

e-mosty

ISSUE 03/2026

SEPTEMBER

Sabiñánigo Viaduct

Spain



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INTERNATIONAL ONLINE PEER-REVIEWED MAGAZINE
ABOUT BRIDGES

It is published at www.e-mosty.cz. Open Access.

Released quarterly:

20 March, 20 June, 20 September and 20 December

Number: 03/2026, September Year: XII.

Chief Editor: Magdaléna Sobotková, MSc.
Contact: magda@e-mosty.cz

Editorial Board

The Publisher: BRIDGES ONLINE, s. r. o. (Ltd.)
Velká Hraštice 112, 262 03 Czech Republic
VAT Id. Number: CZ02577933

E-MOSTY ISSN 2336-8179

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Dear Readers,

*The first two articles focus on the design and construction of the **Sabiñánigo Viaduct in Spain**, whose construction set several engineering records.*

***TYLin** presents the main design aspects, the study of alternatives, deck design and modelling, as well as the construction process, which was decisive for the structure's conception and typology.*

*The article by **TecoZam** focuses on the stability of the Movable Scaffolding System (MSS) during launching operations. It is the **first in a series to be published in the e-mosty & e-BrIM magazines throughout 2026/27**, sharing key lessons learned during the execution of this outstanding structure.*

***SPECIÁLTERV** presents the **Robinson Bridge in Budapest, Hungary**, demonstrating how a 168 m curved deck can cross the river without any fixed intermediate support. The bridge has received international recognition and numerous awards.*

*The next article explores the **evolution of the precast segmental bridge** as part of the Fast Lanes Shfaim Project, showing how structural design and construction methodology can be integrated within a constrained transport environment.*

*Similar challenges were addressed in the **400 m section of cantonal road K2b above Lake Lucerne**, where narrow alignments, sensitive landscapes and the need to keep traffic moving required structural design and modelling to be considered together. The team used **ALLPLAN Civil** to create a parametric model, highlighting the link between structural choices and modelling capabilities.*

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*I would like to thank our **Editorial Board** for their valuable assistance with this edition and for reviewing the articles, with special thanks to **Juan C. Gray**. We are also grateful to **our partners** for their continued support.*

*On behalf of the organisers, we invite you to the **2026 World Bridge Engineering Conference**, with an emphasis on Innovative Bridge Technologies and Accelerated Bridge Construction, which will take place on **1st and 2nd December 2026 in Miami, Florida, USA**. More information is on page 10.*

*The next e-mosty will be published on 20th December, and the next e-BrIM will be released on 20th October, in both **English** and **Spanish**.*

We welcome your articles for both the e-BrIM and e-mosty magazines. You can contact me at magda@e-mosty.cz.

Magdaléna Sobotková

Chief Editor



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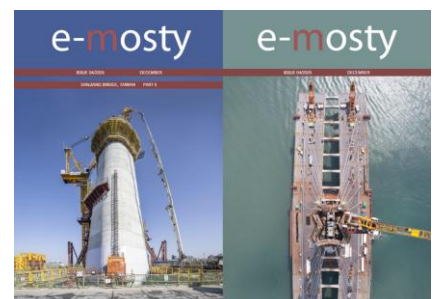
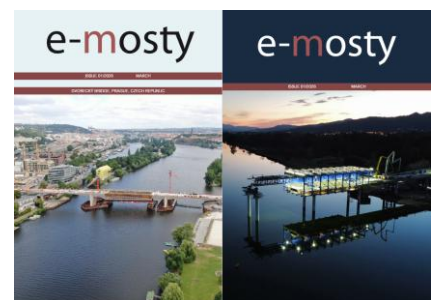
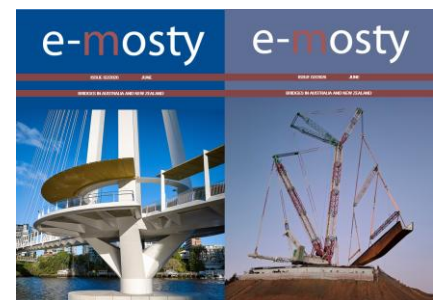
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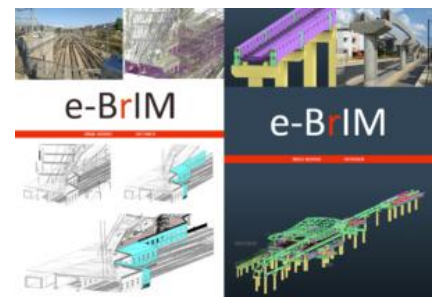
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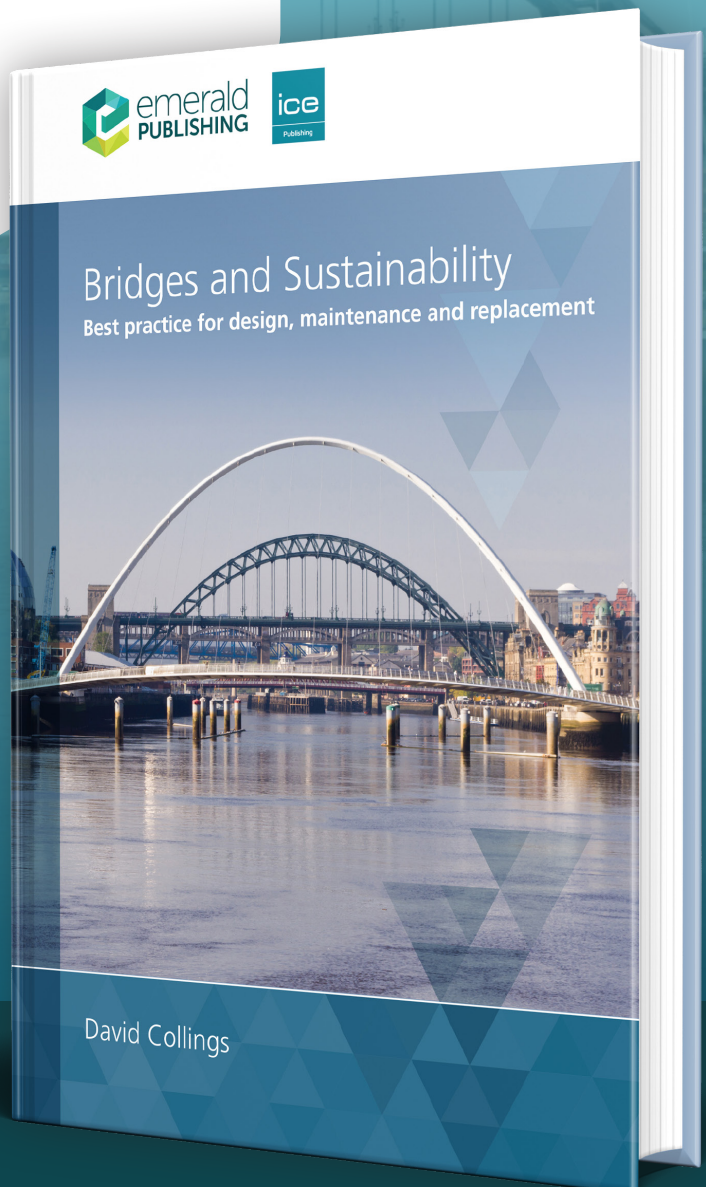
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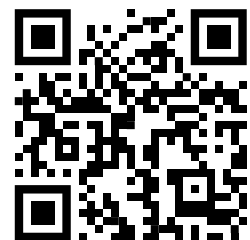
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THE SABIÑÁNIGO VIADUCT IN SPAIN

THE LARGEST SPAN EXECUTED WITH AN MSS

Miguel Ortega, Global Technical Director, TYLin Europe-LATAM, Spain

Fernando Ruano, Bridge Department Director; Jesús Martín, Inspection, Maintenance, and Construction Management Department Director; Jokin Ugarte, Senior Project Manager; David Alcañiz, Senior Project Manager, Bridge Department, TYLin, Spain



Figure 1: Bottom view of the viaduct with a foreshortened view from pier P1

I. INTRODUCTION

The Sabiñánigo viaduct over the Gállego and the Aurín rivers is the main structure of the Sabiñánigo East-Sabiñánigo West section of the A23 highway. The section was awarded to the CJV formed by Rover Infraestructuras and Aldesa Construcciones in September 2019. The UTE contracted TYLin (then IDEAM) to carry out technical advisory work, commissioning it to draft a Detailed Design of the viaduct at the beginning of 2020.

The drafting of the new project was carried out during the central months of 2020. In 2021, the CJV contracted the Tecozam Group to supply the auxiliary equipment for the construction of the

viaduct (movable scaffolding system and wing traveller), at which time TYLin adapted the Detailed Design to the needs of the auxiliary equipment finally chosen. Construction of the viaduct began in 2022 and was fully completed by autumn 2025. TYLin provided technical assistance to the joint venture throughout the construction process.

II. MAIN DESIGN ASPECTS

Conditioning factors

The viaduct layout is strongly conditioned by the local orography. In the vicinity of Sabiñánigo, the highway runs along a plateau located approximately 60 m above the urban area, which ends abruptly at

a steep slope shaped by the erosive action of the Gállego River, descending to the riverbed level (Figure 3). After crossing the Gállego River, the terrain between this river and its tributary, the Aurín River, is essentially flat. Along this stretch, the highway progressively loses elevation, reaching the lower ground level immediately after crossing the Aurín River.

As a result of these topographical conditions, the viaduct had to extend from the slope adjacent to the Gállego River in order to overpass both the main river and its tributary. This led to a total bridge length of approximately 900 m, with pier heights varying significantly, from about 60 m near the plateau to around 10 m in the lower areas.

In addition to its considerable length, one of the main constraints of the viaduct was the presence of the steep slope and the Gállego River itself. Together, these elements defined a zone of approximately 90 m where the construction of intermediate piers was not feasible, thereby imposing a first span of 90 m in length.

Regarding the construction methodology, the overall length of the structure made it advisable to adopt execution methods suitable for span-by-span construction, such as the erection of complete spans using cranes or the use of movable scaffolding systems.

Less efficient solutions, such as balanced cantilever construction, were intentionally limited.

Conventional falsework was only considered viable in the lowest areas of the viaduct; however, its combination with other construction systems — particularly movable scaffolding, whose installation cost represents a significant proportion of the total cost — did not offer sufficient advantages and was therefore discarded in the early stages of the design process.

Another key conditioning factor was the cross-sectional configuration of the bridge deck. The platform comprises two 3.50 m traffic lanes per carriageway, two shoulders, including an outer shoulder of 2.50 m and an inner shoulder of 2.00 m, resulting in a total deck width of 25.1 m for all considered alternatives.

The viaduct also presents a double horizontal curvature, which added geometric complexity to the design but did not significantly condition the adopted structural solution.



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

Study of alternatives

Work for the joint venture began with an alternatives study evaluating different materials, construction methods, and structural configurations in order to identify the optimal solution for the site. Three main alternatives were assessed.

The first consisted of a composite single-cell box girder with lateral steel struts, similar to that previously developed by IDEAM for the nearby Sigüés Viaduct on the same motorway [1], which presented comparable geometric and construction conditions. The second alternative was based on a prestressed concrete single-cell box girder combining two construction methods: a conventional movable scaffolding system (MSS) with 64 m spans for the section between the Gállego and Aurín rivers, and balanced cantilever construction to execute spans 1 and 2, up to 90 m in length, in order to cross the Gállego River. The third alternative proposed a prestressed concrete single-cell box girder constructed exclusively using a special MSS, allowing for 90 m spans throughout the viaduct.

After evaluating the technical and economic implications of each solution, the third alternative was identified as the most advantageous. It relied on a single erection system for the continuous construction of the central box girder from one abutment to the other, avoiding the inefficiencies associated with combining different construction methods. In addition, span-by-span construction using an MSS offered highly competitive production rates. Given the overall length of the viaduct, the investment required for a record-length MSS could be economically justified.

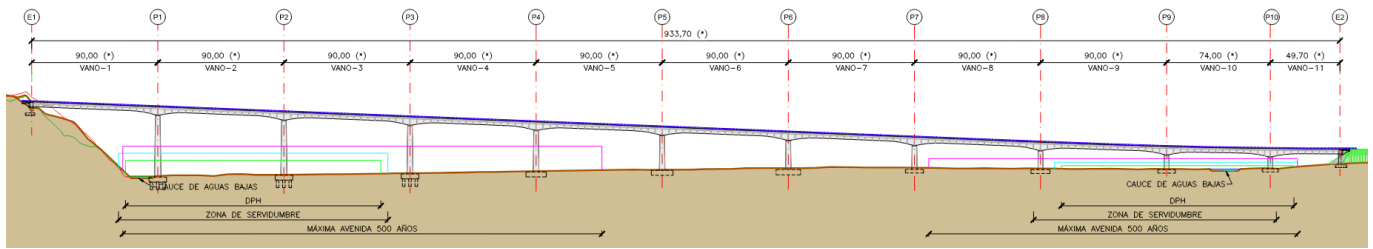


Figure 3: Elevation of the Sabiñánigo Viaduct built with MSS with typical spans of 90 m

It should be noted that MSS equipment capable of constructing spans of this magnitude requires substantial adaptation for each project.

At the time of the study, MSS technology was reaching the threshold for 90 m spans [2]; see Figure 4. The design team had previously participated in the access viaducts to the New Pumarejo Bridge [3], where a large MSS was used to construct a 16 m wide box girder with 70 m spans and a unit weight of 38 t/m. Concurrently, the company BERD had completed four high-speed railway viaducts on the Ankara–Sivas line with 90 m spans and a deck weight of 32 t/m [4].

Following consultations with several MSS suppliers and in coordination with the joint venture, the third alternative was selected. Since the MSS design was governed by the first span (85–90 m), it was advantageous to standardise the remaining spans to 90 m, thereby reducing the number of piers and foundations compared to the other options.

During the drafting of the Modified Project, Project Management recommended increasing the distance between abutment 1 and the steep slope of the Gállego River, ultimately fixing the first span at 90 m. The resulting layout comprised 11 spans arranged as $9 \times 90 + 74 + 49.7$ m.

The MSS was designed to construct the 11 m wide central box girder, with a unit weight of 32 t/m, advancing from one abutment to the other, equalling at the time the world record for the longest span executed using this method [4]. The deck cantilever wings were subsequently constructed using a wing traveller advancing two to three spans behind the MSS. The two shorter spans (74 m and 49.7 m) allowed an appropriate geometric adjustment of the pier locations relative to the Aurín River, ensuring a smooth transition from the typical 90 m spans to abutment E2.

Achieving a 90 m span required rigorous weight optimisation of the deck. A variable-depth box girder was therefore adopted, with a constant depth of 3.20 m over the central 50 m of each span and a parabolic depth increase over the 20 m adjacent to each pier, reaching a maximum depth of 5.50 m at the supports. This configuration allowed a large central region of constant depth, facilitating formwork standardisation and improving construction efficiency with the MSS. A similar concept had previously been implemented by the design team in the Deba River Viaduct in Bergara [5], with an 80 m span, which was the longest MSS span in Spain prior to the completion of the Sabiñánigo Viaduct.

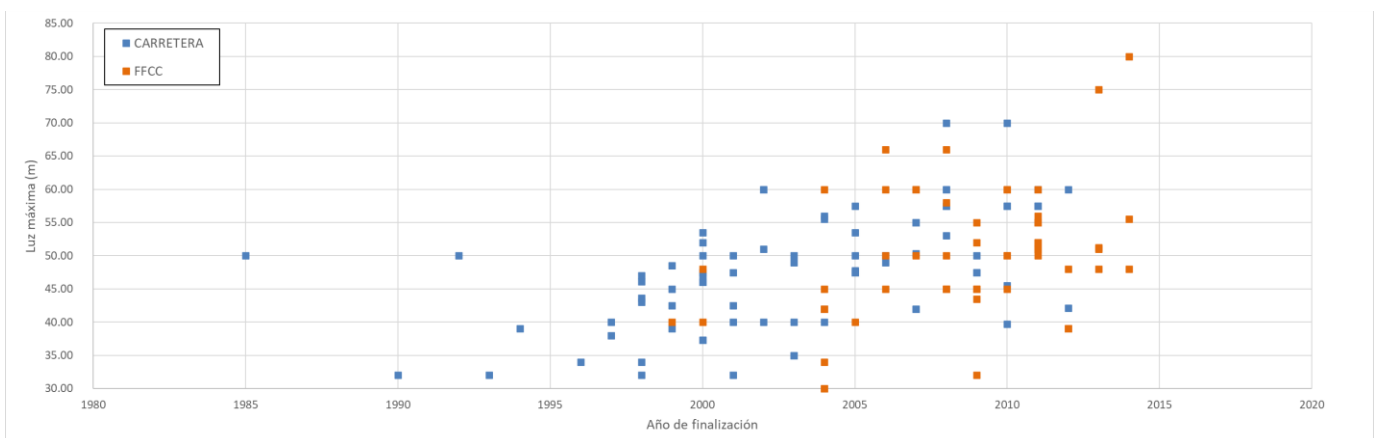


Figure 4: Evolution of maximum span in decks constructed using self-launching formwork in Spain, both on roads and high-speed railways (information up to 2020)



Figure 5: Left: Bottom view of the depth variation. Right: MSS during concreting of the upper slab of a typical span of 90 m

III. DECK DESIGN

Cross-section and longitudinal depth variation

As previously indicated, the structural solution adopted for the Sabiñánigo Viaduct over the Gállego and Aurín rivers consists of a single-cell prestressed concrete box girder. The box section is 9 m wide at the top slab and 8 m wide at the bottom slab, with slightly inclined webs to facilitate formwork stripping. The minimum web thickness was set at 60 cm and locally increased to 70 cm over the 10 m adjacent to each pier to accommodate higher stress demands.

The upper slab between the webs was designed with a constant thickness of 30 cm, which was locally increased to 60 cm above the webs through a linear transition 1.50 m in width. The bottom slab was also defined with a constant thickness of 30 cm, incorporating 1 m-wide chamfers where the thickness increased to 65 cm, see Figure 6. In the 20 m adjacent to each pier, the bottom slab thickness was further increased linearly, reaching a maximum value of 80 cm at the supports.

Outside the webs, the top slab was designed with a reduced thickness of 25 cm. This slender configuration was made possible by preventing the outward deviation of the parabolic longitudinal prestressing tendons. The 8 m long cantilevered wings were supported by an edge beam braced by prefabricated struts spaced every 5 m, which transferred the cantilever loads directly to the junction between the webs and the bottom slab.

The upper slab was designed as a reinforced concrete element, avoiding the use of transverse prestressing tendons.

This decision prevented potential interference with the longitudinal prestressing layout and eliminated the need for an additional construction stage during deck wing execution, thereby simplifying the construction process and improving the overall production rate.

The box girder was designed with a midspan structural depth of 3.20 m, corresponding to a slenderness ratio of $L/28.1$. This depth was maintained constant over the central 50 m of each span, enabling the repetition of reinforcement layouts, prefabrication and assembly modules, and the movable scaffolding system formwork. This approach maximised the level of industrialisation and efficiency of the construction process.

Over the 20 m adjacent to each pier, the girder depth was increased following a parabolic profile, reaching a maximum depth of 5.50 m at the supports, corresponding to a slenderness ratio of $L/16.4$. Within these variable-depth regions, the prefabricated struts were designed with varying lengths and inclinations so that they consistently connected to the intersection between the webs and the bottom slab. This arrangement prevented the struts from intersecting the webs at mid-height, thereby avoiding the need for intermediate transverse bracing elements that could have compromised or complicated internal access within the box girder.

As a result of this geometric configuration, the variation in girder depth caused the struts to define a warped surface (Figures 1 and 5), providing a dynamic appearance when viewed from below, while simultaneously ensuring an efficient structural load transfer and optimal structural performance.

Longitudinal prestressing

Two main families of longitudinal prestressing were defined for the typical spans. The primary family consisted of ten parabolic tendons per web, each formed by 31 Ø0.6" strands and arranged in paired groups. In both the midspan and pier regions, a plan-view deviation was introduced for two of the five tendon pairs in order to maximise prestressing eccentricity and improve structural efficiency (Figure 8).

In addition to this main prestressing system, a second family of negative reinforcement tendons was provided over the piers. This family comprised six straight tendons of 27 Ø0.6" strands, also grouped in pairs, running exclusively within the upper slab. The first tendon pair extended 25 m on each side of the pier, while the remaining two pairs were limited to 15 m on each side.

Owing to the optimised layout and profile of the parabolic prestressing tendons, it was possible to eliminate the need for an additional family of positive reinforcement tendons in the bottom slab within the central regions of the typical spans. This third family was only required in span 1, where the significant imbalance of the end span, which had the same span length as the other spans, resulted in a structural behaviour that was clearly different from that of the other spans, which ultimately justified its inclusion. Three pairs of straight 27 Ø0.6" tendons, each 53 m long, were installed in this span, together with three additional pairs of straight 31 Ø0.6" tendons with a length of 37 m.

To enhance the robustness of the structural design, the project also included the provision of two auxiliary ducts in the central region of the remaining spans.

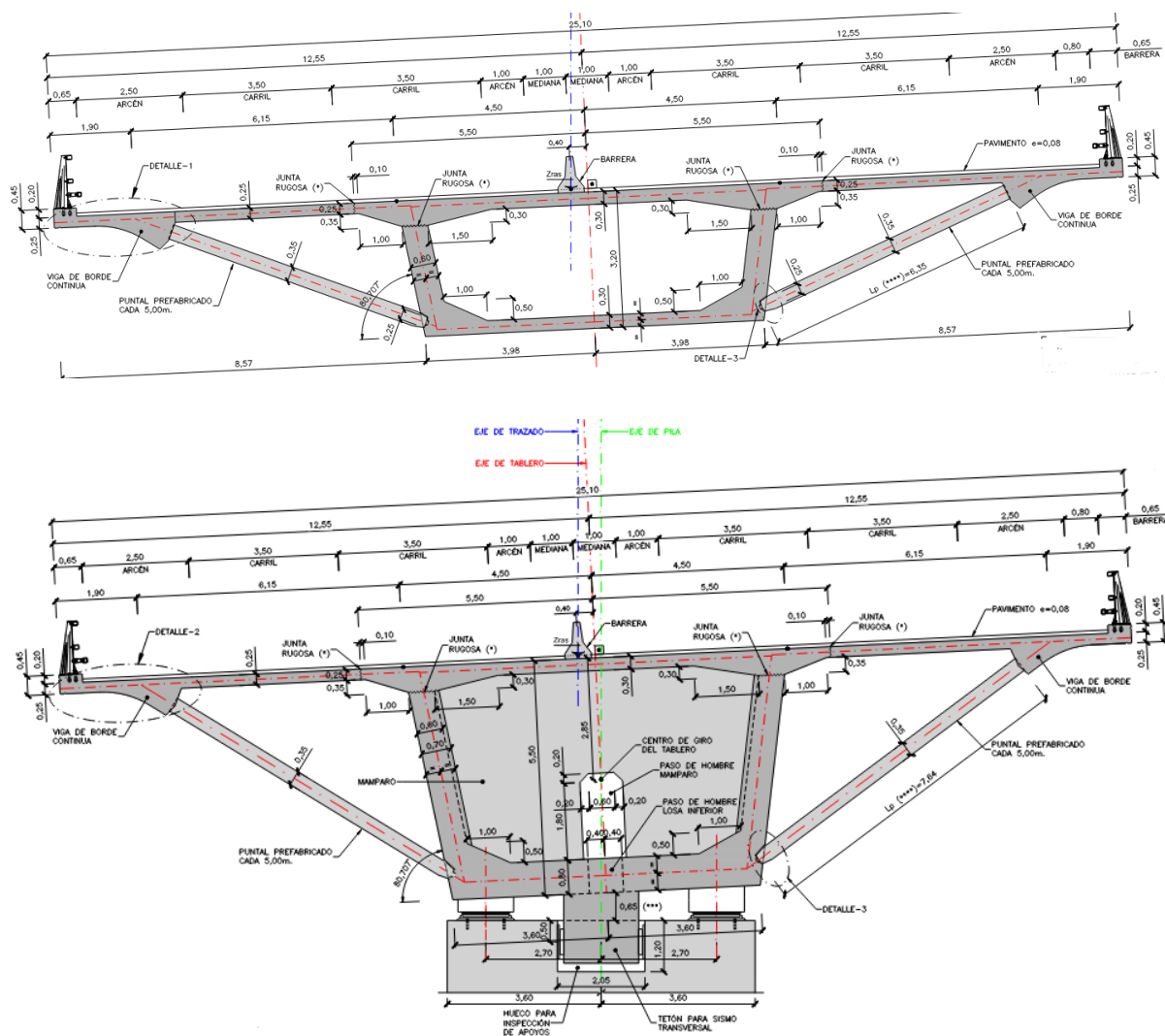


Figure 6: Top: Cross section in mid-span. Bottom: Cross section over piers



Figure 7: Bottom left and right: Bottom views of the deck that show the warping of the prefabricated strut's plane.
Top right: prefabricated strut assembled on site

These ducts, 53.15 m long in the typical spans and sized to accommodate straight 31 $\varnothing 0.6''$ tendons, were incorporated as a contingency measure. Their purpose was to allow a limited increase in prestressing capacity should any deviations from the predicted structural behaviour be observed during construction.

Beyond the typical span configuration, the longitudinal prestressing of the bridge was divided into five main phases:

- **Phase 1:** Corresponding to the 49.70 m end span, where the parabolic tendons were designed as five pairs per web, each composed of 24 $\varnothing 0.6''$ strands.
- **Phase 2:** Corresponding to the 74 m span, in which the parabolic tendons were arranged as five pairs per web, each consisting of 27 $\varnothing 0.6''$ strands.
- **Phases 3 to 9:** Typical span configuration, as previously described.
- **Phase 10:** Corresponding to span 2 (90 m), which was significantly unbalanced relative to span 1, also 90 m. In this phase, the layout of the five tendon pairs per web, each composed of 31 $\varnothing 0.6''$ strands, had to be adjusted to

accommodate a substantially more negative bending moment envelope than that of the typical span.

- **Phase 11:** Final phase in the construction of the central box girder, corresponding to span 1 (90 m), which, as noted, was highly unbalanced with respect to span 2 and subjected to significantly higher positive bending moments than those governing the typical span design.

Evolutionary Behaviour of the Cross-Section

The inherent complexity of constructing a transverse section in successive construction stages should be highlighted. This technique is common in precast and composite bridges and is increasingly being adopted in long-span, wide post-tensioned concrete bridges.

In such cases, the structural behaviour of the cantilever wings closely resembles that of the concrete slab in a continuous composite viaduct.

The wings are executed at a very late stage in the construction process and therefore exhibit a very low stress level at the completion of the structure. When combined with the distribution of permanent loads, this inevitably results in net tensile stresses over the supports at the time of opening to traffic.

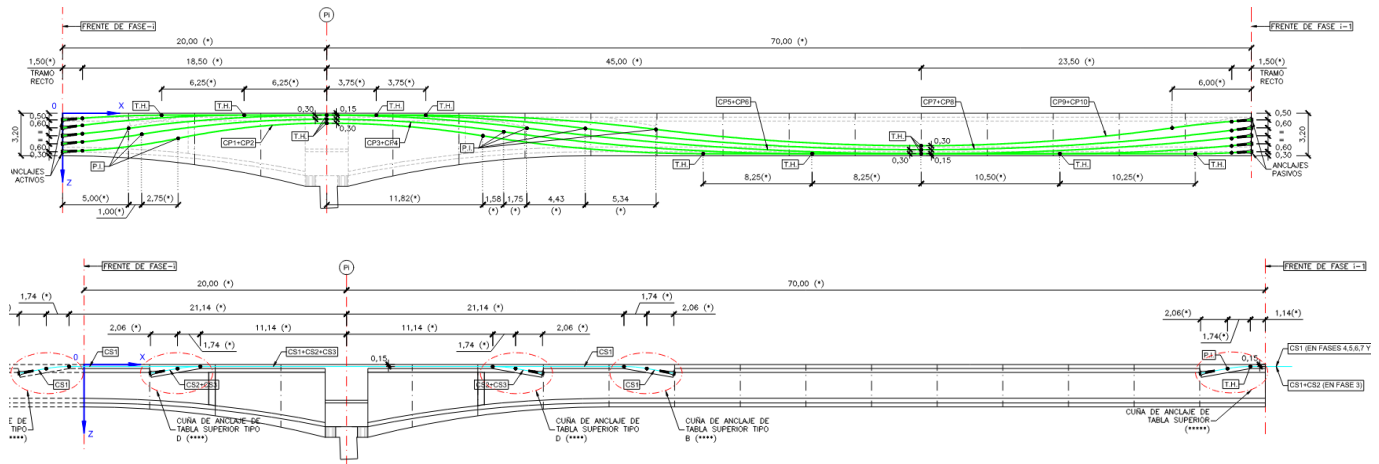


Figure 8: Configuration in elevation of the interior prestressing of one of the typical spans

Moreover, as in composite bridges, the residual shrinkage of the box girder is lower than that of the wings. The resulting differential shrinkage induces longitudinal tensile stresses in the wings along most of the bridge length. Although creep-induced force redistribution between the box and the wings partially mitigates this effect, it remains structurally significant.

In the case of the viaduct over the Gállego and Aurín rivers, all negative reinforcement prestressing was tensioned only after completion of the wings, while the parabolic prestressing was designed to carry the full self-weight of the deck. This strategy introduced an initial compressive state in the wings over the piers, partially compensating for tensile stresses caused by permanent loads. In the positive bending regions, the compression generated by dead loads led to a more favourable stress condition.

Despite these mitigating measures, it is essential to recognise that the wings of a viaduct of this type are inherently subjected to longitudinal tensile stresses. Adequate crack control therefore requires the provision of sufficient longitudinal reinforcement to ensure controlled cracking behaviour throughout the service life of the structure.

IV. SUBSTRUCTURE DESIGN

Longitudinal fixation strategy, seismic arrestors and vertical bearings

The Sabiñánigo Viaduct comprises ten intermediate piers, three of which—P4 to P6—define the longitudinal fixed point of the structure. Due to the considerable self-weight of the deck, spherical

bearings were adopted, and conventional elastomeric bearings were discarded at an early design stage.

The magnitude of the longitudinal forces did not justify a delta-shaped fixed-point configuration. Instead, the deck was fixed directly to the central piers while preserving their optimal vertical geometry. A sensitivity study was carried out to assess the effect of fixing different numbers of piers. Fixing three piers was found to minimise the maximum longitudinal force transmitted to any single pier, as a larger number increased hyperstatic forces induced by imposed longitudinal displacements, while fewer fixed piers led to excessive concentration of braking and longitudinal seismic forces.

Following a detailed assessment with several spherical bearing suppliers, the longitudinal restraint was implemented by locking the longitudinal movement of the bearings themselves. This solution significantly simplified the design of the support diaphragms and pier heads, avoiding complex geometrical arrangements.

Transverse service-level actions, including serviceability seismic forces, were resisted directly by the bearings. For ultimate limit state seismic actions, the bearings were equipped with fuse mechanisms that released the transverse forces beyond a predefined threshold, transferring them to transverse seismic arrestors.

Piers

Pier heights range from approximately 8 m at pier P10 to nearly 44 m at pier P1, following an almost linear variation governed mainly by the longitudinal

gradient of the deck rather than by the underlying topography, which is largely flat in this area. Given the relatively discreet visual role of the substructure within the overall viaduct composition, a sober and functional design approach was adopted to complement the dominant geometry of the deck.

The pier shafts were therefore designed with a hollow rectangular cross-section, incorporating 10 cm chamfers at the edges to soften wall transitions and improve the surface finish. A linear longitudinal taper of 1.5/100 was introduced. The transverse width of the shaft was defined to match the width of the box girder bottom slab at the pier section, reinforcing continuity between superstructure and substructure. This width was kept constant along the height, enhancing the perception of slenderness (Figure 9).

The longitudinal dimension of the pier head was also aligned with the constant-depth zone of the deck support diaphragm, conceiving the pier shafts as a natural continuation of the deck depth variation into the vertical plane and ensuring a smooth transfer of loads to the foundations.

Regarding foundations, piers P1 to P3, located near the Gállego River, were founded on deep foundations with 1.8 m diameter piles, while the remaining piers were supported on shallow foundations, consistent with the more favourable geotechnical conditions.

Abutments

Both abutments, E1 and E2, were designed as closed structures with a total width of 25.1 m. Abutment E1 is founded on deep foundations consisting of 1.8 m diameter drilled shafts and reaches an average height of approximately 12.5 m above foundation level, with a uniform wall thickness of 2.50 m. Abutment E2, by contrast, is supported on shallow foundations, has an average height of about 11 m above foundation level, and incorporates a wall thickness of 2.10 m.

V. MODELLING

Global Model

The global structural model of the bridge was developed in SOFiSTiK using beam elements. The model incorporated the longitudinal vertical alignment of the bridge, the horizontal curvature in plan, and the longitudinal variation of both the structural depth and the deck width.



Figure 9: Longitudinal view of the pier shafts and formal continuity with the deck

All piers were modelled considering their actual heights and the variation of their cross-section along the height. Each pair of prestressing tendons was modelled taking into account both its vertical profile and its horizontal deviations, allowing an accurate evaluation of prestress losses.

A staged construction analysis was performed, accounting for both the construction sequence of the deck and the age at loading of each structural component. The deck cross-section was activated in two phases, reproducing the actual construction process using the movable scaffolding system (MSS) and the wing traveller. Each prestressing pair was stressed according to the sequence defined in the design.

Transverse Model

A three-dimensional finite element model was developed in SOFiSTiK using shell elements over a 30 m deck segment to study the local behaviour of the top slab, internal webs, and prefabricated struts under the load trains defined by the applicable code.

This model enabled a detailed assessment not only of the bending response of the top slab and internal webs under concentrated live loads and other actions, but also of the axial stress state induced by the inclined struts in both the top and bottom slabs.

Local Detail Models

Critical structural components, such as the diaphragms, were analysed using dedicated finite element plate models also developed in SOFiSTiK.

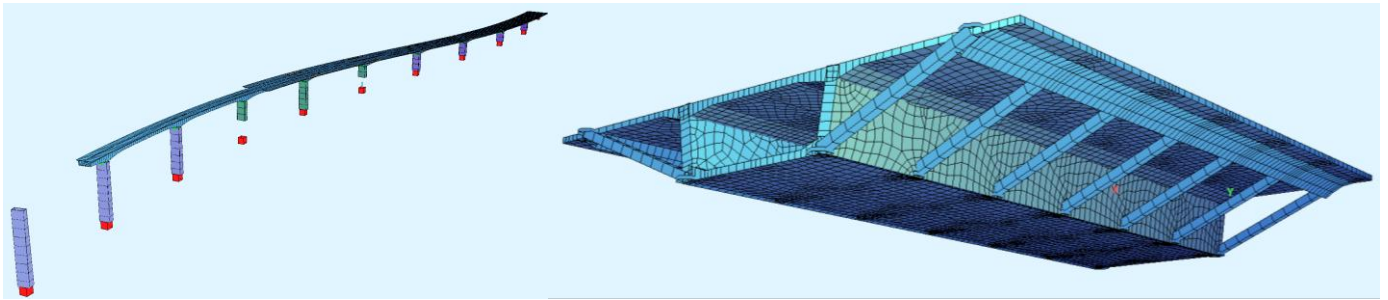


Figure 10: 3D view of the staged construction model of the viaduct

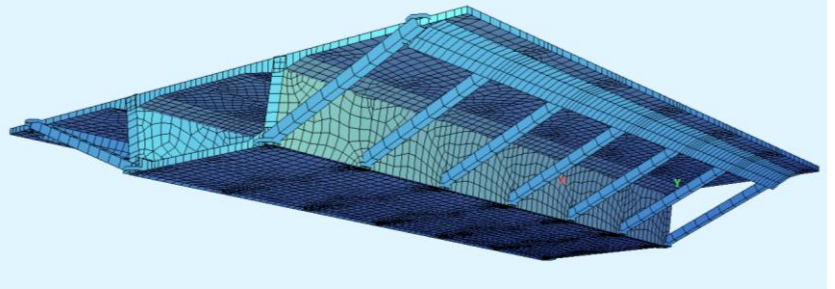


Figure 11: Bottom view of the three-dimensional model used for the local load introduction analysis

These models allowed a detailed understanding of the elastic load paths from the box girder to the bearings, facilitating the calibration and optimisation of the corresponding strut-and-tie models adopted for ultimate limit state design.

VI. CONSTRUCTION PROCESS

Description

As previously described, the construction process of the viaduct is a determining factor in its overall conception and structural typology. The execution of the deck is organised into two clearly differentiated phases. First, the central box girder is constructed span by span using a movable scaffolding system (MSS).

Once the central box has advanced sufficiently, the large cantilever wings are subsequently executed in 15 m long segments using a wing traveller, which operates with a minimum offset of two spans behind the MSS to ensure construction stability and continuity (Figure 13).

In order for the MSS to achieve the required span length of 90 m, the construction of the central box girder is subdivided into several carefully sequenced sub-phases. The process begins with the concreting of a partial U-shaped section of the box girder, comprising the bottom slab and the webs (Figure 13). As is common in bridges executed with an MSS, and with the goal of reducing the duration of the execution cycle, reinforcement was pre-assembled in modules on the ground that are lifted and placed over the scaffolding system. After casting the U-section and once the concrete has reached sufficient strength, two out of the ten pairs of parabolic longitudinal prestressing tendons are stressed, providing the initial structural capacity required for subsequent stages.

The construction then proceeds with the concreting of the top slab between the webs (Figures 16 and 18). At this stage, the self-weight of the newly cast slab is shared between the previously executed

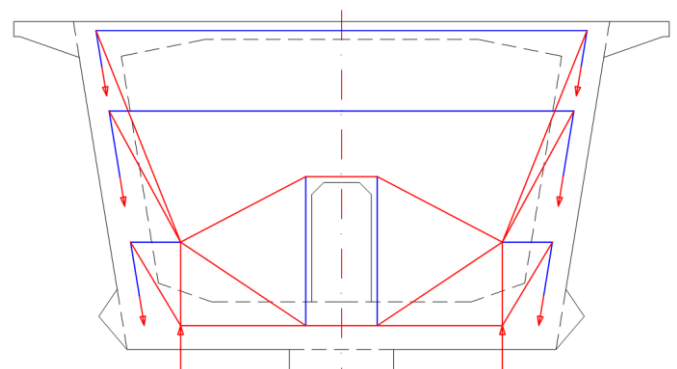
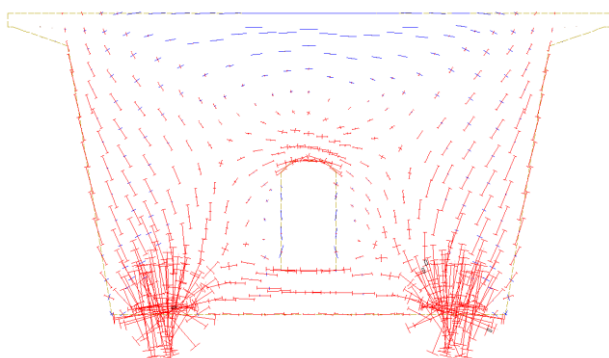


Figure 12: Left: Principal stresses in the elastic finite element model. Right: Corresponding strut-and-tie model for ultimate limit state design

partial U-section and the movable scaffolding system, in proportions governed by their relative stiffness. This construction phase required a detailed assessment of the structural behaviour of the partial U-section, as it temporarily carries a significant portion of the top slab weight and is therefore subjected to elevated stress levels.

Once the upper slab has fully hardened and the composite behaviour of the section has been achieved, the remaining parabolic prestressing tendons of the span are tensioned. The MSS support spindles are then released, allowing the system to be lowered, and the movable scaffolding system is subsequently advanced to the concreting position of the next span, completing the construction cycle for each typical span.

Even though the general erection sequence of the span is the one previously described, other sequences were studied. Particularly, we highlight the option of dividing the execution of the partial section in U into two different sub-phases: one stage consisting in the cast of the bottom slab and, after it hardens, casting the webs of the deck. This alternative was finally carried out in one of the spans (span 2), where the height of the deck and the vicinity of the Gallego River made it difficult to place mobile concrete pumps from the ground and, therefore, it was advisable to reduce concrete volumes placed at once.

The cantilevered wings of the deck section are constructed using a wing traveller specifically designed and manufactured for this viaduct; see Figure 19).



Figure 13: General view of the deck construction, with the MSS and the wing traveller

This equipment allows for the accurate positioning and installation of the prefabricated struts that support the cantilevers (Figure 17), ensuring correct geometry and load transfer during construction.

The post-tensioning of the longitudinal tendons located in the top slab is generally carried out once all the cantilever segments corresponding to a given span are completed.

As a result, part of the compressive stresses introduced by the prestressing are transmitted to the cantilevered wings.

This aspect required particular attention in the design, as the cantilevers are especially sensitive to the effects of differential shrinkage between the central box girder and the later-executed wing segments, which could otherwise lead to undesirable cracking of the cantilevered wings.



Figures 14 and 15: Views of the deck during the concreting of the U-shaped partial section

Control and monitoring of the structure during execution

The construction of a viaduct with these characteristics requires continuous monitoring and control throughout the execution process, with particular emphasis on the control of deck deformations. Under normal conditions, box girder decks constructed using movable scaffolding systems in medium-span bridges do not usually exhibit significant deflections. However, when spans reach lengths of up to 90 m for this construction method, deformation control becomes a critical issue and must be addressed with a higher level of detail and precision.

Deck deformation control and the adjustment of the MSS precamber were performed using a dedicated construction-stage monitoring model, in which both the deck and the movable scaffolding system were jointly modelled. This numerical model explicitly reproduced the successive execution phases of each span, including the suspension of the MSS, concreting of the partial U-section, partial prestressing, construction of the top slab, stressing of the remaining tendons, and finally the lowering and advancing of the MSS.

Development of this calculation model (Figure 20) made it necessary to have a close collaboration between the bridge design team (TYLin) and the team responsible for the design and operation of the MSS (TecoZam), to get a precise representation of the stiffness of both the main structure of the MSS and its formwork system.

