e-mosty

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Front Cover: Ponte Ferreirinha, Porto, Portugal. Photographer: Inês D'Orey Render: lens

Back Cover: Launching Gantry for Cairo Metro Line, Egypt. Credit: BERD

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Dear Readers

In this e-mosty edition, you can read about the new bridge that is going to be built over the Douro River in Porto, Portugal. The authors focus on its structural design and construction process. The article is accompanied by drawings and renderings of this bridge.

The Launching Gantry used for the construction of the Cairo Metro in Egypt is described in the second article. In the article, you can find information about the project, the operation cycle of the LG, its description, and details about the construction process.

In the third article, Wind Engineering in the Chenab Bridge is described. This project's key viewpoints with the main technical wind engineering challenges are addressed.

The East Lake Bridge Project in China presented several intricate design and construction challenges, all of which required precise planning, complex detailing, and rigorous analysis to overcome. In the last article of this edition, you can find information about the complex process behind the construction of the Bridge and the role of BIM in it.

*I would like to thank **Ken Wheeler, Richard Cooke, and Juan Carlos Gray** 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.*

We are happy to announce that we have started cooperation with structurae which is the largest database for Civil and Structural Engineers. We are in the process of uploading e-mosty and e-BrIM articles and special editions.

We are planning one or two special editions dedicated to American Bridges, which will be released in 2024. We welcome cooperation with you and will be happy to publish your articles.

From 25th October to 25th November 2023, I will travelling in the USA, especially the East and West Coasts (Florida, California, and New York). I will be happy to meet you and your teams and visit your bridge projects. In case you are interested in cooperating with our magazines, please contact me. Thank you.

The next e-mosty magazine will be released on 20th December and e-BrIM on 20th October 2023.

Magdaléna Sobotková

Chief Editor

INTERNATIONAL ONLINE PEER-REVIEWED MAGAZINE
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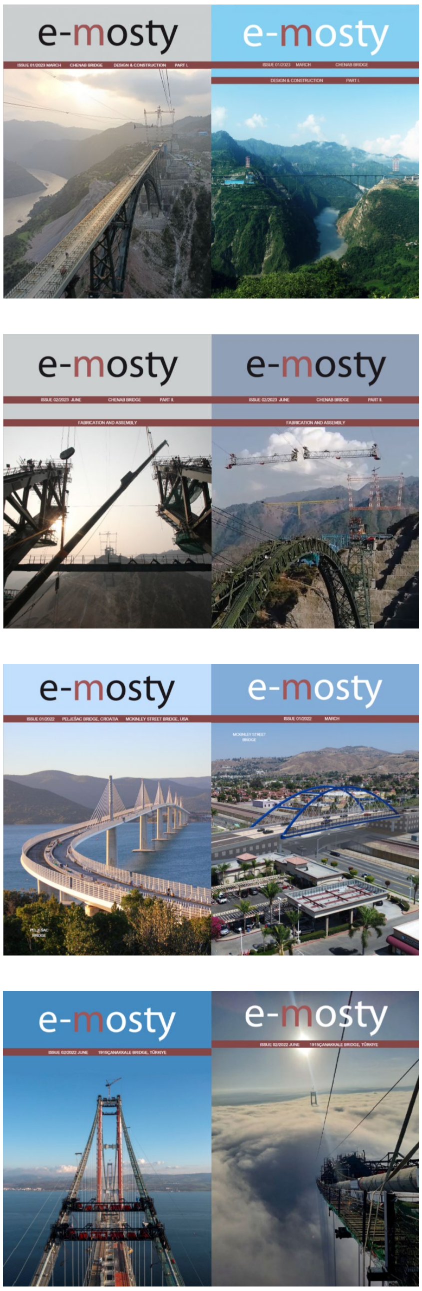

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PONTE FERREIRINHA - THE NEW BRIDGE OVER DOURO RIVER IN PORTO, PORTUGAL

Guillermo Capellán Miguel, Miguel Sacristán Montesinos, Emilio Merino Rasillo
Arenas & Asociados Engineering, Spain

Filipe Manuel Vasques, Edgar Cardoso Engineering, Portugal
José Carlos Nunes de Oliveira, NOARQ Architecture, Portugal

Figure 1: Elevation view in relation with the Arrábida Bridge

I. INTRODUCTION

The new bridge over the Douro River will form part of the new Metro line between Casa da Música and Santo Ovídio, joining the municipalities of Porto and Vila Nova from Gaia, see Figures 2 and 3.

The bridge will support twin metro tracks and pedestrian and cycle paths to encourage sustainable mobility.

The new bridge is a result of an international design competition comprising 27 teams of engineers and architects. The winning design was awarded to the team formed by Arenas & Asociados, Edgar Cardoso and NOARQ in late 2021, based on a preliminary design proposal prepared by these Spanish and Portuguese engineering and architecture firms.

The detailed design was subsequently carried out during the second half of 2022 and completed in May 2023.

The construction tender was launched on 10th May 2023 with bids received by 10th August. The construction contract is expected to be awarded in the second half of 2023 with an estimated duration of 36 months and an estimated construction cost of approximately 70 million EUR.

Figure 2: Location of the Bridge on the map. Source: Google Maps

__CLIENT:__ Metro do Porto, Portugal

Project Director: Vitor Silva, Metro do Porto

__DESIGN:__ Arenas & Asociados, Edgar Cardoso and NOARQ

__MAIN STAFF DESIGN TEAM:__

Guillermo Capellán Miguel, Miguel Sacristán Montesinos, Emilio Merino Rasillo
Arenas & Asociados Engineering

Filipe Manuel Vasques, João Martins, André Costa
Edgar Cardoso Engineering

José Carlos Nunes de Oliveira
NOARQ Architecture

Figure 3: Location of the Bridge showing other Douro River Bridges. Source: Google Maps

The bridge name was chosen by public vote from six alternatives, with the name Ponte Ferreirinha chosen in tribute to Antonia Ferreira, who was a prominent woman in the Porto region associated with the development of the Porto wines during the XIXth century.

The crossing is located 600 m upstream of the Arrábida Bridge, see Figure 5, designed by Portuguese engineer Edgar Cardoso in 1963.

At the time of its construction, it was the world's largest concrete arch bridge with its 270 m span, and in recent years was declared a national monument for preservation.

The new bridge will be 2.4 km downstream from the Luis I. Bridge, see Figure 4, designed by Théophile Seyrig and built in 1886.

It also follows the example of the formidable São João Bridge designed by Edgar Cardoso in Porto in 1991, and is very close to María Pía Bridge designed by Théophile Seyrig and built by Gustave Eiffel in 1877, see Figure 6.

These existing bridges are thus relevant in the design of the new bridge over the Douro River, as Porto is also called the "City of Bridges".

According to our approach, the new bridge should be a natural progression of the existing bridges developing the city bridge tradition one step further.

"Synthesis" thus became the "motto" of the design, minimising the number of elements in the design as the best strategy to reduce its visual impact on the landscape and to better integrate with the adjacent Arrábida Bridge.

In the next paragraphs, the bridge structural design is described together with the decisions and constraints that led to the choice of its typology, dimensions and main technical features.

2. STRUCTURAL DESIGN

The first fundamental design decision was to choose the locations of the main supports of the new bridge.

According to the project's terms of reference, these had to be outside the river bed, due to environmental reasons.

Figure 4: Luis I. Bridge

Figure 5: Arrábida Bridge

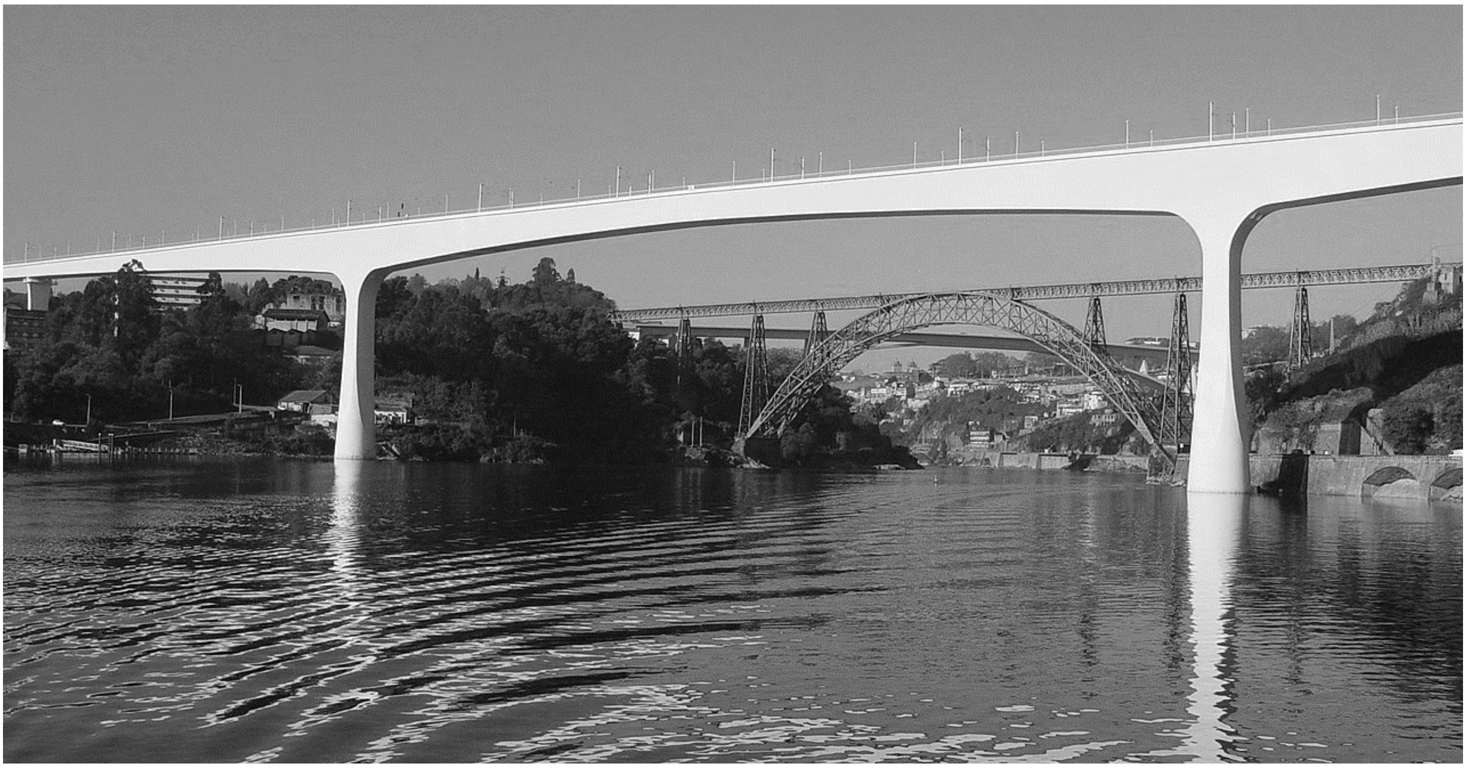


Figure 6: São João Bridge with María Pia and Infante Henrique in the background

If they were arranged exactly at the edge of the river, attached to the docks, the main span would be around 330 m.

However, this position would entail major disadvantages, namely:

- There would be damage to the riverbed during construction;
- There would be a risk of impact from boats, as the vertical clearance at the riverbank would be very low;
- It would create a greater visual obstacle for the buildings on the left and right banks and for viewing the Arrábida Bridge from the waterfront since the Arrábida Bridge has its supports behind the river bank roads;
- It would involve complex and costly foundation solutions, since on the right bank the rock is located at a depth of 20 m, and rising from this point;

Thus, as a starting point, it was assumed that the main supports on both sides of the river should be located behind the river bank roads and, in the case of the left bank, behind the existing building of Armazém da Arrozeira.

The result is a main span of approximately 400 m, with a chosen position for the main supports that optimizes the integration with existing buildings, reduces the visual impact on the Arrábida Bridge, has zero effect on the river and optimizes the foundation conditions.

However, this leads to a very large and challenging structure.

The bridge deck level at approximately 75 m above the water was also a necessary requirement, due to the final connection points, and in order to define a bridge just slightly higher than the Arrábida Bridge to reduce its visual impact on the existing bridge.

For a central span of around 400 m, there is a limited set of efficient structural solutions available, namely arch bridges, frame bridges, cable-stayed bridges, and suspension bridges.

Figure 7: View of the new bridge from the Crystal Palace gardens

Figure 8: The new bridge will be 75 m high

Figure 9: Proximity of existing buildings influenced the design

Cable-stayed or suspension bridge solutions were discarded as they require structural elements above the deck, which do not integrate well with the city's horizontal skyline and the surrounding landscape and also in relation with the other existing bridges in Porto.

With regard to the Arrábida Bridge, these options would also create a visual obstacle due to their greater height and adversely impose on this national monument.

The arch and frame bridge type solutions are able to use the required height of the bridge of 75 m in their structural form, see Figure 8, and make better use of the existing good rock foundation conditions on the side slopes.

Arch-type solutions are efficient for this range of spans and allow intermediate supports for the superstructure on the arch that reduce span lengths.

Usually, similar span lengths are used outside the arch.

On the right bank, however, see Figures 8 and 9, the proximity of the existing buildings makes it difficult to place piers in this area for several reasons - the effect on the stability of the buildings, great difficulties in providing construction access and an unacceptable adverse effect on the urban scale.

Thus, the placement of intermediate points of support in this slope was disregarded, leading to the adoption of a side span of more than 100 m.

Further, multiple vertical supports that typically arise with the classical arch solutions would make the solution less transparent and less "synthetic", thus deviating from the design objectives.

So, the arch solution was not considered optimal.

The frame-type solution allows the main span to be divided into 3 shorter sections for the superstructure and, in this case, proves to be structurally very efficient, which motivated its adoption.

For the main span, with a separation of 428.6 m between adjacent vertical supports, the sequence of support points for the superstructure in the main span of 124.30 + 180 + 124.30 m was adopted.

This span configuration is very appropriate, both from a visual point of view, due to the rhythm and proportion it creates, and from a structural consideration, given that the ratio between the side spans and the main span is approximately 0.7.

Finally, the superstructure is extended on both sides until reaching the abutments, in a balanced succession with span lengths of 21+ 30 + 55 + 104 + 428.60 m (124.30 + 180 + 124.30 m) + 98.40 + 65 + 33 m, where the ratio of adjacent spans is close to the structural optimum.

Assuming this typology as the basis for design, another fundamental decision was the choice of concrete as the main structural material.

The reasons for this choice are associated with the durability, maintenance and construction cost of the structure.

In a marine environment, a concrete structure has greater durability than a metallic structure, ensuring a longer service life with fewer maintenance operations and lower ongoing costs.

It also has a likely lower construction cost and lower economic risk, especially in the current inflationary context, in which the price of steel as a raw material has risen steeply.

A concrete solution also has other advantages related to its superior dynamic behaviour under wind actions and vibration, due to its higher mass.

For this range of span lengths between 100 and 180 m, a superstructure with a pre-stressed concrete box section of variable depth is the most competitive and efficient solution in the context of the life cycle, both structurally and economically, resulting in robust, yet slender and elegant structures, which integrate well in the landscape especially if the superstructure is placed at a high level.

