Construction of the Broadway Bridge Arch Spans over the Arkansas River

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IBC-17-31

KEYWORDS: Network Tied Arch, Arch Erection, Accelerated Bridge Construction, Barge Stability, Falsework Towers

ABSTRACT: The Broadway Bridge connects Little Rock, Arkansas and North Little Rock, Arkansas and spans the Arkansas River. The project minimized the duration of the roadway closure by incentivizing construction completion over a specific time limit. Erection of the new spans was performed at high elevation using steel towers supported on barges. Following fast-track demolition of the existing bridge, the new spans were floated into position and prepared for deck placement and final construction activities.

INTRODUCTION

The Broadway Bridge connects Little Rock and North Little Rock and spans the Arkansas River in Pulaski County, Arkansas. The bridge carries 24,000 vehicles per day and is a critical link between the adjacent communities.

The original Broadway Bridge was constructed in the 1920s and consisted of several cast-in-place concrete spans highlighted by five 200′ concrete spandrel arch spans over the river. The original bridge was modified in 1974. A single 412′ steel through arch span replaced two of the original concrete spandrel arch spans. The original bridge prior to replacement can be seen in Figure 1.

The existing Broadway Bridge necessitated replacement for various reasons. There were many structural deficiencies identified in past inspections that would have required repair. There have been both increasing vehicular demands and navigational demands. Additional travel lanes and shoulder width, along with an improved shared use path for pedestrians, was required. The width of navigable channel and vertical clearance for river traffic also required improvement. Addressing these considerations pointed to the necessity of replacement.

Figure 1 - Original Bridge

After consideration of various structure types, two 440′ "basket handle" tied arch spans were selected to cross the main navigational channel. The new bridge carries two lanes of traffic in each direction along with a 16′ shared use path.

In addition to the twin 440′ tied arch spans, the overall project scope also included three south approach spans, two north approach spans, a southbound exit ramp, and two shared use ramps. While the overall project scope is extensive, the focus of this paper is the construction of the tied arch spans.

The design of the network tied arch spans was performed by HNTB and was completed in 2013. The request for proposal (RFP) was issued by the Arkansas Department of Transportation in June, 2014. An effective bid price was defined to consider the proposed number of days in which the bridge would be closed to traffic. In this way, the selection process considered the initial unadjusted bid price, but also placed significant weight on the duration of bridge closure. Thus, it was essentially defined that an accelerated bridge replacement scheme be selected in order to be awarded the project.

As part of the winning $98.4M bid by Massman Construction, a 180 day closure window was specified. From the day the existing bridge was closed to vehicular traffic, the new bridge was required to be open to vehicular traffic in 180 days. As part of the contractual agreement, should the bridge be open to traffic sooner than 180 days a bonus would be awarded to Massman Construction, but any delay would result in a similar penalty.

From the beginning, the project was planned to minimize the duration of the roadway closure. As the construction of two 440’ network tied arch spans is complex and time consuming, “in-place” construction methods would have proven impossible to meet the 180 day closure window. To meet the imposed time limit, it was decided the two arch spans must be partially constructed prior to the beginning of the 180 day closure window.

Various techniques for construction of the arch spans “off-line”, and moved into position were evaluated. By constructing the arch spans “off-line”, they could be partially complete prior to closing the existing bridge and beginning the 180 day closure window. The selected method was to erect the tied arch spans on steel falsework towers high above the river (at the approximate final elevation) supported on barges as seen in Figure 2.

Following fast track demolition of the existing bridge, the tied arch spans were floated into position and supported on the newly constructed bents. After a successful float in, concrete deck placement and other final construction activities could take place.

Figure 2 - Accelerated Bridge Construction

The selected accelerated bridge construction technique, briefly described above, required the evaluation of many components.