The analytical model was complemented by an extensive topographic survey of both the deck and



Figure 16: Aerial view of the deck during the execution of the top slab (span 1)

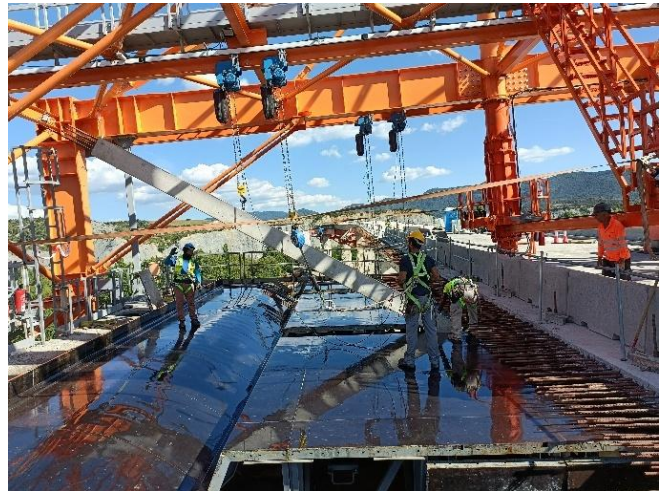


Figure 17: Positioning of the prefabricated strut with the wing traveller



Figure 18: Aerial image during the reinforcement of the top slab



Figure 19: Left: Bottom view of the wing traveller. Right: Upper view of the wing traveller

the MSS, as well as by an instrumentation system based on strain gauges installed on the rear suspension beam of the scaffolding system, with continuous monitoring all along the deck erection process.

This comprehensive monitoring approach made it possible to directly compare the reactions and deformations predicted by the numerical model with those measured on site (Figure 21). As a result, the calculation model could be progressively calibrated, reducing uncertainties and limiting variability associated with construction loads, material properties, and, in particular, the stiffness of the deck–MSS connection.

The implemented control and monitoring system proved to be highly effective in accurately predicting deck deflections during the various construction stages and, consequently, in refining the precamber applied to the MSS formwork. As illustrated in the deflection records shown in Figure 21, the measured deformation values were significant, fully justifying the level of monitoring and control adopted for this structure.

Within the different construction support tasks developed, in addition to the geometrical control described before, we highlight the adaptations done in the areas where prefabricated struts insert into the deck. In order to facilitate the opening and removal of the self-supporting formwork, the struts supporting the wide cantilevers at 5m intervals were anchored into niches formed within the webs.

This area is problematic from an execution point of view due to the reinforcement congestion originated by the combination of global deck reinforcement and local reinforcement of the niches, while the

available space gets reduced but the presence of the niches themselves. This congestion is even more critical in areas where prestressed tendons run near the bottom slab of the deck.

All this led to the need to study, together with the Construction CJV, some solutions that worked in favour of adequate reinforcement placing and casting of these areas. An enlargement of the bottom corners of the deck (Figure 22) was the final solution adopted. This enlargement was created with a vertical formwork over the bottom slab.

This solution allowed the use of more liquid concrete, which could fill the niche area more easily that stayed in place because of the vertical formwork and simplified the vibration of that zone.

To verify the correct structural behaviour of these heavily reinforced zones, a series of in situ load tests were carried out on site (Figures 23 and 24). These tests required the design and installation of a dedicated steel reaction frame against which the hydraulic jacks used during testing could apply the loads.

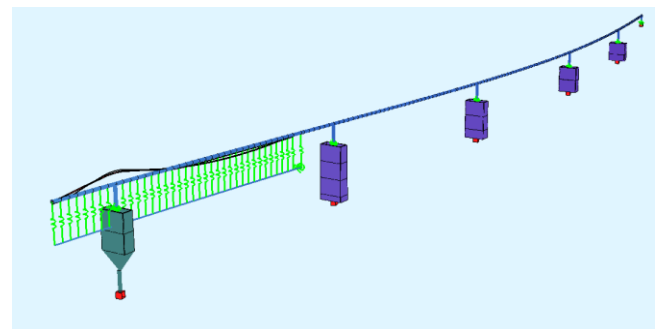


Figure 20: Detail of the calculation model used for the analysis of the interaction between the deck and MSS

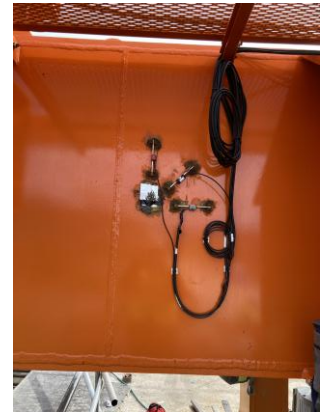
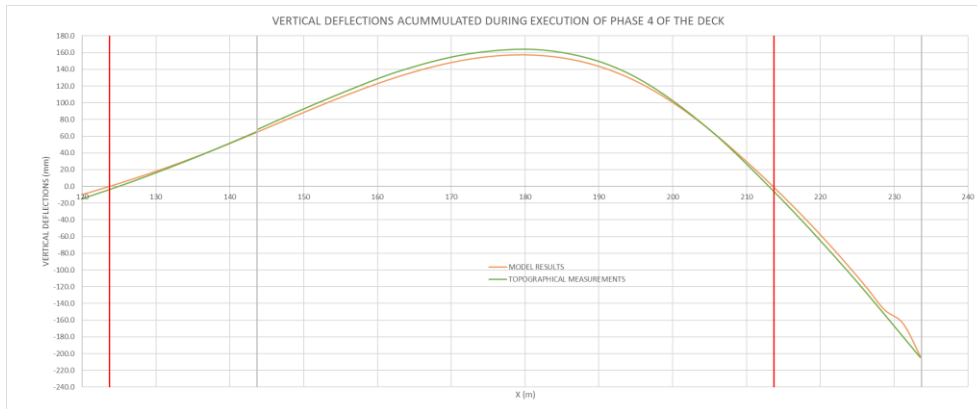


Figure 21: Left: Comparison of predicted deflection in the theoretical model with field measurements when executing phase 4. Right: View of the strain gauges installed on the suspension beam of the MSS

The jacks loaded some concrete pieces with the shape of the end of the prefabricated strut, so the load introduction in the deck could be as similar as possible to the real final situation.

The objective of these load tests was to confirm that no damage—such as cracking, spalling, or other forms of distress—occurred in the box girder concrete under service-level loading.

This requirement was successfully met in all tested cases, validating both the design assumptions and the construction details adopted in these critical areas.

VII. CONCLUSIONS AND REMARKS ON THIS CONTRIBUTION

The Sabiñánigo Viaduct represents a significant advance in span-by-span in-situ concrete bridge construction. The adopted solution demonstrates the feasibility of extending movable scaffolding system (MSS) technology to unprecedented span lengths and performance levels.

With a 90 m extreme span, the viaduct establishes the longest end span ever executed using an MSS. In addition, the use of 90 m typical spans along the main structure equals the current world record for this construction method, confirming the maturity of the design, construction, and monitoring strategies adopted.

In this way, the Sabiñánigo Viaduct provides valuable experience and reference data for future long-span concrete bridges built with movable scaffolding systems.

ACKNOWLEDGEMENTS

The authors wish to express their sincere appreciation to the joint venture *Rover Infraestructuras – Aldesa Construcciones* for the trust placed in IDEAM (now TYLin) in the development of the Modified Detailed Design of the Viaduct over the Gállego and Aurín rivers in Sabiñánigo, as well as for the constructive approach and respect shown toward the technical decisions adopted throughout both the design and construction phases.

The support provided by the *Demarcación de Carreteras del Estado en Aragón* during the development of the project and the subsequent construction stages is also gratefully acknowledged.

Finally, the authors recognise the close collaboration maintained with the TecoZam team from the design phase through to the completion of construction.

The commitment, technical expertise, and professionalism of all parties involved were essential to the successful realisation of this infrastructure.



Figure 22: Interior view of the deck including the enlarged corners



Figures 23 and 24: Load test in strut niches

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Figures 25 and 26: Bottom view of the deck towards abutment 1 and 2, respectively, from span 3



American Icon

San Francisco-Oakland Bay Bridge East Span

TYLin

Photo Credit: Thomas Heinser

GIANT STEPS IN THE PYRENEES: ENGINEERING INSIGHTS ON THE LAUNCHING OF A 90M SPAN MSS

Hugo Coelho, Project Technical Coordinator; Jose M. F. Gutierrez, Technical Director; Alberto Alonso, Chief of Production; Alberto García Conde, Site Engineer; Manuel Vieira Monteiro, Technical Office Site Supervision Engineer; João Tiago Viana, Technical Office Structural Designer
TecoZam, Spain

1. INTRODUCTION

This article focuses on the construction process of the bridge over the Gállego and Aurín rivers, located within the Sabiñánigo East – Sabiñánigo West section of the A-23 Motorway. This structure represents an unprecedented achievement within its typology, setting new span records.

This is the **first article in a series to be published in the e-mosty & e-BriM magazines throughout 2026/7**, aiming to share the key lessons learned during the execution of this outstanding structure. Specifically, this first instalment focuses primarily on the stability of the Movable Scaffolding System (MSS) during launching operations. In this particular project, given the unprecedented scale, geometric complexity, and the specific typology of the MSS, launching stability was addressed as a critical issue.

The A-23 motorway, known as the "spine of Aragon" and part of the European E7 route, holds significant importance for regional connectivity and economic development, primarily by linking the Mediterranean coast to the Pyrenees and France.

Currently in its final stage of construction, the A-23 starts in the city of Sagunto (northeast of Valencia), providing a vital connection to Teruel, Zaragoza, Huesca, and Jaca. While the stretch between Sagunto and Zaragoza has been operational since 2008, the final construction sections are located in the province of Huesca, specifically in the Sabiñánigo area.

Full completion of the motorway is expected by the end of 2026, finally establishing a direct high-capacity link between the Mediterranean Corridor and the Pyrenees.



Figure 1: Sabiñánigo Movable Scaffolding in transition between spans

Status of Key Construction Sections (Huesca):

- **Sabiñánigo East - Sabiñánigo West:** Work on this 8.6 km section is in a very advanced phase. This stretch includes the significant 934m long bridge over the Gállego and Aurín rivers.
- **Lanave - Sabiñánigo South:** Following the contract award in August 2024 with a five-year execution period, this final 8.5 km section is currently progressing to ensure the full operational status of the A-23 by late 2026.

The construction of such a complex structure as the Sabiñánigo Bridge over the Gállego and Aurín rivers involved breaking several engineering records for the industry:

- **Largest span** ever executed using an underslung Movable Scaffolding System (MSS) for multi-span construction (90 m).
- **Largest span** built with an MSS in motorway bridge construction (90 m).
- **Largest extreme span** (abutment span) built with an MSS (90 m).
- **Longest sequence** of 90m spans built with an MSS (9 spans).
- **Largest cantilevered span** of an MSS during the launching stage (90 m).

2. THE BRIDGE OVER GÁLLEGO AND AURÍN RIVERS

The bridge's structural solution was heavily constrained by the first span, which spans a 90-degree bend in the Gállego River. Abutment E1 is situated atop a steep ravine, approx. 20-30 m from the crest of the slope, while Pier P1 is positioned on the riverbank. Even by shifting the abutment alignment towards the slope of the ravine, the minimum achievable length for the first span was 90 m.

Strict restrictions were imposed regarding the placement of Pier P1 and the associated earthworks. Extreme caution and a rigorous mitigation strategy were required during excavation and foundation activities.

The project implemented stringent protocols, such as runoff containment and the strategic positioning of piers, to ensure the riverbed and banks remained undisturbed.



Figure 2: A-23 Motorway layout

Source: Kalh/Wikimedia Commons, CC BY-SA 3.0 [1]

Promoter	Ministry of Transport, Mobility and Urban Agenda (MITMA)
Site Supervision	Ministry of Public Works
Technical Assistance to site supervisor	SERS
Main Contractor	UTE Autovía Sabiñánigo (Joint Venture Rover – Aldesa)
Bridge Designer	TYLin Europe LATAM (IDEAM)
Contractor (deck construction works)	TecoZam
MSS Manufacturer	NRS

This specialised care is justified by the severe environmental challenge facing the Sabiñánigo industrial area, stemming from historical Lindane (HCH) contamination from the former Inquinosa chemical plant. The river sediments now act as a hazardous reservoir that can be easily reactivated by physical disturbance or flooding, posing a continuous threat to the entire hydrological system.

In addition to the Gállego River, the bridge also crosses the Aurín River (a tributary of the Gállego) near their confluence. Both rivers are characterised by highly variable discharge and are significantly affected by flash floods and intense storm events.

Due to its hydrological nature, the basin can experience rapid and violent surges in water volume.

The most striking example of the volatility and danger of river flows in this region is the tragic 1996 Biescas Campsite flood, which occurred in the Arás ravine basin (a tributary of the Gállego River, near the Aurín). This event demonstrated how a localised storm can turn a nearly dry riverbed into a devastating torrent in just a few minutes [2].

For the reasons stated above, and given that the first span was necessarily 90 m long, it was decided to design the bridge with a multiple large-span arrangement. This option represented a significant change from the original solution. The proposed design for the Modified Construction Project maintained the horizontal and vertical alignments of the original project while increasing the total bridge length by 5 m, reaching a total of 934 m between the centrelines of the abutments. This increase results from the 5m setback of Abutment E1, which increases the distance from the steep ravine slope west of the Gállego River.

Furthermore, the span distribution was homogenised to optimise the structural behaviour and to allow for a systematic, uniform construction method throughout the entire structure, utilising standard 90m spans. Consequently, while maintaining the axis of Pier P1 in its original position to avoid impacting the Gállego riverbed, the span distribution was established as follows: 9 x 90 m + 75 m + 49 m [3].

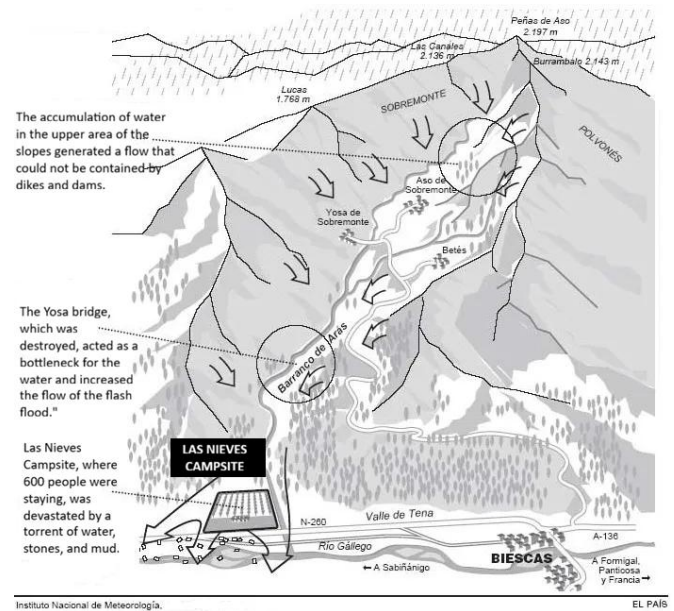


Figure 3: Infographic detailing the Arás alluvial fan morphology and campsite location (Source: El País Graphics) [2]

The elevation and plan views of the modified solution are presented in the following figures.

Regarding the deck geometry, the cross-sectional depth was adapted to the new span distribution.

The bridge features a variable depth box girder, ranging from 3.20 m at mid-span (L/28.12) to 5.50 m at the pier sections (L/16.36).

This variation is achieved through a parabolic haunch at the soffit of the box girder, extending from 1.625 m from the pier centreline to 20 m into the span.



Figure 4: A-23 Sabiñanigo Bridge Span arrangement (plan view) [3]

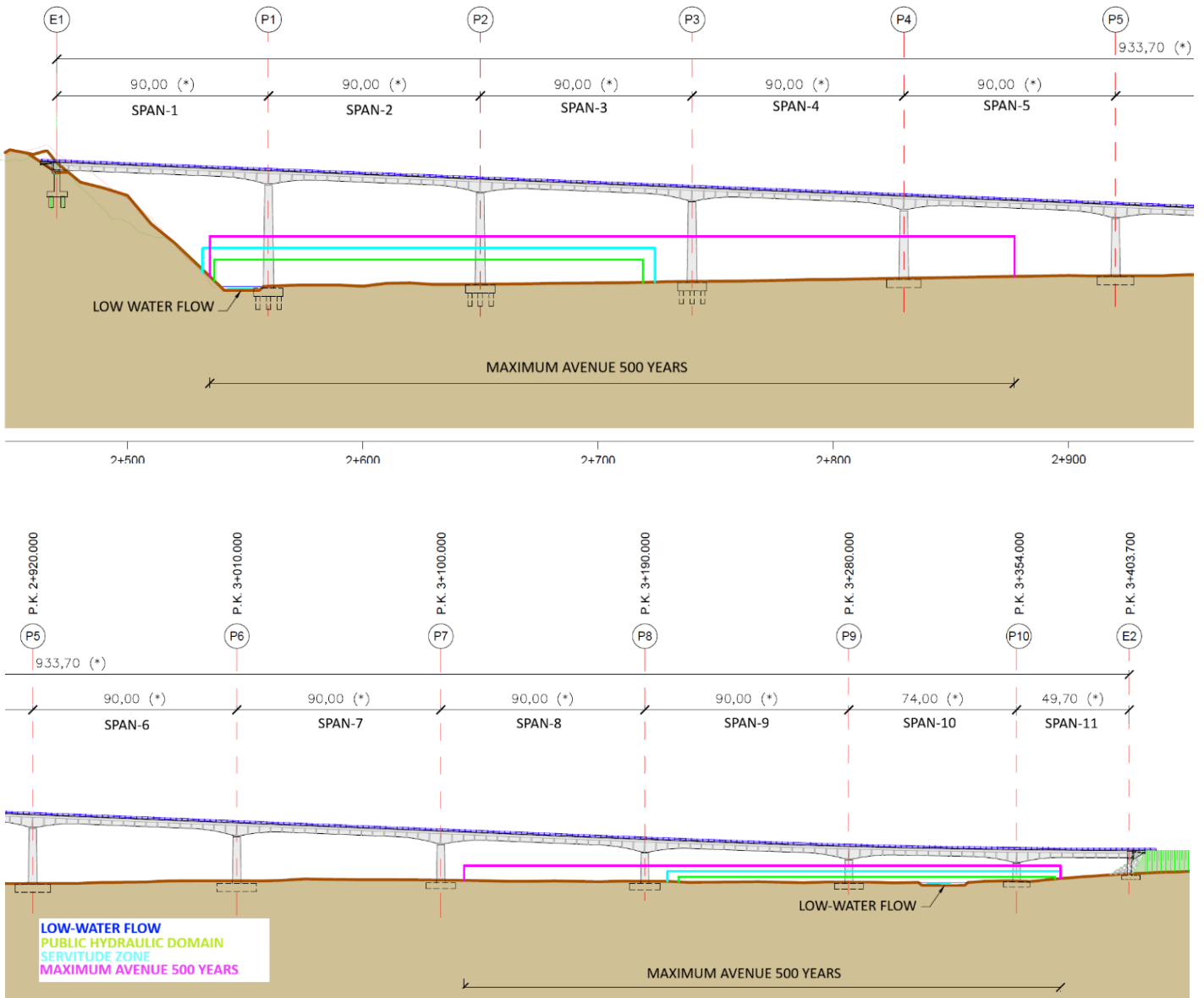


Figure 5: A-23 Sabiñanigo Bridge Span arrangement (elevation view) [3]

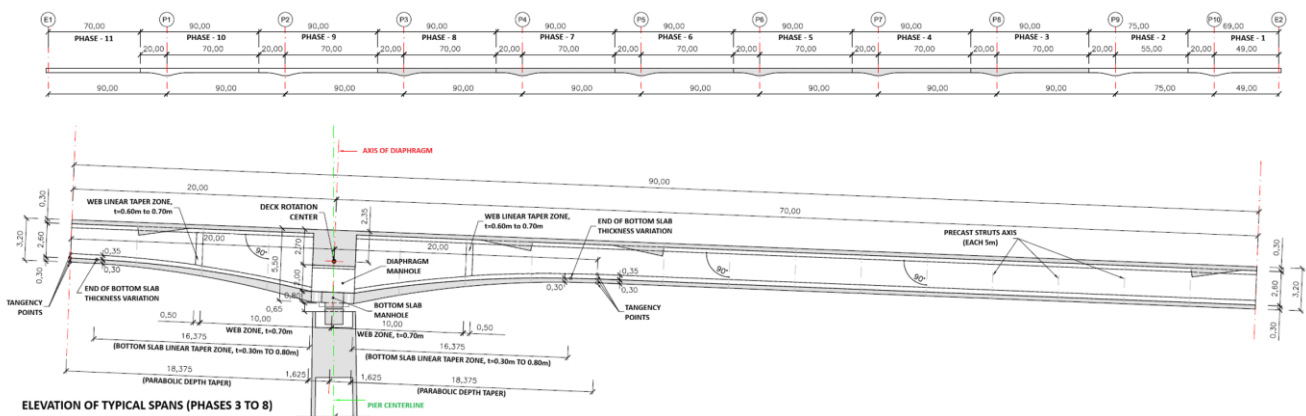


Figure 6: Variation of the box girder depth from mid-span to the pier section [3]

In the typical 90 m spans, a 50 m long central segment maintains a constant depth of 3.20 m. Above the piers, a constant depth of 5.50 m is maintained over a length of 3.25 m, matching the longitudinal dimensions of the pier head.

Regarding the deck's cross-section, the central core consists of a concrete box girder. The extensive lateral wings of the top slab are supported by prefabricated concrete struts, which are inserted into the joints between the webs and the bottom slab. These struts are spaced at approximately 5 m intervals.

At the intersection of the strut and the top slab, a continuous edge beam is provided to ensure rigid support for the outer cantilever slab, thereby limiting deflections and enhancing overall structural performance.

The webs are slightly inclined to minimise the bottom slab width and align it with the bearing requirements at the piers. This results in a box girder with a top width of 9.00 m (measured between the outer faces of the webs) and a bottom width of 7.20 m at the pier sections. This design ensures that the bottom width of the deck matches the pier's geometry, allowing for a vertical pier head with the same transverse dimensions as the box girder.

The cross-sections at mid-span and over the piers are presented in Figure 7.1 and 7.2 [3].

3. GENERAL OVERVIEW OF CONSTRUCTION PROCESS

3.1 Construction Sequence and Phasing

Given the site's features and logistical requirements, the bridge deck was executed proceeding from Abutment E2 towards Abutment E1. This was achieved through a combined system consisting of an underslung Movable Scaffolding System (MSS) for the central box girder and a dedicated wing traveller (formwork system) for the lateral cantilever slabs.

The construction sequence with MSS was divided into 11 distinct phases:

- **Phase 1:** Execution of Span 11 (49 m) plus a 20 m cantilever section of Span 10.
- **Phase 2:** Execution of the remaining 55 m of Span 10 plus 20 m cantilever section of Span 9.

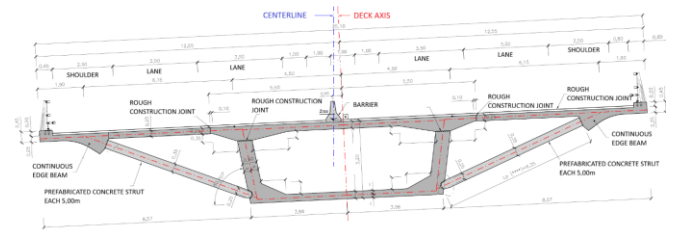


Figure 7.1: Cross-section: at mid-span [3]

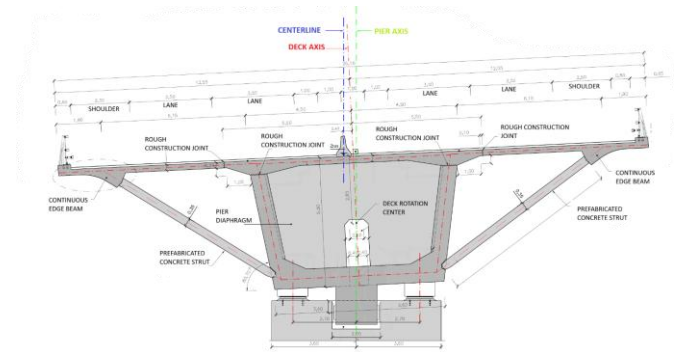


Figure 7.2: Cross-section at pier alignment (bottom) [3]

- **Phases 3 to 10:** Systematic execution of the remaining 70 m sections of Spans 9 through 2, each including an additional 20 m cantilever section of the subsequent span.
- **Phase 11:** Final 70 m section of Span 1, completing the deck closure at Abutment E1.

3.2 Box Girder Execution (MSS Method)

Each phase executed by the MSS involves the construction of the main box girder section (11 m wide) according to the following sequence:

- **Formwork positioning:** Adjustment of the formwork to the new span geometry. The external formwork is composed of lateral and bottom panels that enable independent movements. In the first step, the bottom panels are levelled to their target position by acting on the mechanical spindles that support each individual sub-panel (see Figures 8 and 9). The target position is set by topographical survey, and the final spindle strokes combine the changes required by transversal slope variations and pre-cambering to counteract MSS deformations. In the second step, also guided by topography, the lateral formwork is moved transversally to adapt to the horizontal alignment (plan curvature), see Figure 10.

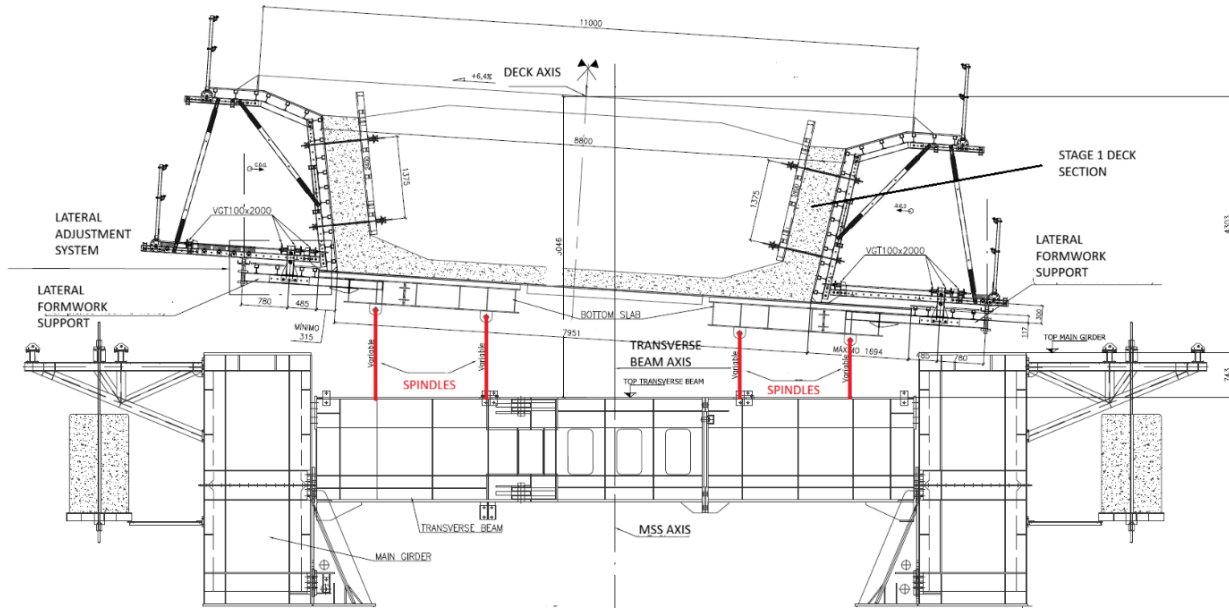


Figure 8: Formwork adjustment and MSS concrete pouring stages (cross section)

- **Stage 1 steel reinforcement:** Placement of passive reinforcement and prestressing tendons for webs and the bottom slab: to increase productivity and safety, pre-assembled reinforcement modules—including prestressing ducts—are lifted and positioned onto the formwork by crane, see Figure 13.

This approach limits in-situ work primarily to the execution of reinforcement laps and connections.

- **Casting of Stage 1 ('U' section):** Concrete pouring of the partial deck section—comprising the bottom slab and the webs up to the intersection of the upper haunch (0.60 m below the top face of the deck)—across the entire span length (different concrete pouring stages are presented in Figure 8).
- **Stage 1 Prestressing:** Partial prestressing of the U-section: The purpose of this stage is to pre-compress the partially built section to counteract the stresses that will later arise during the top slab casting, resulting from the composite behaviour of the deck and the MSS structure. Prestressing between stages consists of the full tensioning of parabolic cables CP1 and CP2 (bottom layer tendons, 20% of total prestressing force. See Figures 11 and 12).

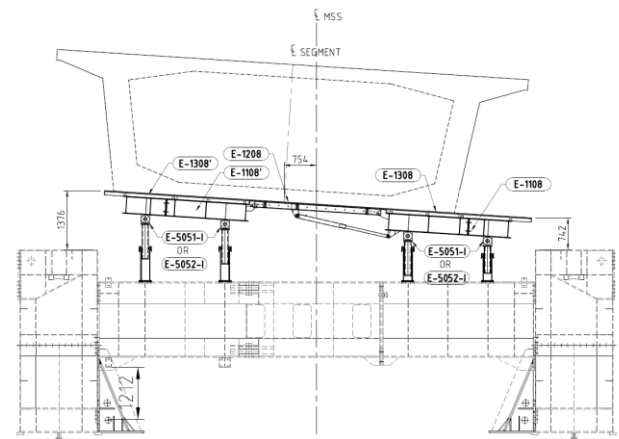


Figure 9: Mechanical spindles for bottom slab vertical adjustments (transversal slope rotation and pre-cambering)

PULLING SYSTEM FOR LATERAL FORMWORK ADJUSTMENT

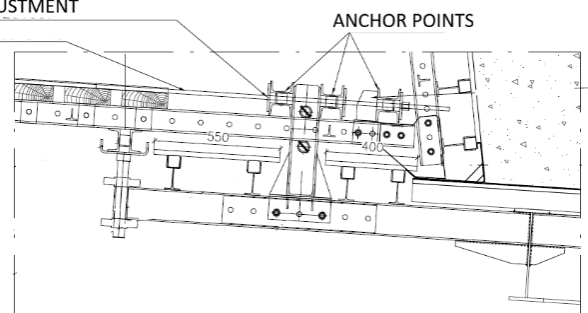


Figure 10: Lateral formwork adjustment system

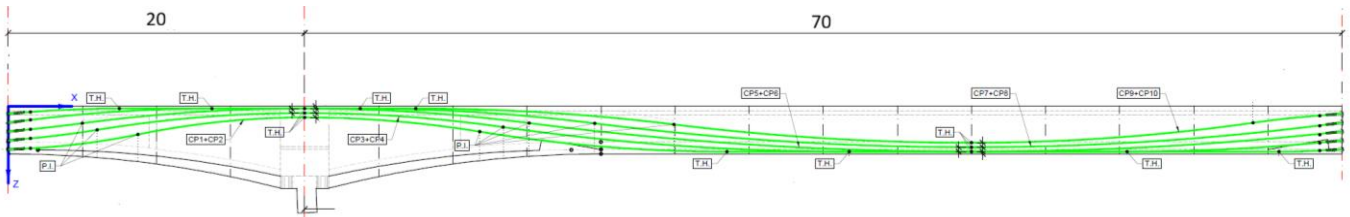


Figure11: Parabolic prestressing cables layout for current span (elevation) [3]

- **Stage 2 steel reinforcement:** Placement of passive reinforcement and prestressing tendons for the top slab.
- **Casting of Stage 2 (Top Slab):** Execution of the top slab between the webs (completing the 11 m wide box section) across the entire phase length.
- **Stage 2 Prestressing:** Final prestressing of the box section completed by the MSS.
- **MSS Launching:** Launching the movable scaffolding system to the next span.

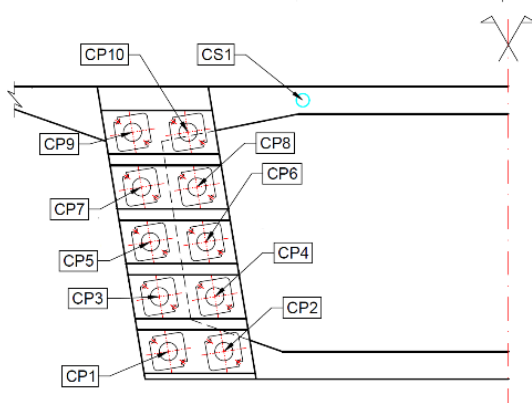


Figure12: Web cables disposition at construction joint (anchoring section) CP1 and CP2 to be stressed between concrete pouring stages [3]

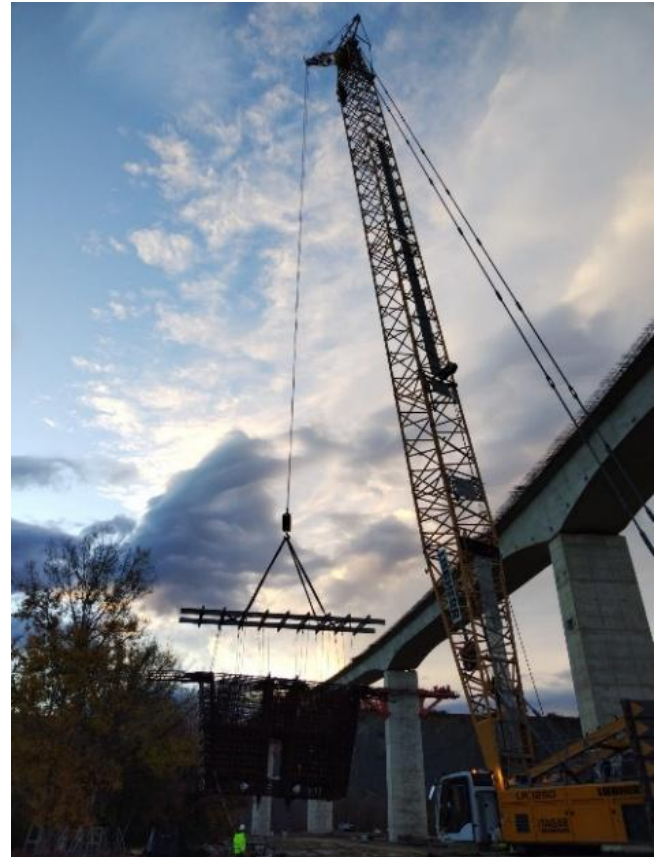


Figure13: Rebar cage positioning



Figure 14: Wing traveller for cantilever slab execution

3.3 Wing Traveller (Cantilever Slabs) Execution

The wing traveller finalises the top slab construction in 15 m long segments, typically operating two spans behind the main MSS unit (span "j+2").

Once the lateral wings for a specific phase are completed, the prestressing of that span is finally concluded.

4. OVERALL DESCRIPTION OF 90M SPAN ZAMORA II UNDERSLUNG MSS

The MSS applied in the Sabinanigo Bridge construction was an underslung piece of equipment, specifically re-engineered for this unique application, and branded as **Zamora II**.

The MSS main technical requirements derived from the bridge features are presented below.

- **Spans arrangement:** 49.7 m; 74.0 m; 9x 90 m.
- **Typical span length:** 90 m.
- **Maximum cantilevered span in launching stage:** 90m.
- **Minimum plan radius:** 1015 m (with variation)
- **Maximum longitudinal slope:** +4.7 % (with variation).
- **Transverse slope:** Variable from -6.4% to + 6.4%.
- **Deck depth:** Variable, changing from 5.5 m (at support) to 3.2 m (at mid-span).
- **Deck width:** 11 m built with MSS (out of a total width of 25.1 m).
- **Deck cross section (built by MSS):** Approximately ranging from 10 m² (in mid-span) to 16.4 m² (pier section).
- **Pier dimension (transversal width x depth):** 7.20 m x 3.25 m.
- **Maximum pier height (P1):** 43.7 m.

While its design is based on a classic underslung model (see Figure 15) - featuring two main box-section girders and triangular trussed launching noses - the MSS is particularly striking due to its unprecedented scale.

The nose trusses possess a primary resistant plane (vertical), which allows them to be oriented to adapt to the alignment's curvature during launching operations.

During this phase, the sliding supports are positioned on brackets mounted on the piers.

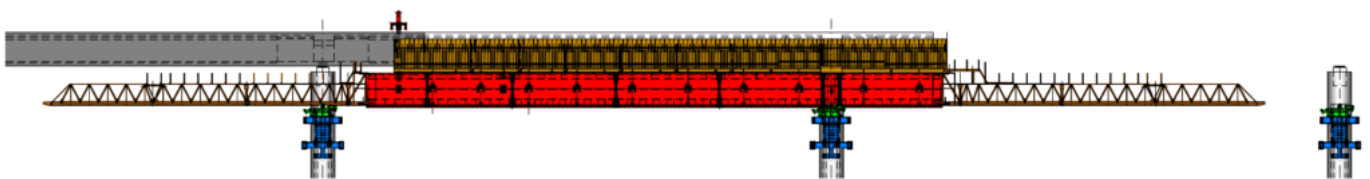
In the casting phase, the MSS adopts the following support configuration:

- **Front support:** Positioned on the same brackets used during launching, with the main girders supported by front elevation jacks (one cylinder per girder).
- **Rear support (Suspension Gallow):** A hanging system suspended from a spreader beam supported by the previously constructed deck, positioned near the construction joint.

This support configuration during the casting phase is advantageous for both the MSS—by shortening the effective span during concreting—and the bridge deck itself, as the loads introduced by the MSS help balance stresses and deformations in the deck during the construction stage.

Furthermore, in the casting phase, the main girders are connected by transversal beams which, besides providing direct support to the formwork, significantly improve the global torsional stiffness of the system.

Longitudinal view



Plan view

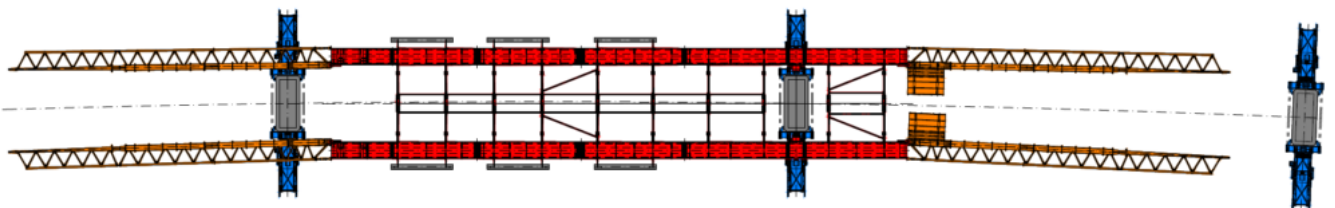


Figure 15: Typical configuration of an underslung MSS [4]

5. CHALLENGES PRESENTED TO MSS DESIGN

The structural design of a large-scale MSS cannot be performed without accounting for its interaction with the bridge under construction and integrating human operational factors during construction.

- As the MSS relies on the bridge piers and the previously cast spans for support, the load transfer mechanism must be double checked (both by the bridge and MSS designers).
- A critical particularity of this process is that, during the pouring stages, the MSS and the bridge segment under construction effectively form a temporary composite structure. This interaction is further complicated by the two-stage casting method for the box girder (U-section in the first stage and the top slab in the second, with partial prestressing in between).

This phased construction approach means that the MSS must withstand the initial dead load of the fresh concrete while subsequently interacting with a partially cured concrete section that already possesses its own evolving stiffness. This composite behaviour significantly alters the deflection profiles and stress distribution, affecting the forces of the combined structures and the final geometry of the bridge deck.

- Furthermore, the deck geometry variability requires an adjusting structure, which increases the complexity of structural analysis since the structure layout is in constant change.

The complexity of the Sabiñánigo Bridge cannot be attributed solely to its span scale. While its dimensions are record-breaking, the geometric complexity of the deck constitutes a primary challenge that would necessarily result in a complex construction process regardless of the span lengths.

Even if more conventional spans had been adopted, execution would still be demanding, mainly because of the following complexity-raising factors:

- **Horizontal Alignment:** Combination of a main curve and a reverse curve with transition spans in between - this asymmetric S-curve alignment is clearly evidenced in Figure 16.

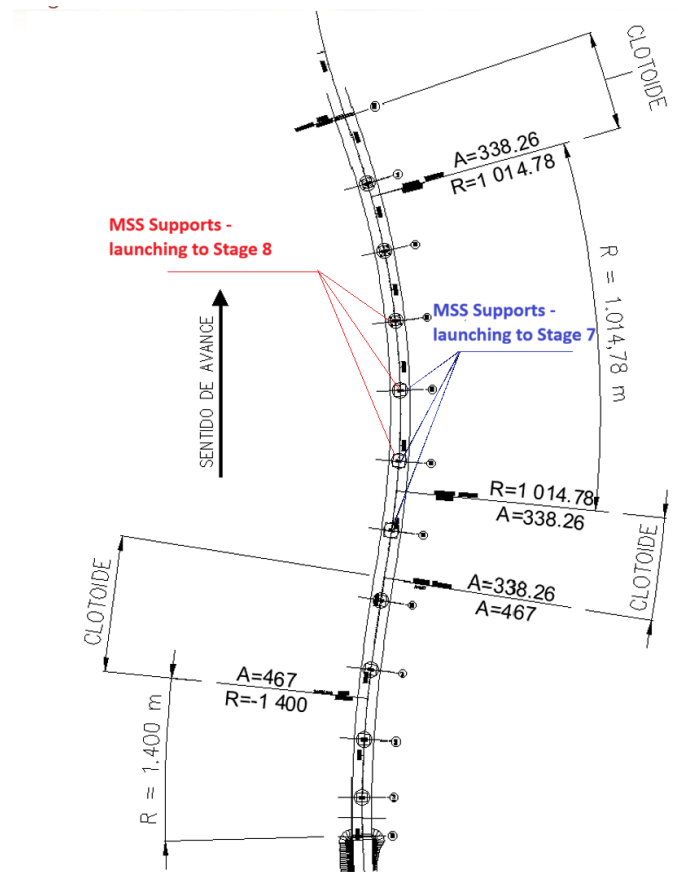


Figure 16: Sabiñánigo Bridge plan view – curve transitions geometrical parameters [5]

- **Longitudinal Profile:** Significant variation in the longitudinal slope, reaching a substantial maximum gradient value.
- **Cross-Sectional Geometry:** A complete inversion of the transverse superelevation (cross-slope), involving substantial limiting values at the edges.
- **Formwork Adaptability:** Extensive formwork adjustments are required to approximate the continuous horizontal curvature using highly discretised polygonal segments. This discretisation is critical for harmoniously fitting the precast concrete struts that support the deck lateral wings.

