SPRING CREEK PEDESTRIAN BRIDGE – A BRAND NEW HISTORIC LANDMARK

DOUGLAS SARKKINEN, PE, SE

GREGORY MINES, PE

BIOGRAPHY

Douglas Sarkkinen is a senior project manager in Otak, Inc's Vancouver, WA office with over 24 years of experience in structural engineering. He has significant experience with concrete structures and bridges, as well as with forensic and expert witness evaluations.

Gregory Mines is a design engineer in Otak, Inc's Vancouver, WA office. Greg has 6 years of structural engineering experience. His background is in both buildings and bridges, with focus on steel and timber design and seismic analysis.

SUMMARY

The Spring Creek Pedestrian Bridge is a 385 foot span cable stayed pedestrian bridge spanning over the Methow River located in historic Winthrop, WA. Scheduled for completion in the fall of 2011, the weathering steel bridge consists of steel stiffening trusses, cables, and built-up, latticed steel towers to capture the spirit of 1890 vintage steel structures.

Detailed modeling and analysis were performed for both static loads and for dynamic pedestrian loads and their vibration effects. One of the unique features of the bridge is the steel lattice towers; both tower legs and cross diaphragms are heavy angle iron members in the four corners with steel flat bars welded for the lattice members.

Erection of the towers, cable stays and truss assemblies required detailed planning and structural analysis to insure stability and adequate structural strength at all stages of erection. Additionally, cable forces, and stiffening truss cambers were all coordinated and checked so the bridge is installed in the final condition with balanced loads in all of the stays.

The completed bridge successfully provides the needed pedestrian access across the river for the Town, and nicely fits in with the historic context. Structural steel was the solution, and will provide the town with a unique amenity for years to come.

SPRING CREEK PEDESTRIAN BRIDGE
A BRAND NEW HISTORIC ICON

1.0 Introduction and Background

In north central Washington in the town of Winthrop, things today look much like they did in the late 1800's; the streets downtown are lined with boardwalks and stores clad in rough-cut lumber with the business name painted on a second-story façade. The feel of the Old West permeates the entire town, and when it came time for the town to contract construction of a new pedestrian bridge, the consensus was to build a bridge that resonated with the period style of the rest of the town.

During the preliminary design phase, one option stood out above the rest: an all-steel structure comprised of an angle iron truss and towers built in the classic lattice-frame style. The use of weathering steel throughout the structure would offer a rustic feel to the bridge in a way that concrete could never offer. This paper covers the design of the bridge as well as the construction and concludes with some lessons learned by the contractor, subcontractors, and engineers during the project.

1.1 Background

The Town of Winthrop received grant funding for a cable stay pedestrian bridge crossing the Methow River near the confluence of the Methow and Spring Creek. The Town retained a design firm for an alternatives analysis and a design was selected; then the Town sent out an RFP for design-build proposals to complete the design and construction of the Spring Creek Pedestrian Bridge. Otak, Incorporated teamed with Mowat Construction and together they compiled the winning proposal.

2.0 Design

2.1 Design Parameters

The Spring Creek Pedestrian Bridge was to be designed as a 385ft long by 12 ft wide cable stayed pedestrian bridge. Final design for the structure incorporated a 60 ft tall, built-up lattice-style towers and a steel truss utilizing X-patterned cross braces. Seven pairs of 1½ inch fore stay and back stay cables connected each leg of the tower to the truss. The deck was to be precast concrete.

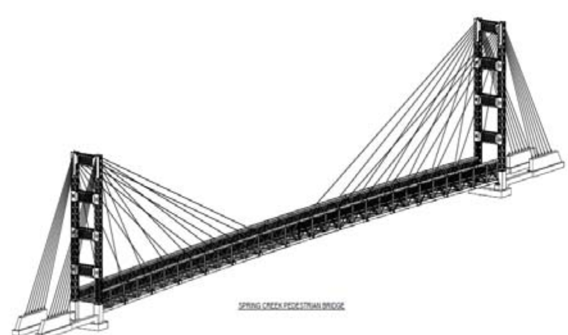

Figure 1 3D view of bridge

As mentioned above, all steel was to be A588 weathering steel (except cables and bolts). Foundations were preliminary sized as spread footings with pilings added for scour effects. The entire structure was outside of the 100-yr flood plain with a minimum 3ft clearance between the bottom of the structure and the 100-yr flood elevation.