DEMOLITION OF EXISTING BRIDGE

The new bridge is located along a very nearly identical horizontal alignment with the existing bridge. Thus the existing bridge had to be demolished prior to construction of the new bridge, with a few exceptions. Prior to demolishing the existing bridge the new substructure could be partially constructed. The substructure layout for the tied arch spans were such that they did not interfere with the existing bridge substructure layout, so the new bents could be constructed up to an elevation that did not interfere with the existing bridge superstructure or demolition activities. An example of this can be seen in Figure 3.

Figure 3 - New Bent Construction

While the concrete deck and spandrel columns were removed with excavators, the steel through arch span and the three concrete spandrel arch spans were removed with explosives. This posed some risk to the new construction as there were new bents constructed below the existing bridge.

The explosive charges were set to pulverize the concrete arch spans into small enough rubble that with minor protective coverings the new bents below would not be damaged, but the new bent below the existing steel through arch span required some consideration.

The portion of the existing structural steel that was directly above the new bent had to be restrained to ensure it did not impact the partially-constructed bent below during the explosive demolition event.

After consideration of many alternatives, the decision was made to support the existing structural steel that was to remain intact following the explosive demolition with the new bent itself. A steel post was cast into the new bent and connected directly to the existing arch rib (just below the knuckle intersection with the girder). This can be seed in Figure 4. Note that it was not connected to the arch rib until after the existing bridge was no longer in service and just prior to the explosive blast.

Figure 4 - New Bent Protection

The explosive demolition caused a dynamic response of the remaining structure, as the supported load was instantly removed. The forces imparted during this dynamic response were analyzed and all structural components were evaluated. The steel arch span was successfully removed without any significant damage to the newly constructed substructure.

BARGE & FALSEWORK TOWER SYSTEM

The assembly and float-in of each span was performed using four 35′ x 195′ deck barges. Although all barges were the same overall dimensions, there were two specific types of barges used in the operation. The "900-Series" barges were 10′-6″ deep and had a specific internal structural system and the "Nugent Sand" barges were 10′-0″ deep and had a different internal structural system. The barges were originally owned by the Massman and had been previously used in other operations.

The construction plan developed in the project planning phase was to assemble the structural steel arch spans on towers supported by the barges so that the float-in operation would deliver the spans directly onto the permanent bearings without any vertical lifting. As such, custom designed and fabricated towers were utilized to support the steel arch spans on the barges. The height of the towers (and associated height of steel erection above the barges) varied between approximately 56′-68′.

The location of the supporting towers on the barges was determined based on the span float-in geometry conditions and the steel erection sequence. The towers were located directly under the first superstructure field segment to be erected for stability and temporary support conditions during assembly. The position and geometry of the new piers and footings limited the position of the barges relative to the new spans for the float-in condition as the barges had to clear the new footings during the span installation. These geometry conditions resulted in the towers being located near to the stern-end of the barges, resulting in eccentric load effects on the barges. Sequential ballasting was used to maintain vertical geometry of the arch structure during assembly. The barge and falsework tower layout can be seen in Figure 5.

Figure 5 - Barge & Falsework Layout

STEEL SUPPORT TOWERS – Steel towers were used to support the arch spans during erection and span float-in. Towers were required due to the bridge height for installation on the permanent bearings.

The towers were designed and constructed for modular assembly and future re-use by the Massman. The main column members were 24" x ½" round pipe members (ASTM A252 Gr. 3). Diagonal and k-frame angle bracing members were included in the tower modules for stability. Tower segments, as seen in Figure 6, with various lengths were detailed and fabricated for use on the project but also in consideration of future re-use.

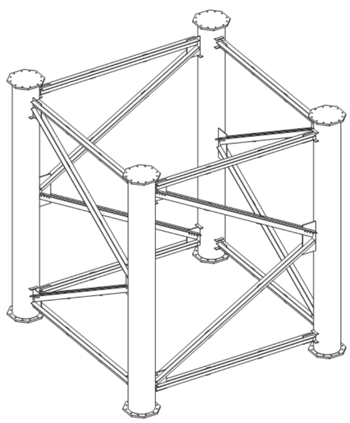

Figure 6 - Typical Tower Segment

The top of the towers included a conventional grillage system that directly supported the underside of the tie girders and consisted of W36 beam members. The base of the towers was supported on a lower grillage system that also consisted of W36 members and delivered load to the barge at the outer edges and at the center bulkhead. The upper and lower grillage systems can be seen in Figure 7.