Another factor of geometric complexity is the variable depth of the deck. This design choice is justified by the span length, aiming for a more optimised structural solution. However, when combined with the previously mentioned factors, it results in a project with a high degree of execution difficulty.

A fundamental issue with scale transitions is that the associated problems are often not visible at first glance. The true engineering challenge lies in discovering and identifying phenomena that, while negligible at smaller scales, become dominant as dimensions increase. Often, designers are not alerted to these factors because they fall outside conventional experience and practice.

Consequently, scaling is inherently a search for emerging problems. It is not by chance that many of the current limits in bridge engineering were calibrated through lessons learned from failures that, in some cases, resulted in structural collapses.

When facing a significant change in scale, one must always remain conscious of the existence of a 'limit value'—an unknown threshold that cannot be crossed without risking catastrophic failure. Crucially, this type of search for the unknown takes time.

With that in mind, and within the scope of this publication, we have selected specific technical points to present more deeply, aiming to share some of the lessons learned during the operation of the Zamora II.

6. INSIGHTS ON MSS DESIGN – LAUNCHING STAGES

6.1 Abrupt change of length in first spans

Counterintuitively, in large-scale MSS applications, some of the most demanding structural scenarios occur during construction of the shorter spans, typically near the abutments.

The mismatch between the machine's design scale and the reduced span dimensions imposes significant constraints on support positioning and the distribution of internal forces. This often leads to **load configurations** that are more critical than those encountered in the typical long-span cycles.

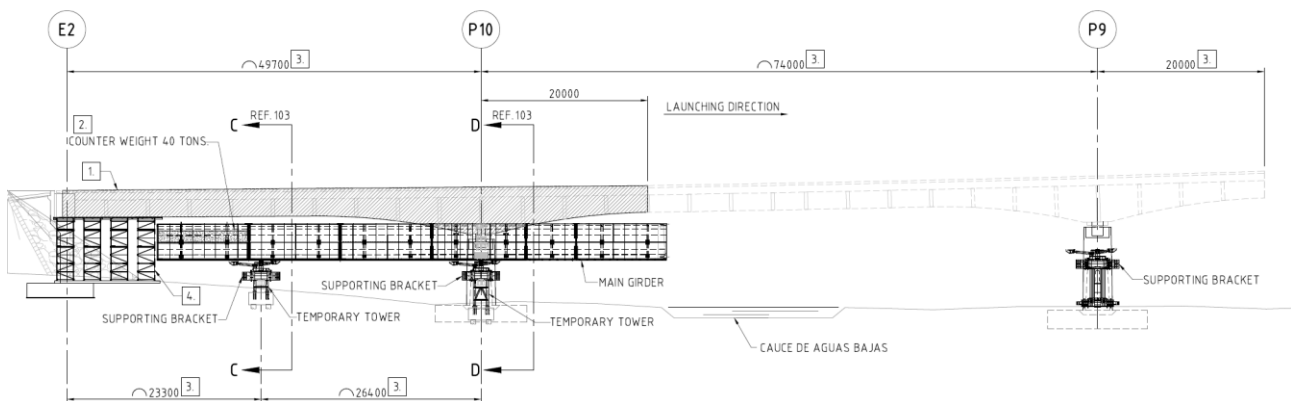


Figure 17: MSS First Span (E2-P10) configuration - elevation

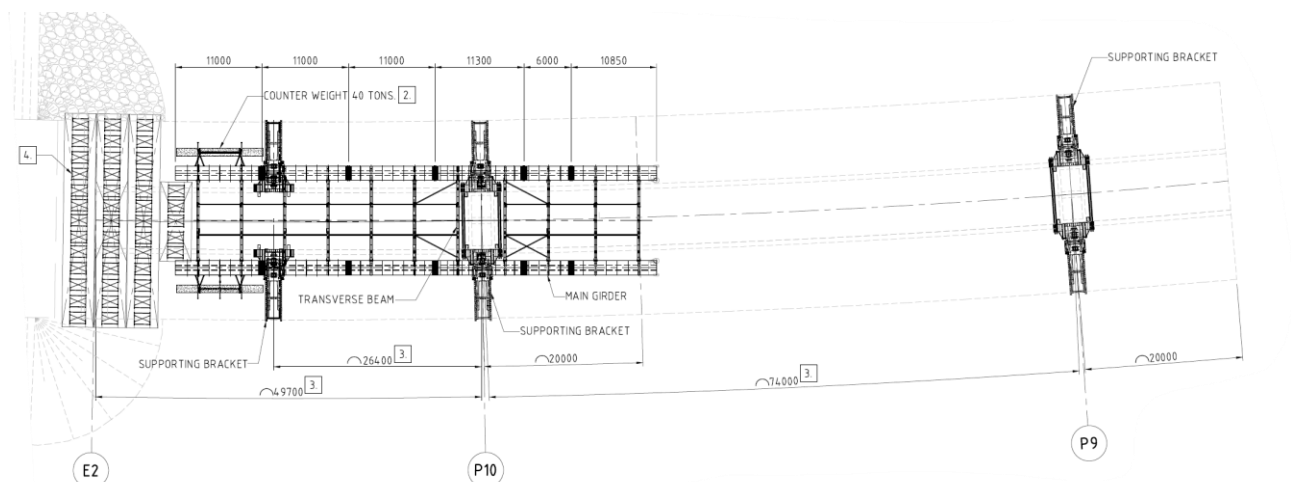


Figure 18: MSS First Span (E2-P10) configuration - plan view

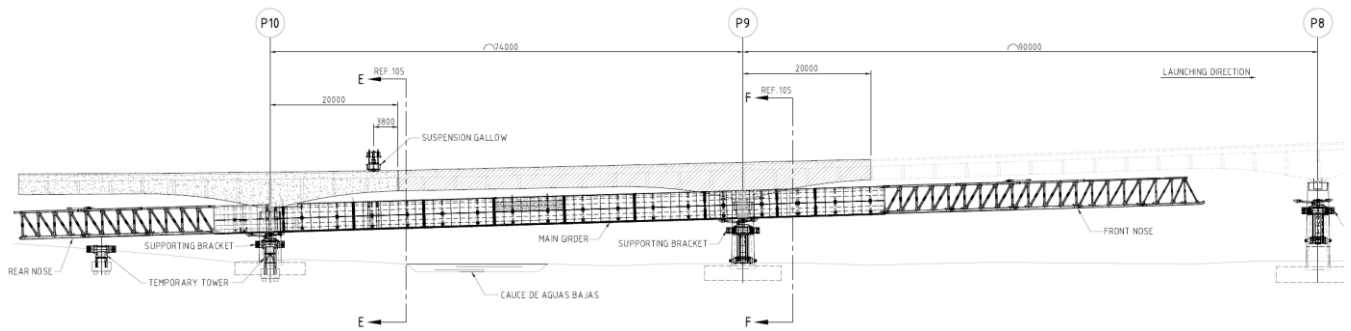


Figure 19: MSS Second Span (P10-P9) configuration - elevation

In the case of the Sabiñánigo Bridge, the initial span sequence followed a progression of 49.7 m, 74 m, and finally 90 m for the third span. The fact that the abutment was executed in a single stage introduced a significant constraint, restricting the available space for the MSS supports in the first span.

This left little margin for operational clearance during the assembly phase, requiring a meticulous approach to safeguard the system's structural stability during the phased transition from the first to the second span.

The assembly operations between the first and second spans were notably time-consuming and were conducted with the MSS in its launching configuration, which inherently decreases stability margins. In such scenarios, safeguarding both the transverse and longitudinal stability of the system is crucial. This requires a continuous re-evaluation of the Centre of Gravity (CoG) as each new sub-assembly is installed; where necessary, counterweight adjustments must be implemented to maintain the required operational safety margins.

From a structural perspective, launching the MSS over shorter spans may appear less demanding; however, this configuration often requires launching with the MSS vertically supported in three different alignments.

This creates a statically indeterminate (hyperstatic) system that is highly sensitive to vertical misalignments between supports and to the rail geometry itself.

In such a rigid configuration, any deviation of the launching rail from a perfectly straight alignment is equivalent to a support settlement. This is particularly concerning because these shorter

spans lead to stiffer girder configurations, in which even minor geometric discrepancies can markedly redistribute shear forces and bending moments, leading to substantial deviations in internal forces compared to a standard launching scenario.

An aggravating factor of this phenomenon is the inherent difficulty in controlling the geometry of the main girders during launching. As the modules are assembled in a rear cantilever configuration adjacent to a particularly short span, geometric control during the assembly phase becomes a significant challenge.

In such scenarios, the final geometry of the main girders and launching noses can only be reliably assessed after the assembly is complete—at which point, given the MSS scale, implementing geometric corrections is complex and technically demanding. Figures 20 and 21 illustrate the constraints of this process and the difficulty of ensuring alignment under these specific conditions.

Furthermore, performing the assembly under such constraints significantly affects the learning curve and operators' ability to assimilate the system's behaviour. In the case of the Sabiñánigo Bridge, the MSS reached its full configuration only when positioned at the third span.

Ideally, for a machine of this magnitude, the initial assembly should result in a configuration close to its final operational state; however, in this project, the team had to manage a system in constant geometric flux until the third span.

The full assimilation of the machine's real scale—including its geometry, weight, centre of gravity, and overall performance—was only achieved during the transition from the third to the fourth span.

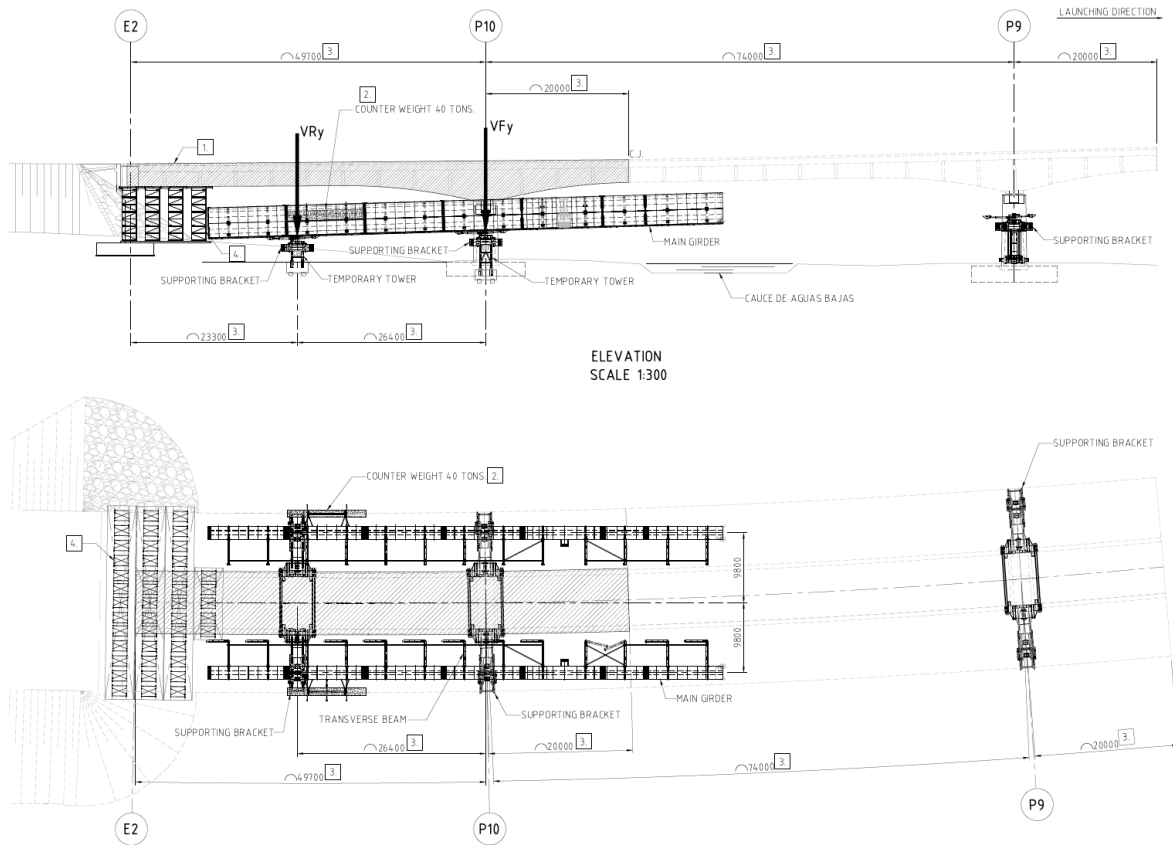


Figure 20: Assembly proceedings and launching from first to second stage (intermediate step with MSS in 2 supports)

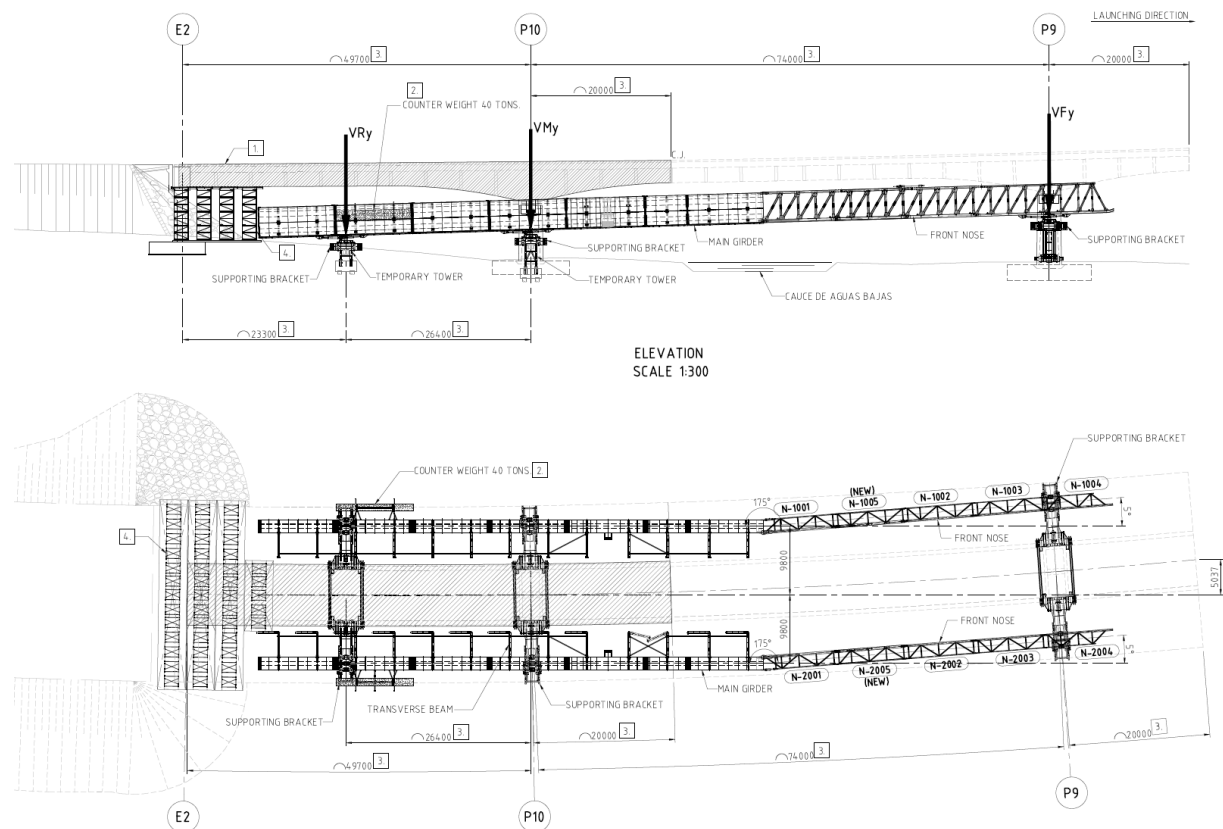


Figure 21: Assembly proceedings and launching from first to second stage (intermediate step with MSS in 3 supports)

The potential consequences of a delayed learning curve must not be overlooked; specific operational measures must be implemented to allow the team to gradually gain sensitivity to the MSS's response. This includes sensory indicators, such as structural noises and vibrations, which are often the most critical clues for detecting anomalous behaviour.

In a machine of this scale, structural noise is expected; the challenge lies in establishing objective criteria to discern normal operational sounds from those indicating a potential malfunction.

Regarding the learning curve, the fact that a machine of this magnitude follows an extended cycle—exceeding four weeks—inherently slows down the assimilation of operational experience.

To address these difficulties, each construction cycle was managed as a dedicated sub-project. This involved a rigorous analytical treatment of data gathered from previous spans and the subsequent issuance of updated operational protocols for each specific stage.

To summarise this sub-topic, the following conclusions are highlighted:

- **Phased Abutment Execution:** Executing the bridge abutment in two distinct phases—ensuring the partially built structure imposes geometric constraints on the MSS similar to a standard pier alignment—is highly advantageous. This approach allows for the assembly of a larger portion of the main girders and launching noses within the starting span, reducing the need for assembly in unstable launching configurations. This is beneficial for geometric control, optimises the assembly schedule, and enables MSS testing in a near-final configuration, significantly enhancing operational safety.
- **Stage-Specific Structural Modelling:** It is fundamental to develop specific analytical models for the different assembly stages. These models must safeguard not only the transverse and longitudinal stability of the partial structure but also its global structural integrity under evolving configurations.
- **Impact of Hyperstatic Forces:** Statically indeterminate forces can significantly impact structural integrity. This factor must be addressed by integrating actions into the

numerical model that simulate geometric uncertainties, such as potential support settlements or rail misalignments.

- **Compensation for the Learning Curve:** The deceleration of the operator's learning curve must be offset by strengthened on-site technical assistance from the MSS designer. Beyond transmitting information to the operations team, the designer's presence is vital for collecting field data. This information is essential for planning subsequent operations and for reducing model uncertainties as the system reaches its final operational configuration.

6.2 Structural Stability in launching Stages

When analysing the launching stability of an MSS of this magnitude, the following aspects demand detailed engineering attention:

- **Stability Against Overturning:** Specifically in the transverse direction, where safety margins are narrower.
- **Integrity of Main Girder Webs:** A critical assessment of the webs at the reaction points of the sliding supports. This involves investigating complex phenomena, ranging from web crushing to local buckling (patch loading), under the combined effects of biaxial compression and shear stresses.
- **Vibrations and Deflection Amplification:** Focused on the launching noses in maximum cantilever positions. This analysis must account for the combined effects of wind loads and the system's inertial forces during movement. In maximum cantilever positions, starts and stops induce vibrations with clearly visible lateral displacements of the nose structure.

A common denominator across these three aspects is the precise characterisation of the self-weight and the system's Centre of Gravity (CoG). Effectively, a vectorial characterisation of the travelling weight is a fundamental requirement for assessing launching stability.

This requirement is particularly critical for an MSS of this typology due to the following factors:

- **Scale and Weight:** The extraordinary dimensions of the system result in massive self-weight loads that fundamentally govern the structural stability assessment during launching.

In the case of Zamora II, the total travelling weight is approximately 13,500 kN, with an equitable distribution between the two Main Girders.

- **Underslung MSS Typology:** Underslung systems are inherently more susceptible to transverse stability issues during launching. The main structural body is split into two independent girders for pier clearance, and the weight of each girder is not transversely centred, requiring the use of counterweights. Furthermore, compared to Overhead MSS, underslung girders are significantly narrower. While an Overhead girder of this scale may vary from 4 m to 8 m in width, the Zamora II girders are 1.815 m wide (measured between rail centrelines).
- **Curved Alignment and Nose Rotation:** The combination of long spans and demanding plan curvature requires the use of the launching noses' rotation functionality to achieve a feasible launching trajectory.
- **Successive Reverse Curves:** The S-curve alignment requires inverting the nose rotation during the launch—a factor that significantly shifts the transverse CoG position and impacts the internal force distribution.
- **Launching Nose Truss Typology:** The noses feature a triangular truss geometry with a single rail alignment. In this configuration, the vertical plane serves as the primary structural plane, while the base and hypotenuse act as bracing elements. During certain launching extensions, with the system supported at two points on the main body and one on the nose, the configuration becomes isostatic. In this "tripod" setup, the three support points form a stability triangle, and the CoG must be strictly positioned within this triangle with an adequate safety margin. Although the system is equipped with anti-overturning devices, launching procedures are designed to ensure these devices are never activated under normal operational conditions.

The analytical determination of the CoG and the travelling weight—including its progressive calibration, on-site verification, and data collection for model updating—must be emphatically highlighted as a key point.

While this task is time-consuming, the data gathered is crucial for maintaining rigorous operational control.

The CoG coordinates are calculated as the mass-weighted average of the positions of n sub-components. From a stochastic perspective, increasing the degree of structural discretization (n) minimizes uncertainty through two primary mechanisms: first, the 'compensation effect', where random errors in individual mass estimations tend to cancel each other out in the global moment summation; and second, the reduction of 'centroidal uncertainty', as smaller elements allow for a more accurate localization of their mass centre [6, 7].

In the analytical estimation of the CoG, global uncertainty is disproportionately sensitive to components positioned at the structure's extremities. Due to the increased lever arm, any error in the mass or position of these peripheral sub-components has a greater impact on the final result. Therefore, to optimise the estimation's precision, a graded discretisation strategy is recommended: the mesh of sub-components should be more refined in regions distant from the geometric centre, thereby minimising the total uncertainty of the global moment.

In the practical case of the Zamora II MSS, the vectorial assessment of the travelling weight was initially conducted through an analytical approach.

The first field measurement of the travelling weight was performed during the demoulding from Stage 2 and the subsequent preparation for launching to Stage 3. At this stage, the MSS was in a near-final configuration, with only the assembly of the two rear launching nose modules and the rear formwork panels—scheduled for Stage 3—yet to be completed.

Initially, a global measurement of the travelling weight and the longitudinal (x-axis) CoG coordinate was conducted. For this purpose, the system was configured in a closed loop, with transverse structures connecting the two main girders. Readings were taken during the lowering process using pressure gauges installed on the MSS elevation cylinders: two cylinders located at the Pier 9 (P9) bracket and two on the rear Suspension Gallow.

The analytical approach and field measurements were compared, enabling numerical model validation before proceeding with the separation of the main girders by disconnecting the transverse structures. Subsequently, with the main girders already separated, the transverse CoG position for each individual girder was assessed.

For this purpose, a specific support set-up was adopted: each girder was individually supported by hydraulic jacks at the front support and by sliding bearings at the rear alignment—the external sliding support incorporated a load cell. The internal rear reaction was then determined through data cross-referencing.

The results of these measurements and the corresponding model discretisation are presented in Figures 22 and 23.

The measurements of the transverse CoG (y-coordinate) were conducted with the launching noses in a straight configuration to enhance stability during the weighing procedure.

The results demonstrated satisfactory convergence with the analytical estimates, validating the continuation of the operation.

The process identified only the need for a specific adjustment to the Right Main Girder's counterweights prior to the Stage 7 launch, marking the transition into the spans with the minimum plan radius.

Ultimately, the collected data provided the foundation for a comprehensive BIM model, designed to integrate every component—from the primary structural elements to the secondary timber ribs used for formwork surface moulding.

Equally critical is the seamless transfer of this component-weight discretisation into the structural analysis model. This calculation model serves as a "testament"; since these structures are reused across multiple projects, the model must be structured to be easily revisited and interpreted by different designers over time.

This underscores the importance of maintaining a consistent language and data coherence between component lists, field measurement records, maintenance logs, calculation models, and the integrated BIM environment.

6.3 Stability against Overturning

For the verification of overturning stability during MSS launching operations, it is recommended to apply the equilibrium limit-state criteria (EQU) as defined in Eurocode 0 [4, 8, 9], which proposes the following load factoring:

- Favourable permanent loads: factored by 0.9.
- Unfavourable permanent loads: factored by 1.1.
- Wind loads (vertical + transverse components): factored by 1.5.
- Live loads: A characteristic value of 20 kN was applied at the most unfavourable position during the launching stage, following the guidelines of the Manual de Autocimbras [10] — a primary Spanish reference for the design and operation of Movable Scaffolding Systems. Live loads were factored by 1.50, and subsequently reduced by a factor of 0.7, as they were not considered the leading variable action.

In accordance with Eurocode 0 [8], the safety verification is defined by the following equilibrium condition:

$$E_{d,dst} \leq E_{d,stab}$$

Where $E_{d,dst}$ is the design value of the effect of destabilising actions and $E_{d,stab}$ is the design value of the effect of stabilising actions. Alternatively, this may be expressed as the ratio between the design moments, with the following acceptance criterion:

$$\frac{\text{Stabilising Moment}}{\text{Overturning Moment}} \geq 1$$

Regarding wind loading, the Eurocode-framed IAP-11 standard [11] was applied.

Specifically, following the provisions of section 4.2.5.1.3, which state that in the absence of specific wind-tunnel data for the combined effect of transverse and vertical pressure, the following shall be assumed:

- For solid-web decks (plate girders): the transverse wind load is applied at 60% of the height of the maximum projected area, measured from the base of the deck.
- Vertical load: applied at a distance from the windward edge equal to one-quarter (1/4) of the deck width.

Pressure in Elevation Jacks - before launching to Stage 3 (25-07-2023)						
X distance between MSS supports <u>57,8 m</u>						
	Pressure (bar)	Acil (cm ²)	F (kN)		Correction Step (*)	MSS Weight (kN)
Front hydraulic cylinder N. 1	155	2123,7	3292	6690		6690
Front hydraulic cylinder N. 2	160	2123,7	3398			
Rear hydraulic cylinder N. 1	262	1176,9	3083	6343	-80	6263
Rear hydraulic cylinder N. 2	277	1176,9	3260			
TOTAL						12953
	Weight (kN)		CoG X coordinate (m) (**)			
Analytical evaluation	12939		27,86			
Measured values	12953		27,95			
Deviation	14		0,09			
	0,11%		0,33%			
(*) - Deduction for the weight of the hanging components and the D75 Dywidag bars supported by the gallow (elements not travelling with MSS)						
(**) - Longitudinal distance of CoG to front pier alignment P9						

Figure 22: Assessment of Travelling Weight and CoG x-coordinate through field measurements (results)

Estimation: List of components Spreadsheet: Right Girder						
Weight	655 ton					
Longitudinal distance from CoG to P9 (x - coordinate)	27,88 m					
Front support lever arm	2,58 m			Load cell calibration factor 0,8		
Rear support lever arm	1,8 m			(*)		
	Front Support Readings				Rear Support Readings	
	CLL 1500012 hydraulic cilinders		HCL-1006 hydraulic cilinders		KOSD-RA2275 2,5 MN Load Cell	
	Display pressure (bar)	Load (ton)	Display pressure (bar)	Load (ton)	Display (ton)	Estimation (ton)
Record N.1	125	260	500	130	138	173
Record N.2	135	281	445	116	155	194
Record N.3	141,5	295	385	100	174	218
	Total weight (ton)	Rear Inside Reaction (ton)	Pext (ton)	Pint (ton)	YG (m)	
	655	92	302,679	352,321	0,522	
	655	64	309,609	345,391	0,516	
	655	42	317,738	337,262	0,522	
Average measured value					0,520 m	
Analytical Estimation					0,469 m	
(**) Deviation					-0,05 m	
(*) Note: Site calibration performed by load cell supplier						
(**) Negative deviation from the safe side						

Figure 23: Assessment of CoG y-coordinate through field measurements for right girder (results)

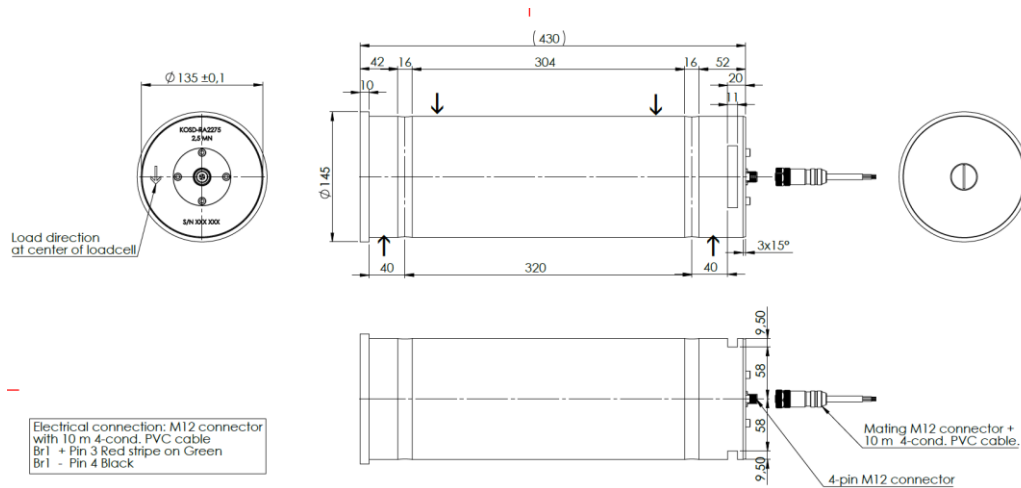


Figure 24: Load Cell installed in sliding supports balance beam (outer support)

6.3.1 Wind speed

With the MSS stationary and unloaded, wind action must be determined in accordance with current regulations for permanent structures, considering all parameters that affect wind speed. For short-term applications, a Return Period correction factor may be applied.

Conversely, during the launching stage, the maximum wind speed is an operational limit defined by the designer or manufacturer. For this specific project, a maximum allowable gust of 43.2 km/h during the launching stage was established [12].

Compliance with this criterion requires the following measures:

- **On-site monitoring:** Continuous wind speed tracking using calibrated anemometer.
- **Forecasting:** A 48-hour wind forecast to ensure operational limits are not exceeded during the launch window.
- **Emergency stabilisation:** Provision for stabilising the MSS through bracing systems in case of unpredictable wind scenarios.
- **Safe-stop protocol:** A formal protocol to halt or withhold launching if a sudden increase in wind speed is detected or suspected.

To mitigate ambiguity regarding the last measure, daily data from the Sabiñánigo Meteorological Station [13] for the year 2023 were analysed.

A statistical variable was established based on the maximum gust amplitude between two consecutive days.

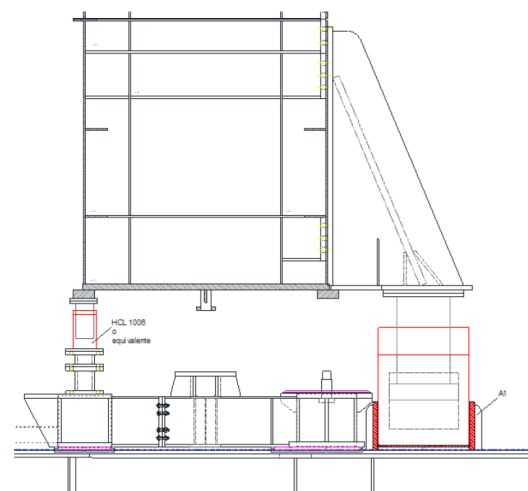


Figure 25: Individual girder front support set-up adopted in P9 for CoG y-coordinate field measurement

This analysis provides the objective basis for the launch of "go/no-go" decisions, as summarised in Table 1.

Based on the statistical analysis of the data provided in the table, the probability of the maximum wind gust increasing by more than 26 km/h between two consecutive days is less than 5%. To ensure an adequate safety level during the MSS operations, this statistical margin was used to define a conservative threshold for commencing the launching process.

Consequently, a maximum wind gust of 15 km/h—calculated approximately as $(43.2 - 26) / 1.1$ —was established as the mandatory limit for initiating any launching manoeuvre. This operation buffer ensures that even in the unlikely event of a sudden significant increase in wind speed, the resulting gusts would likely remain under the operational limit of 43.2 km/h.

Maximum wind speed variation range (Gusts) between consecutive days – 2023 Data															
Cumulative frequency	0%	31%	43%	55%	61%	74%	79%	84%	86%	90%	92%	93%	95%	97%	97%
Cumulative observations		74	105	132	147	180	191	204	208	218	222	226	229	234	234
Recorded days / Range		74	31	27	15	33	11	13	4	10	4	4	3	5	0
Wind speed range (m/s)	0	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-24	24-26	26-28

Table 1: Maximum wind gust amplitude between two consecutive days – 2023 statistical data [13]

6.3.2 Counterweights

In the Sabiñánigo application, and within its base configuration for standard spans, the Zamora II MSS incorporates 45 tonnes of counterweight per main girder [14]. This ballast is strategically positioned to offset the inherent transverse eccentricity of the underslung typology (Figure 26).

The primary objective of the counterweights is to reposition the Centre of Gravity (CoG) of the MSS main girders to ensure equilibrium during launching manoeuvres. Additionally, counterweights can have a favourable effect on the internal force distribution within the primary structure.

As a general rule, the minimum necessary amount of counterweight should be used, provided that both equilibrium (EQU) and structural strength (STR) criteria are met. Depending on the accuracy of the characterisation of the MSS weight and CoG, as well as the nose rotation angles required for each specific geometry, additional ballast may be necessary.

Consequently, a span-by-span analysis is essential to define specific launching kinematics, which must then be integrated into the formal operational procedures.

Therefore, the amount of counterweight and the nose rotation are intrinsically linked. In the Sabiñánigo application, this correlation was synthesised into operational tables, as presented in Figure 27.

6.3.3 Launching Kinematics

The maximum nose rotation is required in spans with the minimum plan radius. In the Sabiñánigo Bridge, the launching to Stage 7 reached the onset of the minimum plan curvature, while the launching to Stage 8 was performed with the MSS fully positioned within a circular curve of 1015 m radius, as illustrated in Figure 16.

Regarding the kinematics, the transition to Stage 7 required a 5-degree rotation of both the front and rear noses, whereas launching to Stage 8 was

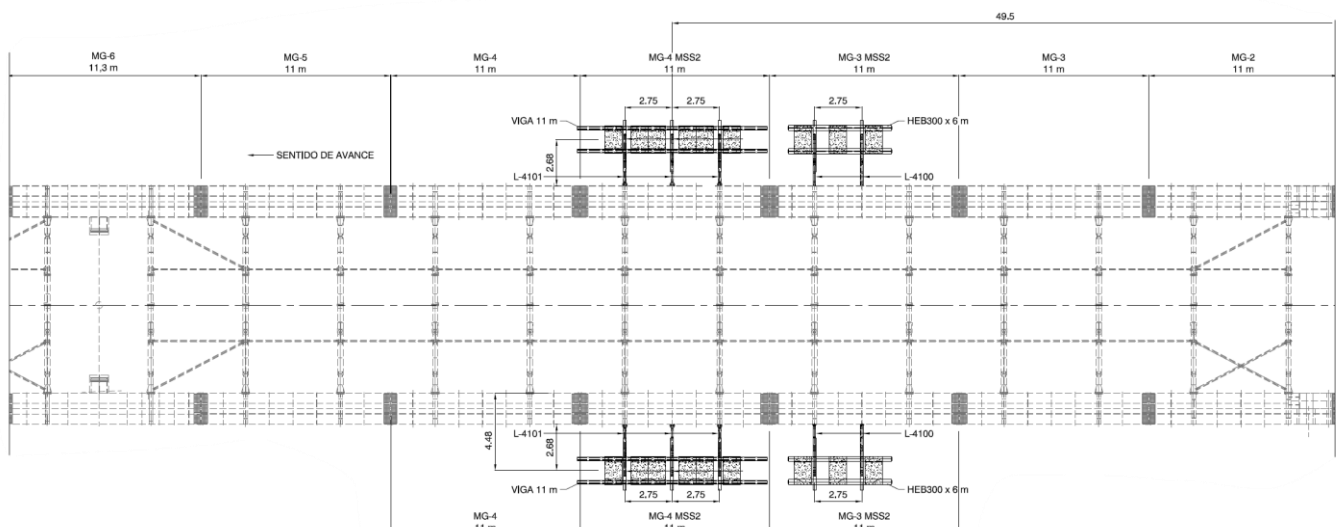


Figure 26: Zamora II base counterweight configuration – plan view [R14]

conducted with a 5.50-degree rotation, nearing the MSS physical operational limit of 6 degrees.

Launching to Stage 8 was more demanding than its precedent operation. Representative steps in right-side girder launching, with summary descriptions, are presented in the following figures.

Launching operations begin by lowering the MSS to de-mould the structure from the deck's weight. Subsequently, the transversal structures are disconnected and partially rotated. This rotation serves a dual purpose: it optimises the CoG y-coordinate, thereby enhancing stability, and enables a shorter transversal sliding stroke on the supporting brackets to achieve the necessary clearance for pier passage.

For the right-side launching to Stage 8, the initial sliding movement was set at 4.57m for the middle bracket (P4) and 4.93m for the rear bracket (P5). These initial sliding movements were governed by the following criteria:

- **Stroke Limitation:** Avoiding the full sliding stroke on the middle bracket (P4) to limit local loads transmitted to the pier walls and to reduce internal forces on the brackets. This prevents the concomitance of maximum sliding stroke, maximum vertical loading, and maximum longitudinal loading.
- **Symmetry and Balancing:** Balancing the sliding strokes of the left and right-side girders. A maximum offset of 1.00m was established for the most heavily loaded bracket to limit the longitudinal (x-axis) bending moments transmitted to the pier.
- **Geometric Alignment:** Maintaining the main girder alignment approximately perpendicular to the bracket axis, with a maximum allowable variation of 2.5 degrees.

Following the initial sliding movement, launching proceeds with staggered longitudinal movements of the right and left-side girders. The sequence is established so that the maximum longitudinal offset between the two girders is approximately 12.5 m, and no girder remains stationary at its maximum cantilever position.

Once the maximum front cantilever position is reached (Figure 30), the main girder is rotated in plan to increase the available stroke at both the rear (P5) and front (P3) supporting brackets.

		RIGHT SIDE GIRDER - FRONT NOSE ROTATION (°) ¹⁾²⁾												
		0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	5,5	6
RIGHT SIDE GIRDER - REAR NOSE ROTATION (°) ¹⁾²⁾	0	0	0	0	0	0	0	0	0	0	0	4	4	8
	0,5	0	0	0	0	0	0	0	0	0	0	4	4	8
	1	0	0	0	0	0	0	0	0	0	4	4	8	12
	1,5	0	0	0	0	0	0	0	4	4	8	8	12	12
	2	0	0	0	0	0	0	4	4	8	8	12	12	16
	2,5	0	0	0	0	0	4	4	8	8	12	12	16	20
	3	0	0	0	0	4	4	8	8	12	12	16	20	20
	3,5	0	0	0	4	4	8	8	12	12	16	20	20	24
	4	0	0	4	4	8	8	12	12	16	20	20	24	24
	4,5	0	4	4	8	8	12	12	16	20	20	24	24	28
	5	4	4	8	8	12	12	16	20	20	24	24	28	28
	5,5	4	8	8	12	12	16	20	20	24	24	28	28	32
	6	8	8	12	12	16	20	20	24	24	28	28	32	32

1) Right side girder in the launching direction

2) Positive orientation in the inward overturning direction

Figure 27: Correlation between nose rotation and required additional counterweights – Right Girder [9]

Concurrently, the position of the bogie on the front bracket (P3) is adjusted in preparation for the skate entrance.

Ideally, this in-plane rotation should be performed in a configuration with only two vertical support alignments. This strategy mitigates hyperstatic forces that may arise from potential delays or misalignments in the horizontal jacking (sliding) strokes. Furthermore, the skate entrance into the front bogie is a critical operation requiring close-range supervision.

The next stage requiring stringent control is the exit of the launching nose skate from the rear bogie (P5), which returns the system to a configuration with two vertical support alignments. This transition coincides with the maximum rear cantilever position (see Figure 31).

As a general rule, the MSS does not remain stationary in this position; instead, it proceeds for several additional metres of launching until a new parking point is reached. At this stage, another in-plan reorientation of the main girder is performed—once again with two support alignments—to further increase the available stroke on the front bracket (P3), as shown in Figure 32.

The next control point occurs during the transition from the launching nose to the main girder at the front support (P3), reached at an accumulated launching distance of approx. 68m (see Figure 33).