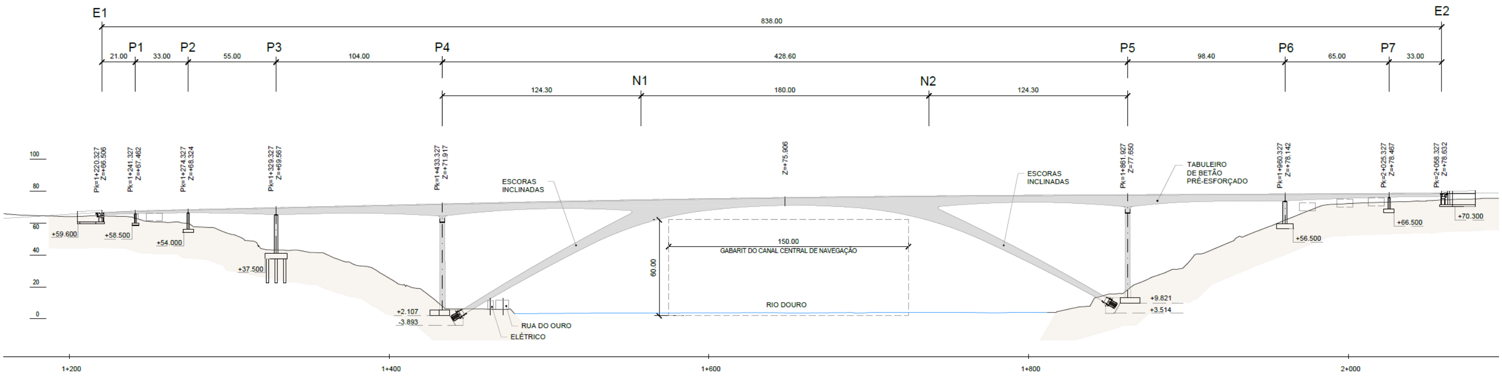


Figure 10: Elevation of the new Bridge over the Douro River

Click on the image to open it in a higher resolution

Figure 11: Plan

Click on the image to open it in a higher resolution

Examples of this type of superstructure with a variable depth section are the existing São João and the Freixo Bridges further upstream.

A variable depth and slightly curved soffit profile is the most structurally efficient solution, as it responds directly to the laws of stress variation, allowing an increase in the span lengths and a reduction of the number of supports.

The form thus follows the structural function, that is, the main function of the bridge.

The depth of the superstructure section is variable, with a maximum value at the supports of approximately 1/18 of the span, that is, around 7 m at the main piers adjacent to the spans of 125 m and around 10 m at the junction with the inclined struts of the frame.

The minimum depth is around 1/40 to 1/45 of the main span, that is, in this case, 4.50 m in the centre of the main span of 180 m.

To complement the integration and balance of the set, a 4 m deep section was considered in the centre of the spans of 100 and 125 m and a structural depth that progressively decreases at the ends of the bridge, consistent with the length of the spans and the height above ground.

It should be noted that these superstructure section depths at an elevation of more than 70 m above the ground are not out of scale but in proportion to the span to be bridged and the remaining elements of the structure.

The frame-type solution is effective for large spans. Previously constructed examples are the Cadore Bridge (Italy) with a span of 275 m or the Sfalassa Bridge (Italy) with a span of 376 m, but all of them are steel.

However, frame bridges do not normally have the visual fluidity that is sought in this case, in which the superstructure and the struts are integrated, forming a single continuous piece, comprising a continuous arch frame that undeniably relates to the arch curve of the Arrábida Bridge.

Thus, the frame was formed by two inclined struts with a very low angle to the horizontal, of approximately 30º, rising to a height of approximately 67 m and with a horizontal projection of 110 m.

The struts are recessed into the superstructure and increase in thickness, from 3 m at the source to 9 m at the connection with the superstructure.

The inclined strut profile connecting to the variable depth profile of the superstructure forms a unique curvature that frames the Arrábida Bridge in the background.

It is considered that this duality of an arch and curved frame is ideal both from a structural point of view and in forming a sympathetic neighbour of the Arrábida Bridge, whose arch positively relates to the new structure.

An additional advantage of the arch-frame solution is that it uses a shallow rock footing, efficiently transmitting the inclined forces from the struts directly to the ground.

With durability as one of the fundamental objectives of the proposed solution, the structure was designed to be monolithic as far as possible, with the least number of bearings and movement joints, which would be susceptible to maintenance and/or replacement during the service life of the structure.

For this reason, both the inclined struts of the frame and the main piers (P4 and P5) are fixed at the bases and made integral with the superstructure.

The central point of the main span is thus the fixed point of the structure for longitudinal horizontal movements.

The remaining piers (P1, P2, P3, P6 and P7) and abutments, shorter and further away from the fixed point, will have sliding bearings, guided in the longitudinal direction.

Durable spherical bearings will be used, as they provide a service life of more than 50 years.

Expansion joints are provided at each abutment.

Another fundamental design decision was that the inclined struts of the arch-frame in the transverse direction split from the connection with the superstructure downwards, into two independent and separate elements, which brings several structural benefits.

Firstly, it allows the total width at the base to be approximately 20 m, which improves transverse stability under wind and earthquake actions, taking into account the height and span of the structure.

This double bracing solution also allows for a better distribution of the load to the foundations and enhances the slenderness of the structure when it reaches the ground, which limits its visual impact and facilitates its integration into the urban scale of the city and the buildings that surround it.

The slender struts are highly compressed, which, given their length of approximately 125 m, requires the addition of an intermediate stabilization element between them.

Similarly, the main pier columns are subdivided into two elements with widths of less than 3 m, with a central opening to reduce the visual impact on existing buildings, increase transparency and avoid disrupting the urban scale by the placement of very wide walls of more than 60 m height so close to the existing buildings.

3. SUPERSTRUCTURE

The typical cross-section of the superstructure has an overall width of 15.40 m, comprising a central platform for two metro tracks measuring 6.40 m wide and two shared paths for bicycles and pedestrians, each with a clear width of 4 m.

Supports for the rail catenary and the lighting are placed at the edge of the clearance envelope of the metro between the tracks and shared paths.

The superstructure comprises a single cell box girder of upper width 10 m with side cantilevers each of length 2.70 m.

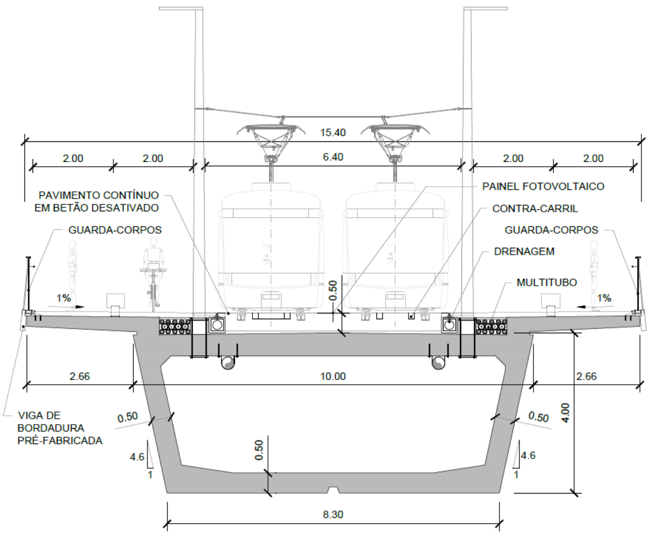
The Metro is supported on a track slab 0.50 m thick, sitting directly on the box girder top flange.

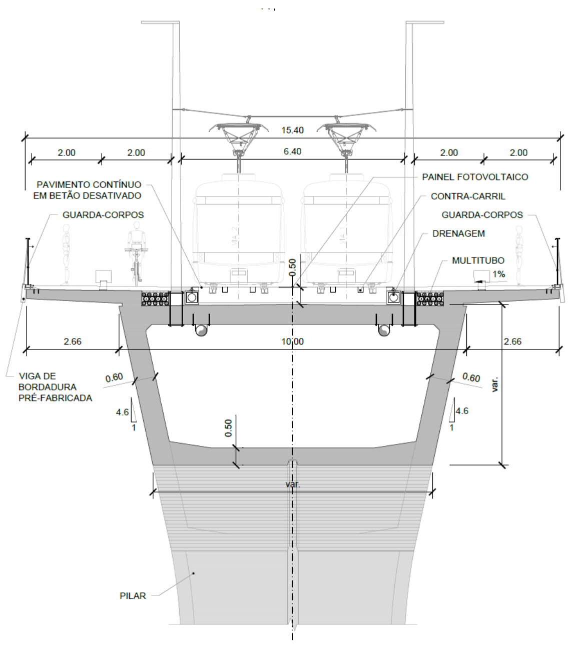
The side cantilevers are raised above the box girder top flange with a maximum thickness of 0.10 m. Service pipes and drainage are integrated into the upper part of the superstructure.

The central free span of the top slab between box girder webs is about 8.60 m.

The thickness of the webs is 0.50 m in the side spans and 0.60 m in the main spans of the central crossing, increased locally to approximately 1 m at the junctions with piers and inclined struts.

The lower flange slab includes a central cutout in the soffit which marks the central axis of the superstructure extending to the piers and frame arch.





Figures 12 and 13: Cross-section of the superstructure at midspan of the main span (on the left) and above Piers 4 and 5 (on the right).

Click on the image to open it in a higher resolution

The superstructure's variable depth, in addition to serving the purposes of structural efficiency described above, reproduces to a certain extent the existing superstructure profile of the Arrábida Bridge reaffirming the idea that the design of the new bridge is an evolution, a synthesis, and a consequence of that bridge.

The superstructure includes three types of post-tensioning.

The construction phase longitudinal prestressing is incorporated in the top flange slab to resist the bending moments induced by the balanced cantilever construction.

The continuity pre-stress is also internal, being arranged on the bottom flange slab with intermediate anchorage blisters, to resist the positive bending moments in the mid-span that occur following the closing of the main span cantilevers.

The superstructure also incorporates external pre-stress, tensioned after the main structure is completed, which is designed for the subsequent permanent load of the bridge and the live load.

This external prestressing can be re-tensioned and replaced, and it is fully accessible for inspection, which constitutes an important advantage for its durability and maintenance and ensures a service life equal to or greater than 120 years.

In addition, the external prestressing allows a reduction in the thickness of the webs, resulting in an optimization of the structure and a reduction in the weight of the bridge.

During the design phase, a detailed study of the wind behaviour was carried out including wind tunnel tests using sectional models and physical models of the bridge for the completed phase and the critical construction phase.

The results confirmed the good aerodynamic behaviour of the structure but also helped to fine-tune some elements such as the bridge railings and improve the proposed construction procedure and deck configuration to avoid any possible risk of vortex-induced oscillations.

The wind tunnel tests were carried out in Canada by RWDI, see Figure 14.

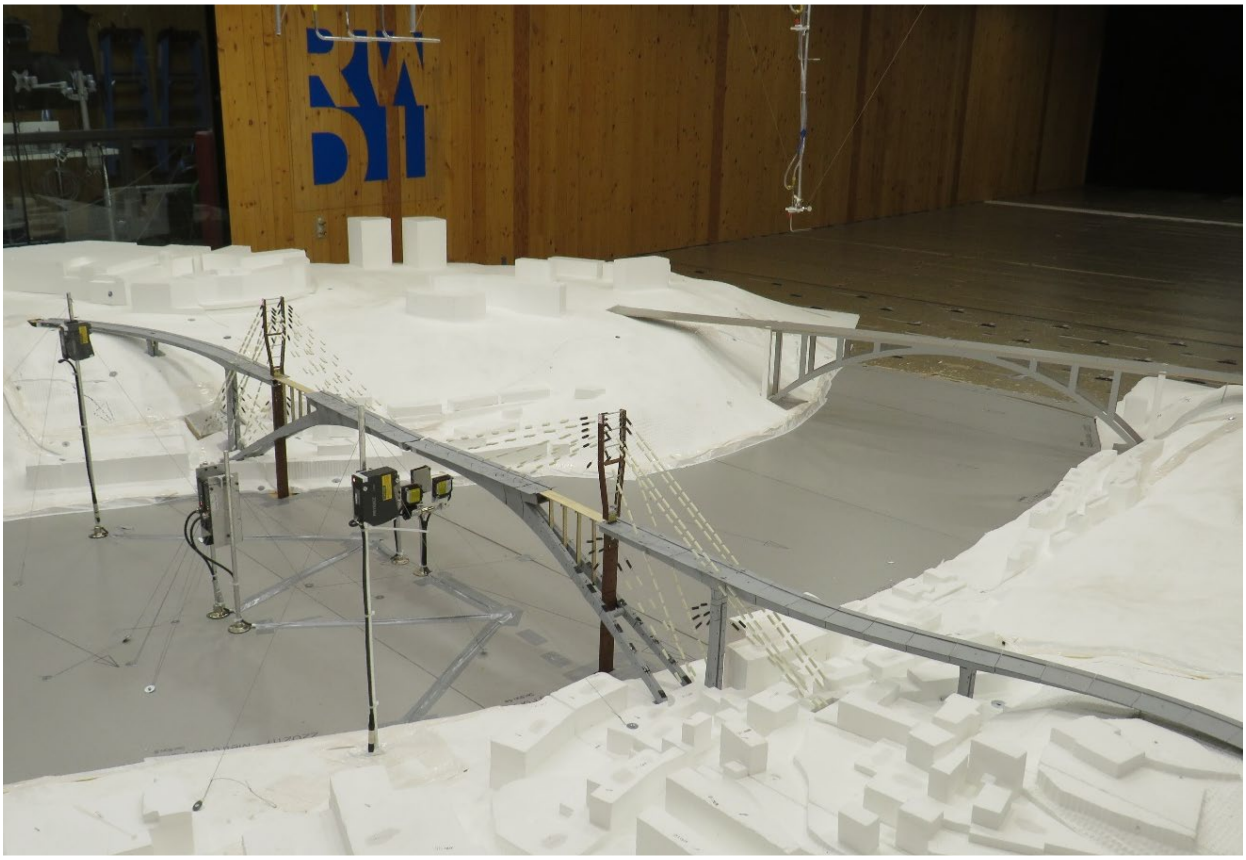


Figure 14: Wind tunnel test

4. FOUNDATIONS

The foundations of most of the supports sit directly on rock, namely on the existing granite formation, except for pier P3 which will have pile foundations due to the unique geological conditions in this location.

The foundations of the frame arch are large concrete blocks about 10 m high, 37 m wide and with a side-sloped surface in contact with the rock perpendicular to the slope of the inclined struts.

The foundations of the piers comprise spread footings of variable dimensions, adapted to the loads to be transmitted to the rock substrata.

In some cases, it will be necessary to reinforce the rock mass by means of rock injections.

5. SUBSTRUCTURE

Pier columns comprise the main vertical support elements.

The main piers P4 and P5 have a height of 57 m and have an integral connection with the superstructure without bearings, which provides better behaviour against wind and earthquake and allows the use of the balance cantilever method of construction of the superstructure.

The slenderness of these elements allows them to accommodate longitudinal movements due to thermal variations, creep, and shrinkage.