Figure 7 - Falsework Grillage & Bracing System

BARGE STABILITY & BRACING - The magnitude and elevation of the supported structure during erection and span float-in required that the barges act compositely as a catamaran system. Although the four 35' x 195' barges provided adequate floatation to support the load, the barges individually did not provide the stability required for support of the system. As such, a system of struts and braces were implemented to create a composite system for two parallel barges. This system was developed to force each pair of parallel barges to act together as in response to lateral loads such as wind, river current, impact, and eccentrically-applied vertical loads. The resulting system essentially transformed two individual parallel 35'-wide barges into a system that was 122'-6" wide with two 35'-wide pontoons.

The struts between the parallel barge towers were 36" diameter steel pipes, and diagonal bracing was wire rope. The tower bracing can be seen in Figure 7. These elements were sized to develop strength and stiffness required to develop the composite catamaran barge system, subject to appropriate load effects anticipated during construction.

LOCAL BARGE REINFORCING - Although the deck barges were adequate for systematic support of the applied loads and for system stability, they were not originally constructed anticipating heavy concentrated loads. As such, support of the steel towers on the deck barges required the barges to be reinforced in the local areas of direct contact.

Engineering efforts for design of the local barge strengthening began with the performance of a detailed inspection of each barge type. As noted previously, the "900-Series" barges and the "Nugent Sand" barges each had unique geometry and internal structure so each set of barges had to be studied independently. The inspections revealed that the barges were in generally good condition and were suitable for use on the project with required reinforcing.

Design details for local reinforcing were developed through the use of three-dimensional shell finite element models that represented the local barge internal structural elements. Design loads were applied to the models representing the effects of tower support and structural results were developed. Local reinforcing details were developed to limit local stresses and achieve acceptable stiffness.

Barge local reinforcing consisted of vertical column members, stiffeners and bearing plates welded to the main bulkhead plates. Vertical column members and bearing plates provided a direct load path for support of applied loads into the main barge steel grillage. Stiffeners increased the load-carrying capacity of the main structural elements in the barge.

All local barge reinforcing details were made prior to installing falsework towers.

STEEL ERECTION

The final bridge design performed by HNTB considered the tied arch span fully constructed supported in the permanent condition for all applicable load effects. As the bridge is constructed one field segment at a time, while supported on temporary falsework on barges, there are many temporary conditions that were not evaluated in final design that had to be considered. The temporary conditions during construction had the potential to induce demands on the structure that exceed those considered during final design. All structural members installed had to be evaluated for each stage of construction.

It is evident structural demands are stage specific, but it is important to also consider that member capacity can vary with construction stage. As member loading condition and unbraced length vary throughout construction, member capacity can be drastically different from one stage to the next.

The construction sequence will be described by relating field segments installed to the field splices at each end. The field splices are labeled in Figure 8 below for reference.

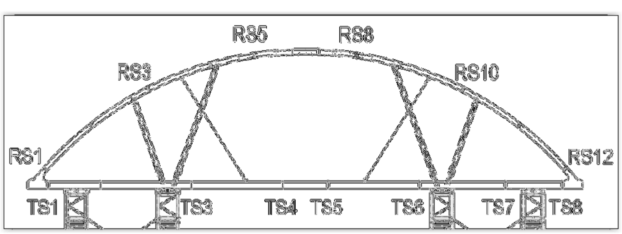

Figure 8 - Field Splice Layout

CONSTRUCTION SEQUENCE NO. 1 - The floor-system located between field splices TS1 & TS3 was installed. The tie girders and associated floorbeams and stringers were assembled on the ground and lifted into position on top of the falsework. This can be seen in Figure 9. This was termed the "mega lift", as it was the heaviest lift for the project weighing in at approximately 625,000 pounds. The lifting and rigging evaluation required to perform this operation (and other lifting operations to follow) will be discussed in a following section.


