Nuevo puente en el puerto de Corpus Christy (Texas, EEUU). Modelos locales del puente de dovelas prefabricadas de mayor luz del mundo

New Harbor Bridge in Corpus Christi (Texas, USA)

Local 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 delta (cercha de hormigón prefabricado), lo que proporciona un ancho total de tablero de 45 m (148 pies). Este documento describe los modelos requeridos para comprobar el comportamiento local de los diferentes elementos estructurales del puente.

ABSTRACT

The New Harbor bridge in Corpus Christi (Texas, USA) will be the word 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 local models developed for the analysis required 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 New Harbor Bridge, cable stayed precast segmental bridge with a double box deck supported by a central plane of stays, will be the longest and widest precast segmental bridge once completed, a general view of the elevation and cross section can be seen in Figure 1 below. Due to the complexity of the structural behaviour of the bridge, the authors were required to develop a significant number of models to both calibrate the global model and analyse local areas where the global model does not produce results in enough detail.

This paper covers the analysis carried out by the integrated team of the Engineer of Record (Arup) and the Independent Technical Reviewer (CFC).

A more detailed description of the structure along with the global analysis can be found in a separate paper [1].

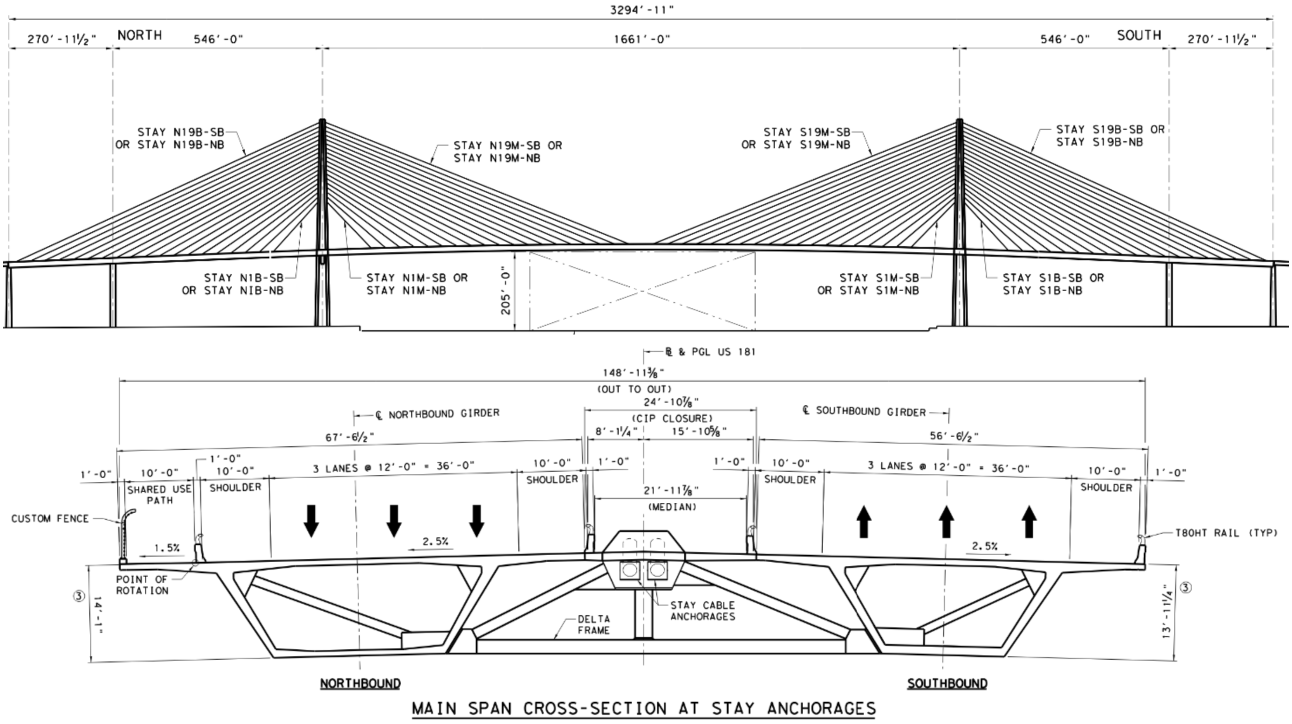

Figure 1. New Harbor Bridge. Main span elevation and cross section at stay anchors.

1.1 Local models. General

This paper covers the following local models required for the design of the New Harbor Bridge:

1. Deck transversal and nodal zones (transversal calibration, stay nodal zone at delta frames and delta frames to deck intersection).
2. Deck diaphragms at transition and anchor piers.
3. Main towers foundations.
4. Anchor towers foundations.
5. Deck-tower intersection (nodal zone).
6. Box anchors and upper tower.

Each model was analysed using different software packages and methodology by the Engineer of Record (Arup) and the Independent Technical Reviewer (CFC).

In the following sections, a description of the models developed, their methodology and the most relevant results for each of these areas of the bridge are detailed.