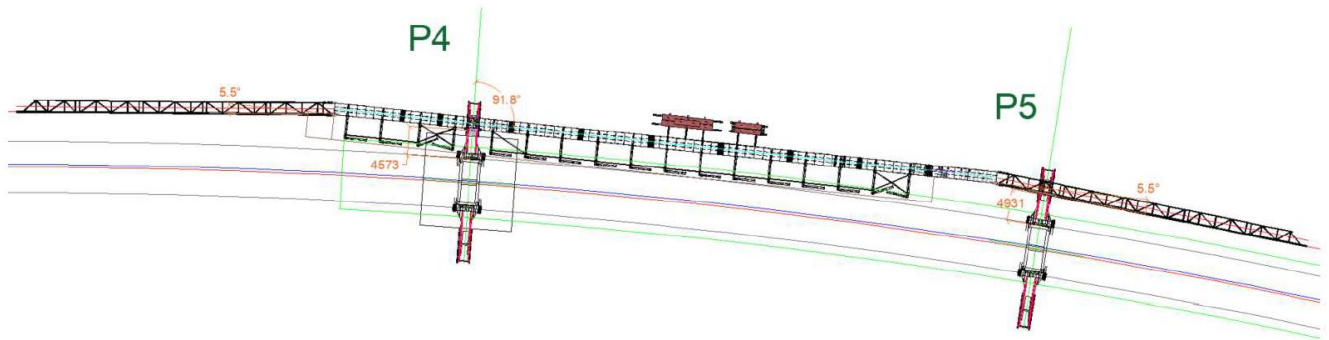


Figure 28: Right-side girder launching to stage 8 – initial transversal sliding movement

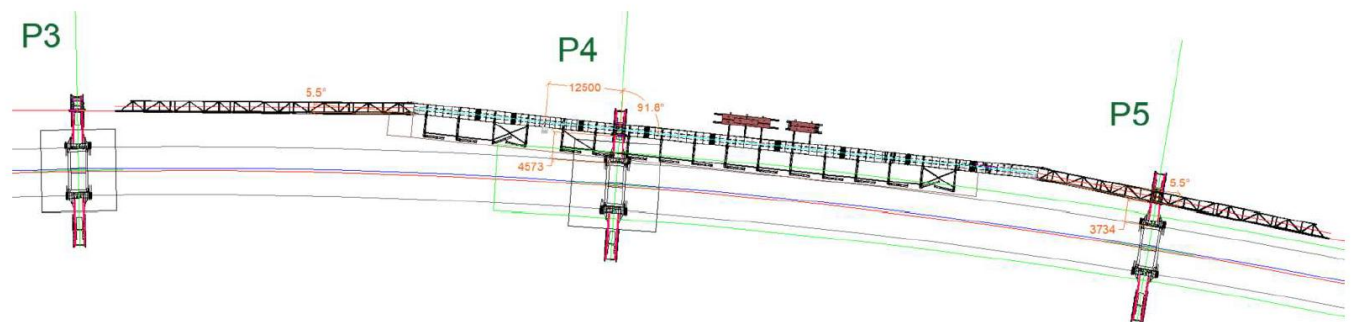


Figure 29: Right-side girder launching to stage 8 – accumulated launching distance of 12,5m

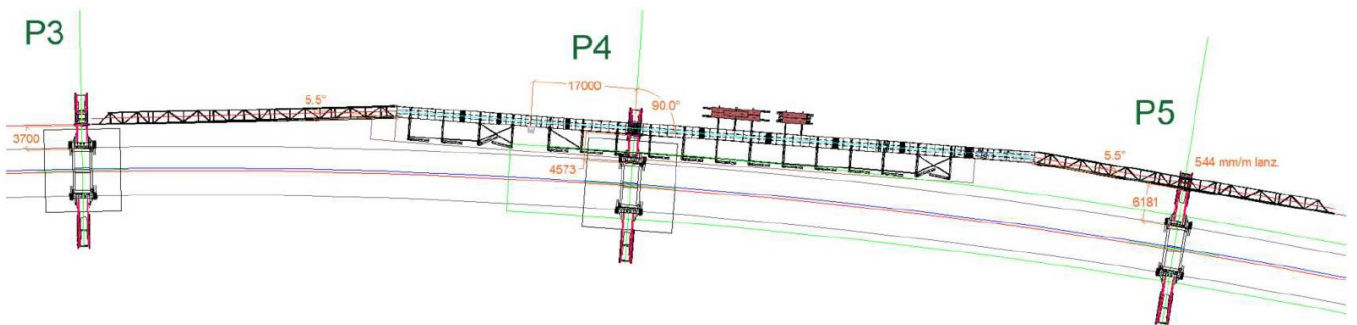


Figure 30: Right-side girder launching to stage 8 – plan rotation in maximum front cantilever position

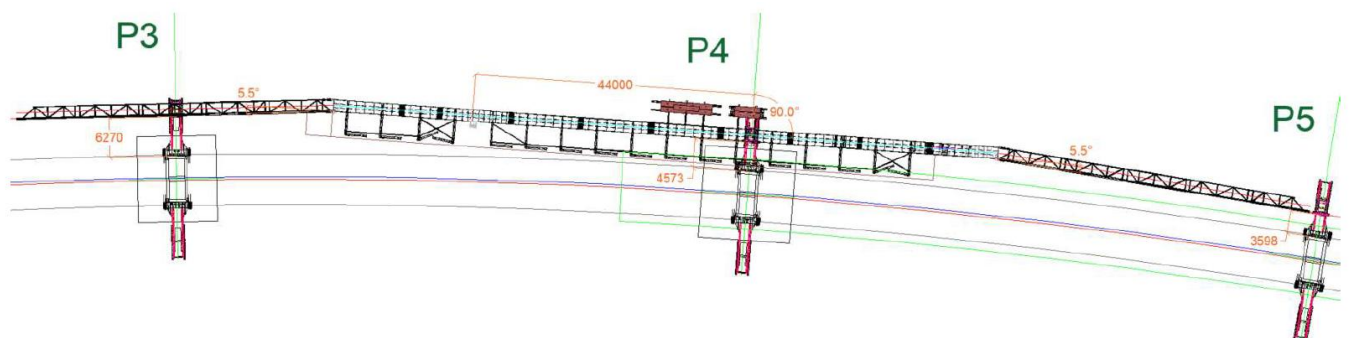


Figure 31: Right-side girder launching to stage 8 –maximum rear cantilever position

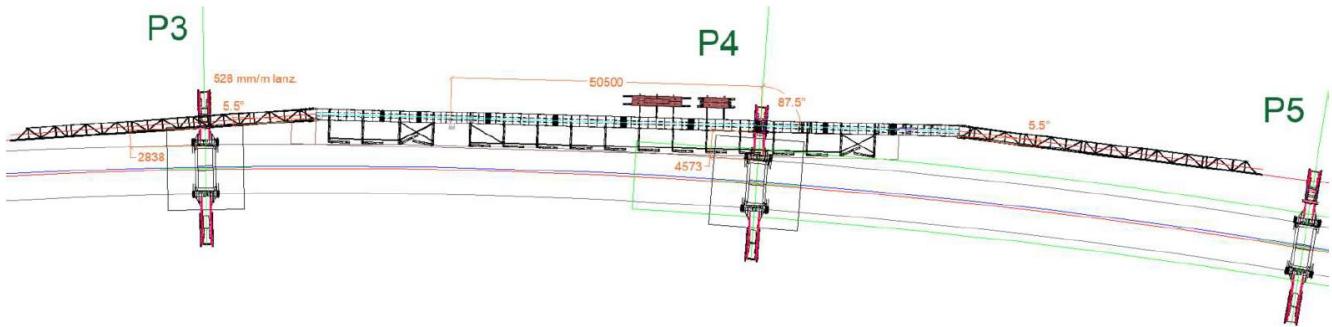


Figure 32: Right-side girder launching to stage 8 – plan rotation with accumulated launching distance of 50.5m

At this stage, a momentary reorientation of the front nose may be required to ensure that the rail joint discontinuity and its angular break (kink) do not lead to potential collisions or blockage within the bogie's lateral guides. This transition demands close supervision.

Launching then continues until the load on the front bracket (P3) exceeds that on the middle bracket (P4) — see Figure 34. At this juncture, the jacking system anchorage is transferred from the middle bracket to the front bracket.

As a general rule, it is beneficial to anchor the launching system to the most heavily loaded support to minimise the longitudinal forces transmitted to the pier.

By doing so, the traction force effectively counterbalances the friction and the force required to overcome the longitudinal slope at that specific support. Consequently, the anchorage should be positioned at the support offering the highest resistance to movement, which corresponds to the point of maximum vertical load.

The new locomotion jacking configuration is maintained until the completion of the launching process. Before the transition from the main girder to the trussed structure at the middle support (P4), the rear nose is oriented into a straight configuration (see Figure 35). This adjustment is intended to ease the passage of the rail joint over the sliding support, minimising potential interference during this transition.

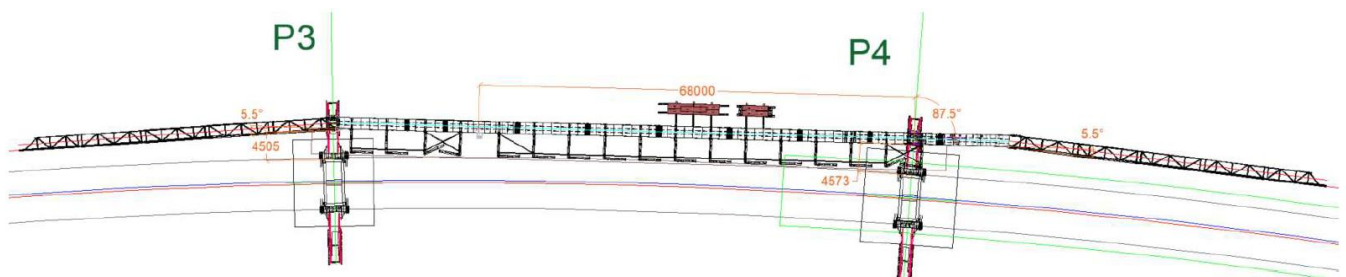


Figure 33: Right-side girder launching to stage 8 – accumulated launching distance of 68m

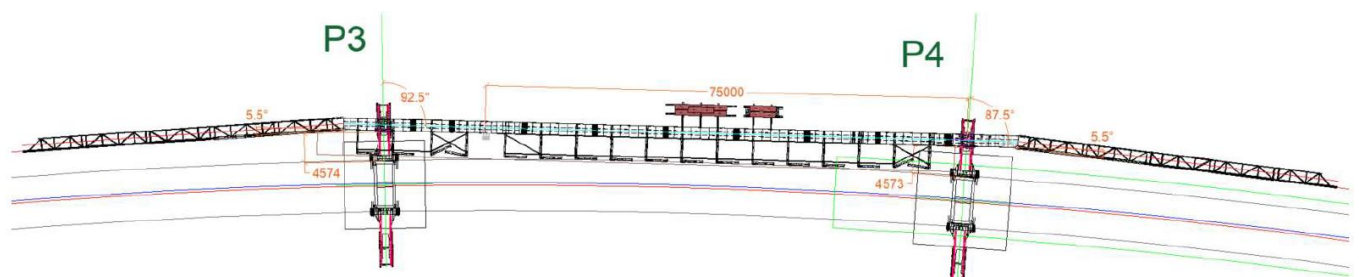


Figure 34: Right-side girder launching to stage 8 – accumulated launching distance of 75m and locomotion anchoring changing from P4 to P3

The launching process then proceeds to the final longitudinal position (Figure 36). Upon completion of the longitudinal movement, the girders slide towards each other (Figure 37), and the transversal structures are rotated and partially connected.

Fine adjustments are performed to ensure the MSS is precisely repositioned in-plan for the next span's construction. Finally, the connections between the transversal structures are fully tightened, and the MSS is jacked up to its definitive casting position.

The operation of the right-side girder with both noses oriented at 5.5 degrees while describing a left-turn trajectory required an additional 28 tonnes of counterweight.

This requirement is fully coherent with the correlation table presented in Figure 27.

The corresponding transversal overturning verification is detailed in Figure 38 [9].

6.4 Structural integrity of main girder webs

The rotation of the launching noses affects more than just overturning stability; it requires a concomitant displacement of the bogies, thereby altering the in-plan support configuration.

In the practical case study of the Stage 8 launch, the impact of nose rotation on the support reactions can be assessed by comparing the reaction values between the right- and left-side girders.

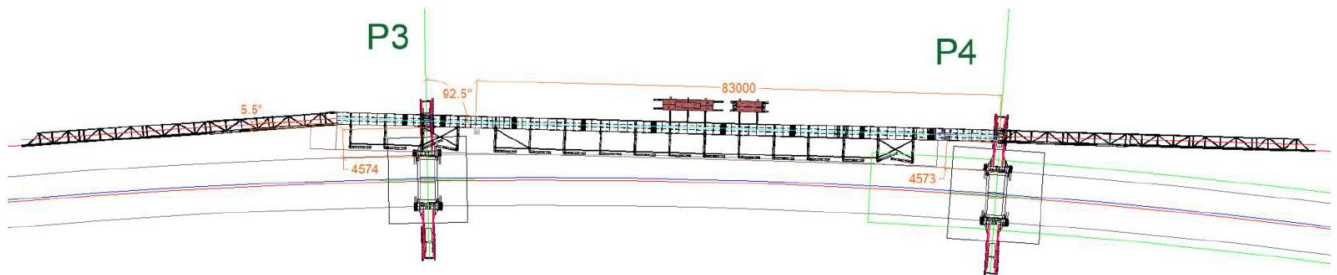


Figure 35: Right-side girder launching to stage 8 – accumulated launching distance of 83m and transition from box section to trussed nose at P4 support

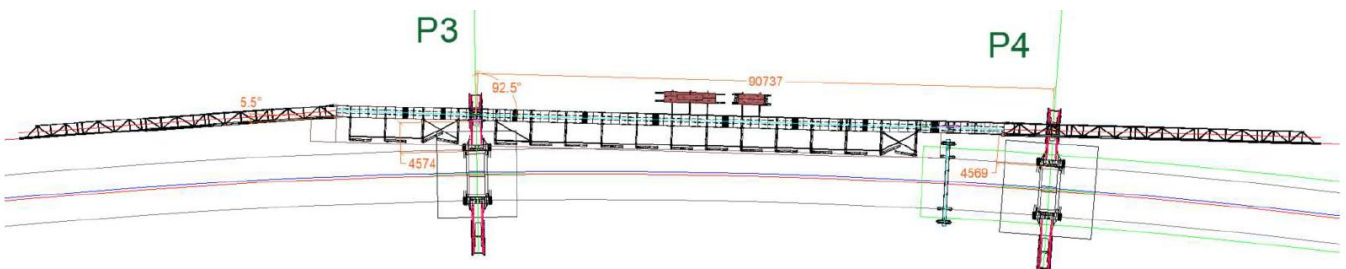


Figure 36: Right-side girder launching to stage 8 – final longitudinal position prior to transverse closing movement

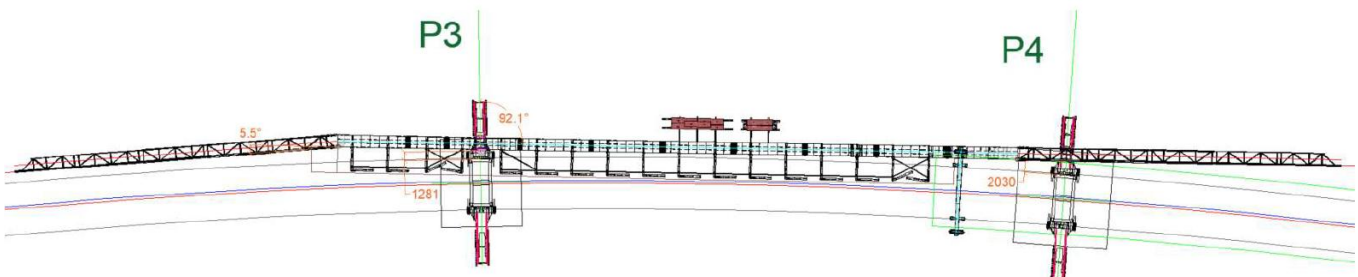


Figure 37: Right-side girder launching to stage 8 – final longitudinal position after transverse closing movement

ANALYSIS ACCORDING TO EUROCODE 0 USING PARTIAL SAFETY FACTORS OF 0.9 AND 1.1

Safety Factors	
Permanent Favourable	0,9
Permanent Unfavourable	1,1
Variable Unfavourable	1,5

	g	Q (kN)	zi (m)	My,base (kN.m)	Factor	M,stab (kN.m)	M,over (kN.m)
MG		3509,1	0,928	3257	0,9	2931	
FRONT NOSE	5,5	487,6	-1,561	-761	1,1		-837
REAR NOSE	5,5	487,6	-1,56	-761	1,1		-837
TB		689,5	-2,393	-1650	1,1		-1815
BOTTOM SLAB		517,2	-1,83	-944	1,1		-1039
WEB AND TOP PANEL		595,0	0,049	29	0,9	26	
CONTRAPESO DELANTERO		466,6	4,520	2109	0,9	1898	
CONTRAPESO TRASERO		278,2	4,505	1253	0,9	1128	
VIENTO 43,2km/h				-951	1,5		-1426
SOBRECARGA 20 kN		20,0	-1,8	-36	1,05		-38

160	Additional
120	counterweights
280	Total

5983 -5993

M,over / M,stab 0,998 OK
Acceptance Criteria > 1,00

Wind Speed (kph) 43,2

Figure 38: Right-side girder launching to stage 8 – transversal overturning stability check

The peak launching reaction occurs in the left-side girder, reaching 7780 kN at an accumulated launching distance of approximately 30m (within a statically indeterminate configuration).

Conversely, the maximum reaction recorded for the right-side girder is 6400 kN, a significantly lower value. Although peak reactions occur in the inner webs (closest to the bridge axis), the outer webs must not be overlooked; in the right-side girder, the maximum reaction is 4405 kN, while in the left-side girder it is 2594 kN.

Given that the outer web is thinner (12 mm versus 14 mm for the inner web) and the outer support length is shorter (1.5 m versus 2.5 m for the inner

support), identifying the most critical scenario is not straightforward. Consequently, all load cases must be rigorously analysed to ensure structural integrity across all web configurations.

The most demanding scenarios described above are presented in Figures 39 and 40. One of the most critical failure mechanisms is web buckling under patch loading (local concentrated loading) from the sliding supports.

Regarding this phenomenon, it is important to note that joint sections are particularly vulnerable. Information from FE models regarding critical load factors and buckling mode configuration must be combined with the design methodology provided by EN 1993-1-5 [15].

	Maximum loads transmitted to the web by sliding supports (ULS launching combination – stage 8)	
	Inner Web (1)	Outer web (2)
Righ-side	6400 kN	4405 kN
Left-side	7780 kN	2594 kN
1) Web thickness = 14mm and sliding support length = 2.5m 2) Web thickness = 12mm and sliding support length = 1.5m		

Table 2: Maximum loads transmitted to the web by sliding supports (ULS launching combinations – Stage 8)

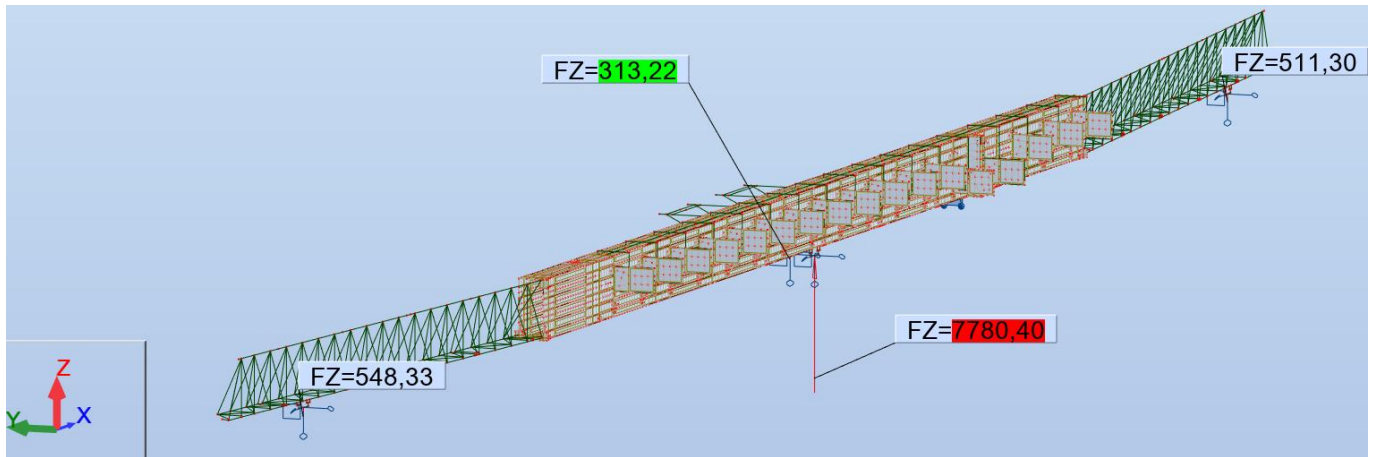


Figure 39: Left-side girder launching to stage 8 – ULS maximum reaction with accumulated launching distance of 30m

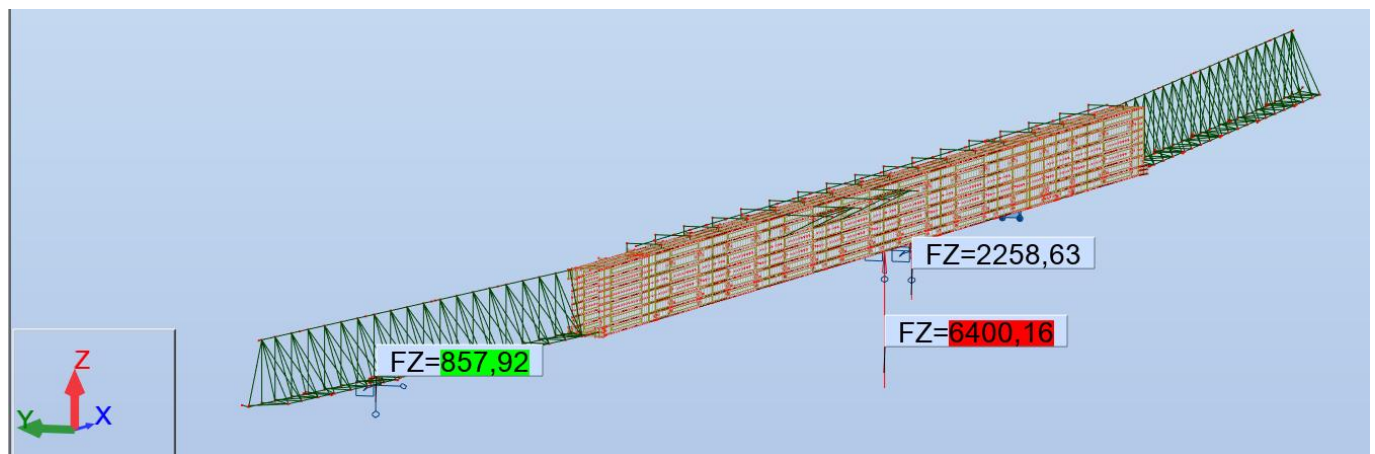


Figure 40: Left-side girder launching to stage 8 – ULS maximum reaction in maximum front cantilever position

6.5 Vibrations and Deflection Amplification in Maximum Cantilever Positions

Beyond its impact on transversal overturning stability, crosswind action during launching is of significant concern during maximum cantilever conditions—a risk further enhanced in this case by the unprecedented weight and scale of the MSS.

Within the bridge construction technical community, some authors propose the need for advanced studies on the dynamic effects of wind during launching for cases where the frequencies associated with transverse vibration modes at maximum cantilever are below 0.20 Hz.

This threshold is primarily empirical, established from various applications involving different types of MSS [16, 17].

However, a representative sample for an MSS of this magnitude does not yet exist. While an analogy could be drawn with mast-type structures, the wind force patterns involved are fundamentally different.

In this case, rather than the second-order effects resulting from oscillatory movement, the primary concerns are the Centre of Gravity (CoG) oscillation and the forces introduced into the hydraulic system that controls the nose rotation. Under extreme conditions, such oscillations could induce the opening of the pilot-operated pressure relief valves, potentially compromising the nose's fixed position.

Furthermore, and in contrast to mast-type structures, motion-induced vibrations and oscillatory movements were clearly visible, especially during the largest cantilever configurations.

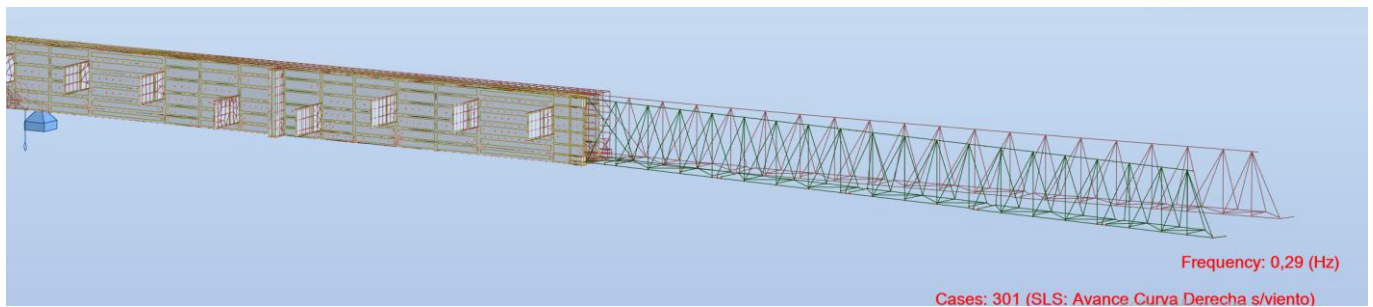


Figure 41: Minimum frequency vibration mode ($f = 0.29$ Hz) in maximum front cantilever position

In the Sabiñánigo application, following the calibration of the numerical model with inputs from weight (mass), Centre of Gravity (CoG), and stiffness measurements (determined by deflection under static loading), a numerical assessment of the launching vibration modes was performed.

The frequency associated with the primary transversal vibration mode at the maximum front cantilever is approximately 0.29 Hz according to the numerical model (see Figure 41).

Although accelerometers were installed to monitor the dynamic response (see Figure 42), the processed data were not available prior to the first maximum span launching.

Consequently, the safety validation relied on the numerical assessment of the calibrated model.

The convergence between the static stiffness measurements and the predicted frequency of 0.29 Hz provided the necessary confidence to proceed with the operation, ensuring that the dynamic amplification remained within controllable limits.

Nevertheless, rather than relying solely on the theoretical vibration frequency, a proactive risk management approach was adopted. Special care was exercised during maximum cantilever stages, with increased focus on the initial launchings.

To maintain the operation under rigorous control, the following specific operational measures were implemented:

- **Adjustment of the nose geometry:** Ensure the nose bears shortly after passing the front bogie (Figure 46). This is achieved by adjusting the nose's geometry altimetrically prior to launching, taking into account actual support levels provided by the field survey.



Figure 42: Accelerometer installed at nose tip

- **Sequential launching protocol:** Defining the launching sequence to avoid having both main girders in the maximum cantilever position simultaneously. First, the girders would park at approximately an 80m cantilever in an enhanced stability configuration.

Then, each girder is launched alternately, only parking again after the front support was reached and loaded, placing the girder in a 3-support alignment, which is a configuration significantly less vulnerable to wind action.

- **Mandatory supervision:** Ensure mandatory supervision of the skid entry and exit points on the bogies.
- **Constant monitoring:** Implement constant monitoring of wind speed and direction using a calibrated anemometer.
- **Tighter wind limits in vulnerable conditions and check control for subsequent launching steps:** Apply stricter limits for admissible wind speed during the sensitive approach phases for bogie entry and exit (**typically limiting speed to 20 km/h** during these moments). Care must be taken not to be overly restrictive, ensuring a compliant and practical response from the operation team.
- **Reverse movement capability:** The MSS must be prepared for the eventual need to reverse its movement and park in a safer position during the most vulnerable stages. This is a fundamental safety feature that is not always a mandatory requirement for this type of equipment, yet it proved essential for risk management in this project.
- **Enhanced lubrication of rails and sliding plates maintenance:** Thoroughly lubricate the launching rails to reduce motion-induced vibrations. Throughout the operation, the benefit of performing transverse shifting movements while locomotion was in progress became apparent, benefiting from a dynamic (lower) friction condition that effectively reduced vibrations and oscillatory deflections induced by transverse bogie movements.

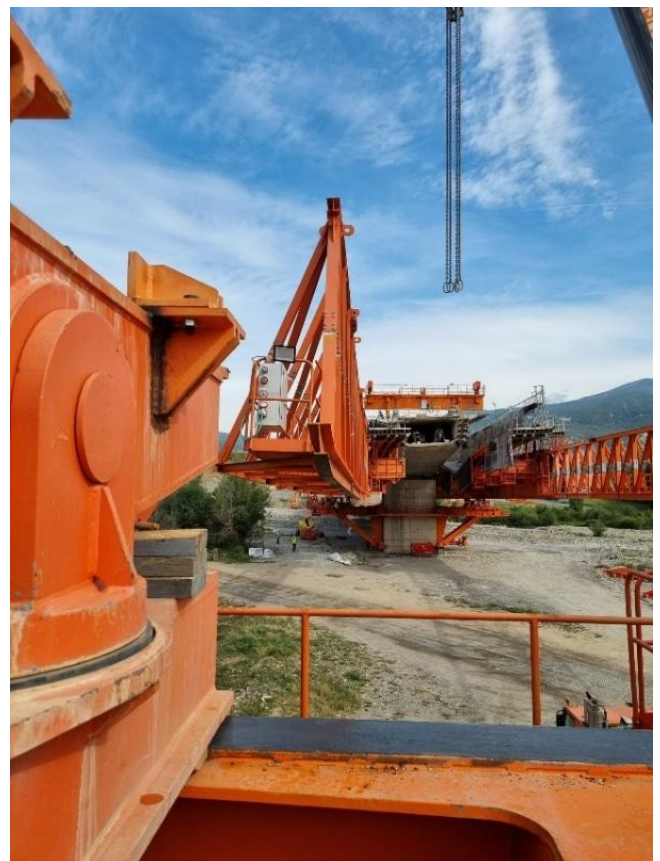


- **Checking rail joints to avoid sudden changes in locomotion forces:** Implement routine checks of rail joints to ensure smoothness and avoid sudden changes in locomotion forces, with the core purpose of minimising motion-induced vibration.

During the first maximum cantilever launching, as an additional precautionary measure, a crane was placed on stand-by. Its purpose was to provide a short-cable connection in the event of excessive oscillation, thereby increasing the frontal cantilever's stability.

As the operation progressed, the team gained greater sensitivity to the structure's movement and response, and no alarming situations resulting from this phenomenon were observed.

Figures 43 and 44 illustrate the approach and entry of the launching nose into the front bogie during the launching stage.



← Figure 43: Supervision at front skate entrance

↑ Figure 44: Front skate approach - view from front bracket



↑ ↗ Figures 45 and 46: Launching to Stage 10

→ Figure 47: Incremental launching for Main Girders disassembly after completion of last span with MSS

7. LAST SPANS AND MSS DISASSEMBLY

The final operations of the project, comprising the launching of Stages 10 and 11, the casting of the closing span, and the subsequent MSS disassembly, represented a demanding engineering challenge. Given the unprecedented scale of Zamora II, these final stages were treated as standalone projects, requiring extensive structural analysis and logistical planning that exceed the scope of this publication.

Figures 45 and 46 illustrate the final launching stages, while Figures 47 and 48 provide a glimpse into the incremental launching strategy specifically adapted for the main girders' disassembly. To maintain the technical depth required for such complex operations, a detailed account of the dismantling procedures and the final span's structural behaviour will be presented in complementary forthcoming publications. This ensures that the pioneering lessons learned during the closing stages of the Sabinánigo Bridge are shared with the technical community in their full complexity.



Figure 48: Incremental launching for Main Girders disassembly after completion of last span with MSS – last steps

CONCLUSIONS

The execution of the Sabinánigo Bridge represents a milestone in the field of bridge engineering, pushing the operational boundaries of underslung Movable Scaffolding Systems to a 90m span record. However, the primary takeaway from this project is that an increase in scale is not merely a linear expansion of existing methods, but a transition into a domain where new, non-negligible phenomena emerge.

The success of the Zamora II operation was fundamentally rooted in the systematic elimination of uncertainty. This was achieved through the development of integrated models that evolved from a theoretical framework into a calibrated operational tool. The continuous process of on-site data collection, engagement with the operation crew, and numerical model updating proved more than once that even the most advanced numerical models require empirical validation and a strict operational framework to manage the risks inherent in large-scale construction.

The technical challenges addressed demonstrate that the designer's role must extend deep into the construction site. Proactive risk management, characterised by a "search for the unknown" and the establishment of conservative, data-driven operational thresholds, ensured that safety was never compromised by the complexity of the alignment or the magnitude of the loads.

Finally, it is urgent to enhance the importance of models as a vital technical legacy. By maintaining a consistent and coherent data structure throughout the project's life cycle, integrated models ensure that the pioneering expertise gained in projects such as Sabinánigo is preserved for future applications.

Acknowledgments

The authors wish to express their gratitude to all partners involved in this project, with particular thanks to the Client, the Main Contractor (UTE Grupo Rover - Aldesa), and the bridge designer (IDEAM, TYLin LATAM), whose collaboration was essential for the execution of such a singular project with remarkable safety standards.

Furthermore, the authors would like to extend a special thanks to the operations team; in particular to Avelino Abal, Valentin Albenda, Sergio Nine, and Victoria Misol, among many others. Their trust in the engineering team, the sharing of invaluable technical insights, and their pioneering execution of the launching operations were fundamental to the success of this project.

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THE 'ROBINSON' CABLE-STAYED CYCLE AND PEDESTRIAN BRIDGE IN BUDAPEST, HUNGARY

Gábor Pál, András Kemenczés, Mihály Szabados
SPECIÁLTERV, Hungary

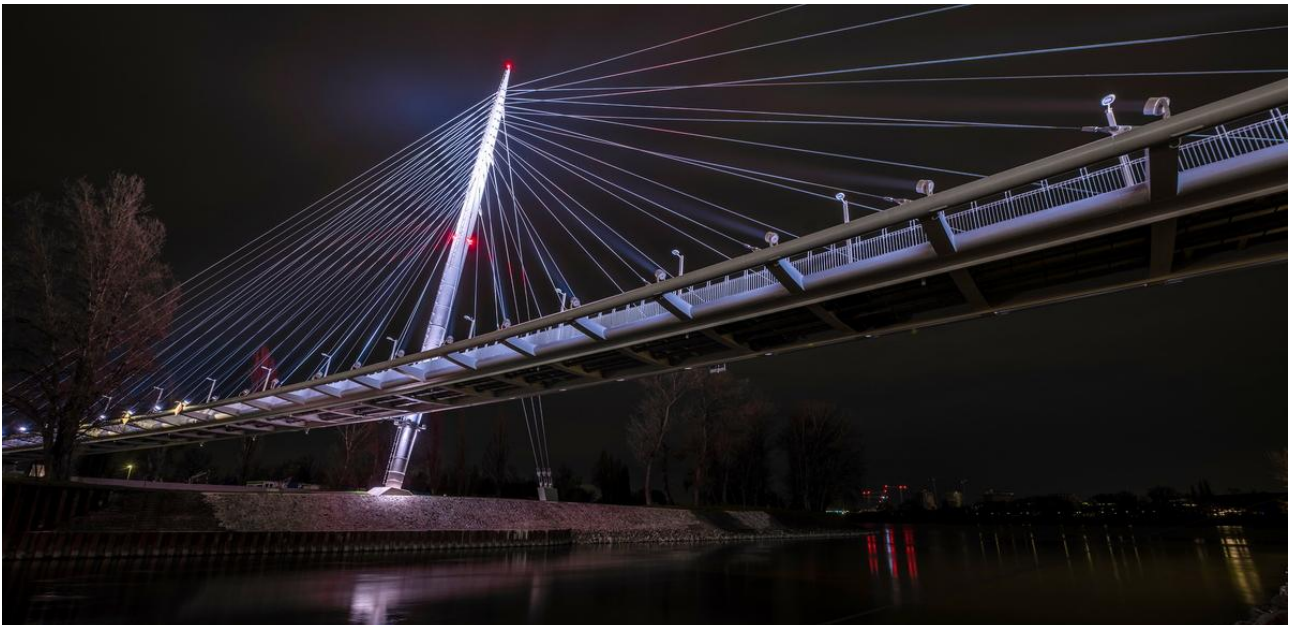


Figure 1: The Robinson Bridge is a new pedestrian & cycle bridge along the Danube

INTRODUCTION

The Robinson Bridge is located in Budapest, Hungary's capital. The bridge was constructed as a key element of the National Athletics Centre project, a brownfield investment which transformed a formerly neglected waterfront into an easily accessible and high-quality public space. It spans the Ráckevei Danube Branch, connecting Central Pest to Csepel Island at the bank of the Danube.

As a structure designed exclusively for pedestrians and cyclists, the bridge plays a vital role in creating access to the Danube waterfront between the city centre and Csepel Island. The bridge follows a curved path along the Danube with its pylon standing on a small island.

The uniqueness of the bridge lies in the absence of any fixed intermediate support beneath the deck. The curved superstructure spans between the abutments without bearing on the island; suspended from the pylon solely by the stay cables, it appears to float above the Danube.

The 168 m long bridge with the 65 m tall pylon is the largest in Budapest and Hungary designed exclusively for pedestrians and cyclists.

The novel bridge features an innovative design utilising concrete-filled steel tubular (CFST) members. Its execution required advanced construction methodologies, as the system shapes an intricate structural behaviour under load.

CONCEPT, CONTEXT

Aesthetic quality was a central consideration throughout the project. The goal was to create a bridge structure that does justice to Budapest's celebrated cityscape, the Danube waterfront, and the new Stadium complex — both by local and international standards.

The pylon is elegantly slim and needle-like, suspending the bridge deck without any fixed intermediate support to achieve a floating effect. The transparent cable network reveals a new and exciting experience from every angle, focusing the user's attention towards the sky.

Designing the bridge alongside the stadium presented a fundamental choice between similarity and differentiation. Ultimately, we decided on contrast, with the stadium's curved and flat mass opposing the bridge's pointed, angular and tall yet transparent structure. While the two facilities allude to each other in their details, they complement each other perfectly as a whole.

The bridge is a one-pylon cable-stayed pedestrian and cycle bridge with a hybrid steel-concrete stiffening girder and an orthotropic steel deck.

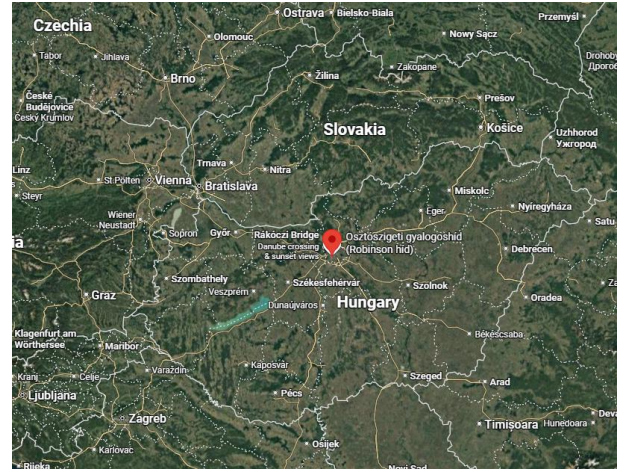
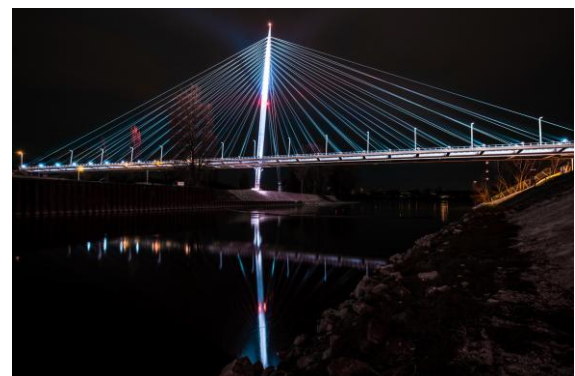
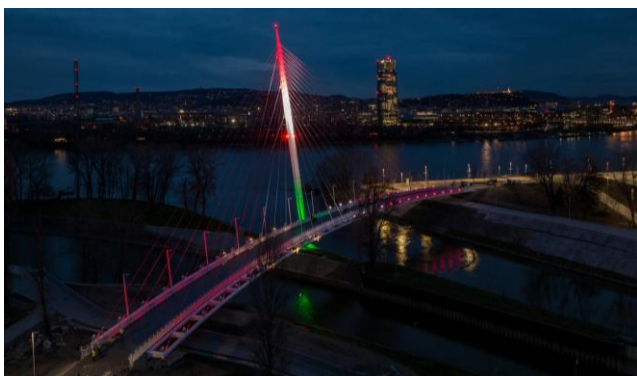


Figure 2: Location of the bridge
Click on the image to open Google Maps

Client	BMSK
Lead designer	Gábor Pál, SPECIÁLTERV
Design team	Mihály Szabados, Péter Pataki, András Kemenczés, László Hunyadi, András Majerszki, Viktória Ódor, SPECIÁLTERV
Authors	Gábor Pál, András Kemenczés, Mihály Szabados, SPECIÁLTERV
Main contractors	ZÁÉV Co. - Magyar Építő JV
Construction	Hídépítő
Steel structure fabrication	Acélhidak
Design	2018-2020
Construction	2021-2022

← Figure 3: The bridge deck is suspended from the pylon without any fixed intermediate support



Figures 4 and 5: The bridge was designed to become a scenic addition to the Danube waterfront in Budapest

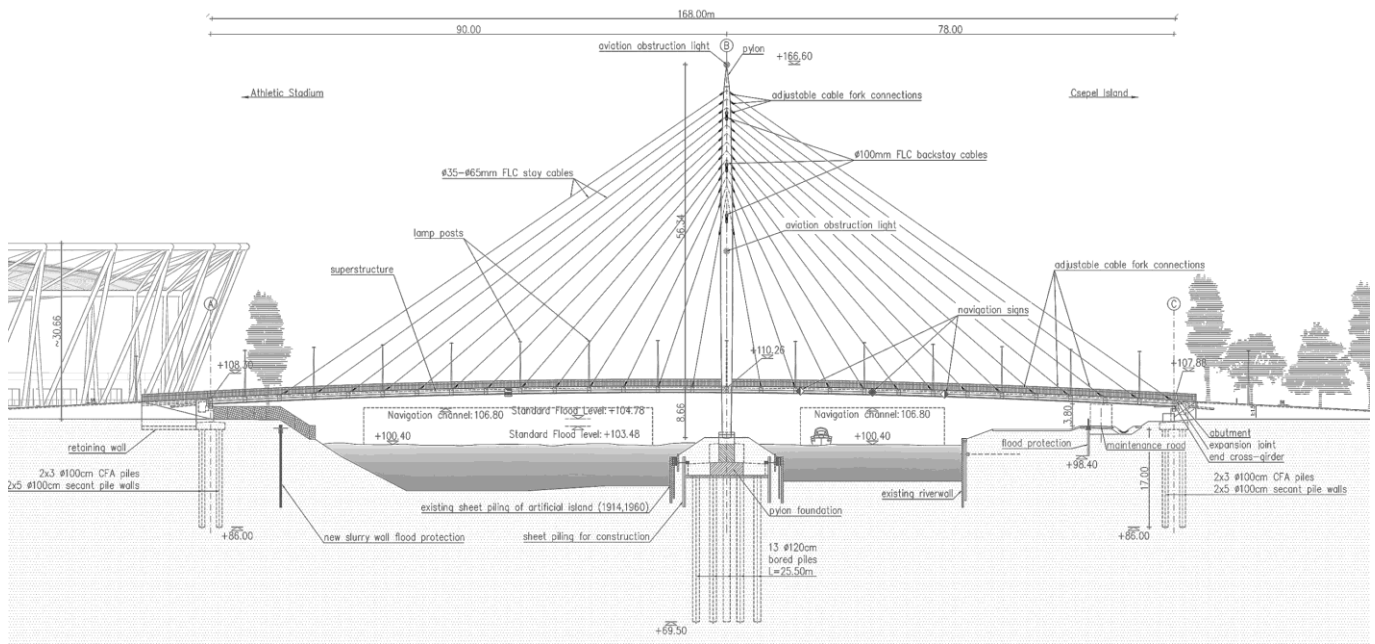


Figure 6: General side view

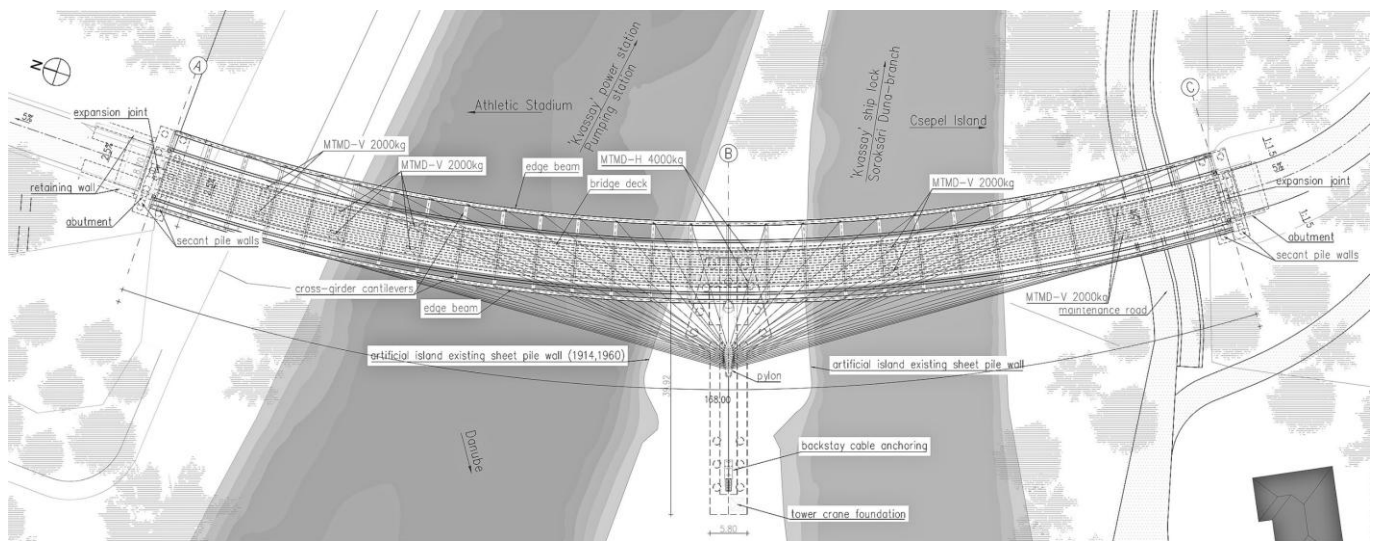


Figure 7: General top view

Span	168.00 m without intermediate support
Bridge length	168.90 m
Width of pedestrian path	7.00 m
Total width	12.71 m
Height of the pylon	65.00 m
Steel structure	1020 t S460 ML and S355 M
FLC cables	3500 m

GENERAL ARRANGEMENT

Due to the site constraints, the superstructure is arranged in a curve with a radius of 275 m. The free width of the pedestrian path is 7 m, and the total width of the superstructure is 12.71 m.

