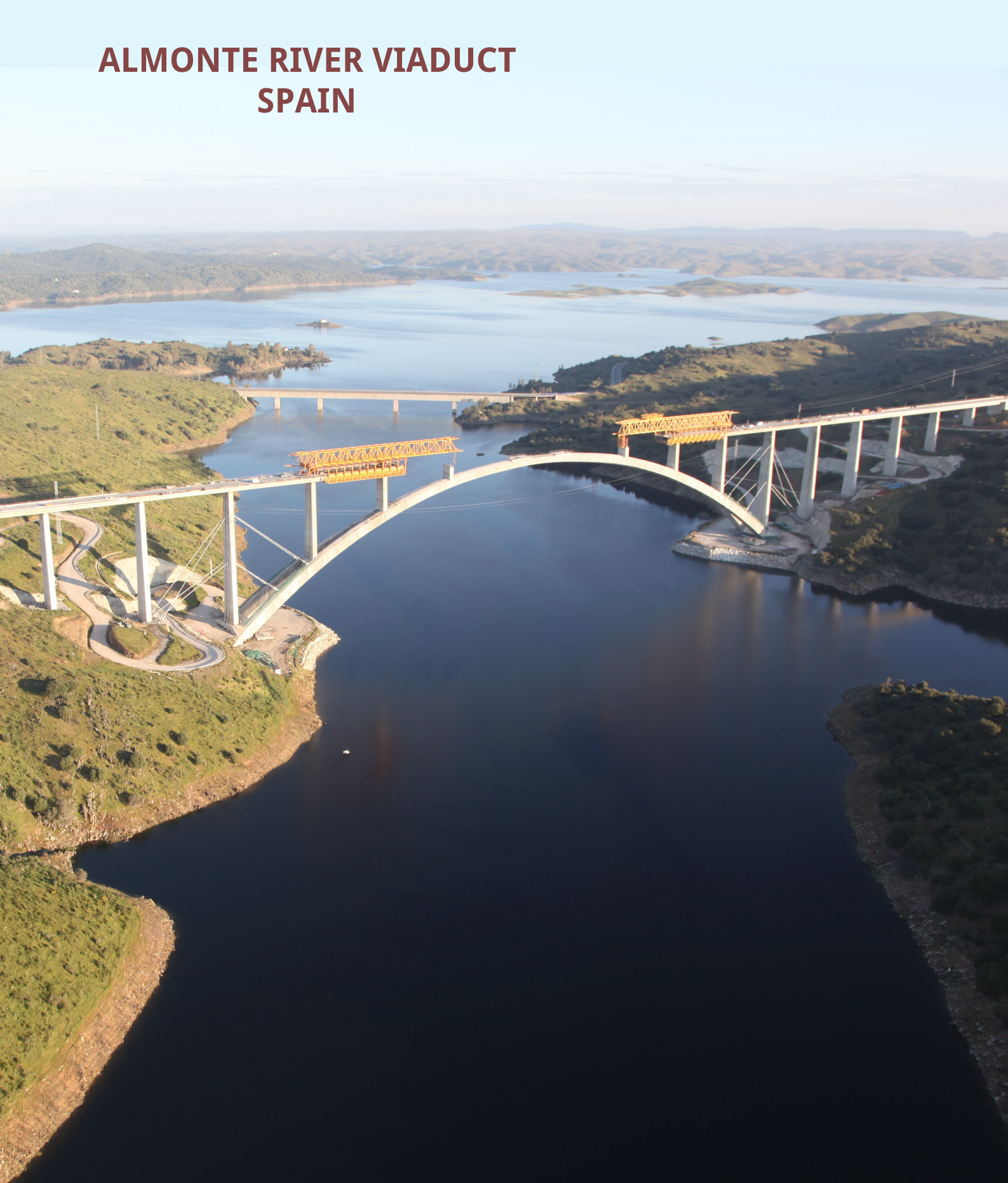
e-mosty

ISSUE 04 / DECEMBER 2016

ARCH BRIDGES

ALMONTE RIVER VIADUCT
SPAIN



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Front Cover: ***Almonte River Viaduct.*** *Photo Credit: David Arribas, FCC Construccion*

Back Cover: ***Tagus River Viaduct.*** *Photo: Carlos Manterola for Carlos Fernandez Casado*

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Prosíme respektujte autorská práva. V případě pochybností nás kontaktujte. Děkujeme.

Dear Readers

This issue of our magazine is focused on arch bridges. We present here three unique bridges:

Almonte River Viaduct (Spain) - the longest railway arch bridge in the world – and Tagus River Viaduct (Spain) – both bridges are part of Madrid – Portuguese border HSR link.

River Irwell Network Arch Bridge (UK) – the first network arch railway bridge in the United Kingdom.

We very much thank to all authors for their contributions, time and excellent cooperation. Let me say that I very much respect and admire you all.

We would also like to congratulate Mr Héctor Beade Pereda, Lead Designer of the Almonte Viaduct, who was awarded the IABSE Prize 2016, "in recognition of his contribution to the design of bridges improving the quality of the built environment, thanks to an enthusiastic and rigorous, aesthetically and structurally holistic approach to bridge design".

Our magazine has developed since last issue a bit. We have new members of our Editorial Board: Let me welcome Mr Richard Cooke (UK), Mr David Collings (UK) and Mr Peter Paulík (Slovak Republic).

Editorial Board

September issue had more than a thousand views in first days after it was released. It is open access and I hope we will be able to keep to this policy in the future. We have started search for partners who could help us and support us.

From December our magazine is solely in English. We bring original articles about bridges from the whole world, all our colleagues can speak English and we have thousands of readers worldwide. So instead of spending time and money on translations into Czech we will rather focus on higher quality and content of the whole magazine.

We were a medial patron of the 8th International Conference on Arch Bridges. We are very happy that we were given the opportunity to support the conference and promote our magazine as well, and we hope for further cooperation in the future.

Thank you all for your cooperation on this issue, especially to Mr Richard Cooke and Mr David Collings, to whom I very much thank for review of the articles and for all their assistance and support.

Magdaléna Sobotková

Chief Editor

Vážení čtenáři,

toto číslo našeho časopisu e-mosty je zaměřené na obloukové mosty. Přestavujeme vám tři jedinečné mosty:

Most přes řeku Almonte (Španělsko) – nejdelší železniční obloukový most světa – a most přes řeku Tagus (Španělsko) – oba mosty jsou částí vysokorychlostního železničního spojení z Madridu k portugalské hranici.

Most přes řeku Irwell (Velká Británie) – první železniční most se sítí závěsů ve Spojeném království.

Velmi děkujeme všem autorům za jejich příspěvky, čas a skvělou spolupráci. Dovolte mi vyjádřit můj respekt a obdiv všem, kteří se na návrhu a realizaci těchto mostů podíleli.

Rádi bychom také pogratulovali panu Héctorovi Beade Peredovi, hlavnímu projektantovi mostu přes řeku Almonte, k získanému ocenění „IABSE Prize 2016“ a to jako „uznání jeho přispění v navrhování mostů a zvýšení kvality stavebních projektů díky jeho nadšení a vytrvalému, estetickému a konstrukčně holistickému přístupu k navrhování mostů“.

Od posledního čísla se náš časopis trochu posunul. Máme nové členy redakční rady, které bych tímto velmi ráda přivítala: pan Richard Cooke (Velká Británie), pan David Collings (Velká Británie) a pan Peter Paulík (Slovenská republika).

Zářijové vydání našeho časopisu mělo více než tisíc zhlédnutí již v prvních dnech po svém vydání. Časopis je „open access“ a doufáme, že jej v této podobě budeme moci udržet i do budoucna. Právě začínáme hledat partnery, kteří by nám mohli pomoci a podpořit nás.

Od prosince je náš časopis pouze v angličtině. Přinášíme původní články o mostech z celého světa, všichni naši kolegové hovoří anglicky a máme tisíce čtenářů po celém světě. Takže místo toho, abychom se věnovali překladům, které jsou časově i finančně velmi náročné, budeme se raději zaměřovat na vyšší kvalitu textů a obsahu celého časopisu.

Byli jsme mediálním partnerem 8. Mezinárodní konference o obloukových mostech. Jsme velmi rádi, že jsme dostali možnost podpořit konferenci a s tím i zároveň náš časopis, a doufáme v další spolupráci v budoucnu.

Děkuji Vám všem za spolupráci na tomto čísle. Pan Richard Cooke a pan David Collings zaslouží speciální poděkování za revizi textů a za jejich podporu a pomoc.

Magdaléna Sobotková

šéfredaktor

e-mosty

PARTNERS AND SUPPORT WELCOME

I would like to offer you partnership with our magazine „e-mosty“ (e-bridges).

Information about the magazine can be found at **www.e-mosty.cz** and previous issues can be viewed in the archive section of the website.

Since May 2015 when the magazine was established we have achieved especially the following:

- Established the Editorial Board
- Released 7 issues (typically 4 per year)
- Achieved an international profile with increasing numbers of readers (over 1000 already) with positive reviews and feedback.

„e-mosty“ magazine is unique and covers a big part of the construction market.

The magazine is **open access** and we would like to keep to this policy.

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We offer you two types of partnership:

- Sponsorship of one issue (eg where your company participated in the construction of a bridge we are writing about), or
- General partnership of the magazine

In both cases we offer you advertisements, PR articles, your logo in every issue and on our web and other promotion. The price is negotiable, the magazine is low-cost and every support is welcome.

We are looking forward to possible future cooperation with you and your company.

www.e-mosty.cz

Madrid - Extremadura - Portuguese Border HSR Link

On following pages we are bringing you information about two of the most important structures on the HSR (High Speed Railway) Line that will eventually connect Madrid and Lisbon:

The viaducts crossing the Tagus (Tajo) and Almonte rivers, at the Alcántara reservoir

Tagus (Tajo) River Viaduct

The width of the obstacles to be bridged, the impossibility of using piles in the riverbeds and the requirement to respect the environmental impact statement, have led to both having a "concrete arch" design, with main spans of 324 and 384 metres, respectively.

The 384 metres without supports of the central span of the viaduct over the Almonte River make it the widest span arched concrete railway bridge in the world. The construction process chosen for erecting both bridges is similar to that used successfully for the Contreras Viaduct, on the Levante line.

Meeting the challenge of constructing this type of viaduct confirms the high level of Spanish infrastructure engineering.

Almonte River Viaduct

ALMONTE HSR VIADUCT

OVER THE ALCÁNTARA RESERVOIR IN SPAIN

Commencement of works:2011

Opening of the bridge to traffic: 2016

Type: concrete HSR arch bridge

Total Length:996m

Main span: 384m

Location: Alcantára Reservoir – Garrovillas, Cáceres, Spain

Client: ADIF AV (Spanish Rail Administrator), Madrid, Spain

Conceptual and detailed design:

Arenas & Asociados, Santender, Spain and IDOM Ingenieria, Madrid, Spain JV

Design engineers: Guillermo Capellán, Héctor Beade, Javier Martínez and Emilio Merino (Arenas y Asociados)

Main Contractor: FCC Construcción - Conduril Engineering JV

Location of the bridge (Source: Google Maps)

ALMONTE VIADUCT - DESIGN

Héctor Beade Pereda, Knight Architects
http://www.hectorbeade.com/

Guillermo Capellán, Arenas&Asociados, Santender, SPAIN

Fig. 1: Photomontage of the viaduct in the natural environment

SUMMARY

The Almonte River arch bridge over the Alcántara Reservoir, which is part of the Madrid-Portuguese Border High Speed Rail (HSR) link, is a challenge for bridge design, engineering and construction. Its 384m main span will make this major project the largest HSR arch in the world and the largest railway bridge in Spain.

In order to solve the specific design problems of a HSR crossing with large span and length, a structurally innovative solution was used: the arch, linked to the deck at the crown, has an octagonal section with variable depth and width in its central 210 m, from where it splits itself into two legs with a variable hexagonal section down to the springings.

The design combines together structural efficiency, out-of-plane stability (required by HSR horizontal deflection limits), improved response to wind loads (proven by exhaustive wind tunnel tests), transparency, aesthetics and durability.

1. INTRODUCTION AND CONTEXT

The characteristics of HSR traffic requires the alignment to comply with strict design parameters both in horizontal and vertical profile and leads to numerous bridges, commonly with significant length and sometimes also height. These bridges and viaducts, which are inevitably flexible elements of rail schemes, are subjected to heavier vertical and horizontal loads than road bridges. They have also to comply with strict deflection and vibration limits in order to guarantee passenger comfort and traffic security (assuring that railroad geometry and curvature, together with wheel-rail contact, are maintained). Additionally, HSR bridges are exposed to considerable dynamic effects, are prone to suffer from fatigue problems due to the intensity and repetitiveness of the loads, and have global-length limitations due to rail expansion joints capacities and track-structure interaction.

As a consequence of these characteristics, HSR bridge spans tend to be shorter than bridges carrying other types of traffic. Nevertheless, there are sometimes obstacles which inevitably require span lengths above the customary and can be considered exceptional for the combination of rail traffic, as in the case of the Almonte bridge described in this article.

The new HSR line at the Madrid- Portuguese Border (with mixed-use and a maximum speed of 330 km/h for passengers and 100 km/h for freight) crosses the Almonte River close to its mouth, where it flows into the Alcántara Reservoir.

The importance of the location from the environmental point of view makes it impossible to arrange supporting elements inside the reservoir infringing on the maximum water level limits, leading to a 350-m distance to be cleared between banks. This made necessary a bridge with a main span of 384 m, the largest on the Spanish HSR network and the third longest concrete arch in the world.

The magnitude of such a main span implies additional requirements to those inherent to any HSR previously built. As an example, aeroelastic phenomena (increasing oscillations caused by the interaction between wind and structure) can be significant for these spans and must be considered in the design. The total length of the entire viaduct is 996 m.

All these particularities make the design and construction of this major bridge a challenge.

The conceptual and detailed design of the viaduct was carried out by Arenas & Asociados, within a Joint Venture with IDOM for the design of a complete infrastructure subsection (Alcántara Reservoir-Garrovillas) of the Madrid- Portuguese Border HSR line. Adif (the Spanish Administrator of Railway Infrastructure) is the owner of the infrastructure and the client. FCC in Joint Venture with Conduril is responsible for the construction of the bridge.

Fig. 2: Site image and bridge alignment

2. BRIDGE CONCEPTION AND DESIGN

2.1 Constraints and typological study

The chosen bridge type is the result of, first, a series of imposed constraints such as the peculiarities of the HSR traffic and the main span (as described in the previous section) and, second, the thorough consideration of the design problem to be solved and how to find the best solution taking into account criteria as functionality, structural behaviour, cost, durability and maintenance, construction method and environmental integration in a spectacular environment.

During the initial stages of the design process, a detailed typological study was carried out, in which several potentially-appropriate bridge-type alternatives (Fig. 3) and different erection procedures (Fig. 4) were analysed. The following types were studied: four alternatives of rigid frame bridges with V-shaped piers (made of steel or concrete), an open-spandrel deck-arch bridge and two alternatives of cable-stayed bridges (single-plane or double-plane).

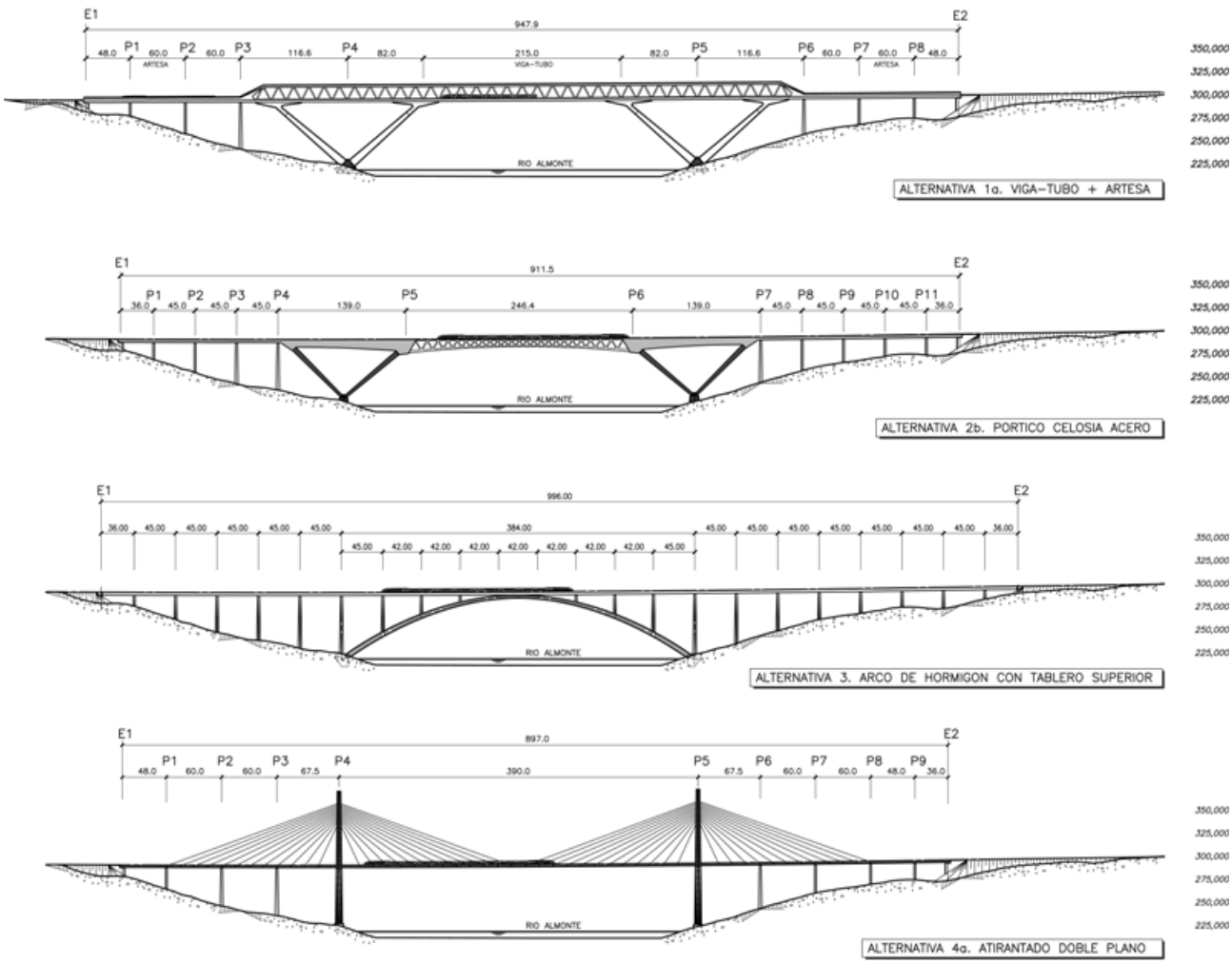


Fig. 3: Some alternatives considered during the typological studies

In the case of the deck-arch bridge option, and starting with the premise of not setting temporary piers inside the reservoir during construction either, three erection alternatives were studied (Fig. 4): pylon-method cantilever erection of the arch using temporary cable stays and provisional pylons over the piers adjacent to the arch (4a), truss-method cantilever erection using the arch as the lower chord of two great truss cantilevers where the deck of the bridge is the top chord (4b), and a variation of the latter where the top chord of the truss is formed by temporary ties that would be removed after the completion of the arch (4c). Another sub-alternative, compatible with all these three erection possibilities, was studied: the lifting of an auxiliary 120-m-to-180-m-long truss for the construction of the central part of the arch (4d).

