PRECAST NETWORK ARCH BRIDGE

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ABSTRACT

The Texas Department of Transportation (TxDOT) has designed what is believed to be the world's first precast network arch bridge. A series of six 163' post-tensioned spans are currently being constructed in Fort Worth. In an effort to reduce cost and minimize on-site construction time, the arches are being cast on their sides four blocks from the bridge site. A specialized lifter is being used to raise and rotate the arches into a vertical position. Self-Propelled Mobile Transporters are being used to move the arches onto columns that are located just outside of the existing bridge rails. After demolition of the existing structure, a total of 102 precast, pre-tensioned floor beams will be connected to the concrete ties of the through-arches. A tight spacing of the hangers (4.81') allows for a small floor beam spacing (9.62') which in turn negates the need for longitudinal stringers thereby simplifying design and speeding construction. The dense weave of stainless steel hangers also creates a unique aesthetic as the metallic shroud will diffuse the light from a series of fixtures embedded down the length of the arch tie.

Keywords: Precast, Network, Arch, Bridge, Concrete, Post-tensioned

INTRODUCTION

West 7th street in Fort Worth is an important thoroughfare that directly links downtown to the Cultural District, home to five internationally-recognized museums including the Kimbell Art Museum and the Modern Art Museum. The area is also experiencing a rebirth of construction including retail, condominium, and apartment developments.

The two areas are separated by the Clear Fork of the Trinity River. The original 138’ arch span with girder approach spans is 100 years old and the concrete is porous, highly carbonated and contaminated with chlorides. It was lengthened from 437’ to 981’ with concrete girder spans 60 years ago when a major flood control project re-channelized the river. After originally intending to strengthen the structure and widen the narrow, 4’-4” sidewalks, the city council decided in 2007 that rehabilitation would be expensive, would not support future light rail, and result in a mish-mash of 1913, 1953 and 2013 architectural styles.

PROJECT CONSIDERATIONS

In addition to carrying four lanes of vehicular traffic, the city requested 10’ wide sidewalks to accommodate the increasing number of pedestrians and cyclists using West 7th street as both a commuter route and a path to the park below. The bridge also had to be designed and constructed to accommodate possible future light rail that would convert two of the vehicular lanes into shared-use lanes. The slab is thickened to 13.5” in two regions to allow for embedded track (Fig. 1).

The 981’ long bridge has to cross a waterway, a roadway, and park trails while ending on an existing levee. A non-skewed span arrangement of six 163.5’ spans managed to clear all these restrictions. The vertical alignment of the roadway had to be raised to meet current hydraulic requirements. However, the amount was limited by the combination of the 5% maximum allowable ADA sidewalk grade and a cross street located 350 feet west of the abutment.

