Nuevo puente en el puerto de Corpus Christi (Texas, EEUU). Descripción general y análisis global del puente de dovelas prefabricadas de mayor luz del mundo

New Harbor Bridge in Corpus Christi (Texas, USA)

Global Structural Analysis of the World's Longest Span Precast Segmental Bridge

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RESUMEN

El puente sobre el puerto de Corpus Christi (Texas, EE.UU.) será el puente de dovelas prefabricadas de mayor luz del mundo cuando se complete en 2025. Se trata de un puente atirantado con una longitud total de 1004 m y una disposición de luces de 82-166-506-166-82 m. El tablero consta de dos cajones de dovelas prefabricadas independientes con un plano central de dobles tirantes que se conectan a los cajones mediante un marco en forma de delta (cercha de hormigón prefabricado), lo que proporciona un ancho total de tablero de 45 m (148 pies). Este artículo describe la participación de los autores en el proyecto junto con el análisis del comportamiento global y desarrollado para completar el diseño.

ABSTRACT

The New Harbor Bridge in Corpus Christi (Texas, USA) will be the world's longest span precast segmental bridge when completed in 2025. It is a cable stayed bridge with a total length of 1004m (3294 feet) and a span arrangement of 82-166-506-166-82m (270-546-1661-546-270 feet). The deck consists of two separate precast segmental boxes with a central plane of double stays that are connected to the boxes via a delta frame (precast concrete truss), providing a total deck width of 45m (148 feet). This paper describes the involvement of the authors in the project along with the global behaviour and analysis developed to deliver the detailed design.

PALABRAS CLAVE: Puente atirantado, dovela prefabricada, Construcción en voladizo
KEYWORDS: Cable Stayed, Precast Segmental, Delta Frame, Balanced Cantilever Construction

1. Introduction

The existing Harbor Bridge spans the main shipping channel entrance to the Port of Corpus Christi, the nation's largest port for exported tonnage. Opened in 1959, the bridge is being replaced due to its low clearance, high maintenance cost and safety issues associated with the lack of shoulders, steep grade and reverse curve.

Texas Department of Transportation awarded the development, design, construction and maintenance contract to Flatiron Dragados LLC (FDLLC) for the replacement of the existing Harbor Bridge.

The bridge main structure consists of a signature precast concrete segmental cable-stayed bridge (the New Harbor Bridge) and approximately 5.7 miles of bridge and connecting roadway. Arup-CFC Design Joint Venture is the Lead Engineering Firm for the New Harbor Bridge.

Figure 1. General view of the shipping canal with the old and new bridge during construction (Sep 2023).

1.1 General description of the structure

The structure is a five span cable stayed bridge which, once completed, will be the longest precast segmental span in the world with a main span of 506m (1661 feet). In cross section the deck is formed by two separate boxes. Due to the presence of a shared used path for pedestrian and cyclist in one side of the carriageways, the two boxes have different widths (67 feet and 6 inches for the Northbound and 56 feet and 6 inches for the southbound). Due to this asymmetry, the central truss, which is also precast, receives the cable loads eccentrically, with the vertical kingpost and the nodal block that takes the load from the central plane of cables closer to the northbound than to the southbound.

The stay system consists of 19 pairs of stays in a single central plane. The number of strands varies from 61 in the shorter stays, closer to the tower to 125 for the longer stays at midspan and closer to the transition piers.