The pier columns comprise variable-width rectangular sections with external dimensions that vary between a minimum upper width of 6.43 m and

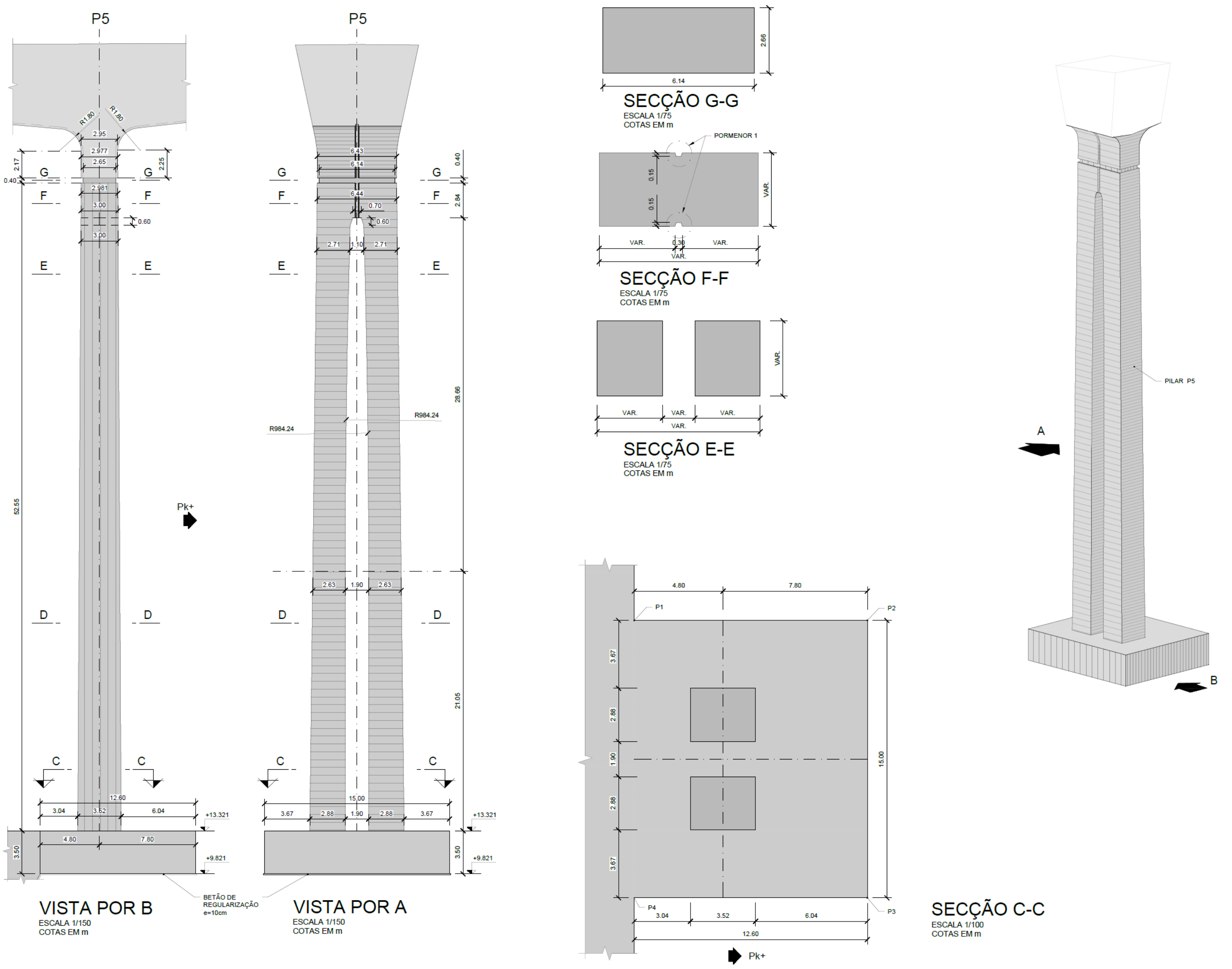


Figure 15: Pier P5. Click on the image to open it in a higher resolution

a maximum lower width of 7.75 m in the longitudinal direction, and with thicknesses also varying between 2.95 m on the upper face and a maximum of 3.54 m at the base in the transverse direction.

Below the superstructure, the section is divided into two parts with a central space that varies between 1.10 and 1.90 m for a maximum cross-sectional dimension of each of the two elements of 2.90 m.

This solution brings flexibility to the pier columns, provides more transparency and improves the relationship of scale with its urban environment.

In the case of pier P4 it also allows the inclusion of an elevator in this space to provide a vertical connection to the new square created in the space of the train station above and connectivity to the streets at the upper level.

Pier P3 is about 24 m high, with bearings supporting the superstructure.

In this case, bearings are required to accommodate longitudinal movements due to thermal actions, creep and shrinkage and also to avoid the concentration of bending moments in case of an earthquake.

The pier columns comprise a configuration similar to that of the main piers with a central space of 1 m and two elements of 2.50 x 2.72 m for a total width of 6.44 m. Piers P3, P4, P5 and P6 are expected to be built with climbing formwork in vertical lifts of 4.5 m.

Piers P1, P2 and P7 are placed in the end spans and have lower heights, having a solid square section with bearings supporting the superstructure.

The thickness of the inclined struts varies from 2.97 m at the bottom to a maximum of 8.52 m at the top.

In the transverse direction, the struts open in an A-shape, starting from an upper section with a width of 6.37 m.

There they are widened and divided into two elements that together reach a width at the base of up to about 21 m, splitting into two separate elements that reduce in width from 3.60 m to 3.2 m.

The rectangular section of the struts is hollow with variable wall thickness, between 0.40 m and 0.70 m on vertical walls, and between 0.70 m and 1.0 m on transverse faces.

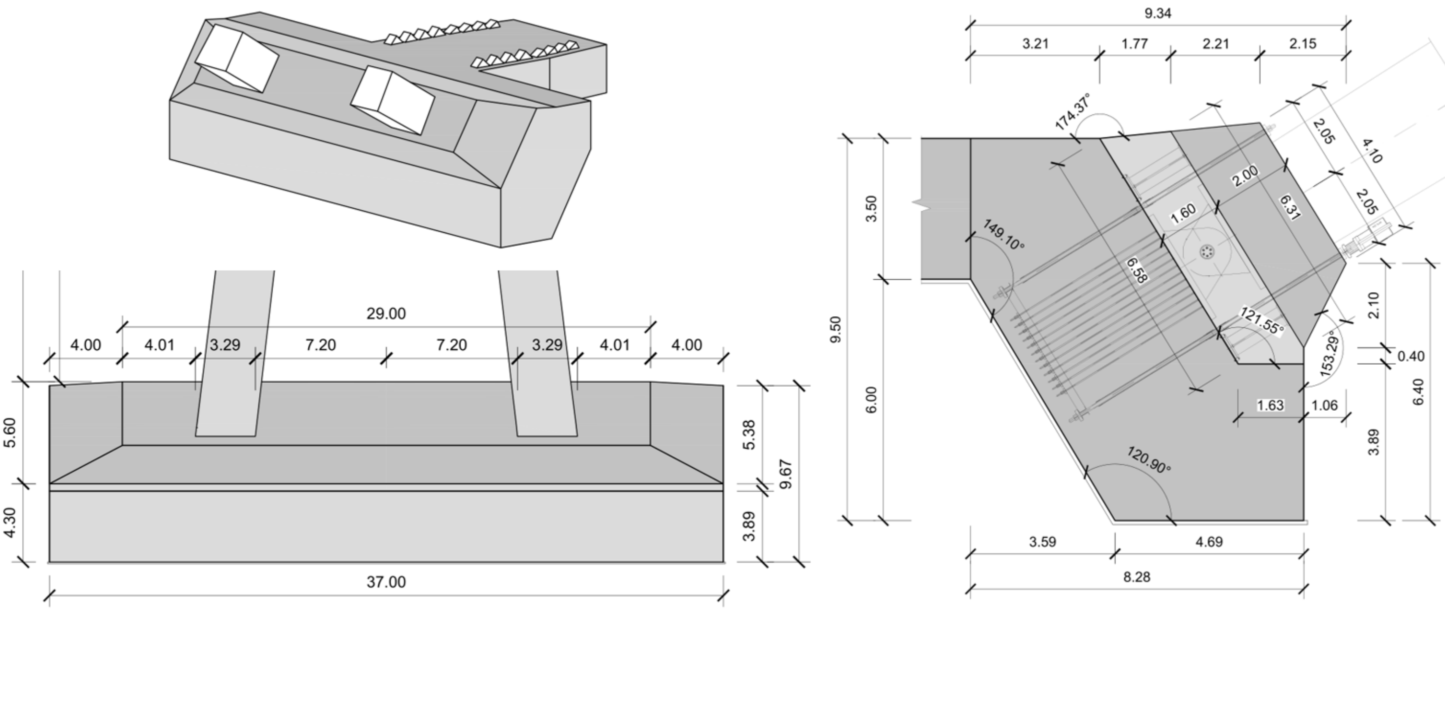


Figure 16: Foundations of the struts

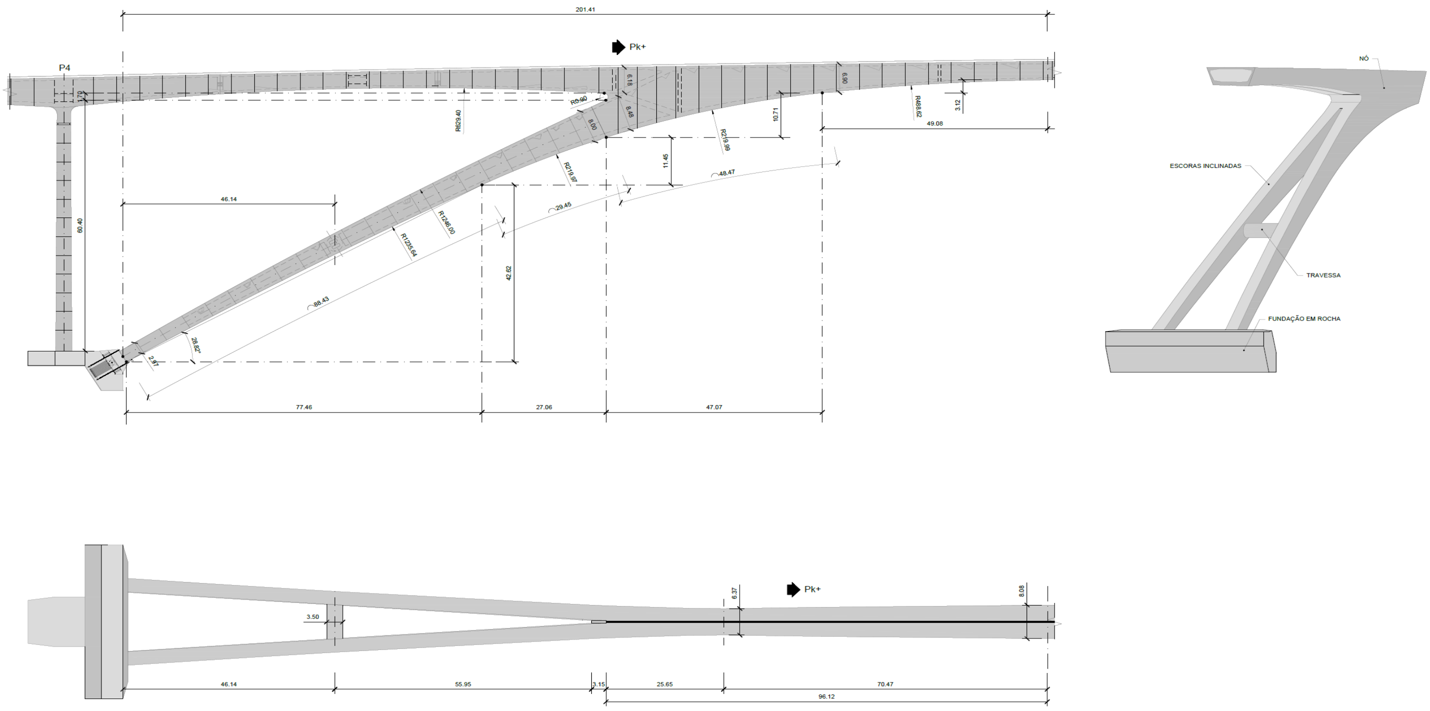
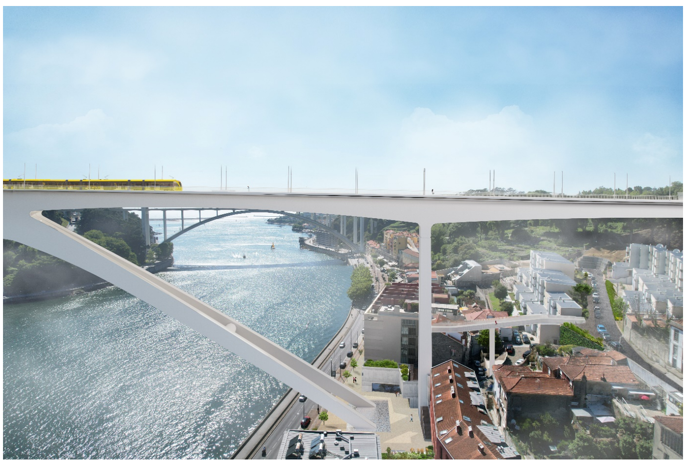


Figure 17: Inclined struts. Click on the image to open it in a higher resolution

The reduction of the section dimensions towards the base reduces the bending moments in the structure, effectively providing a hinge at the base, while the transverse separation provides the necessary stability against wind and earthquake loads.

For these elements, the method of construction is fundamental for their final behaviour.

The temporary cable stays used during construction not only serve as a support during its execution but also introduce a pre-compression in the struts and



Figures 18 and 19: The Bridge is located in an inhabited area of Porto. View of the abutments on both sides of the Douro River in the arch-frame equivalent to the effects of the permanent loads, which minimises bending moments in these elements due to subsequent elastic deformation of concrete under permanent loads.

The abutments are transitional structures between the bridge and the embankment track platform and have the function of retaining the soil in the rear.

The abutments include bearings and important elements such as full-width structural expansion joints and rail expansion devices for the metro.

Special care is given to the architectural integration of the bridge in the access areas as well as at the base of the main supports, creating new urban areas and taking advantage of the opportunity to regenerate some areas to create a better urban space and connectivity.

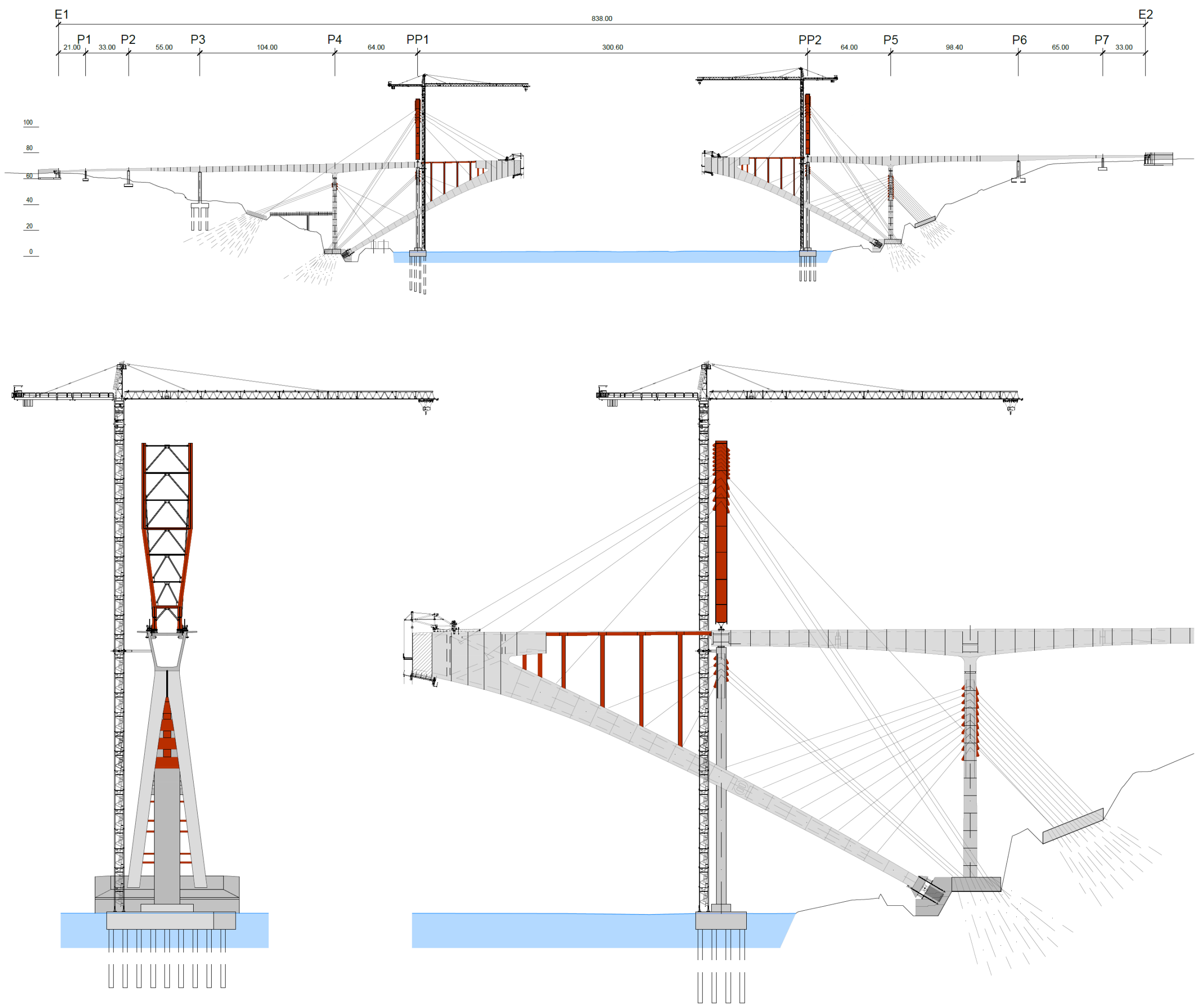
6. CONSTRUCTION PROCESS

The main construction process proposed both for the superstructure and for the inclined struts is the advancement in cantilever with form travellers. In the main span, to carry out this procedure, it is necessary to suspend the inclined struts and the deck using temporary cable stays. For this, provisional piers are placed in the river at both banks to serve as temporary support during construction.

