New Harbor Bridge in Corpus Christi (Texas, USA) Geometry Control of the World´s Longest Span Precast Segmental Bridge

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

Control Geométrico del puente atirantado de dovelas prefabricadas de mayor luz del mundo

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RESUMEN

El New Harbor Bridge es un puente atirantado emblemático situado en Corpus Christi, Texas, EE.UU. Tras completarse su construcción en 2025, será el puente de dovelas prefabricadas con mayor luz y con uso de delta-frames más ancho del mundo. Los elementos principales del tablero, cajones y delta-frames fueron prefabricados e instalados en avance en voladizo. La planificación y ejecución del control geométrico fue clave para asegurar un puente funcional y ajustado al perfil objetivo, incluyendo una conducción suave entre las diversas conexiones entre componentes. El presente articulo describe los desafíos enfrentados, 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, USA. Once complete in 2025, the bridge will become the longest precast segmental span and widest delta frame bridge in the World. The main components of the superstructure, box girders and delta frames, were fabricated offsite and assembled using balanced cantilever construction methods. The geometry control planning and execution was key in achieving a functional bridge on the target profile and with smooth riding across the various component interfaces. The unique challenges this presented are described in this paper, alongside the solutions developed by the design-build team.

PALABRAS CLAVE: control geométrico, atirantando, dovela prefabricada, avance en voladizos, cerramiento
KEYWORDS: geometry control, cable-stayed, precast segmental, balanced cantilever, closure

1. Introduction

The existing Harbor Bridge spans the main shipping channel entrance to the Port of Corpus Christi, the USA's largest port for exported tonnage. Opened in 1959, the bridge is being replaced due to its low clearance, high maintenance cost and safety.

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. ArupCFC is the Engineer of Record for the New Harbor Bridge.

Figure 1. General view of the shipping canal with the old and new bridge during construction (April 2024).

2. The New Harbor Bridge

2.1 General description

The New Harbor Bridge is a five-span cable-stayed bridge: 82,6+166,4+506,3+166,4+82,6 = 1 004,3 m, integrally connected with two 163,7 m-tall inverted Y towers and articulated with two pairs of approximately 45,7 m-tall pier shafts on each back span side. In cross section the bridge deck is formed of two precast box girders connected by regularly spaced precast delta frames.

The 45,4 m wide bridge will carry three lanes of vehicular traffic with two shoulders in each direction and a shared-use path for pedestrians and cyclists on the northbound side.

The twin stay cable system (76 pairs) is positioned along a single central plane and arranged in a semi-fan configuration. Stays anchor in the deck every 11,4 m and in the top portion of the upper tower every 3,4 m.

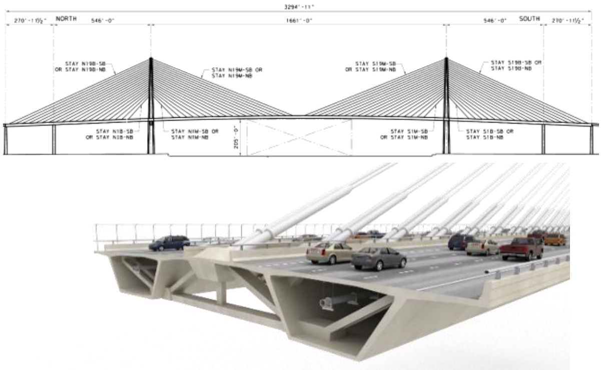

Figure 2. Cable-stayed bridge elevation (top) and cross section isometric rendering (bottom).

The deck profile features a central 609,6 m vertical parabolic curve with 4% end slopes and a navigational clearance of 62,5 m . The 20,4 m increase from the clearance of the existing bridge will open the port to future generations of shipping.

2.2 The prefabricated components

Most of the bridge deck is precast offsite, approximately 35 km away, and assembled onsite. In total, there are 698 precast box girder segments and 84 precast delta frames.

2.2.1. Box girders

The typical northbound and southbound segments are approximately 4,3 m deep, 2,8 m long and 20,6 m/17,2 m wide, respectively.

Bespoke pier segments included diaphragms and additional post-tensioning. To control picked weights, they were either split into shorter segments, or their shell was cast in the precasting yard whilst the diaphragms were cast in situ.

2.2.2. Delta frames

The delta frames are approximately 19,5 m wide with a central node containing the stay lower anchorage. Shear keys either side of the delta frame slot in a blockout at the interior webs of the adjacent segments. These segments each included a diagonal strut that joined with the delta frame to form a transverse truss system across the width of the bridge.

