CONSTRUCTION STRESSES IN THE WORLD'S FIRST PRECAST NETWORK ARCH BRIDGE

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ABSTRACT

This paper reports some of the major findings from the instrumentation of the West 7th Street Bridge in Ft Worth, Texas. This innovative bridge, completed in 2013 as a replacement for a century-old bridge, consists of 12 prestressed, precast network arches. The arches were instrumented with 224 vibrating wire gages that were embedded in the arches prior to concrete placement. The gages were monitored during post-tensioning, handling, and transportation operations as well as deck construction. The instrumentation provided valuable data on the stresses induced in the arches, which were used to ensure a safe environment throughout construction. The measurements also provided a means for evaluating the accuracy of assumptions that were made during design. The results obtained from this study provide a unique insight into the behavior of concrete arches built by an accelerated construction method.

Keywords: Arch bridge, Network arch, Precast, Post-tensioned, Monitoring, Accelerated bridge construction

INTRODUCTION

Arches provide a structural system that can efficiently support large loads while also lending themselves to excellent aesthetics. Historically, arches have been widely used in bridge systems; however, in modern applications, they are usually reserved for signature bridges, where aesthetics play an important role in the design. Arches primarily resist loads through compression, and as a result, concrete is an ideal structural material for their application.

While advances in construction techniques and analytical capabilities have systematically improved in recent years, few concrete arch bridges were built in the past fifty years[1], mostly due to their high construction costs. Concrete arches are usually built using timber or steel falsework or cantilever methods, which are time and labor intensive[2]. As a result, improving construction techniques for this efficient, aesthetic structural form has been of special interest to the structural engineering profession.

An innovative solution for constructing concrete arches was used by the Texas Department of Transportation (TxDOT) for a signature bridge on the West 7th Street Bridge in Ft Worth, Texas. The design utilized twelve 280-ton network tied arches that were precast on their sides, then rotated into a vertical position, transported, and installed in their final locations. To make the substantial stresses induced by rotation and transportation tolerable, the arches were prestressed in both the tie and the rib. This method reduced the on-site construction time and allowed better quality control. However, the designers predicted that some of the most critical times in the life of the arches happened during construction. As a result, accurately estimating the stresses during construction was critical. While sophisticated finite element models were used to predict the stress levels in the structure, the possibility of damaging the arches during construction remained a concern.

Since the designers implemented strut-and-tie modeling techniques developed in previous TxDOT sponsored research, a field monitoring study at The University of Texas at Austin (UT) was funded to evaluate the performance of the arches during construction. As part of field monitoring, the arches were instrumented and data were collected and interpreted to ensure the safety of the arches and verify the design assumptions.

This article presents some of the findings from the instrumentation of the West 7th Street Bridge with a focus on handling stresses. A brief overview of the innovative design of the bridge is presented. The instrumentation, monitoring, and interpretation of data are then described. Finally, comparisons are made between the stresses measured in the structure and those predicted in design calculations.

THE WEST 7TH STREET BRIDGE

The new West 7th Street Bridge was designed to replace a century-old city bridge that connected downtown Fort Worth to the Cultural District. As can be seen in Fig. 1, the bridge spanned over four lanes of traffic, the Clear Fork of the Trinity River, and a number of recreational trails.

Fig. 1- The old West 7th St Bridge (Bing Maps)

The aesthetics of the replacement bridge was of special importance to the city officials. The majority of new bridges in Texas are precast concrete girder bridges, which could also provide an economical solution for this project. However, the new West 7th Street bridge was expected to be a signature bridge and a pleasant gateway to five internationally renowned museums in the Cultural District. On the other hand, due to large traffic demands on the bridge, the new bridge needed to be constructed as quickly as possible. The site conditions allowed TxDOT engineers to use six uniform spans of 163.5 ft. Therefore, they conceived an innovative solution consisting of 12 identical precast, prestressed concrete network arches. The geometry of the precast arches is shown in Fig. 2. While the decision to use arches was highly influenced by aesthetic considerations, the identical design of all arches and the possibility of precasting resulted in a significant reduction in construction costs and the time of street closure and made precast arches a feasible solution for this project.

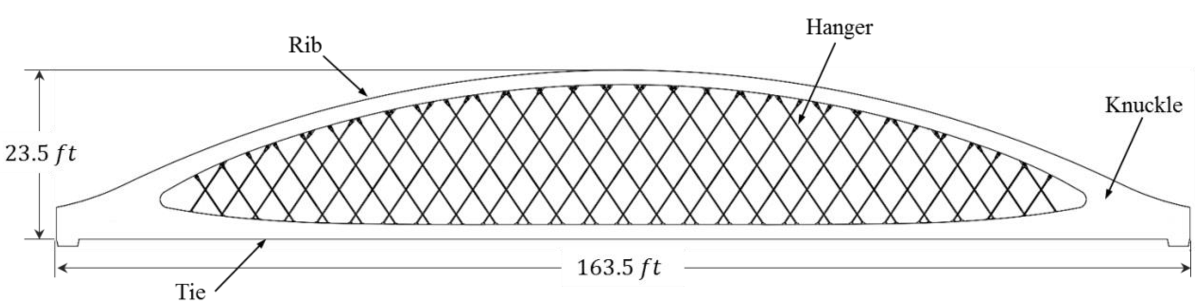


Fig. 2-Geometry of a network arch in the West 7th Street Bridge

By definition, a network arch is a tied arch bridge with inclined hangers, in which most hangers cross at least two other hangers in the plane of the arches. The inclined hangers provide a nearly continuous shear transfer between the rib and the tie, and therefore greatly reduce the bending moments in the arch elements[3]. Due to significant material savings, steel network arches have been gaining popularity. However, the West 7th Street Bridge is believed to be the first precast concrete network arch bridge in the world[4].

Fig. 3 shows the details of a typical span of the new bridge. Each span is supported by two arches, which are located on either side of the roadway. The deck is constructed using precast panels with a cast-in-place topping slab and is supported by 17 prestressed floor beams. The floor beams are suspended from the arches using post-tensioned bars. The bridge carries four lanes of traffic and two sidewalks that are located outside the arches.