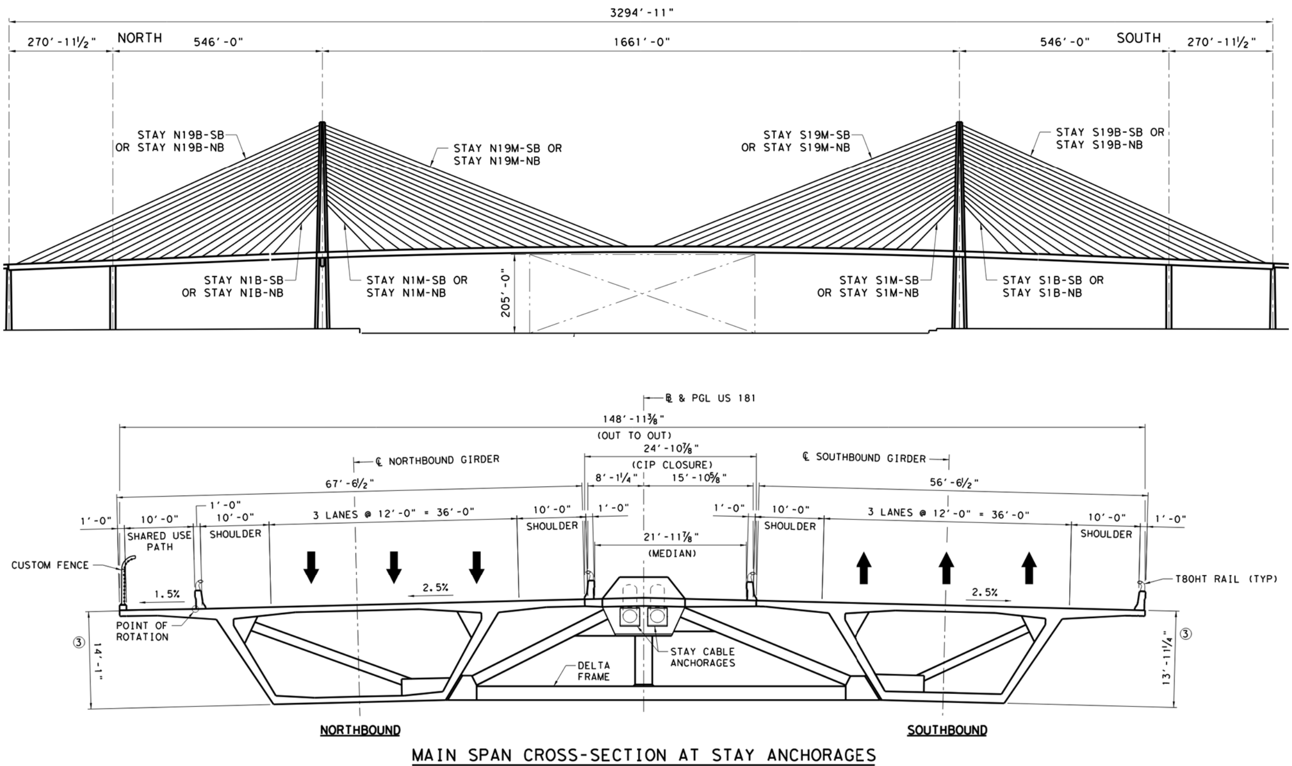

Figure 2. Main span elevation and cross section at stay anchors.

1.2 Background

In August 2020, ARUP-CFC was appointed to take over design responsibilities for the NHB, Arup as Engineer of Record (EOR) and Carlos Fernandez Casado as Independent Technical Reviewer (ITR).

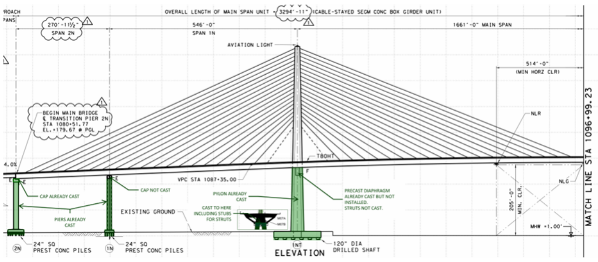

Figure 3. Substructure elements built at the time of the design takeover.

At that time, the substructure, which includes the main span towers up to deck level, along with the transition piers and back span piers were already built (see Figure 3 above).

Also, as expected of a precast segmental bridge, the precast yard had already advanced the fabrication of a small part of the segments (6 out of 89 on the back span and 14 out of 89 on the main span for the north tower, see figure 2 below).

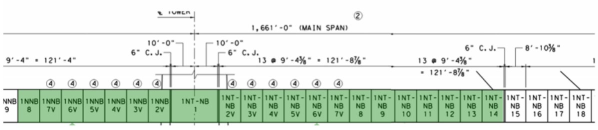

Figure 4. Precast segments fabricated at the time of the design takeover.

The status of the project, both from a design and construction point of view, conditioned the approach of the EOR/ITR team as the time for restart agreed as part of the agreement was limited (15 months) and the impact on those elements already constructed needed to be minimized.