The 65 m tall steel pylon is located on an artificial island that divides the Danube branch. Its geometry was designed to be highly slender; the circular cross section has a maximum diameter of only 2 m, which tapers conically upwards and downwards. The pylon is inclined at an angle of 9°.

The total bridge length is 168.90 m; the free span measured along the curved bridge axis is 168 m, as shown on the general arrangement drawings. The pylon divides the span unequally into 90 m + 68 m parts, but without providing any fixed intermediate support for the deck.

The superstructure is suspended with a two-plane cable system consisting of 53 FLC stay cables. The cable network nodes on the main girder are distributed at 6 m intervals aligned with the radially placed crossbeams, and with a spacing of 1.50 m on the pylon. The cables are arranged symmetrically in a fan shape.

The pylon is stabilised by three backstay cables; their anchoring is located 25 m from the pylon.

GEOTECHNICAL ENVIRONMENT

For the purpose of evaluating the soil and groundwater conditions at the project site during the design phase, a total of two 25m and one 30m deep large-diameter dry auger boreholes, two static cone penetration tests and one dynamic penetration test were carried out.



Figure 9: Cantilevered cross-beams were designed to ensure adequate pedestrian clearance on the deck

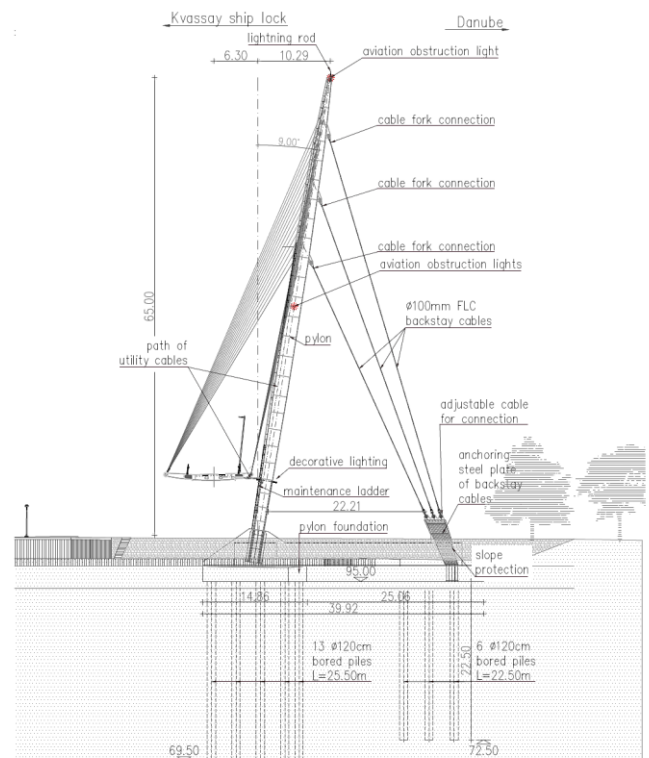


Figure 8: Pylon view

The regional ground profile is characterised by significant anthropogenic fill overlying the historical floodplain surface.

Within the investigation area, fluvial Danube terrace deposits—comprising gravel, sandy gravel, and gravelly sand—overlie the Oligocene substratum. This terrace stratum exhibits high bearing capacity, is typically in a dense state, and possesses high hydraulic conductivity.

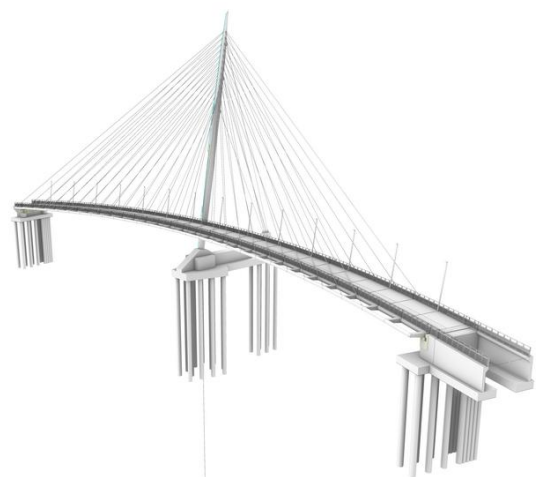


Figure 10: Model of the Bridge, Rhino & Grasshopper

Applied deep foundations

The foundation of the abutments was constructed using Continuous Flight Auger (CFA) piles with a diameter of 1 m, featuring lengths of 17 m at Abutment “A” and 22.5 m at Abutment “C”. Secant pile walls were incorporated beneath the abutments to resist substantial horizontal loads. Each wall segment comprises three primary and two secondary piles arranged at a centre-to-centre spacing of 75 cm. The foundation for Pier “B” (pylon), situated on the artificial island, utilises slurry-supported bored piles with a diameter of 1.20 m and an embedded length of 22.5 m.

To validate and optimise both the piling methodology and axial/lateral load capacities, static pile load tests were conducted:

- Vertical Capacity (Abutments): Verified using a single Ø1 m diameter, 18.0 m long CFA test pile.
- Horizontal Capacity (Secant Wall): Direct load testing of the complete secant pile wall was deemed unfeasible due to constraints in loading equipment capacity and hydraulic jack limits relative to the estimated lateral resistance. Consequently, horizontal testing was performed on an isolated 1 m diameter CFA pile; the response and lateral displacement of this individual pile were used to derive the structural capacity of the secant wall.
- Pylon Foundation: Successfully verified via a Ø1 m diameter, 38.5 m long cast-in-place test pile installed using bentonite/polymer drilling fluid prepared on site.

According to the seismic zonation map for Hungary, the project site is located within Zone 4. Pursuant to EN 1998-1 (Eurocode 8), the reference peak ground acceleration on rock is $a_{gR}=0.14g$.

Although the regional seismic microzonation map for Budapest initially suggests Ground Type “C”, detailed seismic probe testing refined this site response parameter. Consequently, the subsoil was reclassified as Ground Type “B” for seismic design purposes.

ANALYSIS

The bridge is a curved and asymmetric cable-stayed structure. As a result, the 53 suspension cables and three backstay cables together balance the spatial structure.

The main girder is frequently supported in the vertical direction by the stay cables connected at every 6 m, and experiences moderate moments around the horizontal axis.

At the same time, the asymmetric cable forces cause a significant horizontal bending on the stiffening girder. The edge beams of the deck, which are more than 12 m apart from each other, resist this moment. However, this generates significant tension on the side of the pylon, while significant compression occurs on the opposite side.

The C40/50 concrete infill applied to the box girders and edge beams increases the inertia, but even so, more than 20 cm of horizontal displacement occurred during the tensioning of the cables.

The analysis was performed using several structural engineering programs. Detailed global analysis - including exact construction procedures, large displacements and time-dependent composite cross-section properties - was performed using the SOFiSTiK 2020 FEM software, which was verified by an independent structural engineer with ANSYS.

The sequence of cable installation and tensioning, as well as the loadings on the supports for each construction stage, was tracked using these programs.

The AxisVM X5 FEM software was also used for a more detailed analysis of certain structural details.

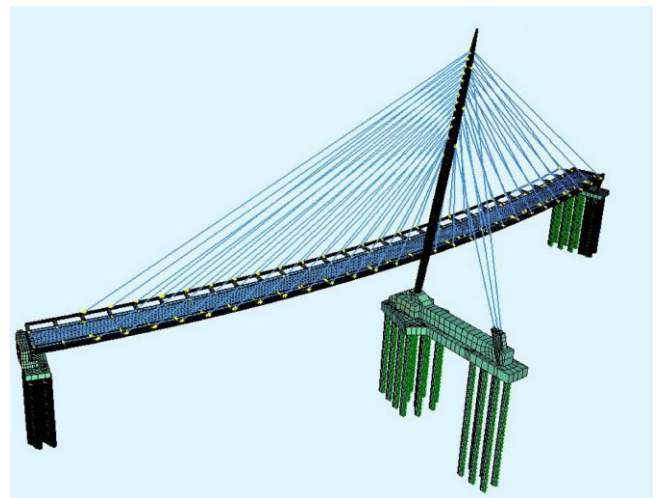


Figure 11: SOFiSTiK model

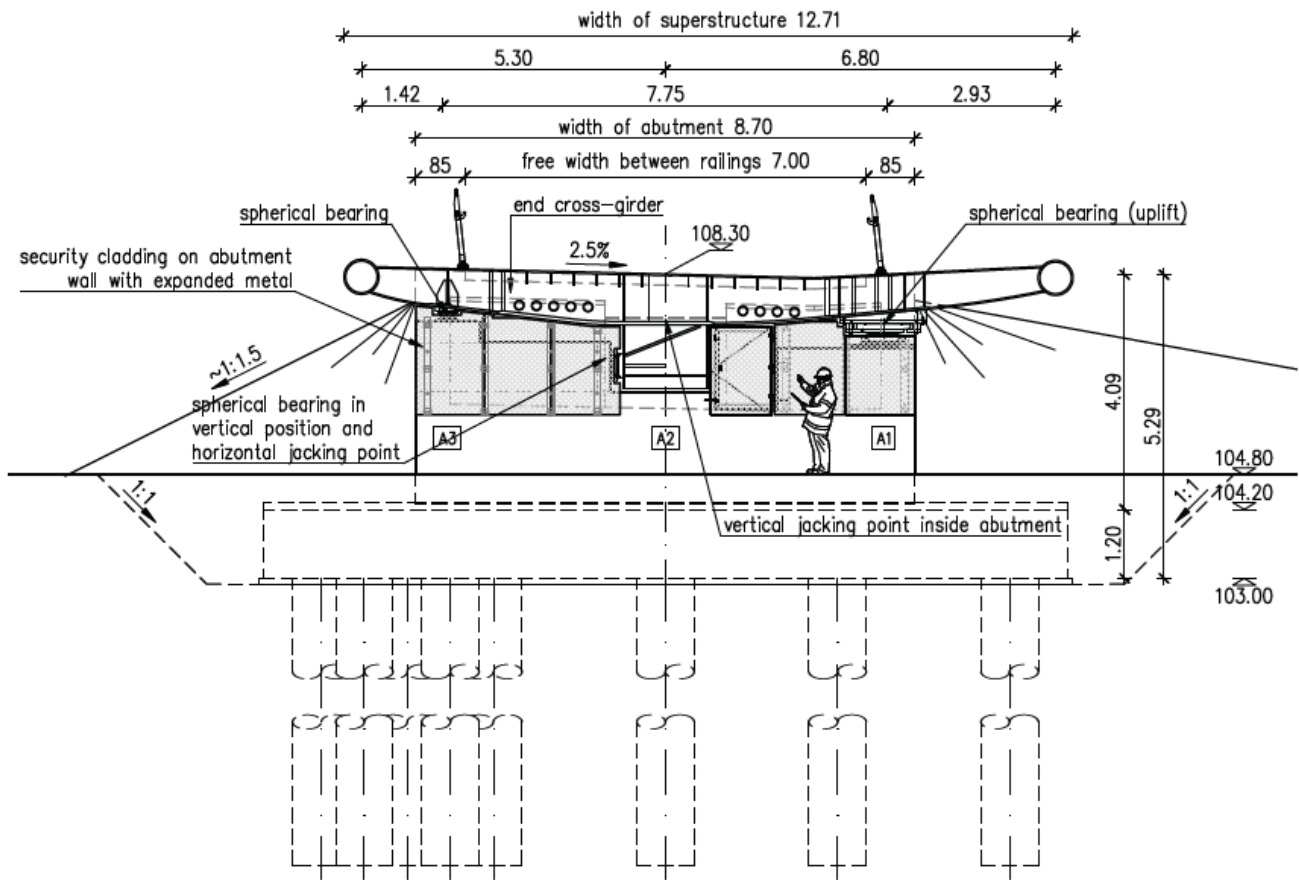


Figure 12: Abutment 'A'

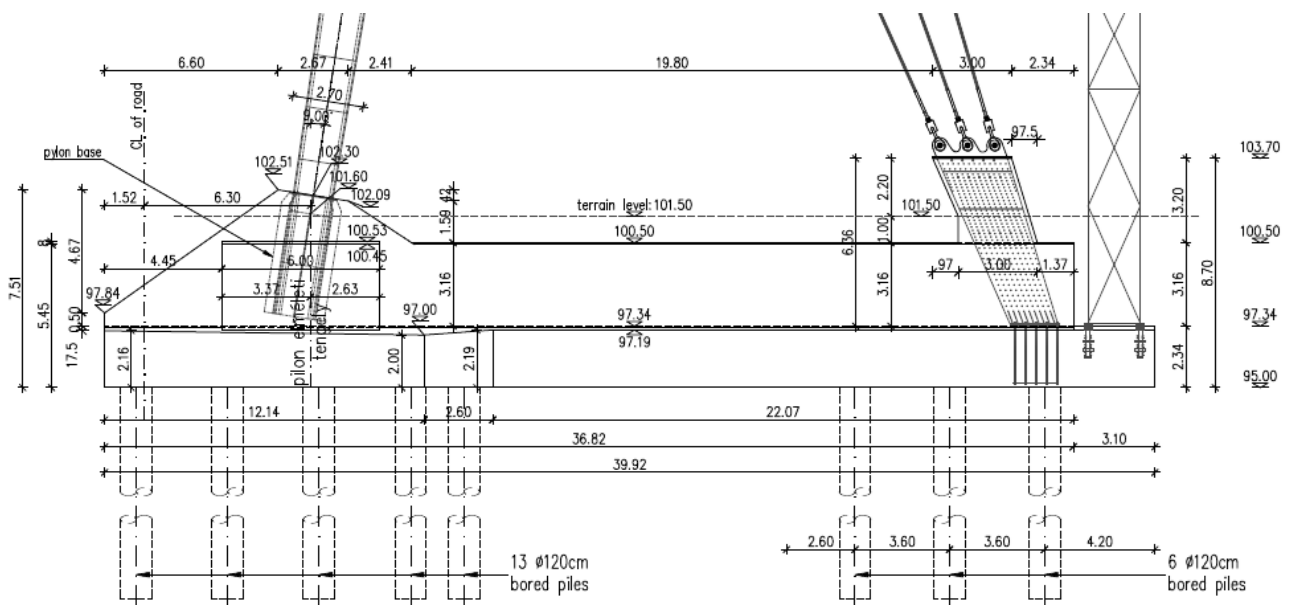


Figure 13: Side view of pylon foundation

SUBSTRUCTURES

The bridge abutments are solid monolithic reinforced concrete structures with pile foundations and parallel wing walls.

Piles and pile walls were designed to withstand the significant horizontal loads acting on the substructure, ensuring excellent lateral stiffness. A specialised bearing arrangement was designed to transfer the reaction forces between the abutment and the superstructure, comprising three bearings supporting the superstructure at each abutment. Due to the horizontal curvature of the deck and the inclined cables, significant lateral force is required to maintain structural equilibrium (~4000 kN under permanent loads). As considerable uplift force is also present, the two actions were decoupled: the outer bearings provide vertical (uplift) support, while a purely compression-loaded intermediate bearing was utilised to transfer the horizontal forces induced by the deck curvature.

These bearings were positioned on shear keys under the superstructure. The bridge is balanced under dead load; thus, longitudinal reaction forces develop exclusively from seismic and live loads.

The intermediate support constructed on the artificial island is a monolithic reinforced concrete structure supported by a pile foundation. This structure provides a rigid connection for the pylon and anchors the backstay cables stabilising it.

To anchor the substantial tensile forces induced by the three large-diameter cables that brace the pylon, a steel plate fitted with shear studs was embedded within the pile cap.

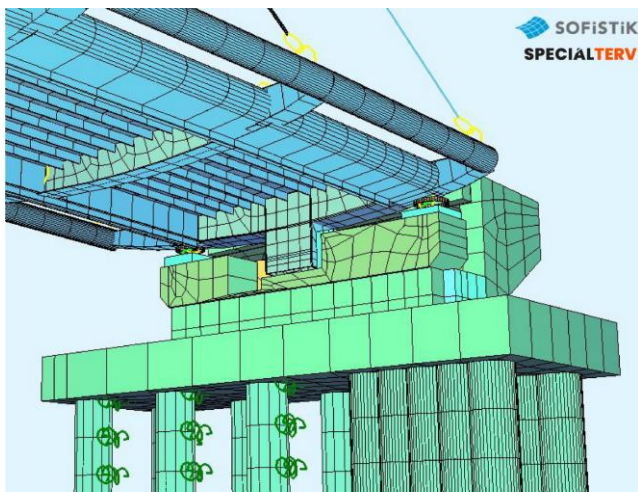


Figure 15: SOFiSTiK model

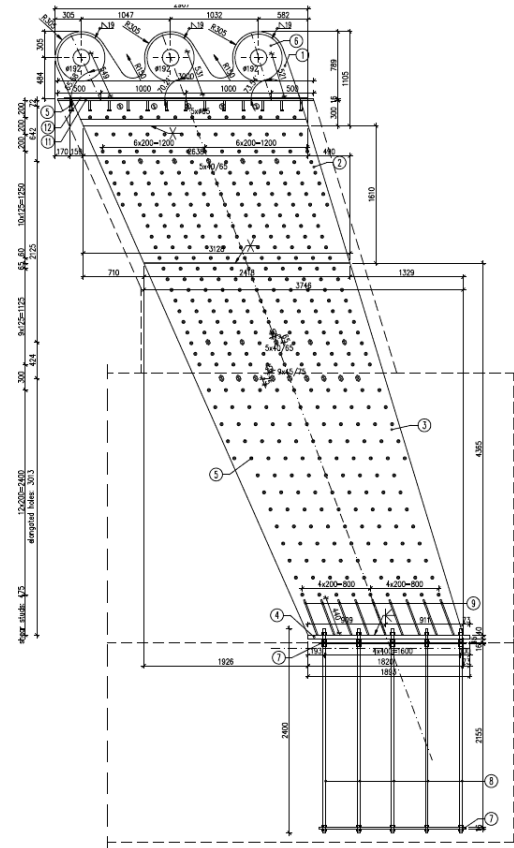


Figure 14: Anchor plate of backstay cables



Figure 16: Pylon foundation detail

PYLON

The 69.80 m long and 159 t pylon is made of S460 ML structural steel plates, formed into cylindrical and conical shapes using a special rolling process. The design of the lug plates to be welded onto the single-curved conical surface was done using a 3D parametric BIM model.

In addition to determining the unfolded contours of the plates and the adjoining lines of the lug plates, it was also necessary to create templates and auxiliary elements suitable for control measurements.

This was achieved by close collaboration between the Contractor and the Designer. The steel pylon was welded together from 5 subassemblies on a TS80 type barge on the Danube River and was lifted onto the pile cap in one piece.

The counter-diaphragms that connect to the internal stiffening rings transfer the forces acting on the lug plates into the pylon, minimising the local bending effects on the shell.

The interior of the steel pylon and parts of the bridge deck were filled with concrete. The composite action between steel and concrete is an efficient and economical design solution to increase the performance and durability of the main load bearing elements.

The internal stiffening rings of the pylon are capable of transmitting the compressive forces to the C40/50 concrete infill without any shear connectors.

The pylon has shear studs on both the outer and inner surfaces only on the bottom that is embedded into the pile cap in order to form a fixed connection.

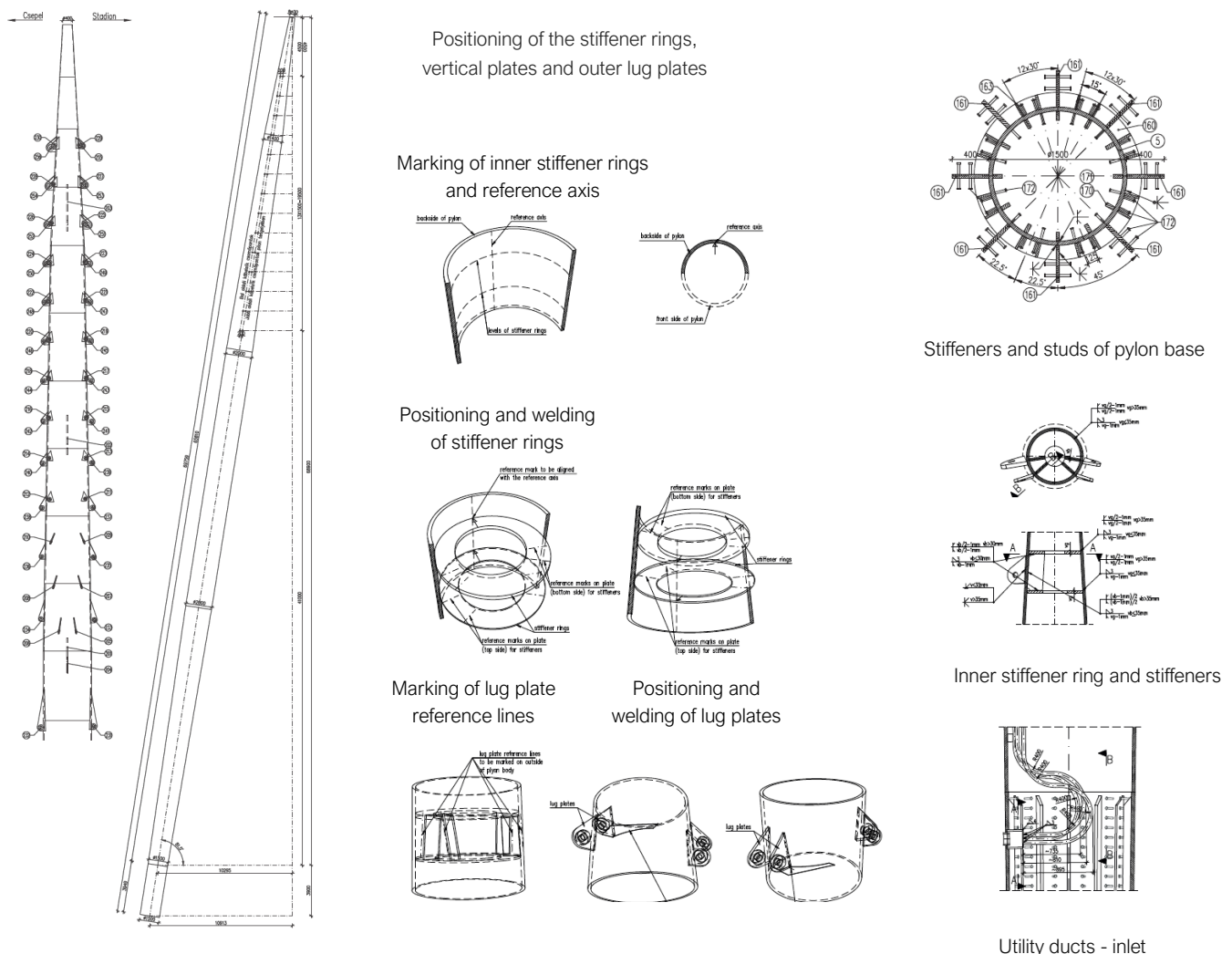


Figure 17: Pylon

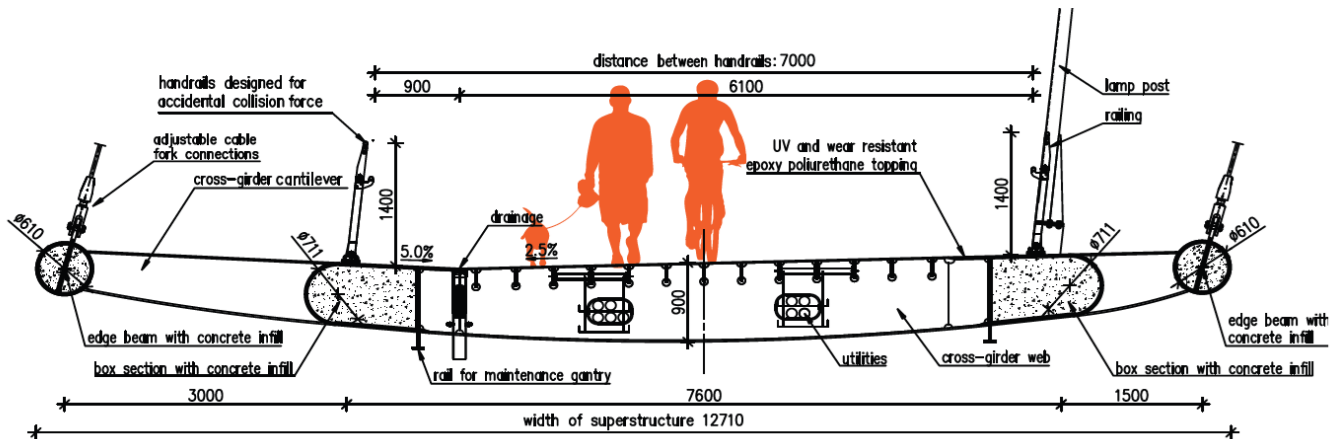


Figure 18: Superstructure cross section

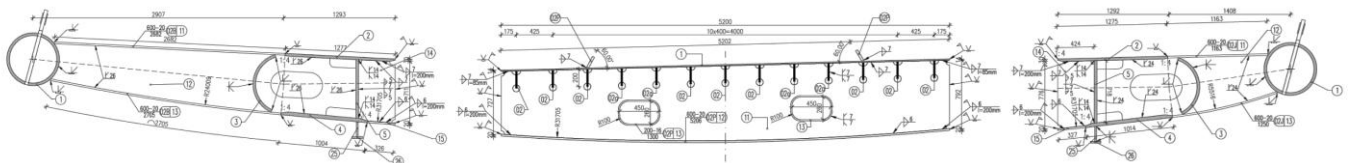


Figure 19: Cross-section assemblies

SUPERSTRUCTURE

The main girder of the bridge consists of two welded box sections, and an orthotropic steel deck plate in between.

Box sections are made of Ø711 mm, 30 mm wall-thick steel half-pipes, a 30 mm top flange, a 40 mm bottom flange, and an internal 30 mm vertical web. The orthotropic steel deck is stiffened with longitudinal open ribs at 400 mm intervals. Crossbeams have a spacing of 6 m, according to the stay cables.

The structural height of the superstructure is only 900 mm. The bottom flanges of the crossbeams are curved to make the superstructure optically slimmer.



Figure 20: Superstructure model

To ensure adequate pedestrian clearance on the bridge's curved plan, the cables were cantilevered outside the main girders. Steel edge beam tubes were used with a diameter of Ø610 mm diameter steel tubular sections with 30 mm wall thickness on both sides, connected to the deck with cantilevers at every crossbeam.

The crossbeams and the orthotropic steel deck were made of S355 M steel, while the box sections, crossbeam cantilevers and edge beams (tubes) were made of S460 ML steel.

Both the edge beams and the box sections of the deck were filled with concrete. The cantilever and edge tubes not only made the roadway more transparent, but also enhanced the spatial experience and ensured a seamless connection between the bridge structure and the adjacent stadium building's key architectural details.

CABLES

The slender curved deck, which is nearly 170 m long, "floats" over the Danube branches without any additional support. The deck is suspended with high-strength steel full-locked coil (FLC) cables attached to the needle-like pylon, independent of the deck.

Their diameters range from 35 to 65 mm, with 5 mm increments. Lug plates are inserted into the cut-outs prepared on the tube to ensure the optimal transfer of cable forces. The three backstay cables that ensure the equilibrium of the outward-leaning slender pylon have a 100 mm nominal diameter. All stay cables have adjustable forked ends. The total length of the applied cables is 3500 m.

DAMPERS

The large pedestrian surface area and slender profile of the bridge make it sensitive to dynamic effects. Considering its urban context, the structure is expected to accommodate heavy pedestrian traffic.

Consequently, the client requested compliance with maximum comfort level criteria pursuant to the Sétra guideline. This objective was ultimately achieved through the integration of 12 tuned mass dampers within the superstructure. Furthermore, the additional stiffness contributed by localised concrete infills proved advantageous.

Dynamic load tests were carried out before and after the installation of the vibration dampers to accurately determine and fine-tune their effect. The results of the two loading tests demonstrated the effectiveness and importance of the dampers, which were designed by using dynamic harmonic analysis. The quantity and spatial placement of the dampers were determined through an optimisation algorithm. The resulting structure satisfied the strict acceleration limits of 0.5 m/s^2 vertically and 0.15 m/s^2 horizontally, thereby attaining the maximum pedestrian comfort classification.



Figure 21: Stay cable joint

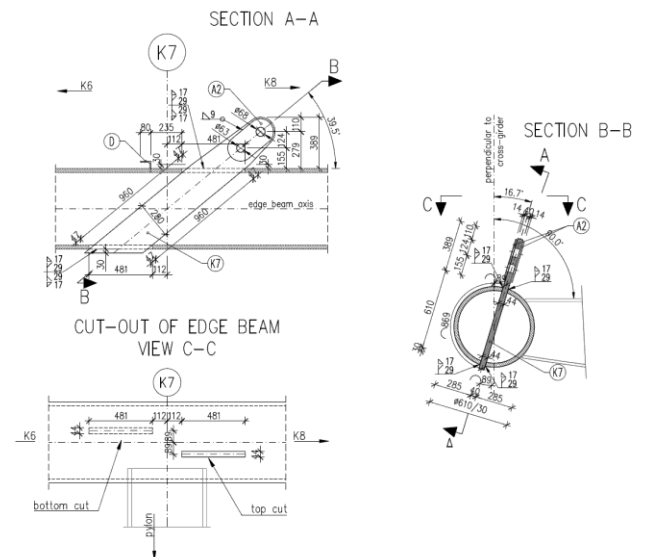


Figure 22: Stay cable joint details

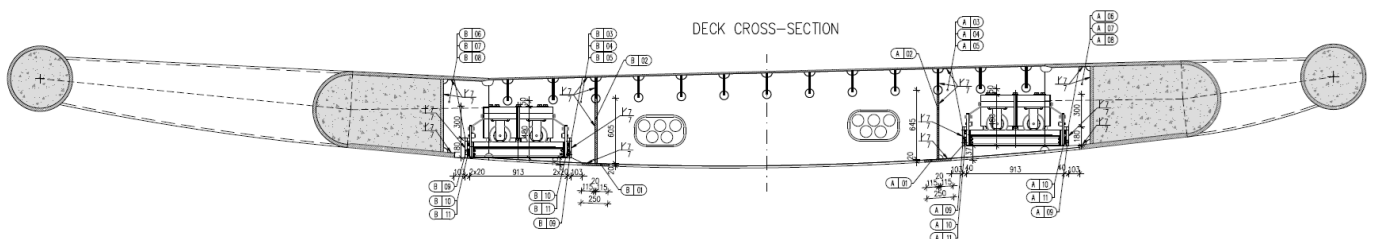


Figure 23: MTMD-H damper supports (horizontal)

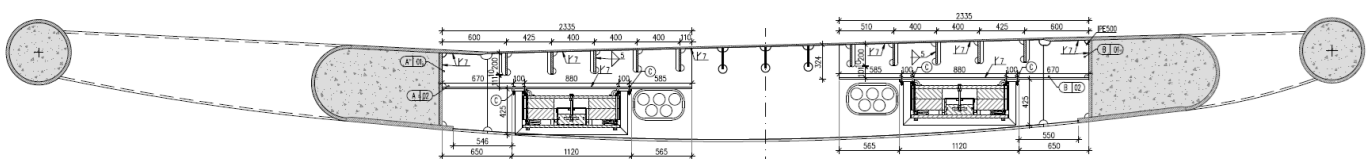


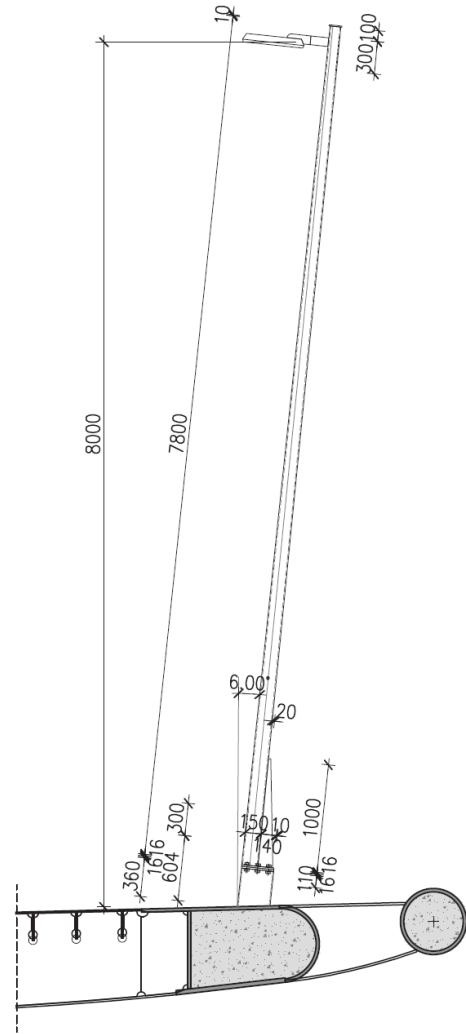
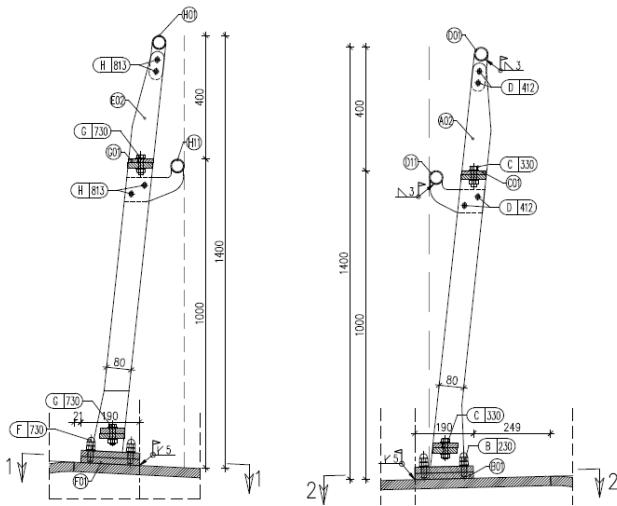
Figure 24: MTMD-V damper supports (vertical)

BRIDGE ACCESSORIES

The bridge railings are tilted to match the path of the stay cables towards the pylon. This effect also emphasises the spatial curvature of the bridge deck and the inclination of the pylon.

The handrails running at heights of 1 m and 1.40 m are made of tubular sections. The railings are also designed to withstand an accidental collision force to protect electric cars that may occasionally appear on the bridge during larger sports events.

The bridge deck is illuminated by 8 m high lamp posts placed at 18 m intervals on the pylon side.



Figures 25 - 27: Bridge accessories

CONSTRUCTION TECHNOLOGY

In addition to the detailed design of the permanent structure, the Designer was responsible for the fabrication drawings of the entire steel structure, together with all associated erection and temporary works documentation. Particular attention was required in planning the transfer of the large steel elements onto the Danube, their assembly afloat and their subsequent movement into position. These operations called for numerous purpose-designed details and continuous coordination with both the fabricator and the erection contractor.

Every element of the operation - from the barge supports, lifting points and rotating cradle to the auxiliary strut, sliding mechanism and temporary support structures - was designed and verified as part of the erection plans. The drawings thus define not merely a general construction concept, but the complete step-by-step sequence, including every temporary support condition and transfer of load.

