

New Harbor Bridge in Corpus Christi (Texas, USA) **Erection Engineering of the World's Longest Span Precast Segmental Bridge**

Nuevo puente en el puerto de Corpus Christi (Texas, USA)

Ingeniería de Construcción del puente atirantado de dovelas prefabricadas de mayor luz del mundo

Jonathan Aylwin ^a, Quentin Marzari ^b, Matt Carter ^c,

Manuel Contreras Pietri ^d, Alberto Muñoz Tarilonte ^e, Antonio Martínez Cutillas ^f

^a Civil Engineer. PE. Arup. Associate. Arup. Jonathan.Aylwin@Arup.com

^b Civil Engineer. PE SE. Arup. Senior Engineer. Quentin.Marzari@Arup.com

^c Civil Engineer. PE SE. Arup. Director and Engineer of Record. Matt.Carter@Arup.com

^d Civil Engineer. PE. Carlos Fernandez Casado SL. Director. MContreras@cfcs.com

^e Civil Engineer. Carlos Fernandez Casado SL. Senior Engineer. AMT@cfcs.com

^f Doctor Ingeniero de Caminos. Director. Carlos Fernandez Casado SL. AMartinez@cfcs.com

RESUMEN

New Harbor Bridge es un puente atirantado emblemático situado en la ciudad de Corpus Christi, Texas. La superestructura se compone de dos cajones de hormigón pretensado prefabricado, conectados con Pórticos-Delta y sustentados por un plano central de tirantes. La superestructura fue prefabricada fuera de la obra principal, transportada y ejecutada en avance en voladizo. Tras completarse su construcción en 2025, se convertirá en el puente de dovelas prefabricadas con mayor luz del mundo y en el más ancho con uso de pórticos-delta. Este artículo describe los retos de construir un puente de esta magnitud y complejidad, además de las soluciones desarrolladas por el equipo de diseño y construcción.

ABSTRACT

The New Harbor Bridge is a signature cable-stayed bridge located in Corpus Christi, Texas. The superstructure is formed of two precast concrete box girders, connected via delta frames, with support from a central plane of stay cables. The precast concrete bridge was constructed offsite and assembled using balanced cantilever construction methods. Once opened in 2025, the bridge will become the longest precast segmental span in the world and the widest ever delta frame bridge. This paper discusses the engineering challenges of erecting and closing a bridge of this scale and complexity, as well as the solutions developed by the design-build team.

PALABRAS CLAVE: Ingeniería de construcción, atirantado, dovelas prefabricadas, avance en voladizo.

KEYWORDS: Erection engineering, cable-stayed, precast segmental, balanced cantilever.

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 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



Figure 1. The New Harbor Bridge is a striking landmark for the Coastal Bend community.

(FDLLC) for the replacement of the existing Harbor Bridge. The project consists of a signature precast concrete segmental cable-stayed bridge (the New Harbor Bridge) and approximately 9.2 km (5.7 miles) of bridge and connecting roadway. Arup-CFC is the Engineer of Record for the New Harbor Bridge (Figure 1).

This paper discusses the unique aspects of the erection engineering, developed by the design-build team, to construct this iconic and highly complex bridge. Separate papers have been written for the Global Analysis, Local Analysis and Geometry Control [1]-[3].

2. The New Harbor Bridge

The New Harbor Bridge (Figure 2) is a five-span cable-stayed bridge with a total length of 1,012 m (3,295 ft) and a main span of 506 m (1,661 ft). In cross section (Figure 3) the bridge deck is formed of two precast box girders connected by regularly spaced precast delta frames and an in-situ median slab.

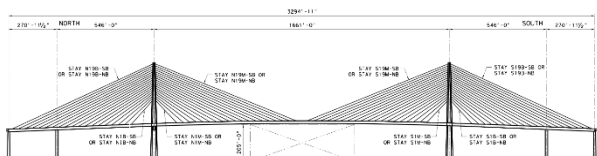


Figure 2. Elevation of the New Harbor Bridge.