2.2.3. Anchor boxes

Steel anchor boxes are used in the upper tower to transfer the stay anchor forces to the adjacent shaft concrete section. They were fabricated offsite.

2.2.4. Tower table

The solid 4,3 m deep and 6,1 m wide tower tables form the integral connection between the deck and the towers. They were broken down into precast segments around 2,1 m in length.

2.3 The construction methodology

The bridge towers and piers were built in lifts, whilst the bridge deck is being constructed using balanced cantilever methods. For more details on the construction process, please refer to the paper dedicated to erection engineering.

3. Analysis

3.1 Setting the target geometry

The target geometry was set as the design Plans geometry at 21,1°C (70°F) and 30 000 days after the end of construction (referred to as "PAL30k", PAL standing for permanently applied loads). The two exceptions are:

- The pier shafts, which were built to the Plans dimensions at the time of erection.
- The UTs, which were targeted to be plumb in the undeformed state, instead of at the reference time.

The implication of both exceptions was evaluated in the design phase.

3.2 Structural analysis

Cable-stayed bridges are highly indeterminate structures. As such, the final state of the structure, including its geometry, heavily depends on the construction path selected.

The Erection Manual is a collaborative document centered around FDLLC's method of construction. It highlights the construction equipment needed and the steps to follow to build the bridge.

The design global analysis finite element model described in the dedicated paper in the series submitted for this congress formed the base for the construction stage analysis model. In an iterative process, the design model was modified to add the appropriate level of granularity required to support the Erection Manual. In particular, the number of construction stages was expanded to 400+, code prescribed construction live loads were replaced by planned equipment, material properties and self-weight estimates were refined. Including transverse post-tensioning loading in the model was found to be important, as it generated compatibility deformations.

3.3 Precamber

The key output for geometry control from the construction stage analysis model was the set of structural deformations. These were reviewed component by component and degree-of-freedom by degree-of-freedom to determine how to offset them to achieve the target geometry.

3.3.1. Tower precamber

The tower pile cap foundations were built high to compensate for total settlement.

The lower tower legs were built with a longitudinal deflection precamber.

The upper towers were built with a vertical precamber to preserve the distance between deck level and the stay upper anchors. Rather than spreading it in small increments at each lift, this was instead lumped into a single value and added to the top elevation of the lift right under the SC anchor region.

3.3.2. Delta frames

Delta frame deformations were small enough not to require a precamber.

3.3.3. Box girders

For the precast segments, a hybrid precamber strategy was adopted.

For small deformations, the precamber was lumped at cast-in-place 152 mm construction joints typically distributed every 12 or 16 segments. Such approach was followed to offset:

- The axial deformation of the deck under the stay longitudinal reactions.
- The inward transverse deformation of the northbound and southbound girders due to the central stay plane longitudinal

reactions and shear lag in the cross section. The first segment after each joint was steered outwards.

For larger deformations, the precamber was distributed among all segments. This was used to offset:

- The vertical deformation of the deck at PAL30k resulting from the chosen stay tuning.
- The twisting deformation due to the non-closure of the rotational lack-of-fit at the installation of the delta frames (see figure 3 below).

  Under dead load, transverse deformation of the cross section leads to opposite twisting of the box girders about the longitudinal axis. When erecting a new cycle, the cantilevered segments inherit this rotation from the previous cycle. It was found not practical (the system is too stiff) to restore the relative geometry of the box girder directly adjacent to the delta frame to be installed prior to casting the stitches between the three elements. As such, this rotation of the girders is locked-in and as erection progresses it leads to a corkscrewing deformation along the length of the bridge.

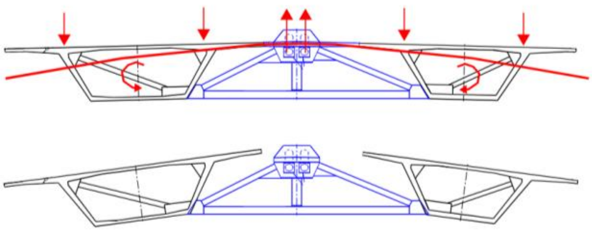

Figure 3. Illustration of the non-closure of the rotational lack-of-fit at the installation of the delta frames

The set of cambered coordinates at each segment joint formed what is referred to as the casting curves and is one of the key deliverables produced in the Erection Manual.

