AN INNOVATIVE TIED ARCH SOLUTION TO SPANNING THE MISSISSIPPI

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BIOGRAPHY

Martin Furrer is a Project Manager with Parsons in the Chicago, IL office and was the Lead Bridge Design Engineer for the Hastings Bridge project. He has over 16 years of experience in the project management, design, analysis, load rating and construction of complex bridges including 14 bridges over major waterways. Mr. Furrer was the kcICON River Bridge Segment Manager and Engineer of Record for the 1000 ft long Christopher S. Bond Cable-Stayed Bridge over the Missouri River in Kansas City. Mr. Furrer received his Master's in Structural Engineering from the Swiss Federal Institute of Technology, Zurich and is a registered professional and structural engineer.

Greg Hasbrouck is a Senior Bridge Engineer with Parsons in the Chicago, IL office and was the River Bridge Design Lead for the main span tied arch segment of the Hastings Bridge project. He has over 8 years of experience working on complex bridge projects including the design of the new Christopher S. Bond Cable-Stayed Bridge over the Missouri River in Kansas City. Mr. Hasbrouck received his Master's in Structural Engineering from Princeton University and his Bachelor's in Civil Engineering from Duke University and is a registered professional engineer.

SUMMARY

The new Mississippi River crossing at Hastings, Minnesota is a design-build project currently under construction, with a 545 ft main span tied arch to create a community icon for this scenic recreation area. Unique features of the project include free-standing steel box arch ribs, post-tensioned concrete tie girders, integral piers, rigorous redundancy requirements and an innovative erection scheme.

With a unique pair of free-standing steel box arch ribs with no upper wind bracing and post-tensioned concrete tie girders, the Hasting Bridge will be a record span in North America for this type of structure. The free-standing arch feature offers not only an attractive bridge but also a cost effective solution. Arch stability and wind resistance are important design and construction considerations for this type of structure.

In addition to the unique structure type, the project is mandated to meet rigorous structural redundancy requirements and a 100-year design life. The redundancy requirement specified by the owner goes beyond the typical considerations of arch and hanger redundancy, with requirements for loss of floor beams and other main structure members, with strict limits in both service and strength conditions. To meet this challenge, a unique steel floor system with both longitudinal and transverse main girders is developed to provide sufficient alternate load paths and validated with detailed response time history computer analysis.

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Introduction

A unique and innovative free-standing tied arch bridge is currently under construction across the Mississippi River in Hastings, Minnesota, for the Minnesota Department of Transportation (Mn/DOT). The new bridge will replace the existing two lane structurally deficient continuous steel arch truss built in 1950 with a 545 foot main span tied arch bridge utilizing free-standing steel box ribs, post-tensioned concrete tie girders and a network hanger configuration. Through the design-build best value selection process the team of Lunda/Ames/Parsons was able to present an alternative design that met the rigorous performance criteria with an innovative cost-effective solution that creates a community icon for this scenic recreation area.

The unique free-standing steel arch ribs with no upper lateral wind bracing sets a record span for this type of bridge structure in North America and is designed to meet stringent redundancy requirements significantly exceeding typical considerations for tied arch redundancy. The final design addresses the loss or failure of any main structural tension member including floor beams and hangers as well as loss of post-tensioning in the tie girder with restrictions for both service and strength limit states. This required a unique floor system utilizing both longitudinal and transverse main girders to develop alternate load paths and the use of additional post-tensioning in the tie girder to compensate for potential loss.

Arch stability and wind resistance were important design and construction considerations that required extensive engineering and the development of structure specific criteria that are not addressed in AASHTO. Erecting the bridge with restrictions on blocking the navigation channel and the unique structure type made the development of the erection scheme in concert with the design and the contractor's means and methods of paramount importance. Additionally, to meet the 100-year design life, stainless deck reinforcement and other innovative details and material usage such as high performance concrete are employed.

Figure 1 – Free-Standing Arch Bridge Rendering

Structure Type Selection

The project criteria issued with the Mn/DOT Request for Proposal (RFP) contained specific design and performance requirements for aesthetics, redundancy, durability and maintenance of the bridge. Key to these requirements was the use of AASHTO LRFD, special strength and service load cases, load path redundancy, bridge security, and a 100-year service life. The RFP identified two acceptable structure types for the river bridge span; a single diamond tower cable-stayed structure and a basket handle network tied arch structure. In addition, the RFP contained very prescriptive design/performance criteria and aesthetic requirements for each structure type.

Parsons served as the design engineer for the Lunda/Ames design-build team which combines the expertise of two major contractors located in the Minnesota region; Lunda Construction Company of Black River Falls Wisconsin and Ames Construction of Burnsville, Minnesota. The design-build team combined their unique skill sets and expertise to evaluate and develop the winning innovative free-standing arch bridge shown in Figure 1 through the Alternative Technical Concept (ATC) process.

The Lunda/Ames/Parsons design-build team thoroughly evaluated the base designs via preliminary engineering and cost estimating exercises and developed and evaluated alternative designs through the ATC process based on the RFP performance criteria and goals for overall evaluation. The RFP requirements, best value scoring, and construction costs/risks ultimately drove the final selection of bridge components.

The tied arch structural system is comprised of two free-standing vertical steel arch ribs with cast-in-place post-tensioned concrete tie girders. The cast-in-place concrete knuckles connect the steel arch rib and concrete tie at the piers. The floor system consists of a grid of steel floor beams, full depth longitudinal stringers and secondary longitudinal stringers composite with a cast-in-place concrete deck. The knuckles and deck are integral with the piers creating a fully framed system. A network of structural strand hangers is used to suspend the floor system from the arch ribs.

The steel free-standing arch ribs shown in Figure 2 provide an easily maintained, aesthetically pleasing element that simplified fabrication cost since each arch can be assembled independently. Mn/DOT specified additional criteria to the design teams alternative including; a minimum Factor of Safety against arch bucking of 2.0 at the Strength Limit state, limiting the lateral deflection to R/300 (where R is the rise of the arch rib at the service limit state under the Service I load combination), and a structure specific wind study.

Figure 2 – Free-Standing Arches

The post-tensioned concrete tie girder and framed knuckle was selected in order to meet the rigorous redundancy and load requirements of the tie system. This alternative also minimizes the inspection/maintenance of the tie system, eliminates any shop fit-up assembly issues with the rib, and provides complete active-tension control of the system during erection. Combined with the greater durability of the post-tensioned concrete tie over a steel tie and the stringent redundancy requirements for the tie element in the RFP, this approach provided a best value win-win solution. The framed knuckle and integral piers shown in Figure 3 eliminate the need for large pot bearings and their associated maintenance and provide for lateral stability of the unbraced arch ribs.

Figure 3 – Framed Knuckle and Integral Pier

A grillage system with full depth longitudinal stringers framing into the floor beams was conceived to meet the floor system redundancy requirements. All floor beams and full depth stringers had to meet stringent service and strength criteria under fracture cases. Additionally, the criteria required that the deck must be able to be fully replaceable one-half at a time. The typical bridge section is shown in Figure 4.