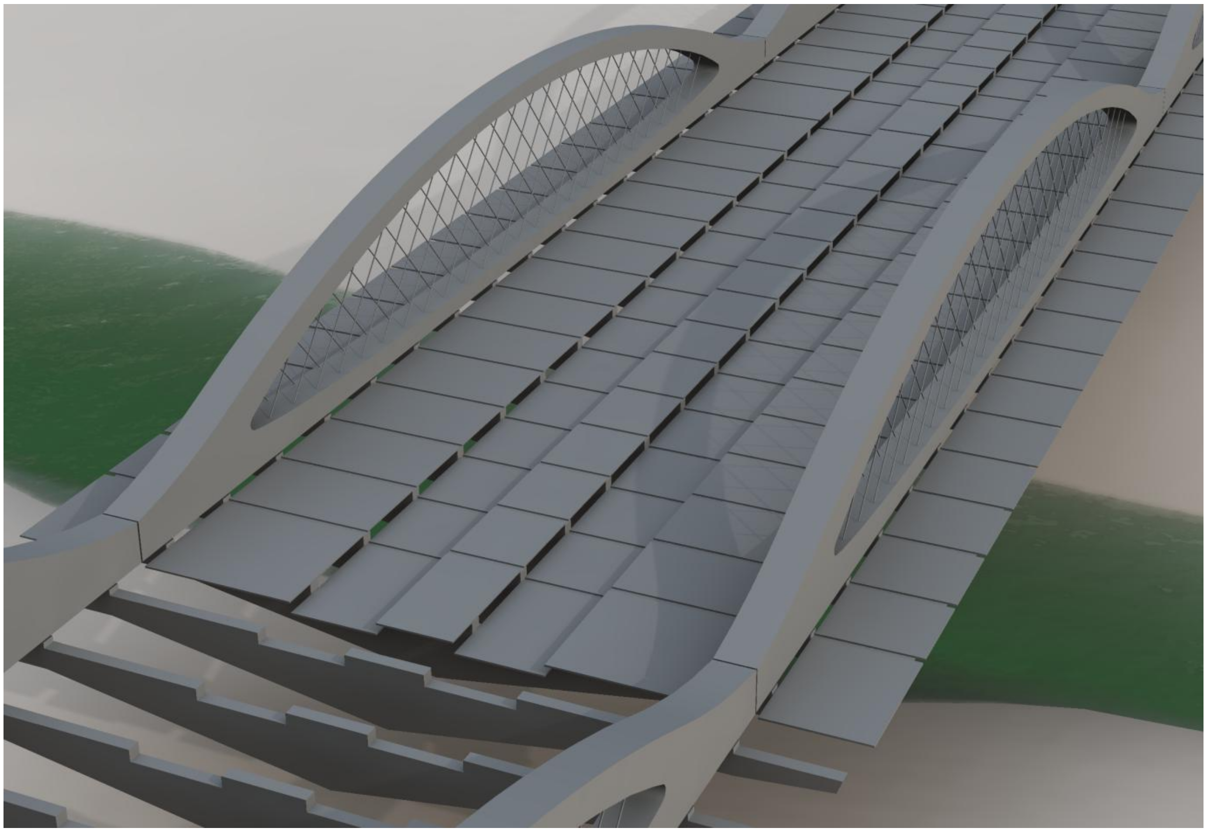

Fig. 1. 88' Wide Bridge with Precast Arches, Floor Beams, and Deck Panels

STRUCTURAL SCHEME

Texas Department of Transportation (TxDOT) engineers who initially worked with the city to provide the structural assessment of the existing bridge and develop rehabilitation and widening plans were now asked to provide ideas for the new bridge. They presented multiple options including a combination steel trapezoidal and concrete U-Beam girder design as well as a one consisting of a series of short (81.5') deck-arches. TxDOT engineers were well aware that aesthetics was of paramount importance at this site given the park setting and proximity to museums designed by such luminaries as Louis Khan, Tadao Ando, Phillip Johnson and Renzo Piano and proposed a unique precast network arch. The city council was overwhelmingly in favor of this option.

The precast network arch—believed to be the world's first—had many advantages over the other alternatives. First, the through-arch system is hydraulically efficient. Even when factoring in cross-slope on a wide bridge, the bottom of the slab is only 15" below the vertical control line. Of course the floor beams are located underneath the deck and need to be accounted for, but their orientation is parallel to the river flow. The through-arch will also allow motorists on West 7th Street to view the structural system: a series of six arch pairs rise just outboard of the travel lanes (Fig. 2). The pedestrians will be physically shielded from the traffic by a short railing and psychologically safeguarded by a metallic mesh of 1 ¾" diameter diagonal hangers. Stainless steel was chosen for its durability, reflective properties, and tactile quality as pedestrians can easily reach out and touch the hangers. Likewise, custom stainless clevises were designed rather than simply selecting available utilitarian hardware. Concrete was selected for the arch due to its ability to be molded into a graceful shape all the while withstanding the complex flow of forces both in the knuckle and hanger connection regions. The economic benefits of concrete became apparent when engineers were able to divide the existing bridge length into six 163.5' simple spans thereby allowing for twelve identical arches. Finally the through-arch presented the opportunity for dramatic nighttime lighting. A series of fourteen light fixtures are set into each arch tie and will illuminate the stainless hangers and underside of the rib.