Among the different erection solutions that were studied, all of them feasible and successfully used for the construction of other long-span arch bridges, the cantilever method with the help of temporary towers (and without flotation) was chosen as the most appropriate in this case. Although this alternative needs more provisional members than others, it allows a more precise geometrical control during erection and, in addition, introduces a pre-compression force in the arch that helps to partially compensate its elastic deformation under permanent loads. This makes unnecessary a final jacking operation in the crown of the arch once the two cantilevers meet, that would be essential with other erection procedures.

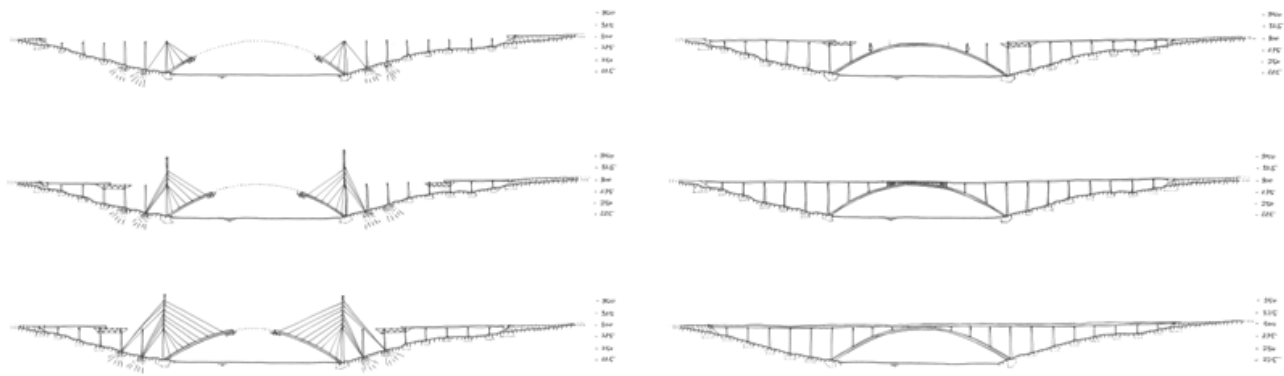


Fig. 4a: Construction alt. 1. Pylon-method cantilever erection of the arch using provisional pylons

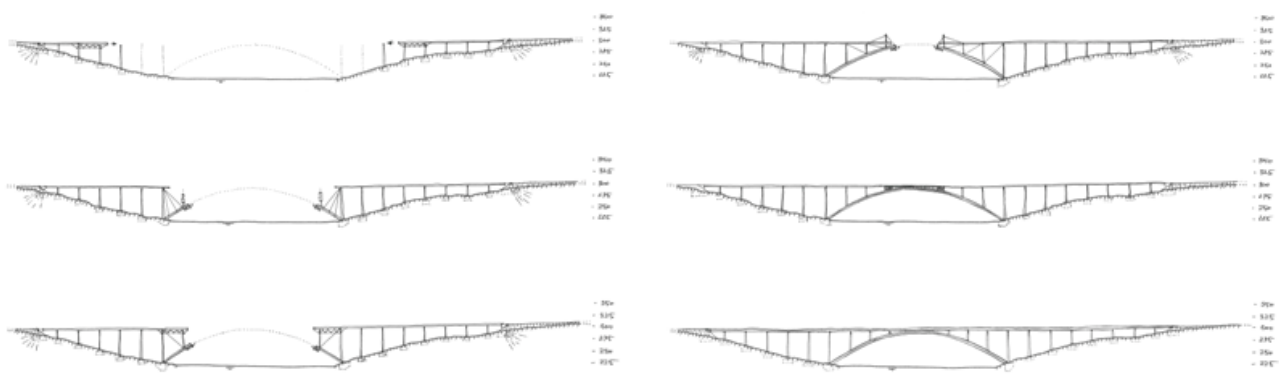


Fig. 4b: Construction alt. 2. Truss-method cantilever erection using the deck as the top chord

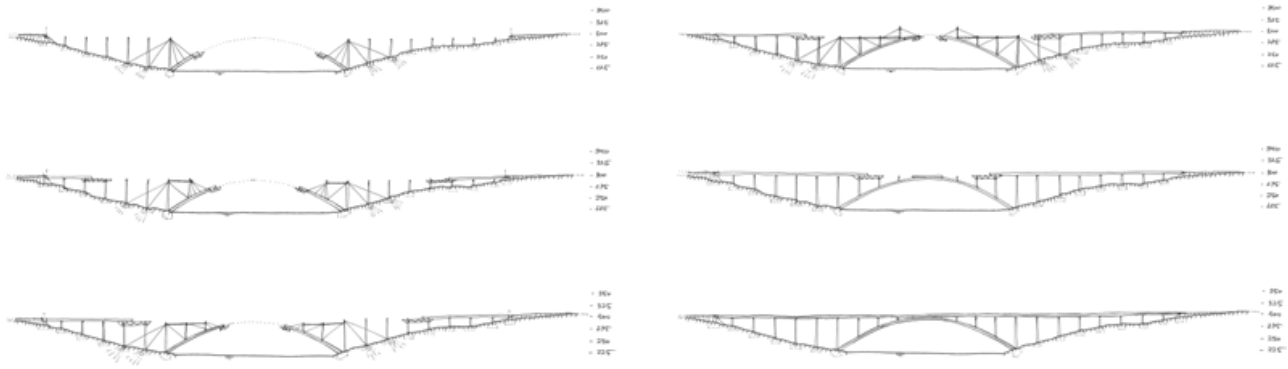


Fig. 4c: Construction alt. 2. Truss-method cantilever erection using temporary ties as the top chord

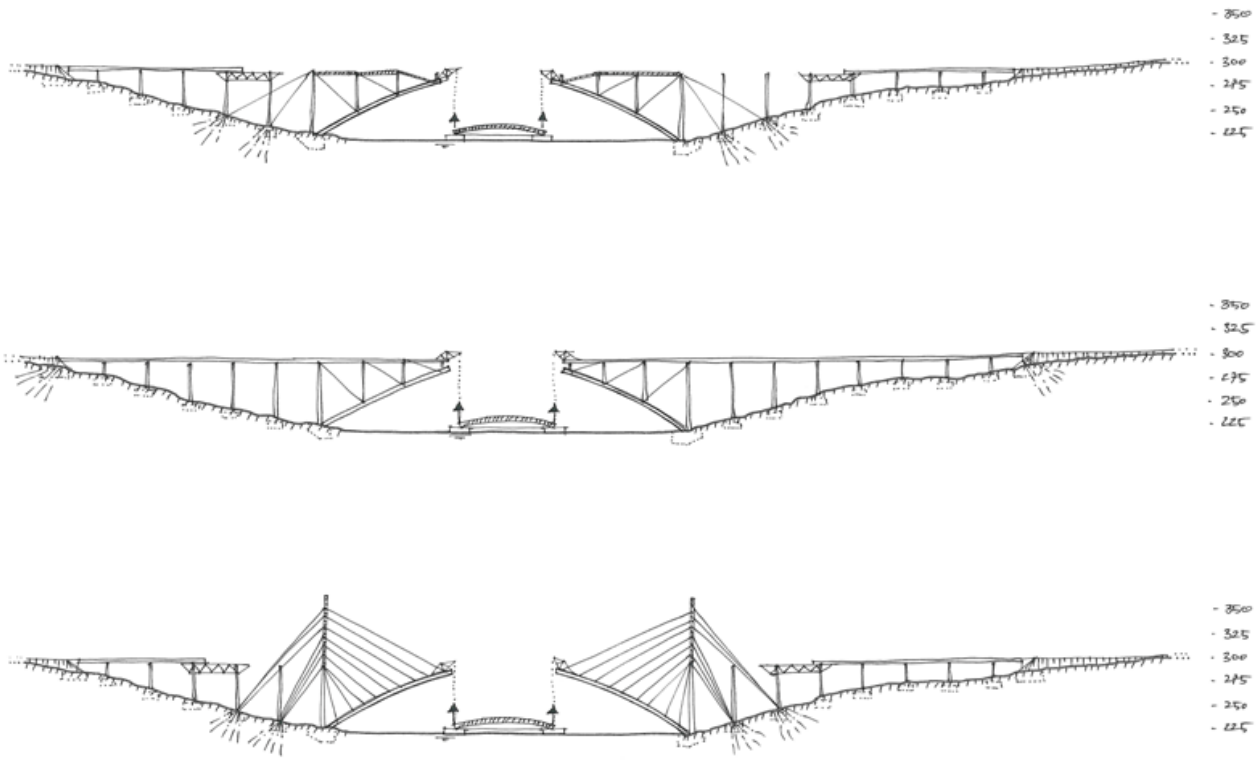


Fig. 4d: Variation of the former alternatives floating+lifting an auxiliary truss for the construction of the central part of the arch

The conclusion of the deep multi-criteria analysis carried out after the bridge-type and construction-method studies was that the most appropriate solution for this crossing was a viaduct with a global length of almost 1000 m and a main span consisting of a large 384-m span open-spandrel deck-arch, erected using the cantilever method with the help of temporary towers.

2.2 Magnitude of the challenge

The viaduct resulting from this typological study will become, in terms of main span:

- The largest railway bridge (and obviously HSR bridge) in Spain.
- The 3rd largest bridge in Spain for all traffic types.
- The largest HSR arch bridge, surpassing the Dashegguan Bridge in China (336 m).
- The largest railway arch bridge made of concrete (including non-HSR bridges), surpassing by more than 100 m the bridge over the Froschgrund Lake, Nürnberg-Erfurt line, Germany (270 m).
- The 3rd largest arch bridge made of concrete without traffic-type distinction, only after the Wanxian Bridge in China (420 m) and close to the greater of the two bridges between the Sveti Marko and Krk islands in Croatia (390 m).

These facts give an impression of the magnitude of the challenge to conceive, design and construct such a major bridge project.

2.3 An appropriate design for the problem to be solved

2.3.1. Suitable bridge type and material for the crossing problem

The open-spandrel deck-arch bridge as it was proposed, with 45m spans between piers and 42m spans between spandrel columns, is the most economical, in terms of construction cost, of those which have been analysed. Two of the reasons leading to this fact are the orographic features and geotechnical properties of the site. The existence of sound rock (slate) close to the soil surface and the shape of the valley make the deck-arch bridge design highly competitive. In addition, the deck was proposed as a continuous multi-supported prestressed box-girder all along the viaduct and, by virtue of the pier and spandrel-column arrangement, it made it possible to be erected using a conventional overhead cast-in-situ movable scaffolding system (Figs. 5 and 11). This design is also the most appropriate from the durability point of view, so it will be also the most suitable of those studied when considering future maintenance costs and it is also the most sustainable.

Another favourable aspect of this alternative is its behaviour under dynamic effects. A better response to vibration phenomena than other solutions is achieved due to the mass and damping provided by the use of concrete for the whole bridge construction.

This bridge type is also more environmentally friendly than most of the analysed options, not only due to its integration in the landscape (which is considered highly successful as well by the authors) but also to the fact that the arrangement of its structural members, with a low number of them and quite massive, minimises accidental bird impact and death. This aspect is very important in a valley which is a corridor for migratory birds.

2.3.2 Specific features of the design which improve the basic bridge-type scheme

In addition to the previously-mentioned features, which make the open-spandrel deck-arch bridge made of concrete the best solution for the crossing problem, the designed bridge incorporates a series of specific features which significantly improve the basic performance characteristics. These features directly respond to the main span dimensions, location of the bridge and HSR traffic to be supported.

Joining together deck and arch at its crown is the first of these features. This joint forms the fixed point of the structure, taking advantage of the excellent characteristics of the arch as a horizontal-load transmitter, in addition to its main role as the vertical-load supporting element. Consequently conventional expansion joints at both ends of an almost 1km long viaduct have been used (Figs. 5 and 6).

The transformation of the single arch into two inclined legs is the second of these features. It improves the behaviour of the bridge under transversal actions (both static and dynamic) and its response to out-of-plane instability phenomena. These improvements are essential for a structure with both a reduced deck width (due to its HSR nature) and an exceptionally large main span. Splitting the arch transversely (Figs. 5 and 6) and the variation of moment of inertia relative to the horizontal axis of its cross section, being a maximum at the springings and a minimum at the span centre, increases arch stiffness while maintaining a similar mass per unit length all along the arch. This leads to a better dynamic-effect response, to both vertical and horizontal effects caused either by wind or by trains, if compared with solutions with constant (or less variable) depth and width.

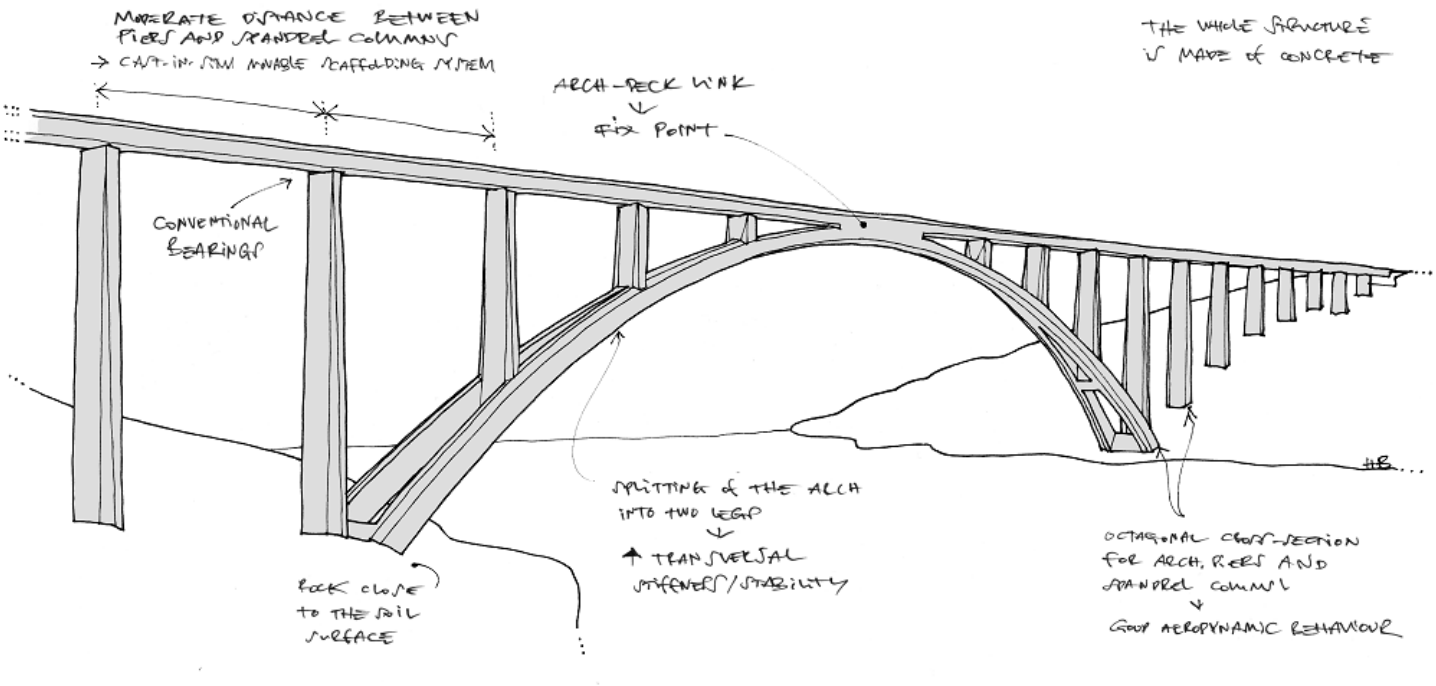


Fig. 5: Some features which make the design appropriate for the crossing problem to be solved

Once a design with a good response to dynamic effects, due to its mass, stiffness and damping, was achieved, the reduction of one of the loads which causes these effects was the objective. With the use of an octagonal cross-section for arch, piers and spandrel columns, a good aerodynamic behaviour was guaranteed, which is fundamental in a long-span bridge as this one is, and it was proved in the exhaustive wind tunnel tests carried out.

The design of this viaduct includes considerable improvements to the basic open-spandrel deck-arch bridge type, with the aim of giving response to specific problems of a HSR-crossing with large main span and overall length. These specific problems are the following: need of transmission of large horizontal reactions to the soil combined with a limitation of deck longitudinal displacements, the need to have suitable behaviour under dynamic effects caused by wind and trains and (partially related to the latter) the need to ensure transverse stiffness sufficiently high for a HSR narrow deck.

The design arrangement of span lengths, both between piers (directly founded on soil) and spandrel columns (using the arch as support), intends, first, to set out enough supports over the arch in order to guarantee its good anti-funicular behaviour with a curved (not polygonal) geometry and, second, to allow the use of a deck with the same cross-section along the whole viaduct, making the construction and the subsequent maintenance of the viaduct easier. The aim of the design is to achieve, within the complex problem that needs to be solved and considering maintenance, the sustainable solution that is most similar to a conventional continuous multi-span concrete bridge with a box-girder deck (in terms of materials, technology, used sections, type of bearings). In fact, if the longitudinal view of the bridge is observed, it is not difficult to compare what is seen to a multi-span box-girder-deck bridge where the terrain was substituted by an arch in the area where the reservoir is crossed. At the same time, the solution aims to create a balanced, harmonious and well-organised image (Figs. 1, 6 and 8a).

