., 1990.

[2] Korea Road and Transportation Association, *Korea Bridge Design Specification*, 2010.

[3] Korea Concrete Institute, *Korea Structural Concrete Design Code Commentary*, 2007.

[4] Architectural Institute of Korea, *Korea Building Code*, 2005.

[5] Architectural Institute of Korea, *Design Manual for timber structure*, 2008.

[6] American Forest & Paper Association, Inc, *National Design Specification for wood construction ASD/LRFD*, United States, 2005.

[7] Shim N-h., Kim D-k., Park Y-s., "An Experimental Study for Structural Evaluation of Timber Truss Bridge", *Korean Society of Civil Engineers Magazine*, Vol.60, No. 9, 2012, pp. 52-62 (in Korean).