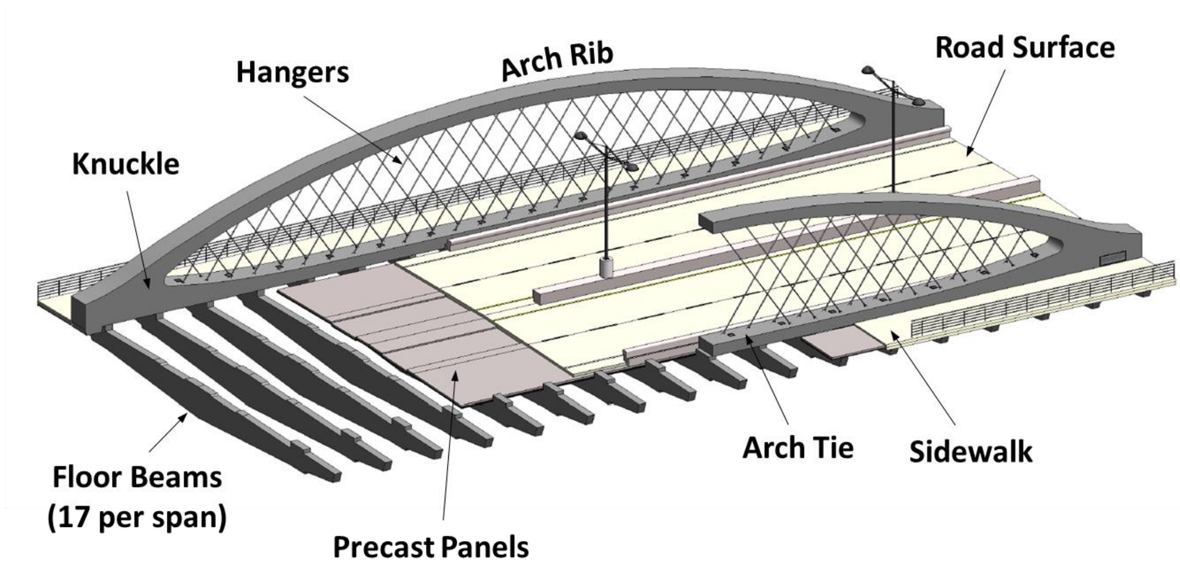


Fig. 3- Details of a typical span in the West 7th Street Bridge
(Image Courtesy of Joel Blok)

The construction of the precast arches was carried out in a casting yard located less than one mile away from the bridge location. The steps for constructing the arches of the West 7th Street Bridge were as follows.

1- The arches were cast on their sides.

2- To prevent cracking during rotation, a first stage post-tensioning was conducted on the arches: two tendons in the rib were stressed to 208 ksi and four tendons in the tie were stressed to 104 ksi. Each tendon consisted of nineteen 0.62-inch strands.

3- The hanger elements were installed. Each hanger was passed through a hanger tube in the tie and was threaded into a clevis at the rib. A nut was put at the other end of the hanger and was hand-tightened before the rotation of the arch.

4- The arches were rotated into a vertical position using a lifting assembly in which six lifting frames were attached to the rib and the tie and were lifted by a massive gantry system. After rotation, the arches were set on temporary supports. The rotation operation for one of the arches is shown in Fig. 4.

5- Since the small gap between the arches in their final position did not allow any post-tensioning operation, all the stressing operations needed to be completed in the precast yard. Therefore, a second stage post-tensioning was carried out, in which the tie tendons were stressed to 208 ksi, and the rib tendons were de-tensioned to 104 ksi.

Fig. 4- Arch rotation

6- In order to prestress the hangers, an upward jacking operation was conducted, which is shown in Fig. 5 (a). A series of hydraulic rams were positioned under the tie at the locations of future floor beams and were simultaneously activated to push the tie upward. When the rams were active, the sag was removed from the hangers and as shown in Fig. 5 (b), the nuts were re-tightened. The rams were then deactivated. As a result, the self-weight of the tie induced a prestress in the hangers. To stiffen the ribs near the knuckle region and prevent excessive tensile stresses during upward jacking operation, prestressed concrete strong-backs, which are visible in Fig. 5 (a), were clamped to the rib before upward jacking. These strong-backs remained attached to the arches until all floor beams were installed in each span.

7- Once all arches and new piers were constructed, the arches were moved from the precast yard to the new piers. As can be seen in Fig. 6, two self-propelled modular transporters carried each arch from the precast yard to their final location, where the arches could be lifted by cranes and installed on bearings.

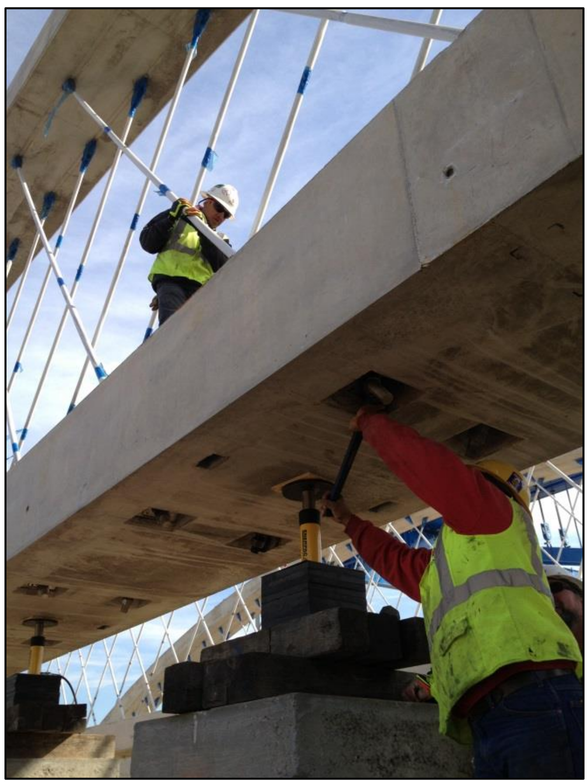
8- When all arches were transported to their final locations and properly braced, the street was closed, and the old bridge was demolished. The floor beams were then installed, and the construction of the deck for the new bridge began immediately to minimize traffic interruption.

This accelerated construction procedure served to limit the time of street closure to 120 days. However, since the arches experienced several post-tensioning and handling operations, concerns were raised regarding the safety of arches during construction, especially against cracking. Potential cracking of the arch rib during construction could be detrimental to the stability of the arches in the finished bridge and therefore, significant efforts were taken during design to lower the likelihood of cracking.

In order to ensure the satisfactory performance of the arches during construction and under service conditions, the design team needed to make careful decisions regarding the area and arrangement of tendons in the rib and the tie. The rib is inherently a compressive element. However, the arches of the West 7th Street Bridge were cast in a horizontal position, and their ribs were not subjected to compressive stresses until the arches were rotated into the vertical orientation. Therefore, excessive tensile stresses in the rib were likely during rotation. Moreover, the designers tried to minimize the weight of the precast arches to facilitate the rotation and handling operations. As a result, even after rotation, the self-weight of the arches was not sufficient to ensure the ribs remained completely in compression during the remaining construction operations. Therefore, the design team chose to prestress the rib elements to minimize the risk of cracking. On the other hand, the tie element transfers the horizontal thrust between the supports through tension, and a relatively large prestress was needed to prevent cracking in the tie when the bridge is subjected to service load conditions. However, the self-weight of the arches generated a small portion of the service load tension in the tie, and as a result, the tie element was subjected to a significant compressive force during the second stage post tensioning. To avoid potential instability in the tie element, the design included a series of small curves in the duct paths so that the tendons would be in contact with the wall of the ducts after a very small lateral displacement[4]. As a result, the second order displacement of the tie element was minimized.