2. Deck. Local models

2.1 Transversal and local behaviour

2.1.1. Modelling strategy

As indicated in the global analysis paper, the 3d grillage used for the design of the longitudinal members of the structure, using mostly frame elements, required calibration due to the complex behaviour of the structure. The coupling of torsional and transversal bending along with local shear lag effects required a comparison between the results obtained.

Since the original model was developed using a database in an abstract coordinate system, it was possible to build a local model using the same coordinate system sharing all the geometry with the global model.

A set of Grasshopper scripts were used to be able to transfer the geometry from the database to a 3D FEM model combining 1D beam, 2D shell and 3d brick elements in the selected part of the structure

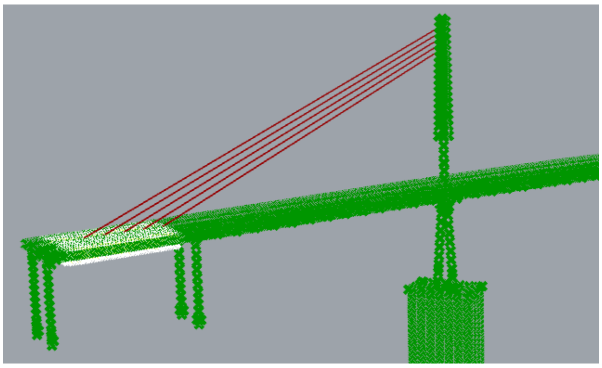

Figure 2. Grasshopper-Rhino model showing the area to be modelled locally

The selection of shells, bricks and beam elements was chosen to balance the correct representation of each area, the model size and a representative extraction of results that allowed to verify the section against the relevant clauses of AASHTO.

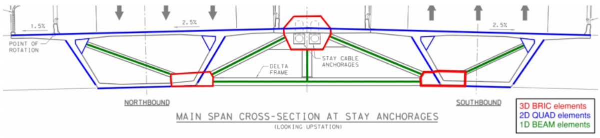

Figure 3. 3D modelling strategy of the deck local model.

The nodal zones of the delta frames were modelled as 3D bricks as the introduction of the anchor forces and the load transfer at the blocks produce forces in every direction of space.

A length of four cables was developed with the boundary conditions extracted for the load case from the global model making compatible interface forces and displacements.

2.1.2. Prestressing

The structure presents two families of transversal prestressing:

1. Flat ducts in the top slab to cater with the transversal bending induced by the traffic.
2. Conventional cables in the delta frame (two pairs in two stages, before and after installation, with the second pair creating a link between the bottom corner of the box and the delta frame (See figure 6 and 7 below).

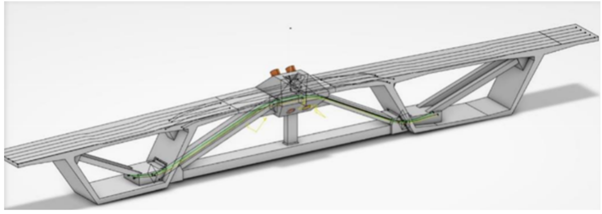

Figure 4. Schematic 3d view of two segments and a delta frame with transversal PT.

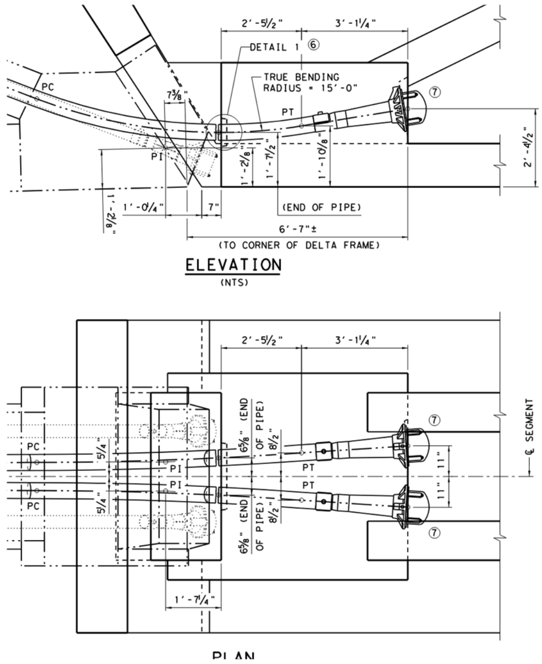

Figure 5. Delta frame-deck connection.

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Sofistik allows for the introduction of prestress directly over shell elements, as well as construction sequence. This permitted to model accurately not only the effects of the prestressing on the local model but also its evolution following the construction sequence of the different elements.

2.1.3. Global model calibration. Main conclusions.

The calibration of the global model with the local model resulted in the introduction of a stiffening virtual element for the inner web of the boxes (see yellow elements in figure 6 below). Results in the 2d shell elements of the local model were integrated to obtain equivalent beam results for direct comparison with the global model grillage approach.