The temporary anchoring is carried out from these piers and from a temporary upper steel tower, which includes front suspension stays and rear backstays, the latter being supported by a temporary anchored mass concrete block to resist the backstay forces.

The variable depth superstructure and struts are advanced in situ in 5 m long segments. During the construction procedure temporary steel hinges are incorporated at the strut bases to eliminate bending in these elements.

Figure 20: Construction procedure with auxiliary elements. Click on the image to open it in a higher resolution



This procedure minimises impact on the river and the banks and allows construction with a minimum of support points on the ground.

In the main spans in the central area of the superstructure lightweight concrete is used with a density of 18.5 kN/m^3 and a strength of 45 MPa, which reduces the weight and forces acting on the structure.

The main auxiliary elements for the construction of the works, see Figure 21, are:

- Temporary pile foundations and their pile caps where the temporary piers will be supported;
- Temporary concrete piers to support the temporary anchorage towers. Some of the cable stays used for the cantilevered construction of the frame arch will be anchored to the piers. These piers are 66 m high, 6.5 m wide and 2.5 m thick, with a hollow section and 0.70 m thick walls. They will be connected horizontally to the superstructure in intermediate stages to increase its stability;
- Temporary steel towers. These towers serve to anchor the longest cable stays. The towers have a height of about 50 m above the deck.
- The temporary stays are placed in two planes, anchored every 10 m on the inclined struts and on the superstructure. They are divided into two types, front stays and back stays;
- The backstay anchorages in the rock are reinforced concrete blocks anchored using rock anchors, tensioned to a predetermined force. In some cases, these anchorages coincide with the foundations of the permanent piers. Anchors will be fixed and tested first and then tensioned in stages as temporary cable stays are installed and tensile forces at these points come into play. At the end of the construction, once the arch is completed, a process of progressive deactivation of the cable stays and their removal takes place, which also ends with the deactivation of the rock anchors, the removal of the temporary towers and the demolition of the temporary piers.

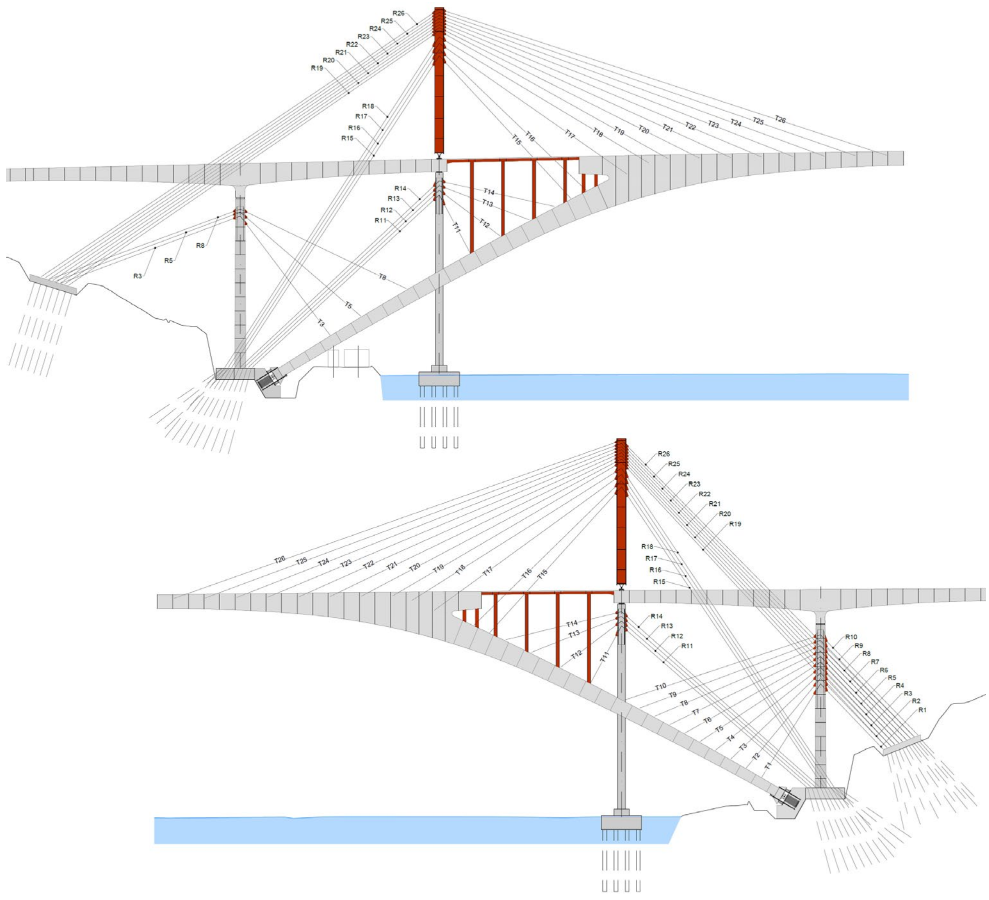


Figure 21: Auxiliary elements. Click on the image to open it in a higher resolution

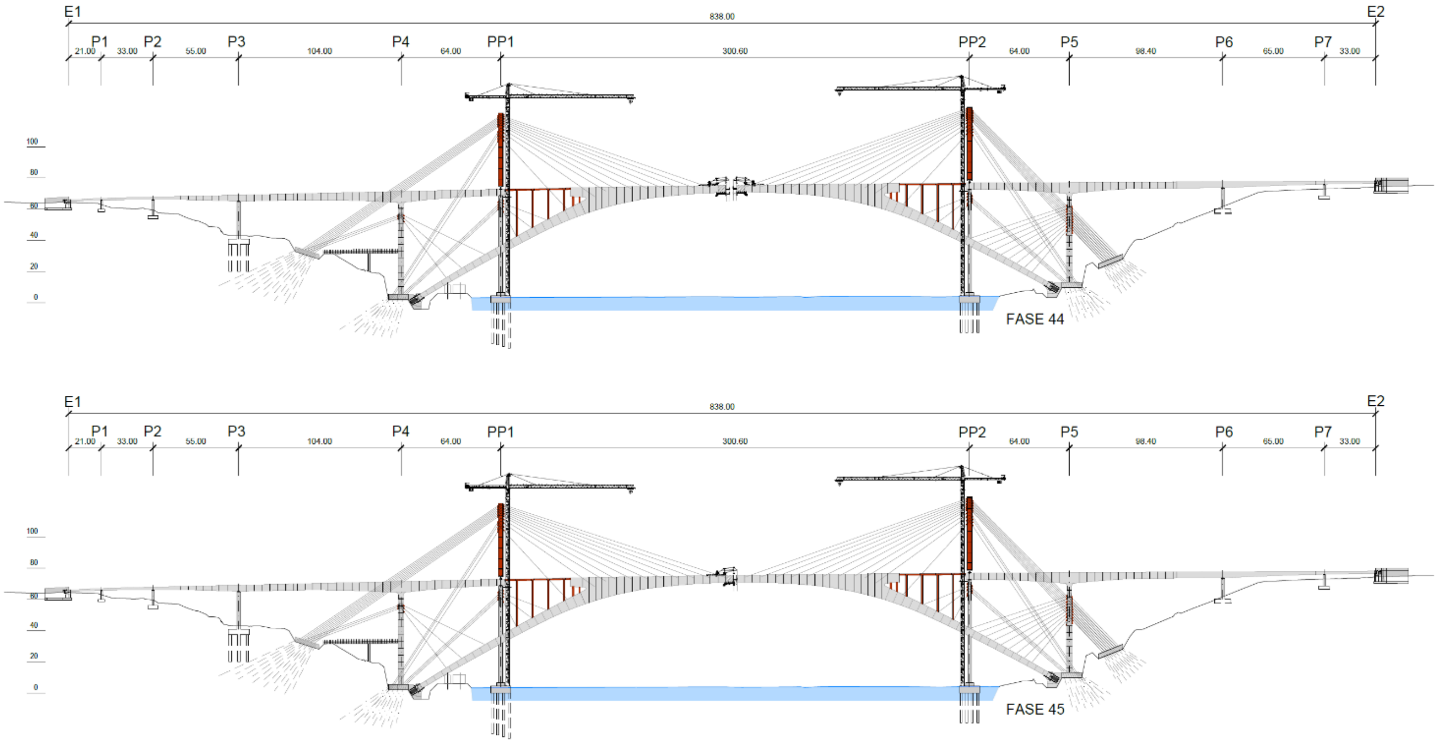


Figure 22: Closing the arch. Click on the image to open it in a higher resolution

7. CONCLUSION

As explained in detail above, the project responds to its inherent constraints with maximum transparency and respect. This is achieved by reducing the number of supports and framing the adjacent existing bridge without blocking any view from different aspects, including from the roads on both sides of the river.

In conclusion, the new bridge over the Douro has a very simple and efficient structural form.

It conveys a great sense of slenderness, which is fundamental for a structure of this scale and size, and is able to be successfully integrated into a sensitive urban environment, where maximum transparency is essential.

Figures 23 and 24: Renderings of the complete Bridge e-mosty

LAUNCHING GANTRY FOR SEGMENTAL CONSTRUCTION AT CAIRO METRO LINE 3 EXTENSION PROJECT, EGYPT

Dinis Sottomayor, Project Manager

BERD

Figure 1: Launching Gantry in operation

INTRODUCTION

Egypt's capital Cairo is the largest urban location in Africa and the Middle East.

With a city population of 10 million persons and 22 million in the Greater Metropolitan Cairo, it requires modern and efficient means of transportation for its inhabitants.

The Cairo Metro was opened in 1987, and with more than 1 billion annual commuters ranks amongst the busiest metro systems in the world.

The construction of the new Extension of Line 3 (Phase 3) of the Cairo Metro with 17 km will serve the main transportation corridors of greater urban Cairo.

This ambitious project is part of Greater Cairo's Transport Master Plan and is expected to greatly reinforce the public transport system in this area, thus promoting a positive mode shift from using private cars to a modern public transportation system, alleviating street congestion and diminishing emissions, with positive effects towards climatic change.

The project consists of the design, construction, and commissioning of Phase 3 of Line 3 of the Cairo metro system, including infrastructure investments, civil works, rolling stock, and a new maintenance area for rolling stock, thus helping to address the shortcomings of Cairo's overburdened public transport traffic system.

The project will:

- Contribute to economic growth by reducing urban congestion and reducing the user's travel time;
- Improve the livelihoods of Cairo's population in an inclusive way by enhancing mobility and improving access to education and jobs; and
- Mitigate climate change and pollution by promoting a more environmentally sustainable means of urban transport.

BACKGROUND

The project is financed by the European Investment Bank. In 2015, The Arab Republic of Egypt and the European Investment Bank (EIB) signed a loan agreement worth EUR 200 million for financing a project to promote public transport in Greater Cairo.

The loan is part of the EUR 600 million approved by the European Investment Bank for the Cairo Metro project.

The European Investment Bank (EIB) is owned by members of the European Union and is one of the leading development finance entities in the Mediterranean region.

The Bank's goal is to support economic and social development by improving people's living conditions, and in this role finances projects related to climate, development, infrastructure and business enterprises.

Client: National Authority for Tunnels (NAT) - Ministry of Transport

Contractors: EFJV, a JV of Vinci Construction Grand Projects, Bouygues Travaux Public, The Arab Contractors Company and Orascom Construction

The EIB aims to establish a tangible presence in the partner countries, focusing on the economic and social priorities of the beneficiary countries to which it not only contributes its financing capacity but also adds value in project implementation and modernization of public policies through its technical and financial expertise and advisory services.

Since operations began in Egypt in 1979, the Bank has provided over EUR 6.4 billion of financing in the country.

Operations in Egypt cover all sectors, including energy, transport, water and industry, as well as support for small and medium-sized enterprises (SMEs) through credit lines and risk capital.

The EIB's aim in the past years has been to deploy its resources to provide an appropriate practical response to the expectations expressed by the Egyptian people.

The total cost of the project (estimated) will be 2,418 million EUR (2,620 million USD).

Figure 2: Location of the project. Source: Google Maps

GREATER CAIRO METRO AL THAWRA LINE (LINE 3) PHASE 3

Cairo Metro Line 3 is managed by French company RATP Dev, a branch of the Ile-De-France/Greater Paris transportation system.

Line 3 is built in three phases, and the route of this phase extends from Attaba to Rod el Farag Axis north of Imbaba, passing through the Ring Road to Etay El Baroud Railway, heading south to Cairo University crossing Gameat El Dewal Street and Boulak El Dakrour to connect with Line 2 at Cairo University Station.

The total length of this phase is about 17.7 km, comprising 15 stations, and it is divided into three parts:

- Phase 3A: 4km from Attaba to El Kit Kat with four underground stations
- Phase 3B: 6.6km from El Kit Kat to the final station at Rod el Farag Axis with six stations - one underground, four elevated and one ground-level station.
- Phase 3C: 7.1km from El Kit Kat to Cairo University with three underground, one elevated and one ground-level station.

Figure 3: Route of the metro line

Credit: National Authority for Tunnels, Egypt

Video: BERD´s LG 36 at Phase 3 of Line 3 of the Greater Cairo Metro. Credit: National Authority for Tunnels, Egypt

Click on the image to play the video

VIADUCT CONSTRUCTION

The viaducts of the Greater Cairo Metro Phase 3B and 3C constitute simply supported spans and cantilevers, both carried out with prefabricated segments designed for two tracks.

For the precast segmental deck erection, BERD designed and built an Overhead Launching Gantry (LG) according to the Contractor´s specifications and technical requirements, from which the following were critical:

- Capability to handle erection for a viaduct with significant inclinations ±4% and tight curvature radius until 200 m (656 ft);
- Variable span lengths, from 14 to 36 m (46 to 118 ft);
- Movement through areas with high construction density (roads and buildings) with significant dimensional and kinematic restrictions;
- Ability to cross several obstacles;
- The construction equipment requires maximum reliability and performance;
- In all cases, provide safe access to work and main maintenance areas without the need for external means;
- Extra safety operation due to proximity to buildings, roads and people.

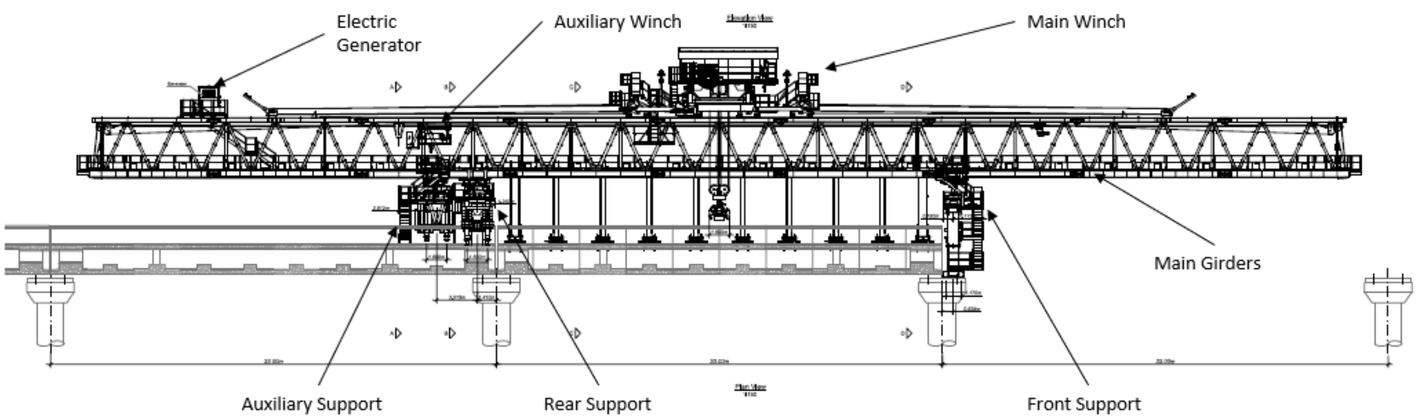
The LG36 was predicted to be used in phases 3B and 3C, but due to constraints in the start of the works it only was used in the Phase 3B.

