ST. PATRICK'S PEDESTRIAN BRIDGE DESIGN AND CONSTRUCTION

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SUMMARY

The St. Patrick's Bridge, recently re-named the George C. King Bridge, is a 182 metre, three-span, continuous network tied-arch structure that spans over the Bow River at St. Patrick's Island on the east side of downtown Calgary. The inherently efficient structural scheme, with generous span to depth ratios, was optimized parametrically with respect to member geometry and sizes. Construction of the footbridge began in April 2012, but was delayed by nearly a year due to extreme flooding of the Bow River in June of 2013 that inundated the island, and damaged and washed out critical temporary falsework supporting the post-tensioned concrete bridge deck and steel arch sections. Replacement of the entire deck was required, altering the erection sequencing and forcing winter construction methods not initially envisioned.

Keywords: *Network arch, post-tensioned concrete, extreme event, flooding, steel fabrication, steel erection.*

1. INTRODUCTION

The planning and design of the iconic St. Patrick's (George C. King) Bridge began in 2010 after the Calgary Municipal Land Corporation (CMLC) held an open international design competition that attracted 33 entries. CMLC selected the design submitted by the team led by RFR, in collaboration with Halsall Associates (now WSP) and Parsons Brinckerhoff (now WSP | Parsons Brinckerhoff).

The bridge provides access between the East Village neighbourhood of downtown Calgary, park space on St. Patrick's Island, and bike paths on the north bank of the river. At the time of the design competition, major redevelopment of East Village was just beginning, and plans were being made to redevelop the park on St. Patrick's Island. The goal was for the bridge to serve as a visual attraction to draw people to the area, linking the redevelopment projects with the existing pedestrian pathways, and thereby contribute to the success of the area's revitalization. The slender arches and slim deck, having minimal contact with the river banks, fulfils these goals by responding to the natural qualities and openness of the site while also acting as a landmark, inspiring area residents to refer to it as the skipping stone bridge.

Fig. 1. *Completed bridge.*

The arches are arranged in spans of 52 m, 31m, and 99 m for the north, centre and south arches, respectively, running continuously between sliding bearings on the two abutments, with each arch rib composed of twin steel pipes joined by welded top and bottom plates forming oblong cross-sections. The long arches over the north and south river channels are bowstring arches with the post-tensioned concrete deck acting as the tension tie.

The deck is suspended from the arches over the north and south channels by diagonal stainless steel cables creating network arches. This hanger arrangement allows the arch and deck to act together similar to a truss beam under asymmetric loading.

The complex geometry of the steel arches required strict fabrication and erection tolerances to achieve the sweeping parabolic shapes with continuous horizontal top and bottom plates throughout. With a continuously variable deck width, the bridge provides ample space for both bicyclists and pedestrians, and includes bench seating areas overlooking the river. A ramp down from the deck in the centre span provides access to the newly renovated island park area.

Construction of the footbridge began in April 2012 and was scheduled for completion in late 2013. Extreme flooding of the Bow River in June of 2013 inundated the island, damaging and washing out critical temporary falsework supporting the already post-tensioned concrete bridge deck and steel arch sections, thereby delaying completion of the project by nearly a year. Removal and replacement of the entire deck was required, altering the erection sequencing and forcing winter construction methods not initially envisioned.

2. DESIGN DETAILS OF THE ARCH RIBS, DECK AND FOUNDATIONS

2.1. Steel arch details

The arch rib cross-sections are 400 mm tall, but vary in width from 1130 to 1550 mm wide, giving the arches significant out-of-plane strength to resist lateral loads from wind, water, ice and earthquakes, which eliminated the need for bracing between the arch ribs, save for one bracing point at the apex of the main span ribs.

The arches rise over the deck on the two river spans at each end, while remaining below the deck on the centre island span. The two river spans include inclined intersecting hanger cables linking the arch and deck to form the network arch. This enables the arch and deck to work together to resist concentrated loads and thus each can be quite slender. Vertical hold-downs at each abutment prevent the ends of the deck from lifting as the arches are loaded.

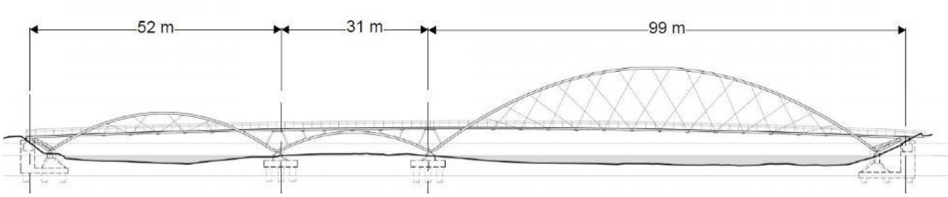

Fig. 2. *Elevation.*

The designers undertook a series of parametric studies of the arch geometry to optimize the efficiency and economy of the slender structure using advanced software interfaces. Rhino was used for the geometric modelling of the bridge structure. Grasshopper was used as a plug-in to allow parametric modelling while updating the Rhino model in real time. Numerous geometries and layouts were studied including the arch rise to span ratios of each arch, arch inclination, and hangar cable layouts.