The 45.4 m (149 ft) wide bridge will carry three lanes of vehicular traffic with two 3.0 m (10

ft) shoulders in each direction, and a shared-use path for pedestrians and cyclists on the northbound side.

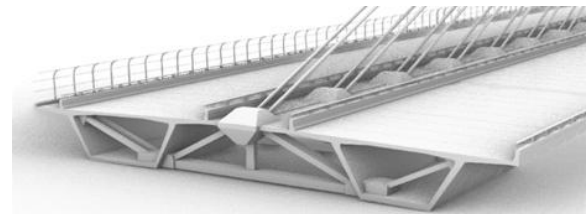


Figure 3. Bridge deck cross section.

The 76 pairs of stay cable are positioned along a single central plane and arranged in a semi-fan configuration. Stay lengths vary from 55 m (180 ft) to 270 m (880 ft).

3. The construction methodology

The bridge is being constructed using balanced cantilever methods, with segments lifted from land near each tower and transported along the cantilevers. By avoiding lifting segments from the channel, maritime traffic at the Port of Corpus Christi remained unimpeded.

Each cantilever heading contains 19 “construction cycles”. A typical cycle consists of four northbound box girder segments, four southbound box girder segments, one delta frame, a cast-in-place median slab and twin stay cables. The stay cables are installed at the end of the cycle and restressed midway through the next cycle.

4. The precast components

Most of the bridge deck was precast offsite, under controlled conditions, and then assembled at the erection site. In total, there are 698 precast segments and 84 delta frames (Figure 4).



Figure 4. Precast components were stored in the yard and transported 35 km (22 miles) to the erection site using specialised haulers.

4.1 High performance concrete

The bridge is located along the South Texas Coastal Bend and exposed to harsh environmental conditions. Despite this, the project required a 170-year extended service life for select non-replaceable concrete components.

The high-performance concrete designed to meet this requirement used probabilistic models to predict the long-term permeability and chloride ion diffusion, whilst also achieving a minimum compressive strength of 69 MPa (10 ksi).

4.2 Box girder segments

The box girders were cast using short-line casting methods. Each segment is typically 4.3 m (14'-0") deep, 2.8 m (9'-4") long and weighed about 100 tonnes. The southbound segments were 17.2 m (56'-6") wide and the northbound segments were even wider at 20.6 m (67'-6").

Bespoke pier segments were cast in separate molds and included diaphragms and additional post-tensioning. These heavy segments were either cast as two half-segments or as a single shell with the diaphragm cast in-situ.

4.3 Delta frames

The delta frames are approximately 19.5 m (64'-0") wide with a central node containing the stay cable lower anchorage. Each one weighed about 125 tonnes.

Shear keys either side of the delta frame slot into blockouts located at the interior webs of the adjacent box girder segments.



Figure 5. There were two molds for the delta frames.

4.4 Weighing the bridge

For the construction of a heavy cable-stayed bridge, any variation between theoretical and actual weights can significantly impact the geometry and locked-in forces. The precast components were therefore weighed prior to erection to allow the stay cable forces to be fine-tuned to the measured weight of the bridge. The average segment weight was 0.3% higher than predictions, with a standard deviation of 1.6%. In total, approximately 75% of the bridge deck was weighed.

5. The Erection Manual

Cable-stayed bridges are highly indeterminate structures. How closely the final locked-in forces match the target forces depends heavily on ensuring the analysis and construction accurately represent one another.

Centred around FDLIC's method of construction, the Erection Manual was used to provide the detail needed to achieve the required internal forces and geometry after completion of

construction, without overstressing any of the bridge components during construction.

The Manual contained drawings of the construction sequence, construction equipment and temporary works required to transport, lift, manipulate and assemble the bridge components. Structural calculations were developed for each stage of erection.

5.1 Construction flexibility

Construction rules were developed to allow FDLIC to make on-site adjustments to the erection sequence, within a typical cycle, without requiring real-time evaluation from the Engineer Of Record.

The use of rules, rather than a rigid sequence, added a significant number of permutations to analyse and evaluate. This computational effort was made feasible by using databases and bespoke scripts for automation.