TxDOT engineers used sophisticated nonlinear FE models of the structure for stress calculations. However, since the structure was the first of its kind, some uncertainties remained about the accuracy of predicted stresses in the structure. Ensuring that the arches were not experiencing excessive tensile stresses was possible through the instrumentation program highlighted herein.

(a)



(b)

Fig. 5- Upward jacking operation (a) Arrangement of hydraulic rams and the strong-backs (b) Re-tightening the hanger nuts when the rams were activated


Fig. 6-Arch transportation

INSTRUMENTATION

The West 7th Street Bridge was instrumented using 224 Geokon Model 4200 Vibrating Wire Gages (VWGs). This model of VWG includes a strain transducer and a thermistor, which allow for both strain and temperature measurements with resolutions of 1 Microstrain and 1 degree F, respectively. Fig. 7 shows a VWG installed in the reinforcing cage of one of the arches prior to concrete placement.

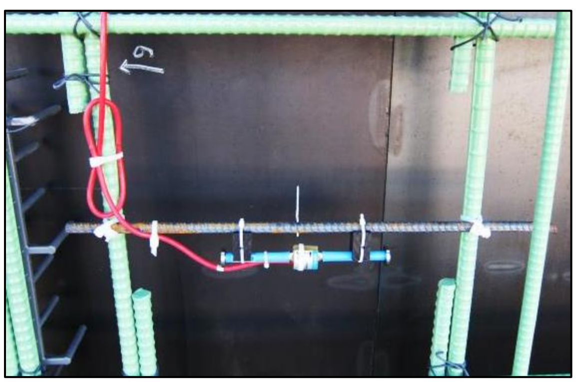


Fig. 7- A VWG tied to the arch reinforcement

The instrumented sections of the structure were selected in coordination with the design team. As shown in Fig. 8, the first two arches were instrumented at every section where large demands were predicted by the designers so that the safety of the arches could be ensured during construction. Following the successful construction of the first two arches, the number of instrumented sections was gradually reduced for subsequent arches.

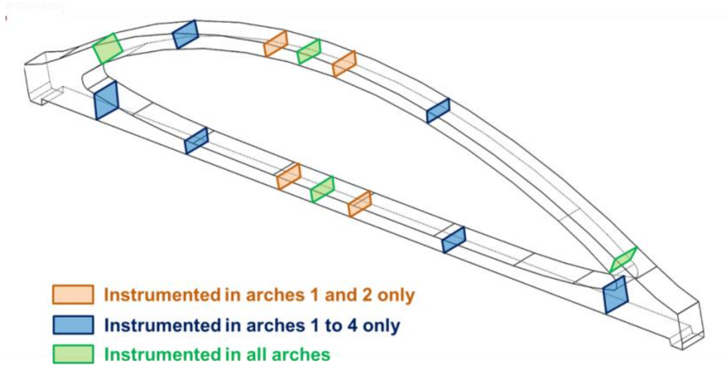


Fig. 8-Instrumented sections in the arches

The VWGs were installed immediately prior to the assembly of the outside forms for each arch. As can be seen in Fig. 7, each VWG was attached to a #3 steel rebar, which was tied to the transverse reinforcement of the arch. Fig. 9 shows the location of the individual VWGs embedded in the first two arches. As shown in this figure, regions of the structure with expected linear strain distributions were instrumented using three or four VWGs. As a result, the stresses at every corner of these sections could be found using the plane section assumption. However, for the knuckle region, the strain profile was expected to be highly nonlinear and the VWGs were expected to represent the local strains. Therefore, in these sections only two VWGs were installed in the anticipated locations of maximum stresses.

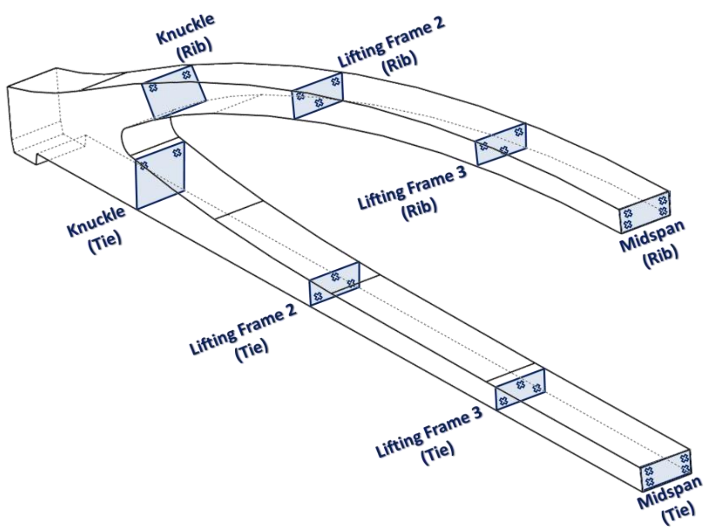


Fig. 9- Arrangement of VWGs in different sections of the arches

To improve the flexibility of monitoring operations for the highly mobile arches, a wireless data acquisition network was used, as shown in Fig. 10. The wires from the embedded VWGs were connected to a data collection box, which received the data and then sent them to the data acquisition system through wireless communication. The wireless connectivity not only eliminated the lengthy wires on the construction site, but also reduced the number of channels needed on the data loggers. The data acquisition system was also connected to a cellular modem, enabling remote monitoring of the structure.

The arches were monitored during the construction until the bridge was opened to traffic. Depending on the speed of the construction activities, different scan rates were used for monitoring the VWGs. The maximum possible scan rate for a single VWG with the available interface analyzers was once every 2 seconds; however, the sensors were scanned sequentially, so the scan rate was reduced. To achieve a suitable scan rate, more data loggers were added to the network when construction operations were being performed on several arches simultaneously. The resulting configuration allowed the researchers to scan the gages once every 150 seconds, which made it possible to detect the changes during rapid construction operations such as post-tensioning. Scanning was also continued at a rate of once per hour when no construction activity was in progress to capture the impact of temperature fluctuations on the structural behavior.