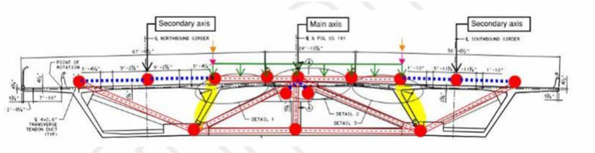

Figure 6. Transversal global model approach (EOR)

2.1.4. Alternative 3D brick model

The independent verifier followed a different approach and produced a local model using Abaqus with 3D brick elements only.

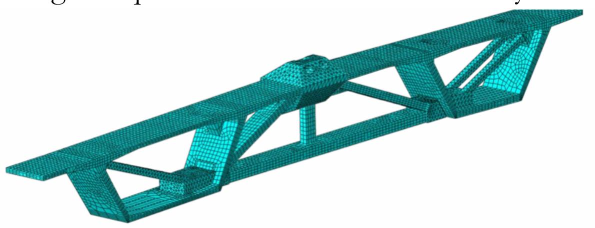

Figure 7. Delta frame segment with brick elements

The length modelled covered several cables which avoided any influence of the boundary conditions.

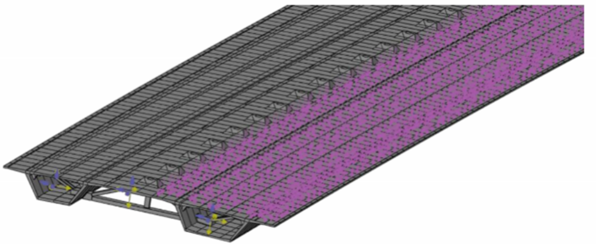

Figure 8. Local model in Abaqus with uniform traffic loading.

The calibration of the global grillage model was carried out for the following load cases:

1. Introduction of the cable load at the anchorage
2. Axial load at the centre of gravity of the section
3. Vertical load at the centre of gravity of Top slab in the North bound deck
4. Vertical load at the centre of gravity of the top slab in the South bound deck.

For these four load cases the same length of global model in Sofistik using the grillage and in Abaqus were analysed and the stiffness of the transversal elements of the Sofistik global model was adjusted to match the results from Abaqus. The graph below shows as an example the comparison of the results for the first load case above.

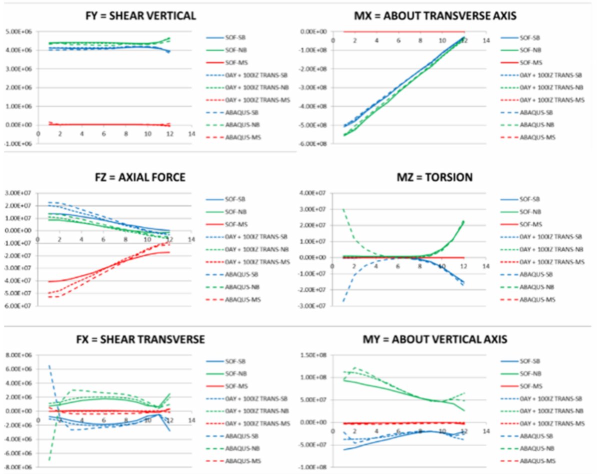

Figure 9. Abaqus vs Sofistik grillage for different Iz

2.2 Results. Deck transversal behaviour.

The local model was used to verify the strength and service behaviour of the top slab under traffic loads. An appropriate section away from the effects boundary conditions was chosen (see figure 10 below).

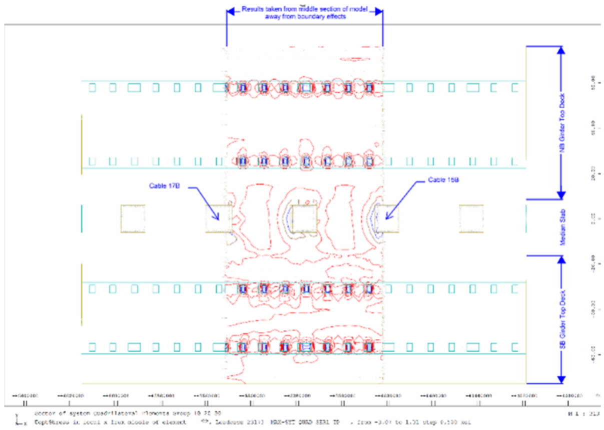

Figure 10. Plan view of the top slab. Transversal strip for checking

The results of the 2D shell elements were integrated a distance sufficiently away from the anchor blocks so the local effects of the intersection between shells were not distorted by numerical peaks. As an example, the figure below shows the north bound box, top and bottom stresses under Service III envelope.

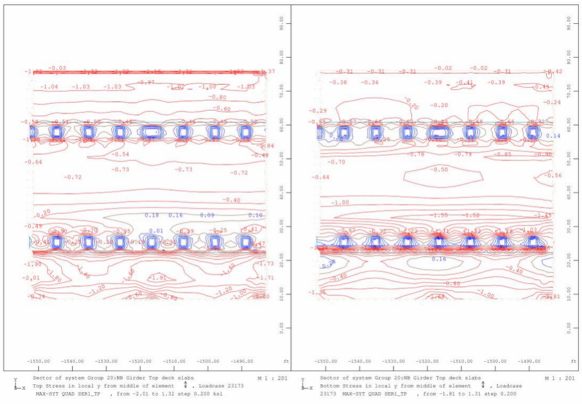

Figure 11. Plan view of the top slab NB. Service I stresses.