5.2 Database workflows

A Structured Query Language (SQL) database was used to store the outputs from the various models and produce the design combinations for each stage of construction. This approach was also used for the in-service design.

A tool was written by Arup-CFC to query the database and produce a dashboard-style interface. These easy-to-read ‘stage-by-stage’ sheets were included in the Erection Manual to document each construction stage (Figure 6).

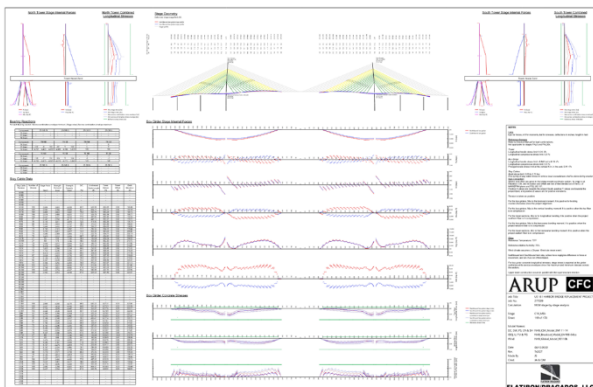


Figure 6. Stage-by-stage sheets detail the geometry, internal forces, stresses, reactions, and Strength and Service evaluations.

This functionality was useful for visualizing the behaviour of a bridge of this scale and complexity.

5.2 Analysis model

The construction stage and in-service analysis was performed using the finite element software SOFiSTiK.

The global construction stage analysis model was represented as a grillage of beam elements and run using an elastic solver (Figure 7). The model outputs were used to produce the information necessary to construct the bridge, such as stay cable installation forces, closure forces and stage deflections.

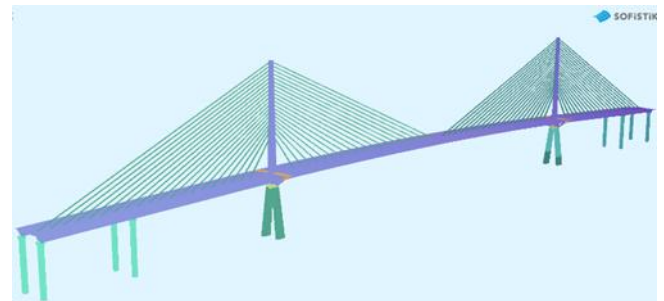


Figure 7. Each of the 400+ construction stages had a unique structural system and construction loading.

Bespoke local models were built to analyse specific structural elements and key construction stages in greater detail (Figure 8).



Figure 8. Local models were built using a combination of shell and brick elements.

5.4 High-wind events

The bridge is being built in a hurricane wind climate, wind effects dominate the critical stages of construction due to the size of the cantilever and width of the deck.

Wind buffeting analysis was performed in the finite element software RM Bridge. Wind parameters were derived from climate studies

and wind tunnel testing for structural behaviour validation (Figure 9).

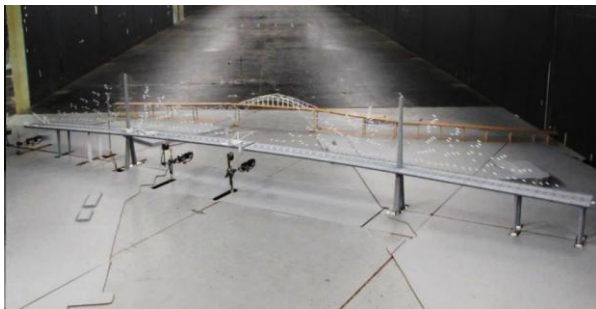


Figure 9. Aeroelastic testing was conducted at critical construction stages.

Wind effects throughout cantilever construction were evaluated for 20-year return period wind event. Reduced return periods were used for specific short-duration closure activities that could be completed within a weather window.

Wind forecasts were continuously monitored during construction, especially during the hurricane season. Special procedures were developed to prepare the bridge for high winds, such as pausing construction at preferred stages within the cycle and shoring deck equipment.