4. Prefabricated component geometry control

4.1 Box girders

The segments were cast using short-line casting methods. In terms of geometry control, the goal is to approximate the cambered geometry as a polygonal line, i.e. curvature is replaced by break angles at the segment joints. For example, casting the target vertical curve would require: $2{,}8 * \frac{4\%-(-4\%)}{609{,}6} = 0.04\%$ vertical break angle at each segment joint within this region.

Even if great care is taken during the casting process, small errors are inevitable. In particular, it is hard to place the match-cast segment perfectly, or it may move slightly as the wet-cast segment is cast. If not corrected, these can lead to large positional errors when projected to the end of each construction unit. Hence, a casting engineer always reviewed the as-cast relative geometry between two segments and corrections were made to the target casting geometry of the following segment to be cast. This process allowed control of the divergence between cambered and as-cast geometry.

4.2 Delta frames

For ease of fabrication, delta frames were cast laid flat using steel forms on a concrete footing. This rigid casting bed allowed to establish a fixed local coordinate system. It was used to set the position and orientation of the stay recess pipe-anchor bearing plate assembly, which was a key operation. Indeed, as-cast stay anchor misalignments can generate localized bending demands in the cable.

The following alignment tolerances were used for the fabrication of the delta frames:

- Each recess pipe was welded orthogonal to its bearing plate to +/-0.1° tolerance.
- Each bearing plate was placed at a fixed location in the DF forms to a tolerance of +/-6.4 mm.

- The recess pipe orientation tolerance of +/-0.125 ° was verified using a digital inclinometer for the vertical angle and direct measurements for the horizontal.

Figure 4. Photo of a delta frame pre-pour

Following casting, recess pipe angles were re-measured and as-cast values recorded.

4.3 Upper tower anchor boxes

A progressive assembly of the steel anchor boxes was performed in the fabrication shop to ensure they would stack up vertically plumb. Each box corner had a position a tolerance of +/-3.2 mm.

Steel shims were welded to the boxes as needed to meet this requirement.

4.4 Tower table

Tower table segments were cast using the long-line casting method, i.e. the six segments on each side of a tower nodal zone were cast together on a stiff casting bed. Like the box girder segments, each segment received its set of six survey work points, which were surveyed in the as-cast configuration before moving the segments apart.

5. Erection geometry control planning

The behavior of an actual structure will always be a bit different from that predicted by models. Recognizing this, planning for onsite geometry control in advance of erection was important.

5.1 Sensitivity analyses

Sensitivity analyses were performed to evaluate the influence of select structural parameters on the final geometry of the bridge. These are presented below with the strategies adopted to identify and control divergences.

5.1.1. Bridge deck self-weight

The most impactful parameter on geometry is the self-weight of the bridge deck. Indeed, only a 5% variation in deck self-weight, if left uncorrected, would lead to 1,0 m change in deflection at the middle of the main span.

To limit this risk, the Erection Manual required enough precast components to be weighed. In the end, 93% of the segments were, which corresponds to 74% or the bridge deck total self-weight. Obtained measured-to-theoretical weight ratios for the typical segments had a mean of 100,3% and a standard deviation of 1,5%. The model was updated accordingly with adjustments to the stay prestress.

5.1.2. Bridge stiffness and time-dependent effects

The stay tuning selected in design limited deck vertical deflections at the end of construction to 76 mm/305 mm, respectively for the back and main span sides, and the longitudinal deflection at the top of tower to 178 mm.

The numbers were so small that naturally reduced the sensitivity of the bridge final geometry to bridge stiffness and concrete creep. Using precast elements limits the risk of noticeable creep and shrinkage variations.

5.1.3. Construction equipment

During cantilever construction the bridge deck is an active construction site with a lot of equipment on it. Capturing its presence matters for geometry control as variations could impact the locked-in internal forces. For example, if there is more equipment near the leading edge, it can affect the prestressing of the stays.

To mitigate this risk, the Erection Manual clearly states that only construction equipment approved by the Engineer of Record shall be used on the bridge deck. Moreover, the position of each key construction equipment at each stage of construction is clearly shown in the Erection Manual sequencing drawings. Finally, the heavier equipment, e.g. the 200 ton S50 derrick cranes, was weighed onsite for added confidence.