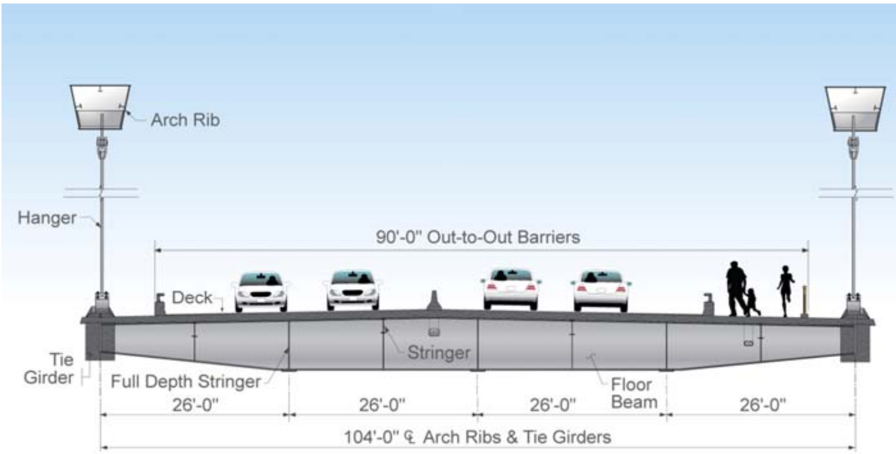


Figure 4 – Typical Section

Structural Redundancy

All structural tension members in the tied arch structure are load path redundant for fracture at any point in a single member or connection subject to tension; consequently there are no fracture critical bridge elements on the structure. The project criteria defined structural tension members as any member with tension under permanent loads (dead loads, locked in forces, creep and shrinkage) and vehicular live load. Consideration of multiple fractures occurring simultaneously was not required.

Project specific redundancy load cases shown below were specified for analysis at the time of fracture considering a fracture dynamic force (FDF) and after a fracture has occurred for both strength and service limit states. Locked-in forces are included in the load cases with a factor of 1.0 with LL+IM consisting of HL-93 loading.

Strength Load Case at time of fracture:	1.25DC + 1.35(LL+IM) + 1.1FDF
Strength Load Case after fracture has occurred:	1.25DC + 1.35(LL+IM)
Service Load Case at time of fracture:	1.0DC + 1.0(LL+IM) +1.0FDF
Service Load Case after fracture has occurred:	1.0DC + 1.3(LL+IM)

The capacity-to-demand ratios for all members under the Fracture Load Case strength limit state are designed to be greater than or equal to 1.0 and the maximum nominal flexural resistance for steel beam design is limited to Fy. For Fracture Load case service limit state, concrete compressive strain is limited to 0.003 inch/inch and reinforcing steel and post-tensioning steel stresses are limited to Fy. Structural redundancy of the tied arch bridge components and its affect on the structure are described in the sections below.

Floor System

A unique floor system utilizing both longitudinal and transverse main girders to develop alternate load paths was developed for this structure to meet the stringent redundancy requirements. In the floor system, the transverse floor beams are the primary load carrying members spanning between the hanger connections and are typically spaced at 27'-6". Three full depth longitudinal stringers span between the floor beams with full moment connection over the floor beams. Secondary longitudinal stringers are located midway between the full depth stringers and are simply supported between floor beams to transfer the deck load. The deck and stringer system was developed to mimic a traditional multi-girder bridge with a concrete deck, which is typically considered load path redundant. The transverse floor beams, full depth longitudinal stringers, secondary longitudinal stringers and tie girder are composite with the reinforced concrete deck which spans 13'-0" transversely as a one-way slab over the tie girders and stringers. Figure 5 shows the floor system framing plan for half of the main structure segment.

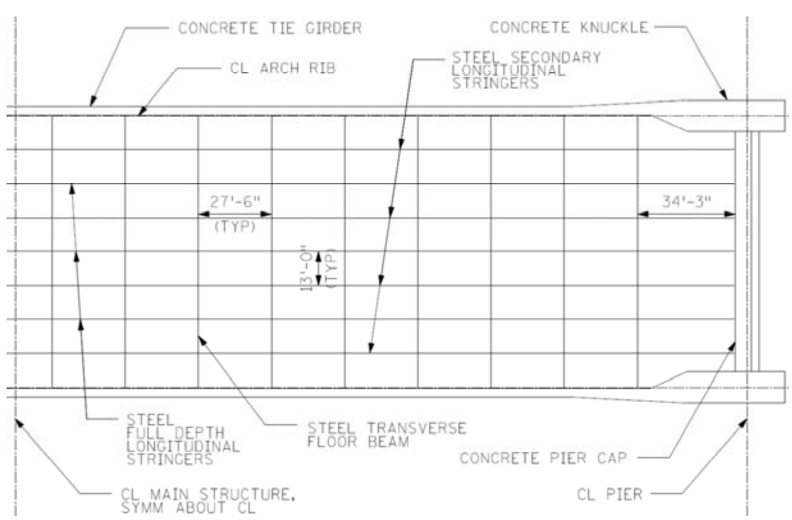


Figure 5 – Floor System Framing Plan

Floor System Redundancy

The transverse floor beams and full depth longitudinal stringers form a grid floor system, which allows load transferring in both the longitudinal and the transverse direction. The grid forms a redundant system with the primary load path through the transverse floor beams. If a transverse floor beam fractures, the full depth longitudinal stringers support the fractured floor beam and transfer load to the adjacent floor beams. The full depth longitudinal stringers provide multiple supports, which minimize deflections from the potential fracture of a floor beam and significantly reduce the resulting fracture energy release and dynamic impact.

The redundant load paths are designed to meet strength and serviceability criteria. Under an element loss the structure load path and member forces change. The new member forces include the steady-state force (the static force which includes the dead load and live load) and the transient force which only occurs at the time of fracture (fracture dynamic force, FDF). The fracture member forces are peak forces that exist directly following member fracture and can be expressed as:

$$F(\text{at fracture}) = F(\text{static, after}) + FDF$$

If the static member force change, or the difference of static member force between before and after fracture, is labeled as ΔF, the fracture dynamic force (FDF) can be expressed as the product of a dynamic load factor (DLF) and the member force change (ΔF), or:

$$\Delta F = F(\text{static, after}) - F(\text{static, before})$$

$$FDF = DLF * \Delta F$$

The moment in the floor beams before and after fracture of a floor beam in the middle of the span is shown in Figures 6 and 7.