6. Construction stage analysis

The unprecedented scale of the bridge results in the need for the structure to be finely tuned to the optimum combination of bending moments in the superstructure and tower as well as reaction forces at the transition and intermediate piers. The stay cables, which are the ‘levers’ that are used for tuning, must also remain within their force limits.

6.1 Box girder verification

During construction, the box girders experience multiple cycles of hogging (negative) bending due to segment erection and sagging (positive) bending due to stay cable effects. The critical location for peak flexural stresses was generally located about three cycles behind the leading edge. The optimum stay cable installation force,

and subsequent restress, was calculated to keep the longitudinal tensile stresses within the deck slab and soffit slab below 4.1 MPa (0.60 ksi) [5].

Additionally, there were dominant torsion and shear effects originating from the derrick crane, wide bridge deck and central plane of stay cables. As such, the project elected to limit principal tension around the perimeter of the box girder cell to 2.4 MPa (0.35 ksi) [5], despite AASHTO LRFD only requiring this limit be checked at the neutral axis of the webs.

Torsional demands were reduced by using practical rules, such as erecting the southbound segments before their northbound equivalents and limiting derrick crane picks at critical stages.

6.2 Main tower verification

Construction progress within each span was coordinated to ensure the main towers did not crack, thus ensuring linearity during geometry control. This was achieved by limiting longitudinal (vertical) tensile stresses to 5.2 MPa (0.76 ksi) [5].

6.3 Stay cable verification

At the Strength/ultimate limit states, the leading twin stay cables experienced their highest level of stress (about 70% GUTS) as the leading cycle reaches maximum cantilever during the construction of the median slab [6].

7. Constructing the substructure

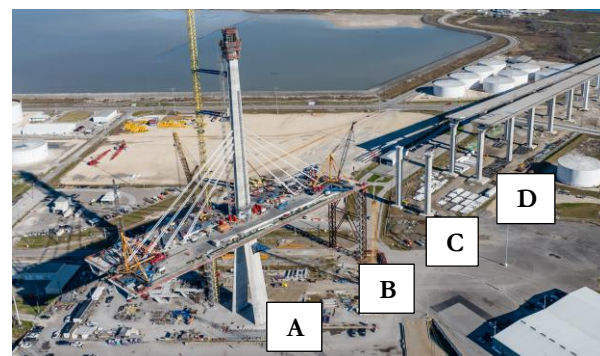


Figure 10. [A] main tower, [B] temporary pier, [C] back span pier, and [D] transition pier.

The substructure elements are founded on drilled shafts and above-ground pile caps.

7.1 Main towers

The main towers are a dominant aspect to the visual impact of the bridge (Figure 11). The lower tower comprises two tapered legs that connect at the bridge deck elevation. The mono-pole upper tower, located above the bridge deck, is 104 m (340 ft) tall and constructed in 20 lifts using jump formwork. Main Towers were built at the same time as deck was being erected.



Figure 11. The stay cable anchor boxes were cast into the upper tower and used as permanent formwork.

7.2 Temporary pier

A temporary pier was installed approximately half-way between the main tower and the back span pier to control out-of-balance bending within the main towers and provide aeroelastic stability to the superstructure.

Arup-CFC developed the performance requirements of the piers which were then designed by Flatiron's Construction Engineering Group (CEG). The steel structure was approximately 55 m (180 ft) tall.

7.3 Back span and transition piers

The back span and transition piers are up to 46 m (150 ft) tall and constructed in 10 lifts using jump formwork.

8. Erecting the superstructure

8.1 Tower table

The tower table (Figure 12) is a critical load path element for the superstructure. In-lieu of casting the tower table in place, FDLIC opted to precast each heading into 6 discrete segments, cantilevered from the central tower. Each segment was produced at each main tower utilizing the long-line casting method.

The first of these segments was connected to the main tower using a 23 cm (9 inches) cast-in-place closure joint. Subsequent segments were connected with epoxy joints. All joints were progressively stressed to one another with high-strength post-tensioning (PT) bars and traditional PT tendons.