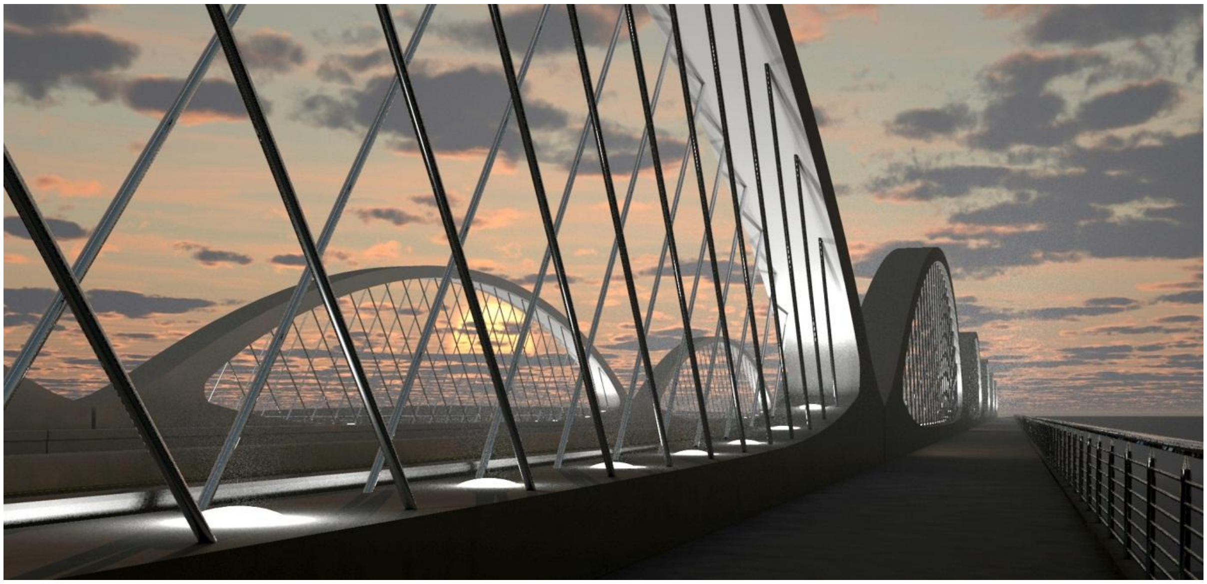

Fig. 2. Rendering of Completed Precast Network Arch Bridge

The substructure consists of 7'-3" x 5'-6" oval columns supported on 84" diameter monoshafts (Fig. 3). The bearings are High-Load Multi-Rotational Disc bearings that are fixed longitudinally and transversely on the back end of every span and guided longitudinally on the forward end.

Fig. 3. 7’-3” x 5’-6” Oval Columns and Demolition of Old Bridge

ARCH DESIGN

The arch design presented a number of unique challenges including determining the best way to cast the arches. Advances in lifting technology meant that designers weren’t constrained to casting the arches in the vertical position. Working at lower elevation, better geometric control of vital embedded items, a simpler forming system, and better access to items in the knuckle favored casting the arches horizontally.

Of course casting in the horizontal position required a means to prevent cracking during rotation (Fig. 4). Designers provided two 19-strand, 0.62” diameter tendons in the arch rib fully stressed to 0.77 fpu and four in the tie stressed to 0.385 fpu prior to rotating (Fig. 5). The contract drawings indicated the net compressive stresses at intervals along the rib and tie so the contractor could develop a lifting scheme that did not introduce tension into the arch. The contractor proposed a lifting system in which six lifting frames connected the tie and rib at defined locations. The lifting frames were attached to the arch to facilitate the rotation of the arch to the vertical position using the gantry system shown in Figure 4. Prior to casting, researchers at The University of Texas at Austin placed vibrating wire gages at critical locations to monitor the stresses during rotation, transport, and other significant construction events.