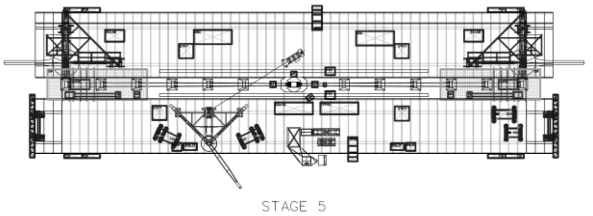

Figure 5. Construction equipment positioning on the deck shown in a sequencing drawing

5.2 Surveying during erection

Ultimately, the actual behavior of the structure is what drives decision making onsite and surveys are the way to acquire the key data to achieve this. The Erection Manual listed the surveying requirements for the erection of the bridge.

5.2.1. Local surveys

Local surveys were those essentially unimpacted by the global behavior of the bridge. They could be performed at any time of the day with few restrictions and were typically used for the bridge component setups

5.2.2. Global surveys

They were intended to capture a snapshot of the bridge at a given time. These were performed within 2 h from sunrise to limit temperature gradient effects, with surveying methods allowing the removal of bridge oscillations, in a balanced back span/main span configuration and under strict construction equipment freeze and mapping rules to maximize accuracy. Schedule wise, they were typically specified around stay stressing or closure operations.

The stay forces were measured using deck anchor lift-off for the leading cable and DYWIDAG DYNA Force® System at the top anchors for all the stays. Geometric survey was conducted using total station, shooting the precast component survey work points established in the precasting yard, or prisms installed post-pour at key discrete locations for the upper tower.

With a bridge of this size, the survey regime prescribed was quite demanding on the survey team. As surveying technology evolves, the possibility of acquiring geometric data automatically would be beneficial, improving accuracy, robustness and increasing the quantity of data acquired at a given time.

6. Erection geometry control implementation

6.1 Evaluation of onsite structural global behavior

As much as planning can be done, variations, usually in construction equipment on the bridge deck or achieved stay force, happened. Before any evaluation, it was thus necessary to normalize survey data to compensate for these differences before comparing against the latest model stage results. The evaluation of the bridge behavior was performed independently and in parallel by Arup and CFC, each with their own tools and models. Close coordination between the two teams allowed to resolve differences quickly and not impact the project schedule.

6.1.1. Projections

The main metrics used in forward processing were geometric projections for positions and slopes, from the segment surveyed at the cantilever leading edge to the end of the construction unit being erected, and at PAL30k. The segments remaining to be erected in a unit are virtually and rigidly attached to the front of the cantilever per the relative as-cast geometry to the projecting segment, and the remaining deformations from the surveyed stage to PAL30k are added.

Because projections rely on the slopes of the leading segment, which are sensitive to survey accuracy, in global surveys it was typically requested to survey all survey work points on the leading two segments. This gave two data points for projections to compare against one another.

6.1.2. Progression

Once bridge deck components are erected, the change in their state between two stages of construction only depends on the change in loading between the two. A pair of normalized global surveys can thus be used to derive a step change to compare against that in the model. Subsequently, the latter can be adjusted, if needed, to increase the accuracy of theoretical predictions and make future corrections.

The two examples below illustrate how progression tracking was used:

If step changes in geometry along a cantilever and in axial force in the leading stay are collected between the stages right before and after that same stay stressing operations, one can isolate the deformations due to an increment in stay cable force and essentially estimate the stiffness of the bridge deck.

If segment vertical deformations and stay forces are tracked from the moment they are installed onwards, one can potentially verify the dead load is correctly estimated and/or if the target stay stress is achieved. Indeed, assuming the cantilevers are deflecting more than expected, if the stay forces are on average higher/lower than anticipated then it will suggest the former/latter, respectively.

6.2 Site setups

Several bridge components required geometry control guidance at installation to ensure they were heading the right way within tolerances.

6.2.1. Tower table

To ensure the tower table re-assembled adequately on site it was paramount to position the first cantilever segments separated from the tower node by a 229 mm construction joint. The positioning of its first cantilever segments was constrained due to the amount of transverse post-tensioning. For slope setting, the difference in elevation between opposite rows of elevation bolts was used. Centerline hairpins controlled the transverse position.

Figure 6. Tower table and upper tower first few lifts

6.2.2. Upper tower

The upper towers were built in parallel with the superstructure, therefore, the lifts were erected tangentially from the lift underneath, not to be impacted by any potential bridge unbalance.

To achieve this goal, a set of two sufficiently apart prisms were installed on the tower bottom lift longitudinal faces and surveyed at the beginning of construction in a balanced back span/main span configuration, before any stay was installed. It formed a reference line for plumbness in the undeformed configuration. A new prism was added every other lift as construction progressed and global survey was performed with the three top prims shot, transferring the reference plumb line to the new prism and the one below.