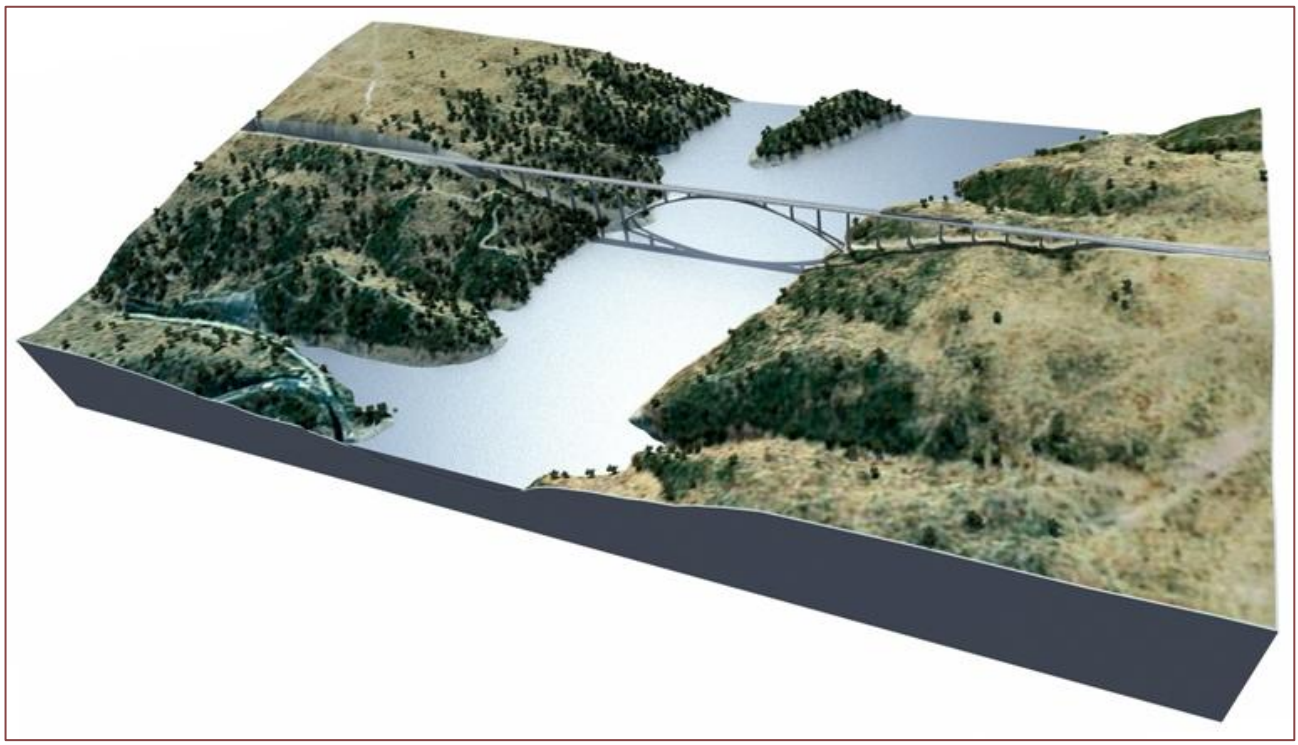


Fig. 6: General (virtual) view of the viaduct

Fig. 7: Virtual view of the viaduct

3. THE DESIGNED BRIDGE

The viaduct is formed by three distinct areas: two sequences of approaching spans from both banks, and the main span. The span arrangement of the former is 36m + 6 x 45m (towards Madrid), while that of the latter is 7 x 45m + 36 m (towards Portugal). The great 384m deck-arch main span gives support, by means of spandrel columns (45m + 6 x 42m + 45m), to the upper deck, which has a continuous multi-supported prestressed box-girder scheme all along the viaduct.

The deck is a double-cantilevered prestressed-concrete box girder with a constant depth of 3.10 m and a width of 14 m (allowing the arrangement of a conventional double-line HSR platform). The inherent flexibility of the arch makes the deck require, when over the reservoir, greater ratios of prestressing and reinforcing steel and a higher grade concrete (C60/75 as against C40/50). This is despite the span reduction between the spandrel columns (42m as against 45m).

The arch is made of self-compacting and high-strength high-performance concrete (C80/95). It has an octagonal hollow section with variable depth and width in central 210m section, from where it splits into two legs with an irregular hexagonal section down to the springings. Both legs are linked together under the first (7 and 14) spandrel columns (Figs. 5, 7 and 8b,c,d). The arch has a depth of 6.90m at the springings, the horizontal distance between side faces being 19.00m, and has a depth of 4.80m and a width of 6.00m at its crown, this distance being coincident with the lower width of the deck, to which it is linked, making up a single concrete cross-section, in the central 30m (Fig. 8e).

Both piers and spandrel columns have a variable octagonal hollow shape with a top solid area. Their heights range from 12m (pier 22) to 65.30m (pier 15). Abutments are U-type with wing walls.

Every element of the bridge in contact with the terrain has a shallow foundation on rock with spread footings except for the arch and its adjacent piers which are founded with massive concrete elements (Figs. 5, 6 and 8a,b,c,d). The latter have an irregular polyhedral shape with a cascade configuration at their bottom areas due to the need to conform, in the most optimum way, to the direction of the resultant arch-pier forces and to the variation in the depth of the sound rock stratum. The cable-stayed system needed for the erection of the arch requires temporary rock anchors under the two spread footings of the piers that follow those which share their foundations with the arch.

Every deck bearing is pot type. Pairs of 83,000kN capacity fixed pot bearings are exceptionally and temporarily required under the auxiliary steel towers of the erection cable-stayed system. They will be replaced by conventional bearings once the arch is closed and the towers removed.

In order to avoid bird-train collisions as transparently as possible, conventional 3-m-high opaque protection barriers have been substituted with a design specifically developed for this project, formed by independent tubular profiles with the same height.

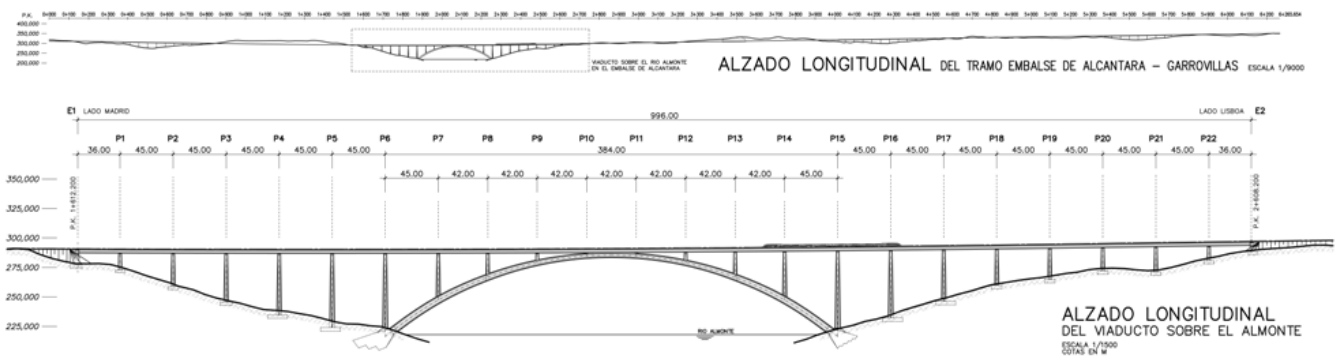


Fig. 8a: General elevation views of the bridge

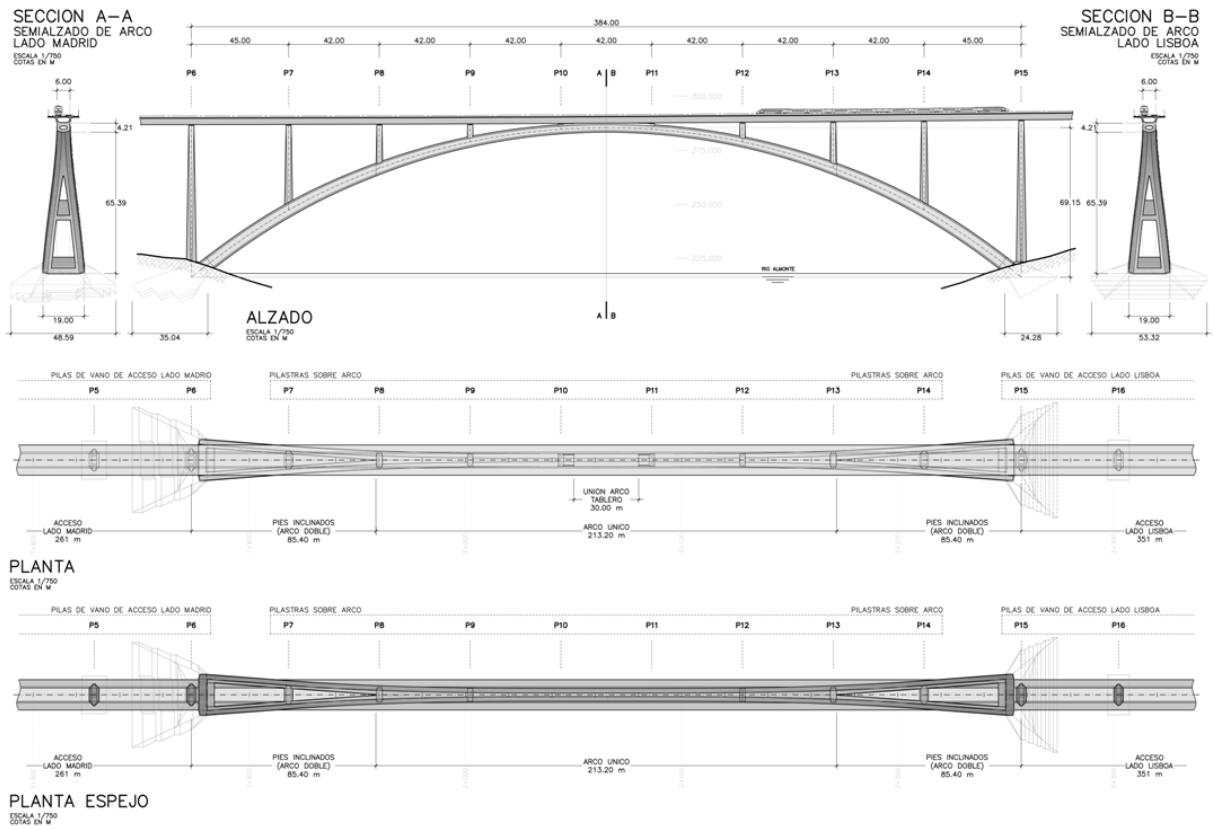


Fig. 8b: Front and side elevation views and plan views of the main span

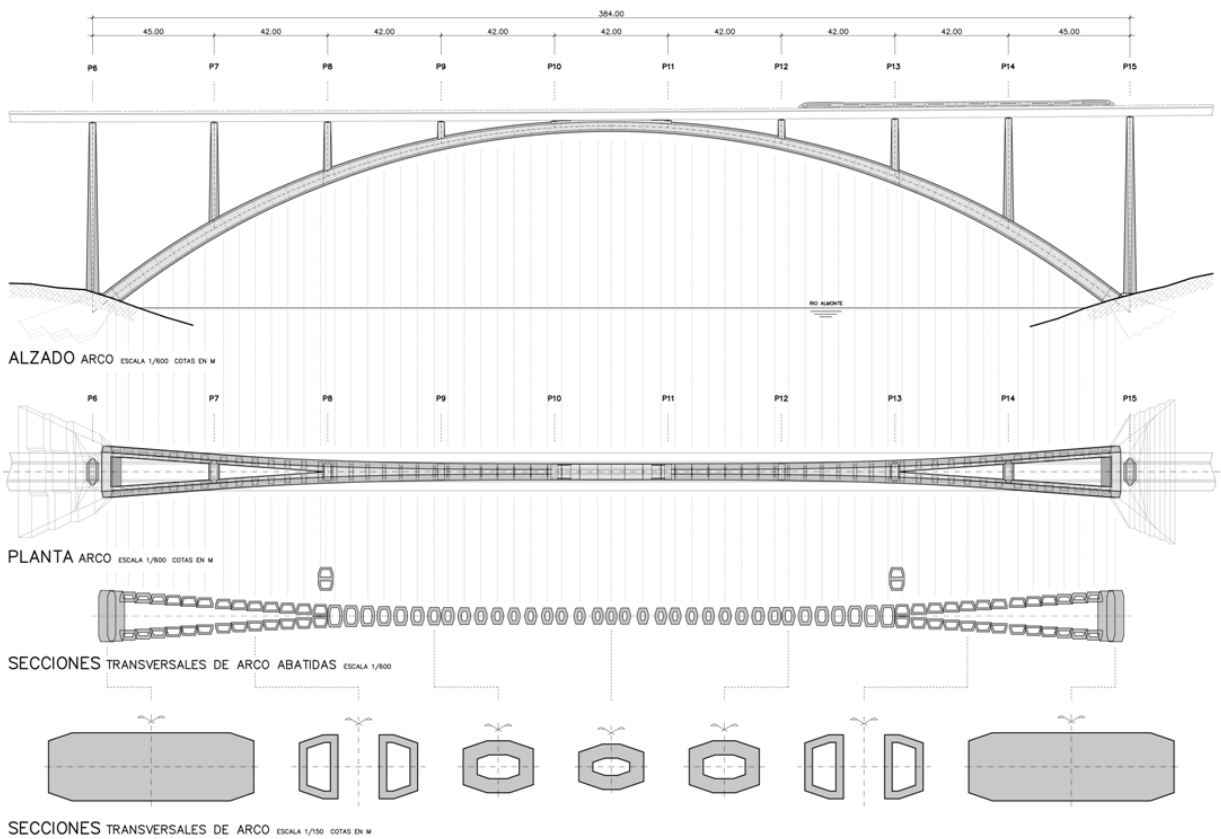


Fig. 8c: Geometric definition of the arch 01. Front elevation, plan view and cross sections

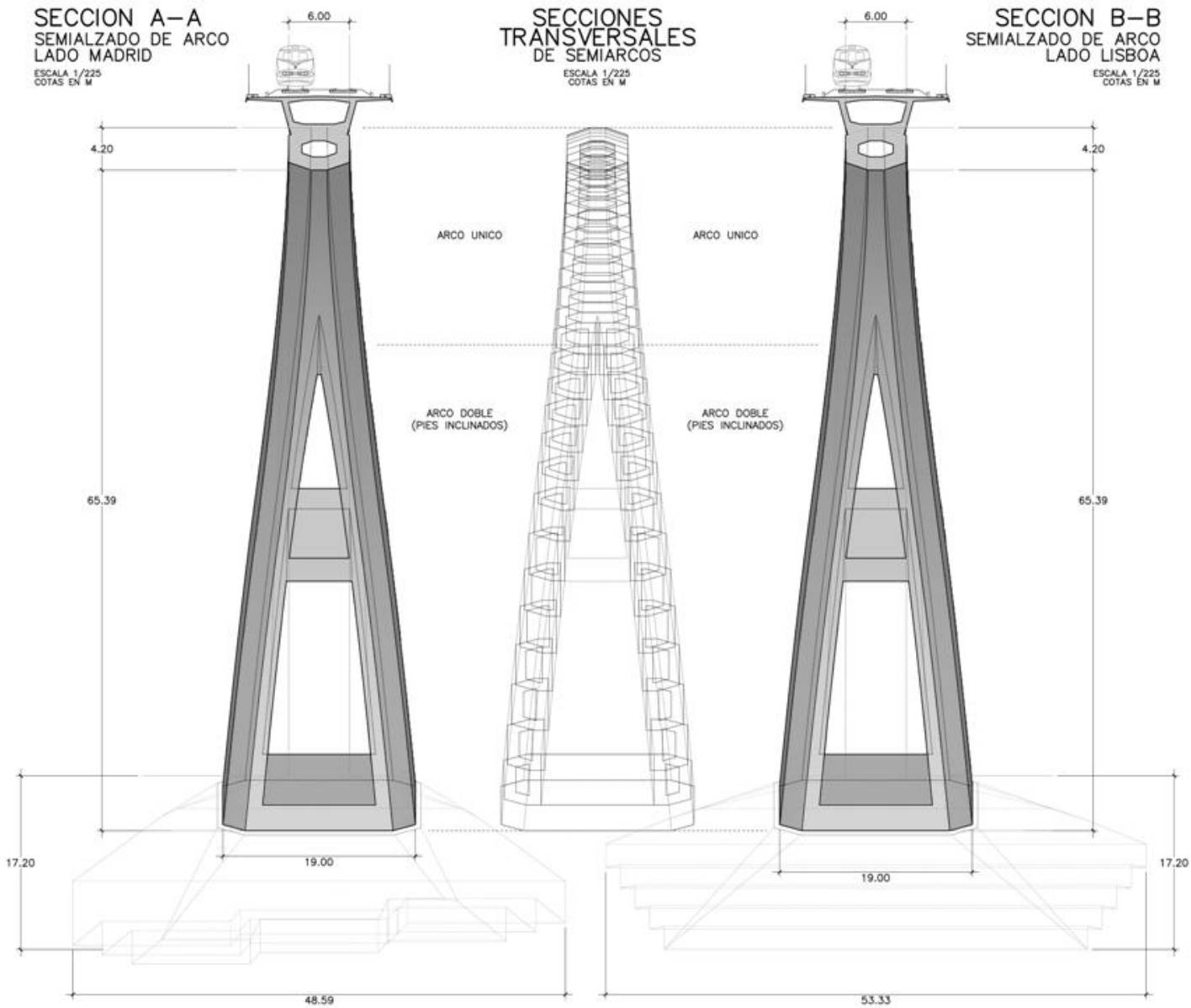


Fig. 8d: Geometric definition of the arch 02. Side elevations and cross sections

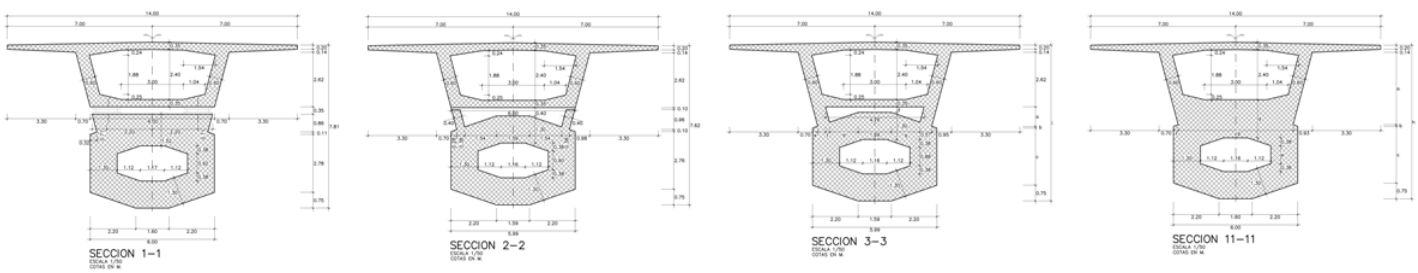


Fig. 8e: Cross sections showing the arch-deck connection

4. ANALYSIS OF THE STRUCTURE

4.1 Computer analysis

The static and dynamic structural analyses carried out for this project would require a specific article of its own, due to its extent and complexity. As a brief summary, it can be said that an advanced incremental structural model, including every stage of the construction, was developed for the analytical calculation of the bridge (Fig. 9). In addition to the influence of erection in the final situation, analysis has also considered geometrical imperfections for every stage and the effects of creep and shrinkage of concrete, as well as material non-linearity both for concrete and steel (using the true tensile stress-strain curves and taking cracking into account).