2.3 Results. Delta frame nodal zone

The nodal zone of the delta frame was verified using a combination of the results coming from the 3D elastic model part of the local model along with conventional strut and tie models.

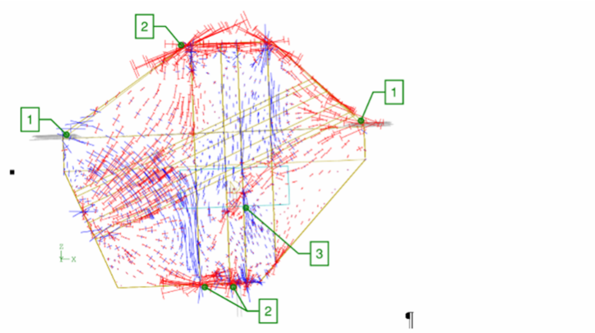

Figure 12. Vertical cut of the nodal stay block. Global XZ plane principal stresses.

As an example, from the cut above the effects of the able anchor and the transfer of the horizontal component to the slab (1) is clearly separated from the vertical component which is introduced in (2) and (3) due to the presence of the delta frame transversal PT.

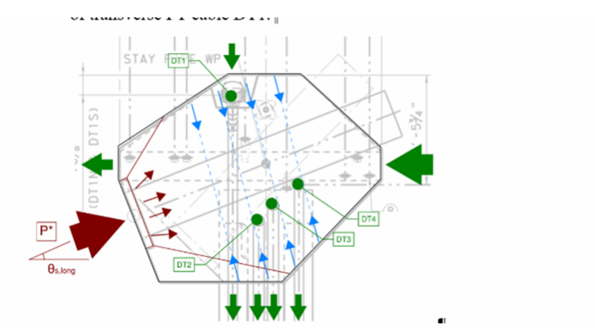

Figure 13. Basis of the STM model for the vertical transfer of loads at nodal stay zone.

A similar approach was followed to determine the reinforcement required in a horizontal plane

As an example, the figure below shows the contour stresses and cuts at different locations on the horizontal plane following the stay.

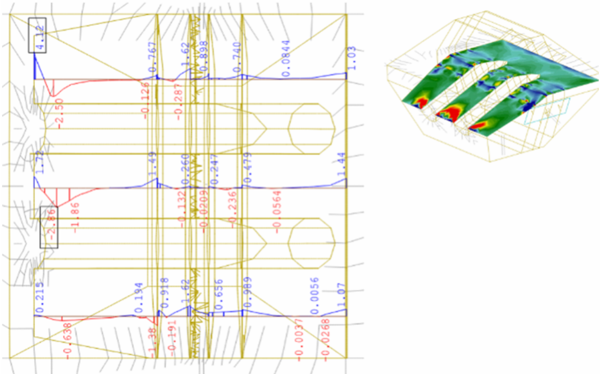

Figure 14. XY plane stresses (contours) and Y transversal stresses.

2.4 Results. Delta frame-deck node

Similarly to the delta frame to stay node, the local model using 3d brick elements was used to analyse the compliance with AASHTO. This section is critical to the behaviour of the bridge as it is the main node that transfers the shear from the inner webs of the boxes to the stays.

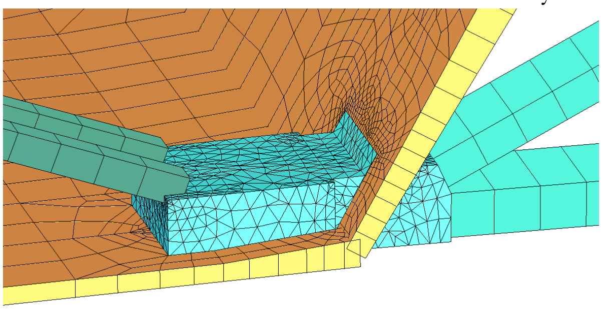

Figure 15. Local model for the delta frame to box connection with 3d brick elements in blue and shells in orange.

To verify the compliance of this critical node, vertical cuts, integrating the non-linear stress distribution and linearized to the section were considered. The figure below shows the vertical plane cuts introduced every 200mm

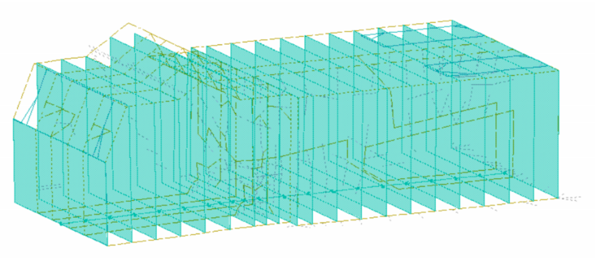

Figure 16. Vertical cuts at the delta frame to box node.

The two fundamental sections are the vertical node at the delta frame to bottom box node, which is only crossed by the second stage PT but no passive reinforcement. The analysis showed that the stresses in the top face were residual (see figure 17 below).