Table 1 Design Criteria

Pedestrian	90 psf
Truck	12,000 GVW
Snow	90 psf
Wind	100 mph
Seismic	
As	0.24
SDS	0.55
Site Class	D

Design was performed per AASHTO LRFD Guide to Design of Pedestrian Bridges with references to PTI Design for Cable Structures and SÉTRA Analysis of Pedestrian Vibrations. Table 1 outlines some of the design criteria used when designing the structure. The structure was modeled in GT STRUDL for gravity design and for dynamic analysis of pedestrian dynamics and earthquake analysis.

2.2 Foundations

Preliminary design called for concrete piles; however, driven steel piles proved to be more economical and faster to install. Piles were steel HP 10 x 42 piles. Eight total piles at each tower were used averaging 30 ft in length at the north abutment and 50 ft in length at the south abutment. The piles at the north abutment were driven to refusal and the piles at the south end were driven to a specified minimum depth. The bridge was designed for soil bearing under spread footings for standard operation, but the piles added redundancy in bearing in the event of scour under the abutments.

2.3 Truss

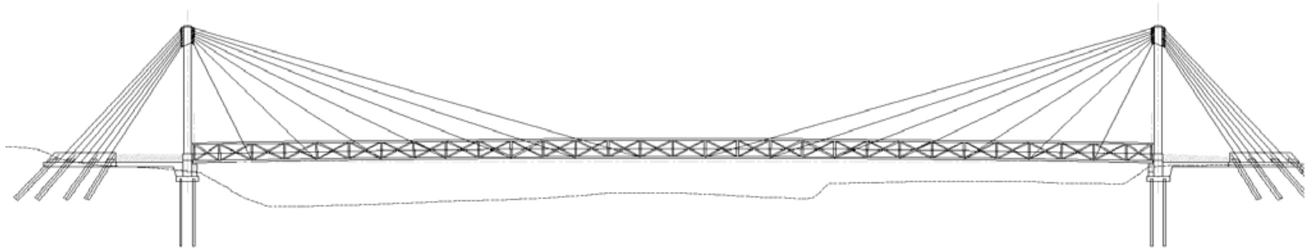

Figure 2 Bridge elevation

Preliminary design of the truss was for chord members comprised of back-to-back angles with X patterned angle cross braces. The sections required to get the specified minimum stiffness for pedestrian vibrations in the structure were prohibitive for angle sections, so the fabricator proposed a bent-plate channel section for the top chord and a conventional tube steel section for the bottom chord. The owner approved both sections and they were adopted into the design.

One of the advantages of steel is that it offers a high strength-to-weight ratio compared to timber and concrete. This is an advantage for reducing the size of supporting members, such as the cables and the superstructure, but can create new issues when looking at structural dynamics. Pedestrian bridges are particularly sensitive to dynamic oscillations, due to a combination of the harmonic nature of walking pedestrian loads, and the lack of any shock absorbing system between pedestrians and the bridge. Additionally pedestrian loads tend to be lighter than truck loads, leading to lighter structures which can experience higher accelerations under dynamic loadings. Lighter structures also mean that the harmonic frequencies of the bridge may be closer to the frequency of footfall of a pedestrian crowd, and the issue of resonance is frequently as large of a factor as strength when designing a steel pedestrian bridge. The Spring Creek Pedestrian Bridge was carefully analyzed for dynamic effects on the structure and fell outside the scope of the simple first order analysis that AASHTO offers for dynamic analysis. This triggered an in depth analysis outlined by SÉTRA which required consideration of:

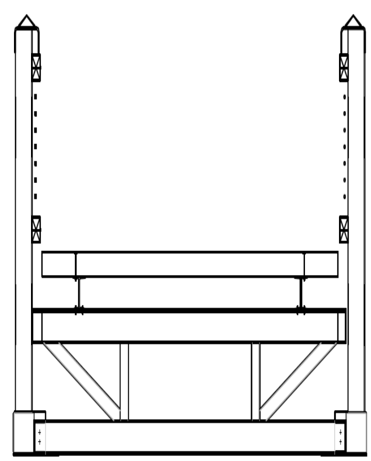

Figure 3 Bridge cross-section

- The frequency and magnitude of pedestrian footfall loadings
- The phasing of loading from multiple pedestrians on the bridge at the same time, including the "lock-in" phenomena
- Appropriate estimation of the structural damping and
- Frequency-dependent limit on acceleration and/or velocity

The detailed analysis outlined in SÉTRA involves estimating the probable crowd size to be using the bridge to characterize the frequency and magnitude of the footfall using probability distributions. This provides a harmonic forcing function for the bridge, and then displacements, velocities, and accelerations of the structure are found using classic structural dynamics theory. These values are then compared to empirically established values that quantify minimum comfort levels for each parameter. This in-depth analysis showed that anticipated accelerations of the Spring Creek Pedestrian Bridge were within reasonable values to ensure user comfort.