The final shape of the arch was determined through a step-iterative procedure, simultaneous to the geometrical adjustments of the different cross-sections, in order to achieve an essentially anti-funicular configuration of the HSR loads (which require a considerably different approach compared to the road bridges), with continuous curvature where the spandrel columns are located.

The arch stability was thoroughly verified for all the critical loading combinations (with a different model for each one of them), making a geometry-and-material nonlinear analysis, and using step-iterative techniques. An initial pre-deformation with the shape of the first vibration mode (which is an in-plane one) was considered. Additionally, a sensitivity analysis with different pre-deformation values was carried out and stability under other deformed geometries was also checked.

Ten different HSR load models, with speeds between 20 to 420 km/h, have been used to evaluate the dynamic behaviour of the finished bridge.

In order to guarantee durability and to avoid dynamic amplifications due to stiffness losses, the criteria of not allowing cracking in the arch in SLS, both during construction and under work-life loads, was established.

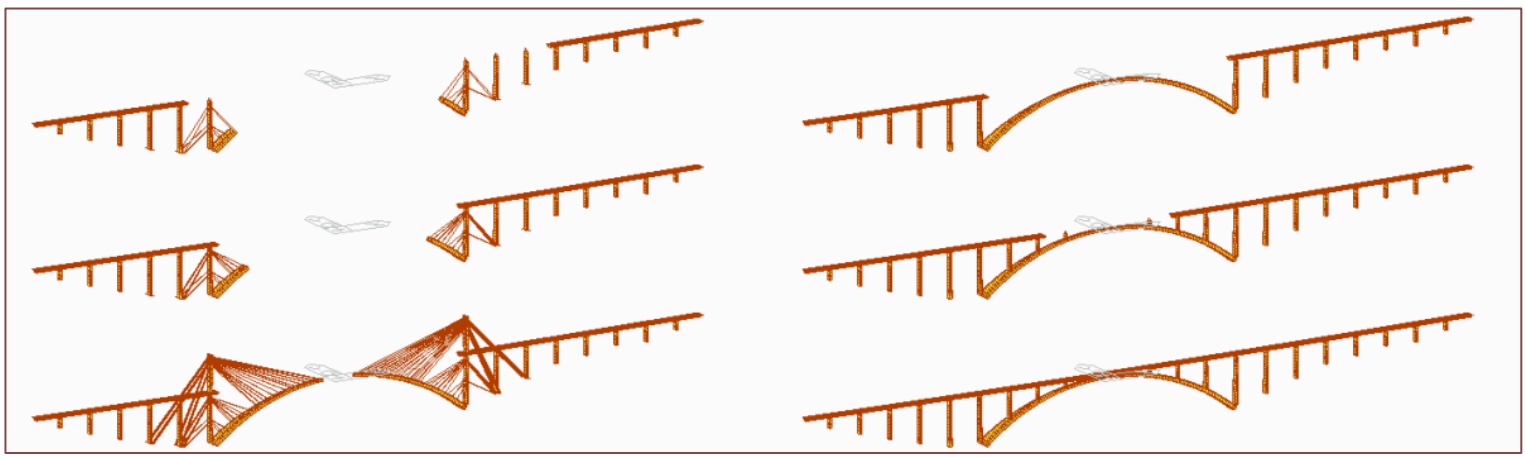


Fig. 9: Structural analysis model for incremental geometry-and-material nonlinear analysis taking into account construction phases

Fig. 10: Wind tunnel test analysis both in construction and final stages

4.2 Wind tunnel analysis

The main span of the viaduct goes significantly beyond the 200m set by the Spanish code IAPF-2007 as a limit after which aeroelastic effects must been taken into account. It was noticed, during early stages of the design, that the high flexibility of the bridge, consequence of its main span, led to reduced values of the first vibration frequencies (under 0,30 Hz) which is indicative of its sensitiveness to these effects. This fact, together with the three-dimensional configuration of the structure and the complex orography of the surrounding terrain, made it essential to carry out, during the design stage, tests using full-bridge aeroelastic models of the maximum-cantilever construction stage and of the finished bridge (in addition to section models). These models were tested by Oritia & Boreas in the Boundary Layer Wind Tunnel II of the University of Western Ontario.

Wind tunnel analysis has permitted to determine the specific wind static loads to be used in this bridge, to validate the appropriate behaviour of the proposed cross-sections (revealing the importance of the aerodynamic shapes used in arch and spandrel columns) and to confirm the good overall response of the design.

5. CONSTRUCTION

The construction of the bridge, which started in August 2011, was carried out with exactly the same erection sequence as the one considered during the project stage. The arch is built by pylon-method cantilever construction with temporary cable-stays (using steel towers and underslung form travellers specially designed for this bridge). The first cable-stays use the adjacent piers as a pylon (Fig. 11). During the erection of the arch, the prestressing forces of the cable-stays must be controlled and adjusted, un-tensioning several of them in some stages. The deck is erected using overhead cast-in-situ movable scaffolding system.

A detailed analysis of all the erection phases of the arch with the real bridge-building equipment was performed together by the designers and the technical department of the contractor during the construction stage. A complete monitoring programme was developed to control every step of the process.

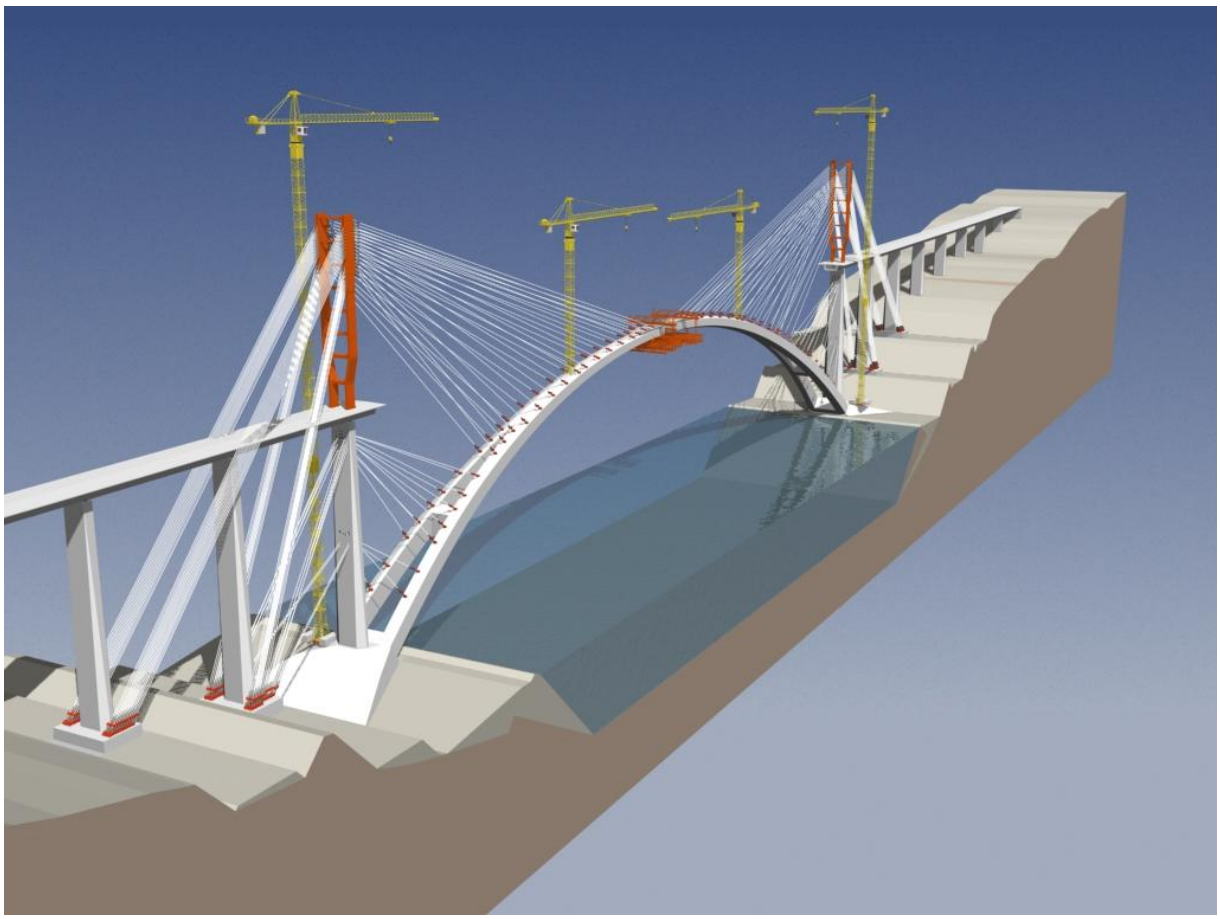


Fig. 11a: Virtual image of the construction of the viaduct as conceived during the design stage

6. CONCLUSIONS AND ACKNOWLEDGMENTS

The Almonte River Viaduct was a challenge for bridge design, engineering and construction. This major project is destined to be a major achievement due to many reasons, such as:

- Its function as a landmark of the HSR link from Madrid to the Portuguese Border.
- Its dimensions, becoming the largest HSR arch bridge, the largest railway arch bridge made of concrete and the third largest arch bridge made of concrete.
- The quality of its innovative structural and aesthetical design. The use of a scheme in which a single octagonal arch forks itself into two hexagonal legs, joins structural efficiency, out-of-plane stability (as HSR deflection limits require), improved wind response (as verified in boundary layer wind tunnel tests) and aesthetics.
- The use of high-strength self-compacting concrete (C80/95) for the construction of the arch.
- The complex erection procedures of its construction. The arch is being erected using pylon-method cantilever system, using temporary cable stays and provisional towers and six tower cranes, four of them over the arch.
- Its sustainability and virtues in terms of maintenance. Within the complex crossing problem, the solution is the most similar that can be achieved to a conventional continuous multi-span concrete viaduct with a box-girder deck (in terms of materials, technology, used sections, type of bearings).

From its apparent simplicity, the design gives simultaneous solution to the multiple functional, structural and environmental requirements of the complex crossing problem. Apparently simple solutions to a complex problem are usually the most difficult to achieve, not only in bridge design but also in many other fields.

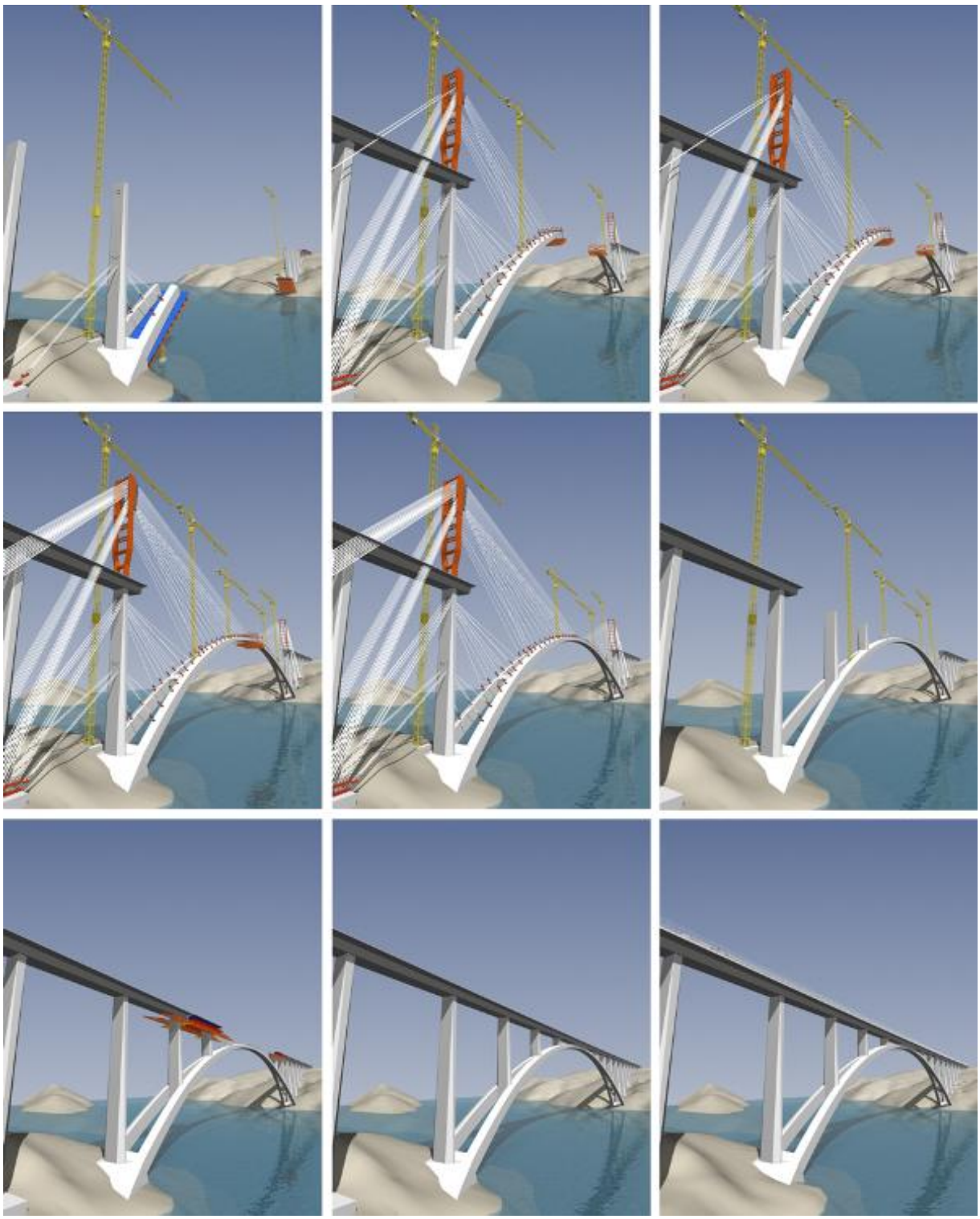


Fig. 11b: Several stages of the construction of the viaduct as conceived during the design stage

ALMONTE VIADUCT CONSTRUCTION PROCESS

David Arribas, Pedro Cavero, David Carnero
FCC Construcción

1. INTRODUCTION

1.1. LOCATION

Within the High Speed corridor that it is being constructed between Madrid and Extremadura, and will eventually connect the European capitals of Madrid and Lisbon, a most outstanding viaduct is being constructed crossing the Almonte river mouth into the Alcántara reservoir. It is located in the southwestern part of Spain, in an area where the river bed is almost 400 m wide.

The Almonte viaduct, whose length is almost 1 km, is within a 6.1 km section of the high speed line Madrid – Extremadura, being all the centre-piece of this unique contract. The whole contract comprises 3 more viaducts (of lengths between 250 and 450 m with spans of 45 m), 2 overbridges and one underpass, besides all the earthworks and drainage works necessary to create the rail platform.

It has to be stated that the Almonte viaduct is nestled in an area of a high environmental value, being qualified as ZEPA (Area of special protection for birds). Consequently, the environmental impact of the viaduct in the landscape should be classed as 'marginal', and the river bed cannot be affected by the viaduct.

1.2. PARTICIPANTS IN THE CONSTRUCTION PROJECT

The main participants in the construction project of the Almonte viaduct have been the following:

- ADIF AV (Administrador de Instalaciones Ferroviarias. Alta Velocidad): Belonging to the Development Ministry of Spain government, is the owner of the line and consequently the property developer.
- Designer: The designer of the whole project is a joint venture between IDOM and Arenas & Asociados, the latter company is responsible for the design of the Almonte viaduct itself. It is notable that this joint venture has been also responsible for the supervision of the construction of the viaduct.
- Main Contractor: It is also a joint venture, between FCC Construcción and Conduril, with FCC holding an 85% share. Furthermore FCC Bridges Department have been liable of developing the final design of the viaduct, which comprises certain adjustments to the detailed design to adapt the project to the resources that were to be used with full involvement with the designer.

1.3. BUDGET AND SCHEDULE TIME

The design of Almonte viaduct concluded in December 2009, and ADIF awarded the construction works to the joint venture FCC-Conduril in April 2010. Construction on the viaduct began in August 2011, and the arch commenced in April 2012. The arch was completed in August 2015 and completion of the bridge as a whole is scheduled for October 2016.

The overall budget of the whole contract section is 81.8 M€, while the budget related to Almonte viaduct is 46 M€.

2. DESCRIPTION OF THE SOLUTION

2.1. VIADUCT DESCRIPTION

During the study of solutions stage, different options were studied to cross the river bed, complying with the environmental impact statement. Finally, an over-deck concrete arch was chosen to bridge the Almonte river bed. Once built it will become the longest railway arch viaduct with a span of 384 m, and the third longest in the world considering all concrete arch viaducts for highways and railways, only surpassed by the Wanxian Viaduct (420m) in China and KrK I (390 m) in Croatia. In order to support the deck over the arch, eight piers were located on the arch at 42 m centres, merging the deck and the arch in a unique section in the 17 central metres of the arch. The approach viaducts of the deck to the edges of the river for each side is arranged with approaching spans of 45 m until the deck reaches the main piers over the arch foundations. This way, a 996 m length viaduct is materialised with a straight alignment and a slight vertical sag curve with a parameter of Kv=45.000 m, being the sag point on the approaching north spans.

In order to improve the behaviour of the arch resisting transverse forces (mainly wind), the arch is divided into 2 arms at the springing point, merging in a single section 87 metres away from the springing, keeping this configuration over the central 210 m crown section.

2.2. CROSS SECTION OF PIERS, ARCH AND DECK

The piers of the viaduct have an octagonal hollow cross section of variable dimensions and with a constant wall thickness of 0.40 m, except in the main piers over the foundations of the arch (P6 & P15), whose wall thickness is 0.80 m at the base, and solid from the middle of the pier to the top. This is necessary to achieve the anchorage of the 8 first families of temporary stays.

The cross section of the arch is variable in its whole length, clearly distinguishing two different sections:

a) Double cross section: The arch has double hexagonal hollow cross section with a variable depth between 6.90 m and 6.10 m and a variable wall thickness between 1.07 m and 0.64 m in the first 87 m from the piers positioned at both banks. The transverse distance between the external faces varies between 19 m at the springing point and 8.37 m where the two legs merge into a single cross section.

b) Single cross section: In the 210 central metres of the arch, it has an octagonal hollow cross section with a variable depth between 6.10 m and 4.80 m in the key, and a variable wall thickness between 0.97 m and 1.16 m. The width of the cross section is also variable, decreasing to only 6 m at the crown.