To comply with the contractual deadlines, the ARUP/CFC Joint Venture performed the full modelling and analysis of the structure in parallel. The first document developed by the EOR was the Design Basis, after review and agreement by the ITR team.

Only where an element was found non-compliant, a conflict-resolution meeting was stablished. This required close coordination between EOR and ITR, to assure the same information was always available for both design teams. Thanks to the system implemented, the lead time between EOR and ITR sign and seal of drawings was reduced to 2 week per design package.

A similar system was implemented during construction, to double check all geometry control values provided to site.

2. Analysis approach

The authors employed up to four different global models with slightly different approaches which required calibration by the use of local models which were also utilized for transversal design and local deck effects.

The EOR used a primary model consisting of a 3D grillage in Sofistik and a secondary model in RM Bridge for the purpose of obtaining the wind buffeting effects. In order to ensure that both models were built up as consistently as possible, a workflow based on a database in MySQL was developed. The database contained all the information required to build the model in a generic way (coordinates, topology, material properties, sections, etc) and also to combine results from different.

The primary global model in Sofistik was developed using a grillage with separate elements for each of the boxes and the central slab (see Figure 5 below).

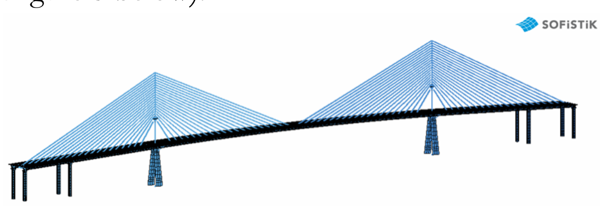

Figure 5. Primary Sofistik model. Global view.

The connection between the grillage longitudinal members and the transversal members was fundamental to ensure that the proper behaviour was properly captured. This is important due to the presence of delta frames every 4 segments and the constraints imposed by the deck at the towers and back span supports.

The Independent Technical Review team from CFC followed a slightly different approach. A single global model in Sofistik was developed as the primary model for the verification of the global components of the bridge and it was calibrated against a local model in Abaqus and verified that the global behaviour was adequate using a secondary global model in their in-house software SICAL (see Figure 6 below).

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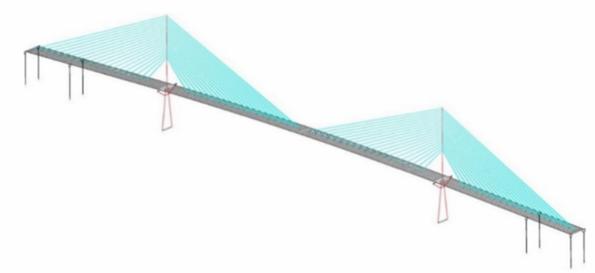

Figure 6. Secondary SICAL model (ITR).

Although both teams used the same software package for their primary model (Sofistik) using a grillage approach, it is important to highlight that the specific strategy to represent the transversal behaviour was slightly different. The EOR model included a stiffening virtual element for the inner web of the boxes (See yellow elements in figure 7 below).

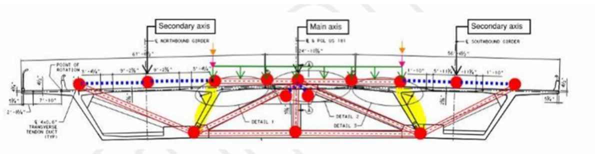

Figure 7. Transversal model strategy (EOR)

On the other hand, the ITR model did not included these elements (see figure 8 below).

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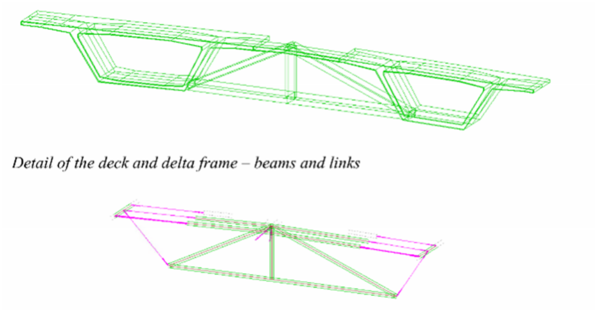

Figure 8. Transversal model strategy (ITR)

In both cases the properties of the transversal and longitudinal elements were independently calibrated using a local model as described in the following section.

