e-mosty

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AMERICAN BRIDGES: USA

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Front Cover: San Francisco – Oakland Bay Bridge, California, USA Photo provided by TYLin (from Shutterstock)
Back Cover: Long Beach International Gateway Bridge, California, USA Courtesy of the Port of Long Beach

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Dear Readers

The first article describes the design and construction of the self-anchored suspension span of the San Francisco - Oakland Bay Bridge (SFOBB). This article was originally published by ICE, however, TYLin offered it to us intending to share it with our readers. With the kind permission of Emerald and in cooperation with TYLin, we publish it again together with some videos and construction photos. For the next special edition "American Bridges" which is planned for March 2025, TYLin will prepare an article about the precast segmental Skyway viaducts.

The next article was prepared by Arup and focuses on the Gerald Desmond Bridge (now the Long Beach International Gateway Bridge) which is California's first long-span cable-stayed bridge.

It is followed by an article about the I-395 Signature Bridge in Miami. The bridge with its six arches will create a new landmark in the city. This bridge is currently under construction with completion targeted for late 2027.

The effort to preserve the Benjamin Franklin Bridge in Philadelphia for the next 100 years is described in the article prepared by AECOM. The majority of works within the five separate design projects consists of repairs to various steel components on the suspended spans, maintenance painting, and installing a dehumidification system for the main cables.

In the articles, we respect US units and spelling. As SFOBB was delivered to the client in metric units, they are put first.

Rúbrica Engineering prepared an article about their projects in which they used customized equipment in America. Those projects are not in the USA, however, and following the opening of their new subsidiary Rubrica USA and their recent projects in Canada, they show how versatile and adaptable their equipment is, and its capabilities in different construction methodologies.

*I would like to thank **David Collings, Juan C. Gray, Ken Wheeler and Richard Cooke** for the review and assistance with the content, and all the authors, people, and companies that have been helping me put the content together.*

We also thank our partners for their continuous support.

*As a media partner of IABMAS 2024, and on behalf of the organizers, we would like to invite you to the **IABMAS 2024 Conference** which will be held in Copenhagen, Denmark from 24th to 28th June 2024. Please find more information on page 9.*

The next e-mosty magazine special edition "American Bridges: Canada", with a focus on the Samuel de Champlain Bridge, will be released on 20th June 2024. The next e-BrIM will be released on 20th May 2024.

In 2025, we will publish another special edition of e-mosty "American Bridges" and we invite you to contribute with articles about your projects in both Americas. We are also planning to prepare a special edition about the Corpus Christi Harbor Bridge in Texas which is currently under construction.

Magdaléna Sobotková

Chief Editor

ACKNOWLEDGEMENT

I would like to thank the people and companies that have been helping me put this special edition together.

Also thank you for your time and for your kind assistance during my US Tour in November. It was nice meeting you and talking about various bridge projects and great bridge engineers. And thank you for showing me some of the bridges and ongoing projects.

Especially:

Juan C. Gray, Member of the e-mosty Editorial Board. HDR

Rafael Manzanarez, Manzanarez Consulting

(Golden Gate Bridge and other bridges around the San Francisco Bay)

Sajid Abbas, Carol Choi, Marwan Nader, Pam Ching, Dan Turner, Fady Bou-Shebel, TYLin

(San Francisco - Oakland Bay Bridge, presentation of Professor T. Y. Lin)

Barry Colford, Joshua Pudleiner, AECOM

(Benjamin Franklin Bridge, Philadelphia)

Jeremy Johannesen, McNary Bergeron Johannesen

Jake Presken, Walsh Group

(Miami I-395 Signature Bridge, Florida)

Luke Tarasuik, Mark Fisher, Jonathan Aylwin, Arup

Jamey Barbas, Project Director for the Governor Mario M. Cuomo Bridge

Khaled M. Mahmoud, Bridge Technology Consulting

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The magazine **brings original articles about bridges and bridge engineers** from around the world. Its electronic form enables the publishing of high-quality photos, videos, drawings, links, etc.

We aim to include **all important and technical information** and show the grace and beauty of the structures.

We are happy to provide media support for important bridge conferences, educational activities, charitable projects, books, etc.

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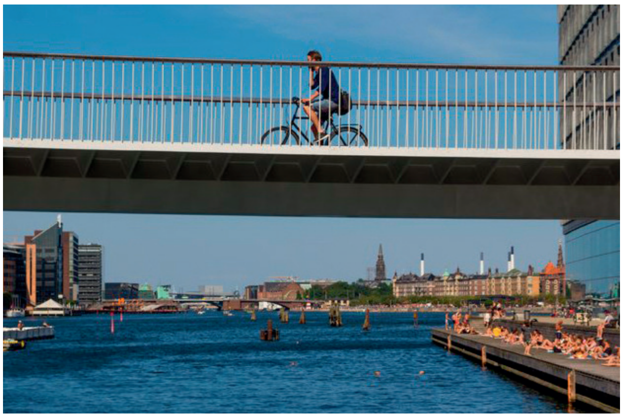

THE SAN FRANCISCO-OAKLAND BAY BRIDGE – EASTERN SPAN, CA, USA

Marwan Nader PhD, PE, PEng, Ing, Senior Vice President and Chief Bridge Engineer; TYLin, San Francisco, CA, USA

Brian Maroney PhD, PE, Toll Bridge Seismic Retrofit Program Chief Bridge Engineer (retired), California Department of Transportation (Caltrans), CA, USA

Figure 1: San Francisco-Oakland Bay Bridge over the San Francisco Bay. Source: Sam Burbank

1. INTRODUCTION

Celebrating its tenth anniversary in September of 2023, the San Francisco–Oakland Bay Bridge, with its daily traffic of 260,000 vehicles, serves as a crucial link in California's San Francisco Bay Area. This vital bridge, designed to stay operational for emergency use after a significant seismic event, has an expected lifespan of 150 years. Spanning 3.6 km (2.2 miles), it comprises four distinct segments: a low-rise, post-tensioned concrete box girder near the Oakland shore; a 2.4 km (1.5 miles) long segmental concrete box girder, also known as the skyway; an innovative self-anchored suspension (SAS) bridge with a 385-meter (1,263-feet) main span across the navigation channel; and a post-tensioned concrete box girder connecting to the east portal of the Yerba Buena Island tunnel.

Opened in 2013, the bridge's standout feature is the 624 m (2,047 ft) long SAS span, which is 79 m (259 ft) wide and accommodates ten traffic lanes alongside a bike and pedestrian path. Procured through multiple contracts, this US $6.4 billion megaproject was delivered using the traditional design–bid–build method. This paper describes the significant design innovations and construction techniques employed to overcome the project's unique challenges.

The seismically vulnerable east span of the San Francisco–Oakland Bay Bridge (SFOBB) was replaced with a dual east-bound and westbound 3.6 km (2.2 miles) long parallel structure. The SFOBB lies between the Hayward and San Andreas faults (CA, USA), which can generate magnitude 7.5 M and 8.1 M earthquakes, respectively, Figure 2. Performance criteria require that the bridge must be operational immediately following a 1,500-year return period earthquake from either of these two faults. Four distinct structures make up the bridge crossing:

- A low-rise post-tensioned concrete box girder near the Oakland shore.
- A 2.4 km (1.5 miles) long segmental concrete box girder.
- A self-anchored suspension (SAS) signature span.
- A post-tensioned concrete box girder that connects to the east portal of the Yerba Buena Island (YBI) tunnel, Figure 3.

Figure 2: Location of the Bridge.
Click on the map to open Google Maps

Comparable with the levels specified in the 1930 Uniform Building Code (UBC, 1930) for buildings, the Bay Bridge was designed for only 10% gravity (0.1g) earthquake accelerations.

2. GEOTECHNICAL CONDITIONS

The site geology varies dramatically along the length of the bridge. Figure 4 shows the geological profile at the bridge alignment. At the western end of the bridge (YBI), the piers are founded on rock.

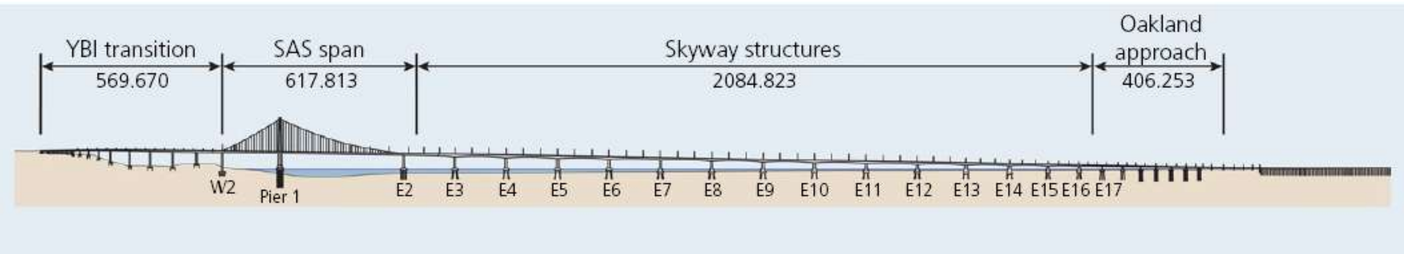

Figure 3: The new SFOBB (dimensions in m)

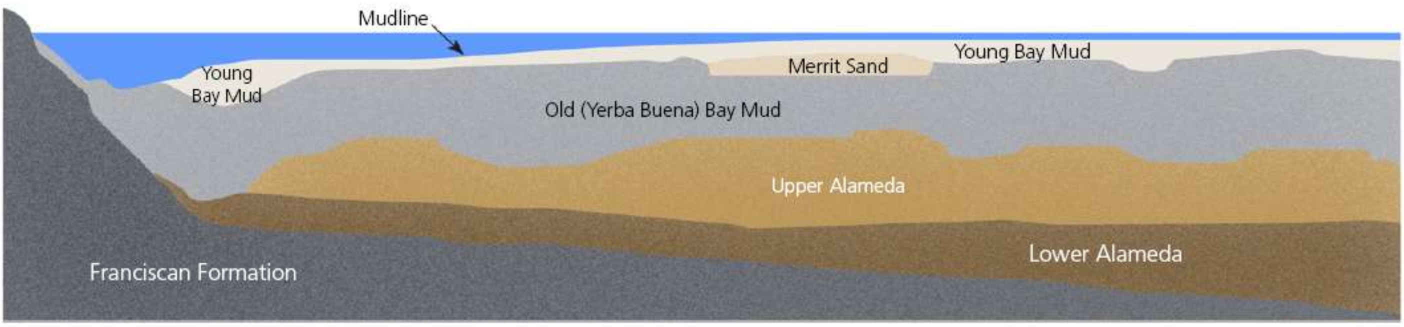

Figure 4: Soil Profile

As the bridge alignment progresses east towards Oakland, the bedrock Franciscan Formation drops abruptly, and the remaining piers overlie deep Bay Muds, followed by the interlayered clays and sands of the Alameda Formation. The main span tower structure is sited on relatively shallow, sloping bedrock. The remainder of the skyway is founded on a significant thickness of sediment.

3. SEISMIC HAZARDS

Seismic hazard evaluations were performed to define a safety evaluation earthquake (SEE) and a functional evaluation earthquake (FEE). The SEE corresponds to an earthquake with a return period of 1,500 years while the FEE corresponds to an earthquake with a return period of 450 years.

Both deterministic and probabilistic approaches were used to set the appropriate seismic criteria for the site. The seismic hazard is dominated by the San Andreas fault and the Hayward fault. The bridge design is controlled by SEE events. Six seismic ground motions were developed and used to determine the structural deformation, strength demands, and drifts.

4. SEISMIC PERFORMANCE CRITERIA

The SFOBB is designed to provide a high level of seismic performance. It is designed to resist two levels of earthquake (i.e. both FEE and SEE events). After a FEE, the bridge will provide full service almost immediately, and there will be minimal damage to the structure. Minimal damage implies essentially elastic performance and is characterized by a minor inelastic response, narrow cracking in concrete, no apparent permanent deformations, and damage to expansion joints.

After a SEE, the bridge will provide full service almost immediately and will sustain repairable damage to the structure.

Repairable damage is damage that can be repaired with minimum risk of losing functionality; it is characterized by yielding of reinforcement, spalling of concrete cover, and limited yielding of structural steel.

5. BRIDGE TYPE SELECTION

The Engineering and Design Advisory Panel (EDAP) appointed by the Metropolitan Transportation Committee evaluated various design alternatives for the SAS signature span and the Skyway. Public outreach meetings were held in which the community was invited to share visions and thoughts on what the new bridge should look like. After several meetings, the EDAP made the following recommendations, Figure 5:

- The new bridge shall have a signature span near YBI.
- The signature span shall be cable supported.
- The tower height of the signature span shall not be taller than the towers of the west spans of the bridge (maximum of 160 m (525 ft)).
- The remainder of the new bridge shall be a parallel structure (not a double deck). This will provide eastbound traffic with sweeping views of the Berkeley Hills.
- The new bridge shall provide access for pedestrians and bicycles.
- The new bridge shall be located north of the existing bridge to provide westbound traffic with sweeping views of San Francisco.
- The bridge shall have aesthetic night lighting and shall be painted white.

The single tower, asymmetrical SAS span with a steel tower and a steel orthotropic deck was selected from a total of four design alternatives that were developed for the signature main span. These included two cable-stayed bridges, Figures 5(c) and 5(d) and two SAS bridges, 5(a) and 5(b). Each bridge type included single tower and dual portal tower alternatives.

(a) (b) (c) (d)

Figure 5: Design alternatives for the main span

The main drive for selecting the asymmetric SAS Bridge was aesthetics. Its allure was that it would provide the bay area with a unique structure not found elsewhere in the world. Its catenary cables resonate with the Golden Gate Bridge and the west spans of the SFOBB. The single tower along with the three-dimensional cable profile provides a cathedral feeling that is unmatched.

The EDAP also selected a segmental concrete haunched girder with spans of 160 m (525 ft) for the skyway from a total of three design alternatives; these included a concrete haunched girder with a 160 m (525 ft) typical span, a 6 m (20 ft) constant-depth concrete girder with a typical span of 120 m (394 ft) and a 6 m (20 ft) constant-depth steel box girder with a 160 m (525 m) span. In addition, the EDAP recommended that a single pedestrian and bicycle path be built on the south side of the eastbound structure, and that the bridge should be designed to carry light rail transit in the future. These concepts demonstrate how the engineer/architect collaboration created a design that satisfies both architectural aspirations and engineering requirements.

6. SAS BRIDGE

The SAS portion of the new eastern span of the SFOBB consists of dual box girders suspended from cables that are supported on the 160 m (525 ft) tower located off the eastern shore of YBI. The SAS spans 565 m (1,854 ft) between piers E2 and W2, with a 385 m (1,263 ft) main span over the navigational channel and a 180 m (591 ft) back span, Figure 6.

The asymmetry of the bridge subjects pier W2 to a vertical uplift while the bridge is lightly supported on pier E2. The main tower therefore carries most of the bridge dead load. The uplift at pier W2 is fully counterbalanced by the prestressed concrete cap

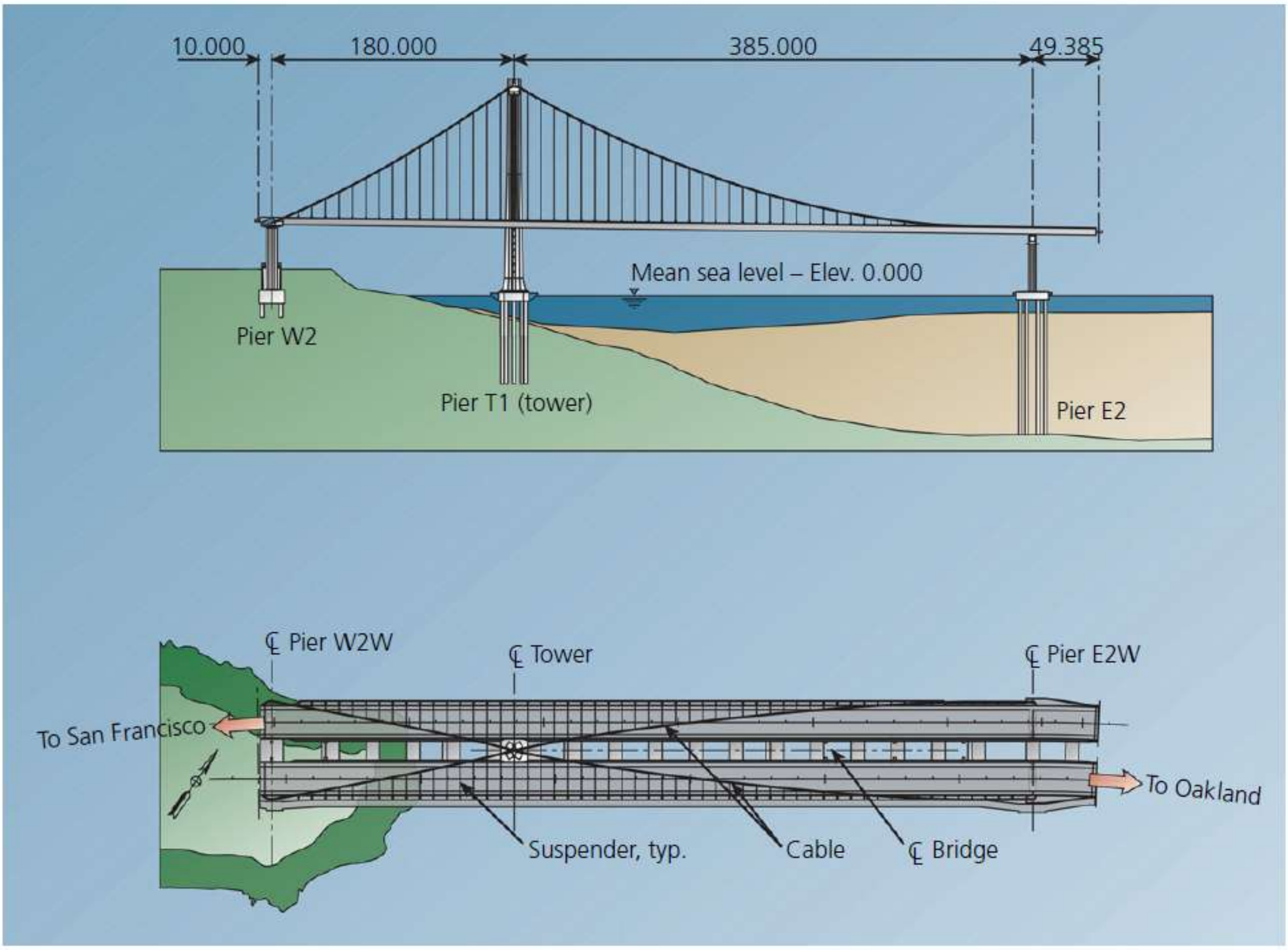

Figure 6: Elevation of the SAS bridge (dimensions in m)

beam at pier W2. This ensures that the bridge is always balanced without relying on the pier to carry any tension. To balance the moments in the box girder caused by the 49 m (161 ft) cantilever at the east anchorage, suspenders do not support a 35 m (115 ft) segment of the eastern end of the main span.

The box girders are not supported by the tower and are therefore 'floating' at the tower, with the suspenders providing the only connection between the box girders and the tower. This implies that, while the tower carries most of the bridge dead load, the tower is not the primary element that carries the bridge seismic loads. The tower seismic response is mainly governed by its own mass and stiffness (the tower acts as a propped cantilever with a spring support at the tower saddle). The gap between the tower and the deck is designed to be large enough to prevent any impact during a SEE. Piers E2 and W2 are designed to provide the main lateral seismic support of the bridge.

The 0.78 m (2.5 ft) dia. cable is anchored to the east anchorage and is looped around the west bent through deviation saddles. The suspenders are splayed to the exterior sides of the box girders and are spaced at 10 m (33 ft).

The superstructure consists of dual hollow orthotropic steel box girders, Figure 7. These girders are in longitudinal compression (reacting against the cable tension force) and are a part of the gravity load system. Transverse diaphragms spaced at 5 m (16 ft) support the orthotropic deck and distribute the suspender loads to the entire box. The box girders are connected by 10 m (33 ft) wide by 5.5 m (18 ft) deep cross-beams spaced at 30 m (98 ft) on center. These cross-beams carry the transverse loads between the suspenders (span of 72 m (236 ft)) and ensure that the dual boxes act compositely during wind and seismic loads (Vierendeel truss action).

The selection of the steel orthotropic deck system, rather than an equivalent concrete deck, was based on the advantages of steel, including its light weight, durability, redundancy, and full integration with the box girder. The steel orthotropic desks for this bridge are noteworthy because they were designed as compression members meeting seismic compactness requirements (Nader et al., 1999) and because of the high degree of interaction among the various structural components. In the SAS system, there is an inherent interdependence of suspended weight and the local design of girders. All suspended weight contributes to the total tension in the main cable, which is anchored into the longitudinal box girders, placing them in axial compression. Each pound of weight 0.45 kg (1 lb) required about 34 kg (75 lb) of structural steel in the girder cross-section to resist additional global compression. Thus, the burden of structural material, added to the box girders to resist additional load, is effectively factored by 1.75.

The bridge carries a pedestrian path on the south side of the eastbound deck. The pedestrian path eccentric load is balanced by a counterweight on the north side.

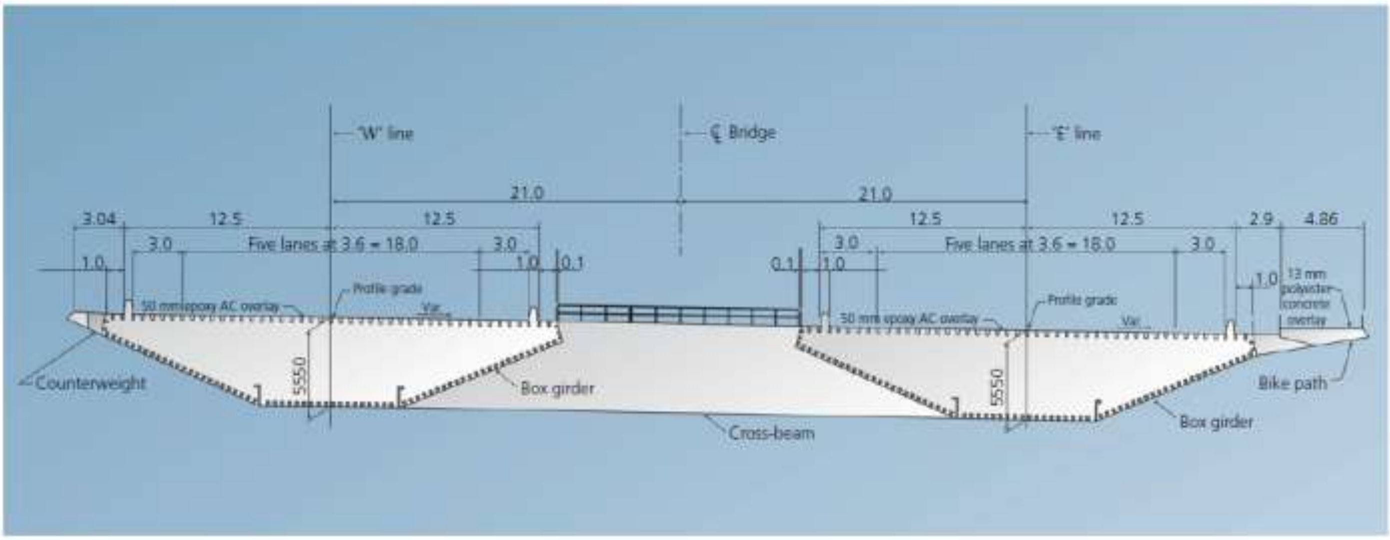

Figure 7: Typical cross-sections of bridge – dual box girders (dimensions in m)

At the west bent, the steel box girders frame into the prestressed cap beam. The connection between the orthotropic steel box girders and concrete cap beam is subjected to the compressive reaction forces of the cables. Additional prestress is added through post-tensioned strands connected at each steel orthotropic rib.

The 160 m (525 ft) main tower is composed of four shafts interconnected with shear links along its height. The tower shafts are stiffened pentagonal steel box sections that taper along the height. They are provided with diaphragms spaced at 4 m (13 ft) and are rigidly connected at the top and bottom by a tower saddle grillage and a tower base grillage, respectively, Figure 8.

The tower is fixed to a 6.5 m (21.3 ft) deep pile cap with anchor rods and dowels. The pile cap consists of a steel moment-resisting frame encased with concrete and supported on 13 steel shell pipe piles (70 m (230 ft) long and 2.5 m (8.2 ft) diameter) filled with concrete and embedded into rock. The rock slope is benched to give the piles equal lateral stiffness and to reduce torsional response (approximate clear length of 20 m (66 ft)).

Figure 9 shows the east bent, which comprises two reinforced concrete piers and a prestressed concrete cap beam. The pre-stressed cap beam was introduced to protect the bearings and shear keys that connect the box girders to the east bent. The bearings are designed to carry vertical loads (with the capacity to carry lateral loads), while the shear keys are designed to carry all the lateral loads. The bearings are made of a spherical bushing assembly capable of large rotations about the transverse axis of the bridge, thus providing an almost true pin connection. The east bent is supported on 16 cast-in steel shell concrete piles. These vertical piles are 2.5 m (8 ft) in diameter and about 100 m (328 ft) long and are founded on bedrock.

The west piers are reinforced concrete columns that are monolithically connected to the prestressed cap beam forming the west bent. The west bent is supported on gravity footings, which in turn are anchored into rock with 10 m (32.8 ft) long corner piles. A tie-down system, designed to ensure that the west piers remain in compression during a seismic event, connects the west bent cap beam to the gravity footings, see Figure 10.

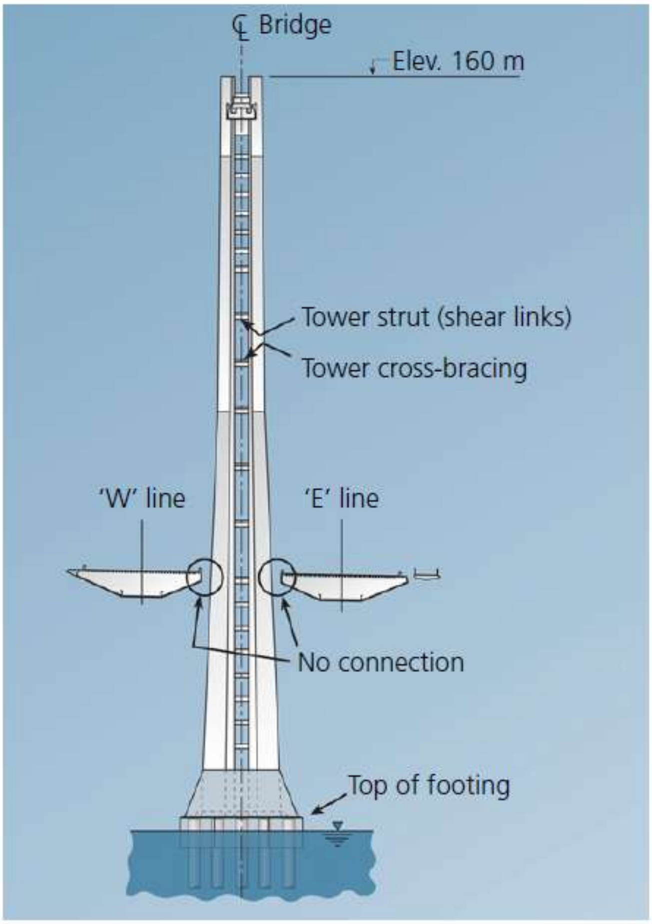

Figure 8: Single tower with four shafts interconnected with shear links

The hinges in the transition spans between the SAS and the skyway and between the SAS and YBI structures are designed to allow the structures to move relative to each other in the longitudinal direction and to rotate about the longitudinal axis of the bridge.