On the other hand, the cross section of the deck is a typical post-tensioned box with a constant depth of 3.10 m, and 14 m of width, throughout the approach spans and the spans over the arch. As mentioned, the arch and the deck merge into one combined cross section at the central 17 metres of the arch.

2.3. MAIN MEASUREMENTS

The main measurements in the Almonte viaduct construction are the following:

- Concrete: 60,000 m3
- Reinforcement bars: 8,250 tonne
- Posttension strands: 500 tonne
- Cable Stays: 910 tonne
- Temporary tower steel: 920 tonne
- 2000 kN Ground Anchorages: 6,000 m
- Cement injected to the ground: 530 tonne

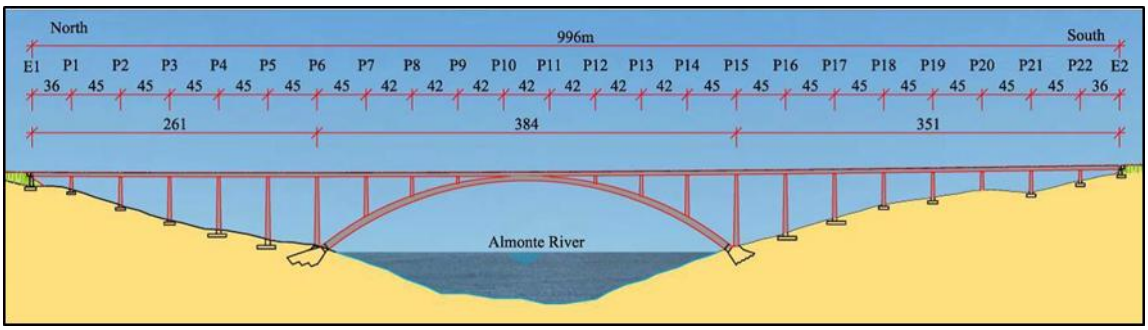


Fig. 1. Longitudinal profile of Almonte River

Fig. 2. Construction process of the arch

3. CONSTRUCTION PROCESS

3.1. GENERAL DESCRIPTION OF THE CONSTRUCTION PROCESS

Initially all the efforts of the construction team were focussed on how to get a good access to the springing points of the arch and also to the foundations of every pier of the approach viaducts. To that end, two haul roads were designed throughout the hillsides of both banks of the river, limiting the minimum radius and the maximum slope allowing the access to the springing points of the arch for huge trucks. The implementation of these access roads involved a challenging labour of earthworks and cutting slopes supports.

Once the access roads were finished, the foundations of the piers and the arch could start. All of the foundations have been achieved with shallow foundations on bedrock. In order to carry out the excavation of the foundations, slope supports have been necessary. Once the foundations were built the implementation of the piers could begin. Built in 40 MPa characteristic resistance concrete, they were executed in lifts of 5.0 m long using climbing formwork. The steel reinforcement is assembled on the ground and lifted with cranes.

The deck of the approaching spans started once the piers were finished. South and north spans were built almost simultaneously with two Overhead Movable Scaffolding Systems (Overhead MSS). When the last span on each side were finished (the one supported by the main pier and the prior), the MSS went back and waited until the arch were finished, because they were liable for the execution of the deck over the arch.

In parallel, the arch started to grow on each side. First of all, segments 1 & 2 were built by means of a conventional falsework of high loading capacity (due to the huge depth and weight of these segments), working perpendicular to the axle of the arch in the beginning. Once the segments 1 & 2 were built, the traveller formworks were assembled and anchored to these segments in order to begin the construction of the arch by means of the successive cantilever construction method, temporarily supported by cable stays. The arch is divided into 32 segments each side, with approximate length of 6.70 m each segment and the key central segment. This construction process is typical of cable stayed bridges. The 8 first families of stay cables were anchored to the main concrete piers while the 18 remaining families were anchored to a temporary steel tower specifically erected over the deck, just above the main piers, in order to get the necessary height for an optimum attack angle of the stays. For the purpose of avoiding longitudinal movements in the temporary tower due to rheological and thermal effects on the deck that could have had damaging effects on the execution of the cantilever, a fixed point between deck and main piers was materialised. From segment 3 to segment 15, a couple of form travellers were used in each side to build the arch. Once both form travellers reach the single section part of the arch they change their configuration and turn into a single form traveller to finish the arch.

When the 32 segments per side were built, and the form travellers of one side were disassembled, only closure of the arch was left in what we call key segment. This segment was built by means of the formwork travellers of the North side, anchored at the end of the cantilever of both sides.

Before casting the key segment it was necessary to block one cantilever to the other in order to avoid differential movements between both sides due to thermal effects on stays and arch that could crack the concrete. For this purpose 4 struts were disposed anchoring both cantilevers. Once the arch was closed, the temporary stays were no longer necessary so the cable stays were loosened and disassembled in order to begin the construction of the 8 piers over the arch.

While the piers over the arch were growing, the overhead gantries were assembled again and were moved forward to the main piers in order to start the construction of the deck over the arch. Once the piers were finished, the construction of the first span per each side could start. It was necessary to resolve the clashes between the two MSS in the construction of the three central spans of the deck. As mentioned, the central span is anchored to the arch in the central 17 metres, achieving the final configuration of the fixed point of the viaduct, and able to transmit all the longitudinal forces of the deck, through the arch to its foundation. In this stage it was necessary to release the provisional fix point in the main piers.

3.2. ARCH AND RETAINING FOUNDATIONS

3.2.1. Arch Foundations

The foundations of the arch are also foundations to the main piers (P6 & P15) and were achieved with shallow foundations over bedrock, which is slate, with concrete volumes between 6,000 and 7,000 m3. The main support plane of the foundations is orthogonal to the arch axis in its beginning and the upper side is horizontal in order to support the main pier, which is also the tallest. Before starting the construction of the foundation and after the excavation was done, a systematic ground treatment needed to be made, in order to fill the gap between the schistosity planes of the slate, which are usually filled with clay. These gaps filled with clay could result in movements in the structure when all the loads were applied, so they had to be filled with an appropriate material.

Due to the foundations being of such a large size, the pouring of the piece had to be divided in several phases, adjusting approximately the volume of each phase to 1,000 m3 and assuring the shear capacity of the layers between concrete surfaces.

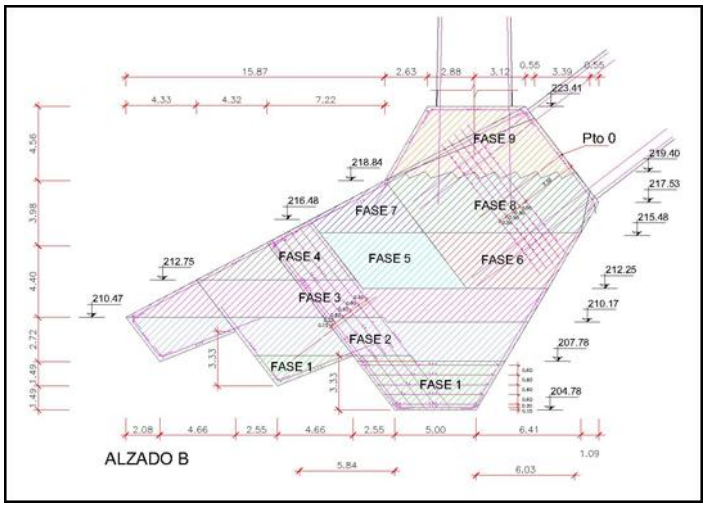


Fig. 3. Concrete performance phases of the arch

3.2.2. Retaining Foundations

The retaining foundations are the two adjacent foundations to the arch foundations. The main purpose of these foundations, besides of being the foundation of a pier, is to anchor the stays to the bedrock, avoiding movements in the structure during the construction of the cantilever. As well as the arch foundations, the ground had to be treated with cement injections under pressure, because of the huge effort during construction that the foundation was going to support in order to avoid movements in the structure.

For this purpose 60 ground anchors of 12Φ0.6” were made in each one of the four retaining foundations, with a fix part grouted to the ground of a length of 16 m, and a free length between 6 and 10 m depending on the anchor. These anchors were stressed to 2,000 KN. The drilling was made after the implementation of the foundations, by means of several pipes embedded in the concrete with the accurate angle and position. On the other hand, the stays were anchored to the foundation by means of Macalloy bars (high tensile alloy steel bars) housed previously in the foundation.

3.3. MAIN PIERS

For the construction of the piers of the viaduct, a climbing modular formwork specially designed for the variable geometry of the piers of Almonte viaduct has been used.

Fig. 4. Auxiliary structure for the crossing of stays at the main piers

In the main piers (the ones starting from the arch foundation), the 8 first families of stays had to be anchored. For this purpose, these piers had to be solid along most of its height. On the other hand these piers must enable the crossing of these stays through themselves.

For this end, an auxiliary structure was designed in such a way that the pipes crossing the piers had the accurate angle, azimuth and position. A 3D model had to be done to check all the interferences between these pipes and the reinforcement of the pier.

3.4. ARCH

3.4.1. Formwork travellers

The success in the execution of the bridge is based mainly on how to solve both 192m long cantilevered arches with a most peculiar geometry with two hexagonal arms that start from a common base with octagonal geometry and that, halfway through the cantilever, blend themselves in a single leg with octagonal geometry. The depth and width of the section are variable along the cantilever what complicates enormously the formwork installation, rebar placing and concrete pouring. That is the reason why the travelling formwork selection has been key in the project success.

Such formwork travellers had to be able to withstand the weight of the fresh concrete of a segment working as a cantilever anchored to the prior segment and pressures up to 90 kN/m2 (as self-compacting concrete) and of course they had to be self-sustaining and automotive.

The travellers' supplier was a Spanish company (Rúbrica) and the detailed design was carried out by themselves in collaboration with the Technical Services of FCC. Two formwork travellers were designed for the construction of each one of the two cantilevers, in such a way they could join into one

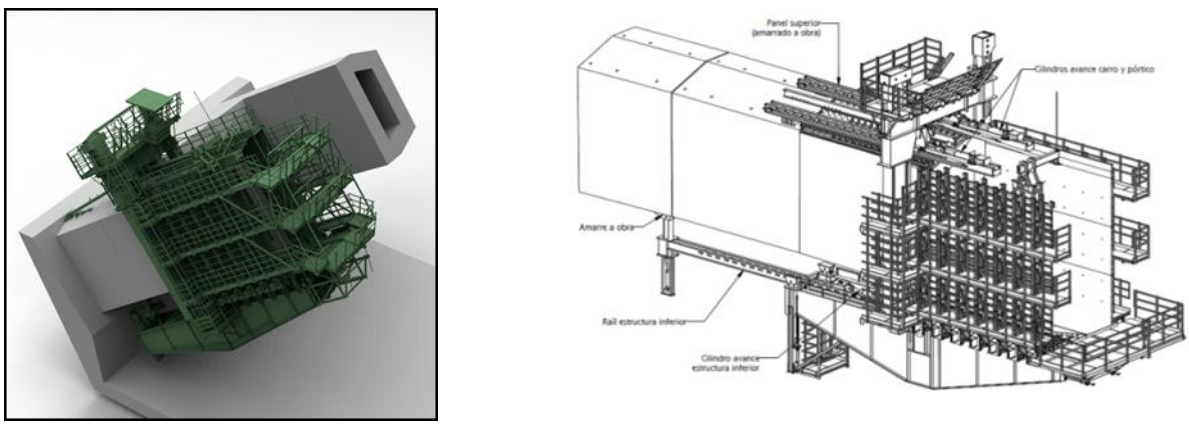


Fig. 5. 3D Model of the formwork travellers used in the construction of the arch

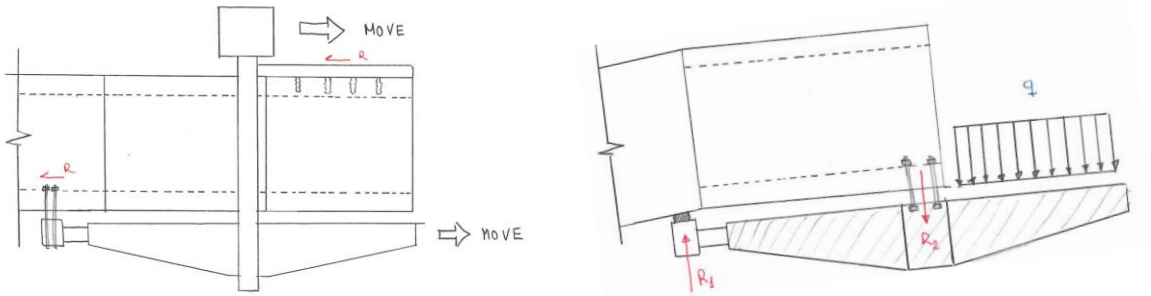


Fig. 6. Two different resistant scheme of the formwork travellers

single traveller to build the one single octagonal leg of the arch.

The formwork travellers have two different resistant frameworks for the two main operations they have to carry out:

a) Casting of a segment: The resistant scheme is based on a cantilever beam connected to the prior segment. For this end, the travellers are formed by a main beam with variable depth supporting the formworks used to cast the segments. The connection of the travellers are materialised with two supports on the prior segment, one in compression and the other one under tensile forces.

b) Advance movement to the next segment: the movement is done by means of a hydraulic system, reacting against the prior segment, both on the top and the bottom of the arch, resulting a repetitive movement similar to the expansion and contraction used by worms.

3.4.2. Geometrical control

In any bridge constructed by the successive cantilever method, the geometrical control during the construction is decisive to get the appropriate geometry in the bridge axle after applying all the dead load and keeping in mind the rheological effects. For this purpose, construction precambers have been considered analysing deeply the construction process and the age of every segment.

On the other hand, thermal factors are essential to control the geometry of the cantilevers during construction. The construction precamber must be amended taken into account the real temperature of the arch concrete and of the stays, known thanks to the bridge monitoring.

In order to avoid the movements in the arch due to the thermal gradient in the arch (hardly evaluable because of a huge depth), the geometrical control was always done in a moment when the gradient was practically zero (generally at first time in the morning).

Technical Services of FCC developed a computer tool for the geometrical control of the bridge in order to take into account the construction and thermal precambers in each segment.

3.4.3. Temporary steel tower

3.4.3.1 Description and operating mode

As mentioned, a temporary steel tower is needed to be erected upon the main piers in order to get an effective attack angle in the 16 last stay cables families. The temporary steel tower was redesigned by the Technical Services of FCC in the detailed project, and its main characteristics are the following:

- The use of double-T profiles, in such a way it was easy to assemble and the welded connections had an easy access to inspect.
- A hinge was designed as a junction between the tower and the deck. Thus the tower could be independent from the deck, and the moments due to unbalances between forward and retaining stay cables would not be transmitted to the deck.

3.4.3.2 Tower lifting

As a consequence of the hinge implemented in the base of the tower, a tower lifting procedure was developed in order to assemble the tower in a horizontal way and then raise the tower around the hinge. This procedure achieved the reduction of bolt connections (in a vertical way of assembling, the tower should have been divided into more pieces) and removed the working at heights.

The tower lifting procedure was developed by the Technical Services of FCC in collaboration with BBR (a subsidiary company of FCC). An auxiliary structure shaped like a tripod was designed. This structure anchored to the tower was pulled by means of horizontal heavy lifting equipment, until

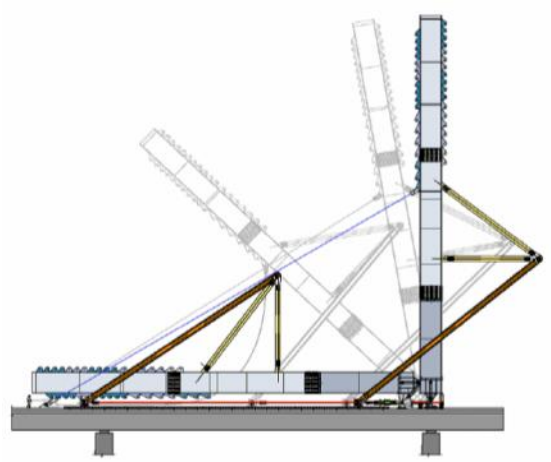


Fig. 7. Lifting procedure and temporary steel tower working

the vertical position of the tower was achieved. This manoeuvre was performed in less than one working day.

3.4.4. Arch closure

One of the most challenging operations was the arch closure. The main purpose of closure operations in bridges built by the successive cantilever construction method is to nullify the differential movements between both cantilevers during the casting of the key segment in order to avoid the cracking. The closure was performed in August 2015, one of the warmest and sunniest times of the year in the location of the bridge. In August, thermally-induced daily movements were at their greatest.

These daily movements ranged as follows:

- Daily stay cable heating was at its most intense. Thermal daily variations of up to 24°C were recorded in the cables, generating downward movements of up to 120 mm.
- Daily transverse solar radiation on the cantilever was also at its most intense, inducing horizontal movements of around 60 mm between 9:00 AM and 11:00 AM.

A steel structure consisting in four longitudinal double-T profiles positioned in the key segment near each corner of the cross section of the arch was designed to reduce these relative movements. The chosen moment of the day to perform the arch closure was between 6:30 AM and 9:30 AM, time lag with the lowest thermal gradient when the movements could be decreased to minor values. In that short time lapse, the temporary struts were received with high-strength, quick-setting mortar and prestressed before the effects of solar radiation appeared.

This structure was set in place on 4 August 2015 and the closing segment was cast on 6 August 2015. The operation was successful and the relative error between the two cantilevers at the key and the absolute error after that operation were both under 10 mm.

3.4.5. Self-Compacting Concrete with high compression strength

There are three circumstances which make the use of self-compacting concrete in the arch of Almonte viaduct essential:

- The variable and complicated geometry of the arch makes the vibration of the concrete impossible in so many points.
- The huge density of reinforcement in the segments.
- The required formwork of the upper face of the arch makes impossible the cast of a segment without a self-compacting concrete.

On the other hand, arches are elements submitted to huge axial forces due to their natural resistant scheme, so high compression strength concrete is required. Furthermore characteristic resistance of 40 Mpa was needed at early age (12 hours) in order to optimize the working cycle, advancing the travellers in less than two days since casting of a segment.