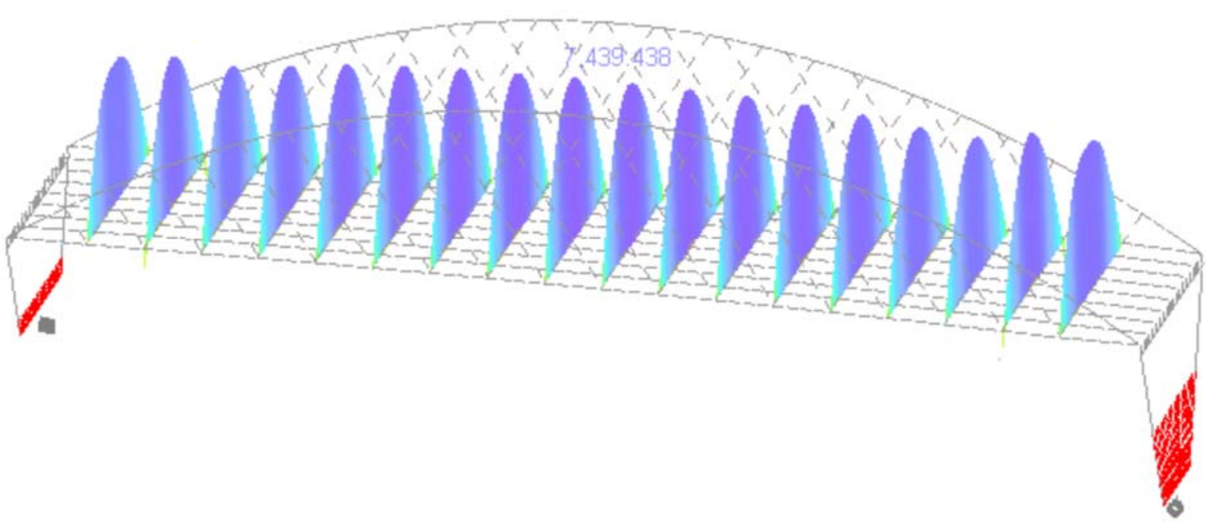


Figure 6 – Floor Beams before Fracture

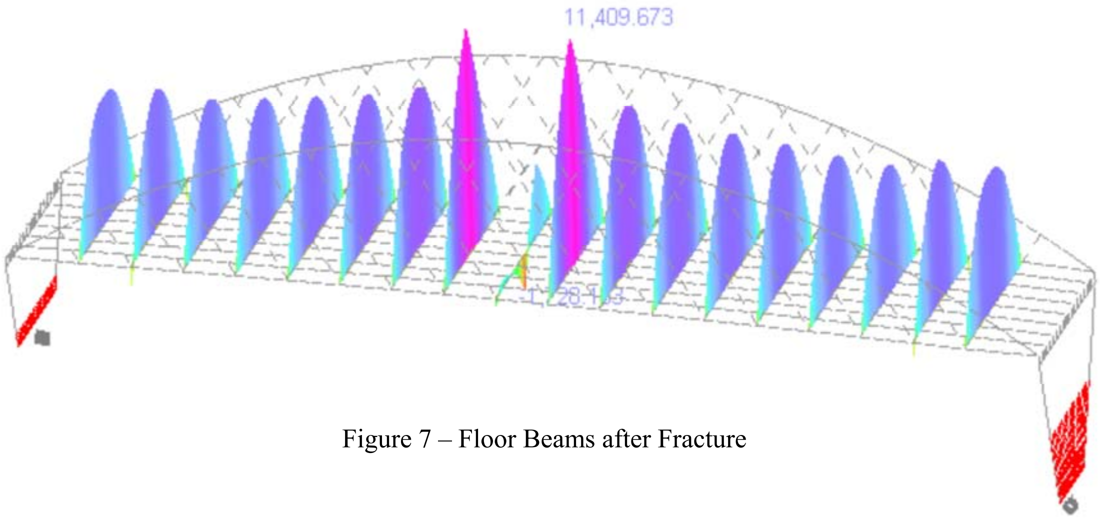


Figure 7 – Floor Beams after Fracture

The dynamic effect was studied by performing a time history analysis to determine the dynamic load factor with the theoretical maximum value of DLF equal to 1.0. This analysis included multimodal structural interaction with appropriate distributed mass and damping distribution. Rayleigh damping was adopted and assumed to be a linear combination of the elastic stiffness and mass. The combination coefficients for the stiffness and mass were computed based on an average damping ratio of 2% over the first 20 modes based of the expected response of the floor system as recommended in AASHTO LRFD Section 4.7.1.4. Model results were not sensitive to the damping ratio with a change in damping ratio from 2% to 1% resulting in an increase in DLF of approximately 3%.

As shown in the Figure 8, the static moment under dead load and superimposed dead load is 7400 k-ft before the fracture of the adjacent floor beam. Immediately after the fracture occurs, the maximum moment of 13950 k-ft is achieved due to dynamic effect, and the steady-state response, or static moment is 11400k-ft, thus:

Static Force Change: $\Delta F = 11400 - 7400 = 4000$ k-ft;

Fracture Dynamic Force: $FDF = 13950 - 11400 = 2550$ k-ft;

Dynamic Load Factor: $DLF = FDF / \Delta F = 2550 / 4000 = 0.64$.

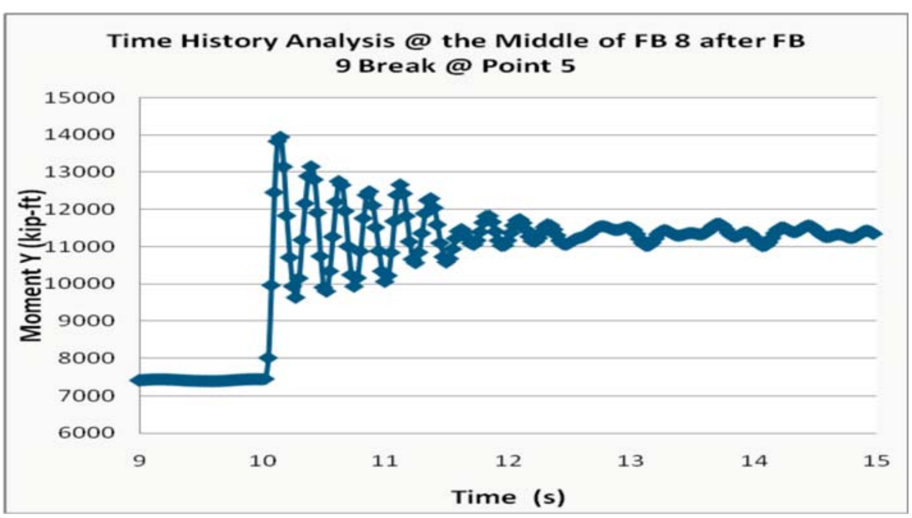


Figure 8 – Time History Analysis

A dynamic load factor of 0.65 was used for design of the floor system elements under a floor beam or full depth longitudinal stringer fracture. In the analysis the entire steel section was assumed to fracture and the deck was assumed to fracture along with the steel section in cases where it was not to the advantage of the structure, thereby assuming no continuity across the fractured joint. In several floor system fracture cases, having continuity across the joint for shear created a larger effect on torsion in the tie girder. For these cases the joint was assumed to have a pin connection at fracture with the load transferred through shear friction in the deck.

Free-Standing Arch Ribs

The free-standing arch is a very unique structure type, particularly with the relatively long span. Chief among the concerns of an unbraced arch of this length are arch rib deflection, performance under wind loading and out-of-plane buckling. The owner's concern with the innovative structure is represented in the additional criteria specified for the structure. Further complicating the design is the lack of AASHTO code applicability to this structure type and the effects on the arch rib of extra post-tensioning in the tie girder required to meet tie girder redundancy requirements.