The tower tables are propped with cast in-situ struts.



Figure 12. Construction of the tower table.

8.2 Erecting the first box girder segments

The first box girder segment within each heading was positioned using a ground-based crane and connected to the tower table with a 15 cm (6 inches) cast-in-place closure joint (Figure 13).



Figure 13. Dry-runs allowed for any minor misalignment of the PT ducts, across the joint, to be resolved in advance of segment erection.

8.3 Creating a working space

The initial segments continued to be erected using a ground-based crane until enough working space was formed to install the deck-mounted stiff-leg S-50 derrick cranes (Figure 14). These cranes operated at the free ends of the northbound box girders and were used to erect subsequent segments and delta frame to the cantilever tip.

The derrick cranes were mounted on bespoke steel sleds that clamped to the deck slab utilising high strength tie-down bars. The 7600 kN (1700 kip) vertical pick reactions governed many of the construction stages.

The derrick cranes launched forward on custom designed tracks to ensure the derrick was properly aligned for subsequent segment installation.



Figure 14. Main span derrick installation.

8.4 Segment staging

The ground-based crane picked segments from the ground to the bridge deck, after which a self-propelled modular transporter (SPMT) delivered the segments to the erection front.

Staging areas located at the base of the tower, bridge deck and erection front allowed a continuous delivery of segments to the derricks.

8.5 Typical segment erection

A segment manipulator was attached to the segment during erection. This contained lifting points for the crane to pick from and jacks within the manipulator to adjust the slope of the

segment to match that of the erection front (Figure 15).

Segmental Bridge Adhesive (epoxy) was applied to both faces of the match cast joints to provide lubrication at the concrete faces during segment manipulation and to seal the box-girder once cured. Each segment was stressed to the cantilever using high strength PT bars.



Figure 15. Segment erection with manipulator and stressing platform.

8.6 Cast-in-place closure joints

15 cm (6 inches) cast-in-place closure joints were located between consecutive casting units to allow on-site adjustments to be made to the projecting geometry of the bridge.

Tie-back beams, anchored to the previously erected segment, were attached to the manipulator to support the weight of the segment and hold the segment in a fixed position whilst casting and curing the concrete.

8.7 Delta frame installation

Accurate bridge steering was critical to maintaining the correct transverse spacing between the box girders. Too narrow and the delta frames will not fit, too wide and the delta frame shear keys will not interlock with the box girders.

A spreader beam, spanning between the box girders, was used to support the delta frame during installation. Hydraulic jacks mounted on sliding surfaces were then used to manipulate the geometry of the delta frame, in all degrees of freedom (Figure 16).

An in-situ stitch connected the shear keys within their respective blockouts. Transverse continuity post-tensioning was installed and stressed to tie the system together. This was initially stressed to 50%, after the median slab had reached sufficient strength, and then restressed to 100% following stay cable installation to balance the development of stress within the delta frame.



Figure 16. Delta frame installation process.

8.8 Median slab construction

The median slab between the box girders was poured in-situ, after which the transverse post-tensioning within the top slab was installed and stressed.

8.9 Stay cable installation

The stay cables were stressed from the lower anchorage with access provided by a temporary stressing platform spanning across three consecutive delta frames (Figure 17).



Figure 17. Epoxy coated 7-wire strand was installed and restressed using a mono-strand. Stays cables varied from 70 strands to 121 strands.

Stay cables were installed and restressed to target chord forces for a particular arrangement of construction live loading; thus,

ensuring the final locked in force is achieved after their removal.

A load monitoring system was installed within the upper anchorage for in-service monitoring purposes. This was also used at specific stages during construction to evaluate the global distribution of the stay forces.

9. Closing the bridge

There were four closure operations on the bridge: three in the back span and one in the main span. Closures were aligned and blocked during periods of minimal thermal gradient to achieve geometry objectives under the reference temperature conditions.

