Stretching the Limits – Modern Timber Bridge Case Studies

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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.
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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.

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.

Figure 10. Tension tie channels connected to heel assembly

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

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[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