Fig. 4. Arch Rotation

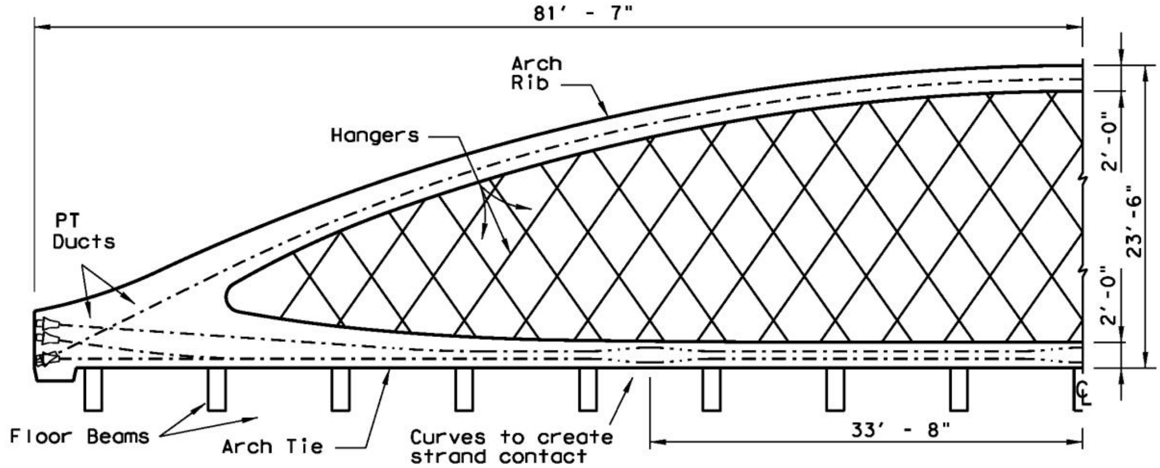

Fig. 5. Arch Elevation

Another challenge arose from the fact that for a multi-span bridge, the arches sit end-to-end. With only a 4" gap separating the spans, there is no room to perform multi-stage stressing in the final position which would result in low-to-moderate compressive forces in the tie at all times. As a result, the tie tendons were re-stressed to 0.77 fpu in the casting yard after rotation. Since the arch self-weight generates less than 25 percent of the total axial service tension in the tie, the slender tie elements experience tremendously high compression forces prior to placing floor beams and other subsequent gravity loads. Also at this stage, the widely spaced rib tendons were de-stressed by half. The contractor was permitted to de-stress the tendons by different amounts if necessary to remove sweep from the unbraced rib. However, this proved unnecessary as the lateral rib deflections were negligible for all twelve arches. The rib and tie tendons were then grouted.

The maximum compression that the tie would experience was still to come. The hangers were installed snug-tight when the arches were in the horizontal position. When rotated vertically and subjected to gravity, the loads in the individual hangers were unknown. The large number of hangers (52), stiffness of the system, and low initial loads on the hangers under self-weight brought into question the wisdom of using a jack to stress the hangers individually. Designers thought it preferable to use the self-weight of the tie to load the hangers. The method proposed was to tighten the nuts while the tie was lifted upward allowing the self-weight of the tie, when released, to load all the hangers simultaneously. A series of hydraulic rams were located at each of the interior thirteen floor beam locations. All were connected thru a manifold to a single pump and each jack was simultaneously loaded to 17 kips. The hangers were pushed by hand into a nearly straight and inclined position, the nuts re-snugged, and jacks removed (Fig. 6). The predicted maximum compressive stress on the tie at this stage was 2500 psi.

Fig. 6. Lifting the Arch Tie with Hydraulic Rams and Tightening Hanger Nuts

If the resultant deflected shape was negative at the midspan of the tie following the application of the 17 kip force, the contract documents allowed for the contractor to increase the force to as much as 20 kips per ram. The additional jacking force would raise the compression at the tie bottom fiber to 3000 psi, the maximum compressive stress for design. When it came to specifying a required concrete strength, designers had to take into account the fact that their analysis used the gross section of the tie. A more rigorous analysis in the already complex finite element model that accounted for the hanger anchorage blockouts, floor beam PT bar anchorage blockouts, light fixtures, post-tensioning ducts and a constantly shifting neutral axis as a result of these voids would surely increase the stress above 3000 psi (Fig. 7). For simplicity, designers specified a conservative 56-day compressive strength of

8000 psi. As it happened, the upward camber at midspan for the twelve arches ranged from 0.5” to 0.63” with a load of 17 kips per jack so there was no need to use a larger jacking force.

Unfortunately calculations showed that the hanger stressing method of jacking the tie upwards created tensile stresses in the top of the arch rib near the knuckle. Designers tried numerous tendon alignments, but to no avail; there was no arrangement of the six tendons that wouldn’t create tension in the rib or tie at some time during construction or service. To eliminate the tensions, prestressed girder strongbacks were clamped to the ribs prior to upward jacking of the tie (Fig. 8). The strongbacks will be removed after the floor beams are installed as there is sufficient rib axial force in the rib at this stage to overcome the bending-induced tensions.

The tremendous compressive force on the tie and (theoretical) lack of contact between the strands and ducts meant that there existed the possibility of the tie deflecting laterally during the final tie tendon stressing phase in the casting yard. The displacement would continue until the interior walls of the ducts contacted the strands and reached equilibrium .[1] In order to reduce the unbraced length and minimize lateral movement of the 2’ x 4.5’ tie during stressing, a series of small curves were added to the ducts causing contact with the four 19 strand tendons at 33’-8” intervals (Fig. 5 and Fig. 7).

The arch width was set at 4’-6” to provide enough area to embed the myriad of elements in the tie, enough room in the knuckle to anchor tendons and sufficient stiffness to resist rib buckling. Two post-tensioning ducts were placed in the tie to accommodate future post-tensioning capabilities should the tie ever experience distress (Fig. 7). The tight hanger spacing resulted in only small shear forces developing along the length of the tie and allowed for a shallow depth. The arch rise was kept deliberately low in order to make rotating and hauling easier. A span-to-depth ratio of only 7.44 keeps the center of gravity at 7.65’ above the bottom of the tie.