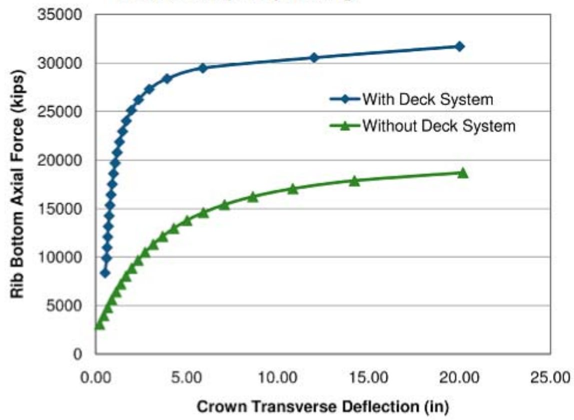
Rib Slenderness and Buckling

The design team utilized our experience on similar arch projects such as the Damen Avenue arch in Chicago, IL and the Martin Luther King Jr. Parkway arch over the Raccoon River in Des Moines, IA to generate a clear methodology for rib buckling and slenderness evaluation of the arch rib system. Lateral stability of the free-standing arch rib is provided by the heavier section modulus of the rib box section to resist out-of-plane forces. The 1:3.33 web slopes provide the visually significant sloped box section and a larger transverse section modulus to resist out-of-plane forces. In addition to a strength approach to stability, the hanger and deck system provides a significant self restoring force to out-of-plane deflections and contributes to the stability of the free-standing arch rib.

A detailed three-dimensional finite element model (FEM), nonlinear stability study utilizing a step-by-step load increment approach provided an accurate calculation of arch stability with the combination of effects. The FEM model included the entire structure and captured the interaction between the arch rib and deck system as well as the self weight and wind loads. The analysis included a sensitivity study of arch behavior resulting from fabrication and erection tolerances. As shown in Figure 9, the factor of safety against buckling in the out-of-plane direction produces a factor of safety around 3.25 with the deck due to the restoring force of the hanger and deck system. Buckling was also analyzed without the deck and ignoring the temporary bracing in place during erection, producing a factor of safety of 1.90, including the temporary bracing increases the factor of safety to around 3.0. The stability without the deck is provided by the concrete cap and knuckles. The difference between the two is the hanger restoring effect of the hanger and deck system.

Case	Buckling Load (kips)	Safety Factor
With Deck	31,700	3.25
Without Deck	18,500	1.90 *

*Without temporary bracing



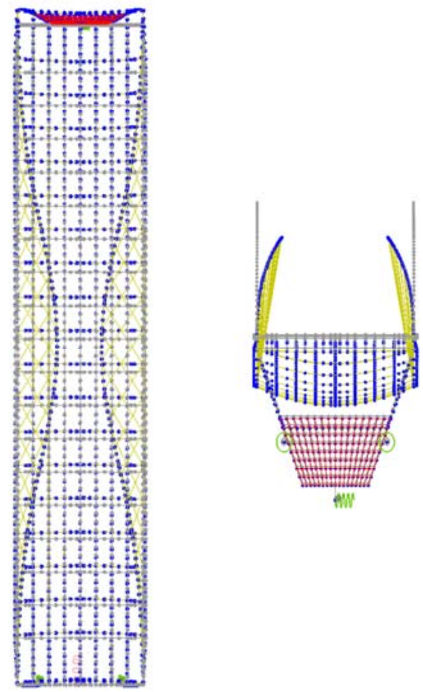


Figure 9 – Buckling Analysis Results

Figure 10 – Rib Mode Shape (Plan & Section)

The final design analysis utilized a "Refined Method" per AASHTO LRFD 4.5.3.2.3, using a P-delta or nonlinear modeling approach. The analysis first utilized a 3D FEM model to conduct a second order analysis to obtain the moment magnification factor for factored axial loads. External loads are then applied to the model to generate the first order modes, shown in Figure 10, in accordance with the AASHTO load combinations. The applied loads are then increased incrementally until the arch rib reaches the stability limit (incremental buckling analysis). Arch rib moments (Mu) were then calculated per AASHTO LRFD 6.9.2.2 based on the moment magnification factors obtained from second order analysis. Equation 6.9.2.2-1 or 6.9.2.2-2 is then used to calculate the arch compressive resistance.

Wind Load and Deflection

A structure specific wind study for aeroelastic stability was performed by Rowan Williams Davies and Irwin Inc. with no unusual findings or specific recommendations. Wind loadings on the structure were derived from the study and implemented in the design of the structure.

The lateral deflection of the rib was limited R/300 at the service limit state under Service I load combination, where R is the rise of the arch rib, equal to the span length divided by 6 or 90'-10" for the Hastings Arch. The deflection criterion is rather stringent for this structure type as it is loosely based on a much more restrictive cantilever bridge deflection under vehicular live load. The resulting deflection of the free-standing arch ribs under Service I load combination was around 3.5" which is less than the deflection criteria of 3.63". To meet the rib lateral deflection criteria, the thickness of the web plates was increased to provide a stiffer transverse section modulus to limit lateral deflection, thus governing the design of the arch rib webs.

Arch Rib Plate Slenderness

Steel plate slenderness in the context of the arch rib flanges and webs for unbraced ribs is not fully addressed in the current AASHTO provisions (1). The design team evaluated current and previous AASHTO specifications (2) (3) and historical AASHTO methodology (4) and three critical observations were made that contributed to the cross sectional design approach used on the Hasting Bridge project:

1) The underlying AASHTO assumption of stocky flanges providing a measure of rotational restraint to the attached webs is not representative of the cross-section proportions of free-standing arch ribs. Therefore, buckling coefficients from the arch rib flange formulations were used for the web to ensure a conservative design.
2) AASHTO employs a broad approach to assumed stress distribution in the arch rib web plates, which can be under-conservative or over-conservative. For this structure, the actual stress distribution is known; therefore the actual stress distribution was used, equivalent to how the stress is calculated for the arch rib flange formulations.
3) AASHTO assumes a doubly-symmetric cross-section and does not account for the contribution of bending to the average stress in an unstiffened web plate. Further, AASHTO does not account for the potential stress increase in the webs due to out-of-plane bending. Therefore, the actual computed average stress in the web plate was used.

Based on this evaluation, design of the unstiffened arch rib flanges applied current AASHTO LRFD formulas for unstiffened rib flanges. For stiffened flanges with a single longitudinal stiffener centered between the web plates, the following equation was used:

$$\frac{b}{t} \leq 2.14\sqrt{\frac{E}{\bar{f}_{panel,max}}} \leq 94$$

The assumptions of the current AASHTO formulations for arch rib web plates are not representative of the free-standing arch rib cross-section proportions. Since the arch rib flange formulations are rigorously derived for the general case of a stiffened plate under a uniform and linear gradient of stresses, they are considered applicable to the design of arch rib web plates. Therefore, the following design methodology was employed.