The self-compacting concrete used at the arch of Almonte viaduct have a characteristic resistance of 80 MPa at 28 days. The selected material and its main characteristics are the following:

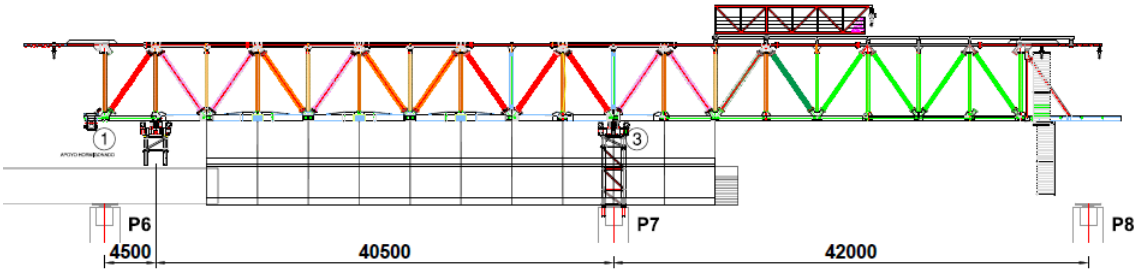


Fig 8. Longitudinal profile of one overhead gantry used in the construction of the deck

a) Ultraval SR special cement: Contains low quantities of AlC3. This fact avoid the possible delayed ettringite formation due to high temperatures reached during the concrete setting. It contains also higher grinding fineness than standard cements getting high beginning resistance (>40 Mpa) in 12 hours and more than 90 Mpa in 28 days with maximum quantities according to Spanish normative EHE 08 (460 kg/m³) It is produced by Cementos Portland Valderrivas (FCC Group) in Navarra (North of Spain).

b) River sand use: Produced 135 km away from the site are essential to avoid concrete blockage and segregation and to make possible pumping the concrete to long (>200 m) and height (>70 m) distances.

c) Fly ash and chemical admixtures use: Fly ash use give higher long term resistance, improving self-compact quality.

d) Besides last generation of super fluidisers have been used to keep the concrete in good conditions to be set in the first 90 minutes.

3.5. DECK

3.5.1. Approach spans

As mentioned, the approach spans of the viaduct (6 spans on North side and 8 spans on South side, all of 45 m) have been constructed by means of two Overhead Movable Scaffolding System (Overhead MMS), one on each side.

These kinds of systems consist in a self-supporting steel structure combined with an exterior formwork, able to withstand the fresh concrete of a bridge span, supported on two piers. In Overhead MSS the main supporting truss is above the bridge span, so the exterior formwork is hanging up the truss, by means of high tensile bars, and can open and close easily because of the hydraulic system implemented. The Overhead MSS is able to move by itself with the help of a hydraulic an electric system.

The whole cross section has been casted in one phase, by means of an inner formwork traveller, able to scroll from one span to the next, by means of a hydraulic system.

3.5.2. Spans over the arch

The spans over the arch have been constructed, likewise the approaches span, with two Overhead MSS, working one on each side. The construction process was designed in such a way the deck over the arch be constructed symmetrically, but permitting an offset of several days between the span on one side and the symmetrical one, so two overhead gantries were essential to fulfil this maxim.

Fig 9. Interferences between the two Overhead MSS in the construction of the three central spans over the arch

In the construction of the three central spans over the arch, the clashes of the two Overhead MSS have been necessary to solve. These interferences have been solved by means of the partial disassembling of the two gantries, reducing their length without affecting their stability. The central span has been built with only one overhead gantry, pulling the other one back, being necessary the performance of a closure operation similar to the one conducted in the arch in order to avoid the cracking in last span.

4. INSTRUMENTATION AND MONITORING OF THE BRIDGE

A bridge with such a complex construction process requires an exhaustive control of several parameters to know exactly how the bridge is behaving at any moment, structurally talking, but most intensely during cantilever construction. For this reason the bridge was instrumented throughout its construction.

The data continuously recorded by the sensors were transmitted to a website where real time and historic information could be displayed.

In addition, each sensor was programmed to immediately send an alarm signal to the site supervisors' cell phones when a given threshold value was exceeded.

Four different types of sensors were used:

a. Geometric monitoring: Targets and prisms on the arch, piers and provisional towers. Clinometer on piers and towers were also disposed.

b. Environmental monitoring: Concrete temperature in arch, piers 6 and 15 and provisional piers and temperature in stay cables. A weather station was also positioned to monitor ambient temperature, humidity, wind, etc…

c. Stress monitoring: Strain gauges on stay cables, on reinforcing steel of the arch and on provisional towers and also load cells on ground anchors.

d. Terrain movement monitoring: Strain gauges on arch foundations to monitor settlements.

5. CONCLUSIONS

Erecting an arch of this scale, overcoming the world record span of a railway arch bridge in more than 110 m, for a structure in which the structural scheme during construction differs widely from its in-service functionality is a complex endeavor. Every single element has required a deeply study, because of being out of scale of what have been constructed until now.

The construction of the bridge has been conducted by FCC and the extra detailing and designing required to address this complexity were performed by FCC's Engineering Department, which also provided worksite support. The authors of the design verified all the work performed in this stage of the project, ensuring full cooperation among all the agents involved.

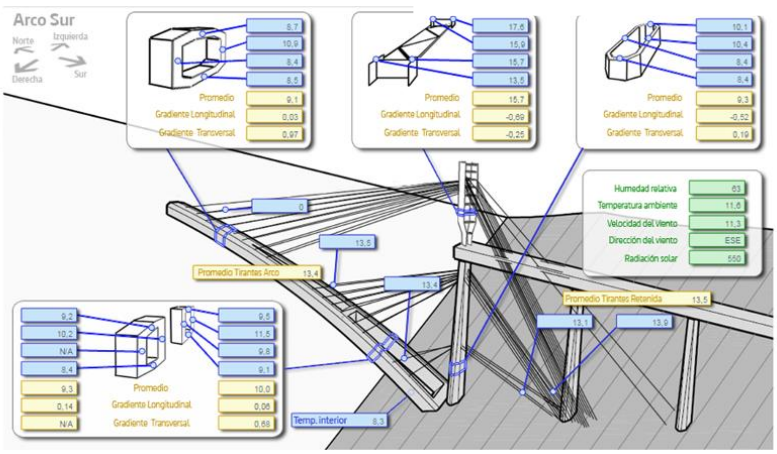
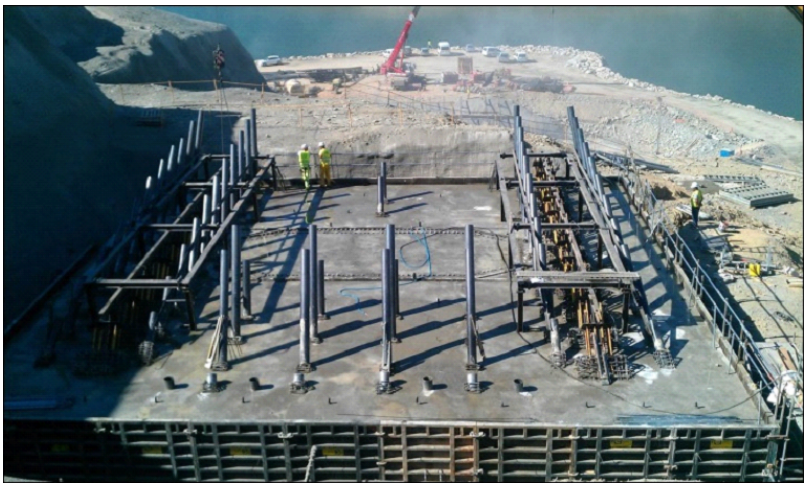
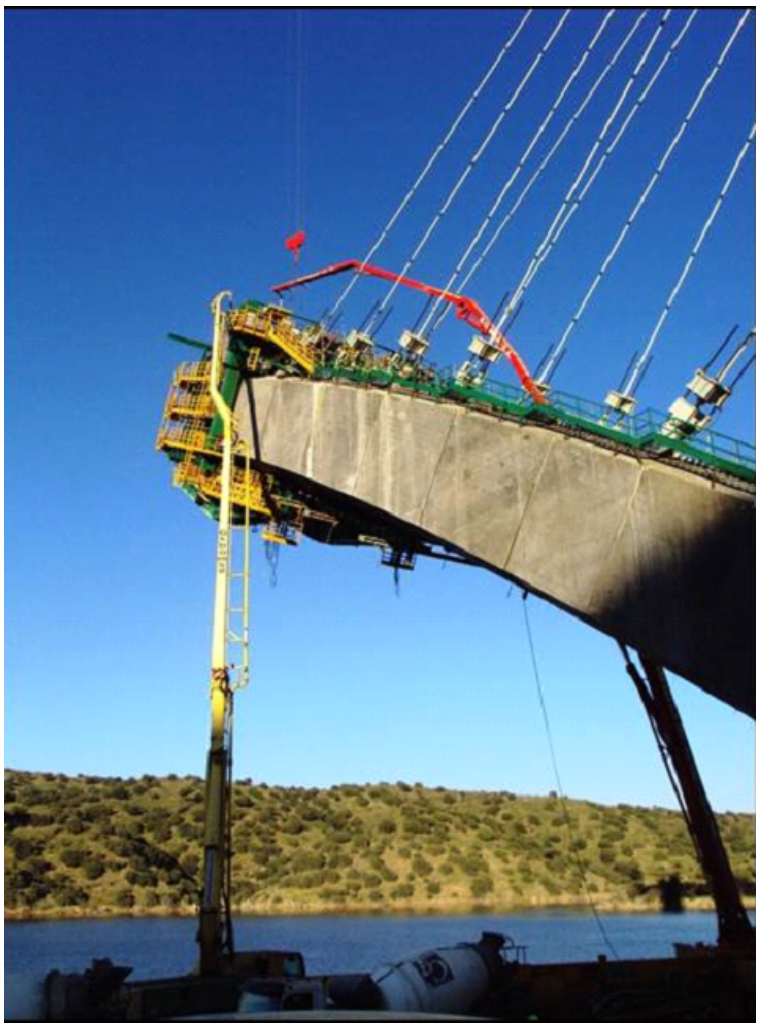
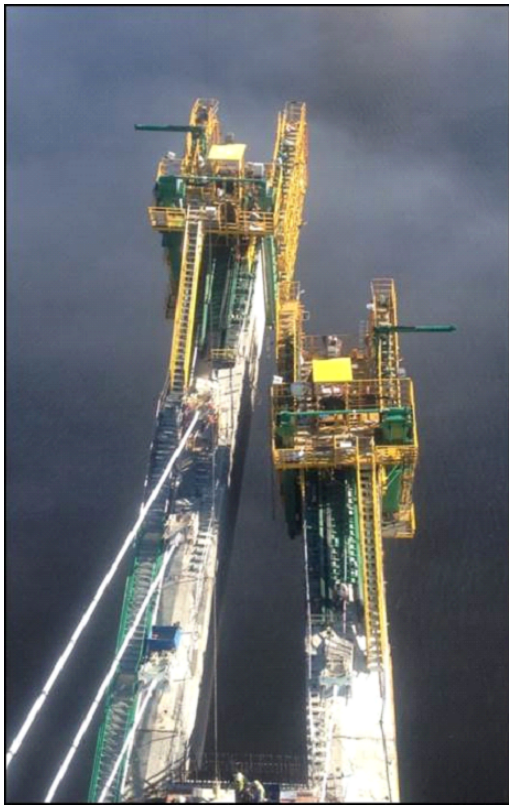


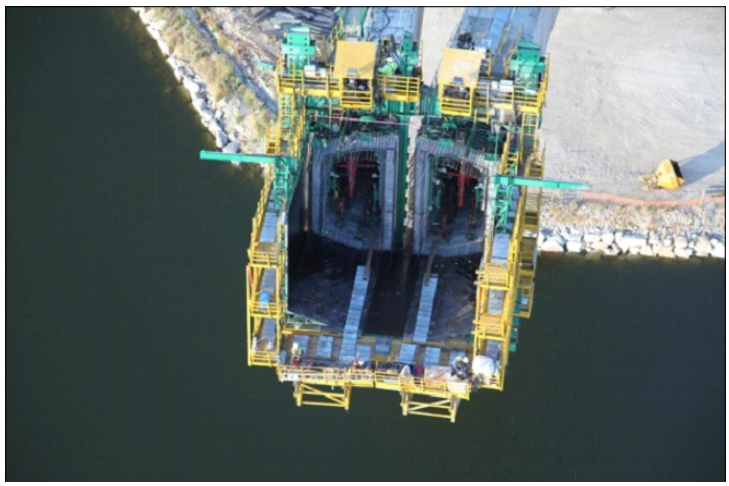
Fig 10. Instrumentation of the bridge during its construction

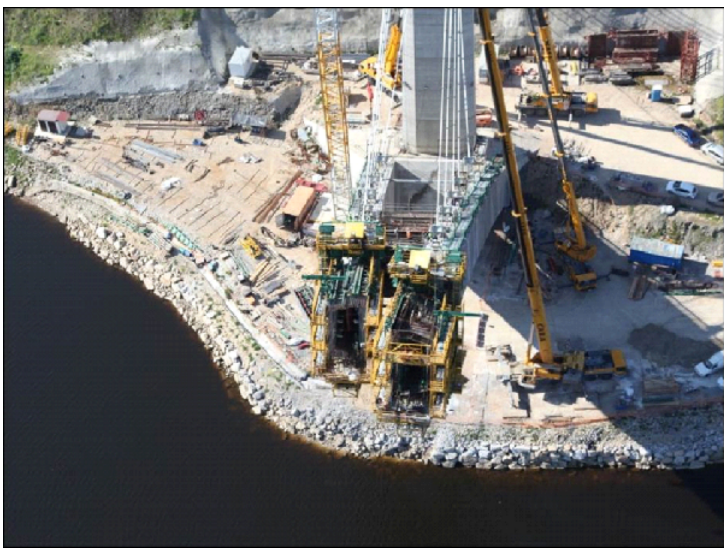
PHOTO GALLERY

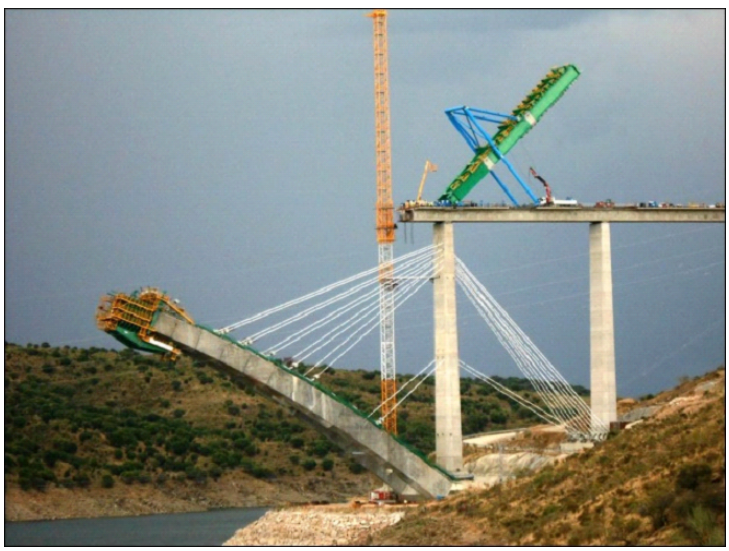


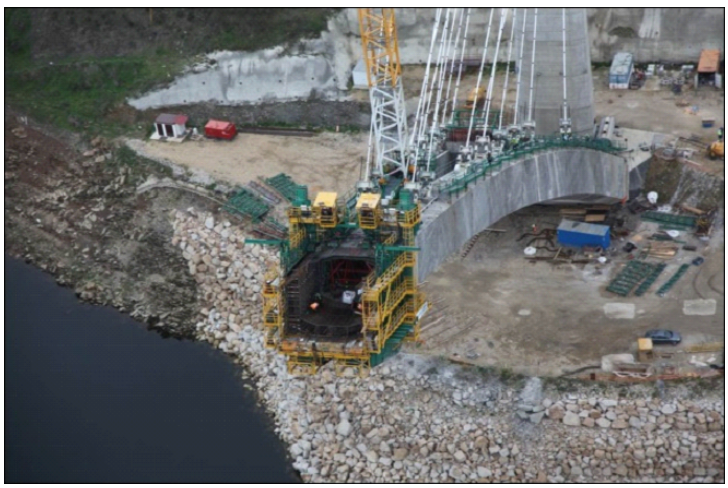












TAGUS RIVER HSR VIADUCT

Photo: Carlos Manterola for Carlos Fernandez Casado

Commencement of works:
Foundations commenced in summer of 2011

Opening of the bridge to traffic:
Completion of works in September of 2017

Type: concrete HSR arch bridge

Total Length: 1.488m

Main span: 324m

Location:
Alcantára Reservoir – Cañaveral, Cáceres, Spain

Client:
ADIF AV (Spanish Rail Administrator), Madrid, Spain

Conceptual and detailed design:
Carlos Fernandez Casado, S. L.

Design engineers:
Javier Manterola, Antonio Martínez, Borja Martín, Silvia Fuente, Miguel Ángel Gil, Lucía Blanco

Main Contractor:
COPISA-COPASA JV.

Location of the bridge (Source: Google Maps)

TAGUS RIVER HSR VIADUCT

Javier Manterola, Antonio Martínez Cutillas, Borja Martín, Hector Faúndez
Carlos Fernández Casado, S. L.

Manuel Escamilla, Luis Miguel Salazar, Jorge Delgado
Pontem Engineering Services, S. L.

Summary

The Viaduct over the Tagus River in the Alcántara reservoir is part of the High Speed Rail Link between Madrid and the Portuguese border. It has a 324 m long arch that spans the river Tagus, widened due to the influence of the Alcántara reservoir. The deck spans over the arch are 54m long.

The two approach viaducts have different layouts due to the difference in height between the terrain and the deck on both margins. They are composed of 60m long spans except the spans adjacent to the arch, which are 57m long.

The arch is formed by a box section variable both in width (12.00 m – 6.00 m) and height (4.00m -3.50m) with its axis designed to reduce the bending moments due to dead loads to a minimum.

1. GENERAL DESCRIPTION

The bridge has a total length of 1488m, with a span distribution of 45 + 9x60 + 57 + 324 + 57 + 7x60 + 45m. After a comprehensive typological analysis, in which cable-stayed and three-dimensional truss alternatives were evaluated, the arch solution was selected as the best structural approach. The harmonic balance of span distribution, deck and arch geometries and pier shapes provide an optimum structural performance, and configures an aesthetically remarkable design.

1.1. Deck

The deck is a prestressed concrete box girder with a maximum height of 4.00 m. The upper slab is cantilevered on both sides so that the total usable surface is 14.00 m with variable thickness. The lower slab is 5.00 m wide and has a thickness of 0.30 m. The web thickness is 0.50 m, although it thickens where it meets the upper and lower slabs.

Concrete: with 50 MPa strength is used in the approach spans and 70 MPa concrete in the spans over the arch. Very often the actual strength of the poured concrete reached 100 MPa.