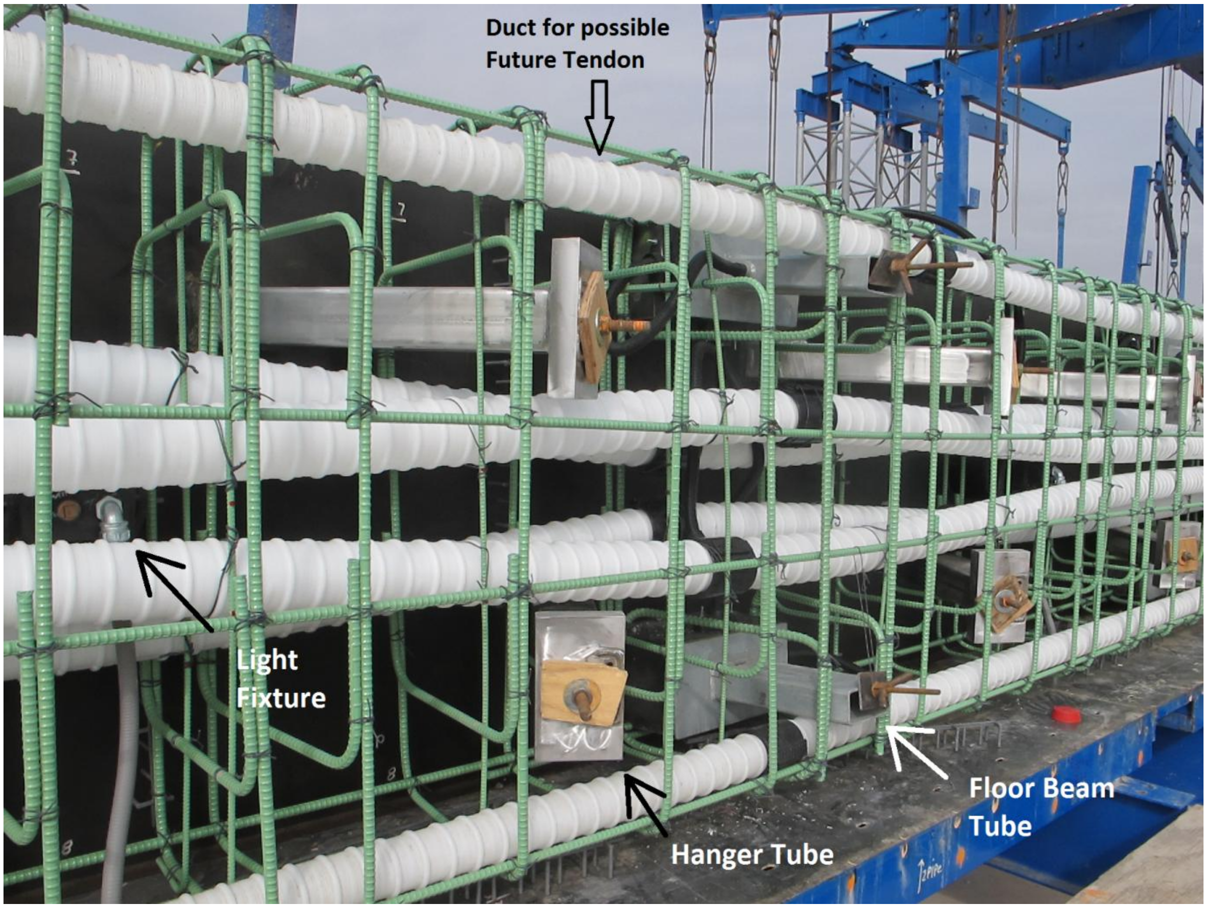

Fig. 7. Arch Tie Elements Including Post-tensioning Ducts with Intentional Deviation

Fig. 8. Strongbacks Attached to Arch Ribs Prior to Stressing Hangers in Casting Yard

Unlike most arches, the West 7th Street Bridge does not utilize longitudinal stringers spanning between floor beams. The modest arch span length and correspondingly short intrados (R=160') requires a tight hanger spacing to prevent the accumulation of rib bending moments in the small rib cross-section. A hanger spacing of 4.813' as measured along the tie and a floor beam spacing of 9.625' results in the floor beam reaction being transferred directly to the four surrounding hangers (Fig. 9). Finally, a standard TxDOT slab of 4" of stay-in-place prestressed panels with a 4.5" topping slab can span between the hangers without the need for longitudinal stringers.