For unstiffened arch rib webs:

$$\frac{D}{t} \leq min\left(1.25\sqrt{\frac{E}{\bar{f}_{plate}}}, \quad 1.06\sqrt{\frac{E}{\bar{f}_{plate}}}\right) = 1.06\sqrt{\frac{E}{\bar{f}_{plate}}}$$

For arch rib webs with a single longitudinal stiffener centered between the flange plates:

$$\frac{D}{t} \leq min\left(1.88\sqrt{\frac{E}{\bar{f}_{plate}}} \leq 90, \quad 2.14\sqrt{\frac{E}{\bar{f}_{panel,max}}} \leq 94\right)$$

Redundancy Requirements and the Arch Rib

Redundancy of the arch rib was not necessary since it did not meet the requirements of a structural tension member set forth by the project criteria; however the effects of the redundancy requirements were felt by the arch rib and erection methods were used to eliminate tension in the arch rib under permanent loads and vehicular live load. A small amount of tension does exist at the base of the arch rib at the service limit state when including wind loading. The interesting part of the tension or minimum compression at the base of the arch rib is that it is on the opposite flange of a typical minimum compression stress at the base of tied arches in that the tension occurs in the bottom flange. This is a direct result of redundancy requirements for the post-tensioned tie girder.

The post-tensioned tie girder is required to meet the fracture criteria specified above under the fracture load cases including the loss of up to 25% of the tendons with a fracture dynamic force of 0.5 times the force in one tendon applied while maintaining a net compressive force under the fracture service load cases. This criterion to lose tie girder tendons and still maintain a net compressive force in the tie girder results in an initial imbalance in the structural system where there is more force in the tie girder than necessary to resist the thrust of the arch under typical AASHTO loading. The extra force on the concrete tie girder results in creep losses in the tie girder and a tendency for the tie girder to shorten over time. If the forces were balanced there would be no tendency for the tie girder to creep. The extra force and shortening of the tie girder creates a tension stress on the bottom flange at the base of the arch rib and likewise creates a tension stress on the top flange of the arch rib at the crown that increase over time.

The increased tension stresses due to these long term effects resulted in the need to implement erection procedures to lock-in moment in the arch rib to partially offset the long term effects on the arch rib and keep the arch rib plates in compression under permanent load. The first erection procedure involves locking in a moment at the crown of the arch during erection of the arch rib sections on temporary towers by placing the middle three crown sections on temporary towers and lowering the supports in the middle of the arch before assembling the arch rib base sections. This results in a movement of the arch rib down at the center and a rotation of the ribs at the splice points prior to assembling the base sections and hence a kink point in the fabricated shape of the arch. The second erection procedure involves stressing post-tensioning for the tie girder prior to casting the knuckle or tie girder to lock-in a moment at the arch rib base.

Post-tensioned Concrete Tie Girders

The solid cast-in-place tie girder is post-tensioned with longitudinal tendons and anchored behind the load transfer area of the knuckle. The grouted multi-strand post-tensioning tendons provide a highly redundant system in the tie girder. The tie girder is designed to be in net compression under service level loads and also allows for up to 25% of the tendons to fracture with the remaining having the ability to resist all loads with a net tensile stress limited to 0.0 psi. Spare post-tensioning ducts are also provided to replace the loss of 25% of the post-tensioning.

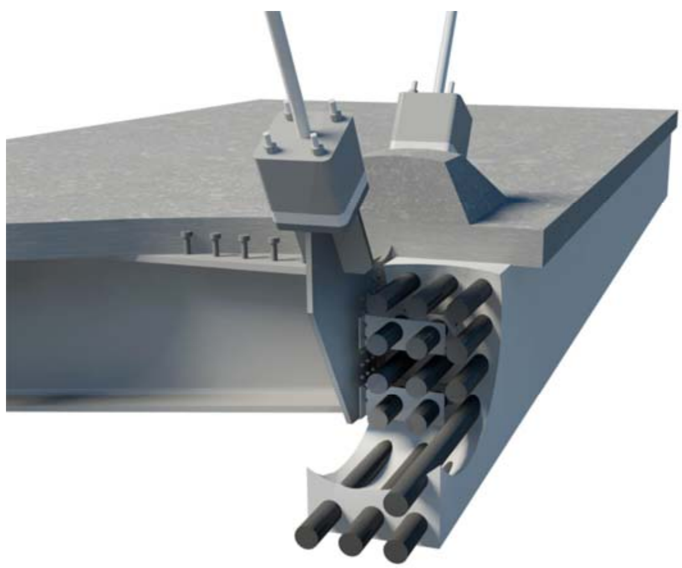


Figure 11 – Post-Tensioned Concrete Tie Girder

The tie girder is designed to limit extreme fiber flexural tensile service stresses in the girder to $0.0948\sqrt{f'c}$ = 0.232 ksi under AASHTO Service III and flexural compressive service stresses in the girder to $0.6f'c$ = 3.6 ksi under AASHTO Service I. The flexural tension and compressive stress of the extreme fiber are not limited under the Fracture Service load cases; however the tie girder must meet the Fracture Load Case Design Criteria for Service as described above.

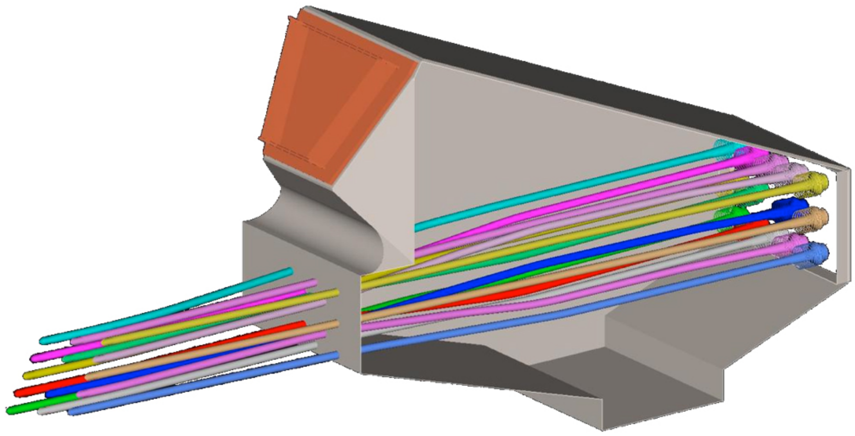


Figure 12 – Post-Tensioning Tendons through Knuckle

To meet the net compressive force design limit under the loss of 25% of the post-tensioning, the tie girders have a significant compressive force under all typical AASHTO loadings throughout the life of the bridge which results in significant creep effects and shortening of the tie girder over time. Due to the integral nature of the arch rib, tie girder and piers, this shortening has an effect on the arch rib, pier and foundation moments.

Tie Girder Redundancy

Fracture of the tie girder system is considered through the failure of individual post-tensioned tendons as opposed to a complete section failure. In the tie girder system, the potential loss of post-tensioning would occur incrementally over time with individual wires in the strands deteriorating enough to fracture. The resulting dynamic force effect is small and theoretically the fracture dynamic force (FDF) is zero for this case. However, a conservative dynamic load factor (DLF) of 0.5 and a resulting FDF equal to 0.5 times the force in one tendon was used due to the importance of the tie girder and bridge.

Girder tie fracture cases included the incremental loss of post-tensioning tendons of up to 25% of the overall tendons. The effects of the loss of post-tensioning is generally localized to the tie girder; however the resulting loss of post-tensioning force varies the forces in the hangers which in turn create moments in the arch rib. Tie girder post-tensioning loss produces minimal effects on the steel floor system and piers.