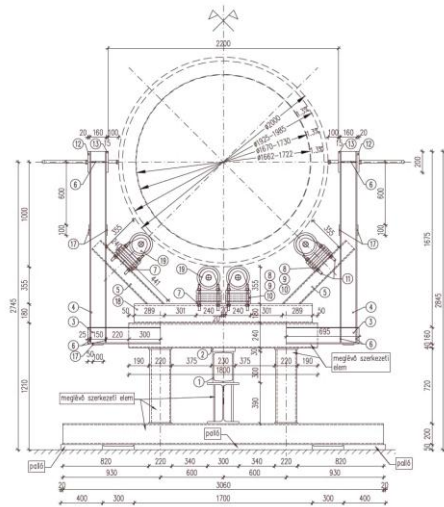
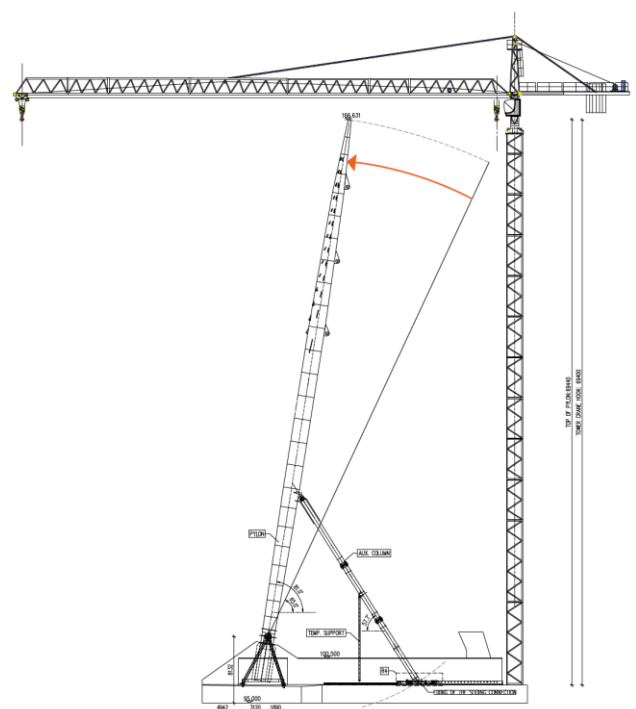
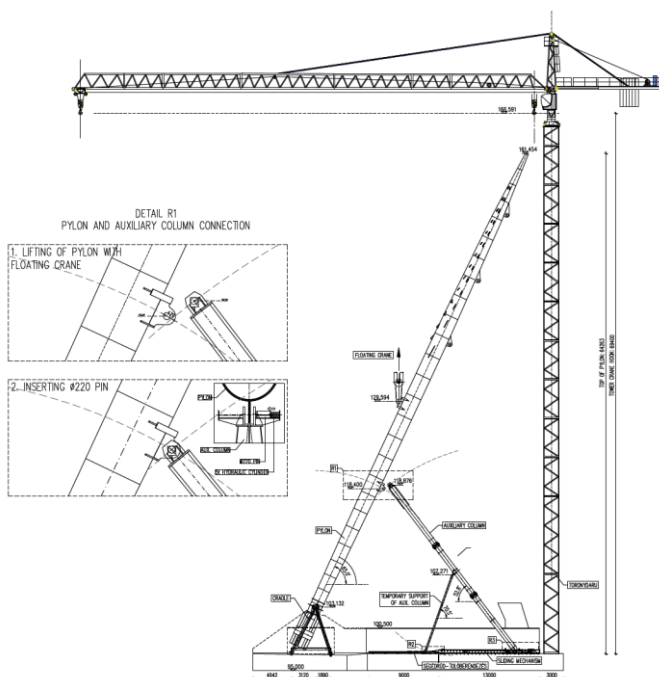
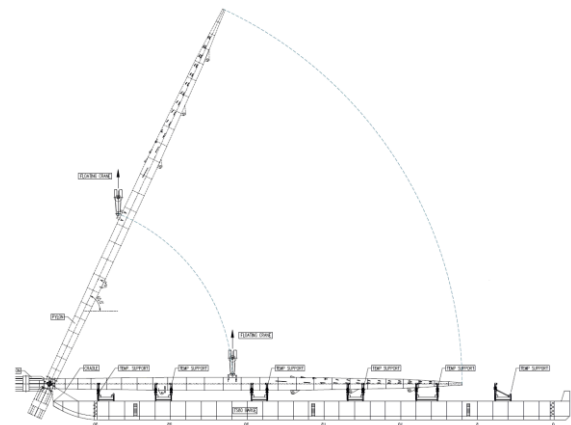


Figure 28: Temporary pylon rotation structure

→ Figure 29: Pylon erection - stage 0

↓ Figure 30: Pylon erection - stage 1

↘ Figure 31: Pylon erection - stage 2



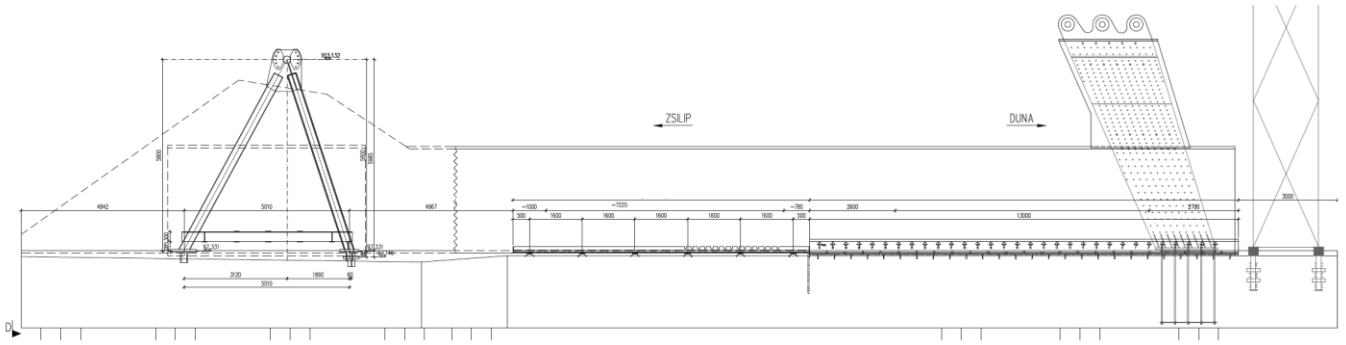


Figure 32: Pile cap side view, with temporary support frame

CONSTRUCTION

FOUNDATIONS

In the first phase, execution of the pile foundations was completed. Because the intermediate support was located on an island, a temporary pontoon bridge was deployed to establish site access. The substructures were cast in situ, whereas the structural steel components were fabricated off-site at a nearby facility on Csepel Island.

PYLON

Due to the very limited construction area and the slender profile of the pylon, a specialised erection methodology was adopted.

The welding of the pylon sub-assemblies was carried out on a TS80-type barge stationed on the Danube. The 69.80-m-long structural assembly was subsequently transported adjacent to the construction site, where installation was performed using Hungary's largest floating crane, the Clark Ádám.

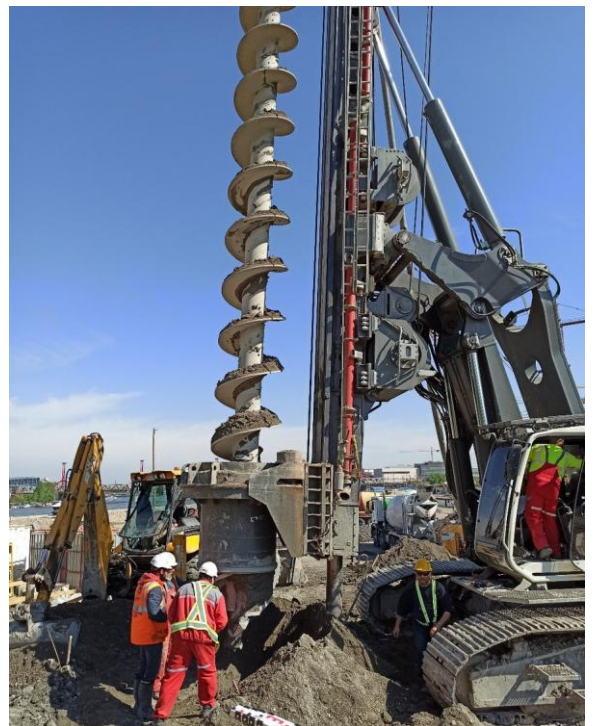


Figure 33: Large-diameter dry auger boreholes



Figure 34: Pontoon bridge

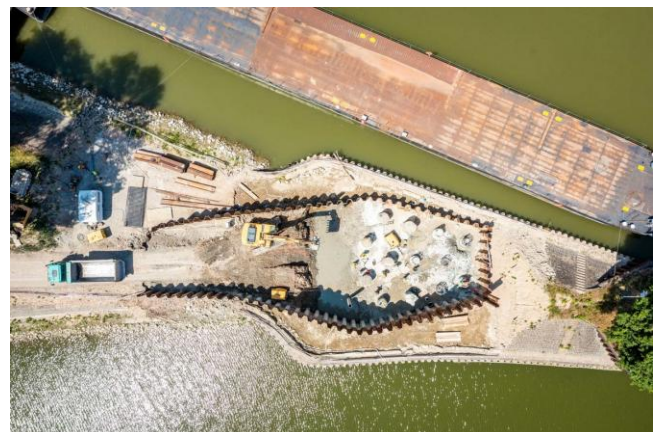


Figure 35: On-site concreting of substructure

The operation utilised the full 200 t lifting capacity of the crane. The pylon was installed on the temporary supports at a 65° angle.

Following the lifting of the pylon, the final alignment was achieved using a movable steel support bracket designed specifically for the task. The interior of the pylon and parts of the bridge deck were filled with concrete.

FABRICATION

The fabrication and on-site erection of the steel pylon and bridge deck were carried out by ACÉLHIDAK Ltd. and Hídépítő Ltd. The project required several innovative manufacturing solutions.

Pylon fabrication began in January 2021 using S460ML steel tubes with diameters ranging from 2 m to 6.4 m and wall thicknesses of 10–45 mm.

The 69.8 m, 159 t pylon was divided into five transport sections and assembled from 28 cylindrical and conical segments.



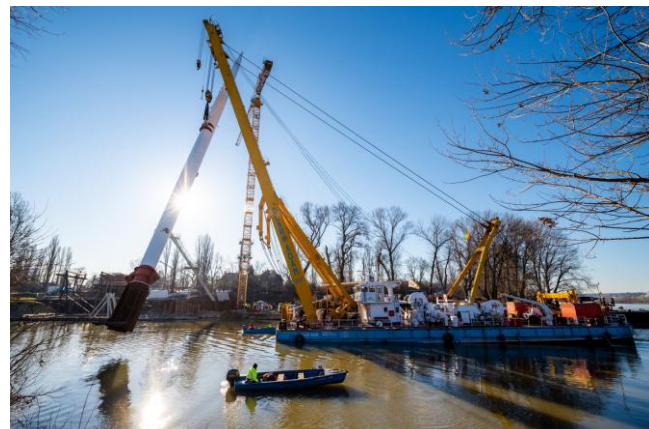
Figure 36: Pylon transport on barge



Figure 38: Pylon shipping



Figure 37: Placement of pylon into temporary support



↙ Figure 39: Pylon lifting

↓ Figure 40: Pylon tilting with jacking

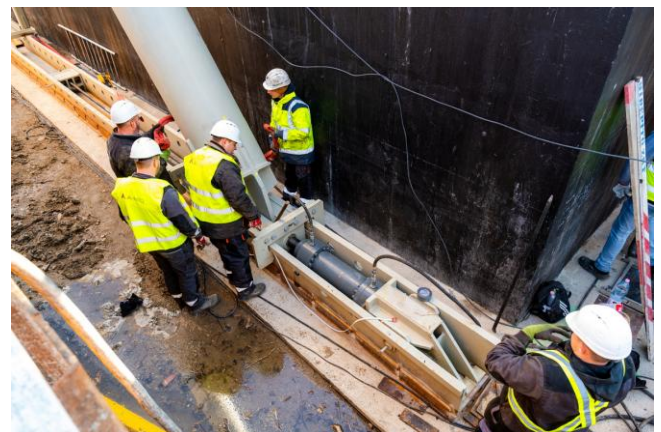




Figure 43: Pylon foundation



← Figure 41: Pylon installation



Figure 42: Pylon in temporary support

Dedicated rotating equipment was developed for circumferential welding, and new welding procedures, including combined MAG, submerged arc, and metal-cored wire processes, were qualified.

Deck fabrication started in August 2021. The 172 m long, 852 t steel deck was manufactured in modular longitudinal and transverse segments to facilitate production and transport.

The fabrication of the cantilever sections involved bending the edge tubes, prefabricating the cantilever boxes, assembling the main cantilever units, cutting the edge tubes and welding the cable anchorage plates, followed by welding the inspection trolley rails.

The central deck sections were produced by splicing the deck plates, welding the longitudinal stiffeners, welding and straightening the orthotropic deck panels in a prestressing bed, and finally attaching and welding the cross girders.



Figure 44: Pylon fabrication



Figure 45: Deck fabrication on site

Throughout the project, precise preparation, controlled assembly, and continuous quality verification were essential to achieving the required geometric accuracy and structural performance.

LAUNCHING

The superstructure was launched into its final position from the southern abutment at Csepel Island. Temporary steel support structures held the main girder in position above the Danube during the launching and cable installation.

After the launching of the superstructure was finished, the bridge bearings were installed, and both end crossbeam assemblies were welded to the deck. This was followed by the installation and tensioning of the stay cables.



Figure 46: Deck launching on temporary supports

↓ Figure 47: Deck launching





Figure 48: Deck launching on temporary supports

TENSIONING

Cable tensioning was performed sequentially in a predefined order; for this purpose, hydraulic jacks were used. Cable forces were monitored simultaneously across all cables in each tensioning step.

The tensioning procedure was expected to cause significant deformation to the curved superstructure, namely a horizontal displacement of approximately 200 mm towards the pylon. This deformation had to be carefully considered during the installation and tensioning of each cable, both in theory and in practice.



Figure 49: Before cable tensioning



Figure 50: Cable tensioning device

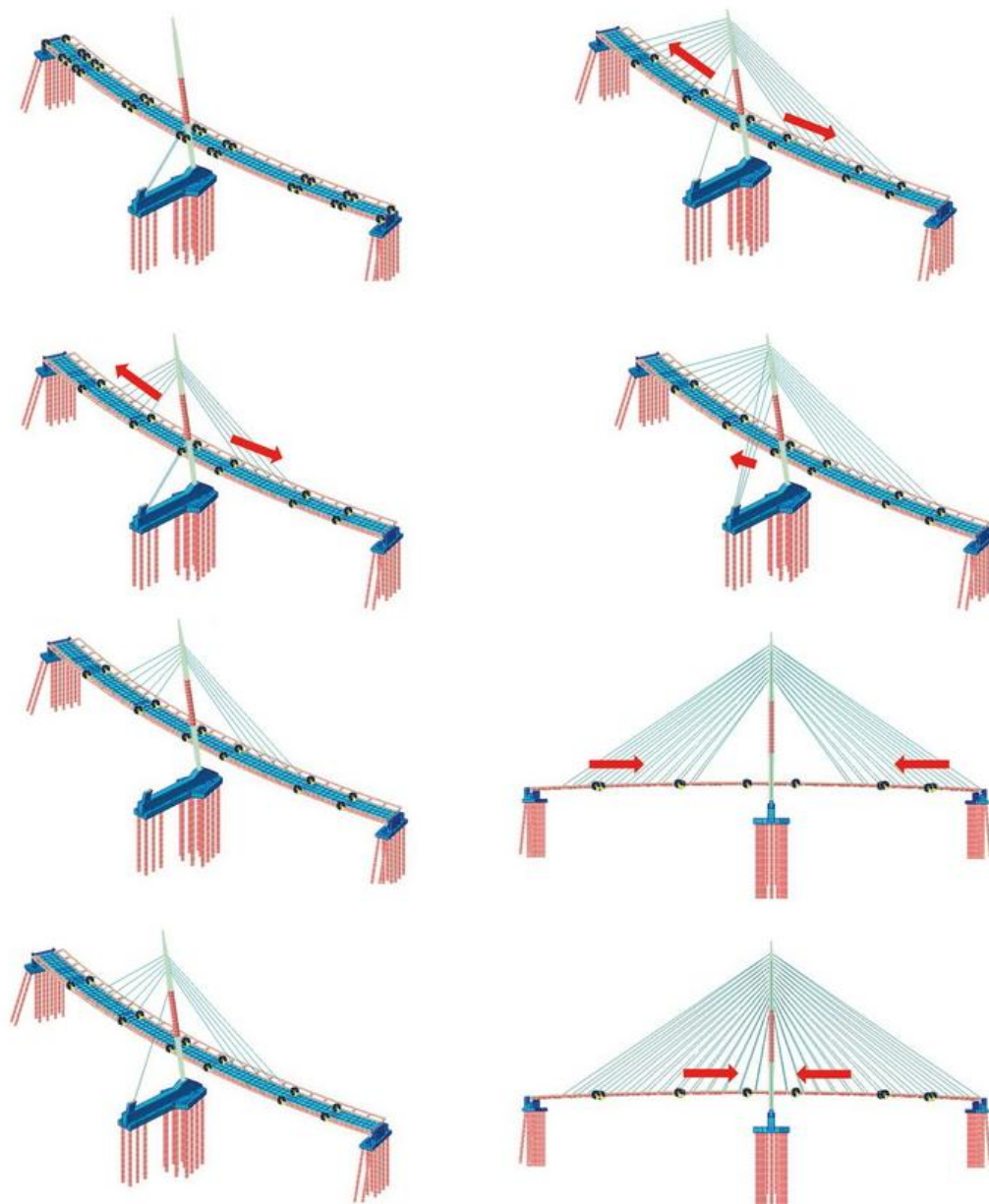


Figure 51: Cable stressing steps

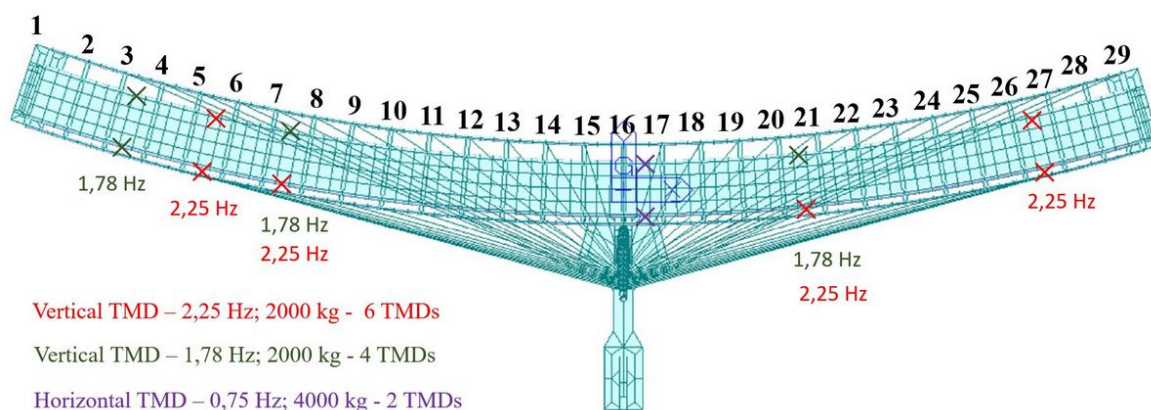


Figure 52: Location of TMDs

STATIC AND DYNAMIC LOAD TESTS

Eigenmode shape

Vertical displacement of the deck and number of cross-girders at peak displacements

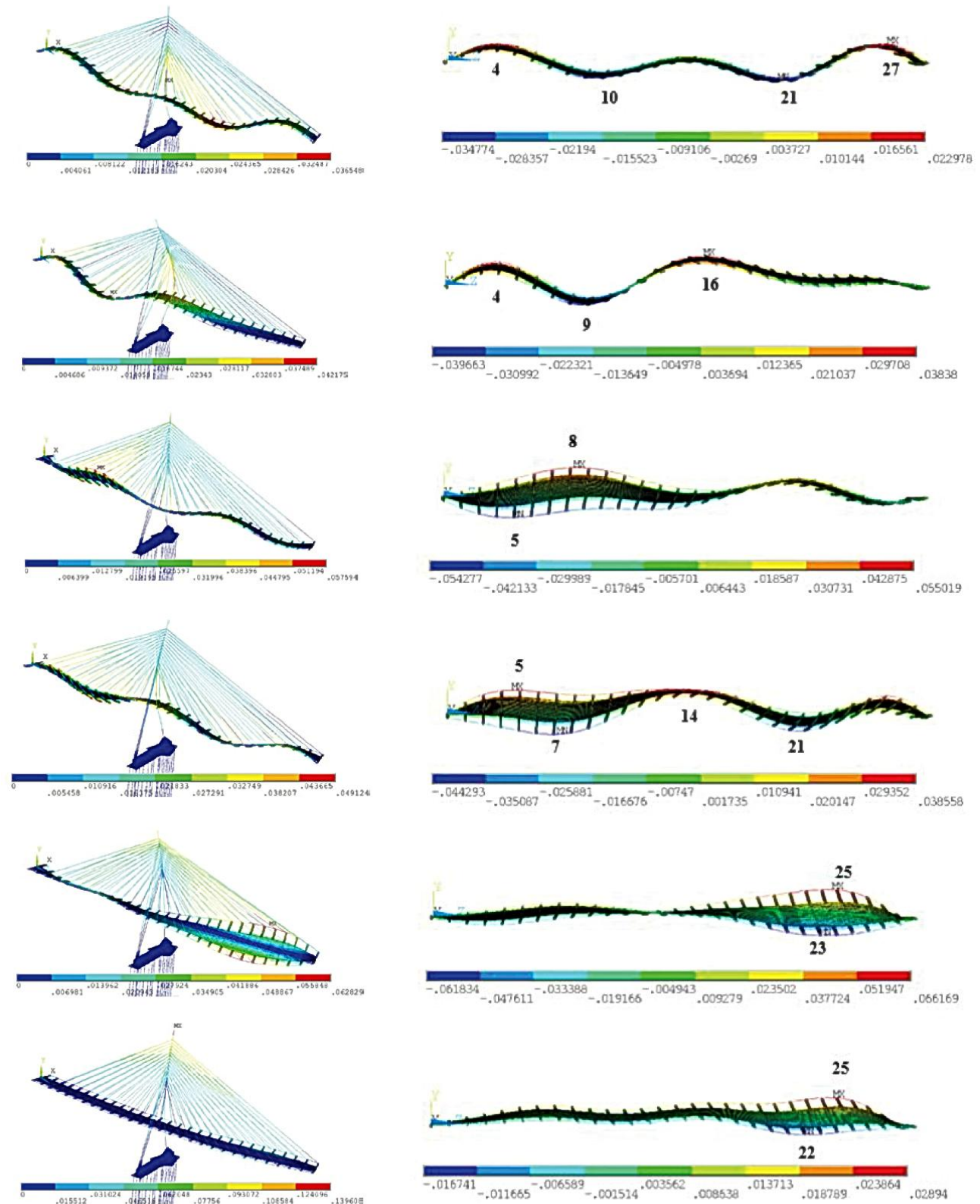


Figure 53: Specific eigenfrequencies and eigenmodes



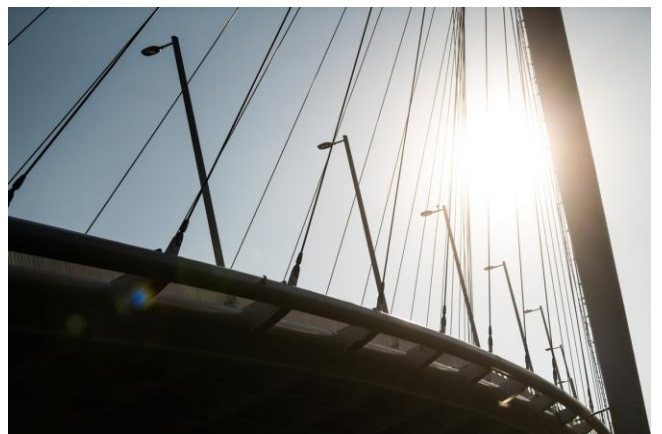
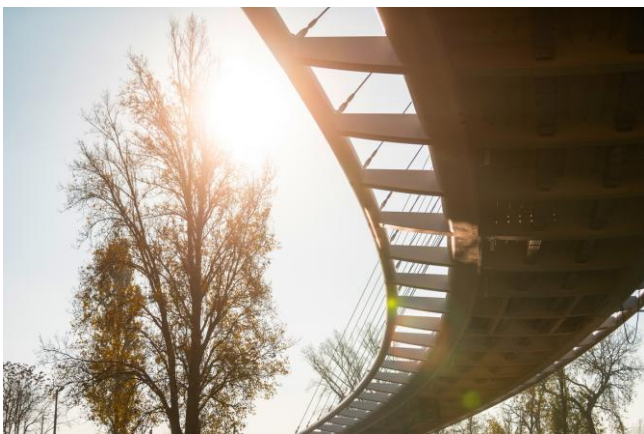
Figures 54 and 55: Robinson Bridge with Athletic Stadium



Figure 56: Robinson Bridge from the waterfront



Figure 57: Northeastern view



Figures 58 and 59: Bridge details



Figure 60: Front view



Figure 61: Cable dynamics



Figures 62 - 64: Decorative lighting

FIGURES CREDIT

Figures 1, 6, 7, 8, 10, 11, 12, 13, 14, 15, 16, 17, 18,
19, 20, 21, 22, 23, 24, 25 – 26, 28, 29, 30, 31, 32, 33,
41, 42, 43, 50, 53, 56, 62- 64

Credit: SPECIÁLTERV

Figure 9

Credit: Zoltán Szabó, SPECIÁLTERV

Figure 51, 52

Credit: SPECIÁLTERV, BME

Figures 3, 4, 5, 27, 34, 35, 36, 37, 38, 39,
40, 44, 45, 46, 47, 48, 49, 54, 55, 57, 58,
59, 60, 61,

Credit: Magyar Építők

MORE IMAGES OF THE BRIDGE

CONCLUSION

The Robinson Bridge demonstrates how a 168 m curved deck can cross the river without any fixed intermediate support. Suspended from a single slender pylon, the superstructure retains a visually continuous, floating character despite its spatially complex and strongly asymmetric structural behaviour.

The project also provided substantial practical experience through the close collaboration between the Designer and the Contractor.

Both parties gained valuable knowledge in coordinating design, fabrication and erection, while the design team further developed a parametrically driven workflow in which the construction stages formed an integral part of the structural design.

Predicted geometry, cable forces and deformations were continuously compared with measurements taken during launching, cable tensioning and load testing, enabling subsequent operations to be refined on the basis of verified structural behaviour.

This measurement-validated connection between parametric design and construction became one of the project's most valuable outcomes.

AWARDS

- Gold Winner; International Architecture & Design Awards 2026; Bridges category
- Gold Winner; Architecture Masterprize 2025; Infrastructure Category; Structure of the Year Award 2025; Hungarian Chamber of Engineers; Bridges Category
- 1st Place; FIABCI Hungary Prix d'Excellence 2024; Special facilities category
- Winner in Architectural Design 2024; BLT - Built Design Award; Bridges and Public Infrastructure Category
- BIG SEE Winner Award 2024; Public, Cultural Buildings and Infrastructure Buildings Category
- Steel Structure of the Year 2023; Hungarian Steel Structure Association
- Award for Excellence 2023; Foundation for Constructors' Masterworks; Transportation Facilities category
- Top 6 Cycle and Pedestrian Bridges Nomination 2023; IABSE



Figure 67: The Robinson Bridge with decorative lighting

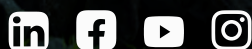
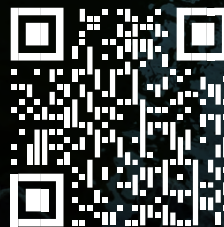


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EVOLUTION OF A PRECAST SEGMENTAL BRIDGE IN A HIGHLY CONSTRAINED TRANSPORT CORRIDOR

*Daniel Elmaleh, Via Bridges
Fast Lanes Shfaim Project, Israel*



Figure 1: General view of the completed BR-541 bridge after opening to traffic

INTRODUCTION

Bridge engineering is often presented through the completed structure: span arrangement, deck type, construction method and final appearance.

In practice, however, the selected form is frequently the outcome of a longer design process in which several feasible structural alternatives are tested against geometry, access, programme, cost and construction risk.

BR-541, part of the Fast Lanes Shfaim Project in Israel, is a clear example of this process. The bridge carries Road 541 across one of the most constrained transport corridors in the region, passing above Ayalon Road, active railway

infrastructure, planned railway development and major gas pipelines. The completed structure is a 237.43 m long precast segmental prestressed concrete bridge, approximately 37 m wide, with five spans and a maximum span of about 65 m. It was constructed using the balanced cantilever method with precast segments.

The final bridge form was not selected at the beginning of the project. It evolved from an original incremental launching concept, through a conventional precast girder bridge and a spliced girder alternative, before the contractor adopted the precast segmental solution as the most suitable and economical construction strategy.



Figure 2: Aerial view of the BR-541 crossing within the Fast Lanes Shfaim corridor. The bridge crosses a dense transport zone containing Ayalon Road, existing railway tracks, planned railway facilities and major utility infrastructure.



Figure 3: Plan view of the completed BR-541 bridge after opening to traffic

Client / Concessionaire	Ayalon Highways
Contractor	Denya Cebus / Danya Cebus
Designer	Via Bridges
Construction Engineering	COWI
Segment Production	Cebus Rimon
Bridge type	Precast segmental prestressed concrete bridge Three precast box-girder cells with in situ longitudinal closures
Construction method	Balanced cantilever with precast segments
Total length	237.43 m
Maximum span	approx. 65 m
Deck width	approx. 37 m
No. of precast segments	222 units
Superstructure cost	1350 USD/m ²

→ Figure 4: Location of the Bridge

Click on the image to open Google Maps

This article follows that evolution and explains how constructability and construction-stage behaviour became decisive design parameters.

PROJECT BACKGROUND: FAST LANES SHFAIM AND PROCUREMENT FRAMEWORK

The Fast Lanes in Gush Dan programme was developed to improve access to the Tel Aviv metropolitan area by combining managed fast lanes along the main northern and southern approaches with large park-and-ride transport hubs and free shuttle services.

The Shfaim hub forms the northern anchor of the system, complementing the Mevo Ayalon hub in Rishon LeZion.



Its purpose is to allow drivers approaching from the north to leave private vehicles outside the congested metropolitan core and continue by shuttle and priority-lane services toward the main employment centres of Tel Aviv.

The Shfaim park-and-ride complex is planned to provide approximately 7,000 parking spaces. It serves as an integrated transport hub, including access roads, bus and shuttle platforms, operational areas, and interfaces with future railway infrastructure. Within this wider system, Road 541 provides an important east-west access connection between the Ra'anana/Sharon area, the Shfaim interchange and the fast-lanes network.

The project was procured under a public-private partnership framework, using a DBOT-type concession model in which the private concessionaire is responsible for design, financing, construction, operation and maintenance before transfer back to the public sector at the end of the concession period.

The concession was awarded to Nativ La'ir, a project consortium established for the Fast Lanes programme.

This procurement framework influenced the evaluation of bridge alternatives: the preferred solution had to perform structurally, but it also had to reduce construction risk, improve programme certainty, support long-term operation and provide a competitive overall project economy.

PROJECT SETTING AND MAIN CONSTRAINTS

The BR-541 crossing is located in the northern entrance corridor to the Tel Aviv metropolitan area, where roadway access, railway facilities, utilities

and future development envelopes are concentrated within a narrow strip. The bridge therefore had to be developed without compromising the operation of the surrounding transport network and while preserving the space required for planned railway works.

The main project constraints were:

- construction above live road traffic and active railway infrastructure;
- the need to preserve the envelope for future railway expansion and tunnelling works;
- major gas pipelines crossing the site and limiting foundation and temporary works options;
- restricted access for cranes, segment delivery and temporary works;
- limited opportunities for intermediate supports in the most sensitive parts of the corridor;
- a wide bridge deck of approximately 37 m, which affected lifting weights, cross-sectional planning, prestressing arrangement and erection logistics.

As a result, the bridge could not be designed solely for its final service condition. The construction process itself became a governing design case: each temporary configuration required verification of segment lifting and positioning, cantilever stability, temporary unbalanced effects, support reactions and work-zone limitations.

Original Tender Solution: Incremental Launching

The original state tender solution was based on an incrementally launched prestressed concrete bridge.

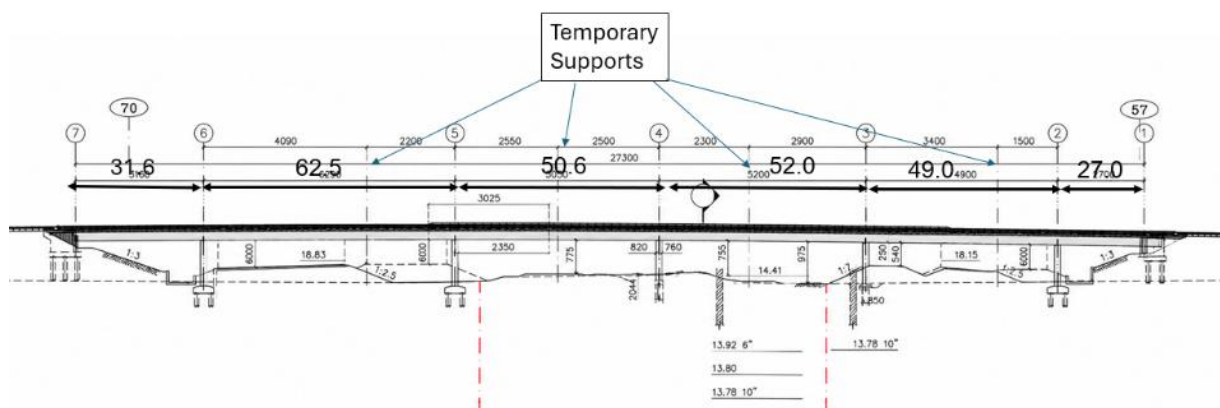


Figure 5: Original state tender solution based on incremental launching. The longitudinal arrangement shows the proposed span distribution, temporary piers and launching concept adopted to limit direct construction activity above the live corridor

Incremental launching is often attractive where work above traffic, railways or other obstacles must be reduced, because the superstructure is assembled behind an abutment and pushed forward in stages. The method can therefore limit the need for extensive falsework directly below the bridge.

For BR-541, this solution offered a clear construction advantage by reducing direct work over the active corridor. At the same time, the method introduced its own sensitivities: the need for a launching bed and temporary nose, careful control of launching stresses, temporary support and reaction management, and compatibility between the bridge geometry and the launching operation. In the context of the design-build phase, the contractor and designer therefore continued to search for a solution that better matched the available construction resources and the evolving understanding of future railway and roadway constraints.

Design-Build Tender Alternative: Conventional Precast Girders

After the design-build tender was awarded, the bridge concept was re-evaluated by the contractor together with Via Bridges. The first contractor-driven alternative was a conventional precast girder bridge.

This option was attractive because it used familiar prefabricated elements and a relatively straight-forward erection concept.

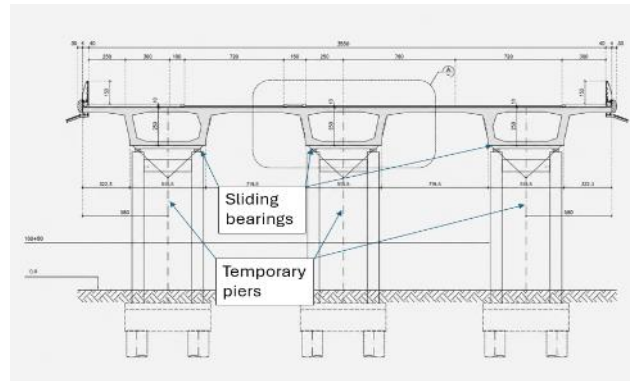


Figure 6: Typical section of state tender solution

However, the conventional girder arrangement created significant difficulties in the BR-541 corridor. The support locations and achievable span lengths conflicted with the envelope required for future railway works. In addition, some intermediate supports had to be integrated with slurry-wall elements associated with future excavation, introducing substantial service-load horizontal effects from static earth pressure.

In this case, a bridge type that was structurally simple in elevation generated a much more complex interaction between substructure, excavation support and superstructure behaviour.

Spliced Girder Alternative

To overcome the span limitations of the conventional girder concept, a spliced girder alternative was developed.

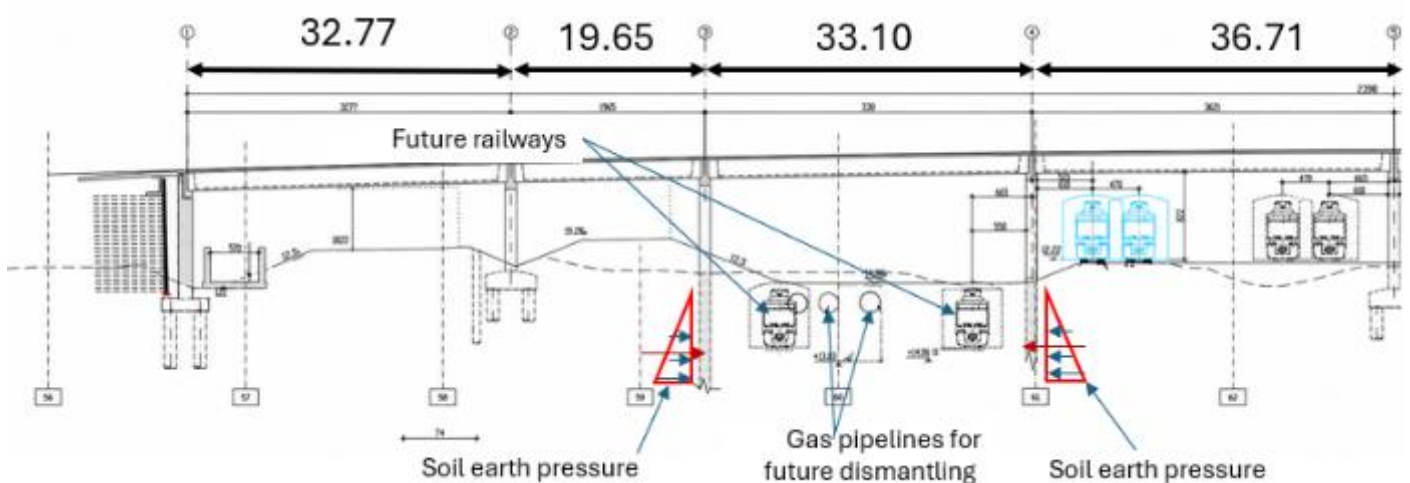


Figure 7: Design-build tender alternative based on conventional precast girders. The arrangement illustrates the limited span flexibility and the sensitivity of support locations relative to existing and future railway infrastructure and utility corridors

1.85m U girder

10.5m

Figure 8: Typical section of Design-build tender alternative

2.0 m U girder

Figure 9: Typical section of the spliced girder alternative

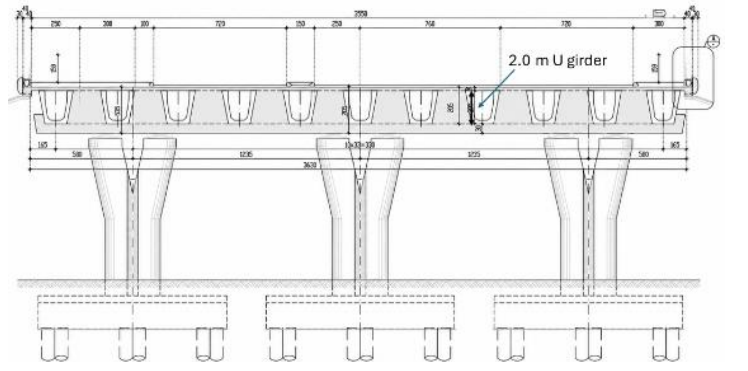
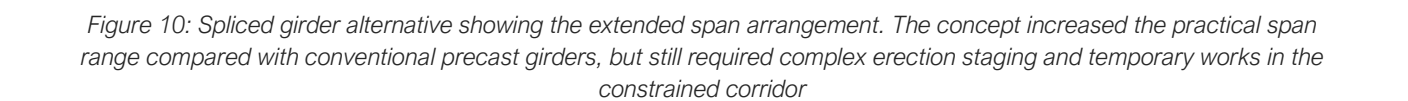


Figure 9: Typical section of the spliced girder alternative



Consequently, the final scheme is not a textbook balanced-cantilever bridge with regular and symmetric span ratios. One of the main engineering lessons of the project is that precast segmental technology, although often associated with modular repetition, can be successfully adapted to irregular span arrangements where support positions are dictated by roads, railways, utilities and future infrastructure envelopes.

The final solution offered several project-specific advantages:

- the longer main span allowed critical below-deck zones to be crossed with fewer intermediate supports;
- balanced cantilever erection reduced the need for temporary support from below;
- precast segment production transferred a substantial part of the work to a controlled casting environment;

- the three-cell arrangement divided the very wide deck into manageable precast and erection units;
- the short-line casting approach benefited from the production experience gained on adjacent ramp bridges BR-06 and BR-07, which used similar precast box segment technology;
- the use of three segmental bridges within the same project helped the contractor balance production demand, reducing pressure on the precast girder factory while other parts of the project continued to use precast U-girders;
- casting and erecting the three cells separately allowed a significant part of concrete shrinkage to develop before the final longitudinal closure pours, reducing restraint effects in the completed wide deck.

The selected solution was therefore not only a structural solution, but also a construction strategy. It allowed the contractor to manage risk, shorten operations in constrained areas and improve the overall economy of the bridge works.

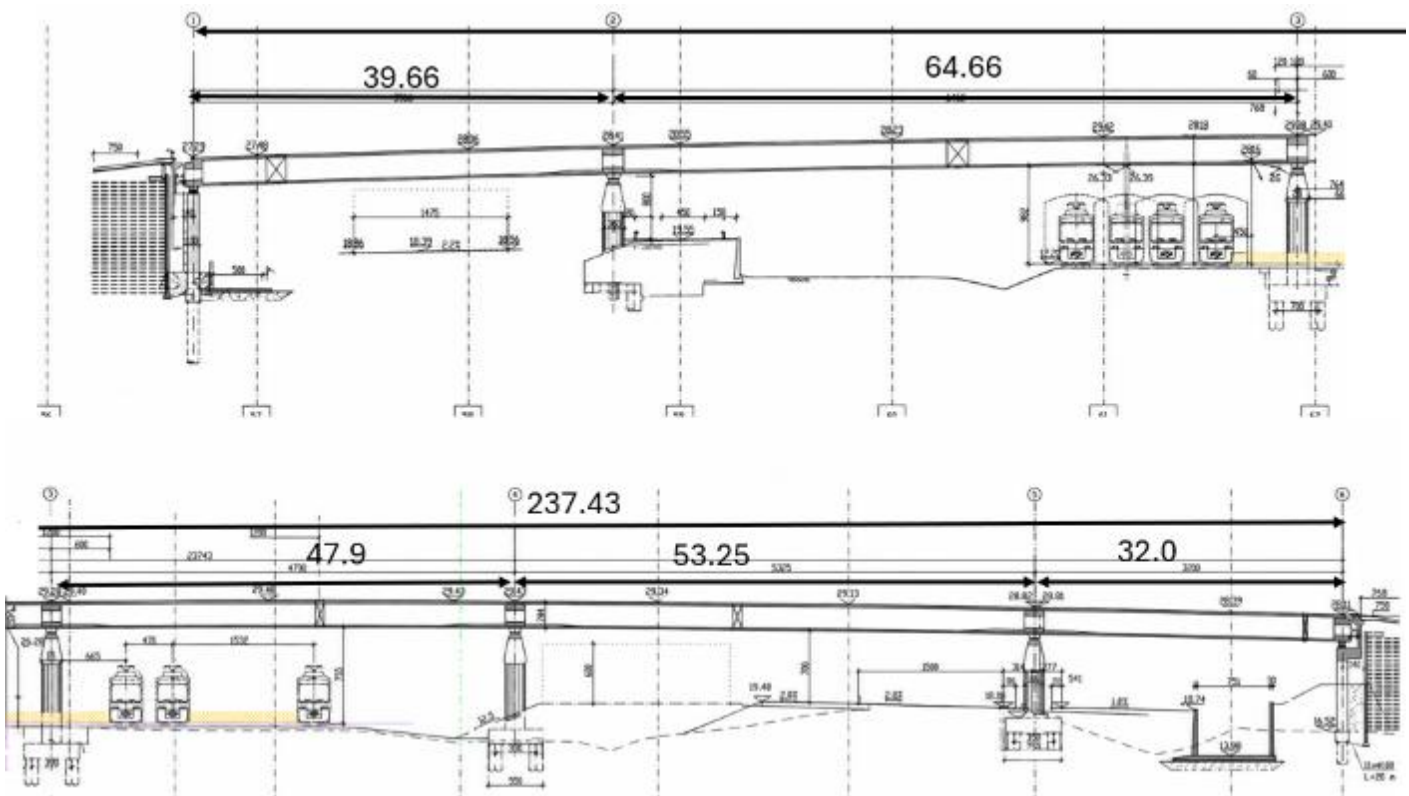


Figure 11: Final selected precast segmental arrangement. The development drawing shows the optimised five-span layout, including the approximately 65 m main span over the most sensitive part of the corridor

CROSS-SECTION, PRESTRESSING AND ANALYSIS CONCEPT

Three-cell deck arrangement. The large bridge width had a major influence on the structural concept. A single box girder for the full deck width would have been difficult to cast, transport and erect. Dividing the deck into three box-girder cells created practical precast and lifting units while maintaining the longitudinal efficiency and torsional stiffness required for the span arrangement.