ARCH ANALYSIS

Designers aimed to keep the arch concrete free of tension for durability, aesthetics and to maintain consistency in the analytical assumption of uncracked sections. Due to the connection details and slenderness of the hangers any model that produced compression in the hangers was deemed unsuitable. Once initial section sizing calculations were performed, numerous post-tensioning layouts were analyzed through the following stages: first stage post-tensioning, rotating, second stage post-tensioning, floor beam installation, deck casting, and wind and live loading. After a tendon layout that satisfied all stress limits was found, strength checks were made and the profile approved.

For speed, cost, and appearance, no rib cross-bracing is used. Eigen buckling studies were undertaken for initial stability analysis. The lowest Eigen buckling value for the completed structure was 13.3 which occurred at Service I with six lanes of traffic and wind load. The resulting arch buckling mode was a maximum deformation at the arch crowns in the direction of the wind.

A more rigorous non-linear buckling analysis was also performed. Seven different factored loadings from AASHTO LRFD Bridge Design Specification Load Combinations I, III and V were examined. The model revealed that even for the most severe condition, Strength III (transverse wind), the crown deflection at a load factor of two was only 2" and the load-displacement curve was nearly linear.

A further study was undertaken to examine the negative consequences of rib sweep that might arise from improper casting, improper storage, variation in modulus of elasticity due to bleed water migration during the horizontal casting, or some other unknown cause. To generate the most severe sweep profile, an Eigen run was made using the load factors for Strength V with six lanes loaded. The displaced profile was then imported into a non-linear model with the same loading applied. This beginning profile was scaled so that the crown lateral displacement was 6". Even with this drastic initial sweep, the additional crown deflection was only 2 ¾" at a load factor of two and the load-displacement curve had yet to plateau.

ARCH CONCRETE

The high performance concrete (HPC) used in the arches is some of the most sophisticated ever produced for a TxDOT project as there are a multitude of difficult requirements: low-shrinkage, low-heat, low-permeability, high-slump, and high-strength. Since the arch is essentially a frame by virtue of being connected by large knuckles, the combination of shrinkage and thermal differential could cause cracking during curing. The following prescriptive requirements were imposed on the mix to minimize shrinkage and thermal cracking: limit maximum cementitious materials content to 700 lbs/CY; set a water-to-cementitious materials ratio between 0.34 and 0.43; require the concrete mix to utilize optimized aggregate gradation; and include the use of a shrinkage-reducing admixture. Additionally, a thermal control plan was required to demonstrate that excessive heat and heat differential within the concrete would not occur during hydration. This required the addition of cooling tubes in the knuckle region. To ensure low-permeability concrete, a minimum of 25 percent of the cementitious materials were required to be Class F fly ash. Consolidation was a major concern because of the congestion in the tie from hanger and floor beam connecting elements, post-tensioning ducts, embedded light fixtures, and reinforcing. As a result the contractor selected a 9” water/admixture induced slump mix that was still able to attain a required 56-day compressive strength of 8000 psi.

FLOOR BEAM DESIGN

The floor beams not only serve to carry the deck loads to the arches but also brace them to prevent buckling. As a result, a global analysis assuming a full moment connection between the floor beams and arch tie was required. The lack of significant torsional resistance of the 2’ deep arch ties result in a small negative service moments (-210 k-ft) in the interior floor beams at the arches. Not surprisingly, the positive service moment of the interior beams at midspan is much larger (1880 kip-ft). Both the negative and positive moments are larger for the exterior floor beams as they support a thicker cantilever slab (10” overall, nominal), cannot distribute load to adjacent floor beams and are connected to the semi-rigid knuckles. The negative service moment at the knuckles is 560 k-ft and the midspan positive service moment is 2790 k-feet. The first interior floor beam moments are slightly larger than those for floor beams near the center of the arch span but are reinforced identically to the exterior floor beams.