The hinges comprise compact steel beam pipe sections capable of transferring live loads and seismic loads. These hinge pipes are designed to fuse before any other bridge components in the case of an overload, see Figure 11.

To form the fuse, the center portion of the hinge pipe, located in the expansion gap between the two box girders, is made with a thinner wall thickness and lower steel grade than the typical portions of pipe that pass through the hinge bearings. This fuse is designed to yield in an unforeseen overload seismic event, thus protecting the remainder of the hinge pipe beam and keeping the rest of the superstructure in the elastic range, including the bearings.

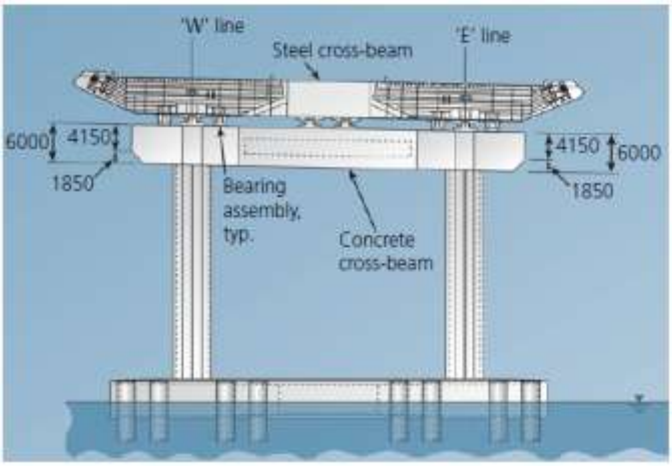

Figure 9: Elevation of east span (dimensions in mm)

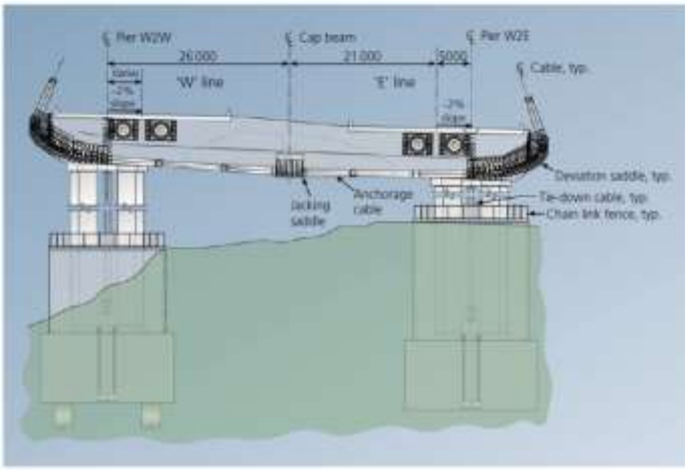

Figure 10: Elevation of west bent (dimensions in mm)

If the hinge pipe beam fuse requires replacement, a strong back, similar to what was used during erection to align the box girder during pipe installation, will be installed to hold the spans in place. Then, the fuse will be replaced by cutting the pipe on either side and lowering the damaged material down through the expansion gap. A new pipe fuse segment would then be brought in and welded into place.

6.1. Single Tower Design

The tower shear links play a significant role in resisting seismic loads as well as supplying the tower with adequate stiffness during service load conditions.

The tower shear links are designed to satisfy the following criteria:

- Supply the tower with the required stiffness for service load conditions.
- Remain almost elastic during a FEE.
- Plastify during a SEE, thus dissipating energy and limiting the damage in the tower shafts (shafts are designed to remain almost elastic).
- To be replaceable after a SEE, if necessary.

To satisfy these requirements, various configurations of the tower were evaluated with variations in the strength and stiffness of the shear links as well as their location along the height of the tower.

Figure 11: SAS to skyway hinge

These studies were primarily done in the form of static pushovers to determine the response of the tower during service loads, wind loads, FEE loads, and SEE loads. An optimal layout of these links was then established, as well as the stiffness and yield strength of the shear links.

Figure 12 shows the cross-section of the tower, and Figure 13 shows the elevation of a typical shear link.

Although the tower shafts are designed to remain elastic during SEE, the shafts were designed as stiffened box sections as per ATC-32 (ATC-32). This ensures that the shafts could undergo large inelastic compressive strains without local buckling.

A pushover analysis of the tower was performed to:

- evaluate the relationship between the base shear and the top of tower displacement;
- optimize the design of the tower shear link and shaft;
- evaluate the lateral ductility of the tower before collapse;
- evaluate the ductility demands on the shear links and tower shafts at various levels of displacement demand.

Figure 14 shows the effects of using shear links on the lateral behavior of the tower. The tower has stable behavior for displacements much larger than the SEE displacement demands (1 m (3.3 ft)).

Figure 15 shows a full-scale test specimen tested at the University of California at San Diego (CA, USA) at a rotation of 0.07 rad.

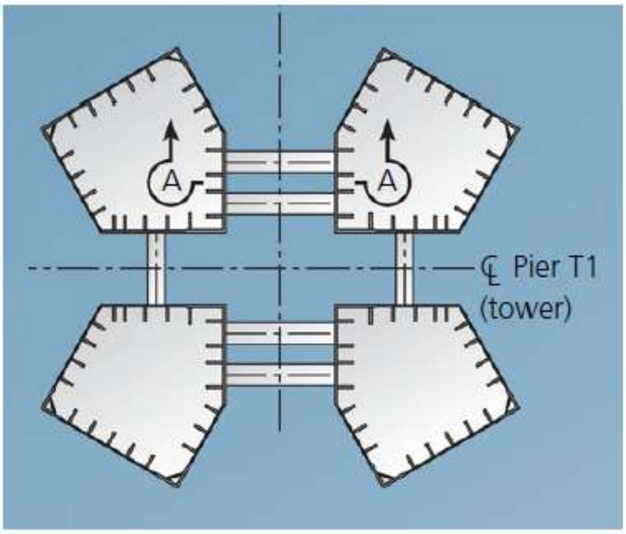

Figure 12: Typical tower cross-section

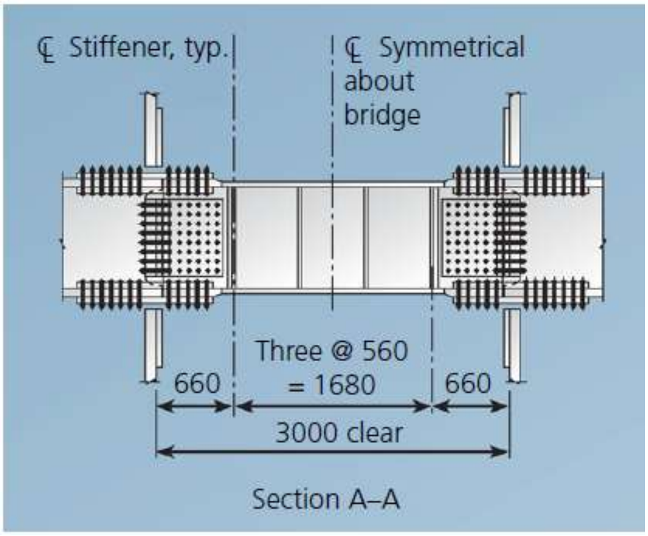

Figure 13: Typical tower shear link (dimensions in mm)

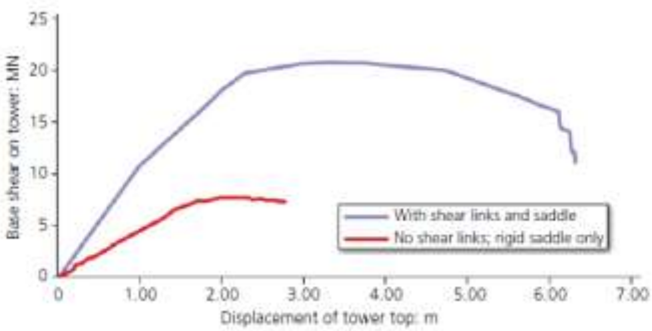

Figure 14: Pushover analysis of the single tower

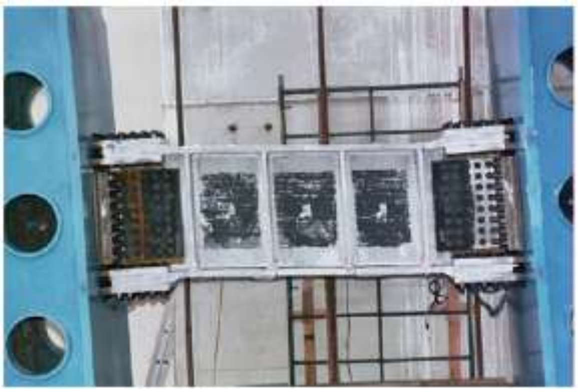

Figure 15: Full scale test of shear link type 1
at University of California at San Diego

6.2. Seismic Performance

Seismic analysis was performed using the general-purpose finite-element program Adina. Three forms of analysis were employed – time history analysis (global model), pushover analysis, and local detailed analysis. Time history analysis was used as the primary means of analysis for several reasons, the foremost of which is that the bridge foundations are subjected to different excitations. The tower and west pier of the bridge are founded on rock, while the east pier is supported in deep soil.

The ground motions at these supports are completely different in character and intensity. This was reflected in the analysis by applying different time histories of ground displacement at the supports.

A large-displacement analysis and the use of non-linear material were necessary to allow the designers to capture the true behavior of the bridge (geometric stiffness of the bridge, P-Delta secondary moment effects slacking of suspenders, plastic hinging of piers, tower shear links, etc.).

The model was 'built' in a single step, in the dead load state. Initial strains in the deck, cables, and suspenders were applied in this single step rather than simulating the construction sequence of the bridge. Time history analyses were done as restart analyses from the dead load state.

The pushover analysis was primarily used to evaluate the ductility of critical elements, and to establish the failure mode sequence. Local detailed analysis was used to establish local strain/stress demands and to evaluate the modelling used for the global model.

The Adina global model of the SAS bridge is shown in Figure 16.

The bridge was designed based on a limited ductility design in which plastic deformations are clearly defined and predetermined. The seismic response of the bridge is summarized in Figure 17: the bridge is designed to remain largely elastic except for the east and west piers, which are designed to form plastic hinges.

The plastic strain in these piers is limited to 2/3 of the ultimate strains based on Mander's equation for confined concrete columns. The shear links between the tower shafts are also designed to yield in shear during a SEE earthquake. The maximum rotation demand on these links is 0.04 rad, compared with an ultimate rotation of 0.09 rad.

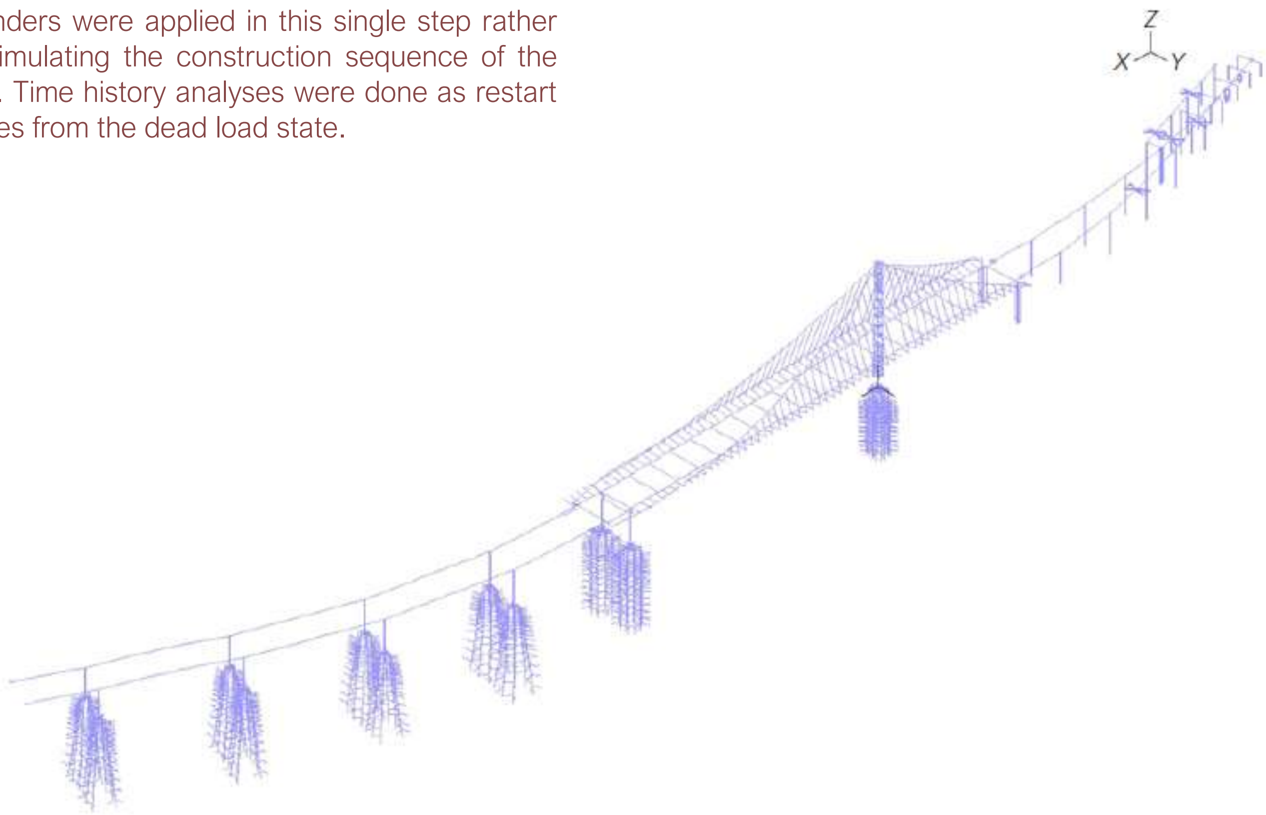

Figure 16: Adina model of the suspension bridge

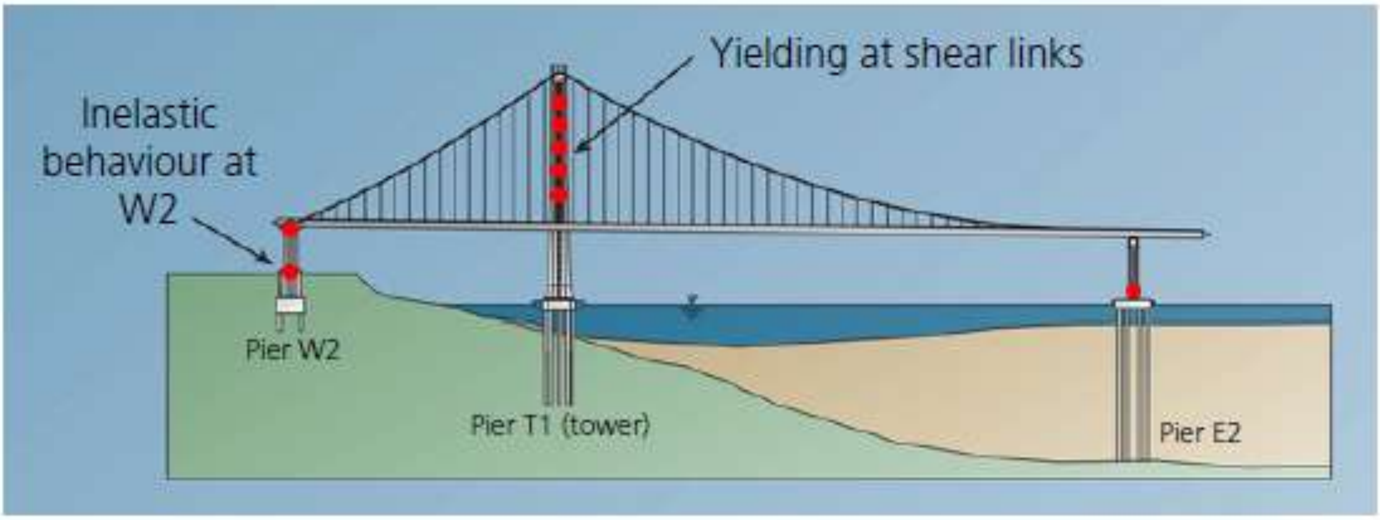

Figure 17: Seismic response of the bridge

The piles were designed to sustain minimal damage (strains less than 0.01 for concrete and 0.02 for steel) when subjected to SEE displacement demands. The tie down at the west pier was designed with a factor of safety of two. An animation of the movement of the bridge in a seismic event is provided in Figure 18.

6.3. Erection of SAS Bridge

In a classic suspension bridge design, the entire design dead load is assumed to be carried by the suspension system, while the stiffening trusses or girders only serve to distribute live loads and limit local deflections. The stiffening system typically has very small bending moments under the design dead load. The hangers are typically vertical, and the longitudinal component of cable tension is a constant. The suspension cable and hangers are erected first. The deck segments are then hung from the hangers, and little falsework is needed. Erection is facilitated by the following factors:

- The suspension system supports the deck during erection.
- The hangers are vertical.
- The deck segments have no dead load moments built into the construction.

In the SAS, these conditions do not apply. Given that the suspension cable supports the girders to which it is anchored, temporary works were required to achieve the final structure. Since the box girder maintains the tension in the cable, it had to be erected on falsework prior to the cable erection, Figure 19.

Parallel trusses were erected for the full length of the superstructure, and the twin girders were erected on them. Cross-beams were installed between the girders as they progressed.

Once the entire superstructure was erected, the cables were erected by towing each parallel wire strand over a catwalk system that led from the north anchorage just east of pier E2, over the tower top, around the back of pier W2, back across the tower top, ending at the south cable anchorage.

Once each strand was in position around the circuit, it was inserted in sequence into the saddles located at the back of pier W2, at the north and south ends of pier W2, at the tower top, and over the north and south ends of pier E2.

Figure 18: Animation of Seismic Response, developed by TYLin and partners for the Toll Bridge Program Oversight Committee

Click on the image to play the video

Figure 19: Temporary truss falsework

Then, the ends of the strand were coupled onto the high-strength rods in the anchorages and tensioned to produce the required sag of the free-hanging cable.

With all the strands in place, the cable was compacted, banded, and wrapped. The suspenders were then placed over each cable band, and selected rope sets were gripped by clamps and pulled towards the suspender brackets. About a quarter of the suspenders were tensioned in this manner to swing the main cable outward into the inclined planes of its final position. With the main cable in position, the remaining suspenders were gripped and prepared for tensioning.

By incremental tensioning of the suspenders, the box girders were lifted off from their temporary bearings, which were removed as they were relieved, to provide a safe clearance between the girders and the temporary trusses.

The temporary works were then removed, and the SAS bridge was in equilibrium and ready for the finishing activities. Led by TYLin as the Structural Engineer of Record and supported by Parsons, NC3D, and Chou's Image, see Figure 20 for the animation, commissioned by Caltrans, demonstrating the load transfer process.

Given sloping hangers, it is a highly indeterminate problem to find the profile of the suspension system hangers, even when the hanger supports do not move. Due to the support conditions at the end piers, there are moments in the box girders throughout the length of the bridge; these were determined by design.

Traditionally, cable-supported bridges are analyzed 'backwards', starting from their intended final configuration, to find the necessary initial conditions from which to base construction. This technique was employed for the SAS, particularly with respect to the cable.

Figure 20: Animation of Load Transfer of San Francisco-Oakland Bay Bridge Self-Anchored Suspension Bridge by TYLin and partners for the Toll Bridge Program Oversight Committee

Click on the image to play the video

However, the main analysis of the SAS bridge was a 'forward' analysis, starting from known or computed initial conditions. This facilitated the determination of critical steps for construction, the evaluation of alternative methods of erection and tracking construction progress.

The analysis and the erection control considered the staged construction of the bridge in great detail, including the major steps of:

- box girder erection;
- tower erection;
- cable erection;
- hanger installation;
- connection to the skyway;
- the addition of superimposed dead load.

7. SKYWAY STRUCTURES

Leading to the SAS bridge from Oakland is the2.4 km (1.5-miles) long segmental concrete skyway, which was constructed using segmental construction and battered piers, Figure 21.

The skyway's decks comprise 452 precast, post-tensioned concrete segments. These segments were fabricated in Stockton Yard, which is located on a deep-water shipping channel, and transported by barge to the project site.

While the segments were being cast, cured, and stored in Stockton, work to build their foundations in San Francisco Bay continued. In this way, the tight production timeline was kept on course and possible environmental impacts to the bay were minimized.

To ensure that the segments would fit together precisely when they were lifted into place, they were match-cast in a giant mold in the order in which they would be installed on the bridge.

Each concrete section was poured, one after the other, with ridges or shear keys on the adjoining surfaces. These shear keys ensured proper alignment and helped hold the segments together during the final assembly.

After the concrete had set, the segments were transported to a curing area by a massive, computerized straddle carrier, designed expressly for this project.

The concrete then cured for 2–18 months.

Finally, the straddle carrier loaded the segments onto a barge for the 10–12 h journey to the construction site in San Francisco Bay.

These were the largest segments of their kind ever cast (BBIO, 2005).

The challenges faced and the innovative solutions that were developed in the design and construction of this unique signature structure signify a major achievement in bridge building.

7.1. Skyway Piers

To provide a relatively stiff foundation system for the tall, flexible piers, large-diameter battered steel piles filled with concrete were used. The piles were driven into the soil at an angle, through a process called battering. This method has been used to create secure foundations for oil rigs for more than two decades but had never been used for bridge construction of this scale.

The piles weighed approximately 331 mt (365 tons) each and were driven by one of the world's largest hydraulic hammers, which generated 5.3 MN (1.2 million pounds) of force – the equivalent of a car hitting a brick wall at 118 m/s (265 mph).

To mitigate the impact of pile driving on fish, and other wildlife, dense columns of air bubbles or bubble curtains, were created around the piles underwater. These bubbles helped to dissipate the shock waves that were produced by the force of the hammering.

Figure 21: Skyway

8. CONCLUSIONS

In the face of numerous challenges, the east span of the San Francisco–Oakland Bay Bridge (SFOBB) was successfully opened to traffic on schedule in 2013. The first-of-its-kind Self-Anchored Suspension span met the demands of high seismic forces and challenging geotechnical conditions, while typical conditions did not apply for the carrying of dead and seismic loads.

The bridge's structural configuration led to the development of an orthotropic deck design controlled by compression stresses, compactness, and stability, with seismic concerns governed by overall economic considerations.

For the 2.4 km (1.5 miles) long skyway, challenges included producing and transporting large match-cast segments for the segmental construction, heavy lifts for the steel to concrete transition spans, and establishing a stable foundation in the Bay Mud. Additionally, seismic forces were addressed through the introduction of hinge pipe beams between the segments.

The devised solutions for this bridge not only pushed the boundaries of existing technology but also conscientiously minimized environmental impacts. The design and details used in both the SAS and Skyway offer valuable lessons for future designs.

ACKNOWLEDGEMENTS

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We would also like to thank Emerald for permission to republish this article.

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Figure 22: San Francisco-Oakland Bay Bridge Credit: Thomas Heinser

CONSTRUCTION PHOTOS

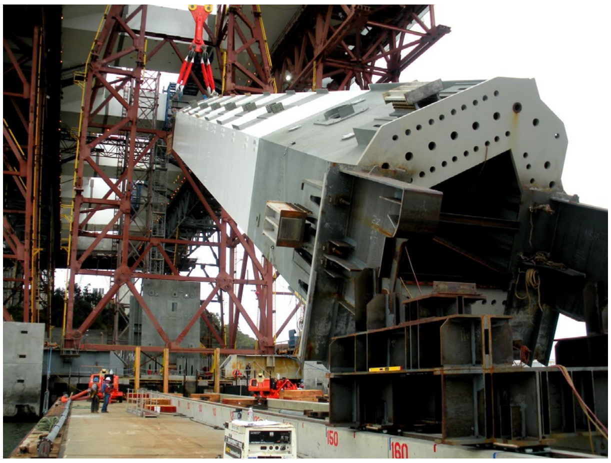
Skyway precast segment at port

Erection of Skyway superstructure

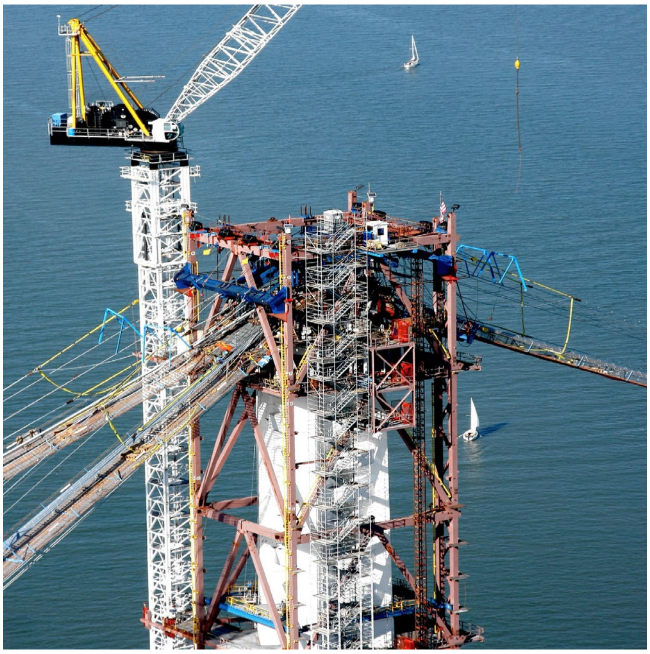
Hinge pipe beam installation

View of cable compactor

Cable compaction trial



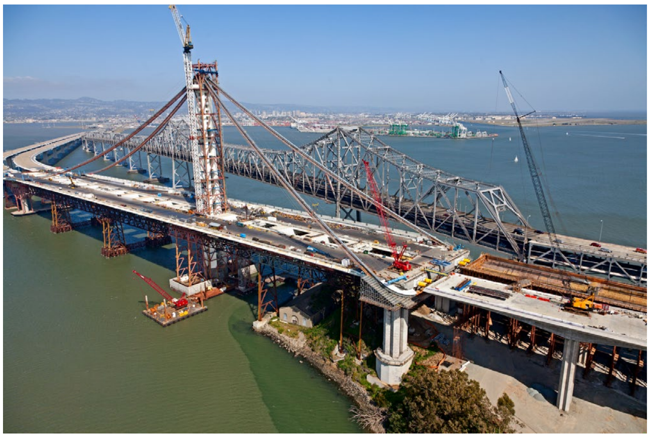
Tower erection



Tower and cable erection

Aerial view of the tower under construction

Tower erection



Looped cable west bent

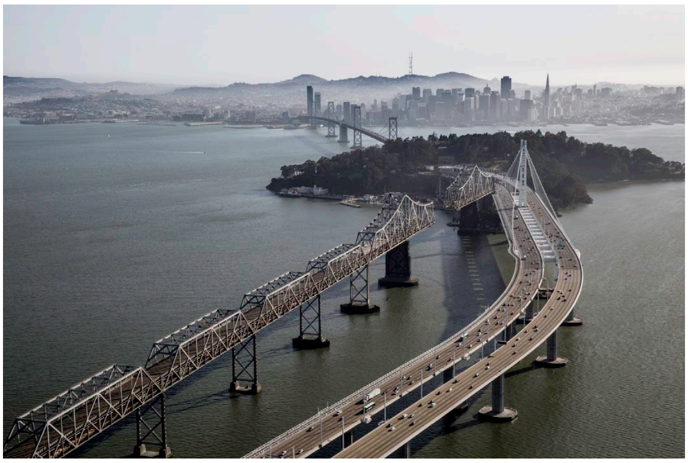
Close-up of looped cable west bent

Dusk view from Yerba Buena Island

Facing west construction

Panoramic view of the SAS bridge under construction

Aerial view of the completed new SAS bridge with the original bridge behind



Aerial view of the SAS bridge under traffic while the original bridge began demolition

TYLin

The Power of Connection

Volunteers from TYLin, in collaboration with Bridges to Prosperity, successfully completed construction of the Bishenge (Masuma) Bridge. This 223-foot-long hybrid suspension bridge spans the Gasumo River at the Nyamagabe municipality in Rwanda.