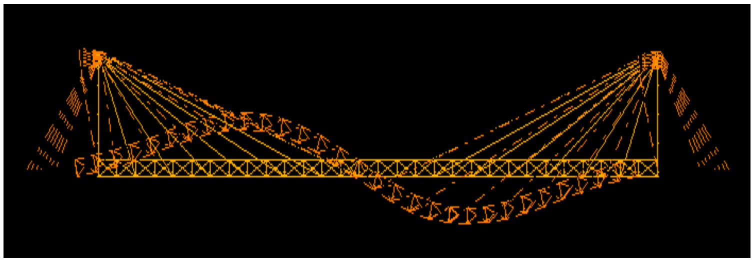

Figure 4 Dynamic analysis mode shape

The cross bracing members on the side of the truss were X-patterned angle sections as originally proposed. Cross bracing members for lateral loads were installed below the deck and were L6x4 angle sections. Using angle sections for the cross bracing members helped to propagate the rustic feel of the construction.

2.4 Towers

The towers were designed as 3 ft by 3 ft built up sections with X patterned lattice stitching between them. The four cross beams were bolted into place prior to standing up the tower. All other connections, including the lattice members, were fully welded. The design for the lattice work was performed per AASHTO, results were compared to AISC using recommendations from other established resources. Although the lattice members tend to be lightly loaded, there is a maximum slenderness ratio as a function of the overall slenderness of the built-up section permitted by AASHTO to prevent the lattice members from buckling before the overall composite section of the tower buckles; AISC has similar limitations on the slenderness. In the end, without sacrificing economy of construction, the design slenderness ratio of the lattice members exceeded the permitted value and additional intermediate diaphragms had to be added to the columns to permit the column to be analyzed without the benefit of the lattice members. This created some redundancy for the members in the tower, since although the lattice members were ignored for analysis, they will still provide some stiffening effect to the angle legs of the tower.

Cross beams in the tower were to provide in-plane stiffening of the towers, primarily to provide stability perpendicular to the plane of the cables. Analytically these loads were relatively low, but a nominal over-sizing of these beams emphasized the lattice pattern and complemented the period styling of the rest of the structure well. A simple pattern in the diaphragms helped echo the lattice diamond pattern and provide some simple ornamentation.

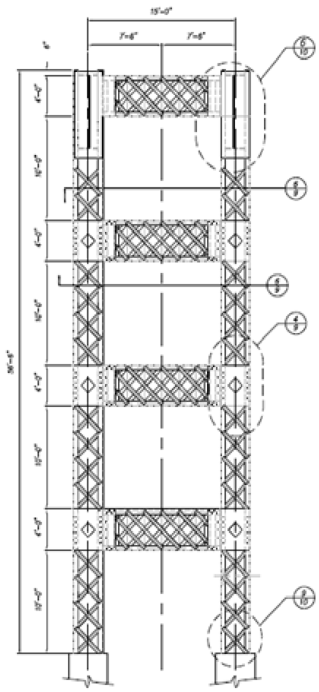

Figure 5 Tower elevation

2.5 Cables

Cables for the stays were sized at 1½ inch galvanized structural strand Class A Coating Throughout (ASTM A586). Due to a combination of the effect of sag in the stay cables and the axial modulus of elasticity (E=24,000 ksi for ASTM A586 strand), the cables have an effective nonlinear modulus of elasticity. Since the cables for the Spring Creek Bridge were not more than 200 ft nor loaded less than around 10% of their breaking strength, the reduction was on the order of 5% of the axial modulus of elasticity. In addition to the nonlinear stiffness of the cables, a detailed analysis of to mitigate wind induced vibration was performed. Per

FHWA-HRT-05-083 Wind Induced Vibration of Stay Cables, the primary modes of wind induced vibration are:

- Vortex Excitation
- Rain/Wind Galloping
- Wake Galloping of Group and
- Galloping of Dry Inclined Cables

Each of these mechanisms was analyzed, and in order to mitigate the cable vibrations, secondary cross-tie cables were added to shorten un-braced cable lengths and to increase effective dampening of the cables.