Integral Concrete Knuckles and Piers

The framed knuckles and integral piers provide lateral stability of the unbraced arch ribs. The concrete in the knuckle takes the compression thrust from the arch rib to the pier and to the post-tensioning in the tie girder. The load transfer zone in the knuckle is in compression from the post-tensioning and compression force from the arch. The reinforced concrete knuckle is designed to take the bending moments from the arch rib and transfer the forces to the pier and pier cap as a reinforced concrete member using strut and tie analysis.

The integral nature of the piers plays an important part in the moment distribution in the arch system and the piers are affected by the long term creep in the tie girder system. The affect of the flexibility of the piers on the rib is also apparent due to the difference in flexibility of the foundations at the two piers. The south pier foundation is anchored by drilled shafts in rock just below the ground level, while the north pier foundation is a deep pile foundation in loose soils below the river bed.

Network Hangers and Connections

Tied arch behavior is similar to that of a simply supported beam, where the arch rib is the compressive flange, the tie girder is the tensile flange, and the hangers are the web. Compared to a vertical hanger configuration, the inclined hangers in the network arch distribute the loads to the arch rib and tie girder in such a way that there is minimal bending in the rib and tie and hence form a more effective web. An efficiency study was performed for this structure and it was determined that hanger angles from horizontal between 55 degrees and 65 degrees provided the most efficient network configuration, and an angle of 60 degrees was used.

The network hangers do provide a challenge when considering the overall structure and erection scheme for this structure. Due to the inclined nature of the network hangers, the hangers not only take vertical load but also have a horizontal component. Therefore, during staged erection and stressing of post-tensioning in the tie girder, half of the hangers begin to take more load while the other half take less. This results in some of the hangers losing all tension and becoming slack if not carefully monitored throughout the erection. Hanger length adjustments are required during the erection to rebalance the forces in the hangers and are taken care of through shim adjustments at the lower hanger connection. A similar phenomenon is seen with the long term effects of creep and shrinkage in the tie girder where some of the tendons become higher stressed and others lose tension over the life of the bridge.

The upper and lower hanger connections to the arch rib and floor beams are seen in Figures 13 and 14 respectively. The upper hanger connections to the rib are individual connections that are bolted to prevent fracture of connection plates in tension from propagating into the rib. The lower hanger connection consists of anchoring two hangers with different alignments at each floor beam location. Each hanger anchors to an individual anchor plate that is bolted to the floor beam end plate and is connected to the tie girder with embedded bent plates bolted to the floor beam and / or embedded anchor rods attached through the floor beam end plate. By separating the anchor plates and bolting them to the floor beam end plate, only the loss of one hanger at a time needs to be considered, and each hanger anchor plate and connection can be inspected and replaced.

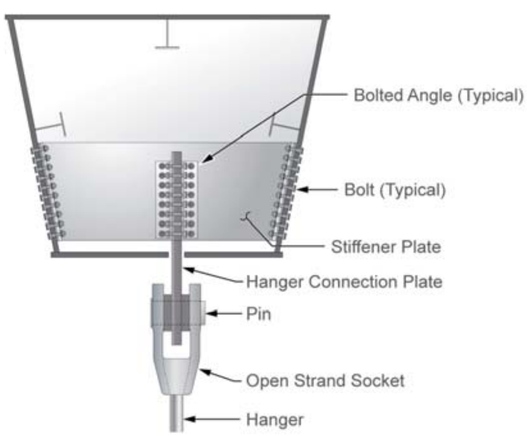


Figure 13 – Upper Hanger Connection

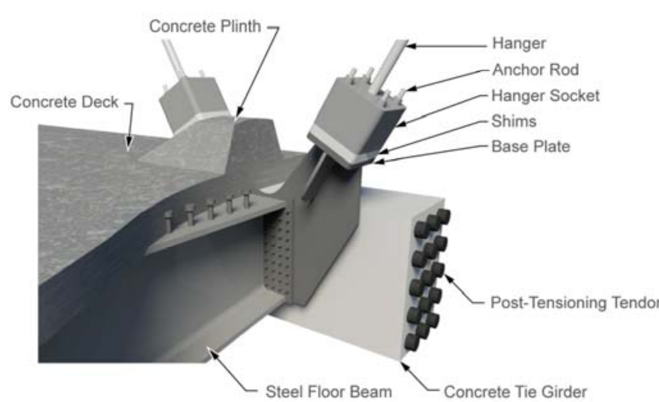


Figure 14 – Lower Hanger Connection

Four rods anchor the hanger socket to the base plate and each of the four rods is tensioned such that the clamping force is larger than any hanger force case, thus there will be no significant fatigue stress range in the rods. The use of four rods also provides an extra level of redundancy in the system such that one can break with no change in the system and along with the shim plates allow for adjustment of the hanger tension.

The hanger design is governed by the allowable stress under AASHTO Service III design load case where the hanger force is limited to 25% of the breaking strength of the hanger. This design criterion is based on historical design criteria of providing a safety factor of 4 under Service dead load plus live load. All hangers are 3.75” diameter structural strand hangers with a maximum hanger force under AASHTO Service III loading of around 400 kips. Under AASHTO Strength load cases, the maximum force in the hanger is limited to 58.5% of the breaking strength. The design criterion was based on 90% of the phi factor for strength in the Post-Tensioning Institute’s (PTI) Recommendations for Stay Cable Design, Testing and Installation, 5th Edition (5), and does not govern design.

Hanger Redundancy

The network hanger configuration and detailing of individual hanger connections also provides an efficient distribution of hanger forces in the hanger system after fracture of an individual hanger or connection. After fracture of a hanger or connection, one hanger remains at the floor beam and can still transfer a portion of the vertical load with the remainder of the vertical load transferred through the tie girder and floor system to adjacent hanger connection points.

Hanger loss is caused either by the fracture of the hanger cable itself or by the failure of hanger connections to the arch rib or floor system. The fracture of any plates in the hanger connection is conservatively assumed to result in the loss of the hanger. The fracture of an individual hanger or hanger connection is accounted for by designing the remaining hangers and hanger connections for the increased forces from the loss of an individual hanger with a conservative dynamic load factor (DLF) of 1.0.

Each hanger is removed from the model geometry to represent the structure at the moment of and after the loss of that hanger. The static force without the hanger from dead load and live load modeling was used to determine the member force change ΔF and in turn the FDF applied opposite the lost hanger force. This is in accordance with the PTI Recommendations for Stay Cable Design, Testing and Installation, 5th Edition. PTI refers to the maximum impact dynamic force resulting from the rupture of a cable as equal to 2.0 times the static force in the cable. In the determination of FDF, the loss of the hanger equals 1.0 times the force in the tendon, while the FDF equal 1.0 times the force in the hanger resulting in 1.0 + 1.0 = 2.0 times the force in the hanger as the impact dynamic force.