Bridges to Prosperity partners with corporations worldwide to build trail bridges that connect isolated communities to critical resources such as education, healthcare, markets, and jobs. The Bishenge Bridge has transformed the lives of many people and TYLin is grateful for the opportunity to collaborate on the construction of this lifeline bridge.

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LONG BEACH INTERNATIONAL GATEWAY BRIDGE, LOS ANGELES

Luke Tarasuik, Josh Mattheis, Matt Carter

Arup

Figure 1: Night View of the complete Bridge

I. INTRODUCTION

After nearly 50 years in operation, the Gerald Desmond Bridge was at the end of its useful life and was replaced with a new six-lane cable-stayed bridge that was completed in October 2020.

The 2,000-foot-long (610 m) Gerald Desmond Replacement Bridge, now named the Long Beach International Gateway Bridge but referred to herein under its previous name, is California's first long-span cable-stayed bridge.

Approximately two miles of new cast-in-place concrete approach viaducts rise 200 ft (61 m) off the ground from both the east and the west as they transition to the main span cable-stayed bridge. The bridge has a clear span of 1,000 ft (305 m) over the Back Channel, providing increased vertical clearance for future generations of commercial shipping. The cable-stayed main span is supported by two faceted 515-foot (157 m) tall mono-pole main span bridge towers.

Enhanced by customizable architectural lighting, the new bridge is a striking landmark for the Port of Long Beach.

With an incredible 15% of all North American container traffic due to cross over the bridge, the Long Beach International Gateway Bridge is a critical infrastructure link and a vital component of the regional and national economy.

The bridge replacement will continue to serve the needs of a growing region and ensure the safe, optimized flow of people and goods, with truck climbing lanes and shoulders on both sides of the highway leading to reduced congestion.

Arup was the prime designer for the project and Engineer of Record for the main span bridge and high-level approach viaducts. Arup also provided the cable-stayed bridge erection engineering support services. The project was developed under a design-build contract.

II. CABLE-STAYED BRIDGE OVERVIEW

Cable-stayed bridge technology was pushed forward in the twentieth century by a need for efficiency in materials and schedule. Post-World War II Europe needed bridges that could be built rapidly with limited supplies of structural steel and quantity-hungry suspension bridges did not meet these constraints.

Existing cable-stayed bridge methods, infused with modern materials and machinery, were a perfect fit and so flourished throughout Europe.

Figure 3: View of the Gerald Desmond Replacement Bridge, next to the original Gerald Desmond Bridge
Photo Courtesy Marie Tagudena

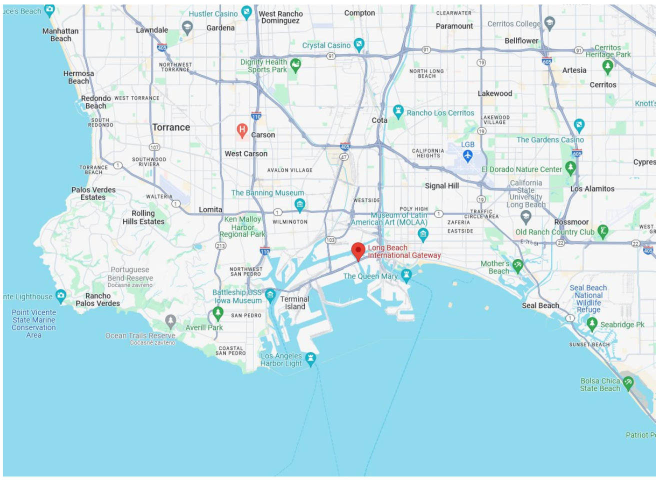

Figure 2: Location of the bridge.
Click on the map to open Google Maps

North America is currently encountering a period of similar needs, requiring a multitude of long-span bridges efficient in cost, schedule and performance.

The Gerald Desmond Bridge Replacement Project is a reflection of this on the West Coast of the United States and is followed by many other similar projects throughout North America.

The cable-stayed main span of the Gerald Desmond Bridge Replacement Project employs precast concrete deck panels made composite with a steel ladder-deck framing system through cast-in-place closure pours.

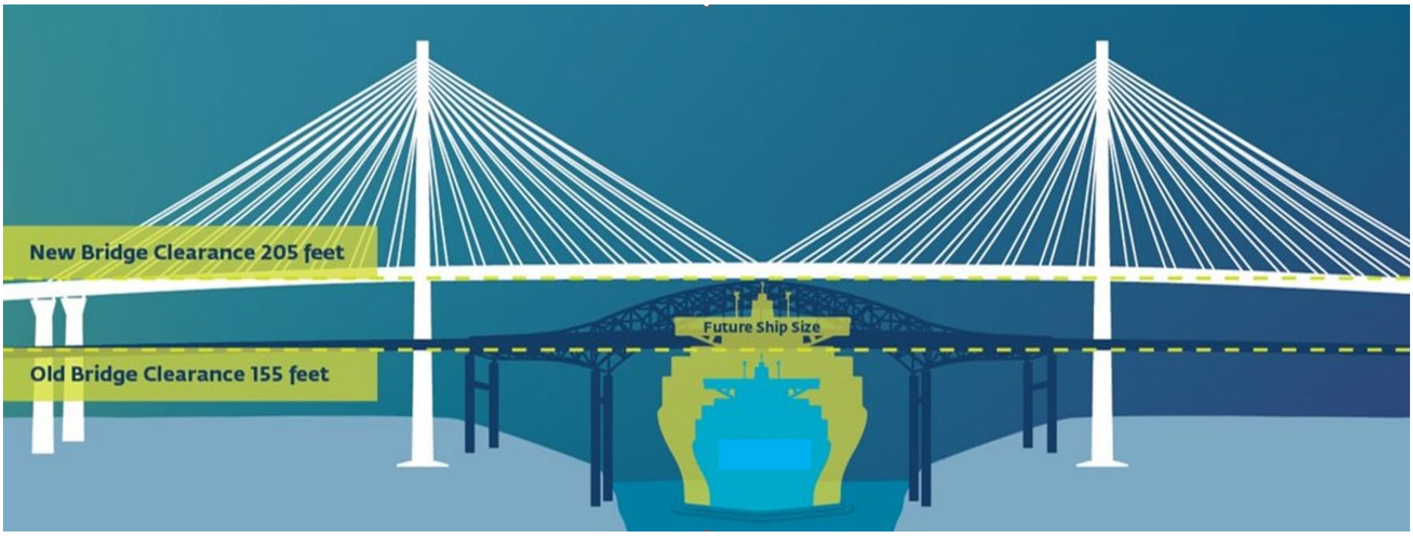

Figure 4: Comparative scale of the old and new bridge. Courtesy of the Port of Long Beach

Two planes of cable stays descend from two slender mono-pole towers in a modified fan arrangement to support six-foot deep trapezoidal closed-box edge girders. 150-foot (45 m) long floor beams span transversely between the edge girders at 16.75 foot (5 m) centers.

Intermediate longitudinal stringers serve to brace the slender top flange of the floor beams while non-composite and provide temporary support for the precast concrete deck panels prior to the closure pour.

The steel ladder-deck is erected stick-by-stick in 50-foot (15 m) long balanced cantilever segments using bolted splices. Through multi-staged stressing of the cable stays, longitudinal compression from the cable stays is introduced throughout the composite concrete deck – one of the benefits of cable-stayed construction.

In the case of the Gerald Desmond Bridge Replacement Project, this intrinsic cable-stayed benefit is augmented by the addition of substantial longitudinal post-tensioning.

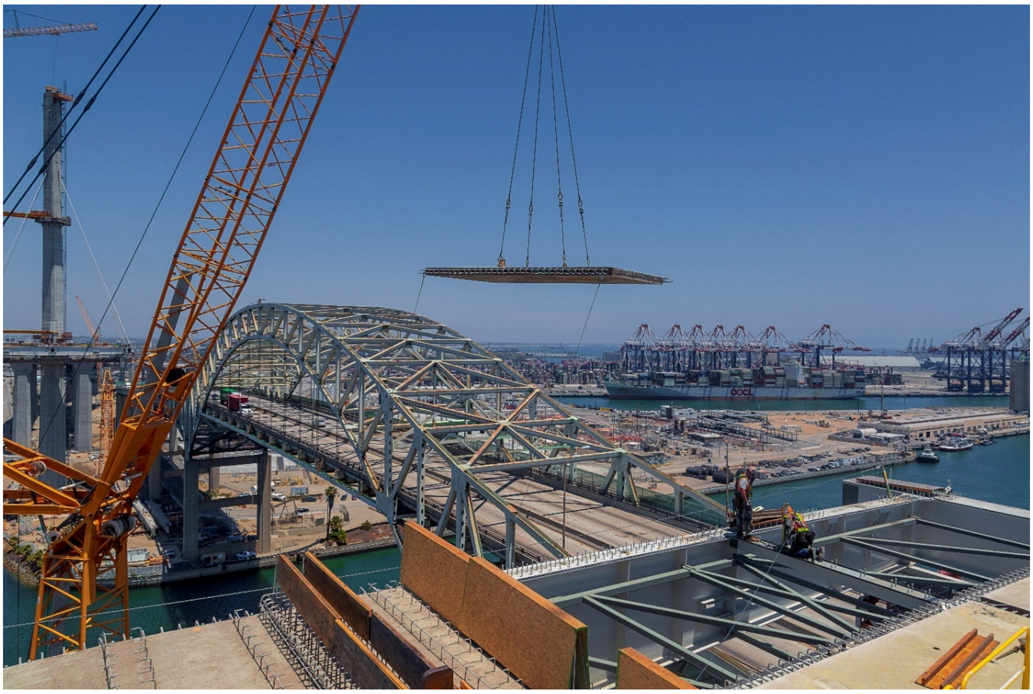

Figure 5: View of precast concrete panel being flown in for installation on the steel ladder-deck framing

Photo courtesy of Marie Tagudena

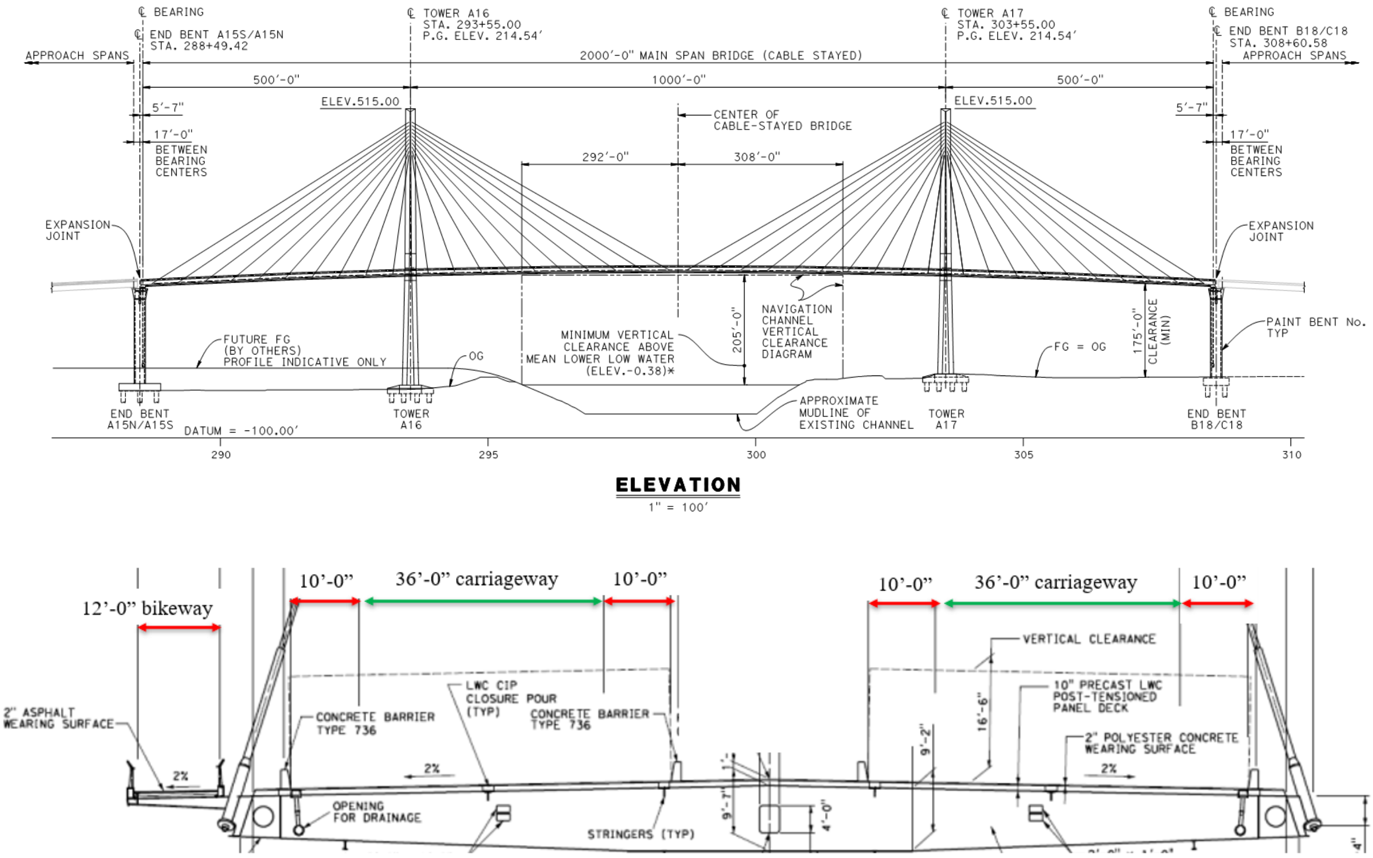

Figure 6: Elevation and Section of the Bridge. Image courtesy of Arup

III. SEISMIC DESIGN

The Gerald Desmond Bridge Replacement Project is the only cable-stayed bridge of its size on the highly seismic west coast of the United States. Arup designed the bridge towers and end bents to remain essentially elastic during seismic events in alignment with an AASHTO Type 3 seismic design strategy.

To achieve this, the bridge deck is seismically isolated from the towers and end bents by an array of 34 structurally-fused viscous hydraulic dampers.

Thanks to integrated structural fuses, the viscous hydraulic dampers only activate during seismic events above the one in one-hundred-year return period event.

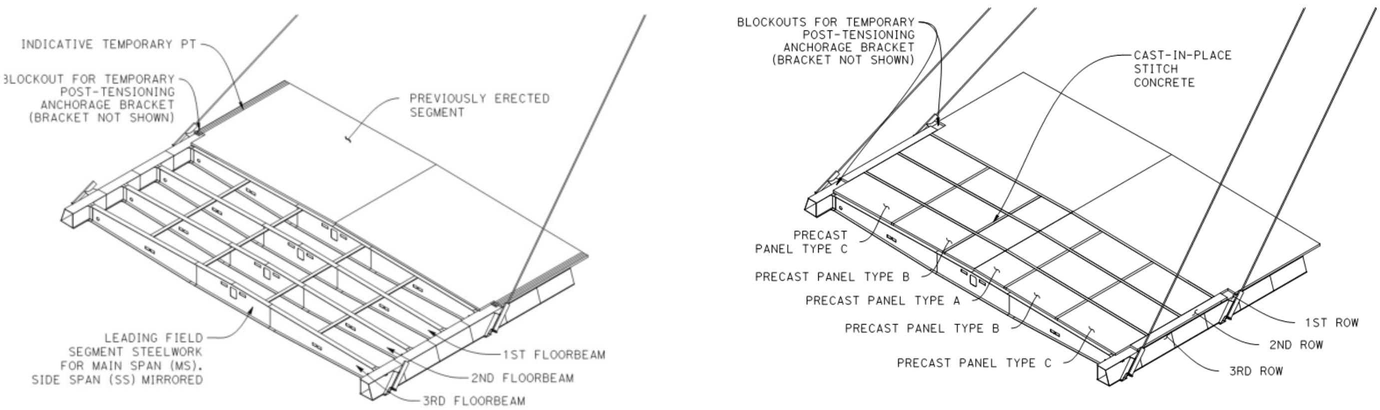

Figure 7: Erection sequence of the composite deck. Image courtesy of Arup

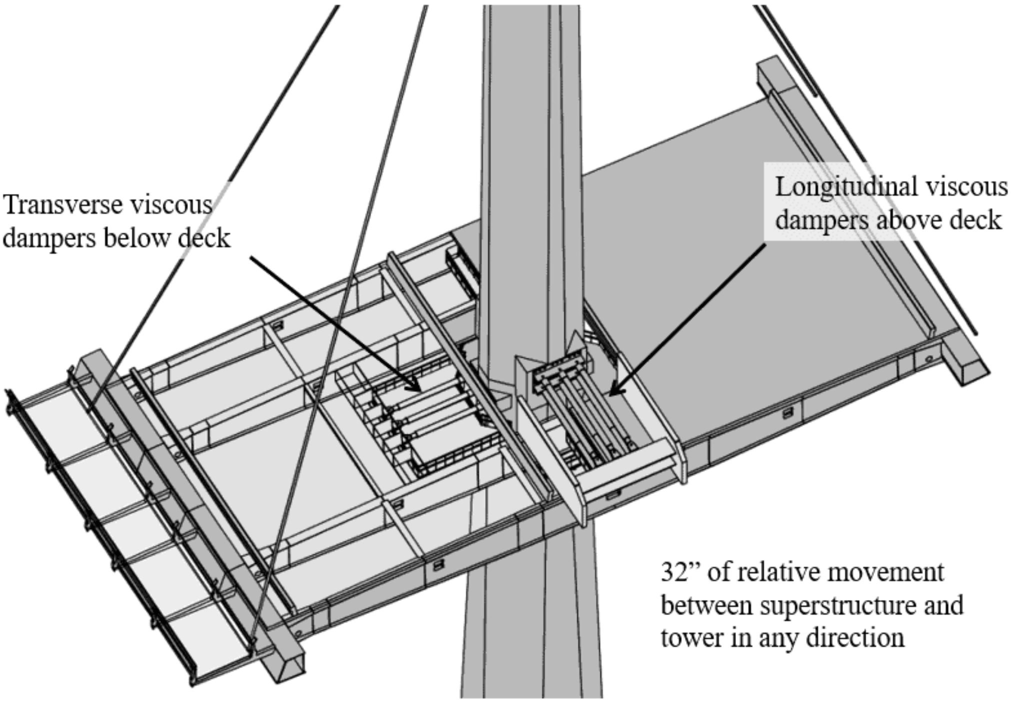

Figure 8: Depiction of tower dampers at maximum seismic distortion. Image courtesy of Arup

The damper fuses take the form of structural steel tubing encompassing the dampers, designed to release at a force corresponding to the controlling seismic event.

After the steel fuse releases, the viscous dampers begin to dissipate cyclic energy in the same way that a car's shock absorbers do on a bumpy road. The fused damper design reduces maintenance requirements by isolating sensitive damper components from ambient cyclical movements, ensuring optimal performance during the design seismic event.

Fuses and dampers are designed specifically for ease of maintenance, redundancy and future-proofing. Integrated pressure gauges, observations windows, and transducers facilitate routine maintenance. The overall quantity of dampers was determined to make the damper size manageable for installation, maintenance, or replacement. Dampers and fuses are provided by Taylor Devices, Inc. Testing of the full-scale dampers was performed at the University of California, San Diego (UCSD) laboratory.

The project's bid package reference design of the tower had a cross section modelled after the San Francisco Oakland Bay Bridge with ductile steel shear links placed between two separate tower legs, see Figure 10.

As the links are difficult to replace after a seismic event Arup's final design instead used a monopole tower and fused viscous damper solution that provides a non-invasive post-seismic remediation plan where the bridge deck is repositioned with jacks, and broken fuses are replaced without the need to alter the bridge substructure.

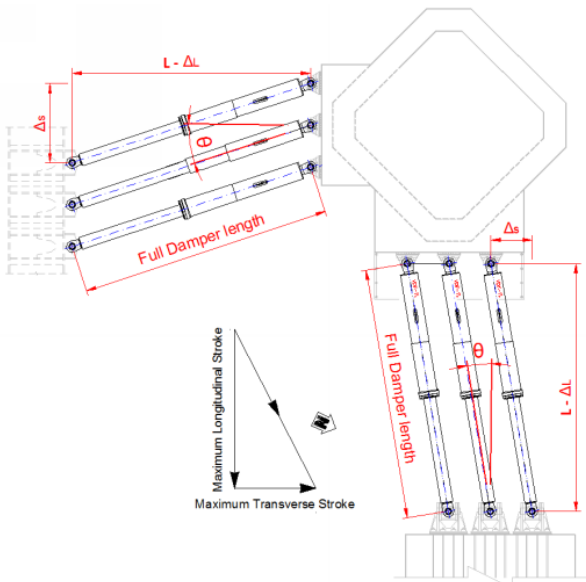

Figure 9: The viscous seismic dampers were designed for the maximum number of cycles, velocity, and displacement predicted by the dynamic model, including angular distortion. Image courtesy of Arup

Towers and end bents are simplified to be less congested with fewer items to inspect and maintain, while means of access are provided to conveniently access each viscous damper for inspection or fuse replacement without the need for hoists or manlifts.

IV. BRIDGE TOWER GEOMETRY, AESTHETICS, AND PRACTICALITY

The two 515-foot (157 m) high main span bridge mono towers are a dominant aspect of the project's visual impact and are visible from viewpoints atop skyscrapers in downtown Los Angeles some 15 miles (25 km) away. The towers also dominate the project's construction schedule and the bridge structural performance, and their geometric form is critical to these aspects.

The cross-sectional form of the tower evolved through a series of workshops between Arup and the contractor SFI. The most obvious starting point was a circular section tapering in a conical manner, as the circular form is visually appealing and well suited to the multi-directional nature of seismic and wind loading. The team reviewed a similar conical geometry adopted for the 1,010' (308 m) tall Stonecutters Bridge towers. In that case, the conical form was constructed with a self-climbing formwork system designed to adapt to the reducing radius of the section.

However, given the shorter height of the Gerald Desmond Bridge Replacement Project towers, such a specialized formwork system would not be economical. The conical form was not developed further.

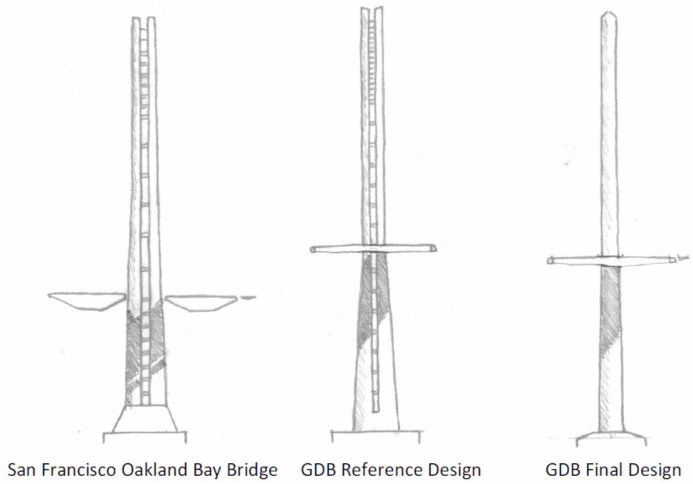

Figure 10: Gerald Desmond Bridge main span bridge tower sectional evolution. Image courtesy of Arup

Two further sections were contemplated:

- A modified cone with constant radius corners and a tapering flat section
- An eight sided geometry transforming from an octagon base to a square form at the top which was the chosen option.

The octagonal transformation is carried out by tapering four of the eight sides while maintaining the other four at a constant dimension. This approach lends itself to an efficient climbing formwork arrangement because of the eight jumping vertical formwork components, only four change dimensions at each jump.

A design decision was made to taper the faces which are orthogonal to the bridge's primary axes.

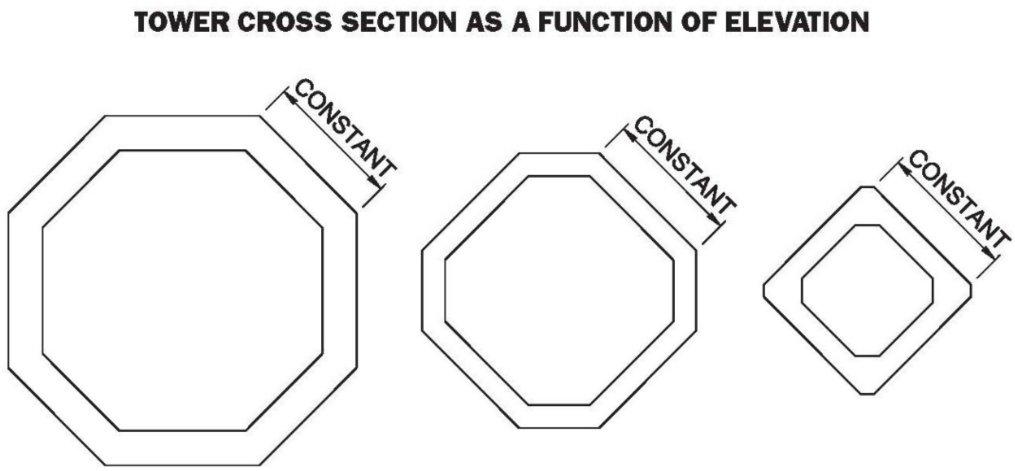

Figure 11: Evolution of tower cross section from tower base (left) to top of tower (right)

Image courtesy of Arup

Tapering faces at 45 degrees to the primary axes would have resulted in a "square geometry" that is incompatible with stay cable geometry: the fan of cables will intersect with the corner of the tower meaning that some of the anchorages will pass through the section's corner.