Five prestressing tendons, using between 25 and 37 Ø15.2 strand units are used in each web. The spans over the arch are complemented with upper and lower straight tendons.

Due to the length of the viaduct, a detailed study on where to place the fixed point was undertaken. This study showed that the arch would be able to withstand the increase of stresses with the fixed point placed at its center. This resulted in the typical configuration of expansion joints in the abutments. The team cast one span per week (launching, steel reinforcement works, formwork, pouring, curing and prestressing).

1.2. Arch

The form of the arch has been designed such that the bending due to the dead loads was reduced to a minimum. The arch axis is completely embedded in a vertical plane.

The cross section consists of a variable rectangular box section with chamfered corners. The width varies between 12.00 m at the abutment and 6.00 m at the crown. The height varies between 4.00 m and 3.50 m with the web and slabs thickness also varying to achieve an almost homogeneous distribution of compressive stresses. Due to the magnitude of the stresses, the 70 MPa concrete was used.

Because of the location of the arch in the valley scoured by the river Tagus (Tagus means “cut” in latin), it was necessary to carry out a study of the effect of the wind on the structure during the construction process. Hence, a reduced scale model was developed and tested in a wind tunnel, confirming that the structure was not sensitive to wind induced instability phenomena.

1.3. Piers

The variable terrain elevation causes the piers to have a very variable height (between 9.60 m and 71.50 m). A basic hollow box section form is used in all of them, 3.50 m wide but with variable transverse dimension.

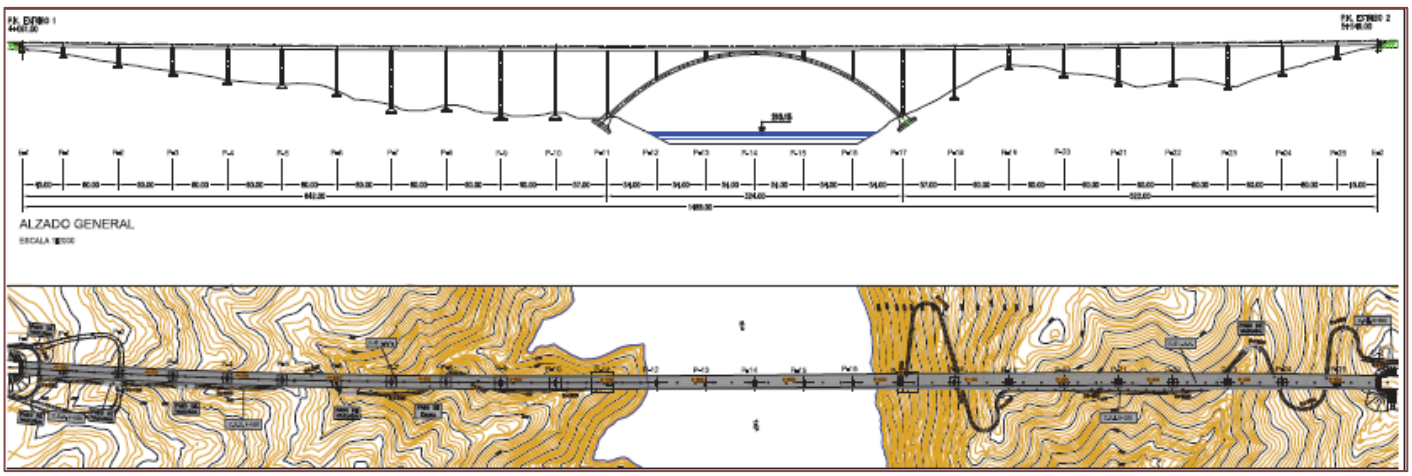


Fig. 1: General definition

Fig. 2: Rendering of the bridge

1.4. Other Elements

The deck is supported by spherical bearings except at the fixed point, where the arch is monolithically linked to the deck.

Box abutments with intermediate walls are designed with maximum heights of 8.63 m at one end and 9.64 m at the other. The bearings and expansion joint devices are supported by these.

Environmental issues with birds made it necessary to use railway edge barriers 3.00 m high. The barrier is formed by steel curve tubes 100 mm diameter every 0.50 m. In the lower part longitudinal horizontal tubes with differing diameters are used. The aerodynamic behavior of this barrier was tested with satisfactory results.

2. CONSTRUCTION PROCESS

2.1. General definition of the deck construction

The deck is built by a movable scaffolding system, consisting of a steel truss. The steel truss is supported by two adjacent piers, this being the chosen for method every span of the viaduct.

To avoid excessive stresses in the arch, the erection of the spans over it was symmetrical. The deformation of the arch and an interaction between the elements of the bridge was considered in the design.

2.2. General definition of the arch construction

The arch is erected from both sides of the bridge concurrently, and was supported by diagonal cable stays strung from the adjacent piers and two steel temporary towers. The towers were in turn stayed to the foundations of the next two piers as they needed to be anchored to the ground to withstand the loads exerted by the stays.

For each semi-arch, several auxiliary elements are needed: A steel tower was placed over the piers and; a form traveler system. A group of cable-stays was used to support the built part of the semi-arch, it was anchored to the tower and piers to maintain the equilibrium of the tower. The pier was anchored to the foundations and the foundations in turn anchored to the ground.

Each steel temporary tower was composed by two columns at a distance of 6.50 m joined by a K bracing. The section of each column is a hollow steel box.

The tower width is reduced where the tower meets the deck, to allow a correct transmission of the loads to the pier top.

The bracing system is composed of horizontal beams placed at 4.00 m spacing and diagonal elements that form a K turned 90 degrees. The bracings were steel double T fabricated cross sections.

The form traveler system was also a steel structure designed to support the formwork of each arch segment which allows the concrete to be cast in place. It was a cantilever structure, with a weight of 95 tonne attached to the already executed and hardened part of the arch.

The arch is composed of 93 segments each about 3.80 m long.

Regarding the cable-stayed system, 15 pairs of stays supported each semi-arch and another 15 pairs held each tower. A pair of stays was placed in the arch every three segments.

Tower cranes:

Materials (reinforcing bars, concrete and prestress) were taken to the form traveler system through two tower cranes installed on each semi-arch.

As in other major infrastructure projects, auxiliary structures are key to the satisfactory completion of the works. The project had a system of four tower cranes and temporary stairs to access the 70 m high

piers, the 54 m high temporary pylon, the cable stay tensioning platforms and the inside and outside of the arch. All of these cranes were required to supply the materials for the casting of each segment as follows: To build the first 21 segments of each semi-arch, two tower cranes were needed. The height of the cranes was approximately 135 m to avoid interferences with the temporary steel towers. They were placed next to the arch foundation and built on a micropiled foundation. Due to their height, they were braced at approximately half of their length to avoid buckling. Once the 21 segments were finished, the cranes were dismantled and moved to the next position over the arch.

Cranes over the arch:

The same cranes, but with its height reduced to 85 m, were assembled on the previously cast 20th segment of each semi-arch. To achieve this, a docking structure that allows the connection between the crane feet and the segment inclined surface was designed. Because of the height where the jib is placed, there were no interferences with the temporary steel towers, allowing complete rotation. Due to the magnitude of the punching loads transmitted by the crane feet, with compressions over 5 MN and tractions over 4 MN, it was necessary to introduce some modifications to the segments where the cranes were placed to ensure a correct load transmission.

Once the 38th segment was completed, cranes 3 and 4 were assembled over the arch. To avoid interferences between jibs, these cranes were 45 and 35 m high. These cranes have no cabin and anchored at 50 m over the arch foundation. The adaptation of the segment to receive the loads from the cranes was also necessary, as well as that of the docking structure between the crane feet and the segment surface.

The cranes' operations were possible because of a platform designed to exchange materials placed on the 31th and 32th segments. The outer cranes placed the materials on the platform and the inner cranes took these materials to the form traveler system.

The concrete feeding was undertaken by two concrete skips with a volume of 2 m^3, so that while the inner crane was pouring one in the formwork, the outer one was refilling the other.

Aerodynamic effects have again been studied to check the behavior of the cranes and the arch under adverse wind conditions.

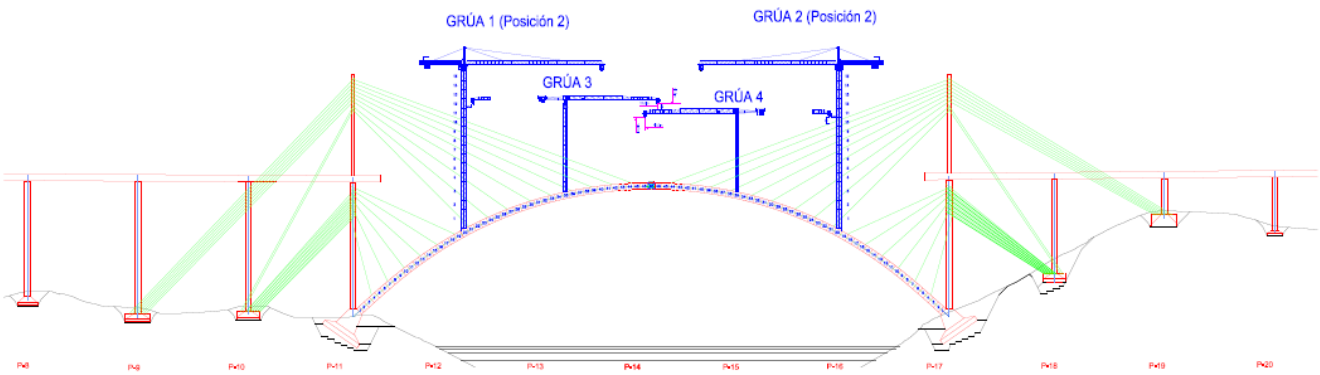


Fig. 3: Schematic of the cranes needed in the arch construction.

2.3. Optimization of the construction process of the arch

During the development of the works, a number of modifications were studied to optimize the use of resources, time and materials.

The main change made to optimize the construction process was in the anchoring of cable stays to the concrete pier next to the arch. This would result in the possibility of starting the construction of the arch without the need to install the temporary steel towers. The main disadvantage of this option was that the cables would have a smaller angle to the horizontal, resulting in turn in great horizontal loads on the pier that would have to be balanced. Additionally, the complexity of the active anchorages at the top of the piers would be greatly increased. Something similar would also occur with the foundations' anchorages. This solution was then conditioned by the maximum number of segments to be built without an increase of the foundations' anchors.

The optimal solution resulted in passing 6 pairs of stays from the arch through the piers, which allowed the casting 20 segments of each semi-arch before the erection of the temporary steel towers and reducing the construction time.

2.4. Provisional stay system

The stays' active anchorages were located at either the pier or the temporary steel towers. The passive anchorages were located at the foundations and each of the semi-arches.

The load applied to each stay could be changed as the anchor was provided with a thread and nut system and the length of the cable stay thus be reduced. The application of tension as well as the decrease of the loads during the construction process was always developed at the active anchorages.

The system was provided with two of the four possible barriers against corrosion defined for permanent cables, so durability was guaranteed while the construction lasted. It also provided thermal insulation which helps to reduce the movements of the semi-arches due to temperature variations.

Substitution of any of the strands of the cables was allowed, should any break or fail. The design also allowed the placement of additional strands once the stay was loaded. Visual inspection of the stays was always possible, so the integrity assured and any abnormality during the execution period detected.

2.4.1 Anchorage in the arch

The anchorage of the cable stays in the arch was achieved by using steel tubes that were left embedded in the concrete during the casting of the segment.

When the concrete of the segment had hardened, they were completed from the outer side with a guide tube with a smaller diameter, welded to the sheath embedded, and which allowed the placement of the deviator in its position.

To support the local stresses that the anchorage system transmits to the arch box, the arch slabs locally increased in thicknesses, with the appropriate reinforcement so that the forces were correctly transmitted.

Fig. 4: Auxiliary steel structure for cable ducts
Photo: Hector Faúndez

2.4.2. Anchorage in foundations

A similar approach was taken for the design of the foundations' anchorages. The foundation has two galleries on its sides so that the passive anchorage might be accessed. The passive anchorage was placed in a passing tube through the concrete section.

Foundations of P-10 and P-18 piers, adjacent to the arch supports, received the first six pairs of cable stays, the other end of the cable stays being in the pier. Two other pairs of cable stays came from the temporary steel towers. This caused a not negligible difference in the angle of adjacent cable stays which resulted in modifications of the geometry of the foundation.

The passive and active anchorages were barely different to those employed in post-tensioned concrete.

2.4.3. Anchorage in concrete piers

The proposed solution had its difficulties. If we recall that 6 type of cable stays will interact with the pier and that each type has 4 cable stays, it is seen that a total number of 24 cables were anchored to the pier. 12 of them provided support to the arch and 12 of them balanced the horizontal loads on the pier by being anchored to the foundation of the adjacent pier.

Because of the high number of anchorages in the pier and its reduced dimensions, a detailed tridimensional model was developed to ensure that the solution was viable.

The possibility of leaving embedded in the upper part of the pier all the anchorages, with no interferences between them, was verified. The real dimensions of the tubes, which depend on the stay system, were accounted for.

To avoid inappropriate concentrations of stresses in the hollow concrete section, the part affected by the anchorages is filled with concrete.

Due to the complexity of the assembly process of the embedded sheaths in the pier, and because a high level of accuracy was needed, a pre-assembly in the workshop using an auxiliary steel structure was undertaken. The accuracy of the assembly was thus ensured and the steel structures transported to site. These structures acted also as support of the reinforcement of the pier and, with an operation easily controlled by classical topography, the anchor

Fig. 5: Form traveler during casting

Photo: Luis Miguel Salazar

system and the reinforcement were placed in their correct positions and prepared for the casting.

Once the arch was erected, and the provisional stay system dismantled, the desired geometry of the pier was achieved by filling the tubes used by the anchorages with concrete.

2.5. Sequence of erection phases of the arch

The first and second segment of each semi-arch were cast with formwork placed over the arch's foundation. Then, the form traveler system installed in the hardened concrete of the first segments and the erection cycle begun.

The form traveler was supported by segment number n and with the formwork placed on segment number n+1. The dimensions of the form traveler system were changed to adjust to the segment dimensions and the reinforcement placed. Concrete was then poured by means of the previously mentioned cranes and some 40 hours after the concrete hardened the form traveler system moves to the next segment where the cycle began again.

Every 3 segments, a cable stay group was tensioned. This was done while the reinforcement of a segment was being placed, thus optimizing the process.

The process was ended by the casting of the middle segment of the arch. The form traveler systems of both semi-arches were adapted and prepared and their ends joined together with an auxiliary steel structure. This reduced the relative movements of both ends and allowed the placing of the reinforcement and the preparation of the formwork. Concrete was then poured and the auxiliary cable stay system dismantled, removing as well the form traveler systems of both semi-arches.

2.6. Erection of piers over the arch

There are six spans over the arch. This requires 5 supports, achieved by means of four piers and the arch key. During the arch erection process, segments of the arch on which a pier would be placed were prepared accordingly.

Once the arch was completed and the cable stay system dismantled, the piers were erected with the same means that were employed for all the piers in the viaduct. A deformational control of the process was undertaken to ensure the adequate behavior of the arch.

2.7. Erection of spans over the arch

Two movable scaffolding systems were employed to allow the symmetrical loading of the arch. They were assembled at the abutments and moved to the spans adjacent to the arch and then launched. The reinforcement was then placed and the concrete poured. Once the concrete has hardened and prestressed, the movable scaffolding system moved to the next span and the cycle repeated.

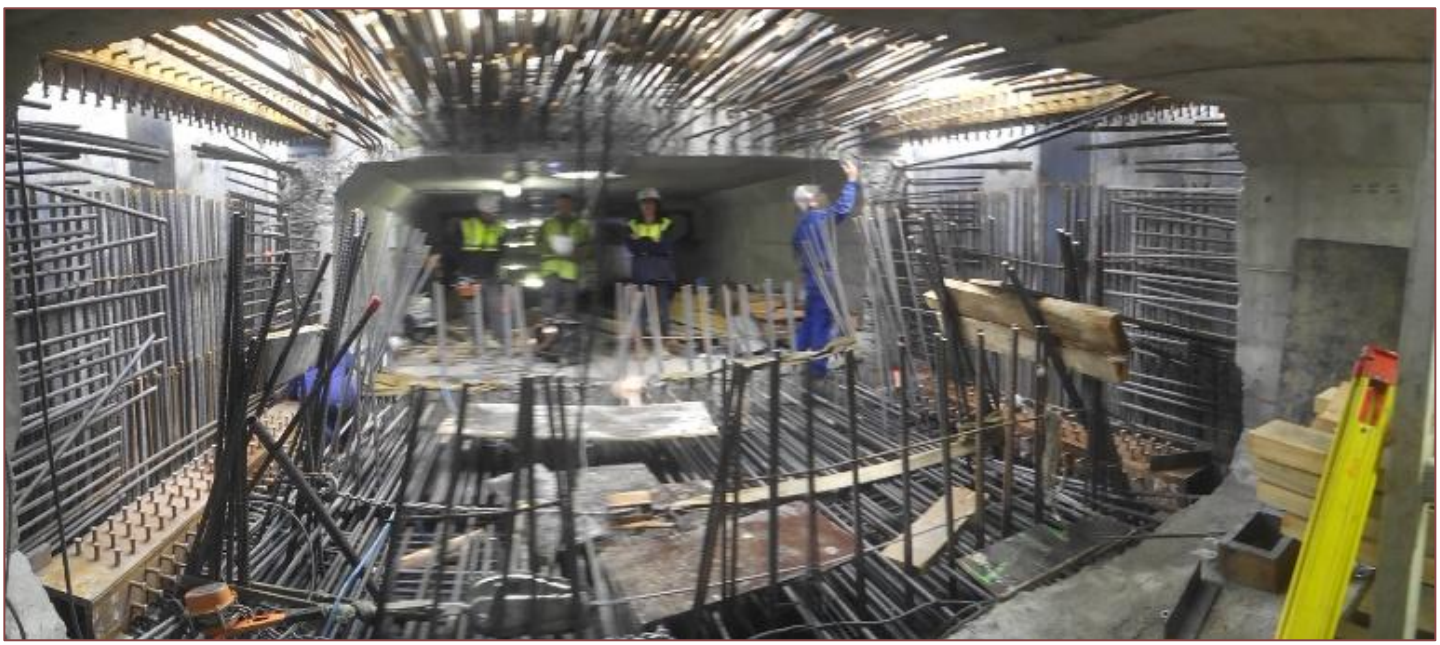
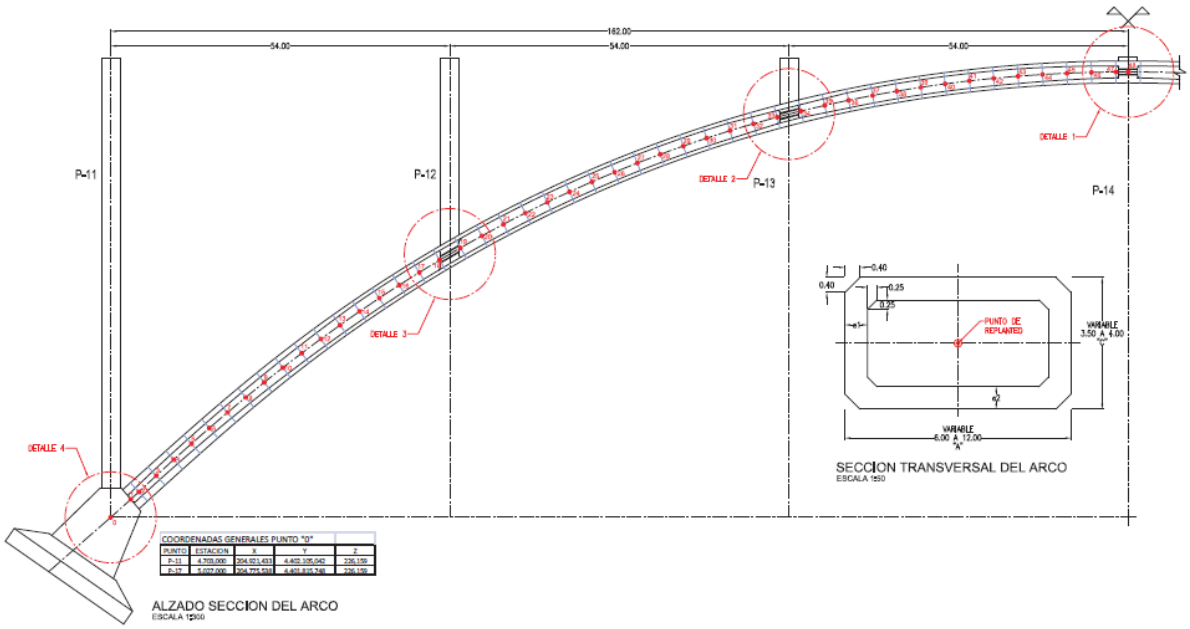