2.1 Model calibration

Given the complex behaviour of the structure, it was necessary to tune the section properties of the longitudinal and transversal elements of the global model to ensure that the proper stiffness of the different components (stay system, deck longitudinal and transversal) was properly captured and the results obtained in the global analysis could be accurately used to demonstrate compliance with the Service and Strength requirements.

The EOR team used a local Sofistik model which combined 3D brick elements for the nodal zones at the intersection of the delta frame with the cables or the boxes, 2D shell elements for the boxes and 1D frame elements for the internal members of the delta frame.

On the other hand, the ITR team developed a full 3d model using all brick elements. Both models included the transversal post-tensioning and in addition to the calibration of the global models, were also used for the local verification of nodal zones and the transversal analysis, which is described in a separate paper [1].

2.2 Wind loading

The NHB is located in a hurricane zone and has very specific wind loading requirements.

Wind tunnel tests were carried out and the correspondent reports were made available to the design team (See [8]])

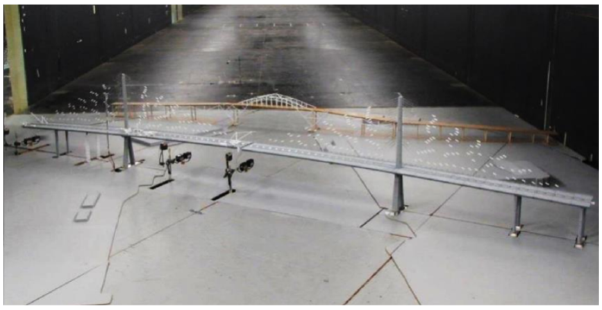

Figure 9. Wind tunnel testing of NHB.

The EOR performed a buffeting analysis using RM Bridge in the frequency domain based on the modal behavior of the bridge and the wind climate definition of the power spectra as well as the turbulence intensities provided in the Basis of Design.

The buffeting analysis was carried out for the In-Service stage once the bridge was completed as well as the critical stages during construction (when connecting with the temporary pier in the back span, permanent pier and transition pier), The wind demands used for checking of the intermediate stages were derived based on an interpolation of results from the critical stages for which the full buffeting analysis has been performed.

The effects of aerodynamic admittance on the buffeting response have been captured by implementing an aerodynamic admittance that follows the square root of the Davenport admittance function to the drag, lift, and moment directions. The ITR followed a different approach, wind loads were calculated by the independent (BLWTL) wind expert contractor using their internal software and provided and results from the Wind Tunnel analysis and dynamic information provided by CFC. This independent approach provided a robust analysis of the complexities of structural response under high wind conditions.

3. Superstructure analysis results

3.1 Cable tuning & construction sequence

In addition to the effects of the different live loads over a calibrated global model, a long span concrete structure is heavily influenced by the permanent stage and, in the case of a cable stayed bridge with integral towers, the long-term effects and redistribution of forces due to creep and shrinkage are critical for both the superstructure and substructure.

A rather complex stage by stage construction model was built, including stages where boundary conditions change significantly such as the connections of back span to towers either permanent or temporary. This allowed the cable forces and locked in effects to be obtained quickly and the bridge tuning to be undertaken for the second stressing of the cables after closure to deliver a compliant structure.

3.1.1 Stay stressing post closure.

As it is well known, the parasitic effects of a cable system in a prestressed box eliminate the beneficial effects of eccentricity of the continuity PT. As figure 10 below shows, the axial effects of the continuity PT are completely centred at midspan.

Since the segment moulds were already built, and this fixes the PT layout, in order to comply with the requirements of the Service stresses of AASHTO, and achieve enough compression in the bottom flange of the boxes under permanent loads so the live load and other effects do not introduce stresses beyond the maximum tension allowed (see section 3.2 below), a third stressing of some stays, post closure, it is required in order to increase the compression in the bottom flange around midspan.