Figure 9 - Sequence 1

CONSTRUCTION SEQUENCE NO. 2 - The knuckle (intersection of arch and tie girder) and associated floorbeams and stringers were installed. This can be seen in Figure 10. It is clear the structural adequacy of the tie girder must be considered for the large cantilevered portion of the floor-system beyond the falsework support. As noted previously all construction stages were evaluated for adequacy.

Figure 10 - Sequence 2

CONSTRUCTION SEQUENCE NO. 3 - The arch members located between field splices RS1 & RS3 were installed. The arch segments and associated lateral bracing members are being positioned as seen in Figure 11. The extreme cantilevered condition of the arch necessitated additional support to reduce the bending demands to within the allowable limit. The use of an arch support strut was required and will be further discussed in construction sequence 4.

To hold the arch support struts in the proper position to receive the arch segment, a strut erection frame was used. This strut erection frame can be seen in Figure 11 to Figure 16 This was required to hold the strut in the proper alignment prior to connection with the arch member.

Figure 11 - Sequence 3

CONSTRUCTION SEQUENCE NO. 4 - The next adjoining arch members between field splices RS3 & RS5 were installed. The arch segments and associated lateral bracing members are being positioned as seen in Figure 12. As with the previous arch segment installed, the cantilevered condition necessitated additional support to reduce the bending demands.

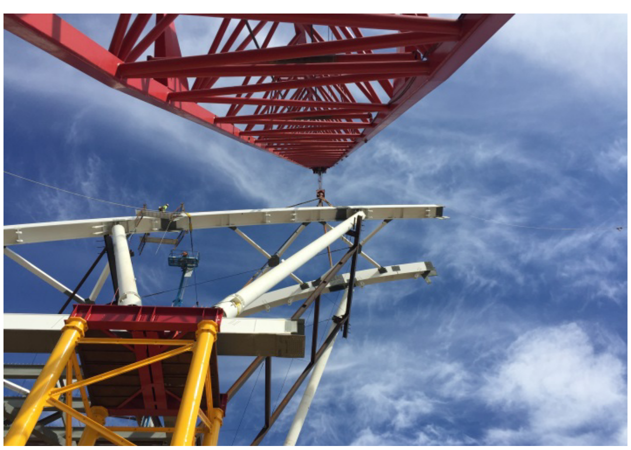

Figure 12 - Sequence 4

There were many options considered for the potential means of providing the required additional arch support, but eventually two (at each of the four corners) 36" diameter pipes in the plane of the arch were selected. The struts were positioned such that they were able to react directly against the falsework. This reduced the demands required of the floor-system.

Disktron bearings, supplied by R.J. Watson, were installed at the top and bottom of the arch support strut to allow minor rotations to occur during installation. With a hard connection to the permanent structure, any deviation from predicted geometry would have proven difficult to install.

Notice that both struts (at each corner of the arch) were positioned to react at the same falsework tower location. This naturally increased the reaction to this falsework tower as compared to the other falsework tower. The falsework tower selected for strut support was more centrally located on the barge. This allowed for less ballasting required to keep the barge level, which resulted in more freeboard available for the float in operations.

CONSTRUCTION SEQUENCE NO. 5 - The adjoining floor-system members between field splices TS3 & TS4 were installed. The tie girder and associated floorbeam and stringer members are being positioned as seen in Figure 13.

Figure 13 - Sequence 5

Without additional support, the tie girder demand, due to the extreme cantilevered condition of the floor-system, would have exceeded capacity. In order to avoid this condition, a temporary cable was connected from the arch (between struts) to the cantilevered end of the tie girder to support the floor-system beyond the falsework (installed prior to release from crane support).

It was desired to use the existing pin connection plates on the arch and tie girder (for the permanent cables) to provide attachment of the temporary cable that was to provide the required support of the floor-system. With the use of the permanent pin connection plates the cable would interfere with the arch support strut. To avoid interference with the strut and still make use of the existing pin plates, a custom bracket was fabricated as seen in Figure 14. The temporary cables can also be seen in Figure 15.