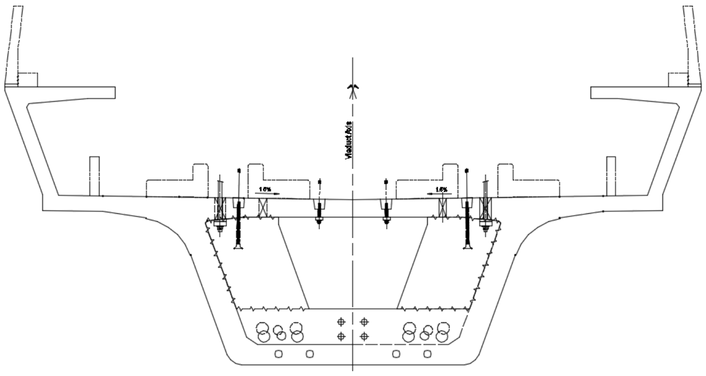
SEGMENTAL CONSTRUCTION

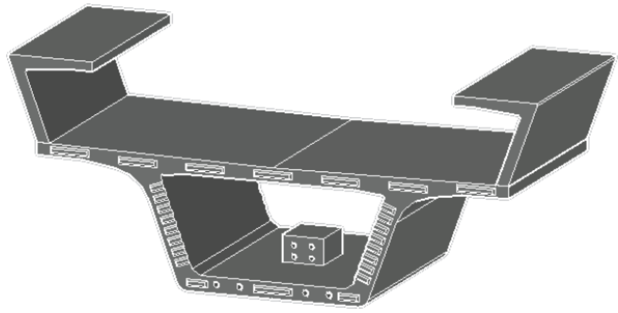
Total number of spans:	165
Spans built by LG36:	101
Minimum plan curvature:	200 m
Maximum longitudinal slope:	4.00%
Segments per span: Variable	Max 11 segments
Weight of a segment:	Between 46 and 63 ton
Length of a typical segment:	9.06 m
Height of a typical segment:	3.6 m
Width of a typical segment:	3.45 m

Figure 5: Route of the extension project metro line 3 built with the LG. Source: Google Earth

↓ Figure 4: BERD LG36







Figures 6 and 7: Typical segment

For the equipment, and after a tender procedure, BERD was chosen as the preferred supplier and an LG36-S model of overhead launching gantry was supplied.

In general terms, this Launching Gantry is able to erect simple supported segments for spans up to 36 m by the typical span-by-span construction method, with a minimum plan radius of curvature of 200 m and a maximal longitudinal slope of ± 4%.

For spans up to 33 m, the launching between the adjacent spans is done without counterweight.

For a span between 33 and 36 m, it is necessary to have a counterweight to ensure longitudinal stability during launching.

Although the project design does not have any spans above 33 m, construction progress could uncover conditions under which a pier needs to be relocated; this event can be covered by the LG special counterweight condition for a maximum span of 36 m.

The LG36 can be moved in both longitudinal directions supported on the already built deck (with Front Support in short configuration). For the erection of the deck the LG36 can only move forward.

The Main Girders are supported on three supports always in the same sequence. From rear to front, the sequence is Auxiliary Support, Rear Support and Front Support.

Figure 8: The construction site in the close vicinity to buildings

Credits Stephane Ciccolini

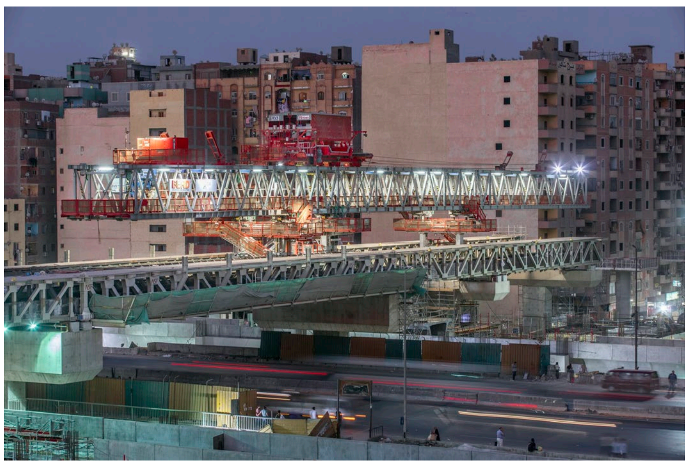


Figure 9: LG special operation crossing over the Ring Road

Credits EFJV

The LG36 also includes two winch trolleys: the Main Winch Trolley (MWT) and the Auxiliary Winch Trolley (AWT).

The MWT is also responsible for the LG36 locomotion. It also includes a Support Launching System used to move the supports longitudinally on the main girders.

TYPICAL WORKING OPERATION CYCLE

After the diagnostic of the launching conditions, the auxiliary support is assembled and the rear support is displaced to the launching position.

The longitudinal slope for launching is adjusted and the Main Winch Trolley is fixed to the Rear Support.

Then, after the first transverse movement on supports (in the case of plan curvature), the first longitudinal launching is started followed by the second transverse movement on supports.

The front support is transported and assembled on the pier cap, followed by the second longitudinal launching.

The auxiliary support is disassembled, suspended on the Main girders and transported to its final position.

After that, the third transverse movement, the third longitudinal launching and the fourth transverse movement on the supports can follow.

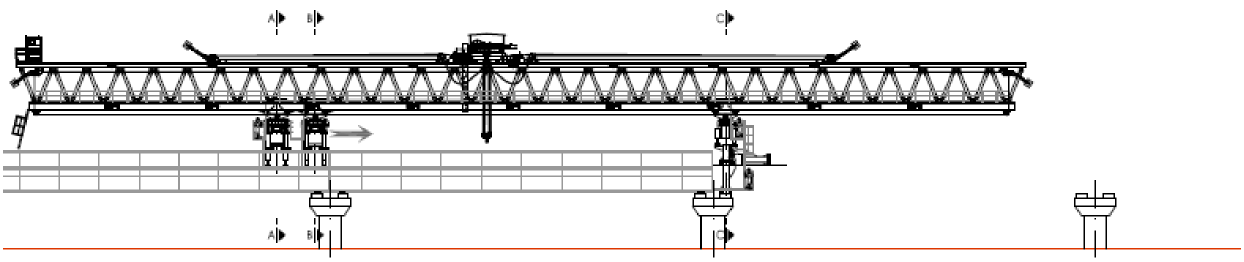
When finished, the main Winch Trolley on the Rear Support is disassembled.

The slope for the erection is adjusted and erection conditions set.

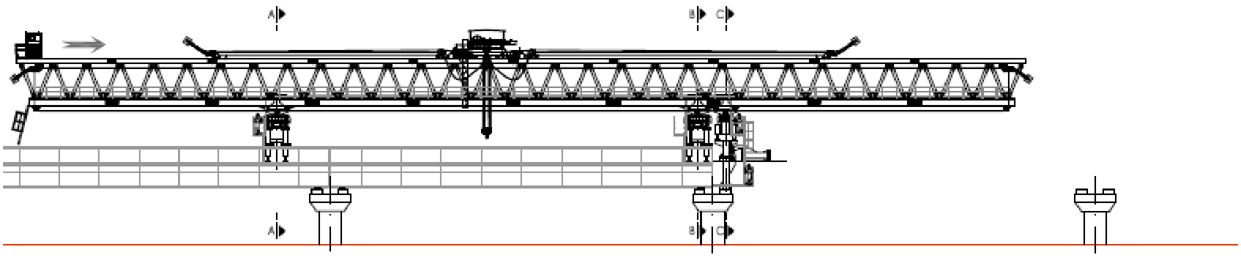
Pre-hanging and hanging of segments are followed by deck stressing.

LG36 Characteristics and Functional Limits		
Brief Description		Overhead Launching Gantry for span-by-span segmental construction for spans up to 36 m
Maximum Span Length without Counterweight		33 m
Minimum Plan Radius (for spans up to 33 m)		201.625 m
Maximum Longitudinal Slope		± 4%
Maximum Longitudinal Slope (Launching)		± 4%
Maximum Longitudinal Slope (Erection)		± 2%
Maximum Segment Weight		620 kN (63 ton)
Maximum Span Weight (33m span)		5,062 kN
Maximum Span Weight (36m span)		5,332 kN
Segment Feeding		From underneath the span to erect
Maximum Lifting Height		30 m
Main Girder Dimensions (LxHxW)		93.4 m x 4.45 m x 6.55 m
Approximate Travelling Mass		395 ton
Approximate Total Mass		465 ton
Wind Limits	Launching	10 m/s
	Erection of segments	23 m/s
	Intermediate without segments weight	35 m/s

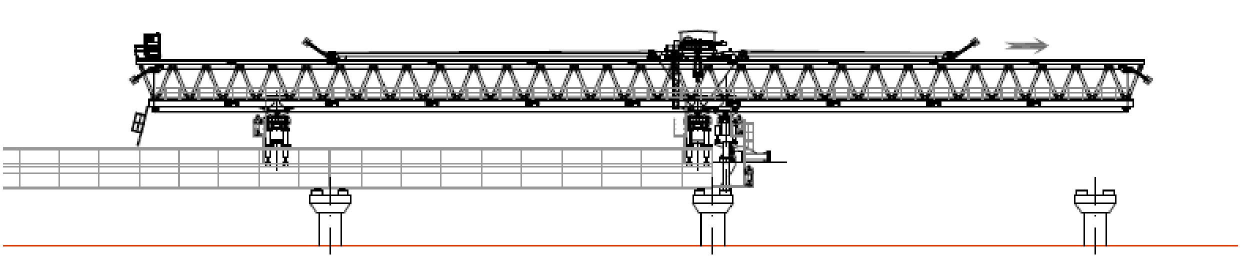
1 - Segments Assembly Position



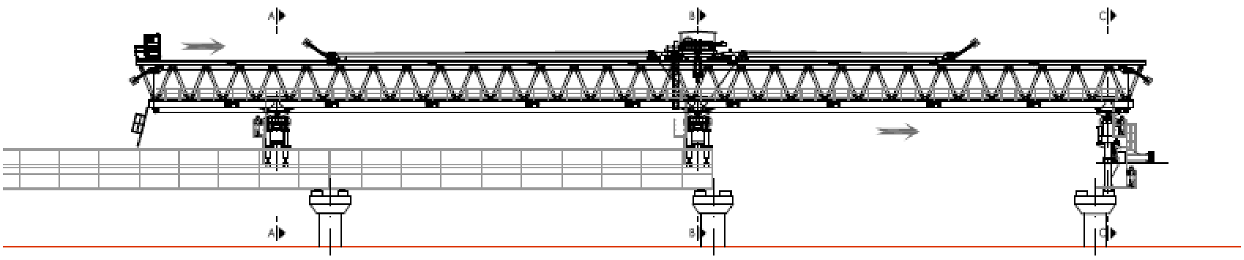
2 - Launching Position / Rear Support Displacement



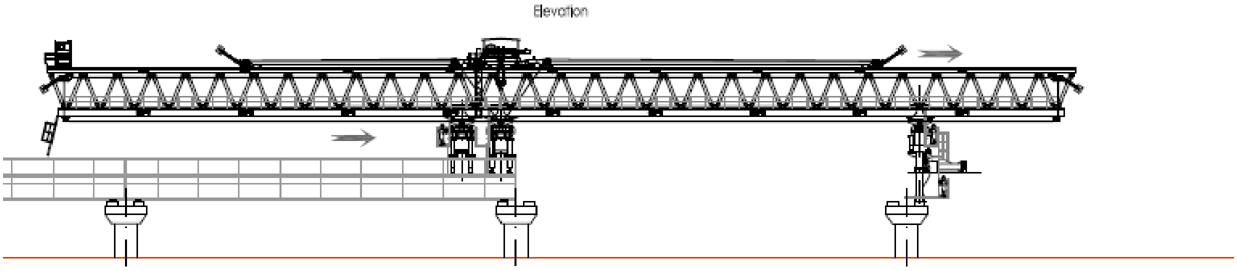
3 - First Step Launching



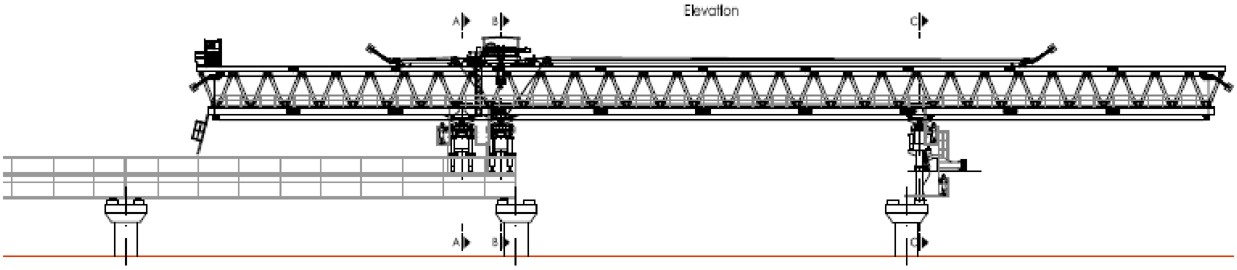
4 - Front Support Displacement



5 - Second Step Launching and Auxiliary Support Displacement



6 - Launching to Segment Assembly Position



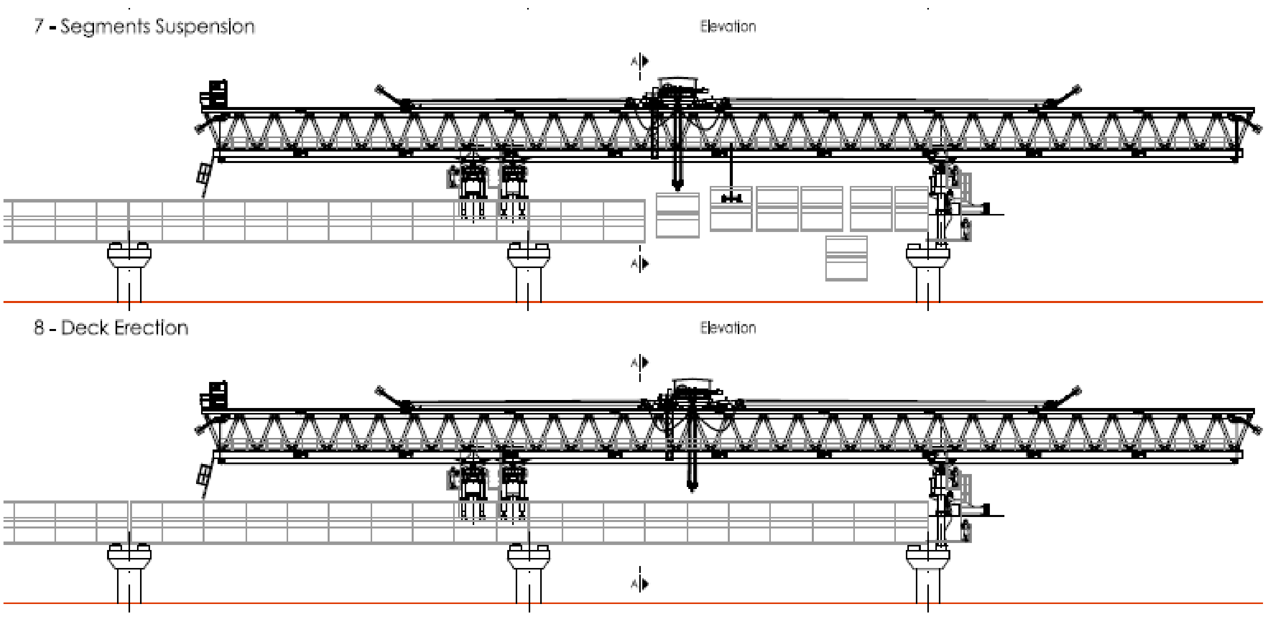


Figure 10: Typical working operation cycle

CONSTRUCTION PROCESS

During the erection of the segments, the Main Girders are supported only on the Rear Support and on the Front Support with the centre of gravity of the Main Girders approximately in the mid-distance of the two supports.