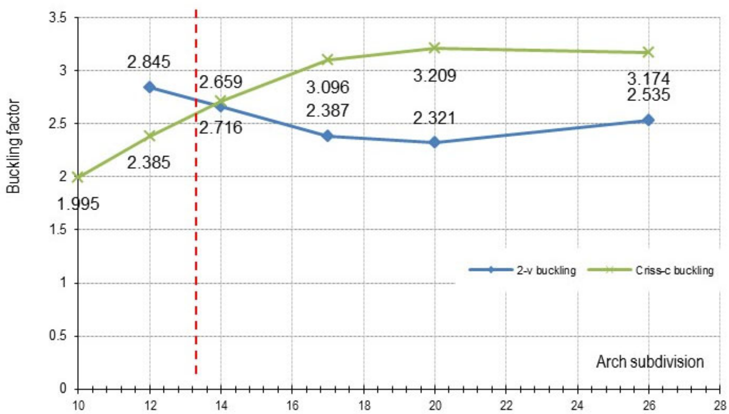

Fig. 3*. Optimization evaluation for number of south span hangers vs. arch buckling.*

Oasys GSA software was used for structural modelling. Geometry Gym's Smart Structural Interpreter (SSI) as used to important the parametric geometric model from Rhino by the touch of a button. This allowed for a quasi-instantaneous evaluation and optimization of the geometric parameters with respect to structural design.

After setting the final bridge geometry and layout resulting from the parametric studies, the final design was carried out according to the Canadian Highway Bridge Design Code CAN/CSA –S6-06 [1] with dynamic studies being carried out according to the SETRA 2006 Assessment of Vibration al behaviour of Footbridges under Pedestrian Loading [2].

More than 300 tonnes of structural steel for the bridge was fabricated by ADF Group in their shop in Terrebonne, Quebec, from Canadian Standards Association (CSA) G40.21 350 WT steel, a weldable notch-tough steel with a yield stress of 350 MPa and a tensile strength ranging between 450 MPa and 620 MPa. The oblong arch section was fabricated from two steel tubes welded together via transverse stiffeners and a heavy top and bottom cover plate, as shown in Fig. 4 and Fig. 5.

Fig. 4. *Arch rib fabrication.*

Fig. 5. *Centre arch knuckle fabrication.*

The tolerance of the steelwork is critical for the stability of the arch structure. In addition, changes in the axis of the arch have a critical impact on the correct length determination for the hanger cables. A fit-up was therefore required to provide geometric control of the arch structure.

Given the size of the arches, it was not possible to perform a vertical fit-up replicating exactly the support conditions on site. The arch was therefore fit-up in a horizontal position with frequent supports to determine the unloaded or "gravity-free" in-plane arch geometry. Particular attention was paid to the support conditions to ensure that the arches were free to move in plane without restraint. This stress free geometry was then corrected for gravity effects to set erection targets on site. The fit-up support conditions were closely scrutinized to ensure that no locked-in deformations would distort the survey results of the fit-up.

In addition to the spatial complexity of the structural steel, the proximity of the arch structure to the bridge user required an elevated level of architectural finishing for the structural steel. The requirements for surface quality varied from AESS 2 to AESS 4, as defined by the Canadian Institute of Steel Construction's Code of Standard Practice [3]. Cost savings were achieved by varying the finish requirements based on the degree of users' proximity to the steel users. In order to set appropriate expectations and limits on surface quality, a mock-up was required to demonstrate the weld detailing and finishing, as well as finish quality of the corrosion protection.

Prior to fabrication of the deck and erection of the steel arches, large temporary beams were lifted over both the north and south channels of the river to support the construction works on temporary berms imported into the river. These temporary beams were intended to support both the deck formwork over the rivers and the scaffolding supports for erecting the main arch. Driven H-piles and steel frames were welded together as temporary abutments to support the large beams over the river channels.

Fig. 6. *Temporary berms.*

Erection of the steel arches began with the centre arch sections on the island, with a single splice near the apex. Two days were required to lift the four pieces. Installation and complete splice welding was required before placement of the deck formwork could begin above centre span arches. The stainless steel rocker bearings were attached to the base of each arch section prior to placement in order to bear on the piers.