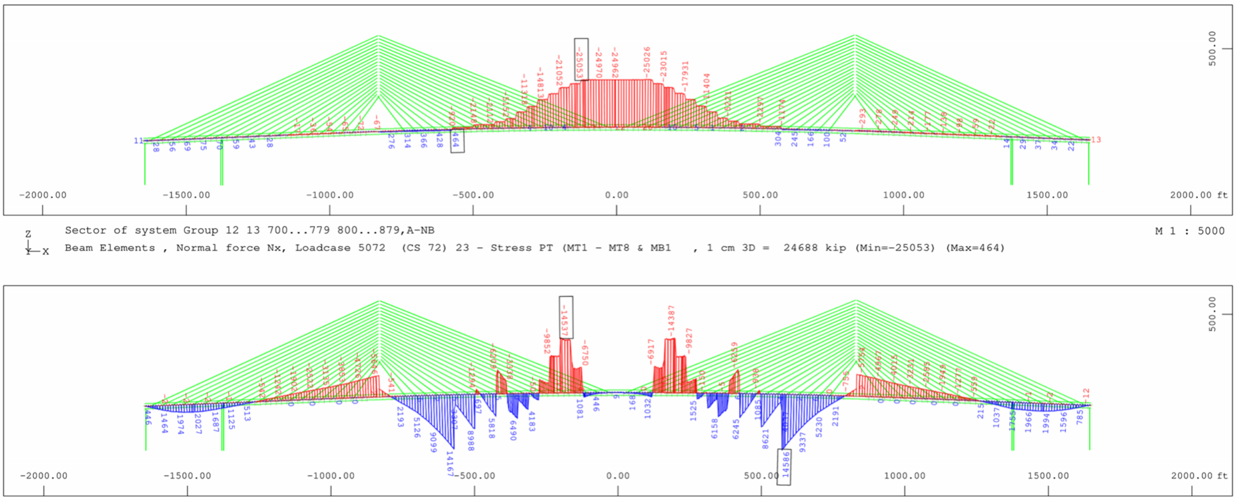

Figure 10. Axial forces (top) and bending moments (bottom) due to the continuity PT.

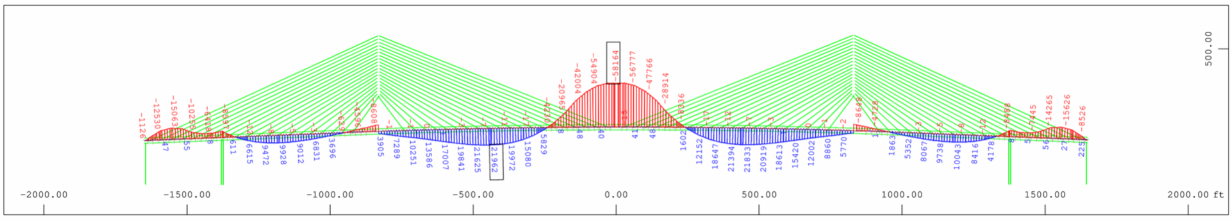

Figure 11. Bending moments due to the third cable stressing post mid span closure.

This additional restressing was limited to cables 16 to 19 around midspan, with this strategy an additional hog bending of 58,000 kip-feet was introduced at midspan (see figure 11), which resulted in over 2.5ksi of additional compression. The bending moment under permanent loads at opening, reduces in around 15,800 kips ft for the long term case (PAL 30 - Permanent Applied Loads at 30 years) from the initial 58,154 kips ft introduced by the third stressing at opening day. Despite this loss due to creep and shrinkage, this remaining hog moment is still beneficial enough to ensure compliance under the service checks.

3.2 Deck longitudinal analysis. Service verification

As indicated in the previous section, the continuity prestress in the top and bottom slabs of the boxes, along with the second stressing of the longest cables was designed to comply with the AASHTO requirements on stress limits (see Table 5.9.4.2.2-1 which allows for a maximum tension of 0.0948 √ f'c and a maximum compression of 0.6 f'c).

This check was carried out for both the PAL 0 (opening) and PAL 30 (long term) scenarios taking into account creep and shrinkage and the corresponding Service envelope for traffic, wind and thermal. As shown in the figure below, the midspan section is critical to comply with the minimum compression requirements governed by Service III.

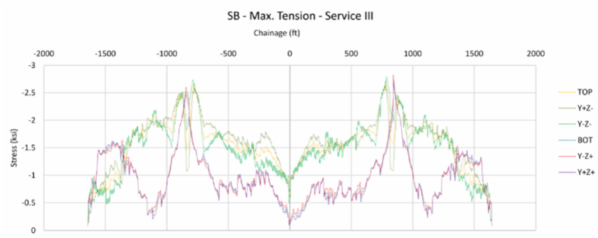

Figure 12. South bound box. Service III (tension) check.