The strong stiffness of the tie girder results in the two adjacent hangers (at adjacent floor beam locations) parallel to the lost hanger picking up most of the force originally taken by the lost hanger, while the remaining hanger at the given location picks up a small portion of the load. The tie girder then behaves like a 55 ft long beam with a point load acting downward on its center (the lost hanger location) creating a maximum positive moment in the tie girder at the lost hanger location. Similarly, the arch rib sees negative moment at the location of the upper hanger connection of the lost hanger. The longitudinal component of the force in the remaining hanger at the lost hanger floor beam location is transferred into the tie girder as an axial force.

Under fracture service load cases, the maximum force in the hanger was limited to 50% of the breaking strength. A safety factor of 2 is provided under this condition to preclude yielding of the hanger under fracture condition. Under fracture strength load cases, the maximum force in the hanger is limited to 85.5% of the breaking strength based on 90% of the phi factor for extreme event in the PTI Recommendations for Stay Cable Design, Testing and Installation, 5th Edition.

Tied Arch Span Erection

As with most complex bridges, the erection method plays into the overall design and analysis of the structure. The complexity was increased significantly for this project due to the framed nature of the free-standing tied arch, the network hanger system and the extra post-tensioning in the tie girder to meet redundancy requirements. Further, since in-place erection was not practical due to the requirement to keep the river channel open throughout the project with minimal interruption, significant consideration had to be given to the offsite erection and final placement methods early in the design development during the design-build procurement phase.

The construction/design team determined very early that the traditional methods of erecting the arch off site on high towers and floating it in over the piers was too risky due to the high center of gravity and the variability of river water elevations which could delay move-in. Therefore, the team elected to erect the arch low on barges, float-in between the piers and lift the arch into place. Alternatively, a land based low build/lift scheme was investigated but the design-build nature of the procurement did not allow for sufficient vetting of this alternative to proceed prior to bid. The land based alternative for steel erection has since been chosen to more easily control arch erection and limit the risk of fluctuating water elevations for erection on barges.

To facilitate the erection of the arch ribs and steel floor system, a temporary steel tie girder is used. The tie also serves to stabilize the floor system and support the formwork for the cast-in-place tie. A steel lifting connection serves as a temporary knuckle connecting the arch rib with the temporary tie. Each seven segment arch rib is erected from the center segment down and supported by temporary towers. The arch ribs are braced during erection and the entire steel system is framed utilizing the arch bracing, floor system, and a lower lateral bracing system. Formwork, rebar and empty post-tensioning ducts are also assembled prior to moving the arch into place.

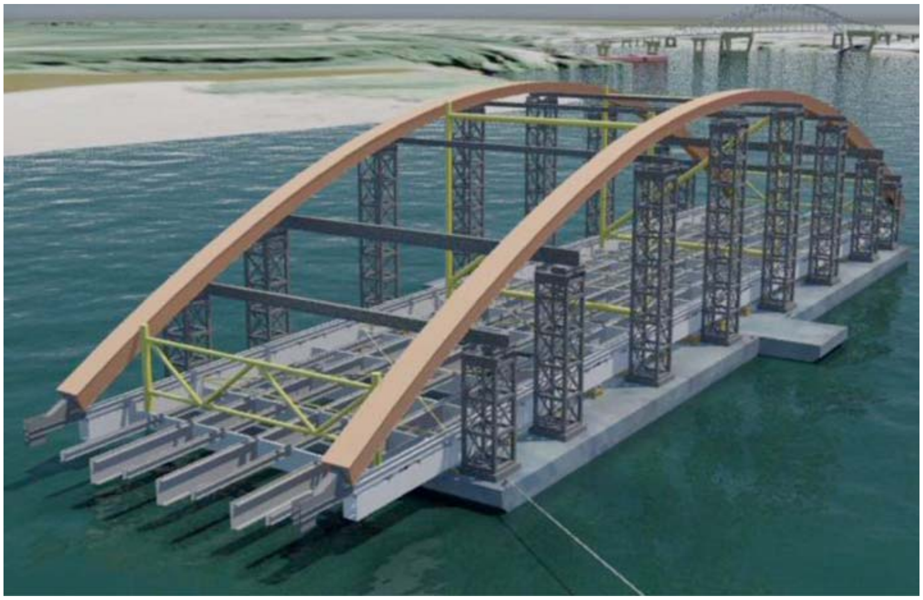


Figure 15 – Arch Erection with Temporary Towers

After erection the steel framed tied arch system is lifted off the temporary towers using SPMT's supporting the arch under each corner and transported onto two barges in the river. The temporary tie is expected to elongate about 4-inches when the floor and arch system is lifted from their supports and the load transferred to the tension tie. The arch is then floated downstream and skidded onto a rail system to slide into final plan position between the piers. Once in place the strand jacks are attached and the arch is lifted into position.

Figure 16 – Arch Lift

The lifting frame supporting the strand jack system is anchored directly onto the top of the pier. Once in place a support frame is moved into position under the temporary knuckle and the bridge is lowered into its final position. The lifting connection and support frame will be cast into the permanent concrete knuckle. To compensate for the additional post-tensioning and long term creep associated with the extra post-tensioning in the tie girder for redundancy, the piers are jacked apart by jacking against the main span to lock-in an initial deflection and moment in the piers. Initial post-tensioning tendons are then installed and stressed prior to the concrete knuckle and tie girder pours to lock in moment in the arch ribs.