Keeping the diagonal faces constant resulted in a "diamond geometry", simultaneously resolving the geometric conflict between cable stays and section corners and creating a unique and instantly recognizable tower form.

The octagonal tower geometry successfully merges what are occasionally competing objectives: construction efficiency and aesthetic distinction.

The octagonal-diamond solution uses light and shadow to identify the structure's unique design at a glance while facilitating optimized construction methods and structural performance - a great example of form meets function and the total architecture perspective of design and construction.

V. DOUBLE TEXAS U-TURN ON THE BRIDGE APPROACH

The project's bid package reference design (RID) proposed a grade-separated flyover ramp for west-bound traffic seeking to exit the main roadway and cross to the southern side of the project.

Arup's value engineering identified that the same functionality could be delivered while eliminating the entire flyover structure. A roadway geometry that passed below the main roadway with a dedicated free-flowing two-lane U-turn was proposed, facilitated by a new underpass constructed through the existing main roadway embankment.

As this is a common geometric configuration in the state of Texas, the arrangement is dubbed the "Texas U-turn."

Through innovative highway engineering, the Port access roads were rearranged so that truck traffic accessing the terminal facilities would use the same underpass both to get on and off the bridge, hence the "Double Texas U-turn", see Figure 13.

The proposed solution reduced project costs by close to $70 million while providing numerous functional advantages.

Figure 12: The play of light and shadow on the octagonal towers facilitates instant identification. Photo courtesy of Arup

Land previously reserved for the RID flyover ramp bridge piers is now free to be used for other, revenue-generating purposes. It also reduced the carbon footprint associated with construction volume, as well as reduced environmental risks.

VI. HIGH LEVEL APPROACH BRIDGES – MOVEABLE SCAFFOLDING SYSTEM

Among the many priorities at the bid stage, the design and construction team focused on developing an approach bridge superstructure design optimized for the Port's objectives. Critical amongst these were:

- Defining an efficient approach viaduct superstructure. This prioritized a constant depth superstructure for construction cycle and thus schedule efficiency, long-span capability to reduce foundation quantities and standardizing the deck width through roadway geometry refinements to reduce formwork variations.

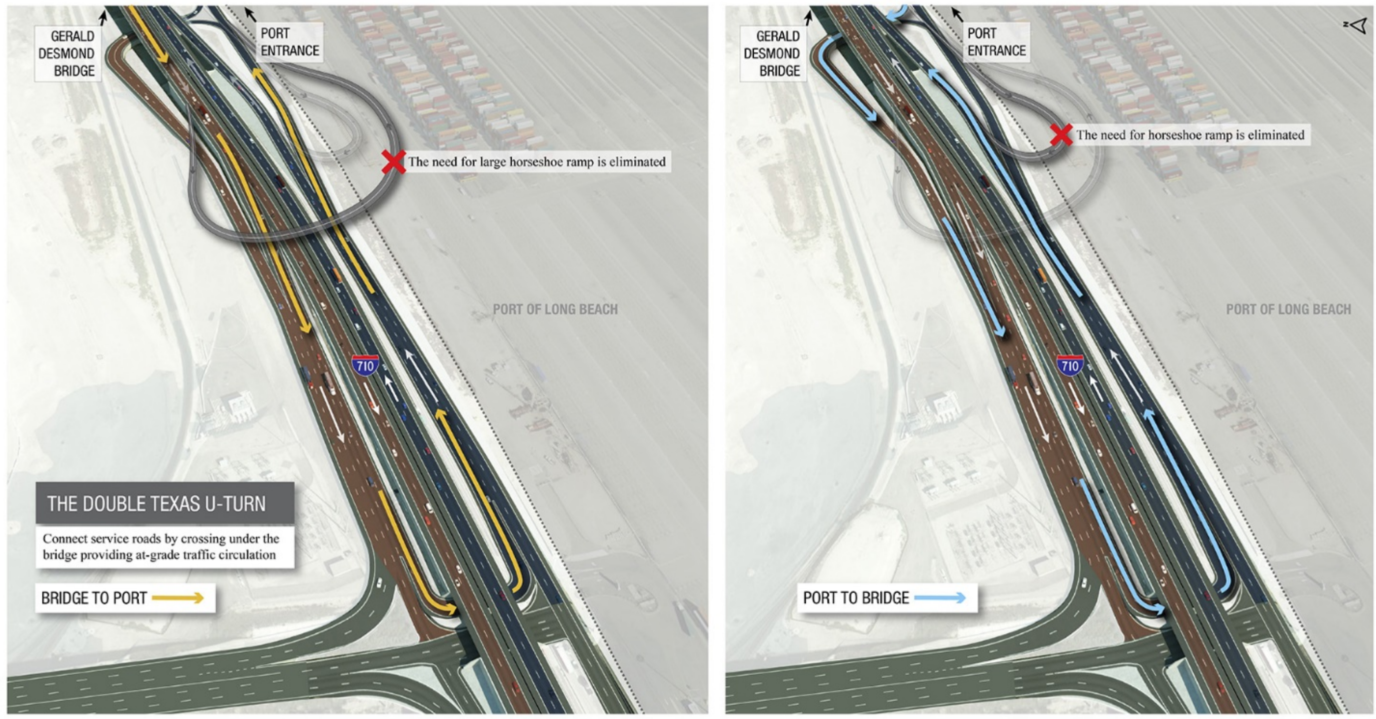

Figure 13: For the connection between the port and the bridge, Arup designed a Texas U-turn, which lets traffic change direction beneath an approach structure rather than on a horseshoe flyover. Illustration courtesy of Arup

- Minimizing the quantity of deep foundations. In addition to reducing construction quantities, this approach also minimizes interaction with legacy oil wells located throughout the project site.

Brainstorming sessions between the design and construction teams determined an optimum solution using a cast-in-place section cast on launching gantry-supported formwork, and large diameter deep foundations augmented in capacity using pile tip post-grouting.

The approach bridge structure type selection process carefully considered site constraints, construction efficiency, height of the deck above ground, and a roadway width varying from approximately 60 to 80 ft (18 to 20 m).

A constant depth, cast-in-place structure was ideal for the contractor SFI's construction methods. From the perspective of foundation quantities and utility conflicts, long-span capacity was none-the-less important.

With column heights exceeding 190 ft (58 m) at the main span bridge transition, falsework had to be minimized or eliminated.

The moveable scaffolding system (MSS) responded well to all these criteria.

The MSS uses traditional under-slung launching gantries to support formwork for single or doubled celled box girders. Final MSS gantry depth and span length was optimized by suspending the tail-end of the gantry from gallows perched on a fifth-point cantilever of the previous span, while the leading gantry end was supported by column brackets.

The contractor SFI procured two MSS systems, essentially identical except in color, so that work could continue simultaneously on the east and west approaches to the main span bridge.

The MSS was delivered in over one-hundred shipping containers and assembled on site. Heavy-lift operations were used to launch or to lower the MSS from then on.

Along with seismic loading, MSS construction loading and capacity governed the longitudinal design of the approach structures. Gallows loading of the MSS on the previous span's cantilever required two stages of post-tensioning to balance short-term and long-term effects.

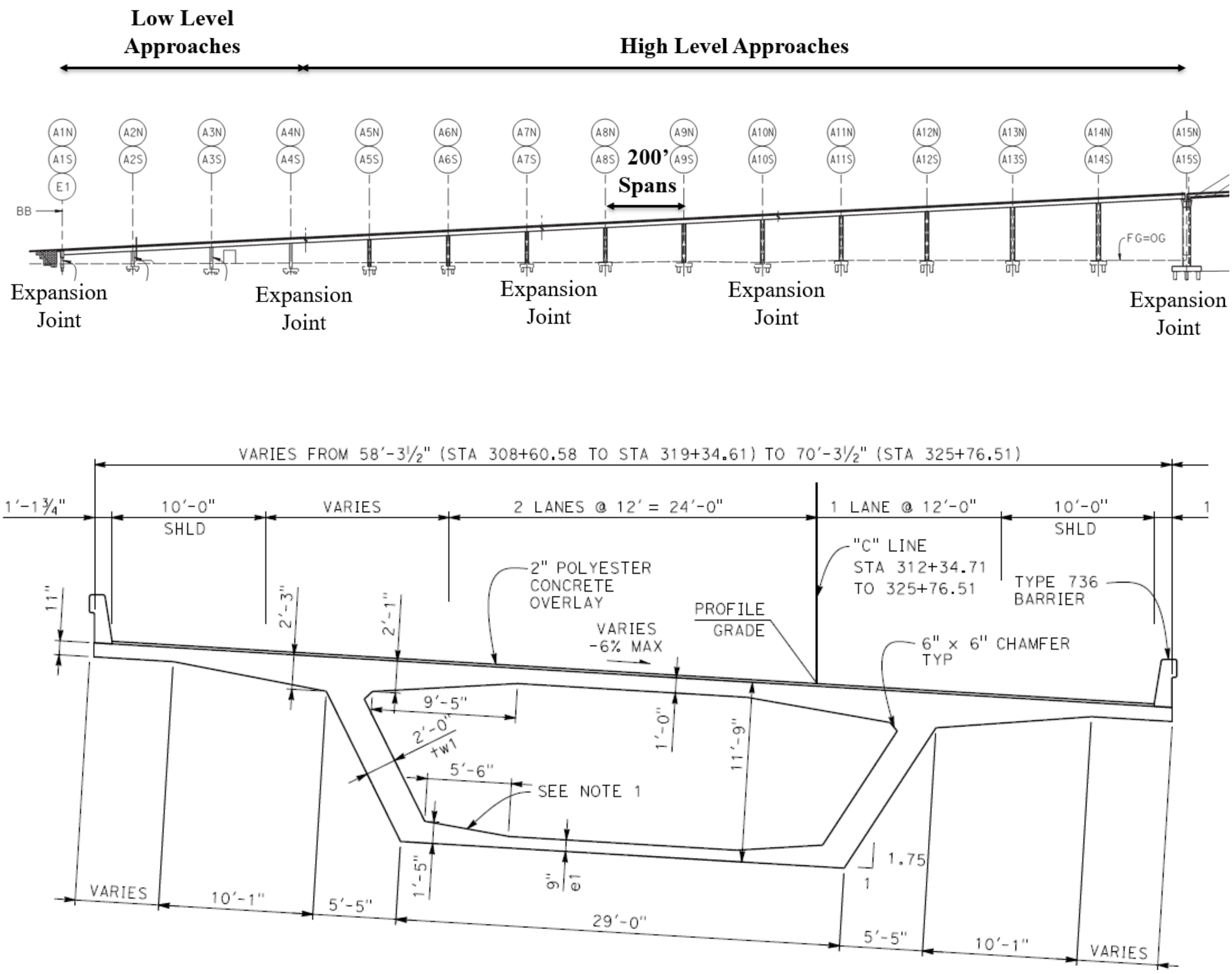

Figure 14: High Level Approach Bridge Elevation and Cross Section. Image courtesy of Arup

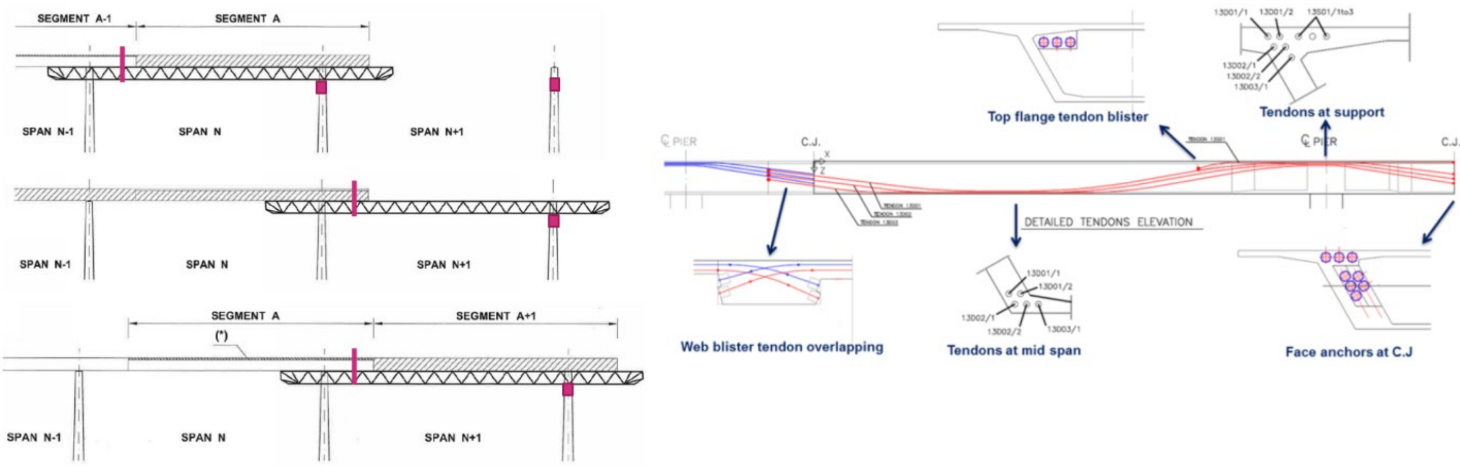

Figure 15: Construction sequence using MSS and box girder prestressing layout. Image courtesy of Arup

Figure 16: Underslung MSS ready for placement of reinforcement and prestressing.
Photo courtesy of Marie Tagudena

Figure 17: MSS heavy lift frame.
Photo courtesy of Marie Tagudena

MSS capacity limited the mass of construction loading and required specific staging of concrete casting and post-tensioning between soffit, stem and deck.

Pier cap design was tailored to account for point loading from the MSS heavy lift operation, requiring coordination of this activity with the design team.

While the MSS presents many advantages compared to traditional falsework, it also represents a significant up-front cost. This was reflected in the design by a strong push to maximize span lengths, to offset that investment through quantity savings.

VII. DEEP FOUNDATIONS – PILE TIP POST-GROUTING

To achieve the contractor SFI's performance goal of minimizing deep foundation footprints project-wide, deep foundation efficiency needed to be maximized.

This required reducing the quantity of piles by increasing pile diameter and increasing pile efficiency with pile tip post-grouting. Optimization resulted in typical square pile groups of four 6-foot (1.8 m) diameter cast-in-drilled hole (CIDH) piles with post-grouted pile tips.

Post-grouting injects grout under the base of the pile after it is installed.

By effectively pre-compressing the pile to mobilize base resistance, concurrent peak side and base resistance are mobilized, unlocking the pile's full instantaneous potential.

Lack of a codified design approach defining to what extent pile tip post-grouting improves pile performance meant that test pile data was instrumental in confirming design assumptions.

Even with test pile data, extensive design documentation beyond traditional foundation design was necessary. This is particularly true in California, as the State Department of Transportation typically does not recognize CIDH pile end-bearing (base resistance) in wet conditions.

Ultimately the team was able to demonstrate to the Department's satisfaction that through pile tip post-grouting, 1,000 psi (7,000kPa) of ultimate base resistance (at 5% pile diameter of movement) was reliable at pile tip elevations permitting significant savings on pile depths.

Different methods of delivering high pressure grout to the pile tips were tested, such as the tube-à-manchettes (TAM) method and the use of specialized grout distribution plates attached below the bottom of the pile reinforcement cage. Ultimately a grout distribution plate approach was adopted and successfully deployed on 350 large-diameter CIDH piles.

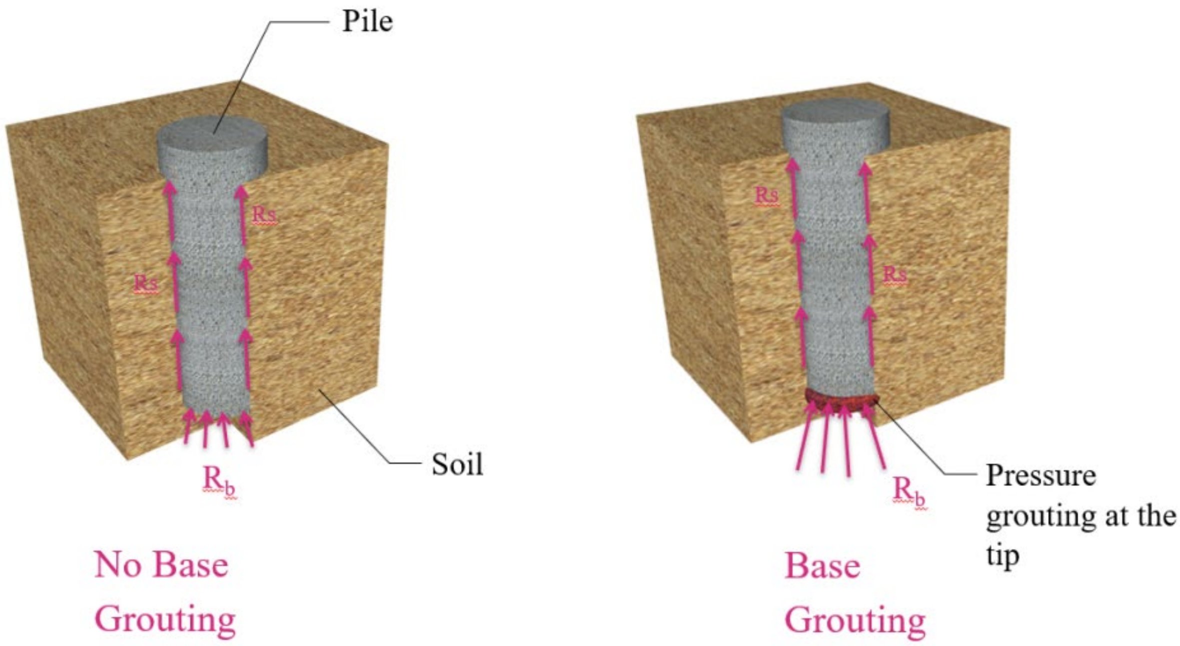

Figure 18: Illustration of the effects of base grouting. Image courtesy of Arup

→ Figure 19: Grout distribution plate. Photo courtesy of Arup

Our experience underlines that whichever pile tip post-grouting method is selected, controlling grout rheology, temperature, injection pressure and volume loss is critical.

Successful production depends on a grout mixture compatible with the selected delivery method and equipment. Several trial runs may be required to perfect the method, considering the particularities of the project geology, site conditions and plant limitations.

Even with the above considered, the contractor found that redundancy in the grout delivery system is key to consistently mobilizing the pile tip with post-grouting and maintaining the production schedule.

Prior experience in pile-tip post-grouting execution is clearly important but may be compensated for with a regime of step-by-step construction methodology validation well ahead of scheduled production.

The benefits in terms of an efficient structure are dramatic and are in line with the environmentally focused objectives of owners such as the Port of Long Beach.

VIII. CLOSING REMARKS

Many technological and performance innovations were achieved during the design and construction of the Gerald Desmond Bridge Replacement Project, of which only a very few have been touched upon here.

The end result is a structure that successfully rationalizes performance, maintenance, aesthetic and environmentally efficient objectives into a best-fit product, to the benefit of all.

The authors would like to thank the Port of Long Beach for their permission to publish this article.

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CONSTRUCTION ENGINEERING OF I-395 SIGNATURE BRIDGE, MIAMI, USA

Jeremy Johannesen

McNary Bergeron Johannesen

Figure 1: Rendering of the completed bridge. Credit: FDOT & I395-Miami.com

INTRODUCTION

The I-395 Reconstruction Project replaces the 1.4 miles (2.3 km) of 1960's era elevated viaduct in Miami, Florida. In addition to improving safety and capacity, the east end of the project will feature a new Signature Bridge that will span 1,200 ft (366 m) and create a new landmark in the city.

This bridge is currently under construction with completion targeted for late 2027.

The project was tendered in 2017 as a fixed-price design-build project. The contractor, Archer-Western de Moya, JV was selected from three competitors as the best-value design.

All proposals were priced at approximately \$800m with the primary differentiator being the design of the signature bridge. In this process, the design and details are driven by architectural considerations which have required interesting and novel construction methods to achieve.

Construction of the bridge is staged to allow the new Westbound lanes to be completed and traffic diverted before removal of the existing viaduct. This results in a split structure, having a westbound and eastbound deck structure separated by a center pier where all six arches converge.

While two of the arches are entirely located between the WB and EB decks, the other four arches straddle the new roadway in an 'X' arrangement. These four arches are referred to as the 'diagonal arches' and carry most of the structural weight.

The arches are precast segmental structures. This method of construction is well suited to arches, since, like a stone arch, the dead-load compression compensates for the lack of continuous reinforcement at the joints. However, the unique requirements of this project introduce additional challenges, not expected in traditional arch construction.

The defining feature of the structure is the six arch ribs which spring from a common center pier to support the suspended deck, see Figure 3.

The largest arch stands over 300 ft (91.44 m) tall and spans over 600 ft (183 m).

DESIGN CONSIDERATIONS

The most challenging consideration in the design is the asymmetry created by the suspender arrangement.

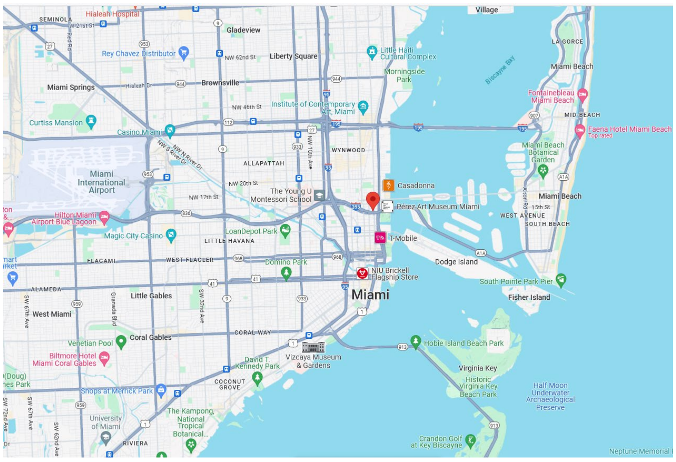


Figure 2: Location of the Bridge.
Click on the map to open Google Maps

PROJECT INFO

OWNER:

FLORIDA DEPARTMENT OF TRANSPORTATION

CONTRACTOR:

ARCHER WESTERN DE MOYA, JV

ENGINEER OF RECORD: HDR

PRECAST SUB-CONTRACTOR:

RIZZANI DE-ECCHER

CONSTRUCTION ENGINEER:

MCNARY BERGERON JOHANNESEN

The out-of-plane component of the cable geometry reverses direction along the length of the arches, meaning that the diagonal arches are loaded transversally.

While arches are inherently strong for in-plane loads, lateral loads generate a significant amount of bending and related post-tensioning.

More interestingly, the lateral loads require the post-tensioning layout to be laterally asymmetric in the diagonal arches.

In addition to tourists and mosquitos, Florida has the seasonal threat of hurricanes. In this and other arch designs, wind demands can be mitigated with large chamfers on the corners of the arch rib.

However, in hurricane alley, these resulting wind loads are always significant and present a design challenge. This is complicated by the fact that the structural configuration during construction and service differ significantly.

For instance, the arches must be able to resist hurricane winds in a free-standing configuration, whereas during service, wind demands act in combination with the lateral loads imposed by the suspender cables.

These stages require different strength requirements which results in some post-tensioning tendons being temporary for construction and other tendons stressed later in combination with the asymmetric suspender cable tensioning.

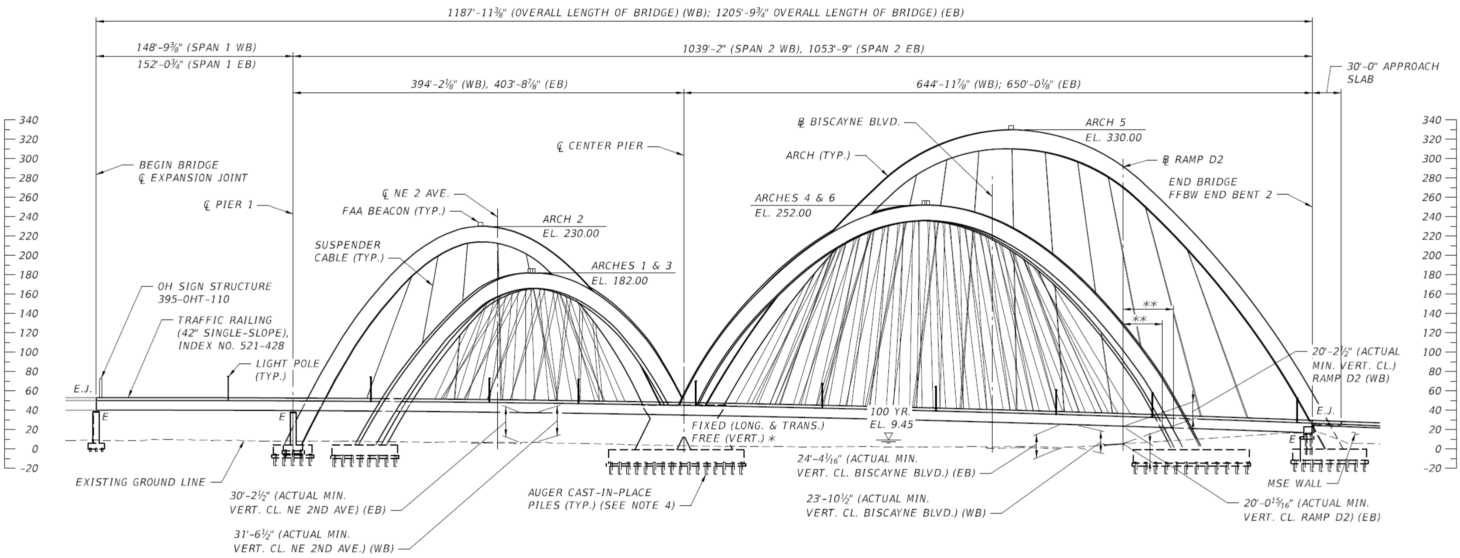


Figure 3: Elevation of the Bridge. Click on the image to open it in a higher resolution

In order to span the streets and utilities within the project site, the arches are not tied. While thrust-arches transmit large horizontal loads and are generally founded on rock, Florida's geology requires deep foundations to support the arches.

Taking a note from local high-rise construction, auger-cast-piles are used in the design. With these deep and relatively slender piles, the foundations experience both horizontal and vertical displacement under load and produce flexural demands in the arch ribs.

Since 2015, the Florida DOT has mandated the use of flexible filler instead of cement grout in post-tensioning for all bridges. This material has been in use in nuclear installations and other types of construction for some time as it permits removal and replacement of tendons.

However, it also requires special measures in design and construction. For bending capacity, the tendons are unbonded, thus the ultimate capacity of the tendons is generally equal to the as-installed force.