3.0 Construction

3.1 Fabrication

Although the goal of the design of the Spring Creek Bridge was to reflect the style of construction during the late 1800's there were practical limits to the lengths to which the style could be imitated. The truss and towers for the structure were all welded in lieu of the historically accurate alternate of riveting. This led to a large amount of labor involved with the fabrication of the structural steel. Additionally, all of the steel was to be ASTM A588 or A847 weathering steel. This special order on steel, combined with the longer fabrication time for the lattice detailing meant that by the time the design team was finalizing the 30% design, the sizes needed to be finalized for the steel so that the steel order could be placed with enough lead time to get the steel to the site on time. A 3D representation of the bridge was built to expedite detailing and to help avoid any dimensioning conflicts with the structure. This method of modeling the structure helped with the construction sequencing analysis and was a good cross-check for the designers to make sure that the clearances and fit-ups were as anticipated and that the finished product was able to be constructed as planned. Inspections were performed by a third party and by the designers, owners, and general contractor at the shop during fabrication.

3.2 Erection

To expedite the erection of the structure, the majority of connections were shop welded. All field connections were bolted. Truss segments came out in 25'-8" long sections with the splice points within 3 feet of where the stay cables connected to the lower truss. Cross braces at splice points were bolted to each segment once it was in place. Top and bottom chords were bolted; the nuts for the lower chord tube steel were tack welded to a splice plate placed inside the tube for the connection; additional splice plates were added as required outside the tubes for connection capacity. The construction followed classic cantilevered construction for cable stay bridges. The center segment required careful attention to geometrics so that minimal amount of adjusting would need to be done to bolt both halves together. The connection holes for the precast concrete deck panels were drilled in the field at the request of the general contractor, to ensure proper fit up.

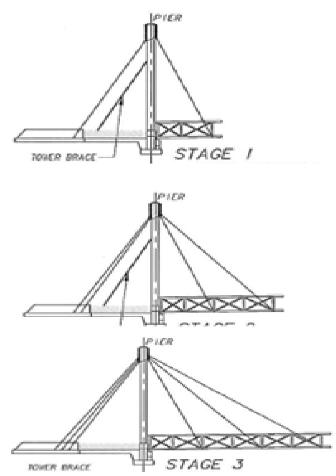

Figure 6 Partial erection sequence

Structural analysis was performed at each stage of the erection and the results were used to determine the proper sequence for erection. The intent was to erect the truss in a cantilever sequence with minimum dead load by erecting without the precast concrete deck in place. The tensions of each of the cables and the elevations of each of the cable connection nodes on the truss at each stage of construction were estimated and monitored throughout construction to make sure that the structure did not experience any undue stress. Due to the light loads in the structure without the deck and the live load, thermal effects on the bridge during this stage were observed to be significant. Elevation differences of an inch or more were observed in the truss between the end of work in the afternoon and first thing on the following cold morning.

Pancake jacks had been placed between the abutment and the end of each half of the truss during the initial erection process. The contractor requested that a gap be left between the two halves and the center splice section to permit some clearance while setting the section and to allow for the splice plates inside the bottom HSS chord to be slid into place once the center segment was in place. The pancake jacks allowed this gap to be closed up for final bolting of the center splice section. After the center splice of the two cantilevered halves of the truss was completed, the cable tensions were checked before placing the precast concrete deck. With the deck in place, a final adjustment to the tensions was performed prior to installing the hand railing cables and opening the bridge to pedestrian traffic.

Figure 7 Setting truss sections

3.3 Inspections

A third-party inspector was retained for both shop inspections and field inspections; shop inspections for steel included welding procedure, ultrasonic testing of specified welds, and general compliance with standard fabrication procedure. Visual field inspections were performed to ensure erection conformed to construction documents. Furthermore, torque tests were performed on an appropriate sample population of field-bolted connections to ensure proper snugness.

Figure 8 Testing tower bolt torques

Additionally, inspectors from the geotechnical design office were present during the installation of some of the piles to ensure that anticipated depths were adequate for the conditions encountered in the field.

4.0 Lessons Learned And Conclusions

A primary lesson learned on this project is that steel is light and limber. The strength-to-weight ratio for steel leads to a light structure, but this lightness can decrease the fundamental frequencies of the structure, and special attention must be paid to the dynamic analysis of the vibrations in the structure to ensure an appropriate level of serviceability.

The design-build process was effective in delivering an economical product within a tight time schedule. The design engineer worked closely with the general contractor and the fabricator to determine the most efficient design and to adjust the details for the bridge to facilitate ease of fabrication and erection. Additionally choices such as the use of steel piling allowed the team flexibility to accommodate conditions encountered in the field that varied from the anticipated conditions.

The lattice pattern on the tower was used to imply antiquated construction styles on a modern structure. This yielded a pleasing structure but did increase both material and labor costs as well as increasing erection time.

The goal of this project was to construct a new structure that echoed the past. The use of weathering steel and vintage detailing practices was able to deliver a product that utilizes modern materials to create a new historical land mark.

Figure 9 Completed bridge

Figure 10 Completed bridge

Figure 11 Completed bridge