A 1¾" diameter XIP wire rope with an open bridge socket was selected and can be seen in Figure 14. This socket type is more easily adjustable than the permanent cable sockets (to be discussed in a following section). The temporary hanger and bracket were designed for the extreme loading condition experienced during construction.

Figure 14 – Temporary Hanger Bracket & Adjustable Lower Socket

CONSTRUCTION SEQUENCE NO. 6 - The floor-system closure segment between field splices TS4 & TS5 was installed. This can be seen in Figure 15. At this time, the two barges supporting each half of the arch were no longer fully independent.

Figure 15 - Sequence 6

CONSTRUCTION SEQUENCE NO. 7 - The arch closure segments between field splices RS5 & RS8 were installed. For the first time, the arch began to behave like an arch. There is now axial compression in addition to the bending stresses that have controlled the design until this stage. This condition can be seen in Figure 16.

As each corner of the arch is supported by a unique barge (four total per span) it is effectively supported by a variable platform. The elevation of the arch at the four field splices (RS5 & RS8), awaiting the arch closure segment, could potentially be at slightly different elevations. As the arch closure segment was installed it was necessary to have the ability to alter the position of the previously installed segments. To achieve this adjustability, the arch support strut was designed to be telescopic. The necessity for the arch support strut to be telescopic will be further discussed in a section to follow.

Figure 16 - Sequence 7

Lifting and Rigging - Each lifted segment, as described in construction sequences 1 - 7, were lifted with a single barge mounted ringer crane using a 4 point pick. To determine the demands to the crane, slings, spreader bars, etc., a detailed weight take off and geometry study of the lifted segment was performed.

When under crane support the lifted segment would rotate until the center-of-gravity of the lifted segment is directly below the hook. After the installation of the first segment (Figure 9), each successive field segment was required to be directly connected to those that were previously installed. To make the field splice connections possible, the orientation of the crane supported segment must match that of the previously installed segment. Thus, not only is a detailed weight take-off required, but a precise center-of-gravity study was required to ensure the lifted segment would naturally swing into the desired orientation.

With the total weight and center-of-gravity known, the sling requirements (force, length, angle) can be defined. As the angle from vertical to the sling is reduced, the sling force is reduced, but the required length of sling and also the required height of crane boom increases. Thus, a balance must be maintained.

The connection to each lifted segment made use of existing bolt holes already present in the structural steel. This included holes for field splices, or floorbeam to tie girder connections, or upper lateral bracing connections in the arch. As these existing holes were not necessarily symmetric about the pick center-of-gravity, the sling lengths, angles, and forces had the potential to vary within a given pick.

Not only were the adequacy of crane and the lifting and rigging components considered, but the structural steel adequacy was also considered. As each field segment was crane supported, the unique loading condition had to be evaluated. The member capacity associated with each braced condition (while crane supported) was then compared to demands.

CONSTRUCTION SEQUENCE NO. 8 – Select permanent hangers are installed to a specified tension. In addition to the specified installation force for each hanger, an associated lower adjustable socket gap dimension was specified. The specified force and gap dimension acted as the guide during hanger installation. Hangers were adjusted to force and the resulting gap dimension was then compared to the theoretically predicted value.

The only disadvantage to the selected position of the arch support strut in the plane of the arch was the interference with the network of cables. All permanent cables that interfered could not be installed until the struts were removed, which could not occur in until after the falsework was removed. Therefore, the permanent cables that interfered with the struts could not be installed until the arch was supported on the permanent bearings (following float-in). This resulted in the installation of 14 of the 22 total permanent hangers.

In addition to reducing the structural demands in the arch during erection, the struts also reduced the demands in the tie girder. As the permanent hangers are installed, the arch begins to pick up more load. The struts provided a load path for the axial compressive load in the arch to the falsework. Without the struts, this axial compressive force would be forced to travel through the knuckle and be transferred to the falsework through the flexural resistance of the cantilevered tie girder. The tie girder demands for this condition would have exceeded allowable limits. Thus, the arch support struts were also required to reduce the bending demands in the tie girder during construction.