Structural continuity between the cells. The three boxes were designed to work together in the final structural system. The cast-in-situ longitudinal slab closure strips completed the roadway deck and provided structural continuity across the approximately 37 m width, allowing transverse load distribution and compatibility between the cells.

At the same time, the three-cell configuration remained advantageous for construction, because each cell could be fabricated and erected as a manageable precast unit before the final closure pours tied the deck into a combined system.

Prestressing concept. The bridge uses both internal and external post-tensioning. Internal cantilever tendons resist the negative bending effects that develop during balanced cantilever erection. External continuous tendons contribute to the final continuous structural system, with deviator segments controlling tendon geometry and transferring deviation forces into the concrete box.

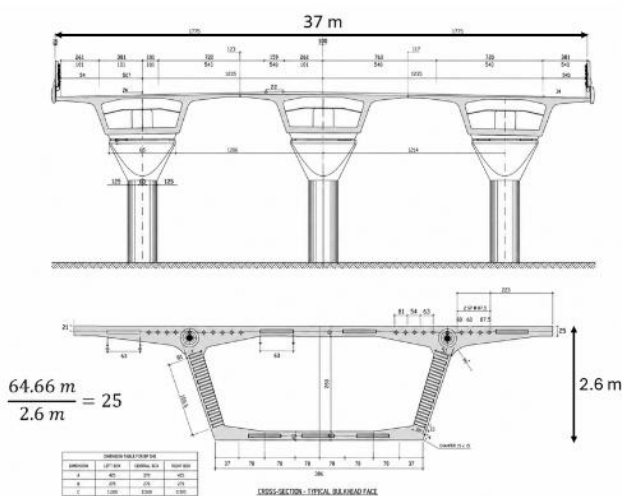


Figure 12: Typical cross-section of the approximately 37 m wide deck. The three-cell arrangement divided the superstructure into practical precast units, while the cast-in-situ longitudinal closures completed deck continuity and enabled the cells to act together in the final structural system

This combination is well suited to segmental construction because it separates the main construction-stage cantilever demand from the long-term continuity requirements of the completed bridge.

Global and local analysis. The analysis strategy combined a global structural model with several dedicated local models. The global model described the overall longitudinal behaviour of the bridge, construction stages, support reactions and prestressing effects. In parallel, refined finite-element models were prepared for critical local mechanisms: solid-element models were used for pier segments, anchorage zones and tendon force introduction regions, while shell-element models were used where transverse deck behaviour and local slab or web response required more detailed assessment. The results of these local models were checked against, and coordinated with, the force envelopes and boundary conditions obtained from the global analysis.

Critical local checks. Special attention was given to local stress verification within each box cell and within the connected wide-deck system, including principal tensile stresses in the webs and the combined effects of shear and torsion.

The relatively tall southern sidewalk introduced additional local demand. In addition, the span-to-depth ratio was close to the upper practical range for this type of structure, making the vertical component of the inclined external tendons an important contribution to shear resistance where applicable under the design assumptions.

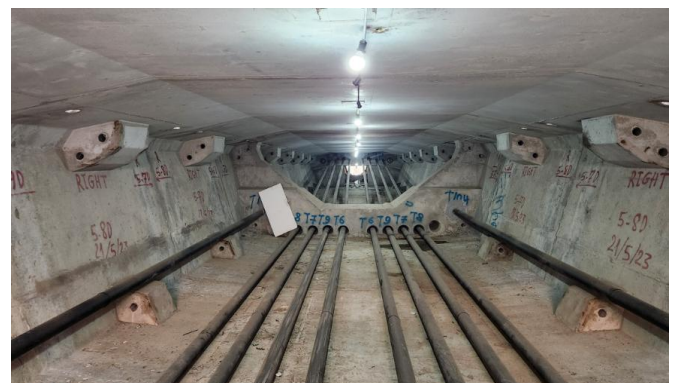


Figure 13: Interior view of a segmental box cell with external post-tensioning tendons. The external tendon system contributes to the continuous structural behaviour of the completed bridge, with tendon geometry controlled through anchorage and deviator zones

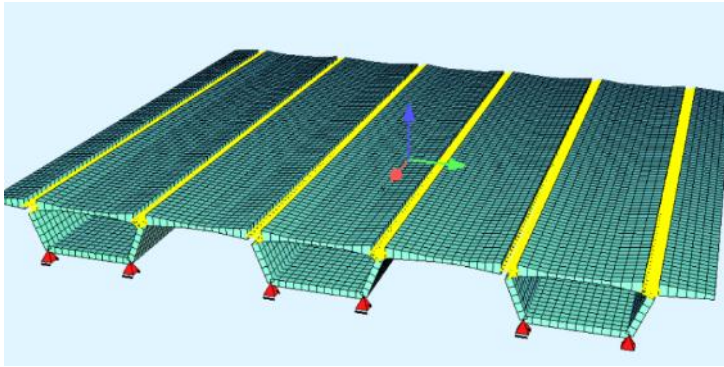


Figure 14: Shell finite-element model used for transverse verification of the wide deck system. The model supported local assessment of slab behaviour, load distribution and the structural compatibility between the three connected box-girder cells

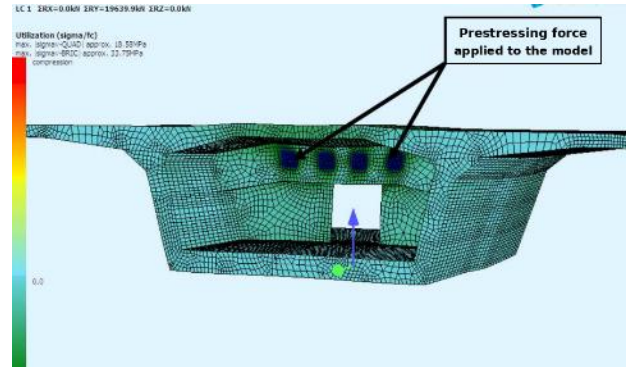


Figure 15: Solid finite-element model of a pier segment used to verify local stress flow around prestressing force introduction regions. The model was used to assess concentrated anchorage effects and their transfer into the segmental box section

CONSTRUCTION-STAGE DESIGN

Construction-stage geometry also had to be controlled throughout erection. Deflections and the erection sequence were monitored and checked at each stage, with adjustments made during construction to ensure that the opposing cantilevers met correctly at the closure segments.

Each new segment was temporarily fixed to the previously erected segment using temporary prestressing bars. After alignment and temporary fixing, the segment was permanently post-tensioned in the conventional sequence before the next segment was erected.

The design therefore had to address the sequence of segment installation. These temporary unbalanced conditions may develop during erection, prestressing stages, support reactions and local force transfer at critical segments.

Pier segments required particular attention because they transfer the large forces generated by the cantilever arms into the substructure. End segments and deviator segments also required detailed verification because they contain concentrated anchorage forces and tendon deviation forces.

CONSTRUCTION ABOVE AN OPERATING CORRIDOR

At construction level, the main implication of the site constraints was that the area below the bridge could not function as a conventional open work zone. The segmental solution reduced both the duration and intensity of operations over the most critical areas, because most of the permanent superstructure was assembled from precast elements and erected without continuous falsework below.

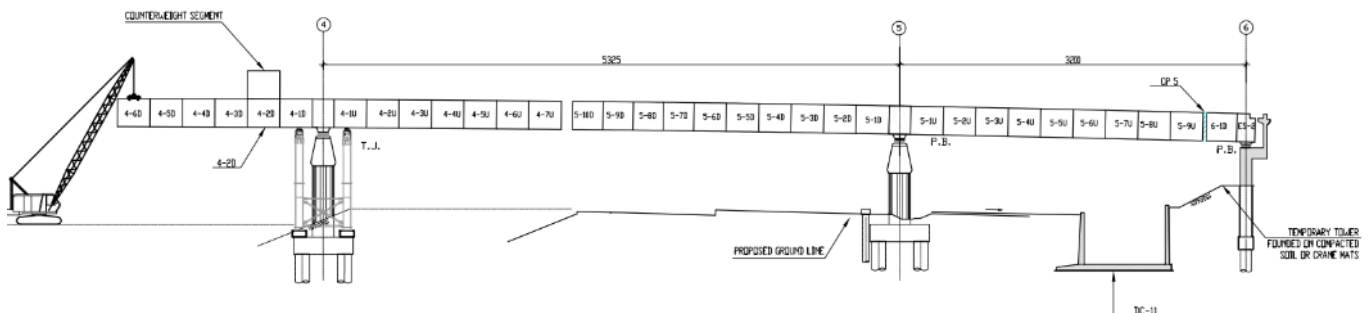


Figure 16: Construction-stage scheme for balanced cantilever erection. The diagram illustrates segment-by-segment assembly from the pier zones and the need to control temporary cantilever behaviour before structural continuity is achieved

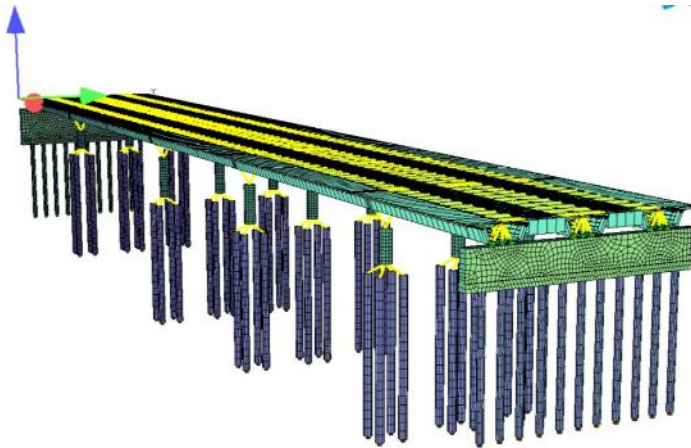


Figure 17: SOFiSTiK FEM global model of the bridge

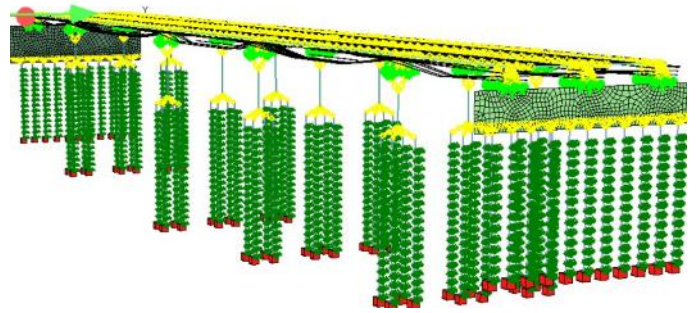


Figure 18: External cable layout in the FEM model

Precast segmental construction also supported quality control. Segment geometry could be produced repeatedly in a controlled casting environment, while erection could be planned around defined lifting, alignment and stressing operations.

This was especially important for a wide bridge with long spans and limited tolerance for disruption to the surrounding network.

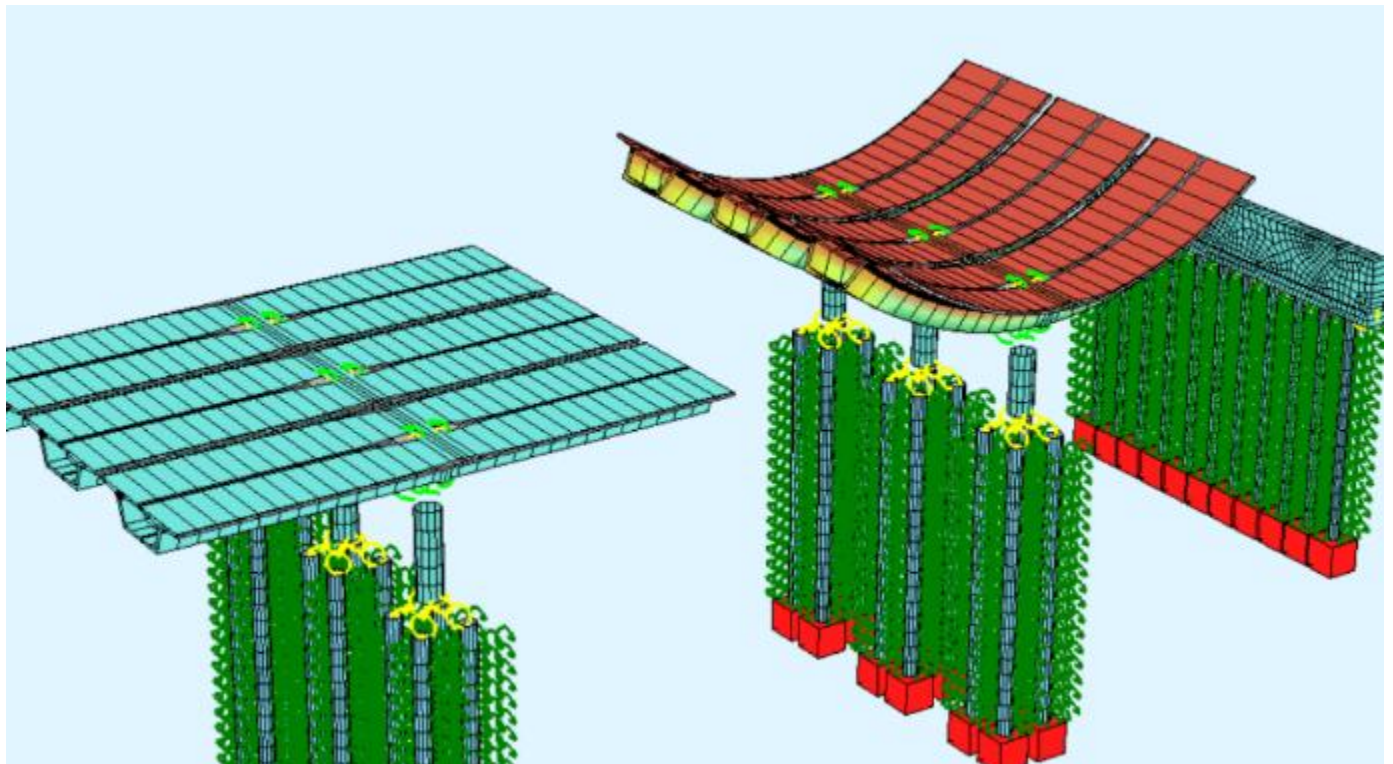


Figure 19: Global SOFiSTiK model used for construction-stage and final-state analysis of BR-541. The view illustrates the staged balanced cantilever erection sequence and the temporary structural conditions checked before the bridge reached its completed continuous configuration



Figure 20: Construction view at a pier segment during balanced cantilever erection. The photograph shows the scale of the precast segmental units and the close relationship between the superstructure, substructure and restricted working area

COMPLETED STRUCTURE AND CONCLUSIONS

BR-541 is now constructed and open to traffic. The completed bridge demonstrates how structural design and construction methodology can be integrated within a constrained transport environment. Its final form reflects a balance between span efficiency, construction-stage stability, future railway compatibility, safety and contractor economy. The segmental solution also provided architectural value. The continuous box-girder form gives the bridge a clean underside profile and a coherent pier-deck rhythm, creating a restrained appearance suitable for a major entrance corridor to the Tel Aviv metropolitan area.

The main lesson of the project is that bridge type selection was governed by the complete delivery problem rather than by final-state structural efficiency alone. The selected precast segmental balanced-cantilever solution succeeded because it combined the required span arrangement with reduced temporary works, controlled fabrication, staged erection and compatibility with future infrastructure.



Figure 23: A view of the abutment of the bridge during construction



Figure 21: Closure pour at the end segment, showing the interface between precast segmental units and in-situ closure concrete



Figure 22: Temporary post-tensioning bars used to fix a newly erected segment to the adjacent segment before permanent stressing

In BR-541, therefore, the completed bridge and the construction process were inseparable: construction-stage behaviour, local force transfer and practical erection constraints shaped the final structural form as much as the permanent service condition.



Figure 24: Completed BR-541 bridge viewed from below after opening to traffic. The final structure presents a continuous segmental deck supported by slender piers, combining structural efficiency, constructability and a clean architectural expression

WHEN INFRASTRUCTURE GETS COMPLEX: PARAMETRIC DESIGN ON LAKE LUCERNE

Gregor Strekelj, Product Manager, Civil Infrastructure, Allplan



*Figure 1: A spectacular piece of alpine infrastructure on Lake Lucerne in Switzerland, designed with Allplan Civil
Copyright: Bänzinger Partner*

INTRODUCTION

A 400m section of cantonal road K2b above Lake Lucerne, built in the 1880s and last widened before the Second World War, was rebuilt as a 224m-long back-anchored cantilever structure.

With landscape preservation and traffic continuity ruling out the obvious alternatives, the resulting geometry – 55 transverse ribs, a polygonised rear wall, and prestressed rock anchors threading past a wartime bunker – would have been exceptionally difficult to detail without parametric modelling.

Europe's secondary road network is full of cantonal and minor routes that no longer meet modern standards.

Many were laid down in the 1880s or 1890s for horse-drawn traffic, have been widened modestly since their construction and have carried heavier and faster vehicles than their builders ever anticipated.

Replacement is rarely an option – these are often the only connections between communities – and conventional widening would have to overcome the original reasons the road took that line in the first place: cliffs, water, geology, or all three.

The K2b cantonal road on the eastern shore of Lake Lucerne is a textbook example. It connects Greppen, Weggis, Vitznau and Gersau to the

regional centres of Küssnacht and Brunnen, and forms the local link to the Swiss national road network. The 400m stretch between Bürglen and the Lucerne/Schwyz cantonal border was built in 1885–86 and widened in 1939.

It sits in a poor structural condition, with lanes too narrow for the current Swiss standard. The Canton of Lucerne's brief was straightforward enough: a minimum lane width of 7 m and a footpath of at least 2.60 m, all delivered in roughly two years and within a budget of CHF 22.9 million.

However, the site is anything but straightforward. The road runs along an exposed cliff face directly above the lake, with very steep terrain above and the water immediately below. There is no room to move the alignment outward without building over the lake, and no room to bring it inward without cutting deep into the rock.

A NINETEENTH-CENTURY ROAD WITH TWENTY-FIRST-CENTURY DEMANDS

The structural concept that resulted – a 313m-long supporting structure that combines a back-anchored cantilever with ribs in the steepest sections and angled retaining walls on the gentler flanks – was not the first option considered.

Bänziger Partner AG, the firm responsible for the engineering structures and technical site management, took on the project after the early concept phase.

According to Stephan Etter, an ETH-trained structural engineer and member of Bänziger's management team, two alternative concepts were examined and rejected before the cantilever solution was chosen.

The first was a roadway slab supported on reinforced concrete ribs of variable height, with each rib adapted to the rock profile beneath it. This was rejected on landscape grounds, as the visible underside of the structure would have looked like a row of mismatched columns against the cliff.

The second was a cantilevered slab resting on, or interlocked with, the rock face on the mountain side. This was rejected because it would have required long-term road closures during construction, which the Canton ruled out in favour of single-lane traffic management with a light system and only two three-week full closures.

For the 20m main span across a rock cut, a separate variant using a prefabricated girder was also considered. It was rejected in favour of carrying a single structural system across the whole 313 m.

The coordination, detailing, and inspection advantages of one consistent concept outweighed the construction-sequencing benefits of dropping in a precast element.

"In addition to the topological and geological conditions and the requirements for the standard road profile," Etter explains, "The decisive factors

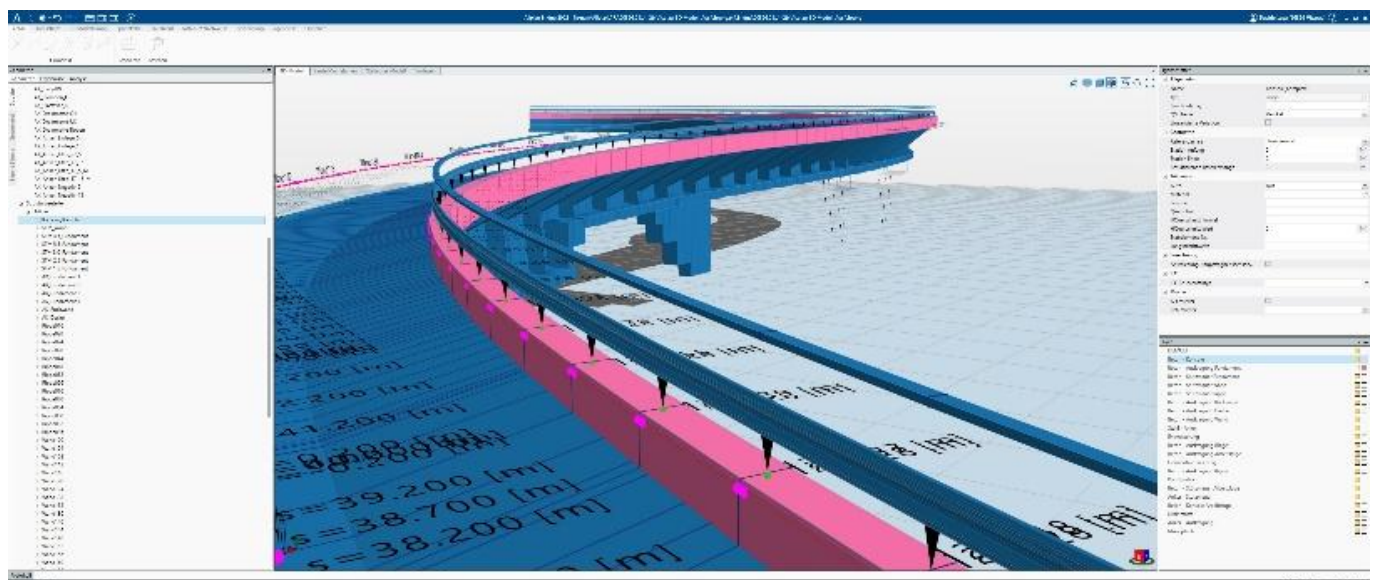


Figure 2: 3D model of the complex retaining structure in ALLPLAN Civil, showing its curved geometry and continuous adaptation to the road alignment. Copyright: Bänziger Partner

were ensuring the road stayed open during construction using a single lane, and conserving the existing landscape.”

What the design team was left with was a structural concept that resolves the constraints elegantly, but at the cost of geometric complexity that would have been incredibly difficult to detail and deliver using conventional 2D methods.

MODELLING 55 UNIQUE RIBS

The cantilever consists of an up to 60cm-thick cantilever slab carried on cross plates spaced at 4m intervals, which connect to the rear wall and a strip foundation to form an integral, robust structure.

The cantilever is primarily anchored into the rock with prestressed anchors. Across the 224m length, 55 transverse ribs sit beneath the roadway slab in a grid running roughly perpendicular to the road axis.

The complication is that none of those ribs is identical. The downhill side of each rib follows the edge of the road, and the uphill side ends at the rear wall.

However, the rear wall itself follows a freely defined line, which is beautifully shaped without too many curves.

It was specifically designed this way as more aggressive curvature would have caused the prestressed anchors to overlap. The rear wall is then polygonised rather than truly curved for cost reasons, with an 8m grid on the wall itself and a 2m grid on the console heads.

Every rib therefore has different geometric dimensions, and each rib has to remain coordinated with its segment of the polygonised rear wall.

Modelling this in a conventional CAD environment would mean drawing 55 ribs individually and then redrawing all of them every time the rear wall geometry shifted. The team at Bänziger built the structure parametrically in ALLPLAN Civil instead. The deck and ribs were defined by cross-sections extruded along the road axis, and the polygonisation of the rear wall and console head was developed from the same parametric model rather than imposed on it afterwards. Geometry changes could then be propagated through the model without manual reworking.

“The parametric model creation with ALLPLAN Civil was perfect for this project,” says Davide Luca, Bänziger’s responsible designer. “It allowed very precise and efficient processing, which would not have been possible a few years ago.”

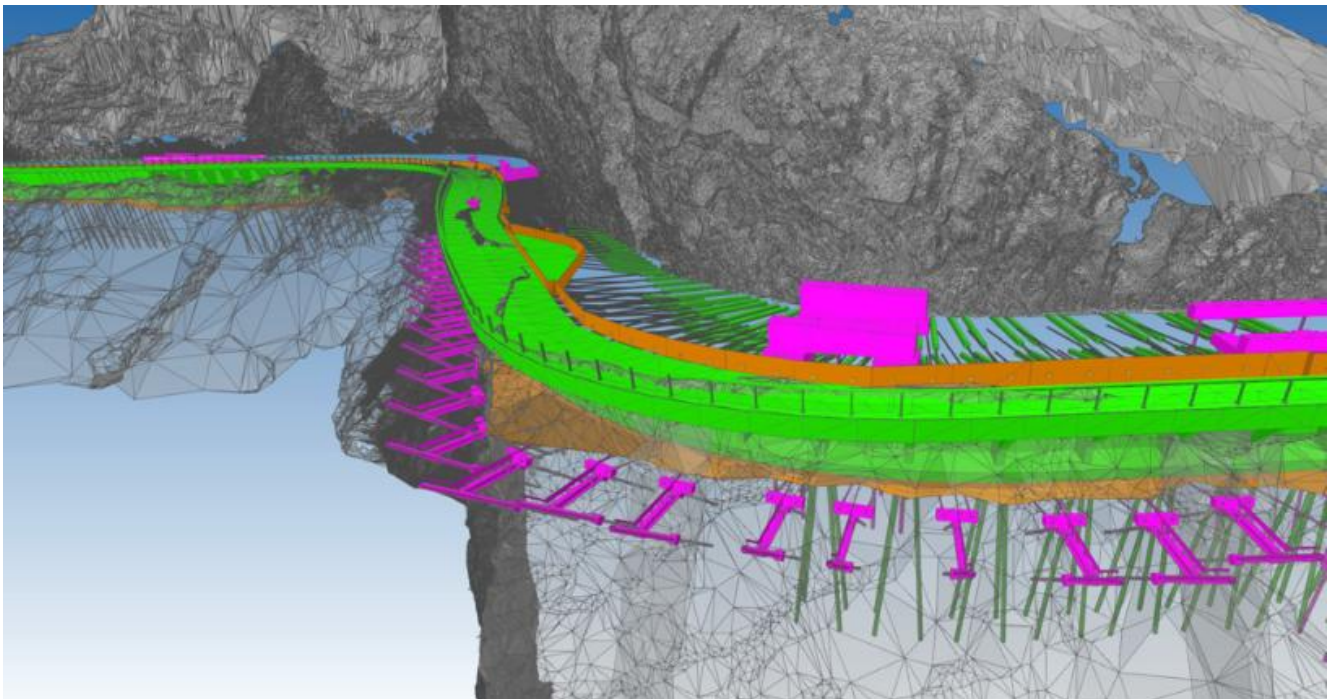


Figure 3: The detailed BIM model integrates the retaining structure, anchoring systems and complex terrain, enabling potential clashes to be identified during the planning phase. Copyright: Bänziger Partner

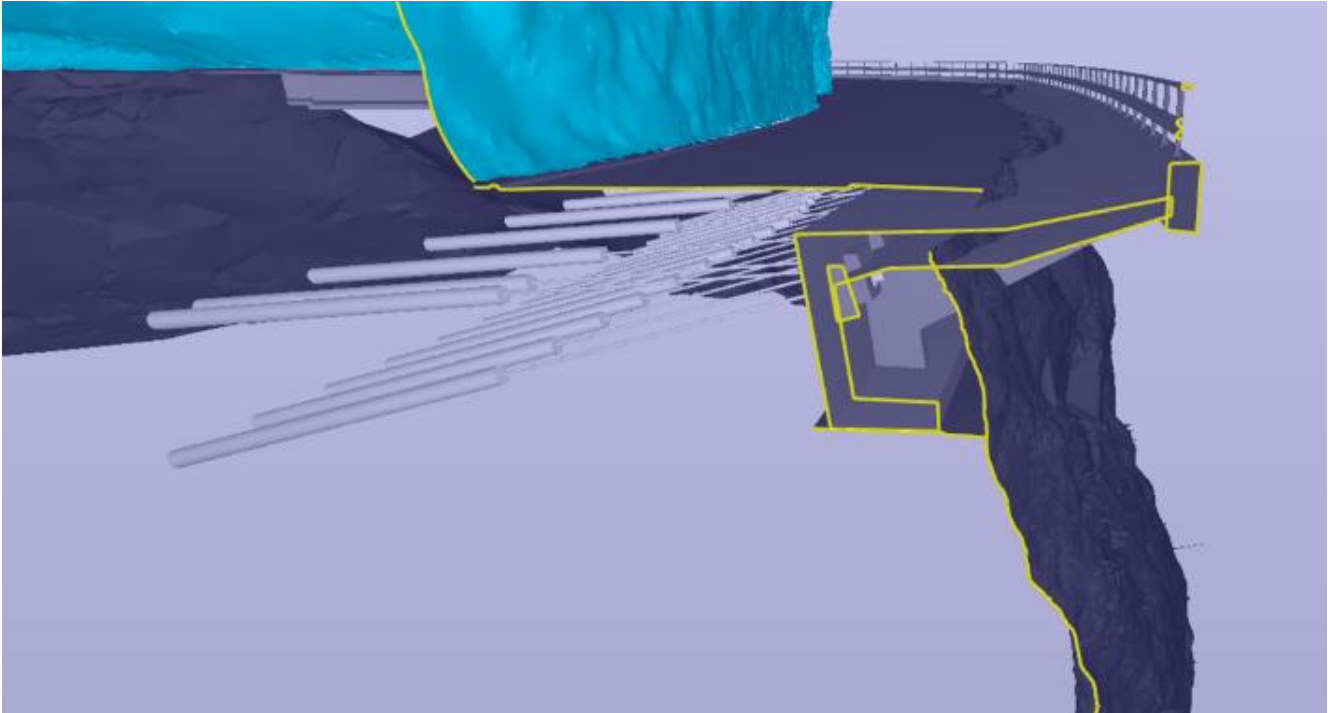


Figure 4: Model-based coordination and clash checking helped identify potential conflicts between the retaining structure, anchors and complex rock geometry at an early stage. Copyright: Bänzinger Partner

Etter is clear about what this means for how Bänziger now approach this kind of project. “In early project phases, parametric modelling is very helpful because changes can be easily incorporated. Sometimes we even start modelling retaining structures before the road edges have been definitively determined. We would never have done that in the past.”

He cites a comparable Bänziger project, also above Lake Lucerne in Gersau and around 700 m long, where the valley side is formed by a monolithic reinforced concrete structure without any breaks.

Minor changes to a continuous wall crown on a project of that length, Etter notes, become very time-consuming if the model is not parametric – and even more so if the team is working from 2D plans rather than a model.

COORDINATION IN A CROWDED ROCK MASS

A back-anchored cantilever in this setting is not just a structural problem. The rock above and behind the road has to accommodate a dense network of intersecting elements: temporary excavation anchors for the construction phase, permanent prestressed anchors carrying the cantilever, unstressed rock support anchors, micropiles, micropiles for the crane foundations, and anchors for the working scaffolding.

Some of these are permanent, some temporary, and some have to coexist with each other and the rock itself.

The rock itself contained a surprise. A sprawling military bunker complex extends through the cliff face across a vertical span of around 60 m, and the design team began the project with only archive drawings to work from.

“During the project, there were always uncertainties regarding a large bunker complex in the rock,” Etter says. “After repeated inquiries regarding conflicts with the anchors, the complex was modelled based on the archive plans. Validating the model, together with the terrain model, was very straightforward. From that point on, the issue was resolved and everyone involved was reassured.”

Other geometric clashes were less dramatic but no less consequential. The team had to coordinate the intersection of anchors with the micropiles supporting the crane foundations and resolve the relationship between traffic routing on the mountain side and the open excavation pit on the lake side. Each of these was identified and worked through in the 3D model rather than discovered on site.

The contractor, a consortium of Marti Bauunternehmung, Contratto, and Walo Bertschinger, also

requested the work platform and crane concept as a 3D model at tender stage, so that conflicts between temporary works and the permanent structure could be checked before construction began. IFC models were exchanged through a planning platform shared across the project team.

Etter is realistic about where the 3D model earned its place and where it did not. Not every coordination question needed automated clash detection: “Of course, there are also intentional conflicts – for example, when you want the foundation to rest directly on the rock. However, for these projects, we generally perform the collision checks visually.”

It is a useful corrective to the assumption that BIM coordination is a single uniform process. On K2b, the model was the indispensable tool for resolving uncertainty about hidden conditions and intersecting linear elements. Visual inspection of the model remained the right tool for intentional, designed-in contact between the structure and the rock.

THE POSSIBILITIES OF PARAMETRIC MODELLING

Construction took place between September 2023 and December 2025. The total amount of rock excavated is around 7,300 cubic metres, and the structure was built under single-lane traffic management with only two scheduled full road closures. The wider lesson of the K2b project sits at the intersection of structural choice and modelling capability.



Figure 5: Construction of the reinforced concrete road deck in the challenging terrain above Lake Lucerne. Copyright: Bänzinger Partner

Landscape preservation and traffic continuity demanded a cantilever solution with demanding geometry: 55 unique ribs, a polygonised rear wall, and anchors threading through rock that already contained a bunker. Parametric modelling kept that geometry manageable from concept through to construction documentation, so the complexity of the chosen concept did not become a project risk in its own right.

Across Europe's older secondary road network, the constraints that shaped the K2b project – narrow alignments, sensitive landscapes, the need to keep traffic moving – reoccur often. On sites like these, the structural concept and the modelling workflow have to be considered together: a concept that resolves the site's constraints is only viable if the team can detail and deliver the geometry it produces.



Figure 6: The completed retaining structure follows the curved road alignment through the steep terrain above Lake Lucerne. Copyright: Bänzinger Partner

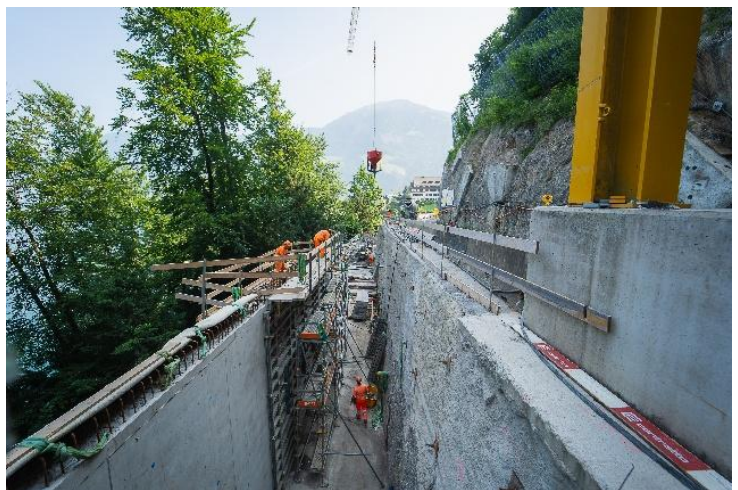


Figure 7: The confined construction site between the steep rock face and Lake Lucerne posed major logistical and construction challenges. Copyright: Infinity Films



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FROM BRIDGES TO PROSPERITY TO FIKA: BUILDING SYSTEMS THAT MAKE RURAL ACCESS INEVITABLE

Nicola Turrini, Global Corporate Partnerships Manager
Caitlin McWhorter, Global Marketing & Communications Director



Figure 1: Bundimulinga Bridge – Uganda Photo Credit: Robb Hohmann

INTRODUCTION: NEW NAME FOR A NEW CHAPTER

After more than two decades of designing and building rural transport infrastructure across 21 countries in Sub-Saharan Africa, Central and South America, and Southeast Asia, Bridges to Prosperity has evolved.

Our new name, **Fika**, reflects a fundamental shift in how we approach rural connectivity and our role in achieving it at scale.

In Kiswahili, Zulu, and other African languages, "**fika**" means **"to arrive,"** with the connotation of arriving **safely**. This simple word captures both our promise and our evolution: from building individual bridges to enabling entire systems that make safe, reliable rural access inevitable for everyone, everywhere.

Over 24 years, we've built or supported more than 800 trail bridges: suspension, suspended, hybrid, and stone arch designs, connecting over 5 million people to schools, markets, healthcare, and

economic opportunity. We've proven that well-designed, climate-resilient, sustainable pedestrian infrastructure can transform rural lives.

A randomised control trial (RCT) of over 100 bridge sites, the most rigorous evaluation of built rural transport infrastructure for poverty alleviation to date, confirmed what communities already knew: when you remove impassable barriers, everything changes; household income, educational attainment, food security, maternal health, and social cohesion all improve measurably.

But proving impact at the bridge level was never the end goal. It was the foundation for something bigger.

Today, over one billion people remain physically isolated from essential services, unable to reach healthcare during emergencies, markets to sell their goods, or schools for their children. Building one bridge at a time, even 800 of them, will never close this gap. What's needed is systems change: government prioritisation of rural access in national plans, sustainable funding mechanisms integrating rural access into larger transport investments, local capacity to design and build independently, and policies that make connectivity a standing commitment rather than a project-by-project effort.

Fika represents our commitment to that systems-level transformation. This article outlines our 2026-2030 strategy, a roadmap for how we're shifting from implementer to catalyst, from builder to enabler, and from demonstration to influence.

THE CHALLENGE: SCALE REQUIRES SYSTEMS

The engineering community understands infrastructure gaps in quantifiable terms: kilometres of missing roads, number of impassable crossings, populations without year-round access. By these metrics, the rural access deficit is staggering.

In Sub-Saharan Africa alone, an estimated 70% of rural populations lack reliable all-season access to essential services. Climate change is making the problem worse, with unpredictable rainy seasons, flooding, and erosion reshaping riverbeds overnight and washing away traditional crossing points that communities have relied on for generations.

Trail bridges, whether long-span cable bridges or short-span stone arch structures, offer a proven, cost-effective solution. A typical 60m suspension bridge costs approximately USD \$100,000 and serves 2,000-4,000 people directly, with total catchment populations often reaching 10,000+ in densely populated areas in Sub-Saharan Africa.



Figure 2: Jigesa Bridge Identify Site – Ethiopia

Photo Credit: Robb Hohmann

These structures are climate-resilient when properly designed and maintained, with clear spans that keep foundations protected from flooding effects and materials selected and designed for durability in harsh conditions.

But here's the fundamental challenge: even if we scaled our construction capacity tenfold, we could build perhaps a few hundred bridges per year. At that rate, closing the access gap would take centuries. The math simply doesn't work.

What does work is this: enabling governments to independently plan, fund, design, build, and maintain rural transport infrastructure at the national scale. When the Government of Rwanda integrates trail bridges into the National Feeder Roads Program, suddenly, rural access becomes a standing budget line rather than a donor-dependent project. When the World Bank commits USD \$35 million to build 373 bridges in Ethiopia through the Rural Connectivity for Food Security Program (RCFSP) in partnership with multilateral development partners, rural access moves from the margins to the mainstream of infrastructure investment. When Ugandan contractors complete their first trail bridges independently, with Fika providing technical assistance rather than direct implementation, local capacity becomes the foundation for sustainable scale.

This is systems change. And it requires a different organisational model than the one that built the 150m long Cyarera Bridge (*e-mosty* - *December 2022*) or any of our previous landmark structures.

OUR STRATEGIC EVOLUTION: THREE CORE THEMES

Our 2026-2030 strategy centres on three interconnected themes, each essential to achieving sustainable rural access at scale.

Theme 1: Rural Access is Prioritised

The Problem: Rural connectivity remains absent from most national development plans and global infrastructure frameworks. When rural access does appear in policy documents, it's often vague, unfunded, and disconnected from measurable targets.

Our Approach: We're working to embed rural access as a tracked, funded priority in national and international development agendas.

This starts with better metrics. We're launching the Safe Access Scorecard, a measurement framework that tracks travel time from rural households to critical destinations (schools, health facilities, markets). Rather than counting bridges built or kilometres of road paved, the Scorecard asks: Can rural families reach essential services safely within acceptable timeframes?

By 2030, we aim to see the Safe Access Scorecard adopted by at least seven national governments and integrated into development frameworks used by multilateral organisations.

When rural access has clear indicators and target thresholds, just as urban transport planning uses travel time benchmarks, it becomes something governments can measure, budget for, and be held accountable for delivering.

We're also positioning rural access within broader development priorities that governments already care about: poverty reduction, economic resilience, maternal and child health, educational outcomes, and climate adaptation. RCT data and ongoing impact research provide the evidence base for these connections.

Theme 2: Rural Access is Funded

The Problem: Rural infrastructure competes for funding against larger, more visible urban projects. Traditional development finance focuses on high-cost interventions, while innovative mechanisms for rural transport remain underdeveloped.

Our Approach: We're mobilizing diverse financing streams, government co-financing, blended finance, climate adaptation funds, and private sector engagement, to unlock sustainable rural access funding.

In Zambia, we've pioneered a co-financing model where local governments commit budgets alongside donor funds, building ownership from day one. We started with USD \$250,000 in government co-financing in our first operational year and are targeting USD \$500,000 in 2026.

In Ethiopia, the Rural Connectivity to Support Food Security Program (RCFSP) represents the largest government-owned rural access program in Sub-Saharan Africa, with trail bridges integrated into a USD \$407 million World Bank investment connecting over 9 million people.

Our target: mobilize **USD \$100 million** in rural access funding by 2030. This includes government budget allocations, multilateral investments, climate finance, and corridor development funds. We are developing investment cases that demonstrate rural transport infrastructure's return on investment in government terms, GDP growth, poverty reduction, healthcare cost savings, educational attainment, making the economic argument for rural connectivity impossible to ignore.

Theme 3: Countries Meet Rural Access Needs Independently

The Problem: Donor-dependent infrastructure programs don't scale and don't last. Sustainable rural access requires governments and local actors to own the full delivery chain, planning, funding, design, construction, and maintenance.