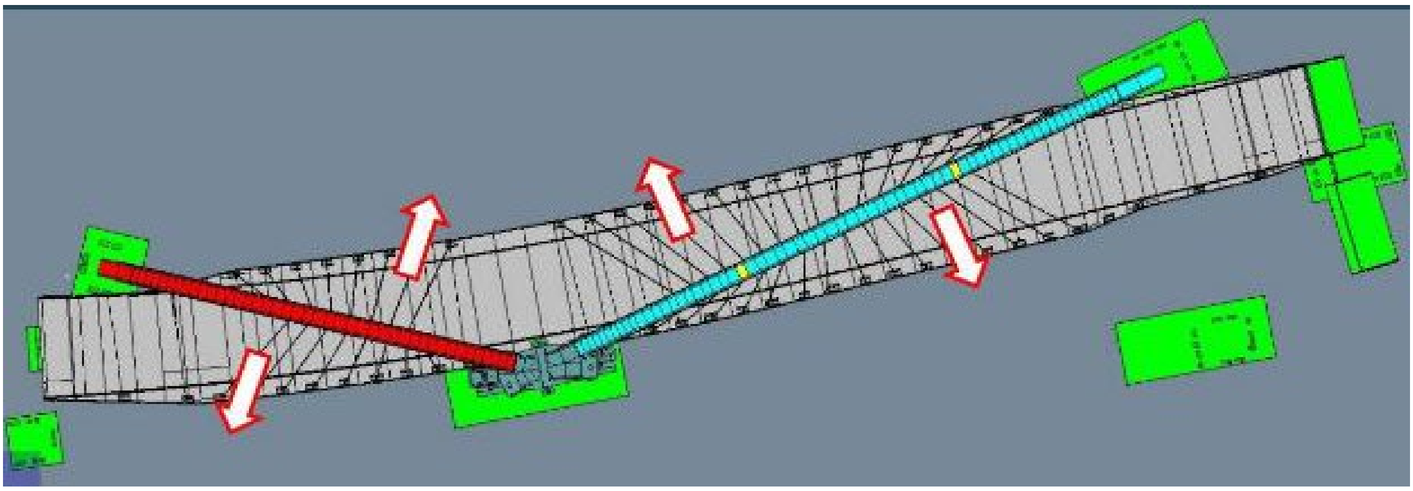


Figure 4: Schematic behaviour of the suspender loading on the diagonal arches. The cable geometry produces an out-of-plane load which reverses direction along the length of each arch.

For shear capacity, sections with unbonded post-tensioning require that the entire area of the duct is discounted when considering ultimate capacity and principal tension stresses. This results in additional concrete being required for shear demands. The overall result is a heavier structure since the strand and cross section are not being fully utilized.

Similarly, in construction, the system requires increased duct spacing and edge distance requirements at the segment joints as well as heavier duct and segmental duct couplers to prevent leakage.

ARCH CONSTRUCTION

The precast segmental arches are constructed using ground-based cranes and are erected in progressive cantilever over spans of approximately 70' from one shoring tower to the next.

Near the base, the arches can slope as much as 70 degrees from horizontal and then vary to a nearly flat profile at the keystone closure.

To accommodate this large variation in angle, segments are lifted using trunnion attachments on the outside faces of the webs. These connections use a conical insert and pre-formed hole to transmit an eccentric load to the concrete and allow segments to be lifted from a flat orientation on the transport truck and rotated up to 90 degrees for erection on the arch.

The everchanging slope of the arch requires various access devices for the crews to do their work safely and efficiently.

Figure 5: Segment rigging lug. Pre-formed holes on the sides of the precast segments receive the conical insert and the bars are secured by knocker-wrench.

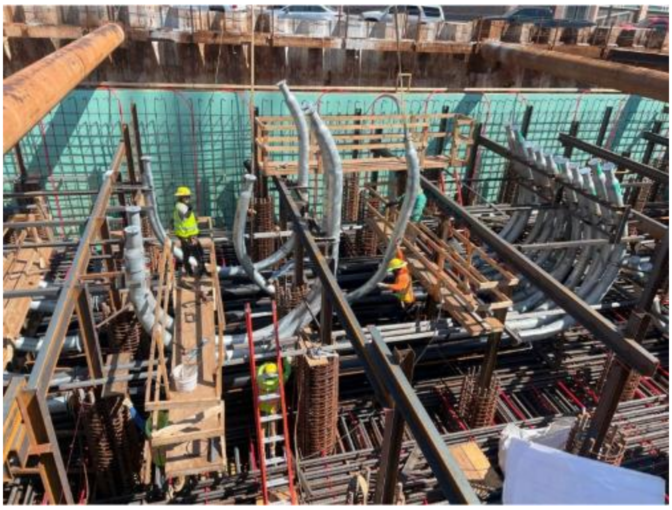


Figure 6: Footing construction. The curved, galvanized steel route post-tensioning cables from the arch ribs to anchorages that are accessible at the top of the footing.

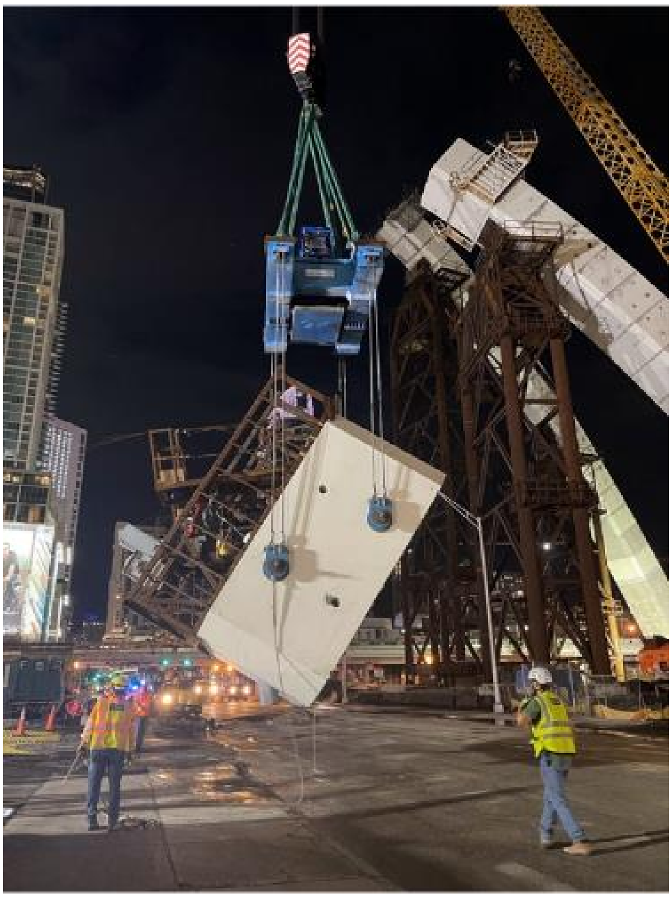


Figure 7: Arch segment hoisted for erection. Rigging connections (trunnions) on the sides of the segment allow rotation from 0 (on truck) to 90 degrees.

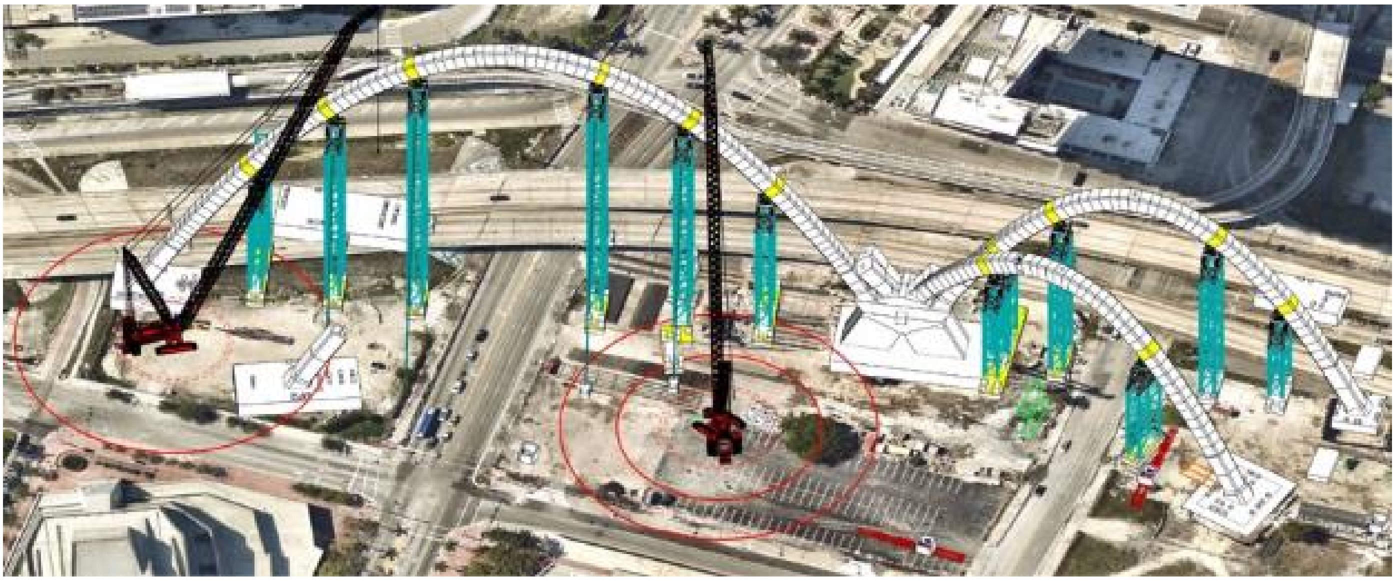


Figure 8: All arch segments are crane erected. This figure shows the 3D modelling used for planning of every crane pick to confirm reach, capacity, and conflict avoidance.

The 'trailing edge' platform provides access to the outside of the work heading in order to apply epoxy to the segment joint as well as providing tool and material storage. A 'leading edge' platform provides access within the lifted segment to install and stress erection tendons.

While the individual arch segments are cast in a rectangular mould, the lower face is skewed to achieve the overall arch shape with the angle change between segments varying as much as five degrees.

Precast segmental construction typically uses high-strength post-tensioning rods to squeeze the epoxy and support the cantilevered segments. However, straight bars in a curved arch did not prove to be a viable option and a strand-tendon

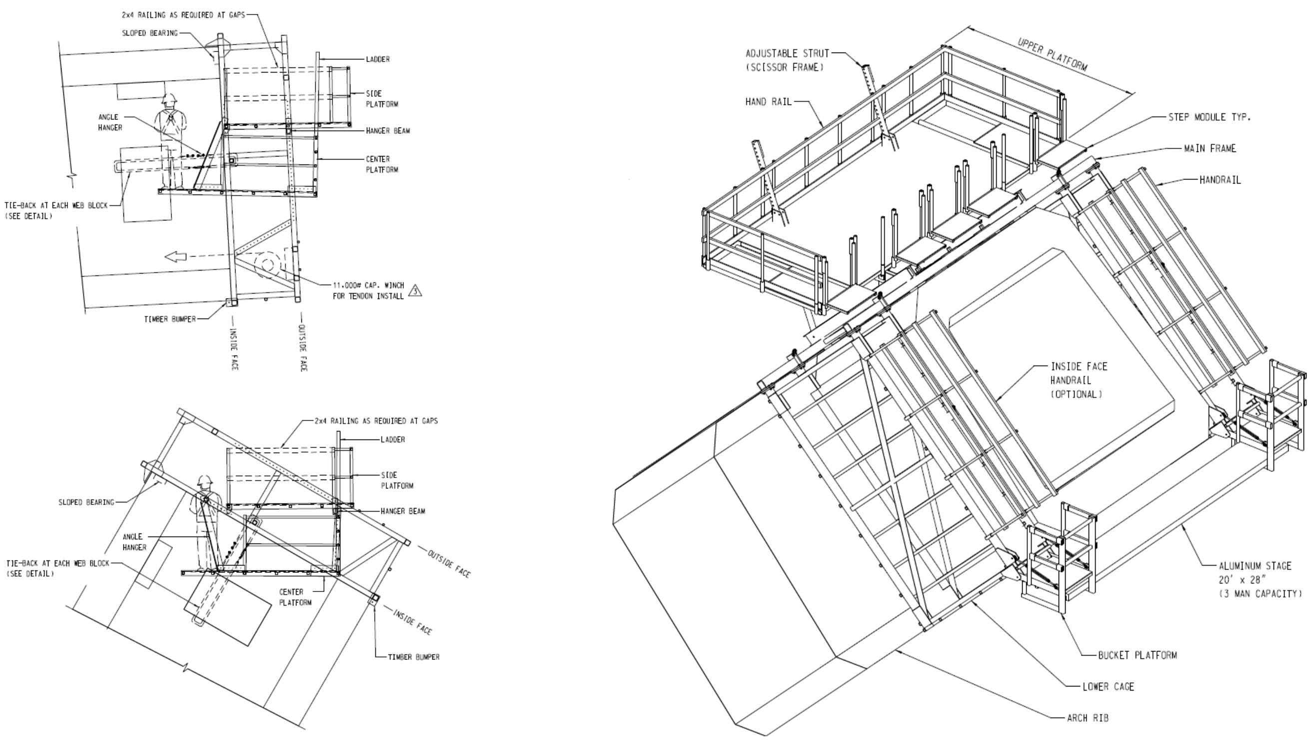


Figure 9: Drawing on the left shows the 'leading edge' platform used access the erection tendons. Figure on the right shows the 'trailing edge' platform used for material storage and applying epoxy to the joint.

option was developed. With this concept, all of the segments are equipped with erection PT blocks having a series of cast-in diabolo holes located on the inside of the top and side faces of the segments.

Erection tendons are secured with face-mounted anchorages and anchor at a block several segments down-arch from the leading edge. The curved diabolos provide a deviation point in each segment such that the erection tendons match the profile of the arch. While the tendons have some curvature due to the arch profile, they are short and straight enough to allow each strand to be individually tensioned with a monostrand jack.

Together, these details (lifting, access, and erection tendons) were developed by MBJ as an integrated system for the erection of the arches on this project.

STARTER ERECTION

Segment erection first requires fixing the first precast segment to the cast-in-situ arch-starter. While there are caveats, the erection tolerances for precast segmental construction on the project are defined as an L/1000 envelope that begins at the first precast segment.

In order to set and aim the precast segment to the necessary precision and given the limits of survey

Figures 10 - 12: ↖ Temporary erection tendons at the interior of the top / 'extrados' flange of the arch rib.

↑ Temporary erection tendons at the interior of the webs. Note lifting sleeve holes.

↓ Web erection block reinforcing in the precast mould. White plastic void formers create diabolo shaped holes to anchor and deviate tendons. Void formers for lifting can also be seen in the wall.

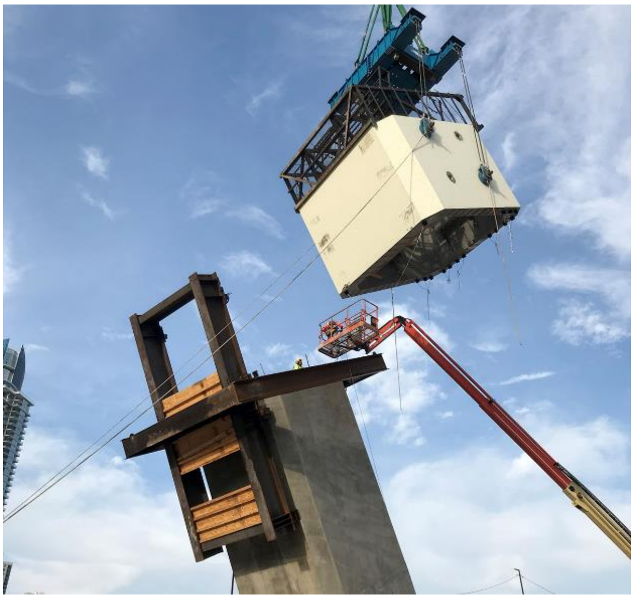


Figure 13: Precast segment being placed at the top of the cast-in-situ arch starter. The starter frame is used to stabilize and adjust the segment prior to casting a wet-joint.

Figure 15: Alignment jacks at the wet-joint between the starter and precast segment

accuracy and adjustment, two segments are assembled prior to casting the wet joint. This increases the effective length of the piece being aligned.

A temporary 'starter frame' is used to support the two segments, assembly on the in-situ starter which is constructed on the foundation footing.

While the starters can be relatively steep, the frame stabilizes the segment from overturning or sliding off the top face. This frame is configured as a basket-handle to both minimize embeds and provide access points for rigging to erect and remove the frame.

With the two segments erected on the frame, jacks located in the wet-joint are used to adjust the trajectory of the precast elements.

Once set, the jacks are secured with lock rings and concrete blocks are cast in the wet-joint to hold the geometry and allow removal of the hydraulic jacks prior to concreting.

Figure 14: → After aligning the precast segment concrete blocks are cast to allow removal of the hydraulic jacks in the closure. Ducts are coupled and splice prior to tying the reinforcement, forming, and pouring.

Note that the flexible filler tendons use smooth, heavy plastic pipe and coupling system.

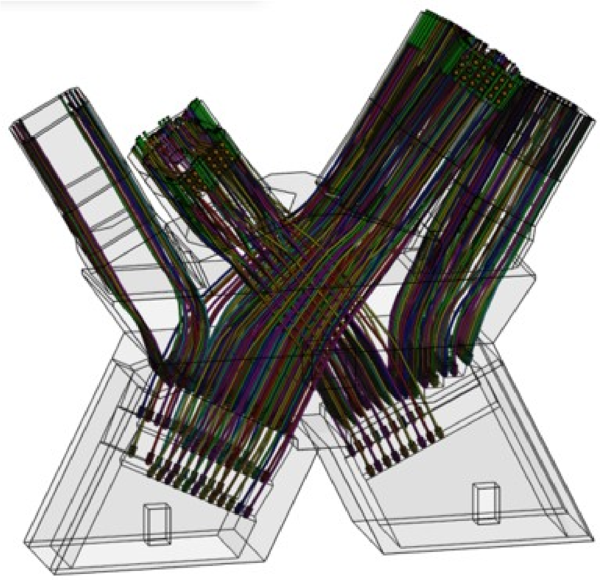


Figure 16: 3D model of the post-tensioning tendons in the center pier

Figure 17: Post-tensioning anchorages at center pier prior to rebar placement

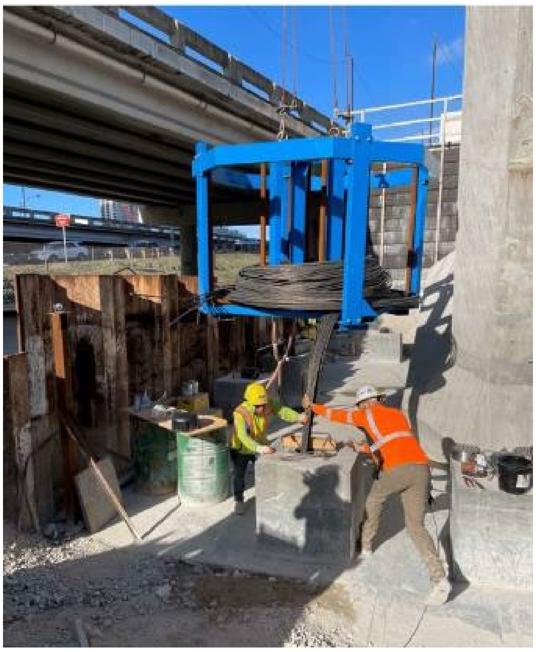
CONTINUING UP THE ARCH

After the starter segments are erected, segment placement proceeds toward the 'key-stone' closure at the top of the arch.

Cantilevering the arch from tower to tower requires post-tensioning to supplement the external erection tendons. These tendons are internal (inside the concrete cross section and not exposed) and generally but not always permanent.

Where they anchor along the arch, the tendons are anchored in anchor blocks (i.e. blisters) inside the box.

At the center pier, many of the tendons stressed during erection find their way to the 'ceiling' of the center pier legs.



Figures 18 and 19: Permanent post-tensioning installation and stressing

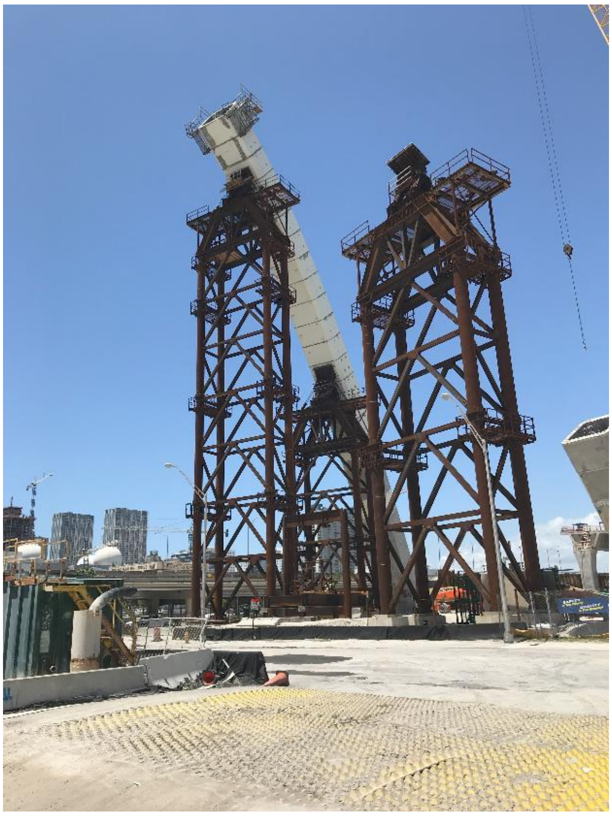


Figure 20: Temporary falsework towers for arch erection

Figure 21: Segments erected in progressive cantilever from tower to tower

At the outer legs of the arches many of these tendons extend to the arch footing where they are routed thru J-bends and anchor on top of the footing.

All of the permanent tendons in the arch are 27 x 0.6” (15.2 mm) which, because of the heavy jack and hardware, requires significant effort and ingenuity to access and stress.

With completed spans of 350 ft (107 m) to 650 ft (198 m), the arches require temporary shoring during construction. These shoring towers are configured as 32 ft (9.8 m) x 3.7 ft (12 m) in plan view and have 36” (0.9 m) steel pipe pile legs.

The towers, like the permanent structure, are founded onauger cast piles and designed to take high lateral loads associated with hurricane level winds. In round numbers, the allowable design loads for the towers is 4,000 kips (1,814 tons) vertical in combination with 400 kips (181 tons) of transverse lateral (out-of-plane) load.

Longitudinal (in-plane) loads are much less since the arch braces the tower.

As a result, the towers are relatively narrow in this direction – not only because the demands are less but to provide flexibility in the longitudinal direction to absorb thermal expansion and other deflections in the arch rib.

Because the wind demands require the tower to be relatively wide for stability, a header truss is used to transfer the arch reaction to the tower legs.

This truss also supports several levels of framework which are necessary to make the tower fit-up to the arch with full consideration of as-built tolerances and potential movements.

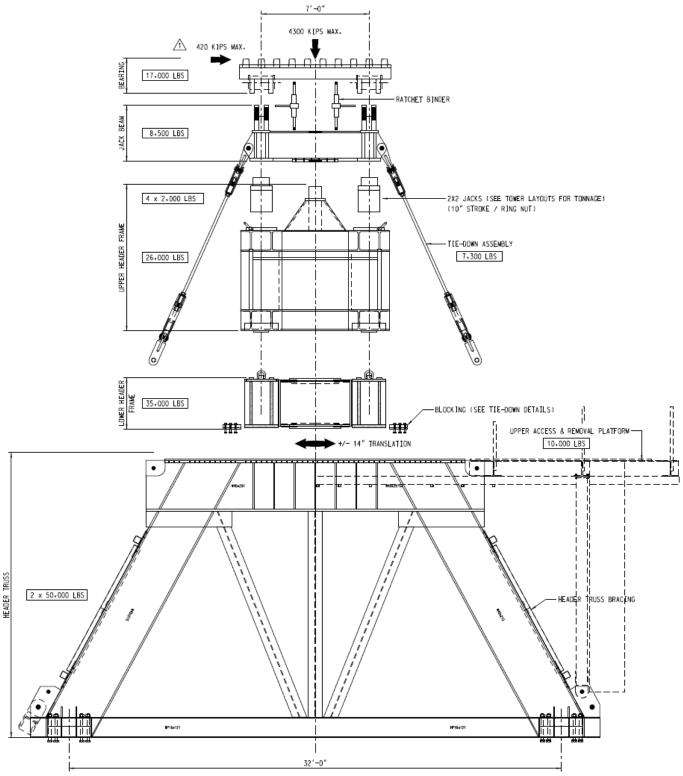
The tower connections are designed to accommodate a range of arch geometry.

Near the base of the arch where the profile is steepest, the load path from the tower into the arch is almost entirely a shear force.

In order to transmit vertical load into an almost-vertical face, the towers are equipped with bearings having shear pintles.

These steel pintles fit into pre-formed pockets on the bottom of the segment and grouted in place.

The bearings then connect to the tower through a pin connection which allows the bearing to match the slope along the arch.



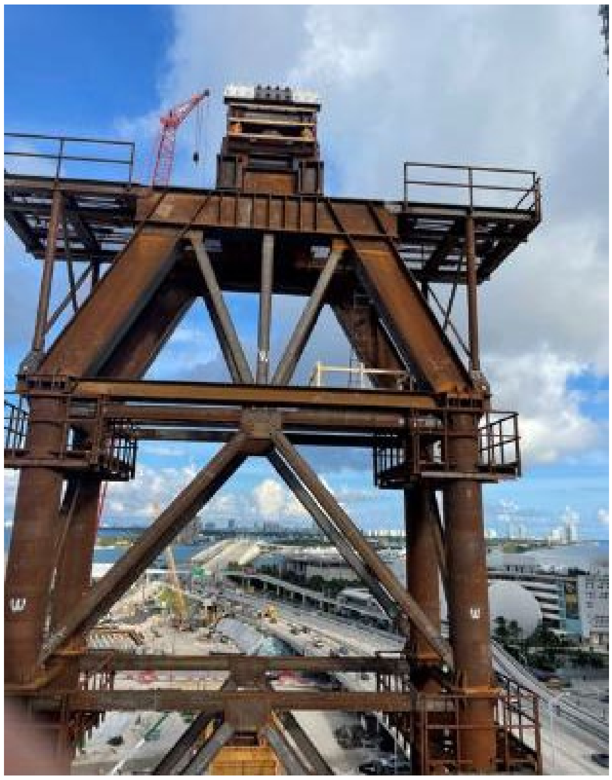


Figure 22: Drawing on left illustrates components at top of tower. The truss supports several layers of beams which allow horizontal, rotation, and vertical adjustment for fitup and adjustment of the arch. Tie-downs are also used to ensure stability in hurricane winds.

Figure 23: Temporary bearing at falsework tower. Hydraulic jacks below the bearing are used for fitup and vertical load adjustment. The bearing can rotate to match the profile of the arch.