Anchor boxes were set per the projected tangent derived from the top two prisms and acted as an internal permanent formwork and spine guiding upper two geometry. Preassembly was good and corrections were sporadically used to achieve the H/2000 tolerance cone.

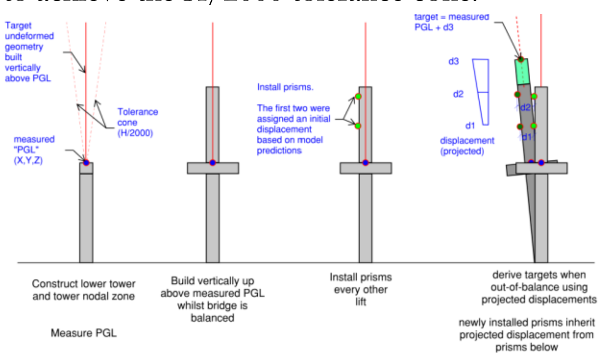

Figure 7. Schematics upper tower geometry control

6.2.3. Box girder segment erection

The most important site setup for geometry control was the positioning of the first segment of each construction unit as it represented opportunities to change a heading course by adding break angles. If a 0.30% (less than 0.2°) vertical angle is added at the construction joint, this will translate into a change in elevation at the end of a 36.6 m long unit of 112 mm.

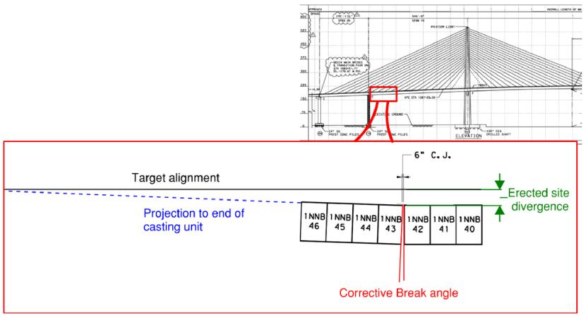

Figure 8. Schematic of construction joint correction

A tool developed by ArupCFC was loaded with theoretical setup data and was used by FDLLC onsite to evaluate tentative surveyed setups until it matched within the specified erection tolerances, +/-3.2 mm for positional coordinates and +/-0.15% for slopes. Since the setup was local and relative to the previously erected segment, it could be carried out independently of other construction activities.

Setting the very first segment adjacent to the tower table was a special case. The position setup was challenging due to the highly constrained interface with many connecting elements without any form of match casting. Dry runs helped identify the best fit and the minor remediations needed to achieve it.

The erection of the segments used epoxied joints and was typically fast. Local surveys were performed at the beginning of construction to verify if segments geometry onsite matched with the as-cast. Divergences measured were typically small and it was hard to discern whether they were real or due to survey variability.

6.2.4. Delta frames

During alignment, a delta frame was supported by a spreader beam spanning across the gap between the two segments and cradle manipulator fitted with hydraulic jacks which allowed geometric adjustments.

Figure 9. Photos illustrating a delta frame setup

The delta frame setup was a highly constrained operation due to the four interfaces with the adjacent segments. Making the mirroring duct ends of the two delta frame transverse continuity tendons concentric within +/-13 mm at each bottom shear key interface left only two degrees-of-freedom to align: the transverse offset position and longitudinal slope.

Accurate and coordinated steering of the corresponding northbound and southbound headings was critical to maintaining the appropriate transverse space for the delta frame to slot in. Too narrow (76 mm tolerance), and the delta frames will not fit, too wide (102 mm tolerance), and the delta frame shear keys will not interlock with the segments.

The longitudinal slope was adjusted by rotation of the delta frame about its bottom chord until a best fit was found between: alignment of the median slab tendon mirroring duct ends at the delta frame top node and adjacent box girder wing tips, and setting of the stay recess pipe vertical angle (+/-0.3°tolerance).

A tool was developed by ArupCFC which derived the theoretical setup data from adjacent segment survey and was used by FDLLC onsite to evaluate tentative surveyed setups until it was deemed to be in an overall best fit position.

6.2.5. Stay cables

The stays were installed to target chord forces provided by ArupCFC, with a tight erection tolerance of +/-2.5%. This ensured the high tuning of the stays was closely followed onsite.