It is important to highlight that the transversal effects due to the specific structural behaviour of this structure are critical. As shown in figure 13 below, the critical section for maximum tension (midspan south bound), shows a significant distribution of normal stresses in the transversal direction.

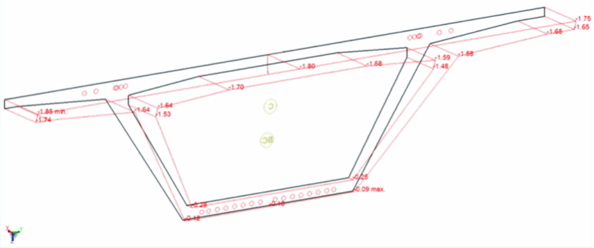

Figure 13. South bound box. Service III (Tension) check, transversal distribution of normal stresses.

Similarly, the concrete strength variation between segments (initially the design included 8ksi in general and 10ksi in selected areas, but the final design upgraded all segments to 10ksi), is tuned towards those points where the maximum compression stress in Service I requires a higher concrete strength (at the towers and the anchor pier)

3.3 Deck Longitudinal analysis. Strength verification.

As indicated in the previous section, the complex nature of the structural system of the deck with a single plane of cables and two separate boxes linked by a delta frame at cable locations, there is a significant coupling between the bending moments due to a horizontal axis (My) which are the conventionally expected for gravitational loads, and the bending under a vertical axis (Mz) induced by transversal bending in the grillage which is also coupled with the torsion.

Figure 14. Strength Envelope My.

As it can be observed in Figures 14, the My bending moments reach values of 350,000 kips-ft in hog at the towers and around 180,000 kips-ft at midspan. While the Mz bending reaches 180,000 kips-ft at the towers and 80,000kips-ft at midspan, in a comparable order of magnitude of the vertical axis bending.
As a consequence of this effect, the strength checks on the deck boxes are clearly governed by biaxial bending rather than the conventional single axis bending on the horizontal axis being mostly de-coupled from the Mz which only occurs under wind or other transversal loading effects.

The main results for the strength check under bending, which was carried out as per AASHTO Cl 5.7.3.2.1-1, is shown in Figure 15 below, with the critical sections at midspan showing a D/C ratio of 0.9.

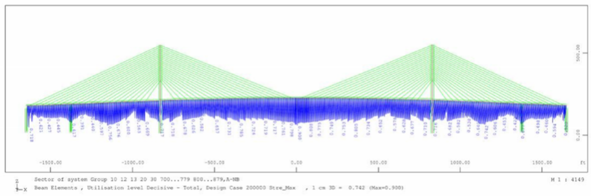

Figure 15. Deck longitudinal strength check under axial and bending.

3.4 Shear and Torsion

3.4.1 Torsion cracked vs uncracked properties.

The wind buffeting loading proved to be governing for the torsional demand at deck level. In a highly redundant system where the deck torsional capacity is coupled with the transversal bending, the ratio between both needs to be analysed properly, particularly when the torsional cracked stiffness is exceeded.

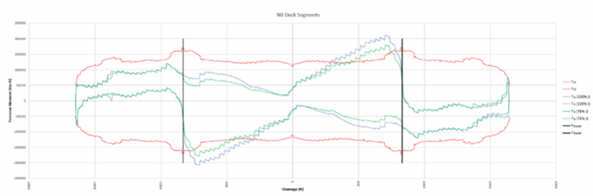

Figure 16. Torsional demand, cracked vs uncracked stiffness (north bound box shown).

For the shear and torsion, the transversal reinforcement combinations were designed using clauses 5.8.3.3 and 5.8.2.1 following the combination rules recommended by Jean Muller in his book Precast Segmental bridges [7] as follows:

- 100% Torsion+Shear demand + 50% transversal bending.
- 50% Torsion+Shear demand + 100% transversal bending.
- 70% Torsion+Shear demand + 70% transversal bending.

Figure 17 below shows the D/C ratio per web when shear and torsion are combined without transversal bending.