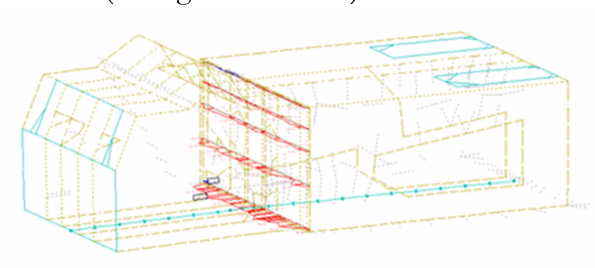

Figure 17. Longitudinal stresses at the nodal zone of the box and delta frame.

The inner web to top of delta frame is completely un-reinforced, which means that compression must be achieved at all stages to comply with AASHTO LRFD 5.8.4.1 to verify the shear capacity of the joint.

Figure 18. Integrated stresses at the unreinforced joint between the inner web and the delta Frame.

2.5 Deck diaphragms. Back span anchor pier

At the back span anchor pier, a single bearing support is provided with vertical posttensioning connecting the deck with pier counterbalancing the tension forces induced by the stays.

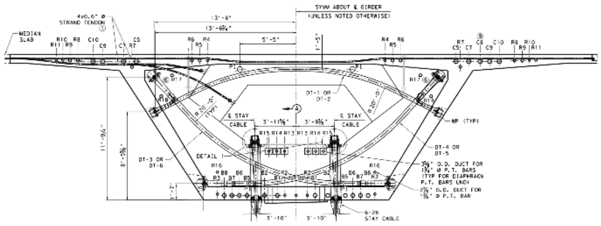

Figure 19. Transition pier diaphragm

As both decks are connected, a local model using shell elements was developed for both boxes simultaneously including the connection slab and the transversal PT that links both (See Figure 20 below).

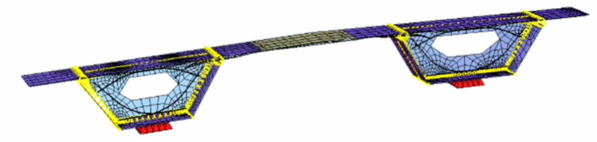

Figure 20. Transition pier diaphragm. Shell model

This model was used to verify the Service scenarios while a Strut and Tie model was developed to verify the Strength combinations as shown in figure 21 below.

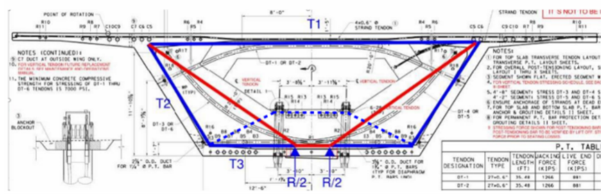

Figure 21. STM model for Strength checks.

2.6 Deck diaphragms. Transition pier

The transitions pier diaphragm required a significant amount of local modelling, not only for the permanent in-service scenario but also for the construction closure stages where the temporary loads associated with the transient stages and loading required to be taken during construction are significant. This procedure is described in a separate paper on erection engineering [3].

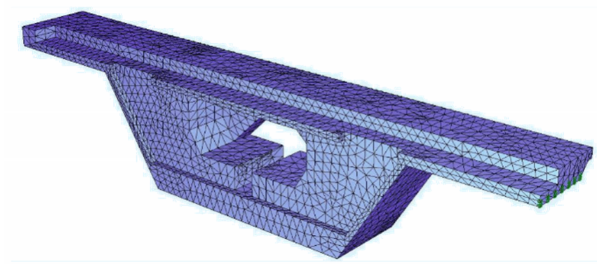

Figure 22. 3D Brick model for the transition segment.

Given the variation of loads and prestressing in the longitudinal direction and the presence of the expansion joint cage, a 3D brick model rather than shell elements was implemented at this location, as shown in the figure above.

The figure below shows the principal stresses for the maximum bearing reaction in a vertical plane in the middle of the bearing.

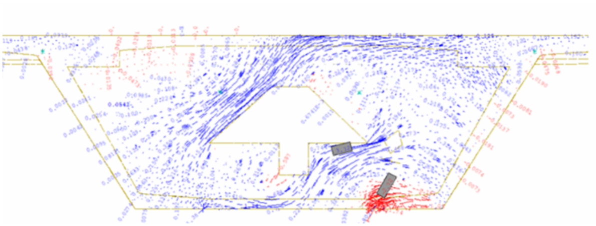

Figure 23. Transition pier local model. Principal stresses in a global YZ plane (without diaphragm prestressing effects).

3. Substructure local models

3.1 Main tower foundations

The main towers of the New Harbor bridge consist of a pile cap of 132ft wide x 72ft long x 18ft deep (40.2m wide x 21.9m long x 4.8m deep) as shown in figure 24 below.

Due to the port quays wall location the main tower legs are skewed to the position of the concrete shafts (large diameter piles of 120 and 96 inches in diameter, 3m and 2.44m).