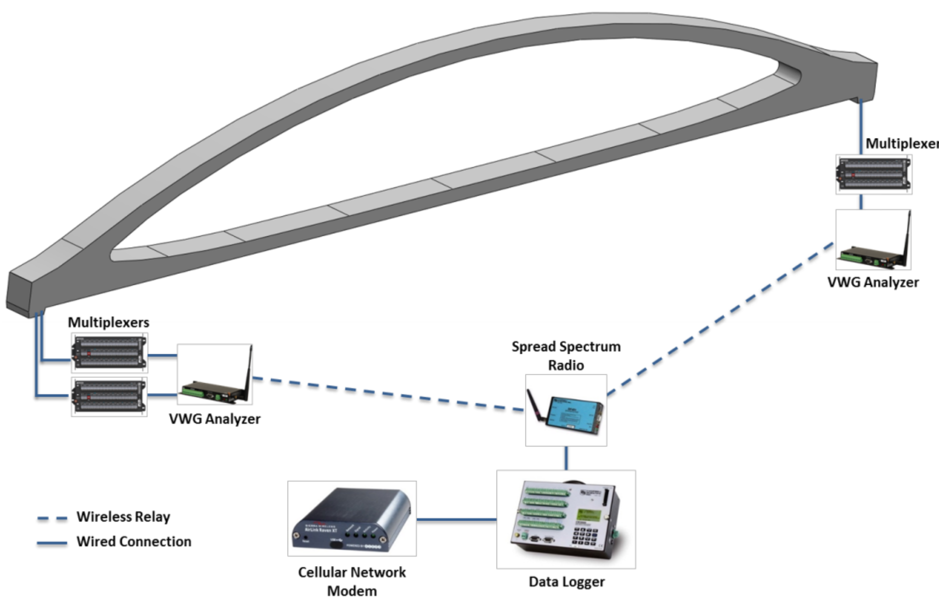


Fig. 10- The configuration of the data acquisition network

ESTIMATING THE MECHANICAL PROPERTIES OF CONCRETE

Realistic values of the modulus of elasticity and compressive strength of concrete were essential in post-processing of the data. Therefore, a material test program was conducted in the Ferguson Structural Engineering Laboratory (FSEL) to measure the modulus of elasticity (E_c) and compressive strength (f_c') of the concrete used in the arches. The results of these tests were used to develop Equation (1), which is a mix-specific equation representing the relationship between E_c and f_c'.

$$E = 39\sqrt{f_c'} + 1350 \qquad (1)$$

In Equation (1), E is the modulus of elasticity of concrete in ksi, and f_c' is the compressive strength of concrete in psi.

To consider the effects of different curing temperatures in the structure as compared to test cylinders, the in-situ E values were estimated based on the compressive strengths obtained from a maturity method and Equation (1). The authors used the results of a maturity study conducted by the contractor to correlate the compressive strength with the maturity for individual arches. The thermal history of the concrete, as recorded by the instrumentation in the structure, was used to determine the maturity at each age. Consequently, the in-situ compressive strength was found as a function of age, as given in Equation (2), and was combined with Equation (1) to estimate the in-situ E values at each age.

$$f_c'(t) = 7500\left(\frac{t^{0.75}}{1.75 + 0.8t^{0.75}}\right) \qquad (2)$$

In Equation (2), f_c' is the compressive strength of the concrete in psi, and t is the concrete age after casting, in days.

POST-PROCESSING OF DATA

A considerable post-processing effort was needed to interpret the data obtained from the instrumentation. The raw data included the strains in the VWGs and the temperatures at the location of these sensors, which were used to calculate the stresses at the corners of the instrumented cross sections in the structure.

The first step in stress calculations was to calculate the strains at the corners of the instrumented cross section. For sections in non-disturbed regions of the arches, plane sections were assumed to remain plane, and strains at any point in the cross section could be calculated using analytic geometry. Due to highly nonlinear distribution of strains in the disturbed regions of the arches such as the knuckle region, the strains could be calculated only at the locations of the VWGs.

The measured strains included several time-dependent and environmental effects, including temperature changes, creep, and shrinkage. These components needed to be excluded from the strain history so that stress-related strains could be multiplied by the modulus of elasticity of the concrete to calculate the stresses.

To minimize the effects of creep and shrinkage in the calculated stresses, the strain changes due to each construction stage was calculated separately and multiplied by the corresponding modulus of elasticity at the time of that construction operation. The total stresses were estimated by adding the stress increments due to each construction operation, as expressed in Equation (3).

$$\sigma(t) = \sum_{i=1}^{n} E_i \Delta\varepsilon_i \tag{3}$$

In Equation (3), $\sigma(t)$ is the stress in the concrete, $\Delta\varepsilon_i$ is the strain change in the concrete due to the i^{th} construction operation, and E_i is the modulus of elasticity of concrete at the time of the operation.

This method neglects the stress changes that occur when no construction activity is in progress. For example, the effects of prestress losses remain undetected by this method. To include the effects of long-term changes on stresses in the arches, the authors used a more sophisticated post-processing method, which incorporates the time-dependent deformations of concrete in stress calculations. However, the effects mentioned above will be covered in a later paper and are expected to have a minimal effect on the short-term stress changes that are the focus of the present paper.

In order to eliminate the effects of thermal changes on calculated stresses, the structure was compared always between data points with equal temperatures. During stepwise construction operations such as post-tensioning, the stress changes due to stressing of individual tendons could be calculated because thermal changes during stressing of each tendon were negligible. However, for slower operations such as arch rotation or transportation operations, stress changes were obtained by comparing the readings taken in the nights before and after the specific event. Temperature effects were minimized by identifying times in the nighttime hours before and after the event with equal temperatures.

RESULTS AND DISCUSSION

POST-TENSIONING RESPONSE

There was some variation in the behavior of a few of the twelve arches. However, in general, relatively similar behavior was observed among the arches. Although the focus of the results presented in this section target the behavior of one of the arches, the data is representative of observations in the other arches as well. Fig. 11 and Fig. 12 show a number of typical stress records during post-tensioning of the arches. As can be seen in these figures, the instrumentation was capable of detecting the stressing of individual tendons inside the rib and the tie. As a result, the recorded data provided the opportunity to evaluate the response of the structure and to check the analysis models of the bridge with respect to each tendon separately. Moreover, the interaction of rib and tie elements could be evaluated by comparing the stress changes in the rib while the tie was being post-tensioned, and vice versa.