The segment feeding is done only from below the LG36 in the section between the piers of the span under construction.

The segments are transported with low-bed trailers trucks and positioned in the MWT range of operation.

Different pre-hanging configurations can be done due to existing restrictions on the terrain below the Launching Gantry.

The erection comprises two phases:

- Pre-hanging of all segments in the approximately final position to control the tension in the epoxy interfaces between segments, and
- Gluing the hanged segments in their final positions.

In both phases, the segments are suspended from the Main Girders by prestressing bars.

The control of the final position of the segments is done by surveying.

After the completed span is glued together, the deck is prestressed and the weight of the deck transferred from the Main Girders to temporary bearings (hydraulic cylinders positioned near the definitive bearings of the bridge) by removal of the prestressing bars.

The transfer of the deck weight from the temporary bearings to the permanent ones is done after the

Figure 11: Transportation of segments from a truck

launching of the LG36 to the next span without its intervention.

During the launching, the Main Girders are moved over the supports and they are supported with three different configurations:

- By the Auxiliary and the Rear Supports;
- By all three supports;
- By the Rear and Front Supports.

The Main Winch Trolley is fixed during the launching to the Rear Support to promote the longitudinal movement of the Main Girders.

Although the longitudinal slope of the Main Girders for launching is approximately the same as the next span slope of the bridge deck (which is up to 4%), for the erection the longitudinal slope of the Main Girders is adjusted by changing the height of the supports to be lower or equal to 2%.

During both the erection and launching, the adaptation of the LG36 to the plan curvature is done only with eccentricity on the supports since the Main Girders are rectilinear.

The power supply of the LG36 is provided by a generator positioned above the Main Girders.

MAIN EQUIPMENT FEATURES

The most significant features of the BERD´s launching gantry are:

- Robust design and fabrication, followed by an extensive factory acceptance test
- Autonomous locomotion and support placement
- Telescopic Leg on Rear Support
- Fully Automatic System - Spreader Beam / Connecting Beams
- Front Support Elevation Cylinder Configuration
- Supports during Longitudinal Movement System
- Platforms and Ladders for safety and comfort
- Global Safety and Monitoring System
- Longitudinal Fixation System

Figures 12 and 13: The works at night

CONCLUSION

The Overhead Launching Gantry LG36 was used for the construction of viaducts for Phase 3 of the Metro Extension in Cairo, Egypt, in a very efficient and reliable way.

High productivity was reached, with cycles of 2.5 days, even during the pandemic period.

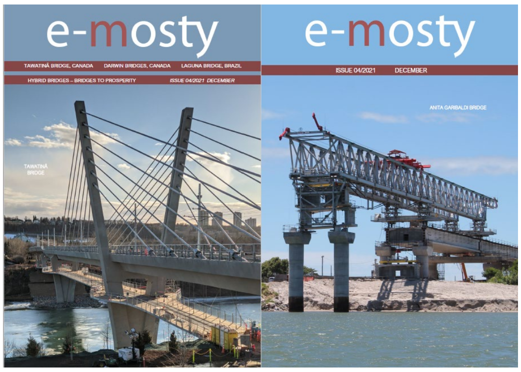
The successful outcome of the project was achieved through a continuous and proactive interaction between the supplier (BERD) and the contractor (EFJV). Some of the key factors that contributed to this outcome were:

- Effective communication: As the supplier-contractor relationship developed and became established, communication was clear and timely;
- Continual improvement of operations: sharing of ideas and feedback, lead to the improvement of operations;
- Resolution of issues: When issues arose during the project development, the two parties facilitated quick and effective problem-solving;
- Continuous follow-up: the presence of the supplier throughout the project timeline assured the necessary support at the most critical operations.

By incorporating these practices, the supplier and the contractor were able to establish a strong working relationship, leading to a fluid project development.

The high quality of LG and its correct performance, contributed to the contractor's successful job in delivering viaduct with zero accidents and incidents during its operation.

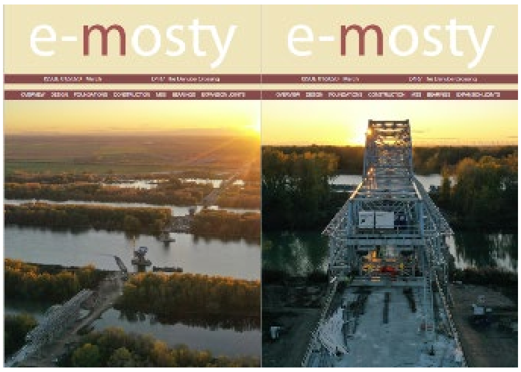
Read about our other projects in previous e-mosty



LG50-S FOR THE SEGMENTAL CONSTRUCTION OF THE ANITA GARIBALDI BRIDGE IN BRAZIL

Pedro Pacheco, André Resende, Alberto Torres, Hugo Coelho

Faculty of Engineering of Porto University (FEUP), BERD S.A., Portugal



M1-70-S MOVABLE SCAFFOLDING SYSTEM (MSS) FOR THE D4R7 THE DANUBE BRIDGE, SLOVAKIA

Pedro Pacheco, BERD; André Resende, BERD; Hugo Coelho, BERD; Filipe Magalhães

Faculty of Engineering of Porto University (FEUP), CONSTRUCT

WIND ENGINEERING IN CHENAB BRIDGE, INDIA

Risto Kiviluoma
WSP Finland Ltd.

1. INTRODUCTION

Wind and turbulence parameters on the Chenab Bridge site are characterized by rough topography with steep-sloped hills and deep river canyons and with the high altitude of the deck structures above the underlying Chenab River.

The bridge has an overall length of 1315 m with the tallest piers of 130 m in height, indicating that the lateral stiffness for wind load is one of the key design issues.

This is to ensure the riding comfort and safety of trains and to keep additional dynamic wind load at a reasonable level.

Reliable estimation of the wind load itself is of special importance for structural and geotechnical analysis.

It became clear to the designer already at the tender stage (year 2004) of the project that specialist wind engineering expertise is needed to analyse wind effects, and that the outcome of such expertise needs to be combined and approved to be used in the design.

A widely used concept in major projects is to employ external experts to do wind engineering with wind tunnel testing on whose results and advice various project parties (designer, contractor, proof consultant and client's expert board) can rely, and on which they can base their own analysis.

In the case of the Chenab Bridge, the approach was to use the in-house wind engineering expertise of the designer throughout the project, outsourcing only the testing part.

Testing was done in an early stage of the project in the year 2005.

This was largely based on a semi-empirical approach, i.e., using section-model type aerodynamic input together with analytical (mathematical) assessments for 3D buffeting and 3D vortex-induced vibration (VIV).

Related aerodynamic input parameters can at the initial design stages be based on reference projects, design codes and literature; and recently more and more often on 2D computational fluid dynamics (CFD) simulations.

As the project has lasted 19 years and included design changes and updated construction plans, it could be said that the semi-empirical approach and in-house wind engineering have been quite versatile and successful for the needs of the project.

Instead of procuring new testing for each change, with a delay in preparing and testing the scale models, the assessments were conducted analytically in parallel with the other design work.

This approach may not be feasible for all projects, and essentially requires wind engineering skills not only by the designer's team but also by the proof consultant and client's experts.

This project's key viewpoints with the main technical wind engineering challenges of the Chenab Bridge are addressed further in this paper.

2. TENDER STAGE ASSESSMENTS

Wind engineering tasks started at the tender stage in the Spring of 2004.

Tenderers were asked to prepare, in a few months´ time frame, technical and commercial proposals for two major bridges on the same railway line: at Anji Khad and Chenab.

Overview sketches of the bridges were given, requesting a design for the steel arch bridges with notable span length and height of the arches.

In technical specifications the wind loads were requested to be based on Indian Standard IS: 875 (Part 3) - 1987 [1] with an addition that the dynamic effect of wind shall be examined.

It was also specified that confirmatory-type wind tunnel testing should be conducted and completed by the tenderer within six months from the award of the contract.

Tasks were conducted using a routine workflow of analytical 3D buffeting and 3D VIV analysis with frequency domain methods.

These were assisted by the in-house specialist software [2]. An envisaged typical aerodynamic and turbulence input was assumed.

The method employs 3D vibration mode shape data of the bridge, which in typical cases is available from the structural finite element (FE) model by the designer, i.e., it is assumed as input to the wind engineering assessments.

In the present case, however, the FE–model for mode shape analysis was prepared by the wind engineer to work in parallel with the rest of the design team to meet the available time frame.

The basic reference wind speed at the site was well defined in the Indian Standards as 39 m/s gust wind velocity (3 s. 10 m altitude, open flat topography, 50 years return period).

With typical gust factor 1.5 this corresponds to 26 m/s mean wind velocity. It was unlikely that any micrometeorological study of local wind speed records would have significant statistical evidence to change this, so it was adopted as a basis for wind engineering assessments.

The main uncertainty in the analysis was related to wind speeds at the altitude of the structures.

Hilly topography, which is partly covered with trees, does not fall into any standard terrain category used in topography roughness change analysis.

For hilly areas surface roughness parameter (z_0) of several metres has been proposed, while z_0 = 0.02 to 0.05 m is generally adopted for open flat topography.

Furthermore, the depth of the boundary layer in storms is uncertain, as theoretical models predict values of the same order as heights of the nearby hills (i.e., one to two kilometres).

This implies that the theoretical relation between the wind velocities on hilly and flat topographies is largely uncertain.

For the tender stage assessments the assumption was made for the Chenab Bridge that the zero plane is approximately on the level of the arch abutments when computing the wind speed and turbulence profiles, and that the effective roughness length z_0 = 0.3 m.

This gave U_m = 43 m/s (10 min mean wind velocity, 100 year return period) and longitudinal turbulence intensity I_u = 18 % at the altitude of the deck.

By assuming flat topography and a full deck altitude of 359 m, the results would have been U_m = 44 m/s and I_u = 10%. So a stronger turbulence was assumed.

Results indicated 0.6 m peak lateral displacement at the deck (100 year return period wind).

In this first design variant fundamental lateral vibration mode had a natural frequency of 0.26 Hz (= period 3.8 s.).

In the technical specifications, a warning system was requested to be built. It is used to close traffic at wind speeds greater than 25 m/s.

This was interpreted as mean wind velocity, and as a basis to check lateral deflection criteria. The lateral deflection was requested to fulfil UIC 762-2R [3] which is set for high-speed trains as L/4000.

Unfortunately, no details were given on how L should be interpreted in the case of bridges of the Chenab type.

If L is taken as an arch span length, it would imply 0.12 m, which is difficult to fulfill.

In the more recent edition of the code [4] the criteria are stated for the change of angle and for the change of curvature radius, which confirmed that the lateral deflection is not an actual issue for trains.

To produce wind loads for the design, first estimates of the equivalent static wind loads (ESWL) [5] were extracted for the in-service bridge and for the construction stage with the longest arch cantilevers.

These were further proposed to be included in the design basis of the bridge.

There exists a general procedural challenge, as ESWLs are dependent on wind-induced vibration of the bridge, which in turn depends on the final design and possible wind-tunnel tests results.

Moreover, the ESWL procedure appears to be standard only in the North American design practice, and less well known elsewhere.

3. DETAILED DESIGN

Once the contract was made for the Chenab Bridge in the Summer of 2004, one of the first tasks was to prepare specifications for the wind tunnel testing and to update the tender stage assessments for the developed design.

The main design variants studied included concrete fill of arch steel tubes and circular vs. rectangular main arch members.

In the developed design fundamental natural frequency of the later mode was 0.31 Hz (= period 3.2 s.).

Wind tunnel specifications were prepared for testing of:

1. Topography model for site wind velocity and turbulence characteristics;
2. Section models for deck and arch (steady aerodynamic coefficients and VIV lateral force coefficient for deck);
3. Full aeroelastic model testing of the in-service bridge.

FORCE Technology in Denmark was awarded the contract to do the testing.

Testing was conducted in the above described order, to best serve the needs of the design.

3.1 Topography model tests

The topography model was designed and manufactured in Finland by WSP and transported to Denmark for testing in a boundary-layer wind tunnel.

The large size of the wind tunnel with a working section of 13.6 x 1.7 m^2 and a length of 15 m allowed the geometric scale of the model to be almost freely chosen, bearing in mind, however, the blockage effect in the wind tunnel.

The geometric scale was chosen to be 1:2200 and the size of the proximity model was 5 m in diameter.

In full scale, this is equivalent to an 11 km diameter. Hills 1 km tall are 0.45 m tall in the scale model.

The model was manufactured by automated milling from the 3D computer model.

The computer model itself needed to be created manually from map elevation contours, as modern Digital Elevation Model (DEM) data was not available at the time.

Hot-wire measurement technology was used, which allowed along-wind horizontal and vertical components of mean wind velocity and turbulence intensity to be measured.

Measurements were taken at 10 points at the location of the bridge structures with 30° wind direction increments.

Figure 1 illustrates the details of the topography scale model.

Inlet wind profile and turbulence were adjusted by spires to present open flat topography, as per the assumed basic reference wind condition from the Indian Standard.

Although the test setup complies with typical wind tunnel and CFD practices, the approach can be considered to be conservative for such wind directions that are not along the direction of the river canyon.

This flat topography does not exist anywhere close to the bridge site.

In the test setup, there is a full-scale fetch of 5.5 km for the boundary layer to develop into hilly topography.