The reinforcement layout followed an orthogonal grid as it is usual in this type of structures with an additional set of layers in the direction of the tie generated by the leg's opening as shown in figure 24 below.

Since the foundations were already built at the time, the analysis considered the verification of the as-built condition based on the results provided by the global analysis.

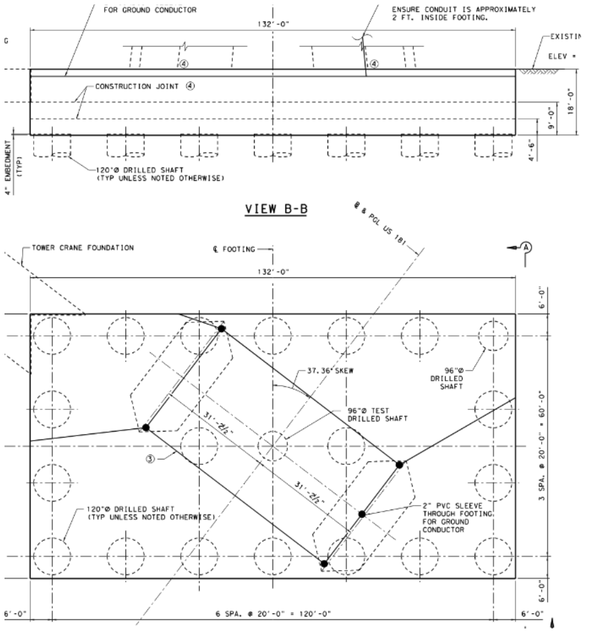

Figure 24. Main tower foundations. Dimensions

A shell model considering the relative flexibility between the piles and the pile cap was developed using Arup in house FEM package GSA as shown in in Figure 25 below.

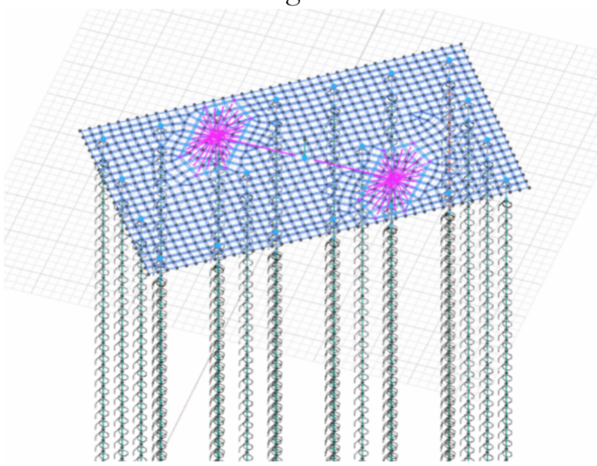

Figure 25. Main tower foundations. Shell and beam elements.

This model was used to determine the tower footing under Strength Limit State and cracking in-service by integrating the results of the shell elements at key locations.

The Independent Technical Reviewer developed a separate three-dimensional model using brick elements in Abaqus which was used for the following purposes:

- Integration of stresses to verify the reinforcement shown in the as built drawings.
- Analysis of load paths and load transfer.

The figure below shows the contour of Abaqus model.

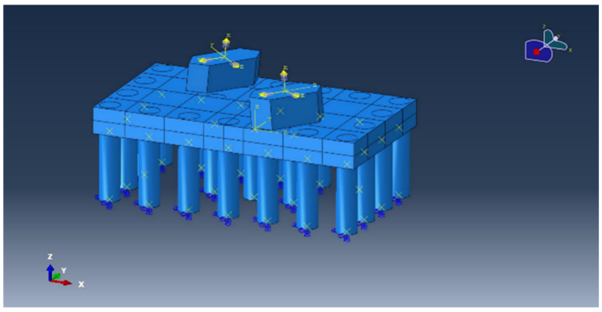

Figure 26. Main tower foundations. ITR Abaqus model.

As an example, the plot below shows the maximum elastic stresses perpendicular to a vertical cut in the long direction of the pile cap (midline). These stresses were integrated in strips of constant width and the resulting forces analysed in cross section to AASHTO LRFD.

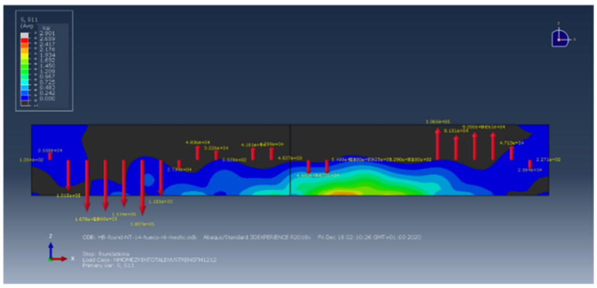

Figure 27. Main tower foundations. YY stresses (short direction) for the maximum vertical reaction.

This model was also used via integration of elastic stresses to verify the tower leg to foundation connection and the right development of the reinforcement in the lower tower. Figure 28 below shows the vertical stresses (S33) for the maximum bending in one of the bottom legs.