Achieved onsite stay forces were tracked and, when necessary, a correction was applied to the stressing data of the next stay to be installed to limit the potential for unwanted flexural build-up in the deck, and the associated geometric divergences. Close coordination between Arup and CFC teams allowed to reach agreement on stay stressing data, a key deliverable to site, with sometimes turnaround times as low as 24 hours.

Essentially, the correction approach detailed above led to a decoupling between the locked-in force goal, controlled via stay stressing adjustments, and the geometric goal, mainly controlled with the in-situ construction joint corrections. That is two tools for two goals.

7. Closures

General geometry control led up to the delicate closure operations, joining the bridge deck to the piers, and the two cantilevers at the main span. Each had its own set of constraints. From a geometry perspective, the closure goal is for connecting parts to fit together within tolerance.

7.1 Temporary pier closure

The temporary pier is there to provide aeroelastic stability during construction. Because all connections are temporary, they were designed with sufficient construction tolerance to accommodate the incoming deck geometry.

7.2 Back span pier closure

Closure with the back span piers was semi-constrained from the geometric perspective. The split pier segments being also the first segments of the construction unit 5, after placement on top of their respective pier they were orientated to meet the targeted construction joint break angle. This rotational setup had negligible impact on the connecting elements.

The single bearing pedestal height under each pier segment was set to match the elevation of the adjacent cantilevers with an anticipation of the movements from the planned alignment loads used to lock-in reactions.

Setting the pier segments in plan was the most constrained part of this closure. The bottom part of the bearing assembly was shifted to accommodate the incoming geometry of the cantilevers, but minor remediation was necessary to improve the longitudinal clearance at the vertical tendon/pier cap interfaces.

In the end, even though 51 mm of transverse jacking had been allowed in design, it was not used. Temporary works held the cantilevers to the pier segment during the pour of the 152 mm closure joint.

7.3 Transition pier closure

Transition pier closure was similar to the back span pier closure in methodology, except it was more constrained by the need to accomplish a smooth transition with the as-erected approach span expansion joint segments.

Vertical jacking was performed using strongback beams in coordination with straightedge measurements across the 1,8 m long closure joints between the pier segments and the cantilevers closure joint to ensure alignment was essentially achieved at both the northbound and southbound headings. Transverse jacking was not required. The large length of the closure joints helped accommodating leftover rotational and minor positional divergences.

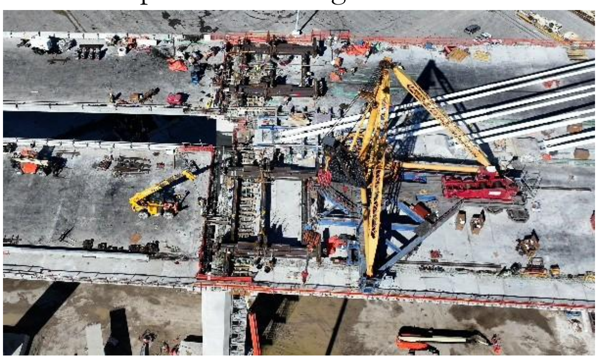

Figure 10. Photo taken during transition pier closure

7.4 Main span closure

Closure between the main span cantilever headings was less constrained than the back span closures due to the absence of fixed connecting elements and because it was relative between two flexible parts. However, it required coordination between the construction joint corrections of the last two construction units of the North and South cantilevers to ensure they were targeting the same meeting point without pronounced kinks.

Instead of having just one closure joint per heading, the main closure relied on a central precast segment connected with a 152 mm joint to its North and a 0,9 m long joint to its South. The former was cast first using standard construction joint procedures, and the latter was the actual closure joint, with similar functions as that described in the previous section. The benefit of having these procedure was to smoothen any potential kinks between the cantilevers by distributing the break angles.

Strongback beams with 102 mm of vertical and transverse correction capabilities were provisioned at the headings. On site, less than 25 mm of these allowances was used.

Figure 11. Photo taken during main span closure

8. Geometry acceptance

Following the end of construction, a final verification of the obtained bridge deck geometry will be carried out to evaluate its functionality in terms of rideability and drainage.

For the former, an instrument called profilograph will be run along different sections of the travelled ways and flag any location with more than 3,2 mm of deviation in 3 m. For the latter, water is typically poured on the deck to check for ponding.

The result of both tests will inform the amount of milling, and possibly localized concrete build-up, required to obtain a smoother deck surface with clean sloping. The bridge was designed with 13 mm of milling allowance.

9. 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

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