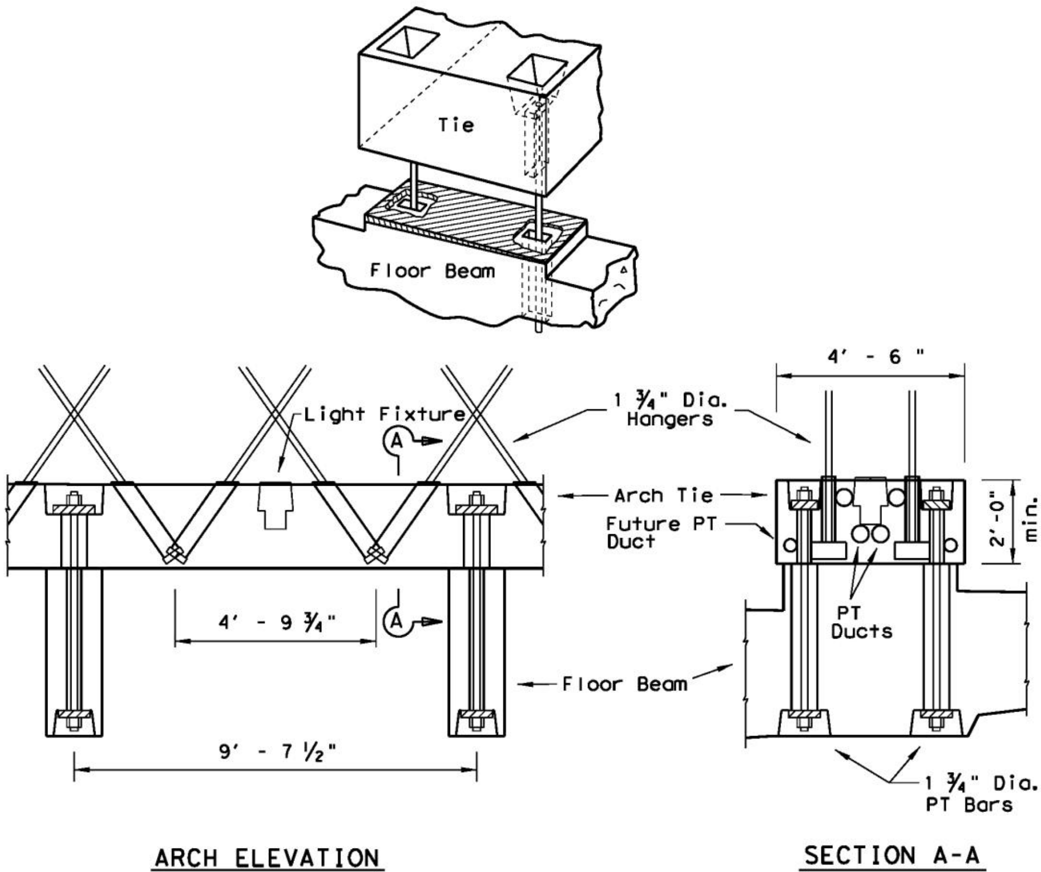

Fig. 9. Floor Beam Connection to Arch Tie with PT Bars and Epoxy Grout

Designers chose to use pre-tensioned floor beams for their quick installation, durability, cost, low-maintenance and aesthetics (Fig. 10). The floor beams have a constant width of 1'-4", a nominal depth of 5'-6" at midspan, and a minimum depth of 3'-0" at the arch with a taper down to 1'-9" at the end of the cantilever. Two different strand layouts were used: thirty-two, ½" diameter strands in the first two and last two beams in the span and twenty-four strands in the interior thirteen floor beams (Fig. 11).

The connection of the floor beam to the arch is a critical component of the bridge as it needs to carry both tension and moment, reconcile two non-matchcast concrete surfaces at each end of the floor beam, and prevent the intrusion of water. The solution was to cast two HSS 6 x 3 x 3/16 sleeves into the beam 3'-2" apart. Companion tubes were cast into the arch tie and knuckle at the same spacing but rotated 90 degrees about the longitudinal tube axis (Fig. 9). This allows for the largest possible range of mis-alignments that will house a 1 ¾" diameter post-tensioning bar. To account for non-planar surfaces, an approximately ½" tall bed of epoxy grout will be placed on top of the floor beam plinths located underneath both arches. The floor beams will then be raised up until contact with the arch is made all around the 1’-4” x 4’-2” bed of epoxy grout. After the grout reaches 4000 psi, the two post-tensioning bars at each arch will be stressed to 105 ksi and the HSS tubes grouted.

Fig. 10. Lifting Precast Floor Beam from Casting Bed

Fig. 11 Strand Layout for Exterior Floor Beam

The floor beams were cast at Heldenfels Enterprises in San Marcos, TX. In order to provide a smooth finish and minimize vibration and finishing efforts, a self-consolidating concrete mix was chosen. It included the use of super plasticizer, retarder, and a viscosity reducing admixture, a cement content of 640 lbs/CY, Class F Fly Ash of 213 lbs/CY and a water/cement ratio of 0.31. The target spread was 24” +3”/-2”. The 12 hour compressive strengths averaged 5600 psi which allowed for a one day turnaround on the casting bed. The average 7-day compressive strength of 9600 psi far exceeded the required 6000 psi 28-day compressive strength.