The north arch sections, again with a single splice near the apex, were erected second, after the deck below was cast and post-tensioned. In addition to the mid-span splice, another splice was required at the island piers to connect the arches to the centre arches.

The south arch sections were erected last. Being the longest span, these arches had two splice points plus the splice to the centre arches.

Fig. 7. *Erection of centre arch segment.*

Fig. 8. *Completed centre arches.*

2.2. Post-tensioned concrete deck details

Typical formwork methods of plywood and timber were used on each facet of the project, and no precast construction was used. The deck surfaces are curved in plan and sloped transversely. The side edges of the deck are sloped at 30 degrees, with a typical 2% cross slope on top of the deck and level monolithic concrete T-Beams at the embedded hanger connections. Special support details were required for the formwork for the drainage trough located on each side of the deck, running parallel to the deck edge.

Fig. 9. *North arches erected.*

Fig. 10. *Deck reinforcing bars and PT ducts.*

Fig. 11. *Deepened deck section with embed plates.*

The deck reinforcement consists entirely of galvanized bars, varying from 10M to 30M bars. Outside of the hold-down areas at either abutment, the reinforcement layout is relatively typical, with special configuration details at the arch to deck connection locations.

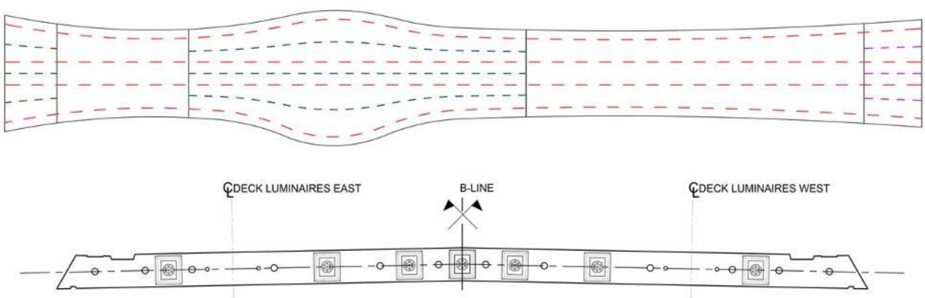

Fig. 12. *Deck plan and section.*

The deck is constructed from 45 MPa high performance concrete, with 15mm diameter low-relaxation post-tensioning (PT) strands. The deck pours are arranged around the multiple post-tensioning sections. There are three individual PT sections, one near each abutment and one over the centre arch, resulting in two infill pours between these three sections. The end section pours are approximately 35 cubic metres in volume, with the centre section pour being approximately 200 cubic metres.

Three specific pour sections, consisting of the centre and end pours, each required individual post-tensioning. Following the stressing of the PT strands in those individual sections and the placement of the infill pours, additional strands were stretched full length of the deck.

The creep and shrinkage of the deck not only impacts post-tensioning loses in the deck but also impacts stresses in the steel arch structure. Longitudinal lengthening of the deck due to creep will have a softening effect on the bowstring tension tie leading to bending in the steel arch and was therefore limited. The original tendered design required a precast steam cured bridge deck system, with a 60-day delay between concrete placement and locking of the arch to deck connection. The value engineering move to a cast-in-place deck required a delay of 120 days between placing of the deck and connection of the deck to the arch.

However, since the tested concrete strength and stiffness values typically exceeded the requirements of the specifications, a reduction of the delay was applied to account for the additional stiffness. In addition to calculation models predicting expected creep and shrinkage, the actual creep and shrinkage of the deck was measured to verify the accuracy of the model predictions.

2.3. Concrete foundation details

The abutments are supported on 12 drilled concrete piles, with a 1.25m thick pile cap supporting the arch bearing plinths and tie-down mass structure to resist uplift forces at each abutment. The north and south abutment pile layouts and pile cap configurations have identical dimensions, although the depths of pile tips vary based on geotechnical data.

Because the pile caps are located below grade, the abutment bearing plinths are extended to the surface via truncated pyramid-shaped extensions. Heights of the extensions vary for each arch bearing location due to the design termination elevations. Each abutment is reinforced with uncoated steel bars and constructed with 35 MPa design strength concrete.

Fig. 13. *Abutment pier foundation.*

Fig. 14. *Intermediate pier foundation.*

Four intermediate piers support the north and south channel arches on St. Patrick's island. All of the piers have identical dimensions, with five drilled concrete piles and a pile cap.

Uncoated reinforcing bars are used in all of the foundations, with the majority of the bars having typical configurations and several areas with special bending details to account for the slopes of the abutment pyramids and the island piers.

To support the arch bearings on the intermediate piers, inclined bearing surfaces were detailed on truncate pyramid-shaped extensions atop the pile caps. The tops of the bearing surfaces are inclined in order to be normal to the support reaction. Since the north and south channel arch sections are continuously welded to the centre arch section over the island piers, the intermediate piers resist all directions of translation.