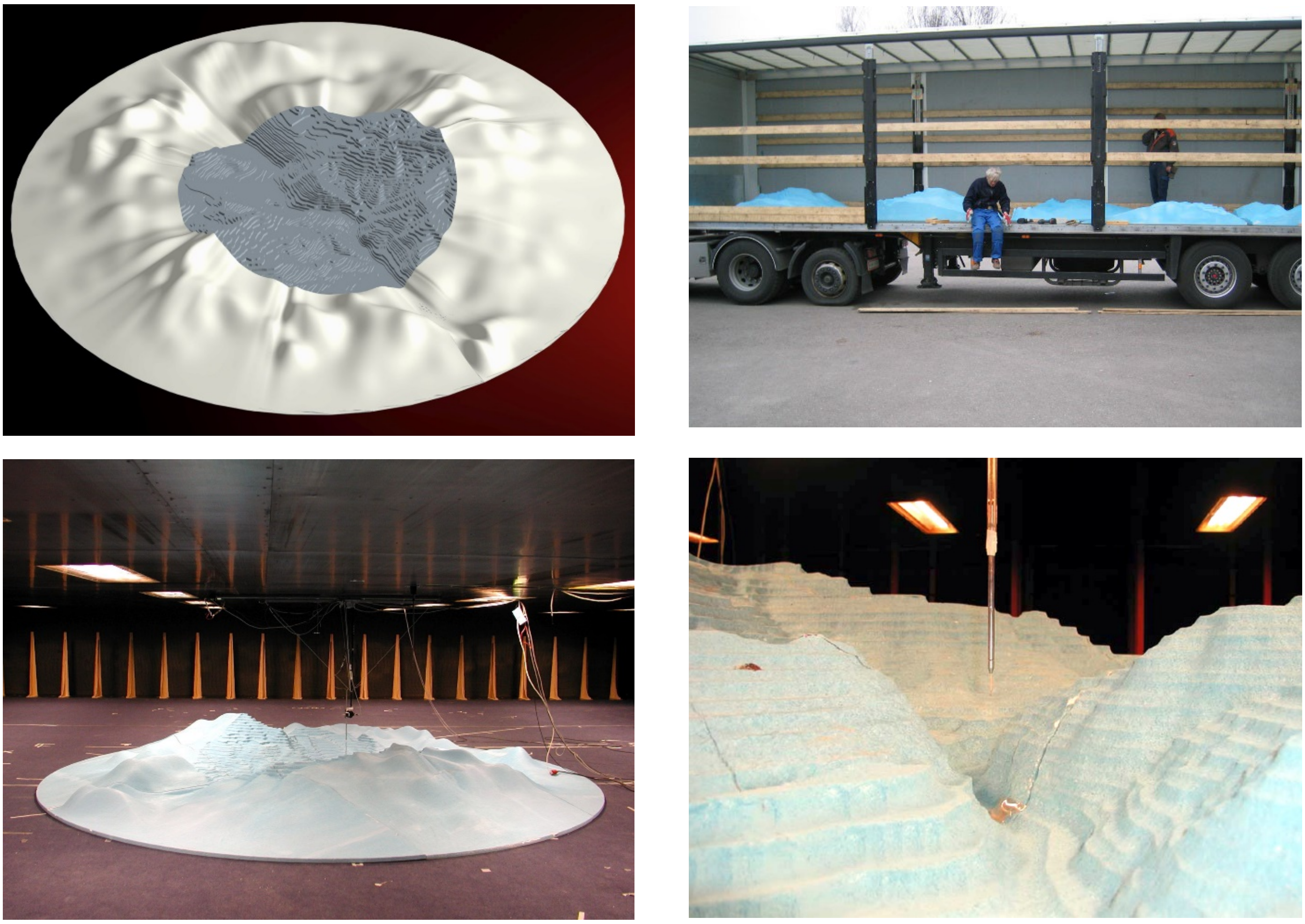


Figure 1: Topography testing: a) computer model b) the scale mode ready for transport c) the model in the wind tunnel and d) hot-wire measurement technology

If this fetch is longer (e.g., 30 km, ideally more than 600 km as indicated in widely used ESDU models [6]) somewhat smaller mean wind velocities and greater turbulence intensities are measured.

As anticipated, the tests revealed a strong variation of wind and turbulence parameters with wind direction.

Turbulence intensities vary between 7% and 55% at the deck level. Assessed 100 year return period mean wind velocities perpendicular to the bridge vary between 34 m/s to 60 m/s in the North-East wind sector.

High mean velocity is related to low turbulence, implying that the 3 s. gust wind velocities are more steady at 70 m/s.

For comparison, the aforementioned tender stage assumption with I_u = 18% corresponded to gust wind velocity 66 m/s.

Mean wind velocity has a vertical slope between -11° to +3° at the deck level.

A negative sign indicates that the wind is bent downwards from its direction. These inclinations are so great, that they needed to be taken into account as a source of increased horizontal wind load and in VIV analysis.

Measured time histories of wind velocities also allowed the extraction of turbulence length scales to be used in custom turbulence models in 3D buffeting analysis.

The effect of this on buffeting results was found to be small, and this possibility was not systematically employed.

3.2 Section model tests and analysis

In the next parts of the test programme, section-model tests, see Figure 2, were conducted to obtain aerodynamic input for 3D-buffeting, 3D-VIV and ESWL extraction.

Two deck options were studied, as shown in Figure 3, and the one with full wind nosing was adopted in design due to lower drag force at zero angle of attack.

The scale models were designed and manufactured by FORCE Technology using conventional methods.

The approach used for VIV analysis was to use the mathematical model [2].

In the model, the basic quantity to extract in testing is the RMS-exciting coefficient (known also as lateral force coefficient).

Based on the findings of the topography tests, the tests were conducted +4°, -13° and 0° angles of attack, with +4° being governing for design.

RMS-exciting coefficients are generally determined by measuring the response of an aeroelastic section model at varying flow speeds.

In this method, the structural damping or the effective mass of the scale model does not need to be similar to the real bridge.

Low damping below the design assumption is preferred as it makes the VIV response well-defined and the exciting forces well-correlated along the length of the scale models.

Assessment of the 3D bridge response is made analytically by taking into actual vibration mode shapes and assumed amplitude-dependent correlation of forces along the bridge deck.

Apart from other bridge types the method is well suited to arch bridges, as the mass of the arch affects the effective mass (and Scruton number thereof) involved in deck VIV.

Obviously for this reason VIV of the deck was not found problematic.

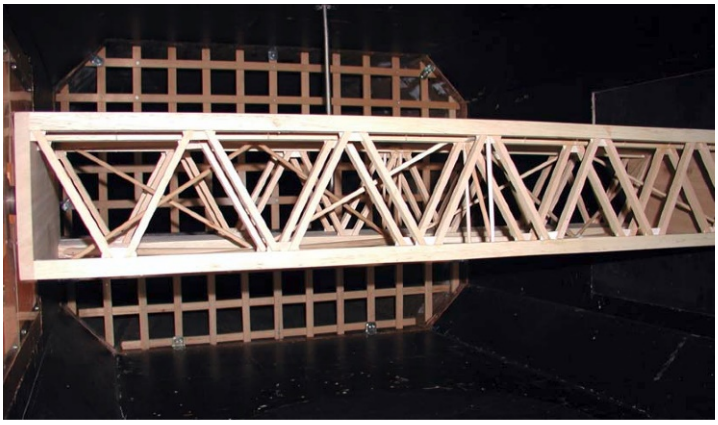


Figure 2: Section models of the deck and arch

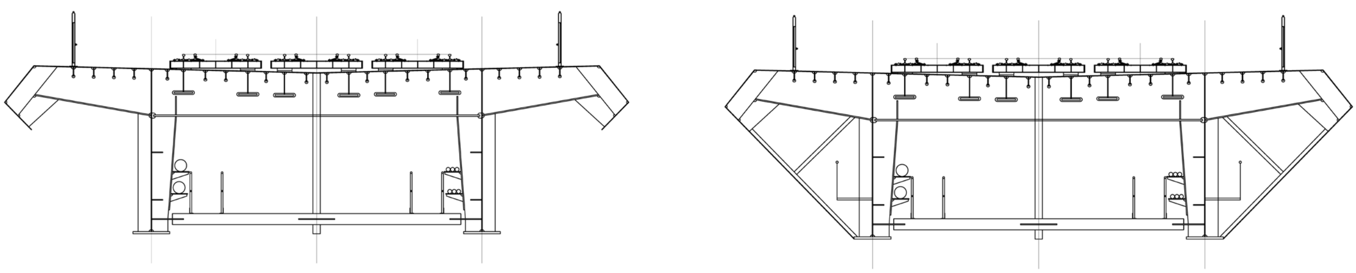


Figure 3: Design variants at the deck section-model tests

The procedural disadvantage of the method is that tests do not approximate the VIV response directly, and a similar type of post-calculation is needed compared to 3D-buffeting analysis.

3D VIV-analysis methods are not as widely and frequently employed as the buffeting analysis methods.

3.3 Full model testing

In the Chanab Bridge wind engineering, the full aeroelastic model testing, Figure 4, was made mainly to confirm the appropriate aerodynamic performance of the bridge in line with the original tender stage client specifications.

The scale model was designed and manufactured by the FORCE technology utilizing 3D printing technologies in a geometric scale of 1:250.

The nearby hills were partly modelled taking into account the local turbulence and deviations in mean wind-velocity vertical angles.

Six wind directions were tested with their vertical inclination of mean wind velocity varying in the range of -11° to +3°. Testing was conducted with and without trains on the bridge. Responses of the scale model were measured with three accelerometers and one strain gauge at the deck.

Results were reported for the mean lateral deflection and standard deviation vibration responses. They allowed comparison with semi-empirical analysis results.

Deviations in the buffeting results were bound to be 20% which is evidently less than implied by uncertainties in full-model testing technology.

VIV response was not observed in the full model, probably due to the mitigation effect of turbulence.

Although this testing type is generally considered technically demanding and time-consuming, its merit is that it brings the results to the attention of various project parties without the need for mathematical post-analysis.

4. DESIGN CHANGES AND CONSTRUCTION PHASE

After the project was paused for a few years, the redesign started in 2013 with some modifications, including the main span length change from 460 m to 467 m.

The changes were reviewed and their effects could be handled by updating the semi-empirical assessments, i.e., without repeating the wind tunnel tests.

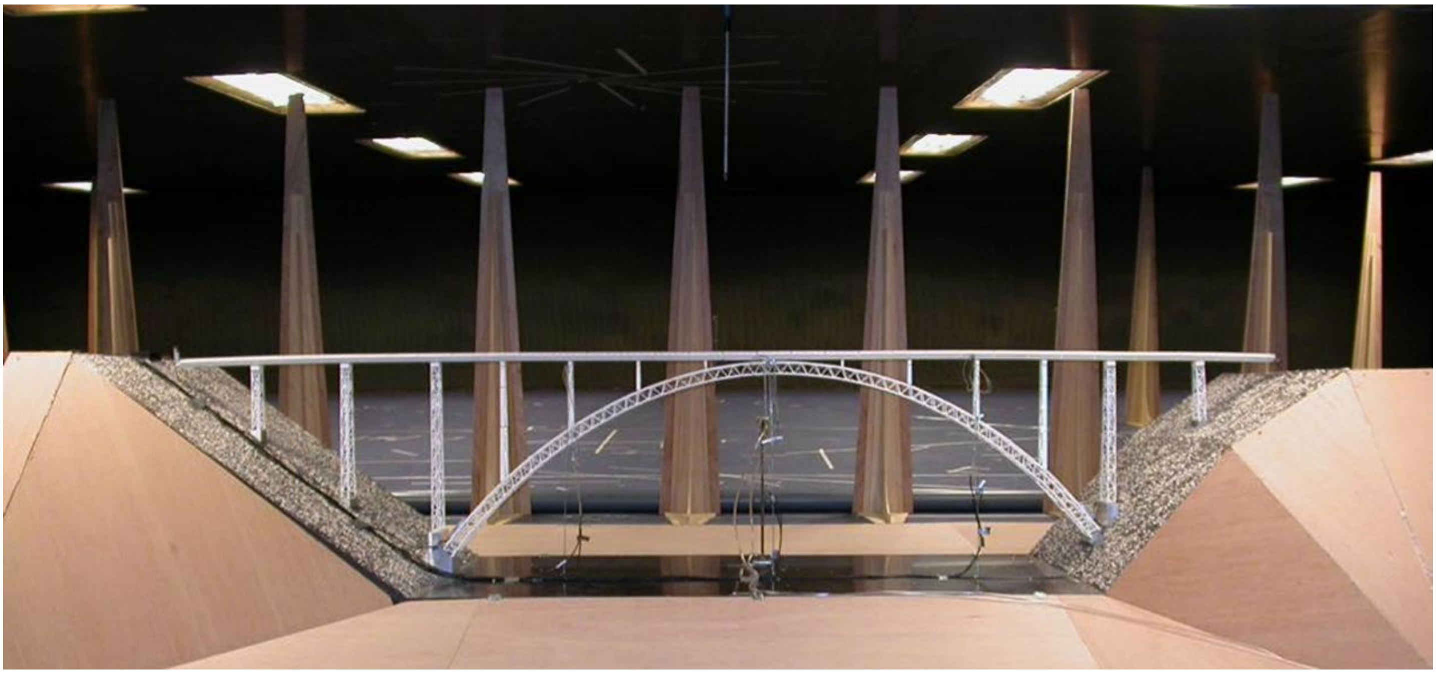


Figure 4: Full aeroelastic model of the bridge

The final version of the wind engineering design report with ESWLs was issued in 2014.

During the construction works, miscellaneous wind engineering issues were handled. These included local VIV risks of steel columns and the effect of access path structures on the wind load of the arch.

5. CONCLUSION

Wind engineering in the Chenab Bridge has been technically and procedurally fascinating, especially for assessing the effects of hilly topography on wind and turbulence parameters, and for bringing the results to design, which has been strongly based on approved design codes.

During the long design period and design changes, the project has demonstrated the advantages of the semi-empirical approach and ESWL procedure.

Bridge design and its design basis have progressively developed into the final stage.

During this process, the main wind engineering design report was updated four times in ten years, but the wind tunnel testing was conducted only once - at the beginning of the project.

It could be restated that this approach requires wind engineering experience not only in the designer's team but also in the proof consultant's team and the client's expert board.

Once the wind warning system of the Chenab Bridge is in use, and the wind speeds are continuously measured at the top of the main span, it would be technically interesting to compare maximum wind speeds to design assumptions.

The assumed return period for construction-stage wind load is 10 years. Wind speeds at the site have already been monitored in other locations for years, and apparently, no major storms have hit the site.

The present revisit of the topography wind tunnel test results suggests that the highest wind velocities may be expected when the wind direction is along the river canyon (North-East or South-West).

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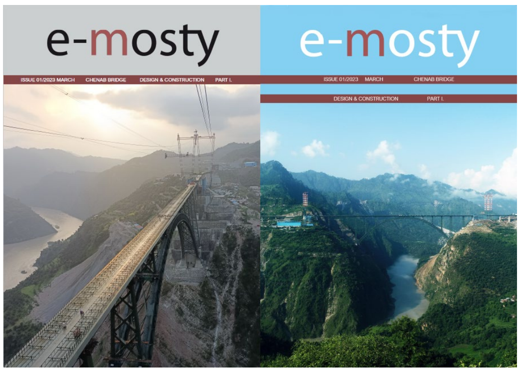

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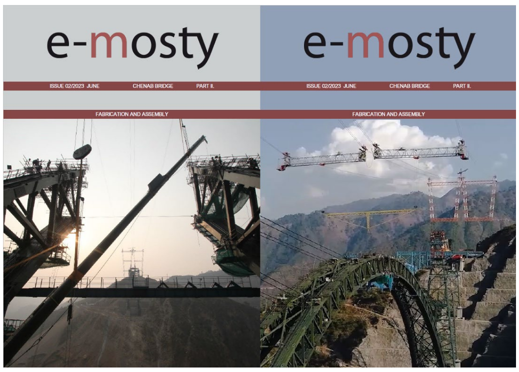


EAST LAKE BRIDGE IN CHINA: OVERCOMING NEW BIM STANDARDS

Zeljka Devedzic

Team Lead Sales Enablement & Consulting Infrastructure
ALLPLAN

Figure 1: Rendering of the East Lake Bridge in China

Xingtai Transportation Construction Group Co. Ltd is no stranger to incorporating advanced infrastructure design tools into its projects.

In their recent project, the East Lake Bridge in Handan, China, they opted to use Allplan's cutting-edge bridge design software.

The East Lake Bridge, a stunning double-deck, cable-stayed composite bridge with a main span of 150 meters, showcases Xingtai's ingenuity and skill.

The intricacies of this modern marvel of engineering, the complex process behind its construction, and the invaluable role of Allplan and BIM in bringing the East Lake Bridge from conception to completion make an interesting read.

The project background is rooted in the shift towards digital transformation in infrastructure design.

Since 2021, when China introduced a unified standard for highway engineering information models, the importance of accurate and comprehensive BIM models has been at the forefront of design and construction methodology.

These unified standards not only emphasized accuracy but also defined its applications at every stage, from preliminary design and construction drawing to the actual construction process and final acceptance.

This new mandate necessitated the development of BIM models that met an LOD of L4.0 according to the new standard, which is a model suitable for construction. Xingtai embraced these standards and chose Allplan for their design tool.

Allplan enabled Xingtai to create a high-precision BIM model that accurately represented every facet of the bridge, from its corrugated steel web composite sections to its double tower cable stayed structure.

This approach allowed for an interactive, highly detailed, and accurate model that could be easily manipulated and analyzed, providing invaluable insights at each phase of the project.