CONSTRUCTION PROGRESS

The bridge project was let in June 2011 at a cost of 209 $/SF. Shortly after being awarded the contract, the contractor created a BIM model of the arch to assist in shop plan preparation (Fig. 12). The first arch was cast in July 2012 and the final one in February 2013. Two casting beds were used in order to meet the accelerated schedule and keep the work crews continuously busy. Site work began in January, 2013. The 4-block transport of the arches from the casting yard to the site using Self-Propelled Mobile Transporters began in

May 2013 as PCI Journal went to press (Fig. 13). Completion is anticipated in November 2013.

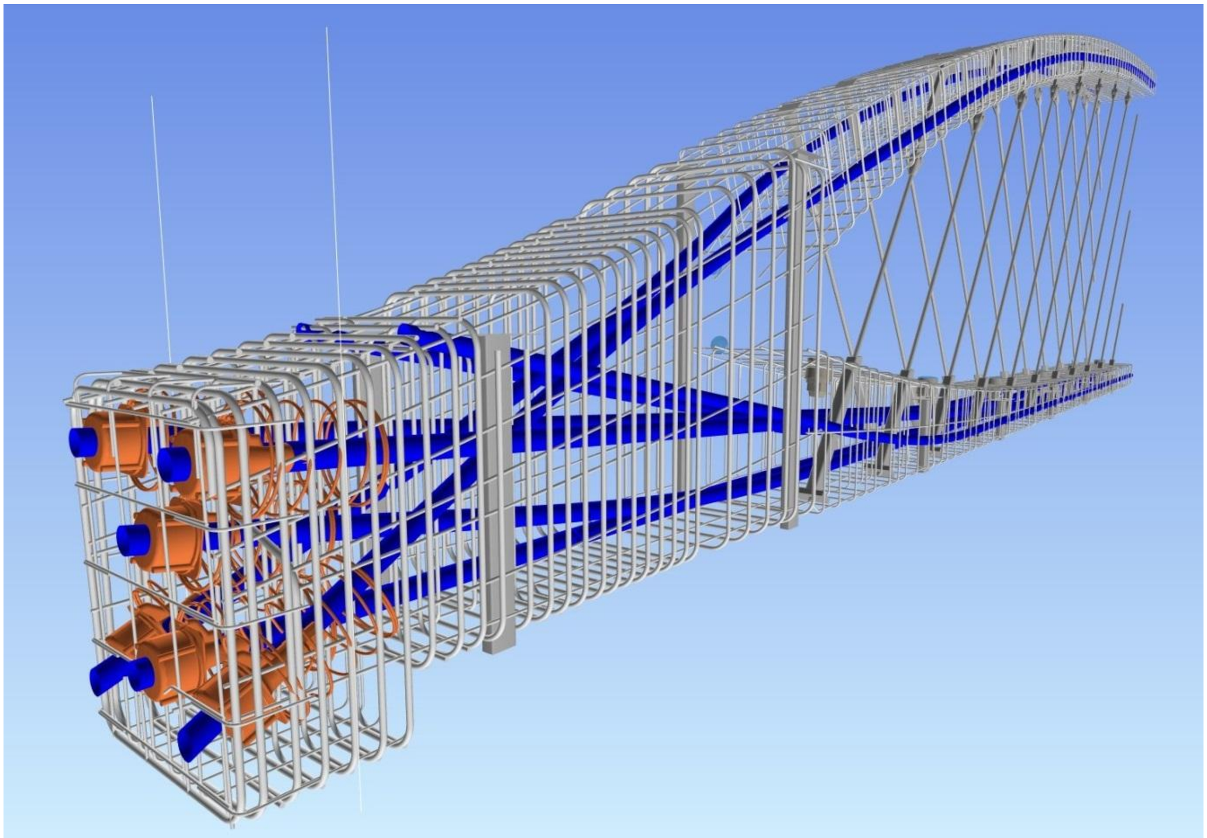

Fig. 12. Sundt Construction BIM Model of Knuckle Region

Fig. 13. View from Downtown of Two 600 Ton Cranes Lifting Arch from Existing Bridge

REFERENCE

[1] Lin, T.Y., and Ned H. Burns. *Design of Prestressed Concrete Structures*. 3rd ed. New York: John Wiley & Sons, Inc., 1981.

CREDITS

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Arch and Column Forms: Aluma Systems, Inc.
Arch Concrete: TXI, Inc.
Floor Beams: Heldenfels Enterprises, Inc.
Floor Beam Forms: Hamilton Form Company

Stainless Steel Fabrication: GST Manufacturing
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