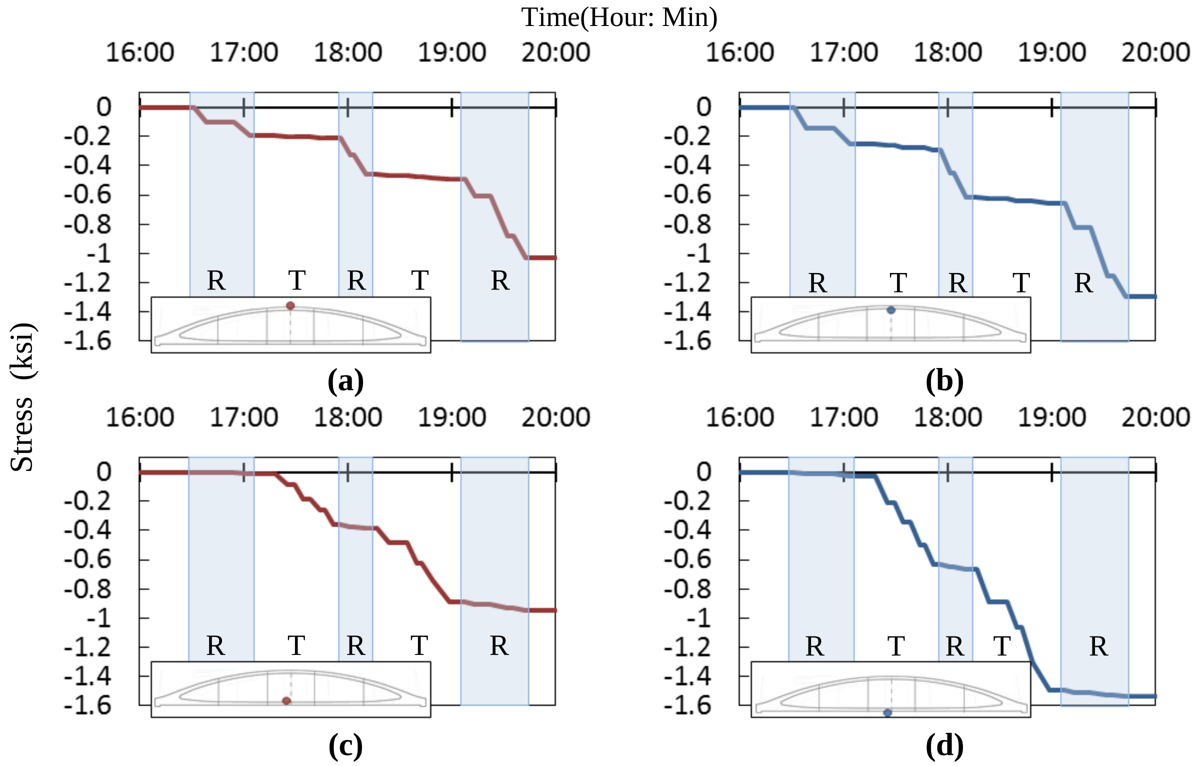


Fig. 11- Stresses in Arch 2 during Stage 1 PT (R: prestressing the rib, T:prestressing the tie)
(a) Midspan, rib, top (b) Midspan, rib, bottom (c) Midspan, tie, top (d) Midspan, tie, bottom

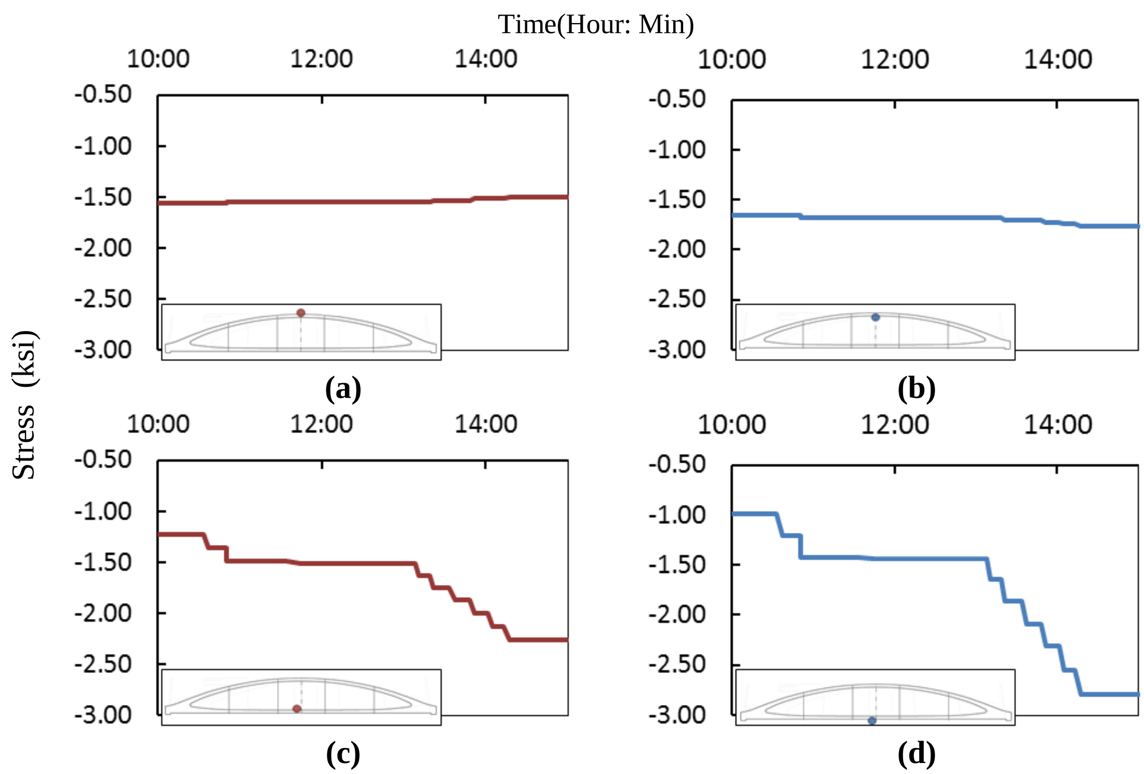


Fig. 12- Stresses in Arch 2 during Stage 2 PT on the tie
(a) Midspan, rib, top (b) Midspan, rib, bottom (c) Midspan, tie, top (d) Midspan, tie, bottom

As can be seen in Fig. 11 and Fig. 12, the response of the arch rib and the tie element were relatively independent of each other during post-tensioning. For each of the tendons, post-tensioning was applied in increments of 52 ksi. During Stage 1 post-tensioning, each increment in the rib tendons increased the stresses at the edges of the rib by more than 100 psi (Fig. 11 (a), R-intervals), but such an increment in the rib caused very small stress changes at the midspan in the tie (Fig. 11 (c), R-intervals). The maximum stress change anywhere in the tie when a prestress increment was applied to the rib was smaller than 20 psi. The stress changes during Stage 2 post-tensioning of the tie show a similar trend. When second stage post-tensioning was being conducted on the tie tendons (Fig. 12 (c and d)), negligible stress changes could be observed in the rib (Fig. 12 (a and b)).

The design team located the centroid of the tendons close to the neutral axis of the tie and the rib at the knuckle region. As a result, very small bending moments were exchanged between the rib and the tie elements due to post-tensioning. Moreover, the orientation of post-tensioning anchorage devices for the rib tendons created static equilibrium of the forces without exerting large shear and axial loads on the tie. In other words, the anchorage force remained in line with the internal axial stresses in the rib, providing the equilibrium without mobilizing the tie.