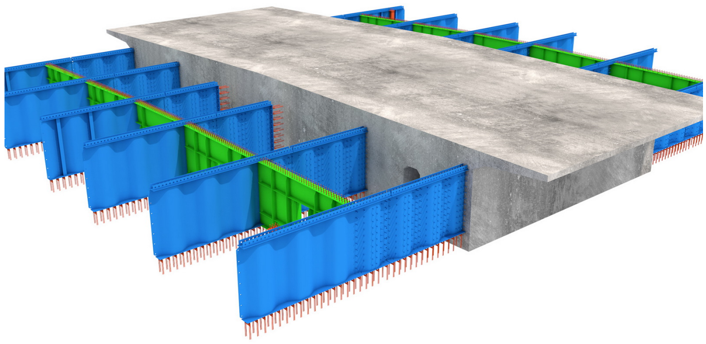
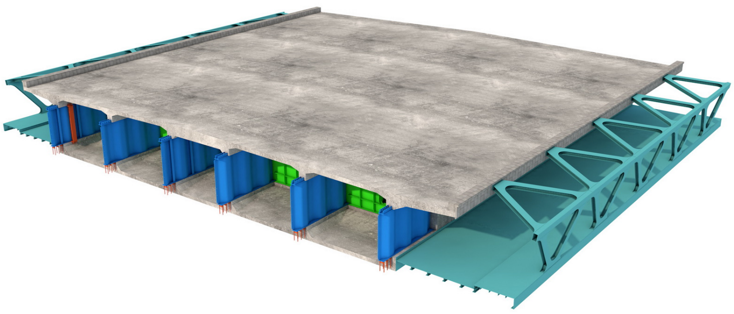
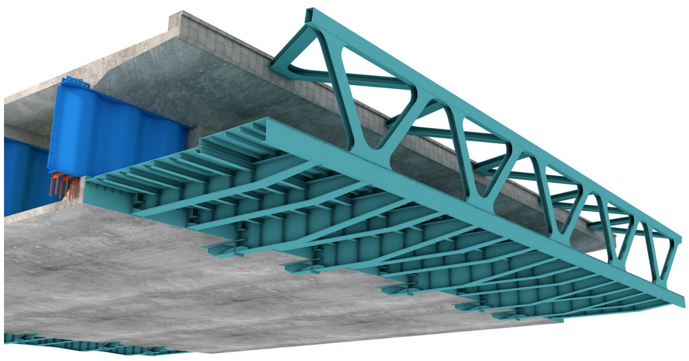


Figure 2: Concrete girder and corrugated steel web

From the foundation's drilled piles to the installation of pedestrian passage components, the BIM model guided each step.

Furthermore, this detailed digital representation made it possible to pre-empt potential challenges, optimize resource allocation, and execute complex construction processes such as the four-stage lower pylon pour and the nine-segment upper pylon pour.





Figures 3 and 4: Modelling of pedestrian passage using Python part

A COMPLEX CHALLENGE

The East Lake Bridge project presented several intricate design and construction challenges, all of which required precise planning, complex detailing, and rigorous analysis to overcome.

One of the initial hurdles was the detailed reinforcement modelling for both the upper pylon and the composite girder decks. Precise simulation was required to estimate the construction feasibility and the exact quantity of rebars needed.

With Allplan's powerful reinforced concrete modelling tools, a detailed reinforcement model was constructed, which allowed for an accurate evaluation of construction feasibility and calculation of rebar amounts.

Allplan's parametrized modelling function, based on the alignment route and bridge cross-section, enabled quick location and updates to the cross-sections and bridge piers based on route images.

This functionality proved essential in modelling the connection between the bridge deck reinforcement and the steel web and in considering the reinforcements in the steel anchor box.

The software was also used to model shear nails for the bridge deck and connection keys at the edge of the web.

With this, it was much easier to plan, model, and visualize the layout of reinforcement and pre-stressing tendon holes in the steel structures.

A safety analysis of the large temporary structures required for the upper pylon templates was performed through detailed simulation and design optimization.

Despite the precision of initial design processes, several unanticipated details for the lower pylon were discovered.

Allplan allowed for precise simulation and correction of these details, ensuring the smooth progress of construction.

Even elements like the bridge deck hanging baskets, disk-type scaffolding, and rigid structural framework during pylon construction were simulated and modelled accurately.

A crucial component of this project was the modelling of repetitive and standardized components, such as corrugated steel webs, steel structures for pedestrian walkways, truss decorative structures, connection components between the main tower and steel cables, and steel templates.

Through secondary development based on Allplan, the efficiency and accuracy of modelling these components were significantly improved.

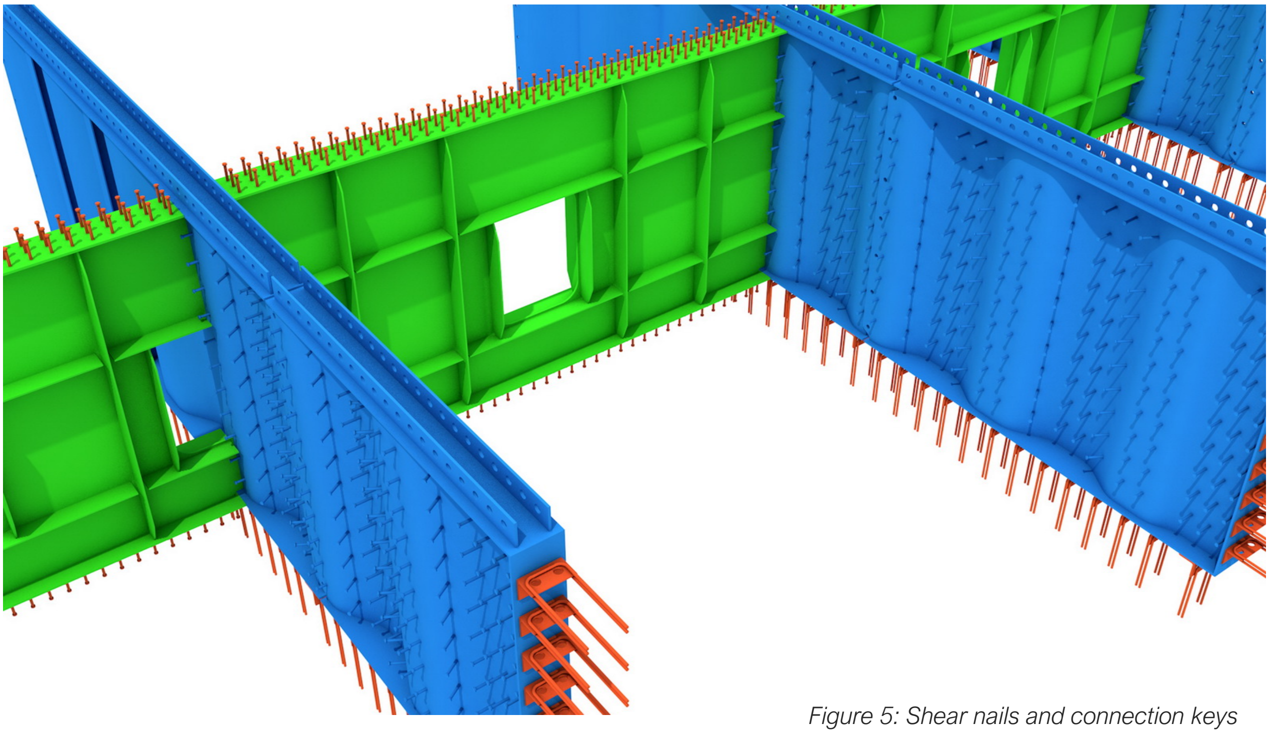


Figure 5: Shear nails and connection keys on corrugated steel web

INTELLIGENT PLANNING FOR ROBUST RESULTS

The team was also able to convert Allplan data into 3D-printed large-scale models of the bridge and other components, providing a tangible representation of the project.

Furthermore, it was possible to convert Allplan models into not only truss beam elements but also frame and solid elements, facilitating complex stress structures or hydration heat analysis for mass concrete.

As part of the intelligent construction approach, Allplan was combined with other technologies for the intelligent processing of reinforcements, including parametric models, segmentation of bridge components for easy assembly, and automatic optimization of reinforcement cutting.

All these processes were automatically transferred to the MES system, ensuring accuracy and efficiency in processing and installation.

Despite the challenges encountered during the East Lake Bridge project, the powerful modelling functions, convenient secondary development capabilities, and open data compatibility of Allplan contributed to the successful application of intelligent construction techniques and the smooth completion of this project.

The East Lake Bridge project sets a new benchmark for infrastructure design in China and, at the same time, it is a testament to precision, innovation, and intelligent construction.

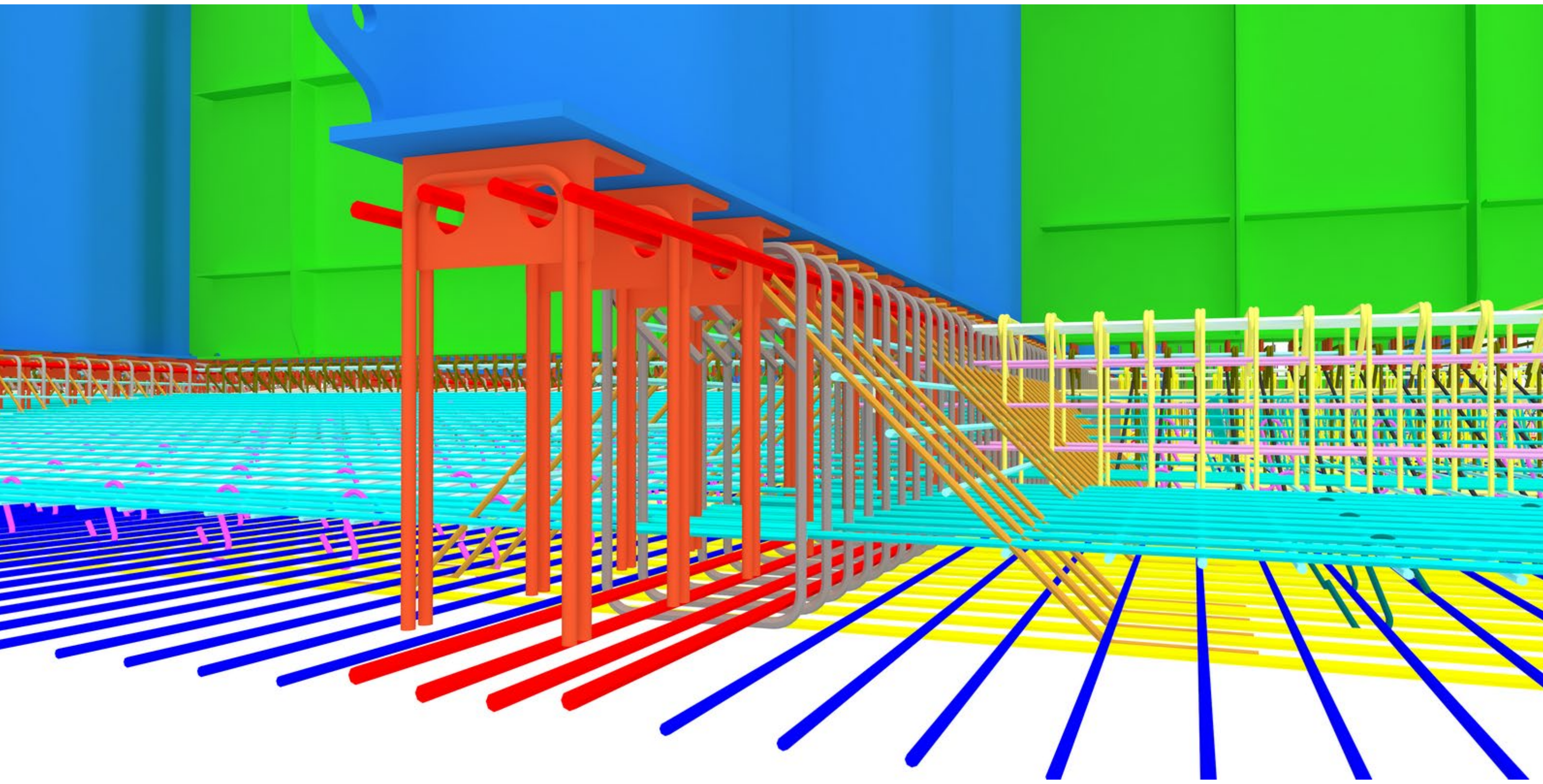
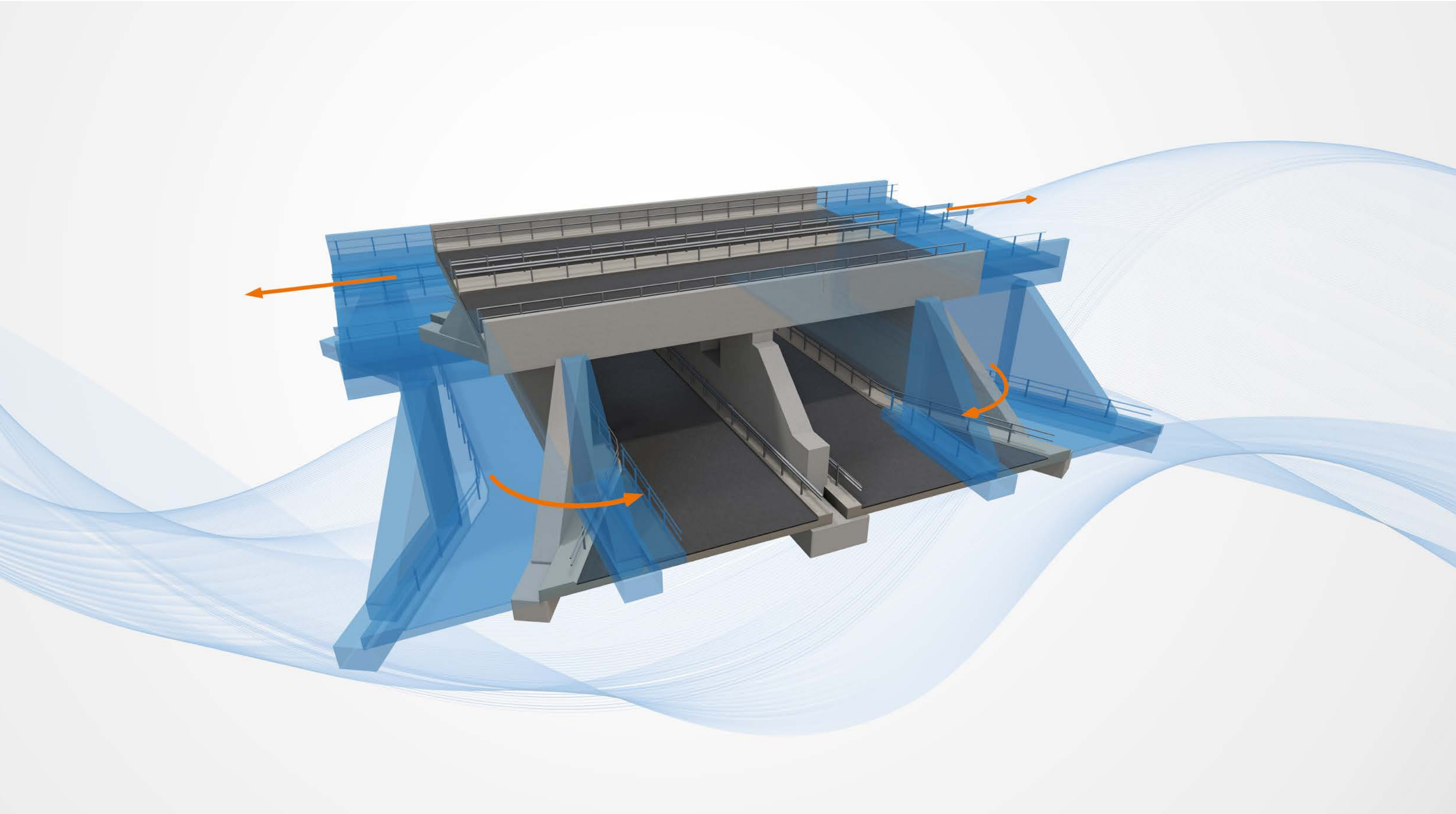


Figure 6: Details of connection keys and reinforcement



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

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