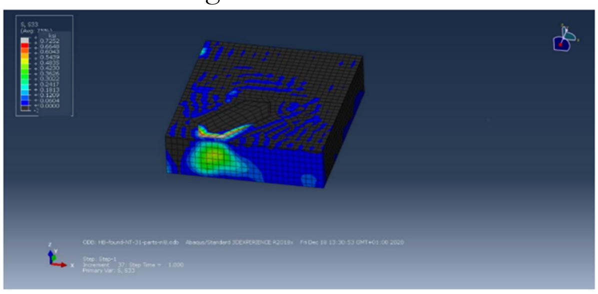

Figure 28. Main tower foundations. S33 stresses vertical (at tower to pile cap connection)

3.2 Anchor and transition pier foundations

The transition pier and back span pier were also built at the time of the design team took over the design. Both columns have piled foundations with different layouts consisting of 24 inches (0.6m) square prestressed piles.

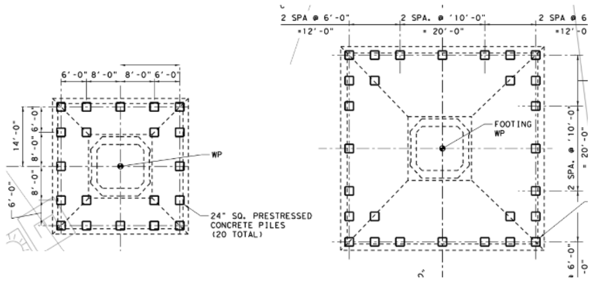

Figure 29. Back span pier foundation (left) and transition pier foundation (right).

As in the case of the main tower, the analysis consisted of an independent verification as the foundations were already built. For the verification of the foundation of the back span pier a conventional STM scheme was used, considering both configurations (tension and compression) as the load case of the column is reversed for the minimum axial force.

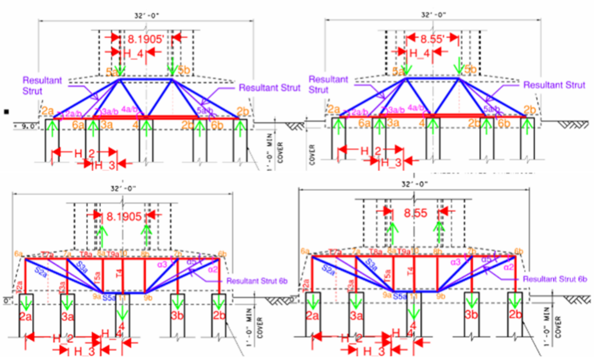

Figure 30. STM for the back span pier verification.

The transition pier on the other hand, due to the significant distance from the tower to the piles in plan (figure 29 right) required a three-dimensional STM with several vertical members which allowed the verification of the shear links as built in the pile cap (see figure 31 below).

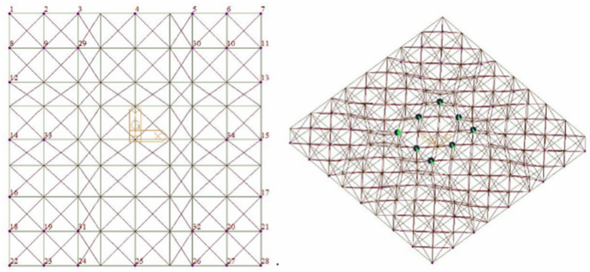

Figure 31. STM for the transition pier verification.

3.3 Nodal zone (deck-tower intersection)

The deck-tower intersection is an extremely complex element where both deck boxes and the tower form a monolithic structure which has transversal PT in addition to the longitudinal PT.

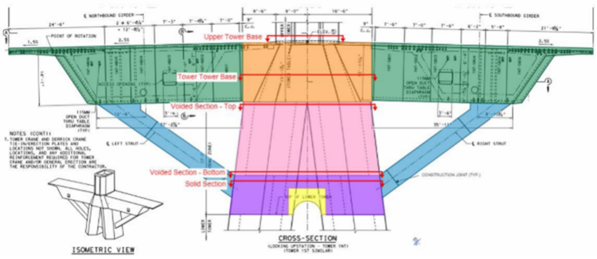

Figure 32. Nodal zone, precast segments (green), props (blue) and in situ tower (purple-pink-orange)

To verify this section, where all kind of three-dimensional effects are superimposed, a non-linear model in LS-Dyna, including explicit definition of the reinforcement, was developed. A general view of the model is shown in Figure 33 below.

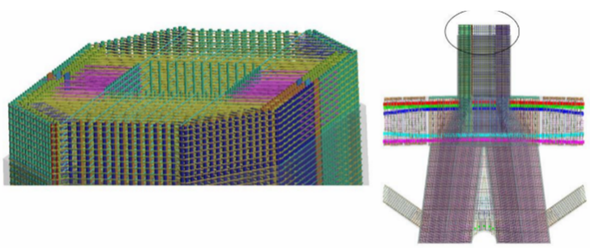

Figure 33. Reinforcement detail at top of tower BC (left) and overall elevation (right).