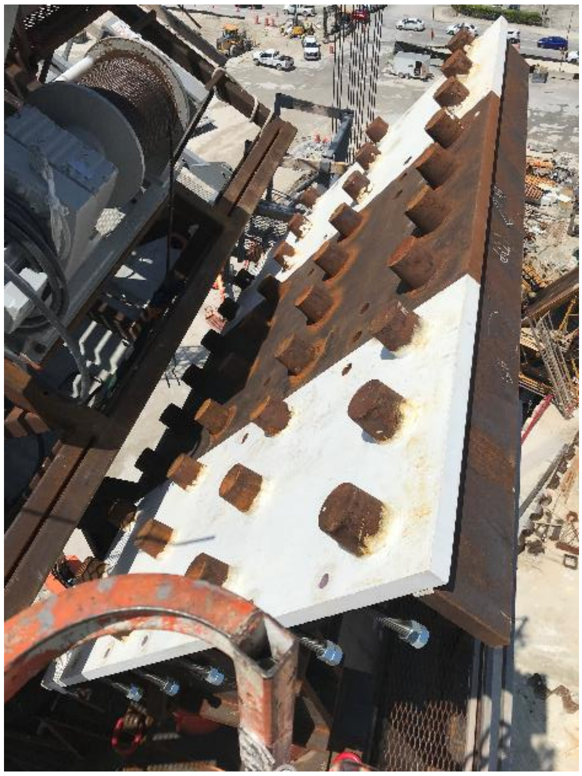


Figure 24: The sloped structure profile requires a shear mechanism to transfer vertical load into the arch rib. The steel pintles shown here are inserted into oversized pockets on the bottom of the arch rib and grouted. The bright steel bolts on the bearing are used to push the pintles out of the grouted pockets when the tower is removed.

ARCH CLOSURE

Once the precast segments are erected, the key-stone closure at the top of the arch is cast. A key consideration when making the closure are thermal effects on the now continuous arch. Specifically, differential temperatures between the steel falsework and concrete arch produce compression in the towers and tension in the arch which can overstress the arch.

In order to control thermal loads, the closure pours are timed to maximize curing during night time hours while thermal gradients are minimal. This is done with the purpose of stressing several continuity tendons prior to dawn at which point tower loads are released in conjunction with additional tendon jacking until the arch is free-standing. The towers remain in place some additional time to provide lateral support to the arch while the remainder of the permanent tendons are installed and the arch is capable of resisting wind loads.

SEGMENT PRECASTING

The precast arch segments, which weigh as much as 200 kips (90 tons) each, are cast approximately 10 miles (16 km) from the project site and transported by truck. Segments are match-cast in the short-line method to achieve the final geometry and camber.

Because the bridge deck hangs from the arches, many of the segments have suspender cable anchorages and guide pipes which extend through the bottom/intrados face of the arch rib. The suspenders consist of multi-strand cable assemblies as are used in cable-stay construction. In order to provide better access and install the cable anchorages and guide pipes, the arch segments are precast upside-down.

This has other practical advantages as it puts the geometry control points used for match-casting on the bottom face of the arch making them more easily visible for arch erection. Tracking the erection geometry is important since a single casting unit has up to 44 segments all match-cast together.

Meeting at the closure within acceptable tolerances requires corrective shimming along the way. Whereas erection geometry for most segmental structures is primarily concerned with vertical deflections, the 2D arch profile combined with lateral effects due to asymmetric post-tensioning requires consideration of deflections in all three axes. During erection, survey of the arch uses prisms that are fixed to the bottom of the arch, allowing the points to be shot from a total station on the ground. These results are then compared against the deflections predicted in the construction analysis.

Figure 25: Suspender segment showing asymmetric cable anchorages. Note asymmetry in PT duct arrangement.

Figure 26: Suspender segment showing suspender pipe. Segments are cast and stored upside-down.

Figure 27: Intrados face of suspender segment. Guide pipes for suspender cables extend out of concrete. Grout tubes for vertical post-tensioning bars can be seen as well as the survey embeds along lower edge of joint.

Figure 28: Foam void formers for suspender anchor blocks. Complex geometry is achieved by cutting foam blocks to shape using CNC router. This geometry is taken directly from the 3D CAD models.

While upside-down casting benefits the surveyors, it does not simplify the fabrication of a structure which is also asymmetric and where all of the 345 precast segments are unique.

In order to coordinate shop drawing details, all of the arch segments were individually modeled in 3D and then inverted for reference in the shop drawings. All of the 3D modeling and shop drawings is done with Bentley Microstation.

Segments are reinforced for shear and torsion demands using fy=75ksi rebar, with reinforcing densities ranging up to 400 lb/cy (240 kg/m^3). These densities dictated the need to use self-consolidating concrete with a required design strength of 10,000psi (70MPa).

The most complicated fabrication details are found in the suspender segments, and are related to the cable anchorages. Because of the architectural requirement of the project, these segments have a broad variety of concrete geometry and reinforcing arrangements.

In order to form the anchor blocks, the 3D models developed in the shop drawing process were used to define 'foam-work' void formers which were fabricated with the use of a computer numerically controlled (CNC) mill. These void formers are then assembled in the moulds to form the complex geometry of the finished segments.

CONSTRUCTION ANALYSIS

In conjunction with the tangible details illustrated here, the construction of this bridge requires extensive analysis. The full construction analysis of the bridge involves over 4,000 members, 2,000 tendons and 5,000 analytical steps. The analysis requires 54 hours to run and generates 279 gigabytes of results. This detailed construction analysis is performed with LARSA4D. Because of the complexity and long run-time, LARSA developed a 'RE-START' feature for use on this project. This allowed the model to be re-run from a specified stage in the analysis, avoiding processing time for the previous construction stages.

For the purposes of developing cable stressing protocols and other detailed procedures, this feature has saved thousands of hours of computing time.

The purpose of this analysis is to provide the information necessary to construct the bridge. Deflections are used for casting camber and tracking the erected geometry. Stress results are used to ensure that the bridge remains within allowable limits during construction. Forces results are used both to define the requirements for the temporary works and ensure that the erection procedure stays within design limits. PT and stay cable results provide elongation and force data used in the stressing and quality control procedures.

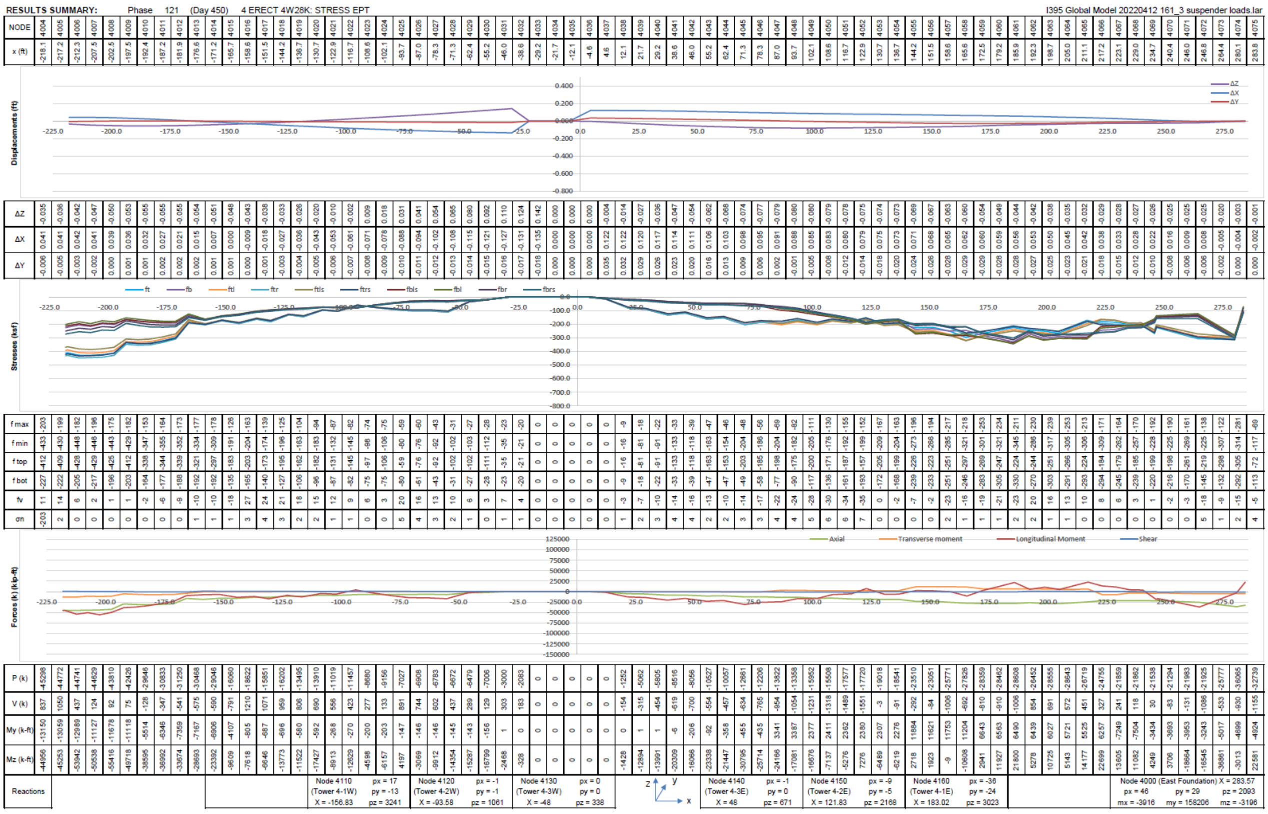


Figure 29: Construction Analysis 'Dashboard'. Deflections, stresses, and forces are reported for each stage of construction.

Broadly speaking, this information is used to help the contractor deliver a structure that will perform as intended but it requires some effort to determine what out of that 279 GB is relevant.

One useful approach is reporting stage results in a 'dashboard' format. This provides a graphical snapshot of the results at each stage of the analysis, allowing one to quickly visualize where forces or stresses are near their limits or maximum demands as well as paging ahead and back to understand the relative change in results due to an operation.

CONCLUSION

Many aspects of construction for this bridge are unique and have required modification to traditional construction methods resulting in sometime interesting and innovative solutions. In the role of construction engineers, it has been our goal to make this vision constructable and as efficient as practical. To that end, we have adapted ideas and lessons learned from our experience across the industry to tackle unique problems.

This bridge was conceived as a landmark to stand beside Miami's growing skyline and provide an impressive transition during commercial breaks of Monday Night (American) Football. As with any endeavour, the engineering is being brought to life by the hard work and dedication of the men and women on site. It is their work that will endure as a new symbol of Miami.

Figure 30: Segment erection crew. Credit: FDOT & I395-Miami.com

PREPARING THE BENJAMIN FRANKLIN BRIDGE FOR THE NEXT 100 YEARS

Joshua Pudleiner, PE, STSC; Barry Colford, PE

AECOM

BACKGROUND

The Benjamin Franklin Bridge in Philadelphia opened to traffic on July 1, 1926, three days ahead of The United States of America's 150th Anniversary of Independence and was dedicated as part of the 1926 Sesquicentennial Exposition. Originally named the Delaware River Bridge until 1955, it was the longest suspension bridge in the world with a main span of 1,750 ft (533 m) until the opening of the Ambassador Bridge in 1929.

Each main cable is comprised of high tensile steel galvanized parallel wires, 0.196 inches (5 mm) in diameter. There are 61 strands of 306 wires each for a total of 18,666 wires making up each cable compacted to a nominal diameter of 30 inches (762 mm) under the helical galvanized wrapping wire.

The Benjamin Franklin Bridge, carrying Interstate 676/US Route 30 over the Delaware River, connects the city of Philadelphia, Pennsylvania with Camden, New Jersey, see Figures 1 and 2.

It contains seven lanes of vehicular traffic divided by a "zipper" barrier which can be shifted using a special barrier transfer machine to configure the lanes for traffic volume or construction. Pedestrian/cyclist walkways run along the north and south sides of the bridge and are elevated and outside the vehicular traffic envelope.

Uniquely, the bridge also contains an outer pair of rapid transit rail tracks on each side of the deck that were originally put into service in 1936 and since 1969 have carried the Port Authority Transit Corporation (PATCO) Speedline passenger rail.

The bridge is owned and operated by the Delaware River Port Authority (DRPA), officially the Delaware River Port Authority of Pennsylvania and New Jersey, which is a bi-state agency instrumentality created by a congressionally approved interstate compact between the state governments of Pennsylvania and New Jersey.

The authority is principally charged to maintain and develop transportation links between the two states with four bridges and a mass transit (PATCO) rail line across the Delaware River. Though the DRPA has "port" in its name, it does not own or operate any ports.

Along with the Benjamin Franklin Bridge, there are three other long span bridges owned and operated by the DRPA across the Delaware River between Pennsylvania and New Jersey.

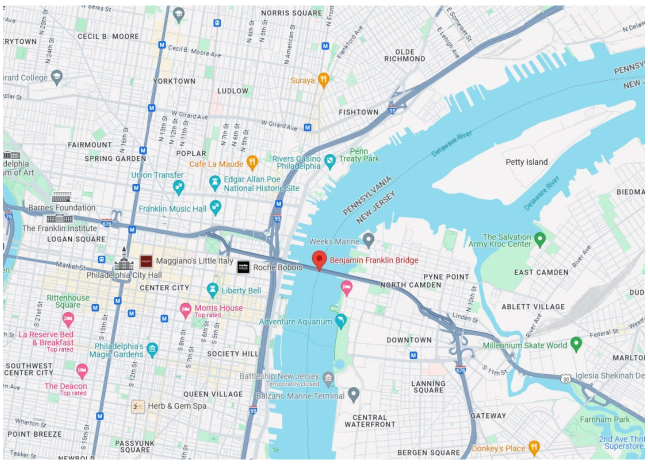


Figure 1: Location of the bridge.
Click on the map to open Google Maps

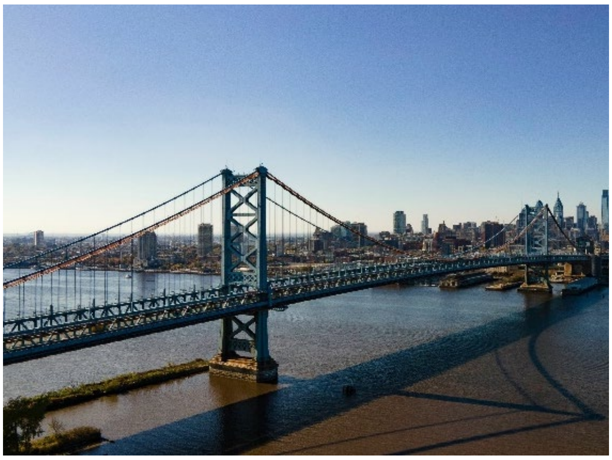



Figure 2: General views of the Benjamin Franklin Bridge, looking toward Philadelphia, PA

They are the Walt Whitman Bridge, the Commodore Barry Bridge, and the Betsy Ross Bridge. All four bridges charge a $5 westbound toll for passenger vehicles (that is into Pennsylvania).

PROJECT DETAILS AND SCHEDULE

The Benjamin Franklin Bridge Rehabilitation of Suspension Spans and Anchorages Project (Contract Number BF-54-2019) consists of five separate design projects combined together into one construction project. The combination of projects was chosen in order to improve the construction coordination among work locations and improve construction access to shorten durations, improve work processes particularly the limited space available in the cities of Philadelphia and Camden, and reduce impacts to PATCO service by allowing multiple work locations during track outages. The general contractor, Skanska began construction on this $216.9 Million project in early 2020 and it is scheduled to be completed late 2024.

The project parts are as follows:

- **Part 1:** Main Cable Dehumidification
- **Part 2:** Walkway Preservation
- **Part 3:** Maintenance Painting and Steel Repairs
- **Part 4:** Decorative Lighting Upgrade
- **Part 5**: North Walkway Widening

The majority of the work consists of repairs to various steel components on the suspended spans, maintenance painting and installing a dehumidification system for the main cables.

The work also includes repairs to reinforced concrete and steel members and painting inside the anchorages, rehabilitation of the North and South Walkways; replacing decorative lighting; and widening of the North Walkway in Camden.

The steel repairs include replacement of the critical components of the suspension system including pin and link assemblies at both the main towers and anchorages using temporary bracing and jacking. The steel repairs also include replacement of the wind lock assemblies and repairs to the lateral bracing.

Access for the work on the suspended spans includes underdeck platforms that also provide containment for the painting operations and main cable access platforms (MCAPs) that provide access to the main cables.

Work associated with the main cable dehumidification includes constructing an enclosure inside the Philadelphia Anchorage and installing a new dehumidification system that pumps air, with a relative humidity of 40% or less, through the main cables. The dehumidification system will preserve the suspension cables and extend their useful life. An existing dehumidification system in the anchorages is being replaced and integrated into a standalone SCADA (supervisory control and data acquisition) system that can be remotely monitored.

The MCAPs are also being utilized to replace all the cable band bolts and some hand rope stanchions. A pair of suspender ropes were removed and replaced at the mid span panel point on the north cable (PP 77).

The removed ropes are being tested to determine if there has been any loss of strength and to estimate remaining service life. An internal cable inspection was carried out at the two midspan panels on either side of PP 77 and a large number of broken wires were repaired.

An existing acoustic monitoring system installed in 2001 has been decommissioned and is being fully replaced to provide continuous remote monitoring of the main cables and detect and pinpoint potential wire breaks.

Part 1: Main Cable Dehumidification

As early as 1969 water was observed dripping from the main cables of the Benjamin Franklin Bridge. In 1988 the observation of corroded, broken wrapping wire initiated additional inspections. Since this time the DRPA has been taking a proactive approach to monitoring and exploring solutions to preserve the main cables which are the most critical, non-redundant element of a suspension bridge.

In 1995 the first internal main cable inspection was performed, and as a result the DRPA proceeded with oiling the entire length of the main cable in an attempt to slow and prevent additional corrosion of the main cable wires. Furthermore, the cable splays were enclosed, and an anchorages dehumidification system was installed in both the Philadelphia and Camden Anchorages.

In 2016 the third internal main cable inspection was performed and following the results of that inspection, the DRPA decided to move forward with retrofitting the main cables with a dehumidification system.

The main cable dehumidification system designed for the Benjamin Franklin Bridge has a buffer chamber, a controls and monitoring system and an airtight wrapping and sealing system on the main cable. The buffer chamber was constructed by sealing a large existing chamber within the Philadelphia Anchorage which involved concrete repairs to the four walls and floor of the chamber, and by installing a structural airtight roof.

Mechanical equipment preassembled on skids lifted using cranes and lowered into the chamber. The processed dehumidified air is supplied from the buffer chamber via fans to the main cable injection sleeves as well as the anchorages splay chambers in both Philadelphia and Camden. Air from the main cable exhausts at intermediate locations between injection sleeves, at the tower top enclosures and into the anchorage splay chambers, refer to Figure 3 for the system layout.

The controls and monitoring consist of a supervisory control and data acquisition (SCADA) system that was installed to gather and analyze real time monitoring data. Sensors at the injection sleeves, exhaust sleeves, monitoring points, splay enclosures and inside the buffer chamber measure relative humidity, temperature, pressure, and flow.

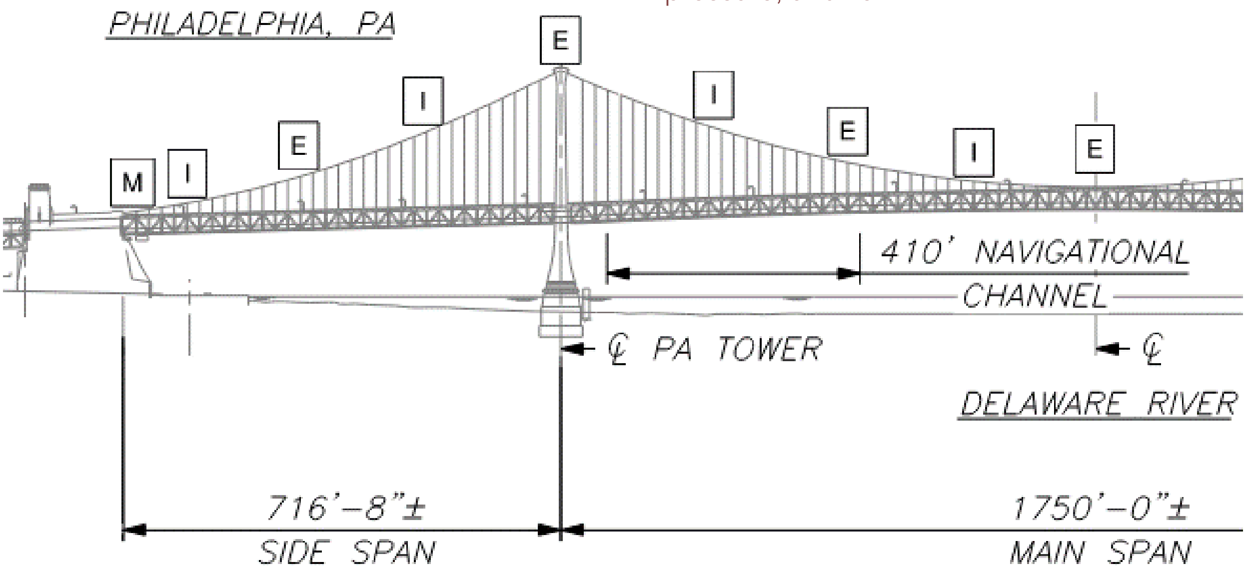


Figure 3: Main cable dehumidification system layout (Philadelphia Side and Main Span), note the system is symmetrical about the centerline. injection points (I), exhaust points (E), and monitoring points (M) are represented on the diagram

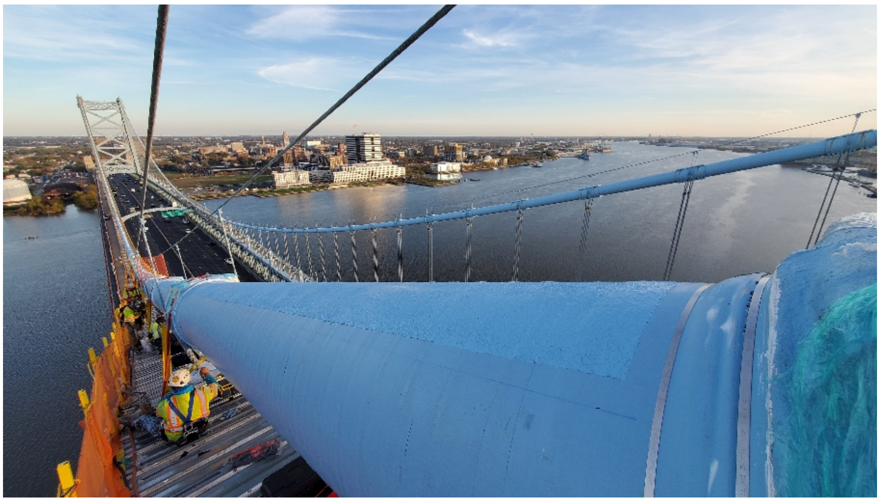


Figure 4: ↑ Elastomeric wrapping material during installation

→ Installed wrapping with wedge seal detail at each cable band and anti-skid walking surface

Cableguard™, a highly durable elastomeric wrapping material was installed and heat bonded onto the main cable. Special end seal detail with wedge seal at the circumferential cable band interfaces was installed and longitudinal cable band joints were sealed with a fast cure elastic sealant, refer to Figure 4.

The end seal utilizing the wedge seal and finishing strip are highly durable, weather resistant and provide a superior airtight seal, refer to Figure 5.

A temporary main cable access platform was designed and installed to provide access to the work beneath each of the main cables' full length. This platform provides an OSHA (US Occupational Safety and Health Administration) compliant work area for the various trades work on the main cable and various other fixtures, refer to Figure 6.

The Benjamin Franklin Bridge contains 4,012, 1 7/8" (47.6 mm) diameter bolts distributed over 302 cable bands which support the suspender ropes from which the deck is suspended.

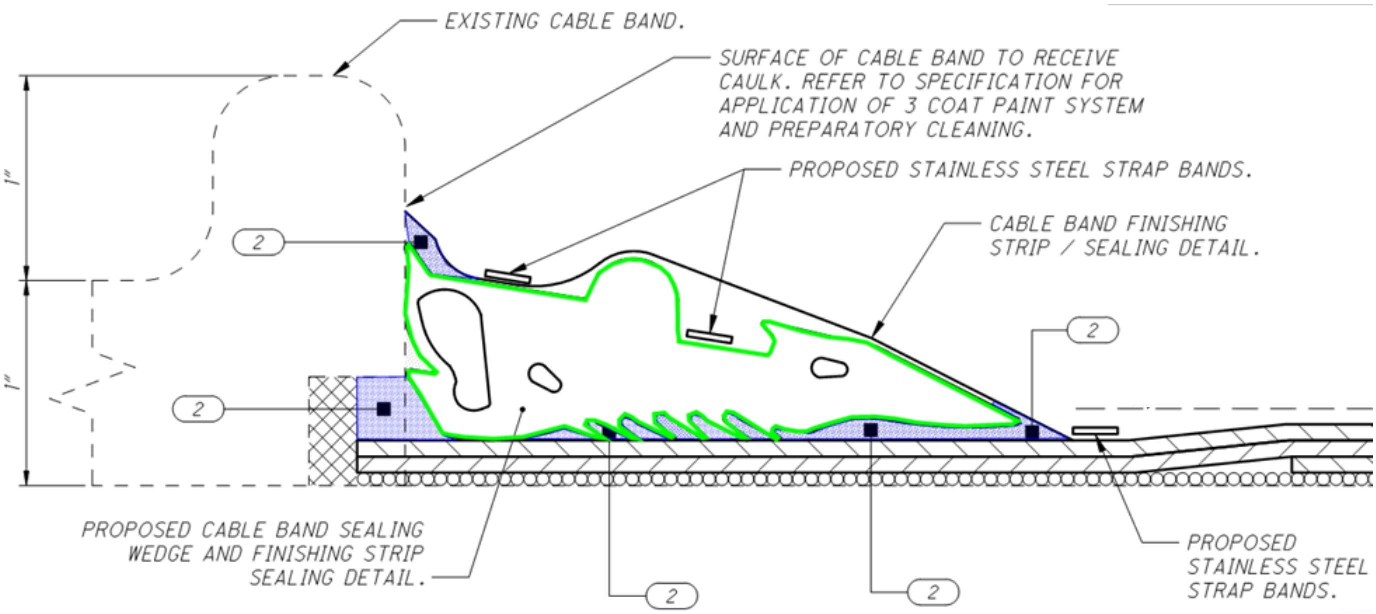


Figure 5: End seal wrapping detail with wedge seal (green) and sealant (blue) highlighted

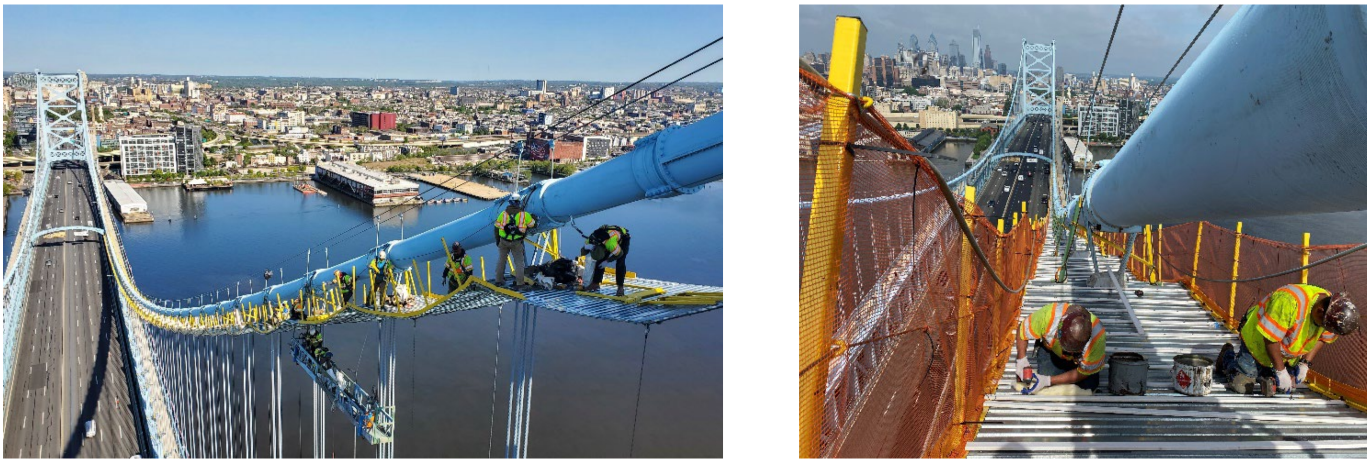


Figure 6: Main cable access platform installation

The primary function of the cable band bolts is to generate sufficient clamping force and friction to prevent slippage of the bands down the main cable under loads applied by the suspender ropes.