The specified installation force was defined to keep the structural demands within acceptable limits. This included the arch and tie girder but also the hangers themselves. The analysis model considered separate stages for the installation of each individual cable, and every structural element for every stage was evaluated for adequacy.

Following installation of the initial 14 permanent hangers the temporary hangers were removed. The floor-system was now adequately supported and the temporary hanger was no longer required.

Permanent Hangers - There are two 2 3/8” diameter A586 strands at each permanent hanger location. There are 22 permanent hanger locations on each side of the arch (44 total), therefore a total of 88 cables are required per arch. Each cable is pin connected to the arch with an open prolite socket and pin connected to the tie girder with an adjustable prolite socket. The lower socket provides approximately +/- 6″ of length adjustability and can be seen in Figure 17.

Figure 17 - Lower Hanger Socket

A detailed length calculation was required for each hanger. The anticipated final deflections from the staged erection analysis were superimposed with the fabricated cambered geometry of the structural steel (as defined in the original bid documents) to define the stressed hanger length. To define the hanger cut length, the associated final hanger force was also specified. The hanger length was cut to target a 10″ gap in the lower adjustable socket to provide the maximum amount of adjustability in either direction.

To increase the tension in the hanger the length of the hanger is reduced by bringing the two halves of the lower adjustable socket together. This gap dimension is adjusted by turning a threaded rod further into the lower socket body. The threaded rod is reverse threaded so that it penetrates into each of half of the lower socket as it is turned.

As the tension in the hanger increases, so does the tension in the threaded rod. The ability to turn the threaded rod (to adjust the hanger force) under load is not possible, therefore the load in the threaded rods must be practically eliminated when turning the rod. A tension bracket was designed to unload the threaded rod as seen in Figure 18.

The tension bracket used four 60ton center hole jacks to transfer the hanger force directly from the upper to the lower half of the adjustable socket. In this manner, all load bypasses the tension rod and allowed the rod to turned. This results in alteration of the length and therefore tension in the hanger. The tension bracket was required for the installation of all hangers.

Figure 18 - Tension Bracket

FLOAT-IN OPERATIONS

CONSTRUCTION SEQUENCE NO. 9 - The preparation for float-in operations could now occur. Miscellaneous tasks such as the installation of the stay-in-place forms and a temporary work platform along each tie girder were installed.

CONSTRUCTION SEQUENCE NO. 10 - The arch spans were floated into position. The arch is transferred from the falsework (on barges) to the permanent bearings on the newly constructed bents. The first arch span during transport can be seen in Figure 19.

Figure 19 - Sequence 10

Weighing in at more than 4,000,000 pounds and with a center-of-gravity nearly 100′ above the river, the transportation of the assembled arch to its final resting place was a delicate operation. Any sudden acceleration or deceleration of the barge support system (such as by barge impact with an underwater obstruction) would have a resulted in a large inertia load to be resisted by the cable bracing between falsework towers. For design, this impact loading effect was assumed to be 10% of the weight of the arch to act in any the orthogonal directions.

Traveling less than 1200 feet to the final position, a series of winch lines were installed to assist multiple tug boats to pull/push each arch into position. With the use of both the winch lines and tug boats the barge supported arch was slowly maneuvered into position without incident.

As the bridge was floated into position the elevation of the knuckle had to clear the bearings atop the newly constructed bents. Specifically, a target clearance of 6″ to 12″ was selected. Without clearance above the bearings the bridge would not have been able to float into position, but too much clearance could also prove problematic as well.