The structure was analysed under the following four load cases that were identified as critical under the linear simplified models:

1. Strength III for the maximum longitudinal bending on the upper tower.
2. Strength III for the maximum torsion in the lower tower.
3. Strength I for the maximum axial load in the upper tower.
4. Strength III for the maximum transverse bending in the upper tower.

For every load case, the appropriate boundary conditions with all accompanying forces, along with a linear transition, was modelled.

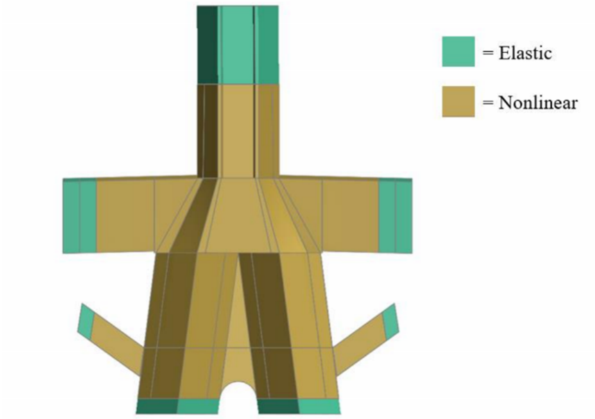

Figure 34. Modelling areas with explicit reinforcement and linear transition.

The results in all the selected cases showed that the nodal zone has additional capacity over the strength combination. As an example, the figure 35 below shows the moment displacement curve for the critical case 1 (maximum longitudinal moment in the upper tower).

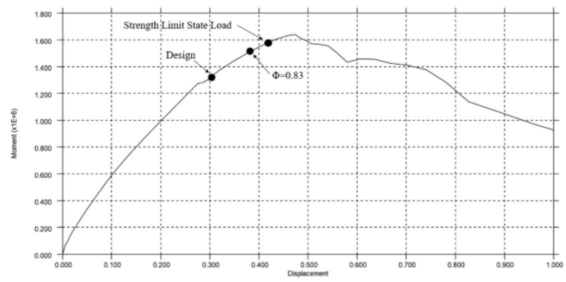

Figure 35. Resultant displacement vs resultant moment for Case 1.

Also, principal stresses and stresses in the reinforcement under design loads were obtained, as an example the figure below shows the principal stresses for Case 1 (longitudinal bending).

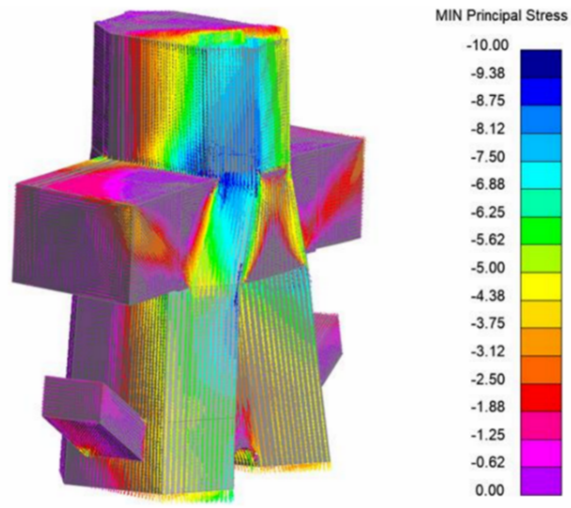

Figure 36. Concrete principal stresses under maximum longitudinal bending.

3.4 Upper tower

As indicated in the global analysis and general paper [1] the designers changed the cable connection at the tower from a saddle to a double anchor steel composite box which is the traditional design in concrete towers.

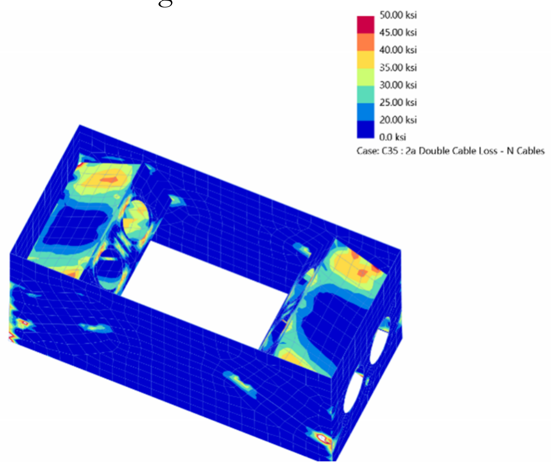

Figure 37. Steel anchor box at the tower. Von mises stresses for double cable loss Extreme load case

Local models for the boxes were developed in the Arup in-house FEM package Oasys GSA and in Abaqus separately, including the interaction with external concrete. These models, in combination with the global model were used to verify the Service, Strength and Extreme load cases (see for example Figure above).

4. Conclusions

A bridge as complex as NHB requires a significant number of local models using state of the art analysis tools to ensure the verification according to the codes of local areas where the global analysis model does not provide accurate information but also to calibrate the global model. The Authors, responsible of the design and independent technical reviewed developed several models on each area using different approaches and software packages arriving to similar conclusions in all cases.

Acknowledgements

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