3. FLOOD DAMAGE AND RECOVERY

As of mid-June 2013, the entire concrete deck had been placed and post-tensioned. The steel arch sections for the centre and north arches were secured on their scaffolding with full penetration splice welds of the arch pipes, while only one of the south arch segments had been erected from the island side.

Scaffolding was supporting the entire concrete deck except for the deck segments supported by the temporary beams spanning over the north and south channels. The struts extending from the Centre Arches were welded in place, but not connected to the plates embedded to the underside of the deck.

On June 19, swift river currents carrying debris steadily increased, as the worst flood in Alberta history began. The flooding impacted the bridge foundations and the scaffolding supporting the deck. Initial failures of scaffolding began on the island near the south intermediate pier, resulting in deflection of the deck. Subsequent flood flows dislodged scaffolding from beneath the deck in front of the southern abutment, which also resulted in significant deflections.

Fig. 15. Upstream side during flood event.

Fig. 16. Flood debris under north end.

Fig. 17. Deck deflection after flooding.

Inspections and analysis following the flood damage concluded that the deck deflections resulting from loss of scaffolding support caused the reinforcing bars and PT strands to yield, therefore requiring replacement of the deck. Due to the full length post-tensioning ducts having been previously grouted, the two deck sections that experienced significant deflection could not be independently removed and replaced. Consequently, it was necessary to remove and replace the entire concrete deck.

Before deck removal and recovery could begin, a substantial amount of debris and damaged scaffolding needed to be removed from under the deck, and temporary support of deck sections needed to be re-installed. The clean-up required safe working practices and a large effort from the contractor to restore the site back to normal working conditions.

Removal of the deck occurred via saw cutting and lifting off of small sections with a crane. However, before the sections of deck could be removed off the temporary beams over the river, the arch support scaffolding needed to be removed, along with the single south arch section previously erected but not securely welded. In addition, the north arch top and bottom plate splice welds were completed before the removal of the scaffolding and deck sections. A rare opportunity to examine the PT duct grouting was available to inspectors, revealing that the ducts in all cut sections had complete encapsulation of the strands.

Upon full removal of the deck concrete, additional survey and observations were made regarding the centre arches. It was observed in the aftermath of the flood and debris that the deck was being supported by the centre arches, but not in a manner expected by the erection sequencing. The resulting conclusions determined that the centre arches deflected within tolerances, but the protruding steel struts had deflected plastically out of tolerance and required adjustments.

After removal of the deck, the erection sequencing was adjusted, such that all arch sections were erected first and followed by full deck replacement. Due to the delay, pouring of the deck occurred during the winter months in order to ensure the new deck was complete and supported by the arches before the following June when the potential for more flooding is the highest. This required full enclosures to hoard and heat the deck. The south arch sections were lifted in three sections, and spliced to each other and to the centre arch prior to removal of the temporary scaffolding. Two temporary support frames were left in place to support the as-built geometry until the hanger cables were installed and tensioned.

Fig. 18. *Reinforcing within hoarding.*

In all, the restoration effort of replacing the deck required adjustments to the arch erection sequencing and support details, as well as reconstruction of the deck formwork and steel embed plates. Consequently, this caused a significant schedule delay, with a full year extension to the delivery date. After replacement of the damaged deck, installation of the hanger cables, and field application of the final paint coating, the bridge was opened to the public in October 2014.

Fig. 19. *Hoarding and heating of entire bridge deck during winter construction.*

4. CONCLUSIONS

The City of Calgary now has an elegant new pedestrian bridge linking downtown with St. Patrick's Island and the north bank of the Bow River. The bridge design is an aesthetic response to the site conditions and to the aspirations for the redevelopment of both the East Village neighbourhood and St. Patrick's Island.

Fig. 20. *Completed bridge and island access ramp with lighting.*

Innovative calculation methods enabled the designers to create a slender and graceful crossing without compromising its strength. Construction of the bridge had many challenges to overcome, but the construction team combined their skill and knowledge to develop an alternate erection sequence and methods to complete the project. The devastating flood delayed the construction completion, but also demonstrated the resilience of the bridge with no damage to the permanently installed bridge components, indicating the bridge will be a lasting part of the Calgary landscape.

REFERENCES

[1] CAN/CSA-S6-06, *Canadian Highway Bridge Design Code*, Canadian Standards Association, Mississauga, Ontario, Canada, 2006.

[2] SETRA, *Assessment of Vibrational Behaviour of Footbridges under Pedestrian Loading*, Service d'études techniques des routes et autoroutes, France, 2006

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