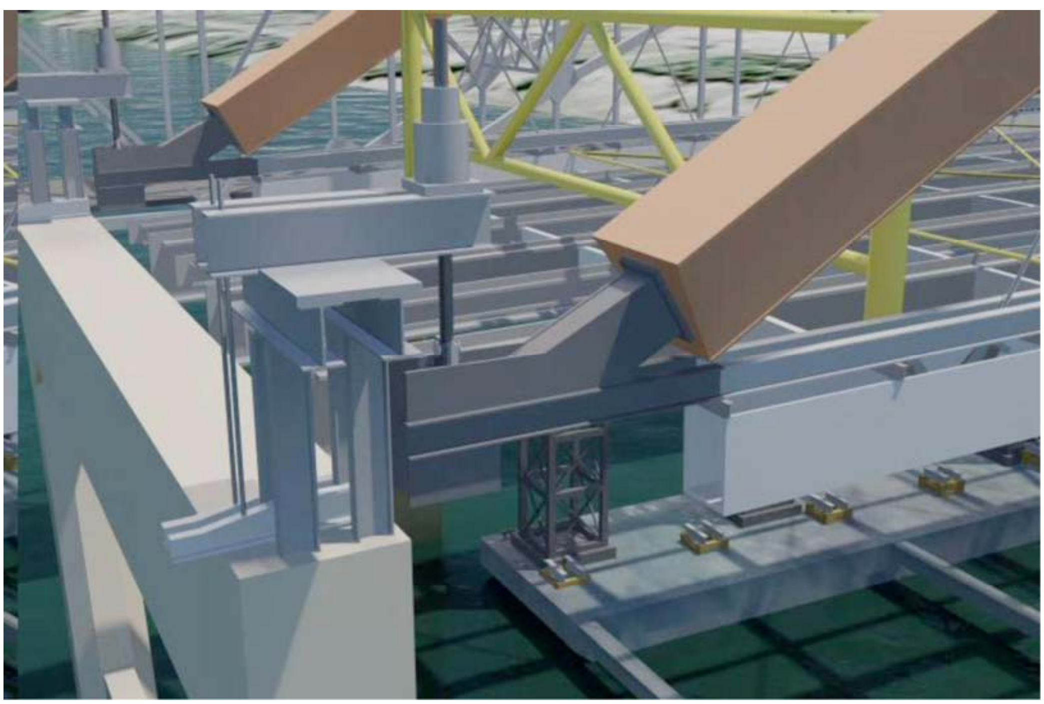


Figure 17 –Lifting Frame and Strand Jacks

The concrete knuckle concrete is placed at all four corners, simultaneously at the two corners at one pier, followed by the simultaneous placement of the tie girder concrete from the center of the span outward. The tie girder is then post-tensioned in stages to relieve initial creep strain and remove the tension in the temporary tie girder such that it can be removed from the system prior to placing the deck. The staged post-tensioning of the tie girder prior to the deck pour results in the need for a hanger shim adjustment to rebalance the forces in the hangers.

Figure 18 – Knuckle and Tie Girder Placement

The deck is then placed in a single pour beginning at the center of the bridge and moving outward simultaneously to each end. The temporary arch bracing remains in place until the deck is cast and hardened to provide the lateral stability for the free-standing arch ribs. Hanger adjustment for final grade geometry is then made by modifying the shim packs of the hangers.

Figure 19 – Completed Structure

100-Year Service Life

As a new, modern structure on a critical transportation link, the structure is required to be designed for an extended service life of 100 years. Certain limitations and material requirements to achieve this goal including stainless steel reinforcing bars in the deck, residual compressive service stresses in most post-tensioned elements, non-structural wearing coarse, and a stainless steel deck drainage system were requirements for the project. A Corrosion Protection Plan quantifying the 100-year life goal was also required and included an assessment of concrete service life span and a life cycle cost analysis for bridge wear elements.

The overall approach to achieving a 100-year service life is based on combining proper detailing, durable materials, detailed quality control, and easy inspection and maintenance. The Corrosion Protection Plan that was developed and approved early in the design process provided an overall approach to achieving a 100-year service life for the bridge. Each bridge component was identified with its corresponding environmental exposure and relevant degradation and protective mechanisms were identified with respect to time. The expected service life of each component was then estimated and enhanced if necessary either by selecting materials with reduced corrosion potential or by selecting materials and details to resist degradation.

The quantitative assessment of the bridge component durability is based on modeling corrosion of uncracked reinforced concrete using Life-365 with reliance on general bridge inspection and evaluation experience. The Life-365 model was further used to assess the effect of variations in materials, concrete cover and the environmental exposure conditions to assess the sensitivity of each factor on the service life. Based on this analysis, the critical factors or conditions were identified and design detailing or material selection was adjusted to meet the requirements of the corrosion protection plan and achieve the criteria for a 100-year service life of the bridge.

The elimination of cross bracing and secondary members in the case of the large box section of the vertical free-standing arch ribs offers significant durability, inspection, and quality advantages by eliminating smaller, hard to inspect areas where corrosion can begin unnoticed. The solid post-tensioned concrete knuckles and tie girders provide durability and protection to the post-tensioning system and eliminated maintenance of any interior spaces. The integral connection of the knuckles to the piers also eliminated the need for large specialty bearings that typically present significant maintenance and life cycle costs. For enhanced durability the concrete deck and tie are to be constructed continuously without construction joints with the post-tensioning compression adding to the crack resistance of the concrete systems.

Durability was also provided by specifying materials with reduced corrosion potential and that resist degradation processes. Corrosion-resistant stainless steel rebar is used in the main bridge deck and painted weathering steel is used throughout the structure. The fully replaceable hanger system which utilizes galvanized structural wire lubricated with a protective grease inner coating and painted using a noxyde paint system provides three levels of corrosion protection. A two-inch low-permeability wearing course on the deck riding surfaces acts as a sacrificial layer that can be easily removed and replaced in the future, while maintaining the structural integrity of the deck. A high performance concrete specification for mix designs, testing, and placement was developed which focuses on thorough testing of materials and the use of pozzolins such as fly-ash and slag to control permeability and heat of hydration for mass concrete.

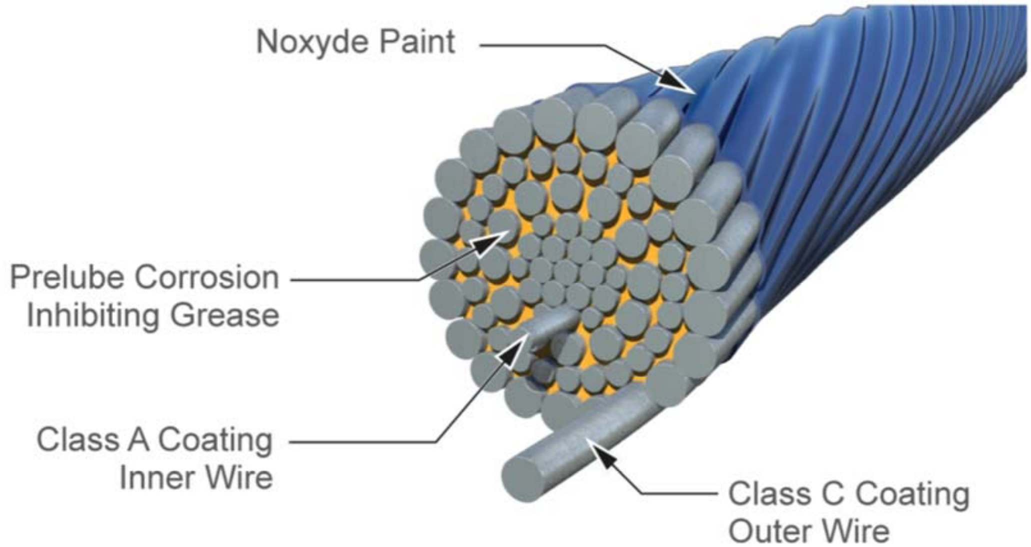


Figure 20 – Hanger Strand Protection

The projects Quality Management Plan is integral to achieving the 100-year life through enhanced quality testing and performance matching of raw materials combined with controlled construction practices. Pre-screening testing of concrete mixes and raw materials were required to ensure aggregates, cement and admixture combinations meet the criteria of the corrosion protection plan. Enhanced steel fabrication quality practices are identified for the steel floor system, which require weld testing above the minimums required for AWS 1.5 specifications for redundant structures. This will also be the first US bridge project to utilize Computed Radiographic Testing (CRT) for the non-destructive weld testing. CRT is enhanced over traditional Radiographic Testing by utilizing digital photographic technology to provide an immediate and permanent digital record of the radiographic test.

Achieving a 100-year service life depends on periodic inspection and maintaining protective systems over the life of the bridge. As noted above, the free-standing arch rib and concrete knuckle and tie provide for ease of inspection. Further, the hanger system is easily accessible from deck level or with a vertical lift providing end-to-end visibility and access to the structural elements during maintenance and inspection activities.

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