When originally installed the bolts were tensioned to a force ranging from 60 to 75 kips (267 to 334 kN) depending on their location along the main cable.

However, over the service life of the bridge, the cable band bolts will relax and lose tension due to the continual self-compaction of the cables over time. Given the visible corrosion on a number of the bolts and access provided by the MCAP, it was decided to perform a full replacement of all the bolts on the North and South Cable.

Figure 7 shows the replacement of cable bands and indirect measurement of the pre-tensioning of each bolt using a rigid framed extensometer.

Part 2: Walkway Preservation

The Benjamin Franklin Bridge Pedestrian Walkways on the north and south sides are vital panoramic routes for walkers, runners and cyclists between Philadelphia and Camden. The improvements made as part of the rehabilitation project with work limits extended from abutment to abutment, have allowed for a safer access by the public.

The major items of construction work included temporary shielding below the walkway directly above the PATCO Speedline track envelope, repair of structural steel, replacement of expansion joints and painting of steel. In addition, portions of the ornamental railing were replaced with new casted components, the concrete walkway deck panels were repaired and sealed, and the lighting fixtures were removed and replaced, refer to Figures 8 and 9.

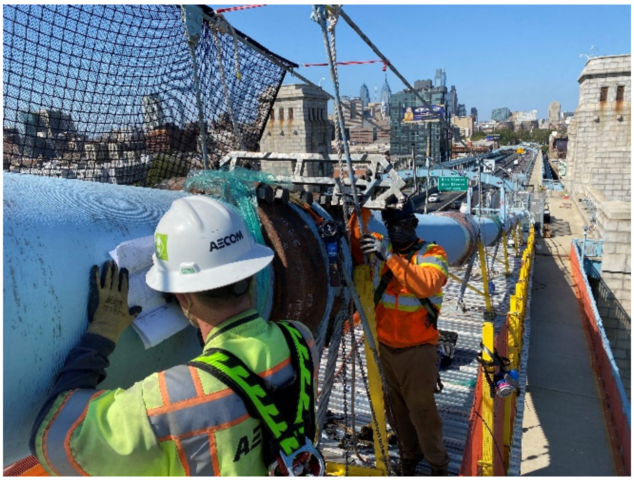


Figure 7: Cable band bolt replacement

Figure 8: Expansion joint replacement

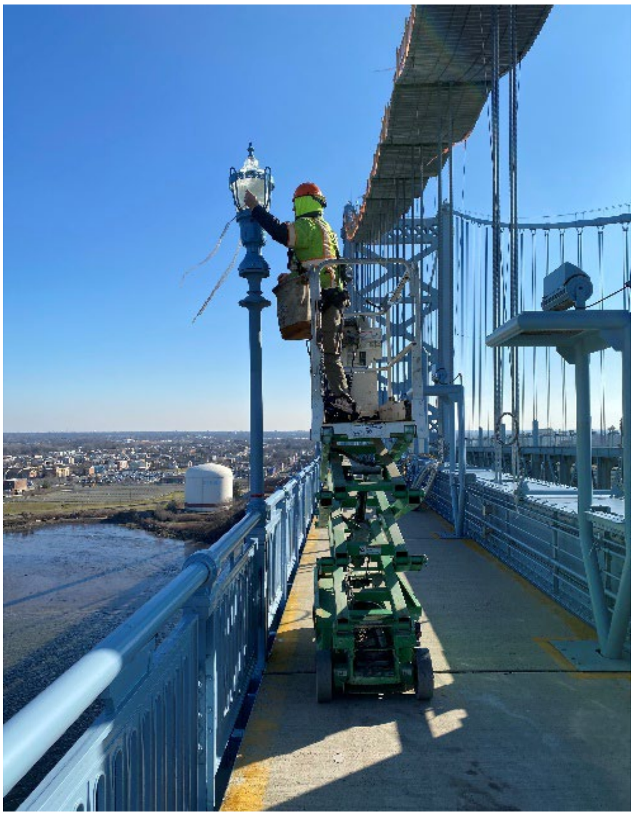


Figure 9: Walkway lighting replacement with new LED fixtures

Part 3: Maintenance Painting and Steel Repairs

In 2018, a routine inspection revealed significant deterioration to lateral bracing members which resulted in emergency structural repairs. In conjunction with these repairs, the bridges' stiffening truss end links and wind tongues which hold/restrain the truss structure at the towers and anchorages were being monitored with structural health monitoring (SHM) sensors as they were approaching the end of their service life.

The decision was made to combine this work with main cable dehumidification and other walkway and decorative lighting projects that were also under design. This work included large scale maintenance painting, removal of abandoned utilities, major steel repairs, miscellaneous steel repairs and rehabilitation of the anchorages. Access for this work included a full width full length under deck platform from anchorage to anchorage which integrated into containment during painting operations.

Maintenance painting consisted of limited blast cleaning, lead abatement and overcoating areas of the bridge in both the suspended spans and anchorages.

Along the suspended spans, the stiffening truss was power tool cleaned and overcoated, lower tower struts were partially overcoated, and the lower tower cells were blasted cleaned and painted. Inside the anchorages, the steel supporting the roadway and PATCO tracks were blast cleaned and painted.

Critical components classified as major steel repairs located on the suspended spans included replacement of the stiffening truss end links assemblies, lateral bracing repairs, retrofit and replacement of the anchorage wind lock assemblies and replacement of the tower wind tongue assemblies. The Contract Documents provided the contractor guidance pertaining to PATCO track outage allowances during specific temporary conditions and load transfers while performing these major repairs. Prior to moving forward with the repairs during the mobilization and planning phases of the project, Skanska re-engineered many of the suggested temporary support configurations that eliminated the need for track outages which greatly reduced the disruption to the traveling public. Figures 10 - 12 show various Part 3 work elements.

Figure 10: Lower tower strut overcoat painting

Figure 11: Removal of stiffening truss top chord

Figure 12: Placement of new stiffening truss end link

Part 4: Decorative Lighting

Decorative lighting was first installed on the bridge in 1987 with spotlights highlighting each of the 256 vertical suspender ropes outlining the geometry of the structure, visible from up to ten miles away.

In 2000, the lighting system received an upgrade adding colored LED lights to the outboard side of the deck. The lighting system was connected to a network that could be controlled remotely from a computer.

This contract work consisted of updating and replacement of the entire decorative lighting system on the bridge and other ancillary facilities. A new color kinetics system was installed that operates on a fiber backbone which runs across the bridge. It upgraded the conduit and wiring, the catenary lighting, tower lights, suspender uplighting, anchorage lighting including the historic lanterns and lights at the Lightning Bolt Monument.

Following work being completed on the north then south sides of the bridge, the system was adjusted and commissioned into service, refer to Figures 13 and 14.

The DRPA works and liaises with the local communities within Philadelphia and Camden and various organizations to honor and help bring awareness to various holidays, special events, sports teams, and philanthropic causes.

Part 5: North Walkway Widening

The North Walkway on the Camden approach span of the bridge contained a reduced pathway which introduced challenges when accommodating both pedestrian and cyclists' traffic.

Therefore, as part of this project, the aforementioned portion of the walkway was restored to its full width, enhancing the conditions for the public. Utilizing the temporary platform located directly above the PATCO track envelope, the existing walkway slabs and fencing were demolished. Abandoned utilities were removed and miscellaneous structural steel repairs were performed along with the installation of additional steel framing.

Precast concrete deck panels were primarily used, and erected during overnight track outages.

Figure 13: North Cable decorative lighting commissioned in Spring 2023

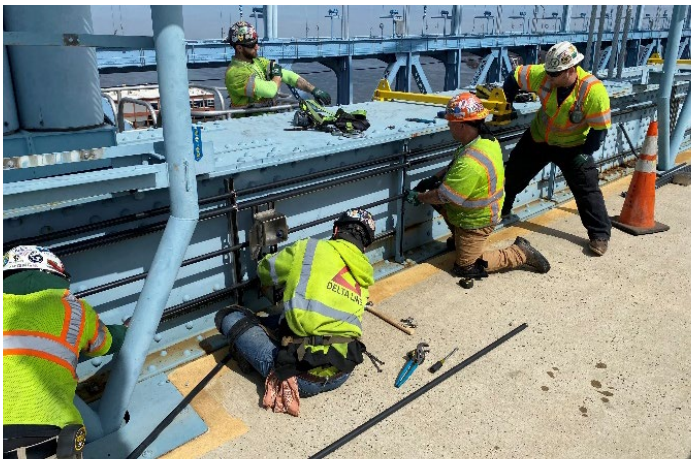


Figure 14: Electricians installed new conduit and wiring for upgraded decorative lighting system

Cast-in-place concrete deck panels were placed where custom layouts were required due to the existing features of the bridge, refer to Figures 15 and 16.

WHAT'S NEXT

The DRPA is committed to maintaining their vital infrastructure assets which connect communities along the Delaware River.

The installation of the main cable dehumidification and replacement of major steel components of the Benjamin Franklin Bridge will extend its service life indefinitely.

In the future, the DRPA will continue with proactive maintenance procuring future contracting to paint the approach span steel, repointing the anchorage masonry, and deck resurfacing and expansion joint replacement.

Figure 15: Temporary platform above the PATCO track envelope, existing walkway components demolished

Figure 16: Placement of new precast walkway slabs during overnight track outages

FORMWORK TRAVELLERS FOR BRIDGE CONSTRUCTION IN PANAMA AND COLOMBIA

Juan Novoa

Rúbrica Engineering

Figure 1: Cantilever overhead formwork system for the Guaitara Bridge, Colombia

INTRODUCTION

The Spanish Company Rúbrica Engineering has more than 25 years of experience in the design and manufacturing of special equipment for heavy civil work including bridges.

Rúbrica´s background includes a large selection of solutions for different construction methodologies such as underslung formwork travellers for progressive cantilever, heavy lifting equipment such as segment lifters for segmental bridges, precast moulds, wing travellers and so on. All of them are typically customized and adapted to the client and design requirements.

FORMWORK TRAVELLERS

A formwork traveller is a specialized piece of construction equipment used in bridge construction. It is a movable framework or structure that supports the formwork (temporary mould) for casting concrete deck segments of a bridge, with constant or a variable cross section.

The formwork traveller is designed to move horizontally along the bridge span, allowing for the incremental construction of segments. The inner formwork is a specialized structure incorporated within the traveller system, designed to achieve the particular geometry of the box girder and support the concrete during the casting process.

Sometimes, the inner formwork requires a more thorough engineering study to adapt to different configurations and peculiarities of the geometry (Delta frames, post-tensioning blocks, stay cable anchor blocks and so on), as well as providing a number of folding and tilting mechanisms to be able to pass through very confined spaces.

Formwork travellers are equipped with hydraulic systems or other mechanisms that enable them to move smoothly and precisely, ensuring accurate alignment and placement of concrete segments. Steel or timber platforms, ladders, and auxiliary lifting equipment are included to improve the accessing and help with other specific tasks.

In this article, we will present three Case Studies where we had the opportunity to participate in Panama and Colombia. We aim to show how variable and adaptable the formwork travellers are so that they can be used in different bridge projects.

CASE STUDY 1: FORMWORK TRAVELLERS FOR THE ATLANTIC BRIDGE

THE BRIDGE

The Atlantic Bridge is the third bridge over the Panama Canal. It is part of a local connection road between the Bolivar Highway in the east and the undeveloped western area. It allows vehicles to cross the Panama Canal on the Atlantic side regardless of whether the locks are in operation or not.

This double-pylon, concrete cable-stayed bridge carries two lanes of traffic in either direction. With a main span of 530 m and two back spans of 230 m, pylons 212.5 m high, and a vertical navigational clearance of 75 m, it allows the largest container ships, the Post-Panamax, to pass through the Canal.

The east approach is 1,074 m and the west approach is 756 m long.

Construction of the bridge and access viaducts commenced in January 2013. The bridge was opened to traffic in 2019.

The owner is the Panama Canal Authority and it was constructed by Vinci Construction Grands Projets and designed by HPDI and Louis Berger Group.

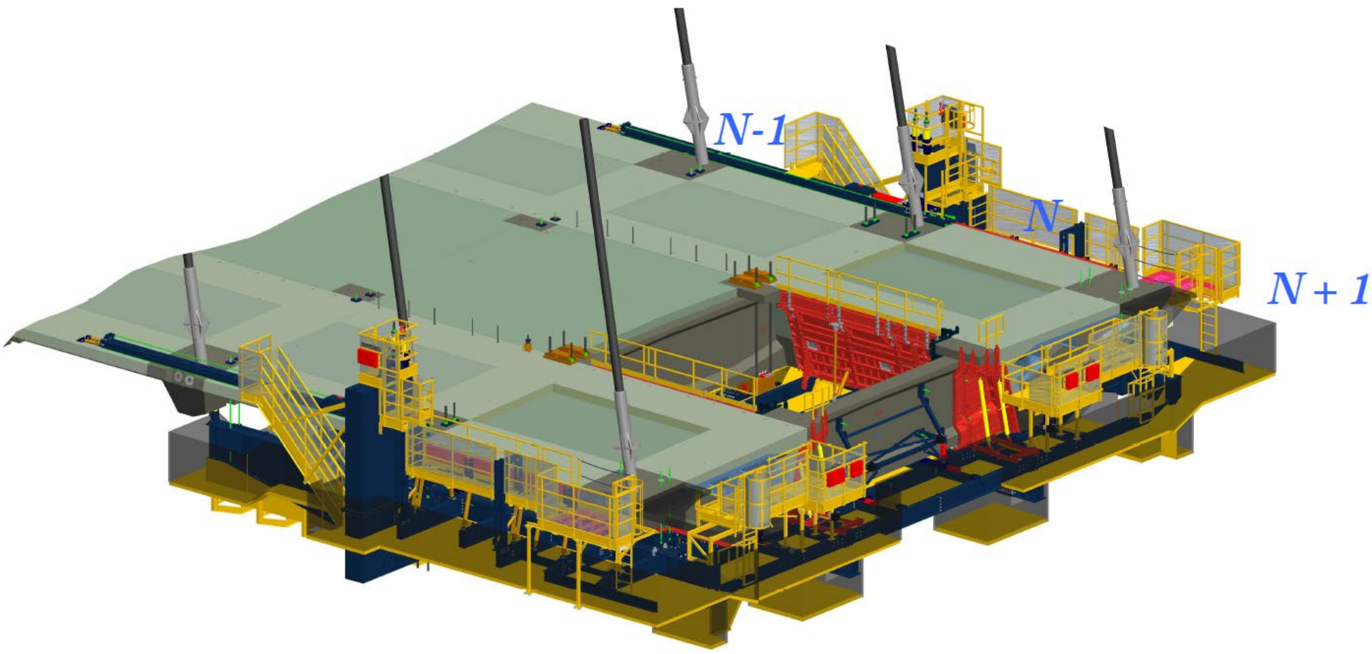


Figure 2: Formwork traveller for cast in situ deck – Atlantic Bridge, Panama

CONSTRUCTION

The Panama Canal is a very busy shipping route, especially for container ships, so one of the requirements for the construction was to keep the canal traffic uninterrupted. As a result, the bridge was constructed with a cast in situ methodology.

Due to the adversity of the Panamanian tropical climate on the Atlantic side, the Panama Canal Authority opted to build the bridge entirely from concrete.

Rúbrica Bridges designed and supplied four underslung formwork travellers (two for each bridge pylon), which were used to build the central span and both back spans. Due to high load restrictions during cantilever construction, each traveller weight was 220 tonnes. For the lifting of the travellers and also the first casting, Rúbrica Bridges designed and provided four lifting structures supported off the pylons, which allowed the construction of the first deck segment and the final assembly of the traveller into the final configuration of the formwork traveller.

The formwork traveller consisted of a steel casting cell supported by a grillage of transverse and four longitudinal steel girders that were located under the deck. The equipment was designed for a constant cross section of 8 m length but with the capacity to be adapted by removing some panels to construct 7 m long deck segments, as required by the client. During the casting of each typical deck segment, each longitudinal girder was anchored with 4 Macalloy bars 50 mm in diameter to the previous concrete deck segment creating a cantilever supported structure.

During the launching, the whole system moved on a pair of rails, using a skidding shoe (pushed by hydraulic jacks) as the launching surface, where the whole structure was supported off this pad with an articulated C frame connection. The C-frame is a superstructure used during the launching to hang the whole formwork and transmit the self weight to the rail installed on the deck. Since the centre of gravity of the traveller was located ahead of the point of rotation and ahead of the anchor point of the structure, the rotation of the traveller was resisted by two back supports bearing against the underside of the completed deck composed of a rolling system built with 24 units of high capacity Juwathan polyurethane castors, to avoid damage to the concrete and ensure a smooth and safe launching.

Due to the reduced space available at the pylon, the first deck segment at each pylon was cast using only the front part of the travellers. The back

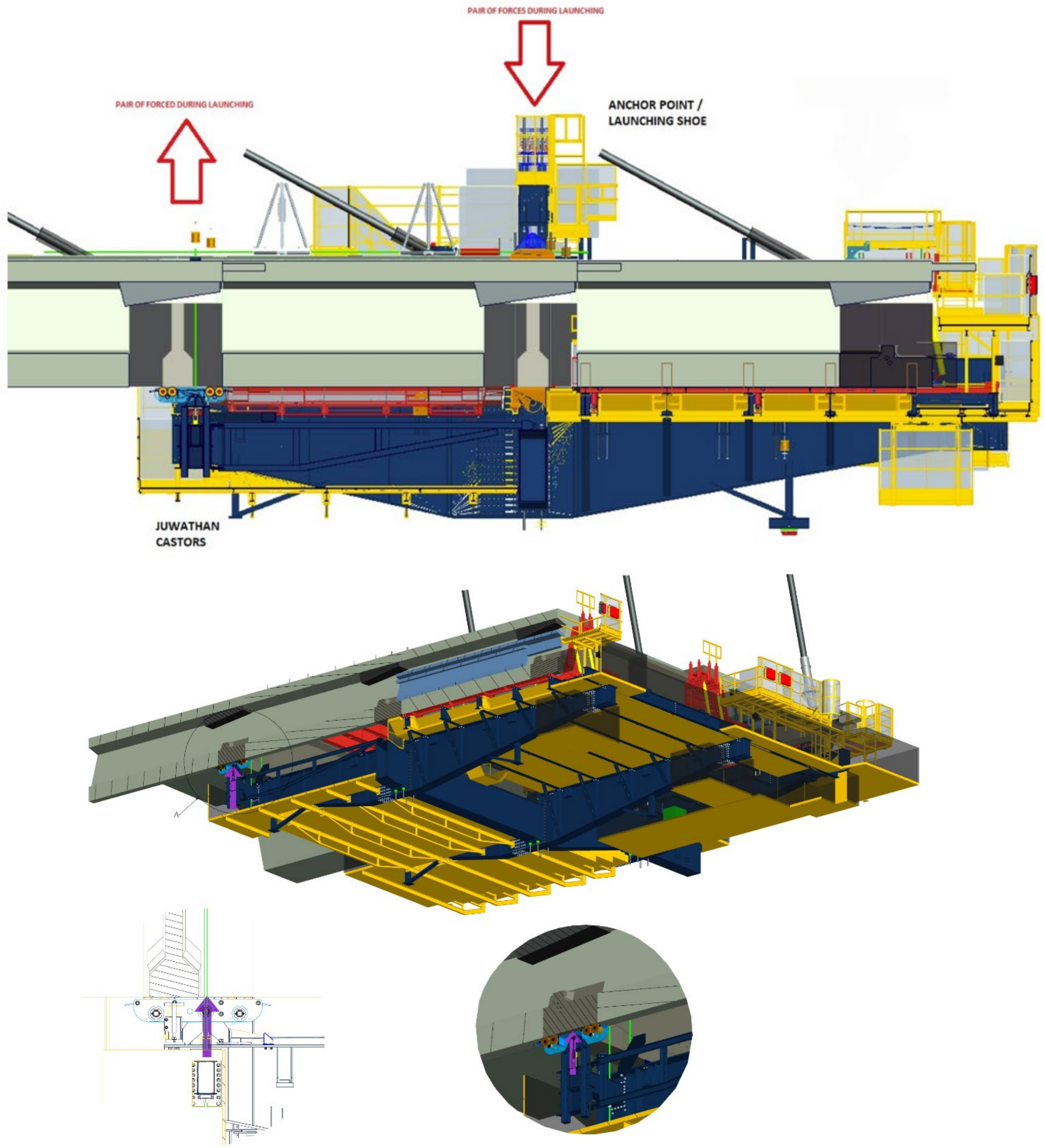
Figures 3: The Bridge during construction

of this traveller section was lifted using a strand jack system supported by the lifting structure which was fixed to the rear part of the deck.

For the front section, the traveller was supported off the lifting structure with Macalloy suspension bars. With this solution, during the casting of the first segment, the traveller acted as a double-supported structure, ie. using front and rear supports.

After this first segment construction, the front part of the traveller was lowered and the rear parts of the traveller were assembled and connected on the ground to complete the full configuration of the formwork traveller.

After this final assembly, the traveller was lifted again with the same strand jack system into the advanced position and the traveller was then ready to start the typical operation.



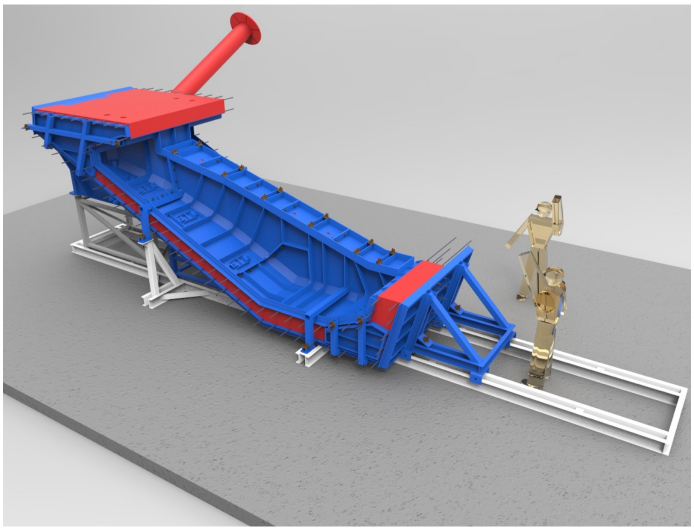
Figures 4 and 5: Back support and launching system


Figures 6 and 7: Casting of the first deck segments

A particular requirement of the construction method, and therefore of this traveller was the need to construct the anchor blocks for the stay cables and a central diaphragm using prefabricated elements. The forms for the casting of the stay cable anchorages were also designed and supplied by Rúbrica Bridges.

Furthermore, the method of placing these pre-cast items, to avoid overloading the deck cantilever with the use of cranes, was designed and integrated into the traveller, providing added value to the product, see Figures 9 and 10.

↑ Figure 8: Formwork travellers in full configuration and during typical operation



→ Figure 9: Pre-cast forms for cable anchor blocs

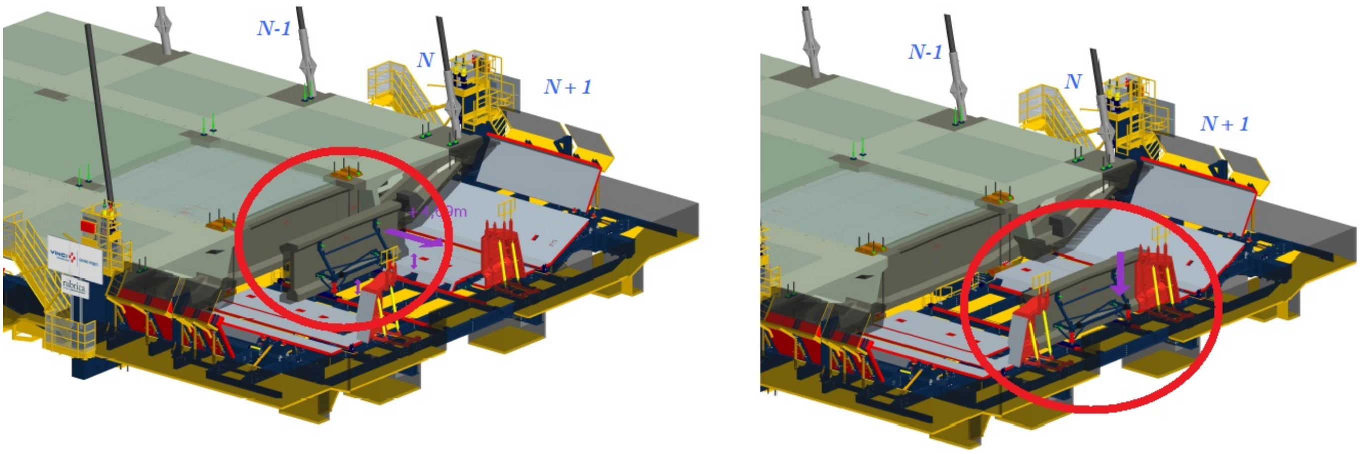


Figure 10: Launching precast items using railing system designed by Rúbrica and integrated in the formwork traveller

CASE STUDY 2: BRIDGE CANTILEVER FORMWORK FOR THE RUMICHACA – PASTO HIGHWAY PROJECT IN COLOMBIA

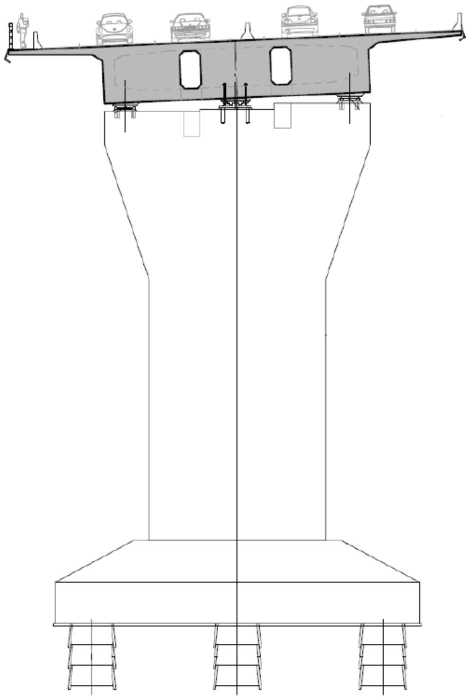
The Rumichaca – Pasto Highway Project in Colombia is located in a terrain with mountainous topography which had a big influence on the construction method chosen.