Our Approach: We're transferring technical capacity, tools, and systems so countries can deliver rural access without us. This includes:

Technical Training: We're training government engineers, private sector contractors, and university faculty in trail bridge and rural infrastructure design, construction, and maintenance.

In Ethiopia, we've trained 21 fabricators in steel component manufacturing, 50 consulting firms (100+ engineers) in surveying and design, and 140 local contracting firms in construction. In Uganda, our training of trainers (TOT) approach embeds expertise directly in government institutions and the private sector.

Design Standards & Manuals: We're working with governments to adopt national trail bridge design standards and construction manuals. Rwanda's RTDA (Rwanda Transport Development Agency) has adopted our Trail Bridge Manual, integrating our designs into national road standards. Uganda's Ministry of Works and Transport (MOWT) is finalizing similar adoption. These are not vanity documents. They are the technical specifications that enable governments to tender, procure, and oversee rural infrastructure implementation independently.

Digital Tools: Our Fika Digital suite, including Fika Collect (mobile data collection), Fika Map (geospatial analysis), and impact modelling tools,

helps governments identify where rural infrastructure is needed, prioritise investments, and track outcomes. We're onboarding governments in Rwanda, Uganda, Ethiopia, and Zambia, with a target of 10+ government entities using Fika Digital tools by 2030.

Local Private Sector Engagement: In Uganda, we've piloted private sector-led bridge construction, with contractors now building bridges under government contracts, with Fika providing supervision and quality assurance rather than direct implementation.

In Rwanda, we're targeting five bridges built by private contractors in 2026. This model proves that local construction firms can deliver quality rural transport infrastructure solutions when given proper training and oversight.

Maintenance Systems: Rural transport infrastructure programs are only as good as their maintenance programs. We are establishing maintenance frameworks in each country, training bridge committees within the served communities, setting up inspection schedules, and creating government-managed maintenance budgets. In Rwanda, we are piloting a model where districts directly contract maintenance to local actors.

Our boldest goal: by 2030, sunset our support in at least two regions where governments are delivering rural access programs independently, without ongoing Fika involvement. Success, for us, means working ourselves out of a job.



*Figure 3: Kasenga Bridge – Zambia
Photo Credit: Robb Hohmann*

WHAT THIS MEANS FOR ENGINEERING & IMPLEMENTATION

For the engineering and construction professionals reading this, our strategic shift has concrete implications:

Trail Bridge Design Standardisation with Local Adaptation: We maintain a portfolio of standardised trail bridge designs (suspension, suspended, hybrid, stone arch) covering spans from a few meters to 150+ meters. Standardisation enables efficiency, cost control, and rapid deployment. But standardisation must be balanced with local adaptation, soil conditions studies, hydrologic analysis, material availability, and construction capacity, all of which vary significantly across geographies. Our approach: standardised core systems (cable configurations, tower designs, foundation types) with site-specific customisation (substructure sizing and positioning, lateral stabilisation systems for long spans).

Climate Resilience by Design: Every infrastructure asset we design accounts for climate variability and extreme weather events. The right type of bridge is selected for each specific valley, with the goal of protecting the foundations from flooding and scouring: this sometimes means working with long-span cable solutions to place the foundations far away from the present and future placement of the riverbed, and other times means locating strong soil or rock where to place a resilient short-span stone-arch solution. Materials are selected for durability, and local availability, which ultimately could mean designing steel elements to be galvanized (like we do in Uganda) or painted with protective paint (like we do in Rwanda) depending on what the local possibilities are. The Cyarera Bridge's 150m span required custom wind-guy cables and additional anchor blocks, used to stabilize long-span bridges against lateral wind loads, precisely because climate resilience demanded a span beyond our standard designs.

Buildability & Local Capacity: A design is only successful if local teams can execute it. We design for hand construction methods and locally available materials wherever possible, in addition to being constructed by local labour from the same served communities. While an international pipeline of donated reused wire-rope cables, cable clips and steel pipes (used for the fabrication of suspension

bridges' towers) is bringing non-locally available materials to our country programs; all the rest of the construction materials are locally available, and the needed steel fabrication works for elements like towers, deck panels, and crossbeams happen in the shops of our local fabricator partners. In Rwanda, for example, stone arch bridges, built entirely with local materials and labour, are becoming a very appreciated bridge typology for certain site conditions, being extremely cost-effective and fully "made in Rwanda".

Quality Assurance: As we transition from direct implementation to technical assistance, quality assurance becomes even more critical. We've developed design, quality and safety procedures, inspection protocols and construction protocols that government engineers and private contractors can be trained on, with the intent of coming to use it independently.

Maintenance-First Design: Rural transport infrastructure assets must be designed for maintenance from day one. That is why we work closely and train government partners in asset management, while making sure local community members are upskilled not only in bridge construction methods, but also in inspection and maintenance practices. The main structural elements of our bridges are designed to last for the entire lifespan of the structure, while more vulnerable elements (ie fencing) are designed to be easily budgeted for and replaced when needed. A well-maintained bridge is designed to last more than 30 years!

Cost Optimisation Without Compromising Safety: Government budgets are constrained. Our challenge: deliver safe, durable bridges at the lowest possible cost. We've driven costs down through design efficiency, material optimization, efficient local and international procurement, and strong global partnerships.

EVIDENCE AS THE FOUNDATION

Everything we do is grounded in evidence. The RCT of our bridge sites in Rwanda (2019-2025) demonstrated that trail bridges increase household consumption by 31%, secondary school enrollment by 22%, and market participation by 18%. Women's mobility increases by 60%, enabling access to healthcare during pregnancy and childbirth.

Agricultural productivity rises as farmers can transport goods to higher-value markets.

We are continuing to build this evidence base. Current research includes:

- **Impact Modelling:** Using machine learning to identify rural access routes and crossing sites from satellite imagery, then modelling the potential impact of rural transport infrastructure investments using actual demographic and economic data.
- **Cost-Benefit Analysis:** Quantifying the return on investment of sub-tertiary transport assets (from trail bridges to footpaths to ancillary transport infrastructure) in economic terms, cost per DALY (disability-adjusted life year) averted, cost per child-year of education gained, contribution to GDP growth.
- **Maintenance Economics:** Studying optimal maintenance intervals, cost-effectiveness of different maintenance approaches, and long-term infrastructure durability under various climate scenarios.
- **Comparative Bridge Typology Analysis:** Evaluating when long-span cable bridges, short-span stone arch bridges, or other typologies of rural infrastructure are most appropriate based on geotechnical and hydrological site conditions.

This research serves two purposes: First, it informs our technical approach, helping us design better bridges and more effective, holistic rural access programs. Second, it provides the evidence governments and funders need to prioritise and finance rural access at scale.

BY THE NUMBERS: OUR 2030 TARGETS

Clear goals drive accountability. By 2030, we aim to:

- 10 million additional people connected to essential services through safe, reliable transport infrastructure
- USD \$100 million mobilized for rural access programs (government budgets, multilateral investments, climate finance, private sector)
- Rural access embedded in 7+ national development plans, with measurable targets and dedicated budgets
- Safe Access Scorecard launched and adopted as a standard metric
- At least 2 regions delivering rural access programs independently, no longer requiring Fika's ongoing support
- 10+ governments using Fika Digital tools for rural access planning and monitoring
- **5+ research papers** published in peer-reviewed journals, advancing the evidence base for rural transport infrastructure



Figure 4: Rwanda Cyabami
Photo Credit: Fika



Figure 5: Bambala Bridge - Ethiopia
Photo Credit: Robb Hohmann

With the right partnerships and sustained commitment, these targets are achievable.

Country Programs: Where Strategy Meets Implementation

Our four country programs demonstrate different stages of this systems-change journey:

Rwanda (14 years): Our longest partnership is transitioning from implementation to technical excellence. Rwanda has integrated trail bridges into national road standards, adopted our design manual, and is building capacity through partnerships with Rwanda Polytechnic.

Uganda (12 years): We're accelerating the private sector transition. The first pilot bridges built by local contractors are complete, with 10+ more planned for 2026. We are also embedding technical capacity directly in government through a seconded engineer position in the Ministry of Works and Transport, a model for sustainable capacity building that puts expertise where it's needed long-term.

Ethiopia (4 years): The scale-up at a systems level is happening - and working. The TRAIL Phase II project's success (61 bridges, extensive capacity building) led to a USD \$35 million World Bank investment for 373 bridges by 2029. We are registering as a local entity, building a foundational in-country team, and integrating the TRAIL III follow-on with the USD \$407 million Rural Connectivity for Food Security Program that will connect over 9 million Ethiopians. Ethiopia shows what is possible when government ownership and multilateral finance align.

Zambia (2 years): Starting from scratch, we're proving that rapid scale-up is possible with the right approach of government-aligned partnerships for systems change. We built 2 pilot bridges in year one, scaling to 14 in 2026, with USD \$250,000 in government co-financing secured and USD \$500,000 targeted. Zambia is our testing ground for climate finance mobilisation and corridor development integration.



Figure 6: Kasenga Bridge – Zambia
Photo Credit: Robb Hohmann

THE ROAD AHEAD: PARTNERSHIPS FOR SCALE

We cannot do this alone, nor should we. Achieving our 2030 goals requires partnerships with:

Governments: The ultimate owners and deliverers of rural access. We provide technical assistance; they provide leadership, budgets, and long-term commitment.

Global Development Finance Partners: World Bank, AfDB, and regional development banks have the financing scale and policy influence to mainstream rural access. We're working to position trail bridges and rural networked transport infrastructure as fundable, scalable interventions.

Engineering Firms & Construction Companies: We are excited to collaborate with the vast array of partners who can bring technical expertise, project management capacity, and construction excellence to rural access programs, whether as advisors, trainers, or implementers, and who are interested in collaborating with Fika to make rural access an embedded, non-negotiable component of cross-sectoral programs and investments.

Research Institutions: Universities and research organisations help us strengthen the evidence base, refine our technical approaches, and disseminate findings to the broader infrastructure community.

Technology Providers: Digital tools, satellite imagery, machine learning, and data analytics are transforming how we plan and monitor rural infrastructure. We are seeking partners who can help scale Fika Digital and integrate new technologies.

Funders: Philanthropic foundations, bilateral aid agencies, and impact investors provide the catalytic funding that proves new models, builds government capacity, and enables innovation.

FROM BRIDGES TO SYSTEMS

The Cyarera Bridge, our 150-meter suspension bridge in Rwanda, is an example of how we constantly push the technical limits of what we can achieve. It required custom engineering, an additional complex lateral cable-stabilisation system, customised towers much taller and heavier than usual, and a total of more than 1300m of main cables to withstand the huge load of this exceptional structure and of the thousands of community members that cross it every day.

It cost twice as much as a typical bridge. And it connected three communities that had been isolated for generations.

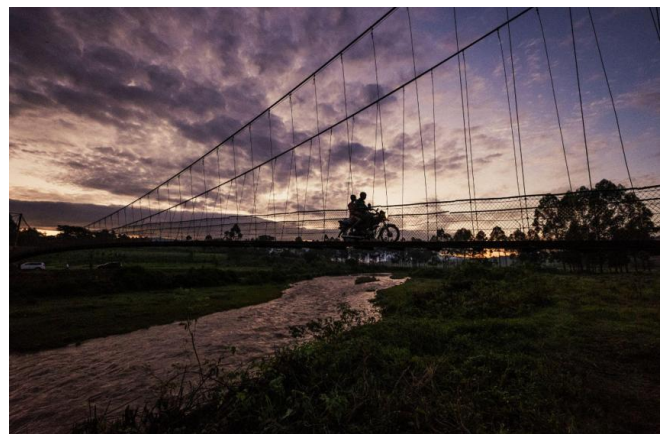
But Cyarera, remarkable as it is, is still just one bridge. Our 2026-2030 strategy is about building something bigger than any single structure: the systems that make bridges like Cyarera, and thousands more, inevitable. It is about governments knowing where bridges are needed, having the budget to build them, possessing the technical capacity to design and oversee construction, and maintaining the infrastructure for decades to come. It's about rural access becoming a standing priority rather than a project-by-project effort.

Fika means "to arrive safely." Over the next five years, we are working to ensure that safe arrivals become the norm, not the exception, for the one billion people who still lack reliable access to opportunity.

The engineering is proven. The evidence is clear. The partnerships are forming. Now it's time to scale and make safe access inevitable.

ABOUT FIKA

Fika (formerly Bridges to Prosperity) partners with governments to enable safe, reliable rural transport access at national scale. Over 25 years, we've connected 5+ million people across Rwanda, Uganda, Ethiopia, and Zambia through trail bridges and rural networked transport infrastructure. Our 2026-2030 strategy focuses on systems change: embedding rural access in national policies, mobilising sustainable funding, and building local capacity so governments can deliver independently.



*Figure 7: Tongole Bridge – Uganda
Photo Credit: Robb Hohmann*



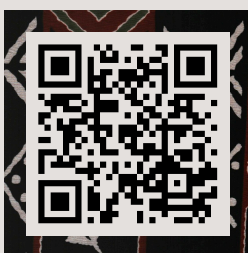
We envision safe arrivals for everyone, everywhere.

A world where over one billion currently underserved rural people are connected to essential services and opportunities.



Our mission is to enable safe, reliable rural transport access by helping governments and local actors independently plan, fund, design, build, and maintain the infrastructure their communities need.

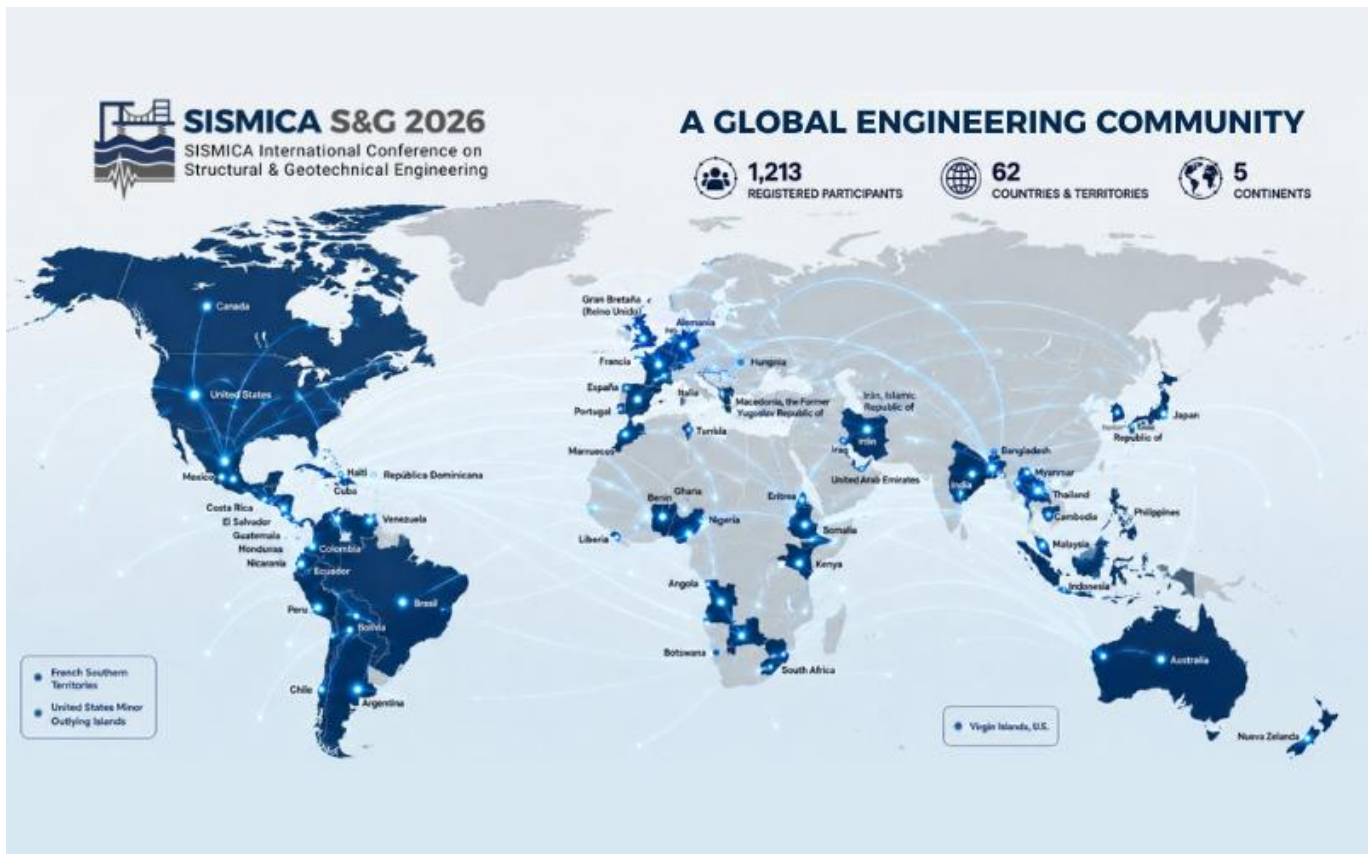
Partners help us turn this vision into reality by supporting rural infrastructure that transforms lives and strengthens economies.



Together, we're building the systems that make safe access inevitable.

SISMICA S&G 2026: BRINGING STRUCTURAL AND GEOTECHNICAL ENGINEERING TOGETHER IN REAL-WORLD PRACTICE

Edinson Guanchez, CEO, Sismica Institute



From 18 to 20 June 2026, SISMICA Institute hosted the first edition of SISMICA S&G 2026 — International Conference on Structural & Geotechnical Engineering, an online international conference created to bring together two disciplines that are often discussed separately, but are deeply connected in real engineering practice: structural engineering and geotechnical engineering.

Over three days, the event gathered **1,213 registered participants** from 62 countries and territories across **5 continents**, creating an international space for technical exchange, professional reflection, and knowledge sharing around some of the most relevant challenges in civil engineering today.

The conference was held online through Zoom Events and featured a bilingual program in English and Spanish, with the participation of 20 speakers from different countries and professional backgrounds.

A CONFERENCE DESIGNED TO CONNECT SOIL AND STRUCTURE

One of the main purposes of **SISMICA S&G 2026** was to create a technical meeting point where structural and geotechnical perspectives could be addressed within the same conference program.

In professional practice, structures and soil cannot be understood as isolated systems. The behaviour of a building, bridge, industrial facility, retaining system, tunnel, or foundation is directly influenced by the interaction between structural demands, ground conditions, seismic actions, foundation response, construction constraints, and long-term performance requirements.

For this reason, SISMICA S&G 2026 was organised around a 3-day program designed to reflect the way engineering problems are solved in real projects: from structural performance and seismic behaviour to foundation design, geotechnical response, industrial facilities, special structures, and complex infrastructure systems.

Day 1: Structural Engineering & Bridges

The first day of the conference focused on Structural Engineering & Bridges, addressing topics related to structural performance, seismic behaviour, infrastructure, and service life.

This day included keynote lectures and technical sessions on concrete infrastructure, gravity dams, seismic behaviour of buildings, structural reliability, tunnels, underground works, and buildings with base isolation systems.

Speakers discussed topics such as service life extension of concrete infrastructure, seismic analysis and instrumentation of concrete gravity dams, lessons learned from recent earthquakes, uncertainty propagation in structural engineering, seismic considerations for underground works, and the structural performance of base-isolated buildings.

Day 2: Foundations & Geotechnical Engineering

The second day was dedicated to Foundations & Geotechnical Engineering, with a strong focus on soil behaviour, foundation systems, seismic response, and geotechnical applications.

The sessions addressed advanced methods for determining pile length in infrastructure projects, the estimation of soil deformation parameters and their influence on foundation design, the undrained shear response of clays, helical piles as an alternative deep foundation system, uplift mechanisms in shallow foundations, and the state of the art in drilled shafts.

Day 3: Industrial Structures & Special Projects

The final day of SISMICA S&G 2026 focused on Industrial Structures & Special Projects, bringing the conference into the context of complex engineering systems and highly applied case studies.

The program included presentations on structural stability in steel buildings, seismic design and stability of industrial structures, structural assessment of reinforced concrete silos, seismic retrofitting of concrete tower systems, embodied carbon in bridge structures, and foundation design for industrial facilities such as steel tanks and silos.

The final day also reflected one of the central messages of the conference: real-world engineering requires coordination between disciplines, practical judgment, technical rigor, and a clear understanding of how different systems interact.



ACKNOWLEDGEMENTS

SISMICA Institute would like to express its sincere gratitude to all speakers who participated in this first edition of the conference. Their generosity, preparation, and technical quality made the event possible and gave meaning to the purpose behind SISMICA S&G 2026.

We also extend our appreciation to the technical moderators, participants, and the entire team behind the scenes, whose work, commitment, and attention to detail contributed to the successful development of the event.

THE BEGINNING OF AN ANNUAL CONFERENCE

SISMICA S&G 2026 marks the **beginning of a new annual conference organised by the SISMICA Institute.**

The objective is to continue creating a meeting point where structural and geotechnical engineering come together in real-world practice, allowing professionals from different countries to share technical knowledge, project experience, research findings, and applied methodologies.

SISMICA Institute will announce the dates and details of the next edition of SISMICA S&G soon.

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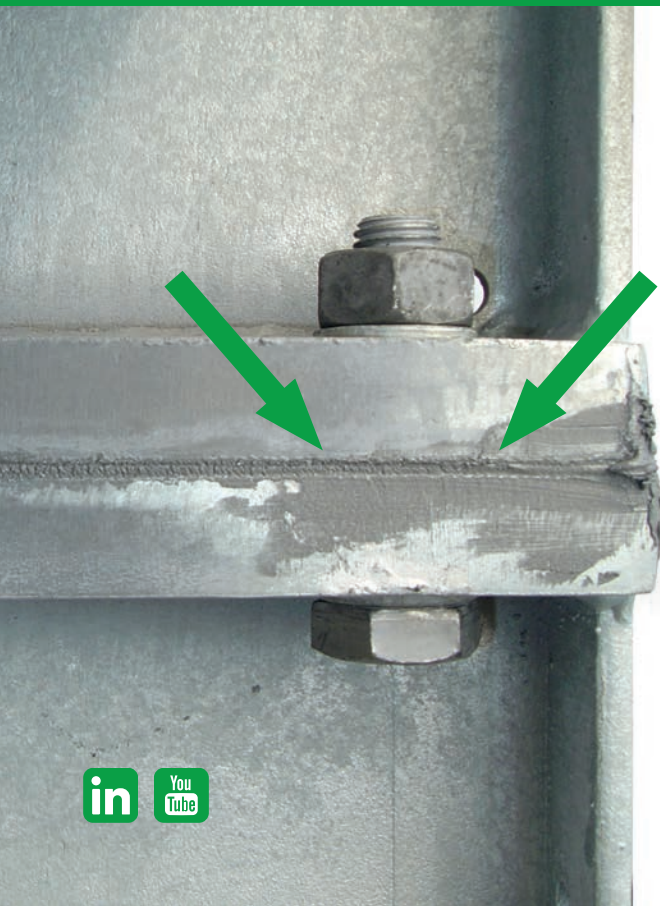


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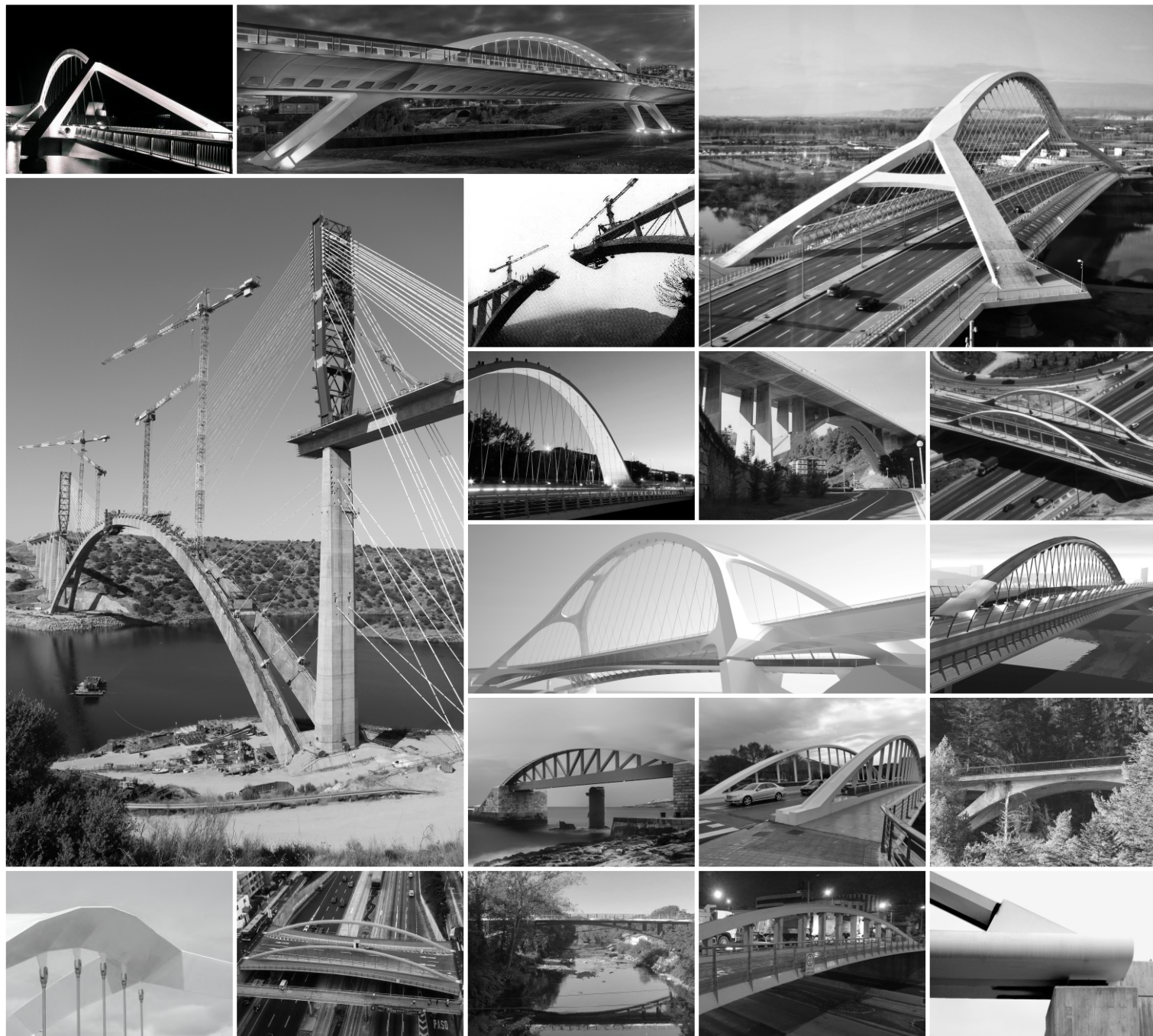
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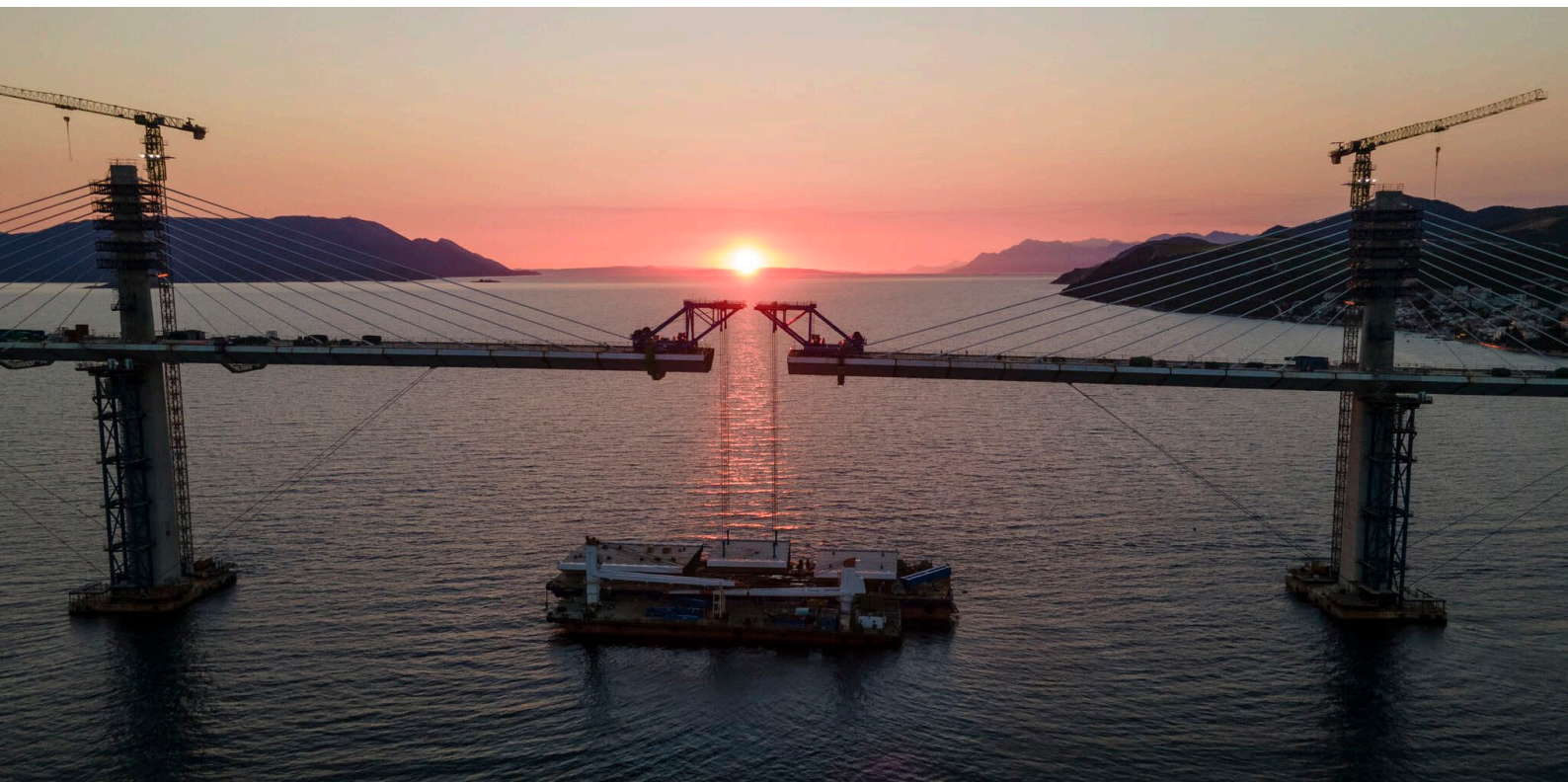
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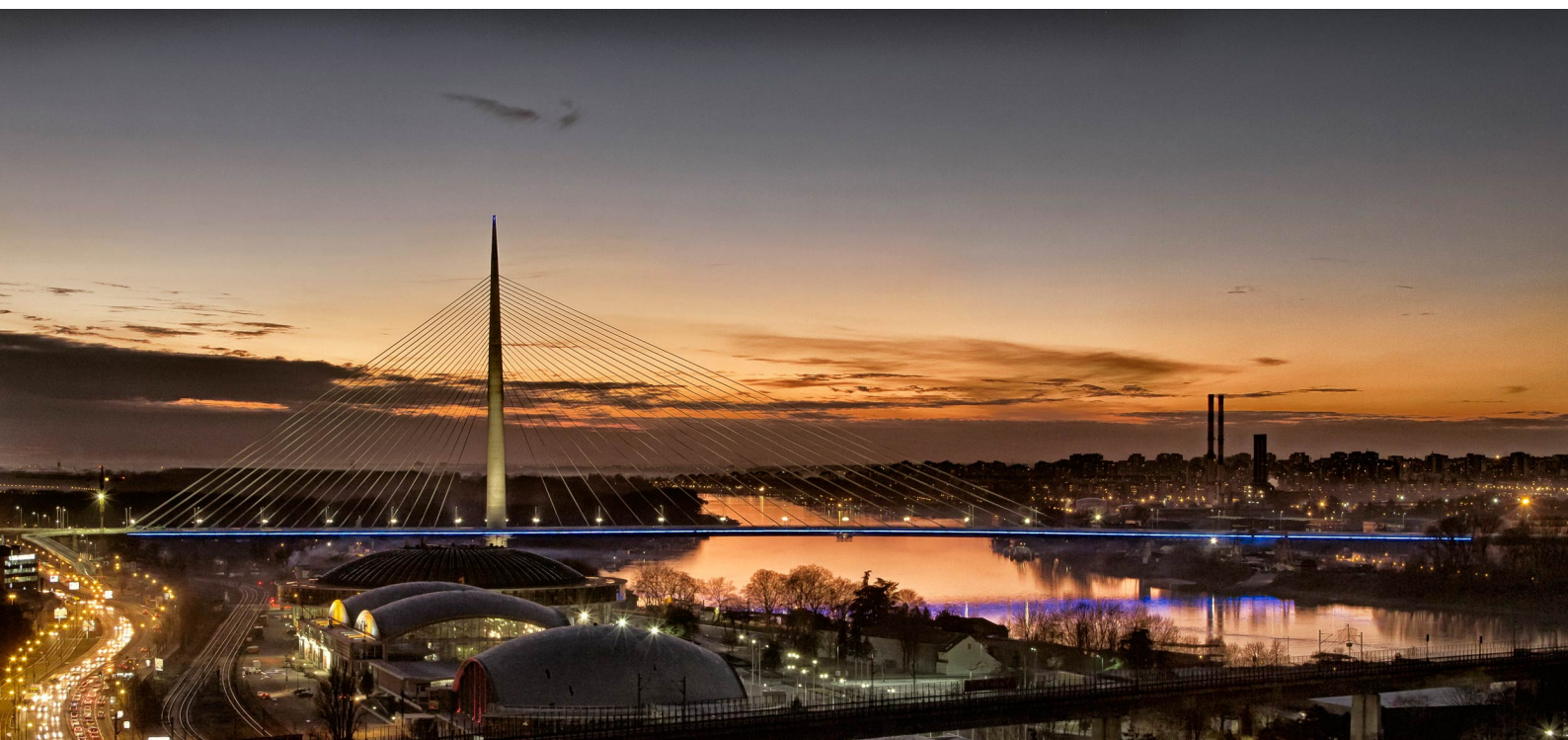
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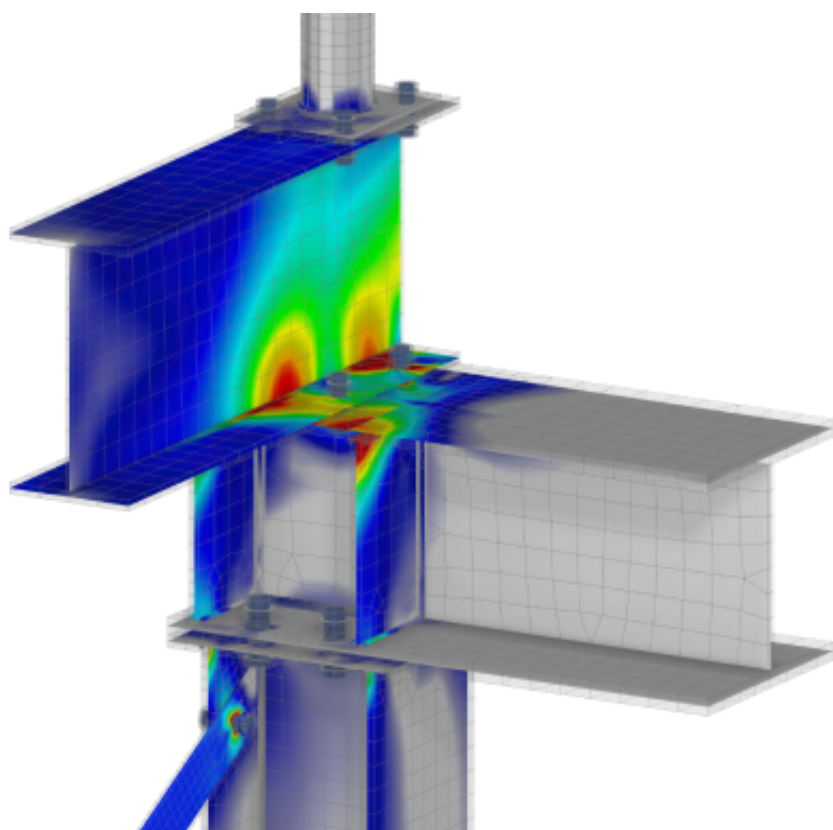
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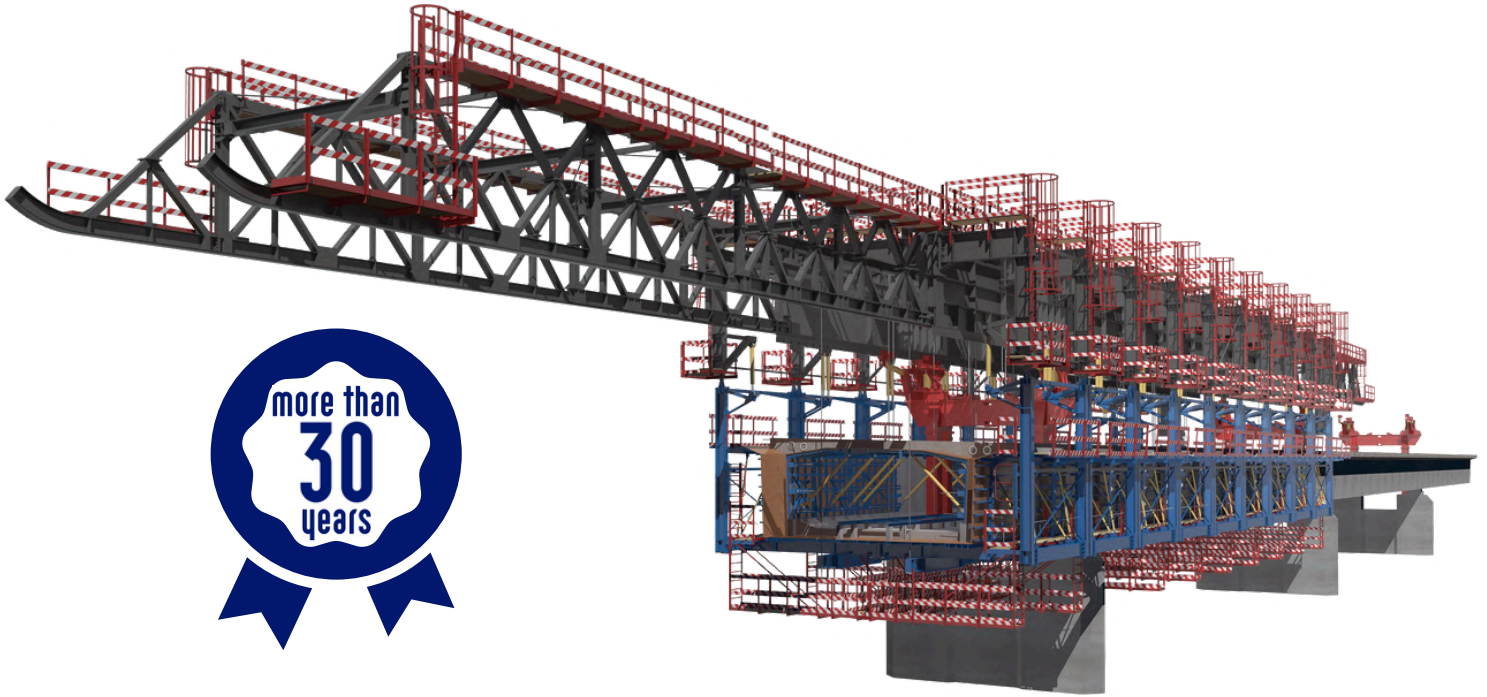
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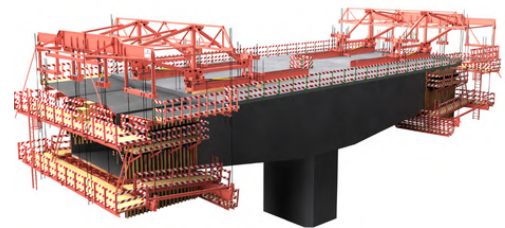
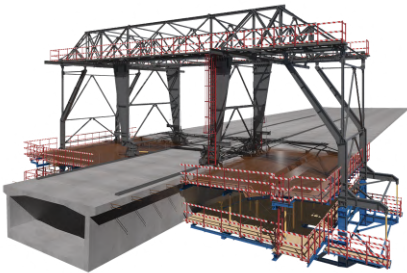


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Long Span Bridges
e: richard.hornby@arup.com

Ngai Yeung
East Asia
e: ngai.yeung@arup.com

Deepak Jayaram
UK, Middle East, India & Africa
e: deepak.jayaram@arup.com

Sabine Delrue
Bridge Assessment & Retrofit
e: sabine.delrue@arup.com

Marcos Sanchez
Europe and Global
e: marcos.sanchez@arup.com

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- Hochmoselübergang, Germany
- Osman Gazi Bridge, Izmit, Turkey
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Build Bridges Connect Communities Attract Top Talent

Join a team of volunteers to build a life-changing footbridge or water project alongside an isolated community as part of our **Free Agent Program**. Or bring a whole team from your company as part of our **Industry Program**. More interested in recruiting? **Become a mentor** for students in our **University Program** to pass on your skills while attracting top talent from over two dozen elite universities around the world. Or donate **in-kind** tools, services such as engineering or project management, or materials like wire rope.

We're breaking the barriers that perpetuate poverty by building essential infrastructure, and we need your help. Contact us today to learn how you or your company can get involved.

Think about it: you, us, a bridge.



Partnership
Opportunities
Binder

PARTNER WITH US TO BUILD BRIDGES AND WASH SYSTEMS THAT
CONNECT PEOPLE TO HEALTH, EDUCATION, AND OPPORTUNITY.

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**BRIDGING
THE GAP
AFRICA**
BUILDING BRIDGES &
TRANSFORMING LIVES



**THE NEED IS
GREAT.
THE NEED IS
NOW.**



BE A PART OF THIS. BE A BRIDGE.

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ISSUE 03/2026

SEPTEMBER