Another important observation was that post-tensioning did not induce bending in the arch rib except for the eccentricity of the tendons from the neutral axis of the rib. In other words, the curved shape of the rib element per se did not result in significant bending. This behavior is attributed to the circular profile of the arch and careful selection of the tendon paths and anchorage orientation by the designers. As can be seen in Fig. 13, the response of the arch rib to post-tensioning is similar to a compressive ring, which is in pure compression due to the effect of pressure applied along its radius. This characteristic of the arch design was very important in relieving the arch ribs from long-term bending deformations under sustained post-tensioning forces.

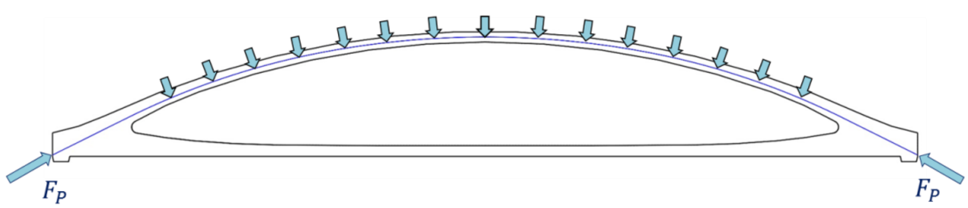


Fig. 13-Compressive stresses acting on the arch rib due to post-tensioning

ROTATION RESPONSE

Fig. 14 shows typical stress changes at the midspan of the arches at the rib and the tie during rotation. As can be seen in this figure, the rotation response consists of two stages. The first stage, supported rotation, represents the change of the arch from a horizontal position to a vertical position while the arch was supported by the lifting frames, as shown in Fig. 4. The second stage, arch setting, represents the release of the arch from the lifting assembly and its installation on temporary supports.

The stress changes during supported rotation were gradual and relatively small. Although the arch was loaded by its self-weight during this stage, it was supported by six lifting frames, which were distributed along the length of the arch, and therefore, the arch action was not mobilized during supported rotation. Most of the observed changes were caused by the bending of rib and tie elements as they behaved similarly to continuous beams during this stage.

The stress changes during setting of the arch on temporary supports were quick and relatively large. During this stage, the arch action was fully mobilized. Therefore, this stage was associated with a significant increase in compressive stresses in the rib and a significant decrease in those in the tie. Although these stress changes were relatively large, the arches were designed to withstand much larger demands due to bridge deck and live loads in the finished bridge. As a result, these stresses were easily tolerated by the arches in the vertical orientation. Consequently, monitoring the stresses during supported rotation was more critical for ensuring the safety of the arches.

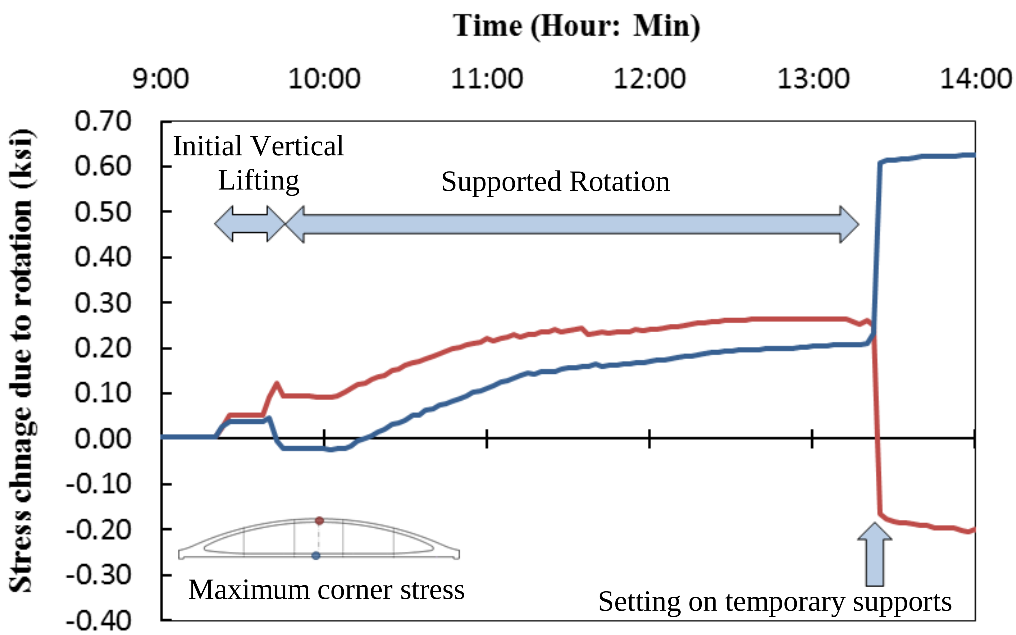


Fig. 14- Typical stress changes in the rib and the tie during rotation

Fig. 15 shows the maximum and minimum stresses observed in the arches during supported rotation. This figure shows that none of the arches experienced tension under this operation and therefore, the arches remained crack-free during rotation.

The predicted stresses in Fig. 15 are based on the calculations carried out by the construction engineering team, assuming a 50% dynamic allowance for the self-weight of the arch. The figure shows that the measured stresses were in reasonable agreement with the calculated results, and where a difference exists between measured and predicted stresses, the predicted response generally overestimated the risk of cracking. Although the rotation procedure was carried out relatively slowly, the 50% dynamic allowance appears to have contributed to obtaining more realistic stress levels.