It was estimated that under the full weight of the tied arch span, falsework towers, and temporary work platforms, the barges would have approximately 2.5′ of freeboard remaining. Once situated hovering directly above the permanent bearings the bridge had to be lowered into position. To set the bridge down, a ballasting plan was defined to lower the barges by flooding specific tanks in the barge. It was estimated that once the initial touch-down occurred the barge would be required to drop an additional 6″. This accounted for structural displacement of the arch at the falsework location and providing clearance between the top of the falsework and bottom of the tie girder once the transfer has occurred. With 12″ of clearance and 6″ of structural displacement, it was estimated that the barge would require a ballasting plan to drop 18″. Starting with 2.5′ of freeboard, there would only be 1.0′ of freeboard remaining. Thus, a maximum target clearance was selected to ensure there would be remaining freeboard following the successful transfer of arch support from the falsework to the permanent bearings. The knuckle clearance above the permanent bearing during float-in operations can be seen in Figure 20.

To define the falsework height, many factors had to be considered to assure the desired clearance was achieved. This included considerations such as the river elevation anticipated during float-in, the expected barge freeboard while under the full weight of the supported load, barge ballasting requirements to maintain level conditions, the tip deflection at the cantilevered knuckle, and the unique pedestal elevations at each bent.

Figure 20 - Knuckle Clearance above Bearing

In addition to reducing structural demands, the arch support strut also reduced deflections at the tip of the cantilevered tie girder. This deflection would have increased without the struts providing the direct load path to falsework. This allowed for a reduced falsework height required to provide the target bearing clearance during float-in. This allowed for less ballasting required, and more freeboard remaining after touch-down on the permanent bearings.

CONSTRUCTION SEQUENCE NO. 11 - With the arch fully supported on the permanent bearings the barge and falsework could simply be floated out from under the arch.

FINAL CONSTRUCTION ACTIVITIES

CONSTRUCTION SEQUENCE NO. 12 - The arch was no longer being supported on falsework, therefore the struts are no longer required. In addition to geometric control, as discussed in construction sequence 7, the telescopic abilities allowed for easy loading or unloading of the strut as required during construction. To unload the struts the two 200 ton jacks, located within the telescopic end of each strut, were simply engaged and shim plates were removed. This allowed the struts to be unloaded in a controlled manor without risk to personnel performing the field operations. The telescopic end of the strut with window opening for jack access can be seen in Figure 21.

Figure 21 – Telescopic Strut

With removal of the support strut the remaining 8 cables, which previously would have interfered with the support struts, were installed. As with the initial round of hanger installation (construction sequence 8), these hangers were installed to a specified force and the resulting lower adjustable socket gap dimension was compared to the theoretically predicted value.

CONSTRUCTION SEQUENCE NO. 13 - At this time the deck was installed according a specified pouring sequence. The sequence was selected to reduce the potential of unloading hangers, while considering the structural adequacy each temporary condition.

An extensive survey of the floor-system was performed prior to the deck pouring operations to define the haunch dimension at the quarter point of each stringer the full length of the bridge. This survey was to account for steel fabrication tolerance and also any minor variation in actual verses predicted structural deflections. This was required to ensure that the deck thickness was as intended.

CONSTRUCTION SEQUENCE NO. 14 - The force in all 22 hangers (88 actual hangers) were adjusted. As with hanger installation, each cable was adjusted to a specified force with an associated gap dimension. The adjustment was intended to match final target geometry (tie girder and arch deflections) and hanger forces as specified in the contract documents. Each hanger adjustment considered the fact that the adjustment of the hangers to follow would affect the adjusted force. The analysis model considered separate stages for the adjustment of each cable.

SUMMARY

The successful float-in of Span 5 was completed on November 15th, 2016 and the successful float-in of Span 4 was completed on December 2nd, 2016. After being closed for 152 days, the Broadway Bridge was officially opened to traffic on March 1, 2016, which was 28 days ahead of schedule.

The two network tied arch spans supported in their permanent position and open to traffic can be seen from various vantage points as seen in Figure 22 and Figure 23.

Figure 22 - Final Bridge

Figure 23 - Final Bridge

ACKNOWLEDGEMENT

The author would like to recognize, HNTB, Garver, and the Arkansas State Highway and Transportation Department for their involvement during the construction review process.