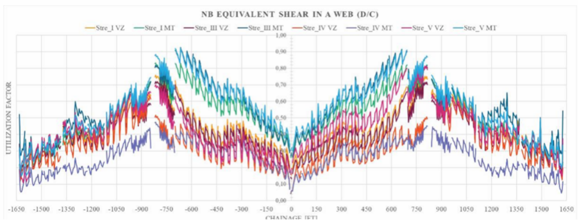

Figure 17. Shear and torsion combined demand to capacity ratio per web. North bound box.

When combined with the transversal bending demand with shear and torsion, the reinforcement requirements in the transversal direction are shown in Figure 18 below for one of the two boxes.

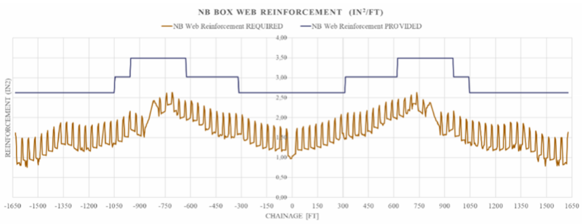

Figure 18. Transversal reinforcement in the webs, combined demand requirements (in2/ft).

4. Global analysis. Substructure results

For the design of the main tower foundation, the global model incorporated the full pile design.

This model was calibrated against 6x6 stiffness matrices that were used in the superstructure and construction analysis. Figure 19 below shows the global model incorporating the foundations.

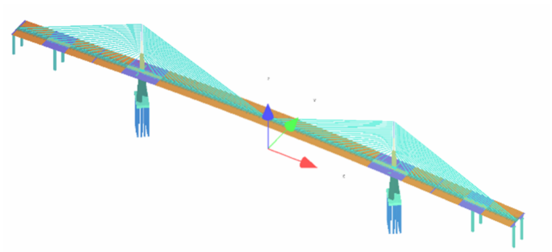

Figure 19. Global model with main tower foundations.

4.1 Optimization of bending moments at main towers

The main towers are integral with the deck, this means that the main span behaves as a 506m span portal frame once the main span is closed. The lower tower bending moments are heavily influenced by the axial shortening due to the application of the continuity PT as well as creep and shrinkage. To compensate these effects, a horizontal jacking of 1350 kips (615 tons) was introduced at main span.

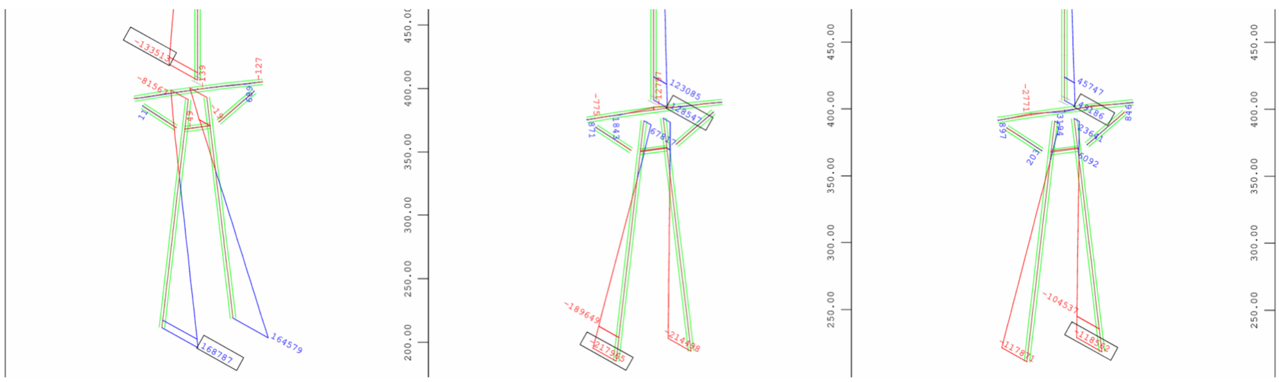

Figure 20. Tower bending moments due to horizontal jacking (left) vs. total bending due to permanent effects: PT+Stay Restressing+ Long term (middle) vs. creep and shrinkage only (right). All units kips-ft

As it can be seen in Figure 20 above, the jacking force over-compensates the predicted creep and shrinkage (167,000 kips ft vs 117,000 kips ft) and sits in between the total bending when elastic shortening due to PT and stay re-stressing is considered (217,000 kips-ft vs 167,000 kips-ft).