9.1 Temporary pier closure

The temporary pier segment, located in Cycle 6, was erected following typical segment erection methods. After this, a counterweight was installed to the southbound box girder to balance the construction live load and thus reduce the locked-in torsion.

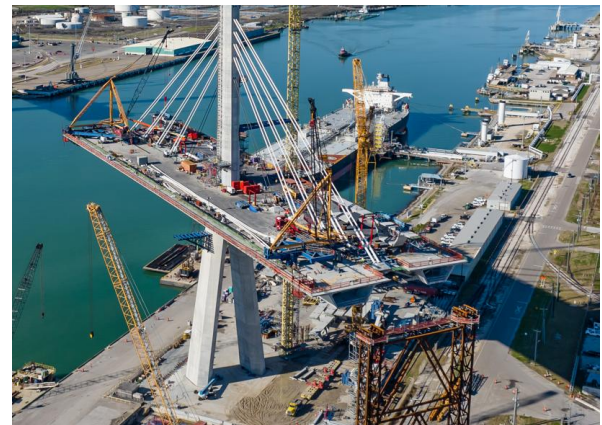


Figure 18. Cantilever deflections hinder closure operation. To prevent this, the free end of the back span was vertically jacked 5 cm (2 inches).

Jacking the back span created sufficient reaction to prevent decompression due to environmental effects. Shims were then placed between the segment and cap beam to form longitudinal, transverse and vertical restraint.

Vertical tiedowns, installed to 13,300 kN (3000 kip) per box girder, prevent uplift.

9.2 Back span pier closure

The back span pier segment, located in Cycle 13, was staged above the pier and secured in advance of closure. A counterweight was installed on the southbound box girder.

The stay cable 12 installation force was calculated to ensure the cantilever aligned approximately 5 cm (2 inches) lower than the pier segment, allowing the final alignment to be made, under more controlled conditions, using a strongback and jacking system (Figure 19).

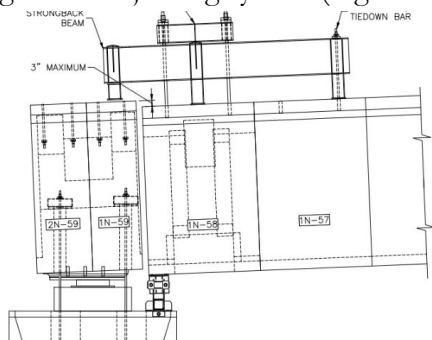


Figure 19. PT bars were aligned and coupled within the 15 cm (6 inches) closure.

Grout blocks were poured at the four corners of the box girder cell and clamped by partially stressing the PT bars. This acted to lock the connection and thus retain geometry whilst pouring the closure. The permanent vertical tiedowns to the pier were installed last.

9.3 Transition pier closure

The transition pier segment was staged above the pier and secured in advance of closure.

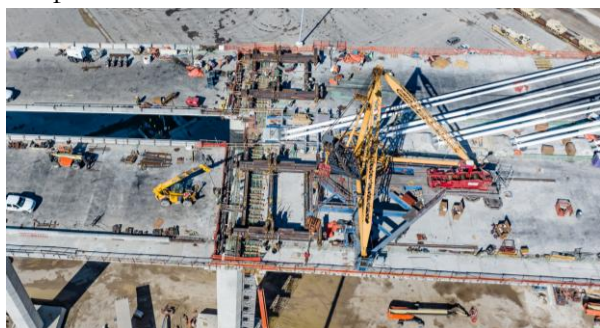


Figure 20. Asymmetry of construction live load was compensated by jacking higher on the northbound, rather than counterweighting the southbound, to reduce overall vertical jacking requirements.

Vertical alignment to the transition pier was achieved using a combination of stay cable force and strongback jacking (Figure 20). The optimum balance was found between not overstressing the main span soffit (due to stay install) and not locally overstressing the webs below the strongback (due to vertical jacking).

Once aligned, the 253 m (830 ft) long back span cantilever becomes susceptible to dominant thermal and wind effects. To resist this, a blocking system was installed within the joint and clamped by partially stressing the permanent PT tendons (Figure 21).