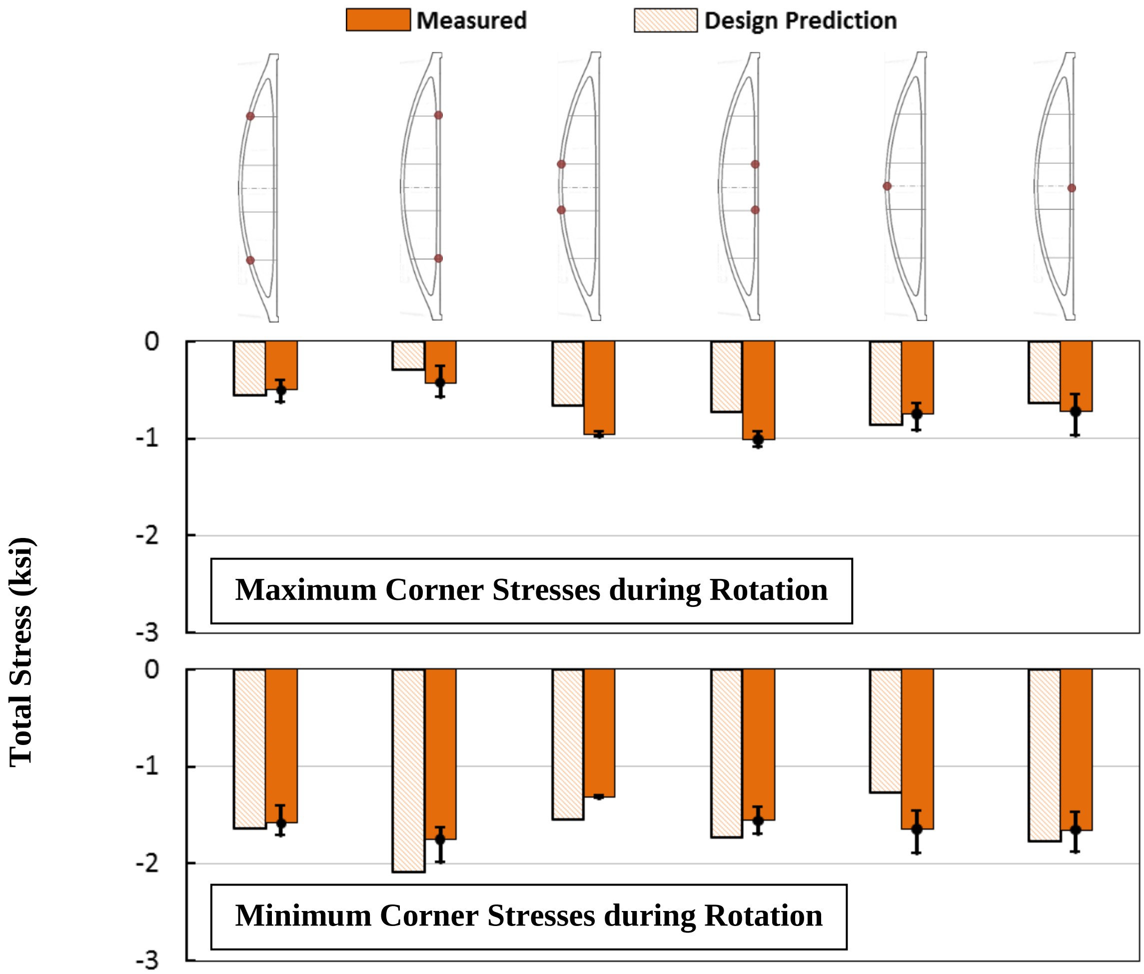


Fig. 15- Average maximum and minimum corner stresses in all 12 arches during rotation (The black lines show the range of stresses in different arches)

UPWARD JACKING

The upward jacking operation was the most critical stage for the knuckle region of the arches. The analysis during design had predicted that during upward jacking, tension would have been induced at the top of the rib in the knuckle region if the arches had not been temporarily strengthened.

Fig. 16 shows the measured stresses in the arches when the hydraulic rams were activated but the hanger nuts were not re-tightened yet. As can be seen in this figure, the stresses in the knuckle region were in good agreement with the design predictions. Although some arches experienced tension during upward jacking, the tensile stresses were well below the modulus of rupture of concrete.

The figure shows that the stresses at midspan of the arches in the tie were highly variable during upward jacking. This observation was not surprising, as the arches included hand-tightened hangers with unknown forces before upward jacking. The stresses at the bottom of the tie were higher than design predictions but below 50% of the compressive strength of concrete.

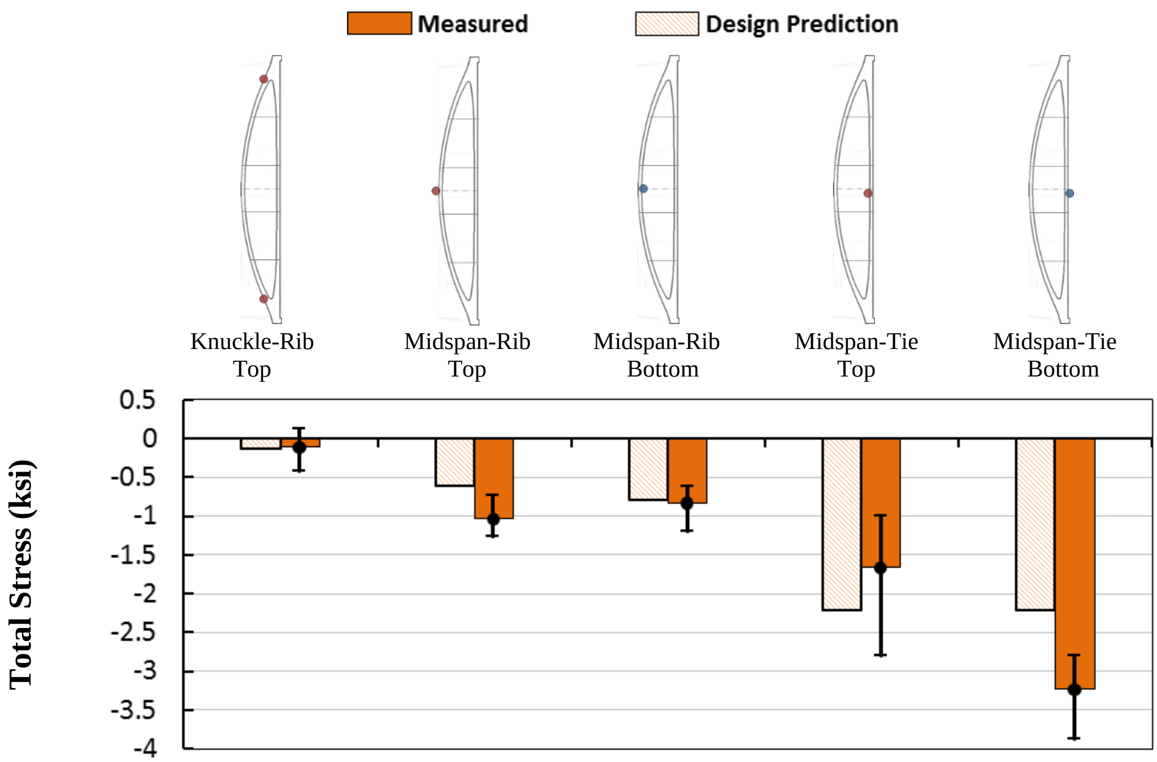


Fig. 16- Maximum arch stresses during upward jacking operations before deactivating the hydraulic rams

STRESSES AT THE END OF EACH CONSTRUCTION OPERATION

Fig. 17 shows the stresses at the end of the main construction operations on the arches in the precast yard. As can be seen in this figure, design calculations were generally successful in predicting the stress levels in the structure during precasting operations. While variability was observed in the concrete stresses between different arches, design calculations could identify the major parts of stress changes in the structure and provide safety for the arches during handling operations. The largest differences between measured and predicted stresses were observed at the bottom of the tie, where design predictions underestimated the compressive stresses. However, since the stresses were below the linearity limit of the concrete, such a difference should not be a source of concern.