4.2 Vertical tendons on the back span piers

In order to avoid the uplift of the bearings in the back span piers, vertical hold downs were provided. It was modelled explicitly in the global model, with its activation during construction captured both in the global analysis and erection engineering models.

The vertical hold downs are extended in the back span piers up to the bottom of the pier in order to guarantee the full mobilization of the pier as counterweight without creating tension in the column and also to ensure that the horizontal movements can be achieved at deck level.

4.3 Global design. Upper tower

The design as received by ARUP-CFC, showed a saddle/cradle crossing the towers. As an alternative FDLLC proposed a more conventional double box anchor.

The upper tower, consists of two different sections:

- A vertical reinforced concrete section up to the first anchor and
- A steel composite section from anchor level to the top where the anchor boxes transfer the loads from the stays to the steel boxes and then to the concrete (see figure 23).

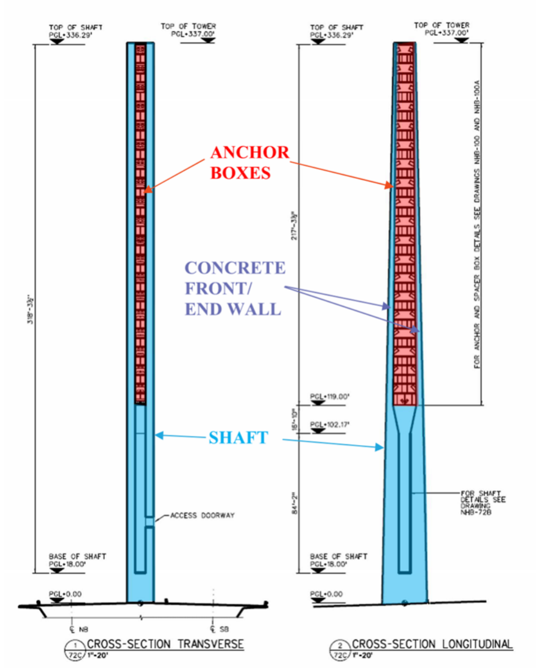

Figure 23. Upper tower schematic design elements.

The design methodology of the towers was as follows:

- Determine forces from the global model (including wind buffeting from RM).
- Determine second order effects due to slenderness
- Section verification under Strength Limit State, control of cracking under Service Limit State and Section verification under Extreme Limit State
- Verify reinforcement detailing including requirements described in the Security Report

The Extreme event conditions required to be analysed were as follows:

- Stay cable loss in accordance with PTI and Extreme Event III, also including double cable loss due to a delta frame loss.
- Fracture of the anchor box web beam at Extreme event III

The amplification factors due to second order effects are relatively low as shown in the figure below:

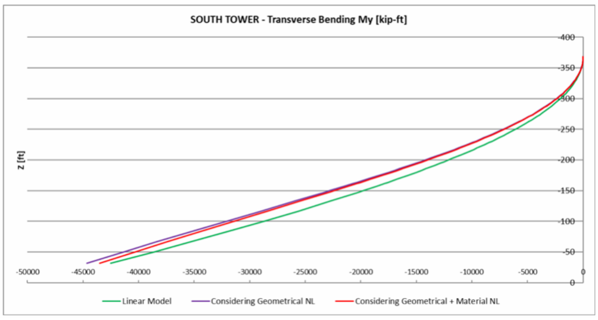

Figure 24. Upper tower (south) linear vs non-linear bending moments (transversal effects).

5. Conclusions

This paper covers the global analysis of a world record breaking precast segmental bridge. The results shows that the design is very optimized . The modelling and complexity of the structure required several model calibrations and independent revisions, reaching up to four different global models and two local models used for the calibration of the global models.

The fact that several components of the bridge were already built or fixed (precast segmental moulds, etc) at the time of the authors getting involved in the project, limited the possibility of changes to be implemented in order to achieve compliance with the AASHTO and project requirements.

The ARUP/CFC Design Joint Venture was successful in providing a complete analysis and re-certification of all elements in less than 15 months. The close coordination between EOR and ITR, working independently and in parallel was a key to the project's success.

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