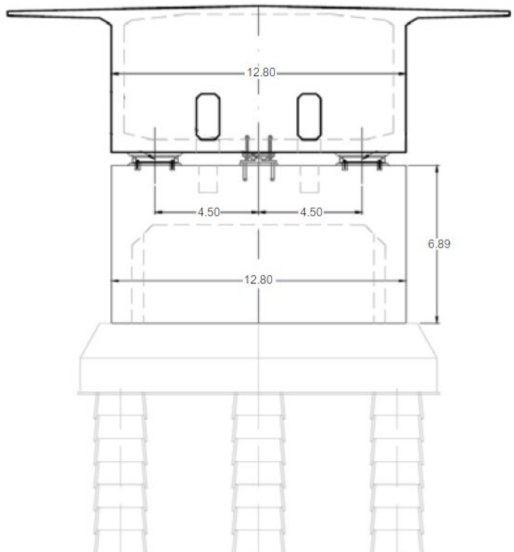
In this case, Rúbrica Bridges designed and supplied overhead formwork travellers for segmental balanced cantilever methodology, capable of being adapted to the different crossfalls (with a maximum of 8%) and horizontally curved alignment. The project was constructed between 2015 and 2021 by Sacyr Consorcion SH.

The Boquerón Bridge is a 260 m long and 105 m high viaduct and includes two carriageways, one in each direction. The superstructure has a constant longitudinal grade of 4.17% and a variable crossfall, a total width of 21.80 m including a pedestrian sidewalk 1.0 m wide together with its respective railing.

The deck is a single-cell concrete box section with variable depth between 2.75 m and 6.37 m, and vertical webs. The bridge has five supports outside the channel of the river; two abutments and three intermediate piers.

The formwork traveller was designed to have high adaptability for longitudinal grade, crossfall, variable depth and capacity to construct the curved horizontal alignment.





↗ → Figures 11 and 12: Boquerón Bridge. Cross section at Piers 2 and 4 Source: Consorcio SH, 2017

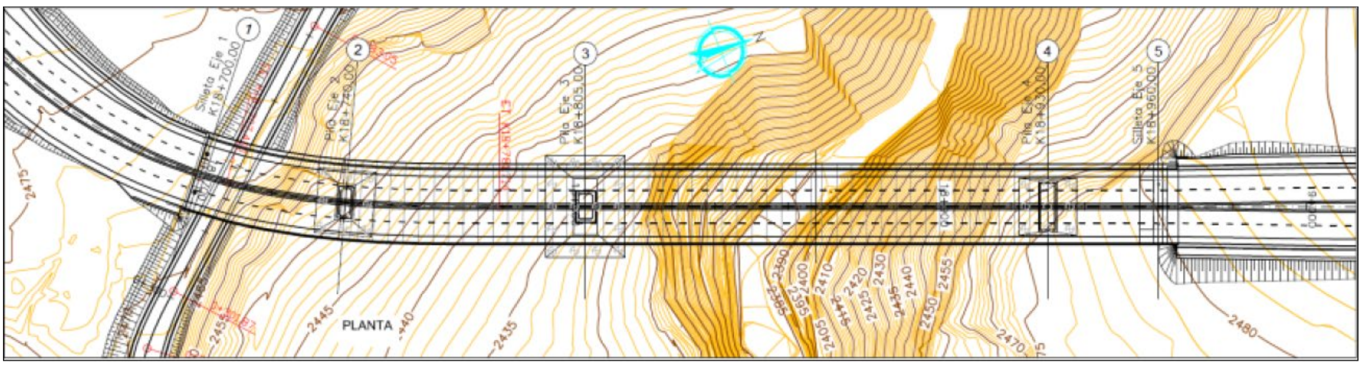


Figure 13: Boquerón Bridge Plan. Source: Consorcio SH, 2017


Figures 14 and 15: Formwork travellers on the Boquerón Bridge

The Porvenir Bridge is a 225 m long and 60 m high viaduct located in the municipality of Iles and crosses the canyon with a minor stream. The superstructure has a constant longitudinal grade of 1.0%. The deck's horizontal alignment is completely straight, see Figure 17.

The deck is 21.80 m wide, with two vehicular carriageways (one in each direction) and a pedestrian sidewalk 1.00 m wide together with its respective railing.

Figure 16: Porvenir Bridge

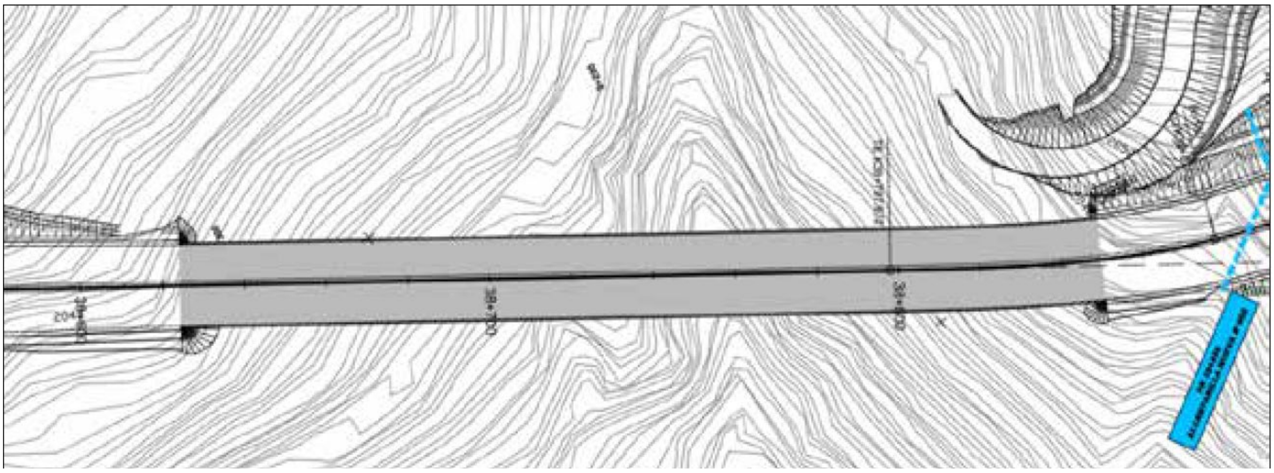


Figure 17: Plan of the Porvenir Bridge. Source: Consorcio SH, 2017

The Rio Sapuyes Bridge is 200 m long with a horizontal curve, see Figure 18. The superstructure has a constant longitudinal grade of 7.50%. Due to the terrain's topography, it is a concrete box girder bridge constructed using the balanced cantilever method.

The main span is 100 m, side spans 50 m. It is 21.80 m wide, with two carriageways (one in each direction) and a pedestrian sidewalk 1.00 m wide together with its respective railing.

The deck has a variable-depth box section, with a maximum depth of 5.00 m at piers P-1 and P-2, and a minimum depth of 2.50 m in the centre of the span and at the abutment support. As a result, with two 2 pairs of cantilevers from piers 1 and 2, the client completed the three spans that made up the viaduct.

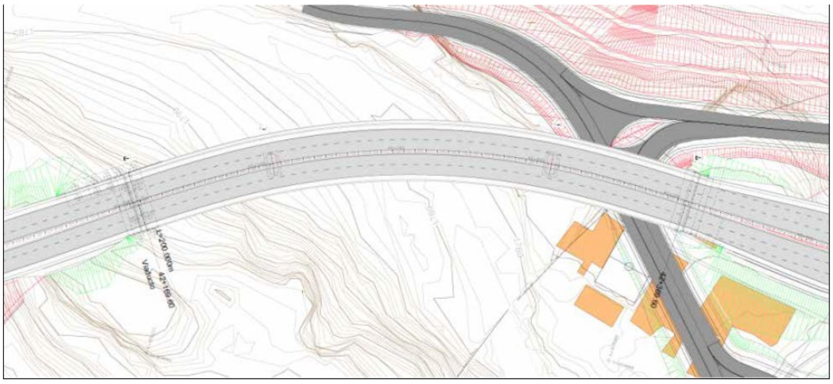


Figure 18: Plan of the Sapuyes Bridge. Source: Consorcio SH, 2017

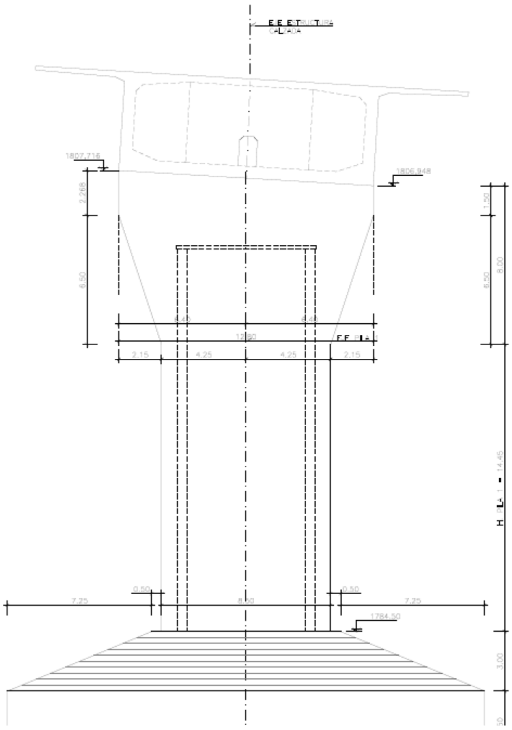


Figure 19: Cross section of Pier 1
Source: Consorcio SH, 2017

The box section has webs with a constant thickness of 0.60 m. The base of the section's core has a constant width of 12.80 m. The thickness of the bottom flange was a maximum of 0.6 m at the piers varying linearly to 0.25 m. The webs' intersection with the upper flange incorporated haunches to accommodate the deck segment longitudinal prestressing tendon anchorages.

For these three bridge projects (Boquerón, Porvenir and Sapuyes Bridges), Rúbrica Bridges supplied its RC800 series Overhead formwork traveller, see Figure 20.

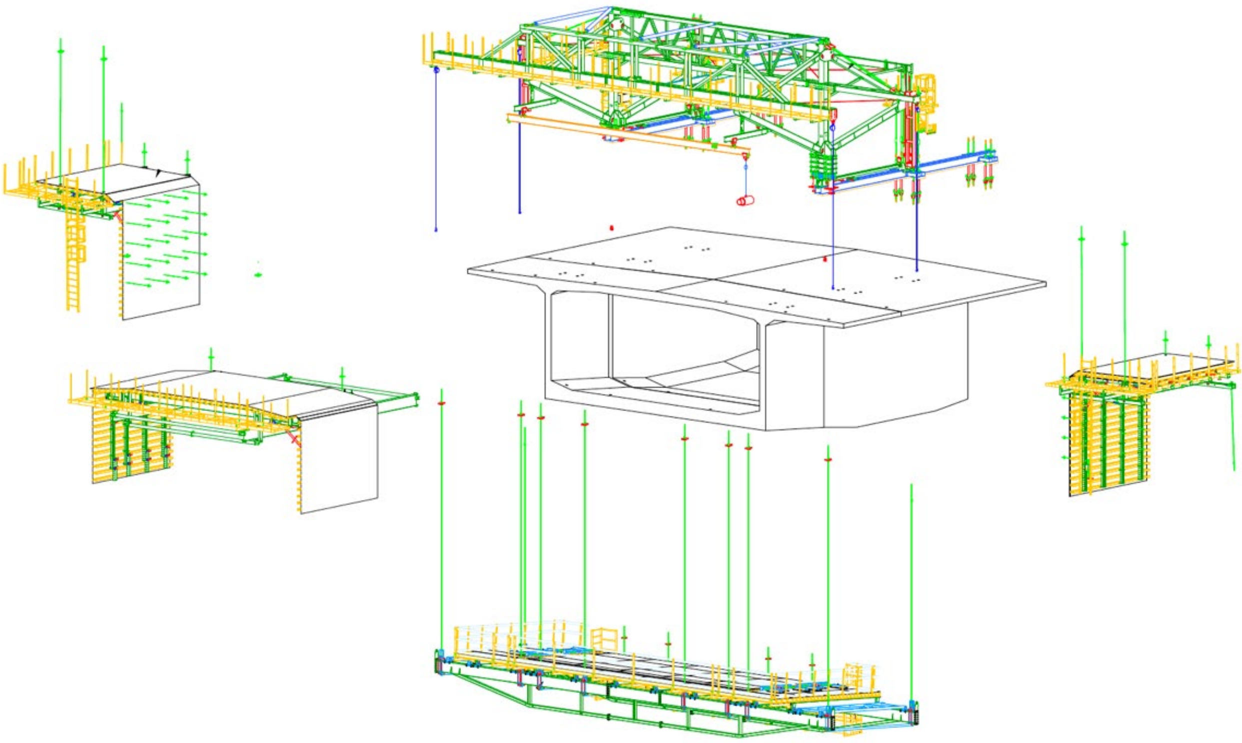


Figure 20: RC800 Overhead form traveller

It is adaptable to different configurations which was necessary for the Colombian highway geometry. Its capacity is 800 tons per metre and its maximum casting length of 5 m.

Taking into account the terrain's topography and the span length to be bridged, the balanced cantilever construction method was used, with single-cell concrete box segments of depths varying between 2.75 and 6.37 m, and vertical webs.

It consists of a single platform with a total width of 21.80 m, which can accommodate four vehicular lanes and a pedestrian sidewalk 1.00 m wide together with its respective railing. The bridge consists of four supports, two abutments and two intermediate piers 28.98 m in height.

Within the construction of the same Rumichaca-Pasto line, Rúbrica Bridges participated in the design and supply of formwork travellers for narrower superstructures with our RC500 (500 tons per metre and a maximum casting length of 5 m) series for Macal, Guaitara and Magdalena Bridges.

In this case, and following the line of the previous ones, the formwork traveller was designed for the rugged and changing topography of the Colombian mountains which comprised very tight horizontal curved areas, significant longitudinal grades and transverse crossfalls reaching sections of up to 8%, which required frequent changes in the configuration of the form traveller.

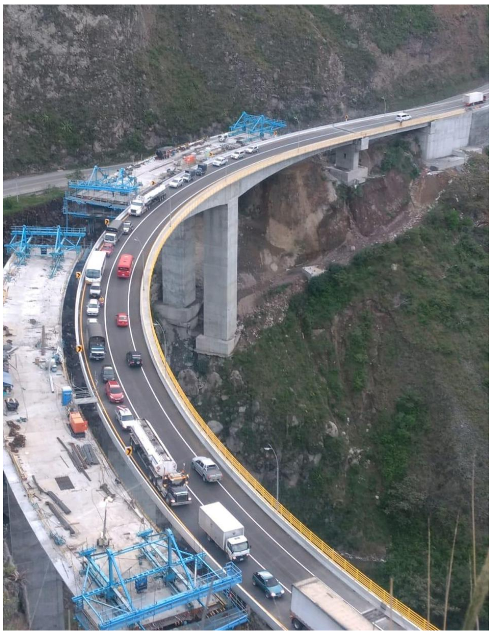


Figure 21: Magdalena Bridge

Figure 22: The Guaitara Bridge

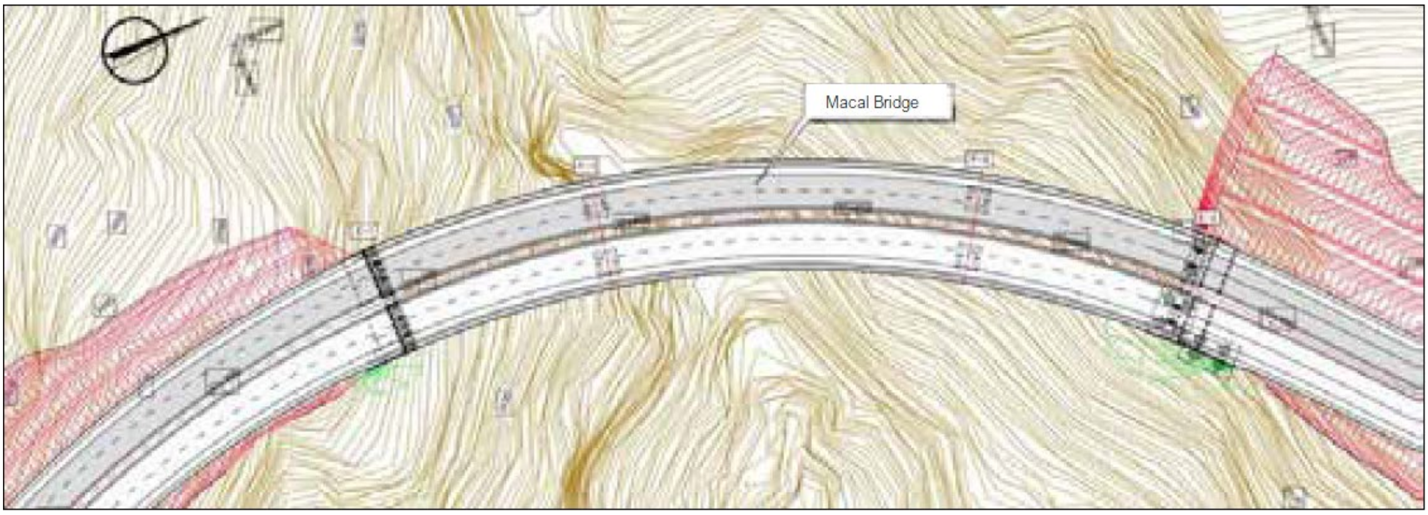


Figure 23: Plan of the Macal Bridge. Source: Consorcio SH, 2017

CASE STUDY 3: CANTILEVER FORMWORK CONSORCIO VÍA AL MAR1, COLOMBIA

The main objective of the project Autopistas para la Prosperidad is to construct a road connection between the city of Medellín and the main commercial exchange centres such as the Caribbean Coast, the Pacific Coast and the Magdalena River.

The Al Mar 1 Highway project is one of the Fourth Generation road projects, an initiative of the National Government of Colombia, and it is approximately 176 km long.

In 2018, for SACYR, Rúbrica Bridges designed and provided an adaptable RC500 Overhead formwork traveller for the construction of 4 different viaducts. These viaducts were designated as the K6+900, K6+240, K5+240 and K1+940 viaducts.

The formwork traveller could be adapted to the specific longitudinal grade and transverse crossfall of the bridge as well as to the variable depth section, as in the previous case study.

One of the particularities of this equipment was the overhead cross member configuration for the casting of the first two deck segments.

Due to the minimal space available at the top of the pier, the Cross Member solution was chosen, see Figures 25 - 27.

This configuration allows us, in extremely reduced pier hammerhead lengths, to be able to accommodate both formwork travellers at the same time and to cast both initial deck segments avoiding unbalanced situations.

To achieve this, the rear or tail support of each traveller overhead main structure is removed and replaced by a structure that connects both travellers, creating a single arrangement in which the removed rear support of each traveller is replaced by the front support of the other.

After the construction of these first two deck segments and once there is enough space for the complete assembly of each unit, the traveller is launched and assembled into its full configuration to start the typical casting sequence, see Figures 28 and 29.

Figure 24: Bridge 12 K6+900

↑← *Figures 25 - 27: Bridge K1+940 in cross member configuration during assembly*

↓ *Figures 28 and 29: Bridge K1+940 after formwork traveller split and typical operation configuration*

MAURER

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MSM® Swivel Joist Expansion Joint

OSMAN GAZI BRIDGE, IZMIT, TURKEY | WORLD NO. 4 SUSPENSION BRIDGE WITH HIGH SEISMIC LOAD

Scope of application:
The installation of the MAURER Swivel Joist Expansion Joint shall allow access to and protect the bridge deck from horizontal over load during a seismic event.

Features:
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- Serviceability of the structure after the earthquake
- Protection of the bridge deck from horizontal overload caused by extreme closing movements during the earthquake
- High life time expectation through use of high performance components
- Longitudinal seismic displacement of ca. 4 m
- Service velocity up to 20 mm/sec (10 times higher than for a regular bridge)
- Watertight across the bridge width
- Maintenance free

References:
- Bahia de Cadiz, Spain
- Hochmoselübergang, Germany
- Osman Gazi Bridge, Izmit, Turkey
- Mainbrücke Randersacker, Germany
- Millau Viaduct, France
- Rheinbrücke Schierstein, Germany
- Rion Antirion, Greece
- Russky Island Brigde, Vladivostok, Russia
- Tsing Ma, China

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forces in motion

Helgeland Bridge, Norway

Photo : Jules van den Doel

AAS-JAKOBSEN

Pipenbaher
Consulting Engineers

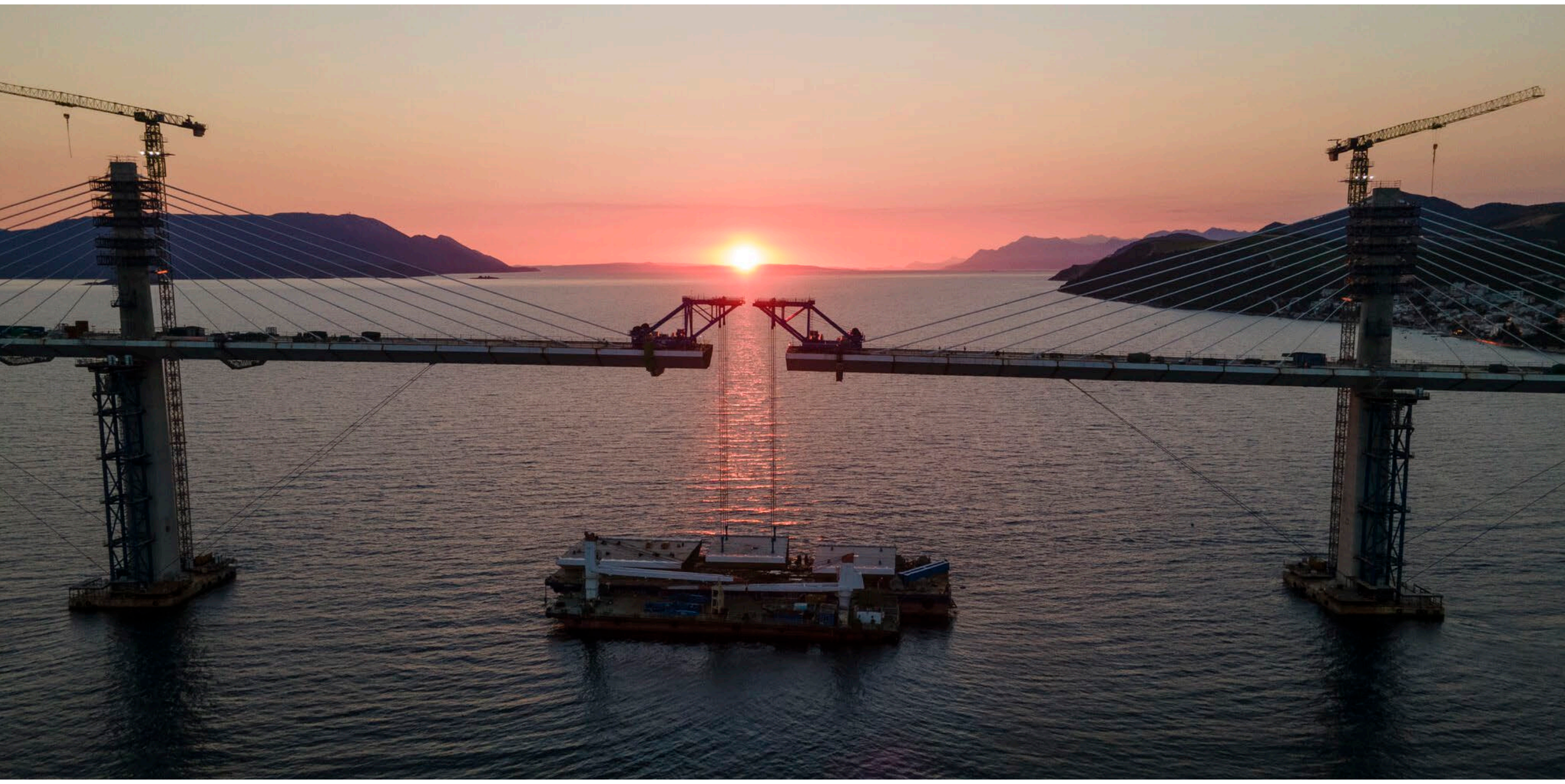
PIPENBAHER INŽENIRJI d.o.o., Slovenia
www.pipenbaher-consulting.com

PELJEŠAC BRIDGE
CROATIA

Pipenbaher Consulting Engineers / PIPENBAHER INŽENIRJI d. o. o.
Žolgarjeva ulica 4a, 2310 Slovenska Bistrica, Slovenia

We design bridges

ponting bridges



Pelješac Bridge, Croatia

Conceptual/Preliminary/Final design
Joint Venture Faculty of Civil Engineering, University of Zagreb; Ponting; Pipenbaher Consulting Engineers

Ada Bridge over Sava in Belgrade, Serbia

Winning competition design/Preliminary/ICE for final and detailed design

Ponting inženirski biro d.o.o., Strossmayerjeva 28, 2000 Maribor, Slovenia

EIA ENGINEERS IN ACTION

We connect Students, Industry Partners and Communities through collaborative bridge building.

Isolation caused by impassable rivers is a root source of poverty all over the world. Fueled by the passion of University Students, our Industry Partners and the wisdom of our Local Partners, we build footbridges with isolated communities to ensure they have year-round safe access to essential resources such as education, healthcare, and markets.

eiabridges.org | EIA Bridge Program | eiabridgeprogram | EIA Bridge Program

BE A BRIDGE.

Together we can transform lives.

Everyone can be a bridge to a better world.

A BRIDGE THAT BRINGS SOCIAL CHANGE. A BRIDGE OF HOPE. A BRIDGE OF LOVE.

Bridging the Gap Africa believes everyone deserves access to the basic necessities of life: Better healthcare · Quality education · Robust commerce. We build bridges *with* the communities we serve. This approach enables Kenyan communities to be involved with the building process and empowers them to expand beyond geographies and borders to include corporate and private donors from around the globe.

BtGA is a 501(c)3 in the US that also has Charitable status in Canada. For more information, please visit the Bridging the Gap Africa website at bridgingthegapafrica.org.

BRIDGING THE GAP AFRICA

bridgingthegapafrica.org

Get involved. Be a bridge.

Bridges to Prosperity

We envision a world where poverty caused by rural isolation no longer exists.

Corporate Partners make this vision possible.

Join Bridges to Prosperity in helping isolated communities gain safe access to healthcare, education, jobs, and markets through simple, sustainable, trailbridges. Together, we can build more than a bridge; we can build a pathway out of poverty.

+60%	+75%	+35.8%
Women Entering the Labor Force	Farmer Profitability	Labor Market Income

*Wyatt Brooks and Kevin Donovan - "Eliminating Uncertainty in Market Access: The Impact of New Bridges in Rural Nicaragua," 2017.

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