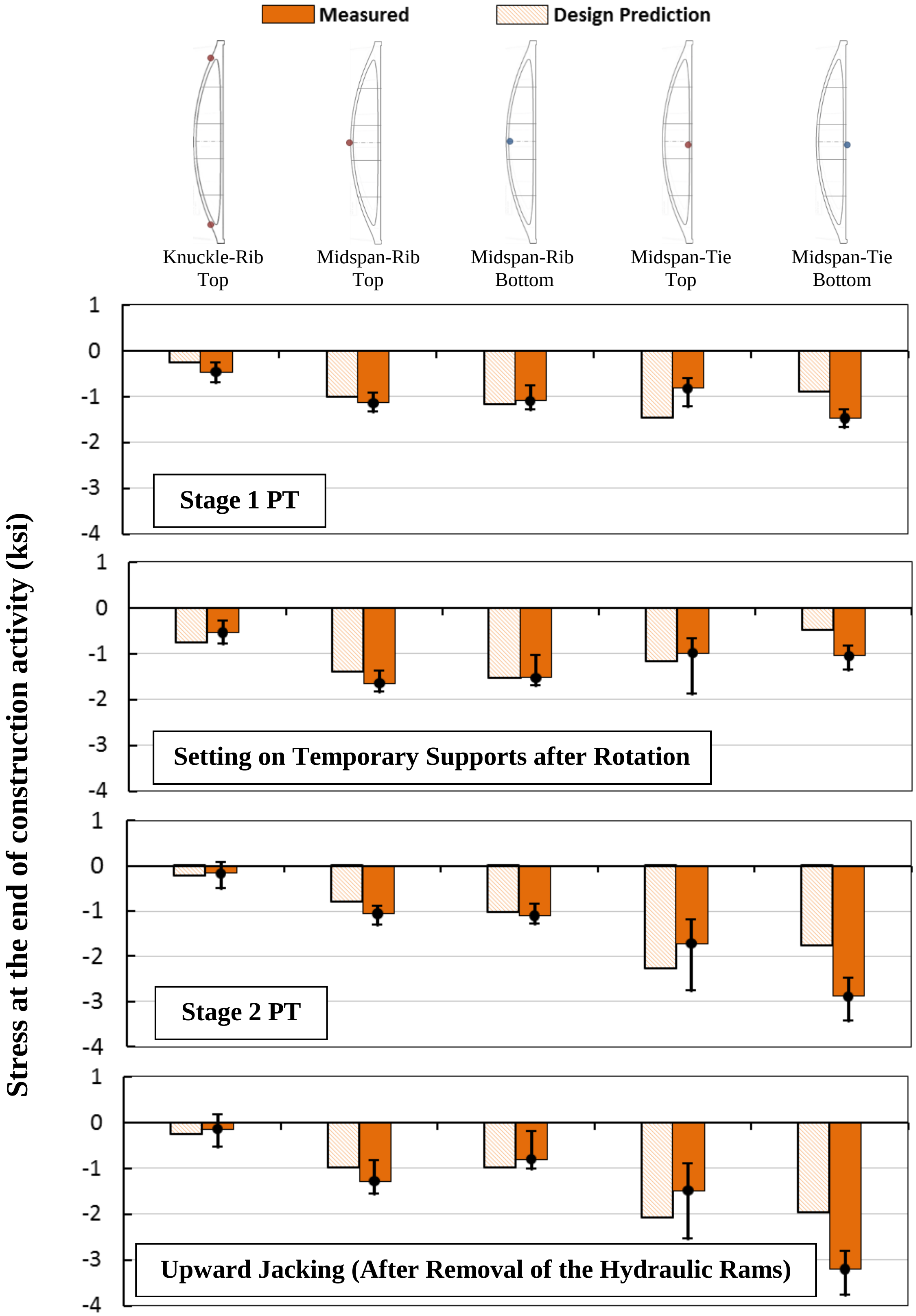


Fig. 17- Stresses at the end of each construction operation

The total stresses in the finished bridge are highly influenced by the post-tensioning and dead load stresses. After the bridge was opened to traffic, a static live load test was conducted on one of the most heavily instrumented spans of the bridge using four sand trucks weighing approximately 50 kips. The largest live load stress measured in the instrumented sections of the arches was approximately 200 psi, which is relatively small compared to the construction stresses shown in Figure 17. The details of the live load test on the bridge will be discussed in a later publication.

CONCLUSIONS

This paper evaluated the stresses during construction of the first precast network arch bridge in the world. The West 7th Street Bridge was monitored using embedded vibrating wire gages throughout construction to develop an improved understanding of the structure's behavior and to ensure the safety of the arches.

The primary conclusions of this paper are as follows:

- Design calculations were generally successful in capturing the essence of the structure's response during post-tensioning and handling operations. As a result, the arches were successfully constructed without experiencing excessive stresses or cracking.

- The short-term construction stresses in the identical arches could be highly variable from one arch to another, particularly after the arches were rotated into the vertical orientation. Uncertainties due to presence of hand-tightened hangers may have contributed to such variability. In this project, the observed variability did not result in endangering the safety of the arches. However, reliable stress predictions for network arches must consider uncertainties due to unknown hanger conditions, regardless of the level of sophistication used in modeling. A successful handling design would best be obtained by assuming multiple conditions of hanger forces and making sure that the structure will not undergo excessive stresses due to an unforeseen stiffness distribution.

- Before finalizing the design of other structures that might be sensitive to cracking similar to these arches, a material study is highly recommended. The modulus of elasticity of the concrete and creep and shrinkage parameters will affect the stress calculations. Therefore, these parameters must be realistically estimated before design. Although such a study is often impractical in initial design calculations, it is possible to analyze the model with the updated parameters once the final mix is determined and make sure of the suitability of the design.

To the knowledge of the authors, the study presented herein is the first ever on the construction responses of a concrete tied arch bridge of any type. The data obtained in this study are a useful validation tool for future modeling of concrete arches.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the Texas Department of Transportation (TxDOT) for providing the financial support for this implementation study. Dean Van Landuyt and Courtney Holle, the TxDOT engineers who designed the arches, provided a great deal of guidance on the project. In addition, there were numerous engineers and representatives from TxDOT and the contractor, Sundt Construction, who provided significant assistance throughout the investigation. Many thanks are also due to Jose Gallardo, Ali Morovat, David Garber, Kostas Belivanis, Vasilis Samaras, and Hemal Patel, current and former graduate students at The University of Texas at Austin, for assisting with the instrumentation. The findings, opinions, and recommendations presented in this article are those of the authors and do not necessarily reflect the views of TxDOT.

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