The 1.8 m (5'-10") cast-in-place closure was poured and the PT tendons stressed in phases to develop strength as the concrete cured. The permanent vertical tiedowns to the pier were stressed last to complete the closure.

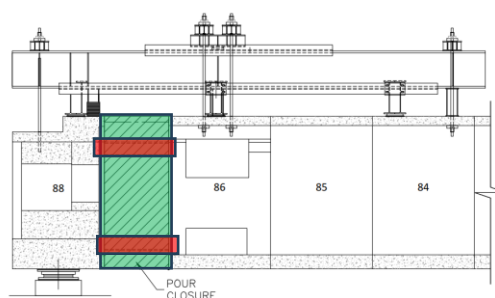


Figure 21. Two strongbacks and 4 struts per box girder allow the construction the cast-in-place joint.

9.4 Main span closure

The central closing segments were erected following typical segment erection methods, cantilevering from the northern bridge. The resulting asymmetry of permanent loading, causing the northern cantilever to vertically lower 25 cm (10 inches) relative to the south, was corrected using strongbacks.

A temporary blocking frame was installed inside the box girders to join the spans. Each strut, designed for factored demands of 29,000 kN (6,500 kip), was clamped by partially stressing the permanent PT tendons that pass through the closure joint (Figure 23).



Figure 22. The bridge was longitudinal jacking apart by 3,600 kN (800 kip) to offset future creep and shrinkage effects at the base of the main tower.

A 1.0 m (3'-0") cast-in-place closure closed the bridge, with longitudinal PT tendons stressed in phases to develop strength as the concrete cures.

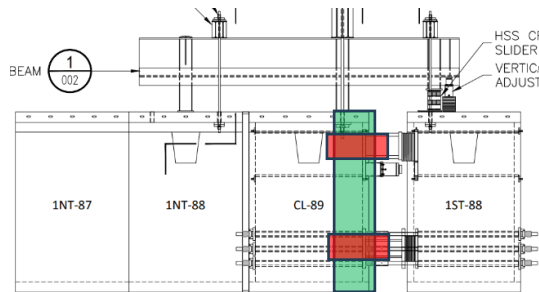


Figure 23. Two strongbacks and 4 struts per box girder allow the construction the cast-in-place joint.

10. Stay cable acceptance

A third restress was performed for stay cables 15 through 19 to lift the main span and compress the soffit slab (Figure 24). This was implemented as a force step change, which differs from the cantilever stage where force targets were used.



Figure 24. Stay cable forces were measured and used for Final Acceptance according to PTI [6].

11. Conclusion

The many unique construction challenges specific to the New Harbor Bridge result from the structural form and unprecedented scale. The solutions described in this paper were realized through detailed engineering and an effective design-build partnership.

Acknowledgments

The authors would like to thank the Texas Department of Transportation and Flatiron Dragados LLC for their permission to publish this article.

References

- [1] Carter M., Martínez Cutillas A., Sanchez M., Contreras Pietri M., Long C., Martin Martínez B. Global Structural Analysis of the World's Longest Span Precast Segmental Bridge. ACHE Congress, Granada June 2025.
- [2] Carter M., Martínez Cutillas A., Sanchez M., Contreras Pietri M., Tarasuik L., Fuentes S. Local Structural Analysis of the World's Longest Span Precast Segmental Bridge. ACHE Congress, Granada June 2025.
- [3] Marzari Q., Aylwin J., Carter M., Contreras Pietri M., Martínez Cutillas A., Pérez Monreal A. Geometry Control of the World's Longest Span Precast Segmental Bridge. ACHE Congress, Granada June 2025.
- [4] Alywin J., Marzari Q., Carter M., Contreras M., Pérez A., Marines A., Construction Engineering on the US181 New Harbor. International Bridge Congress, Austin. July 2024.
- [5] AASHTO LRFD Bridge Design Specifications, 7th Edition + Interim Revisions (2015).
- [6] PTI DC45.1-18 Recommendations for Stay Cable design, Testing and Installation