Fig. 6: Auxiliary structure on the key of the arch

Photo: Luis Miguel Salazar



COORDENADAS GENERALES PUNTO "0"				
PUNTO	ESTACION	X	Y	Z
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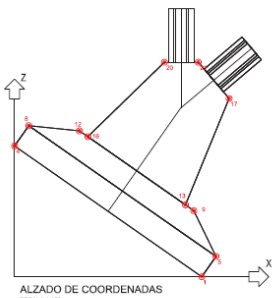
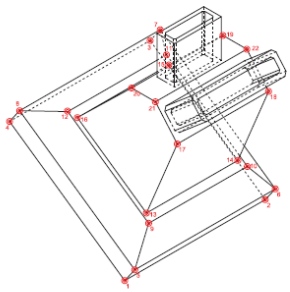
P-12	5.070.000	264.795.518	4.402.610.746	236.159



ALZADO DE COORDENADAS
ESCALA 1:150



VISTA ISOMETRICA DE COORDENADAS
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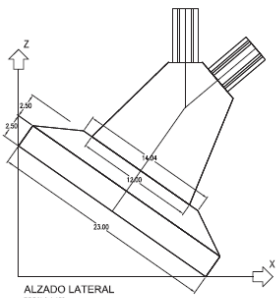

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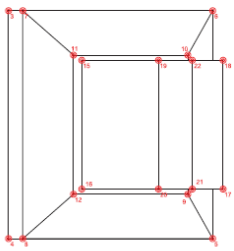
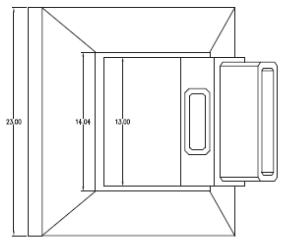
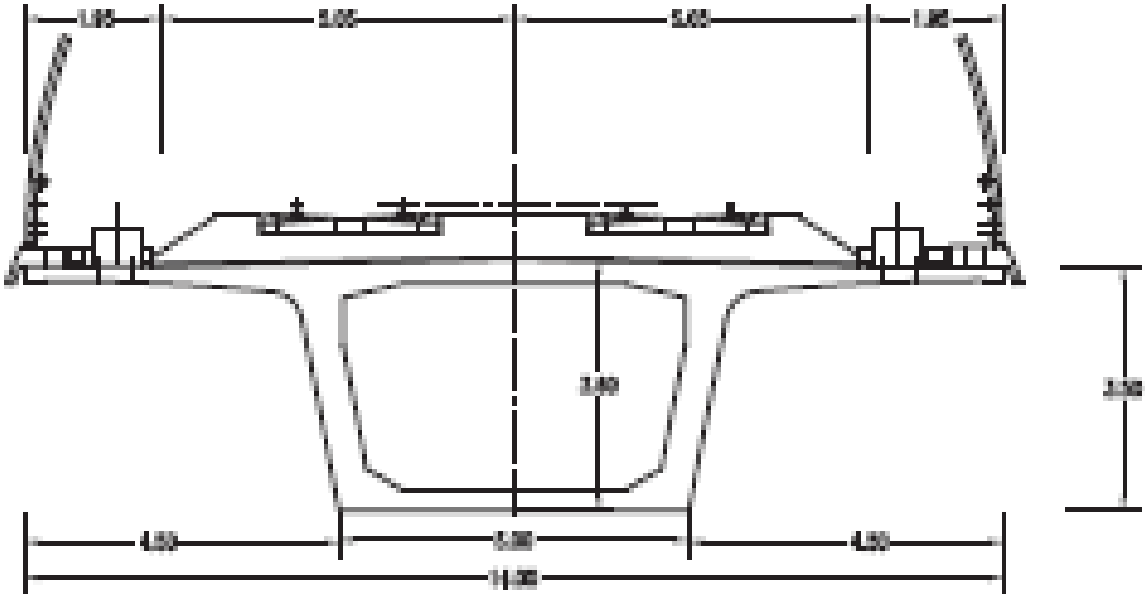


PLANTA DE COORDENADAS
ESCALA 1:150

PUNTO	X	Y	Z
1	0.000	0.000	0.000
2	0.000	23.000	0.000
3	−23.000	23.000	0.000
4	−23.000	0.000	0.000
5	0.000	0.000	2.500
6	0.000	23.000	2.500
7	−23.000	23.000	2.500
8	−23.000	0.000	2.500
9	−2.980	2.980	5.000
10	−2.980	17.020	5.000
11	−17.020	17.020	5.000
12	−17.020	2.980	5.000
13	−4.000	3.500	5.000
14	−4.000	16.500	5.000
15	−16.000	16.500	5.000
16	−16.000	3.500	5.000
17	−8.560	3.500	16.312
18	−8.560	16.500	16.312
19	−13.990	16.500	15.577
20	−13.990	3.500	15.577
21	−11.203	3.500	17.527
22	−11.203	16.500	17.527



PLANTA
ESCALA 1:150



SECCION TRANSVERSAL DE TABLERO

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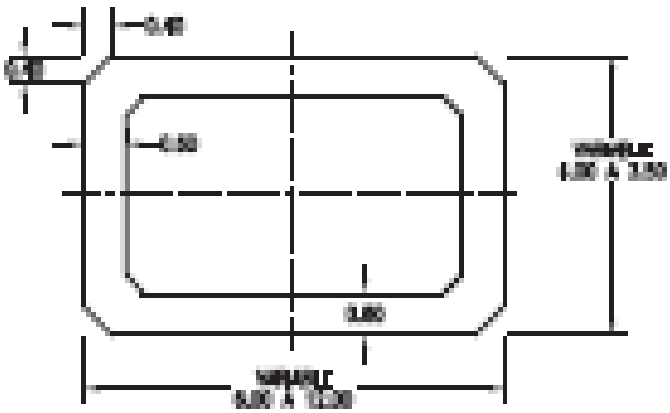
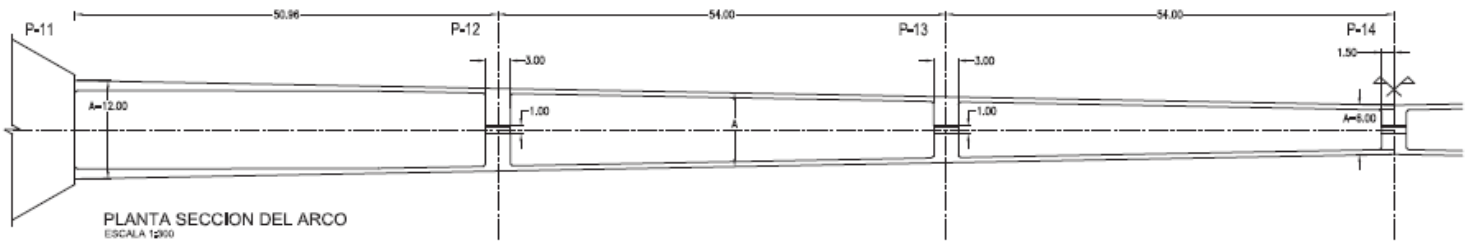
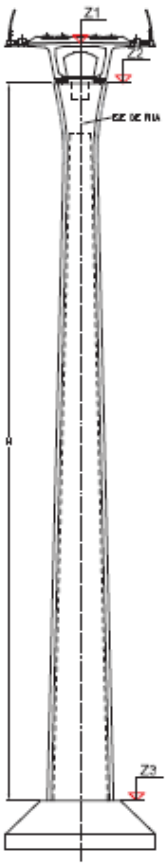


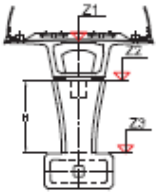


PLANTA SECCION DEL ARCO

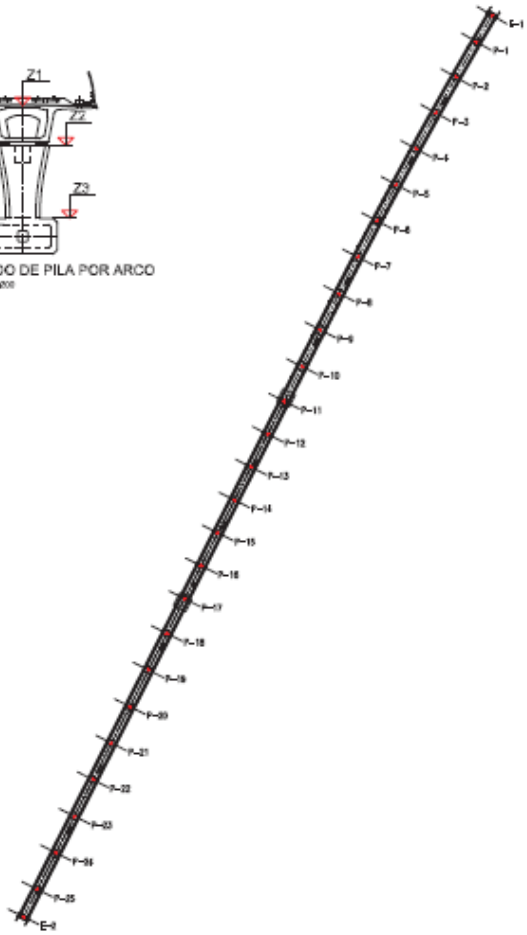
ESCALA 1:200



ALZADO DE PILA



ALZADO DE PILA POR ARCO



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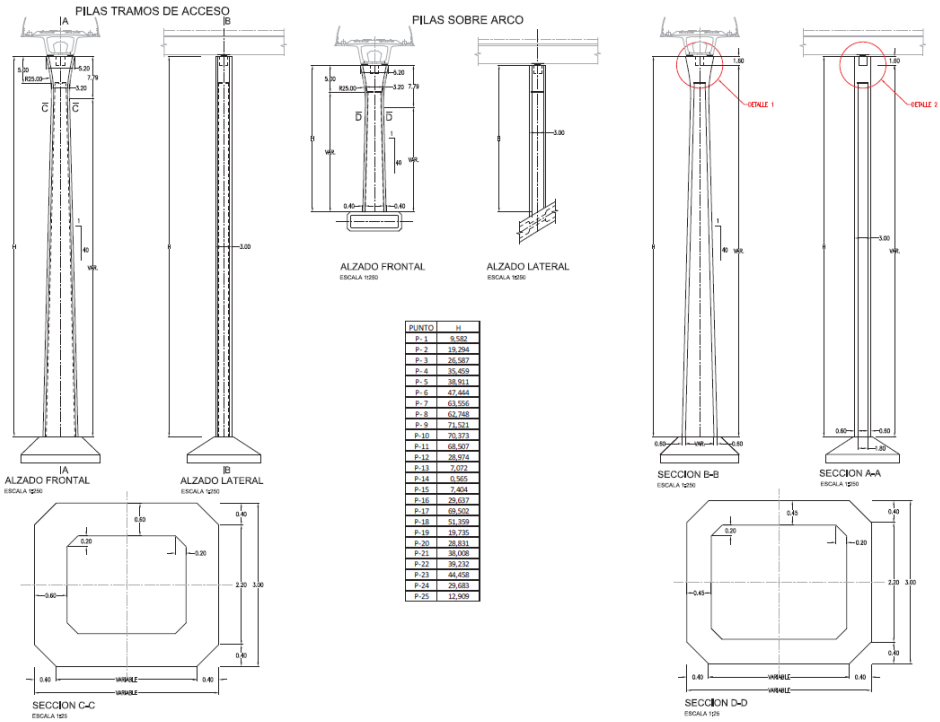

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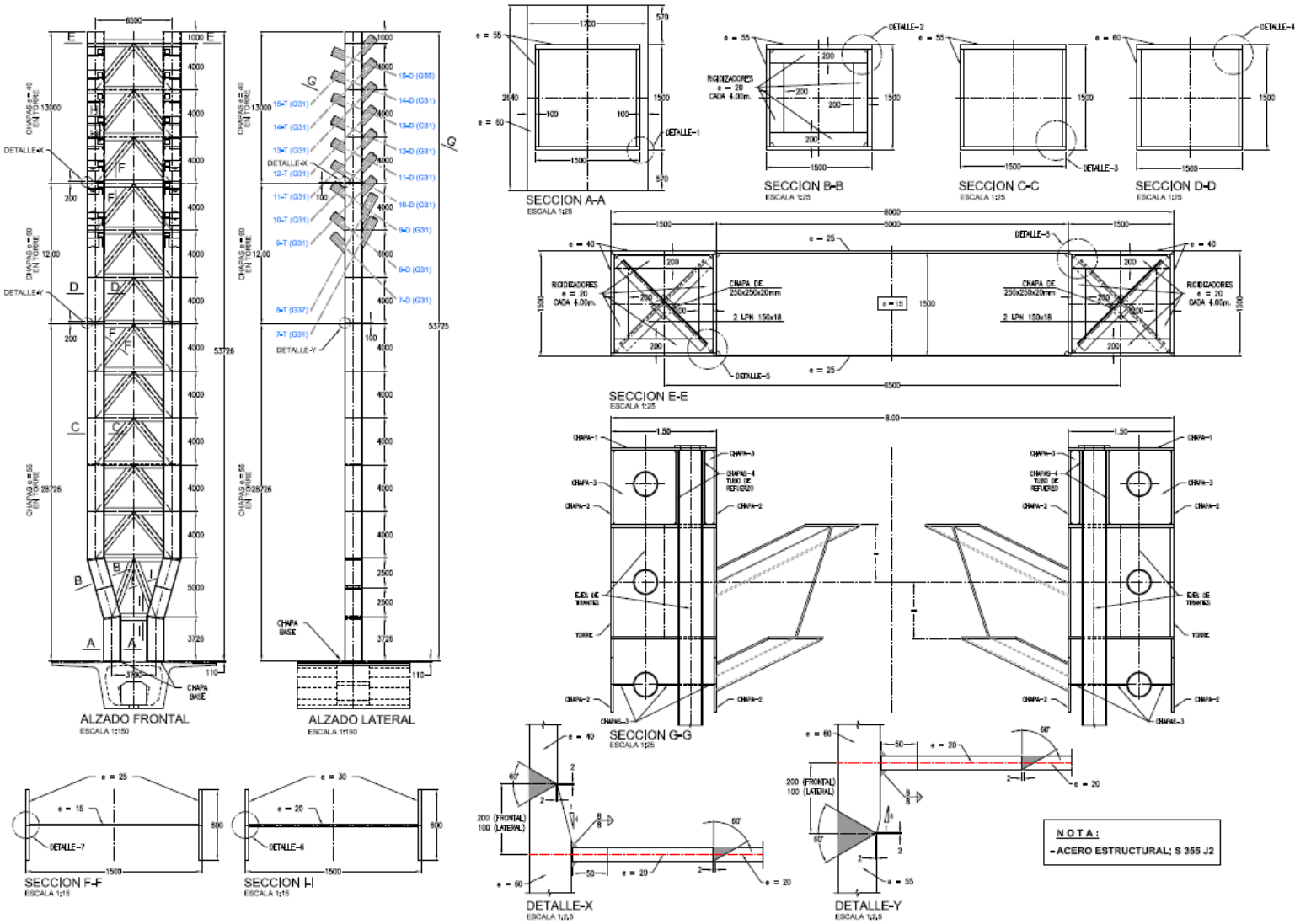
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SECCION C-C
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SECCION D-D
ESCALA 1:25
SECCION E-E
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SECCION G-G
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DETALLE-X
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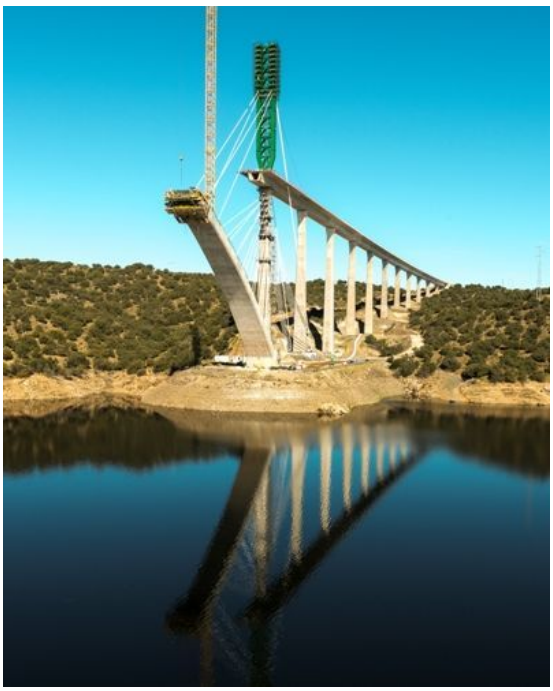
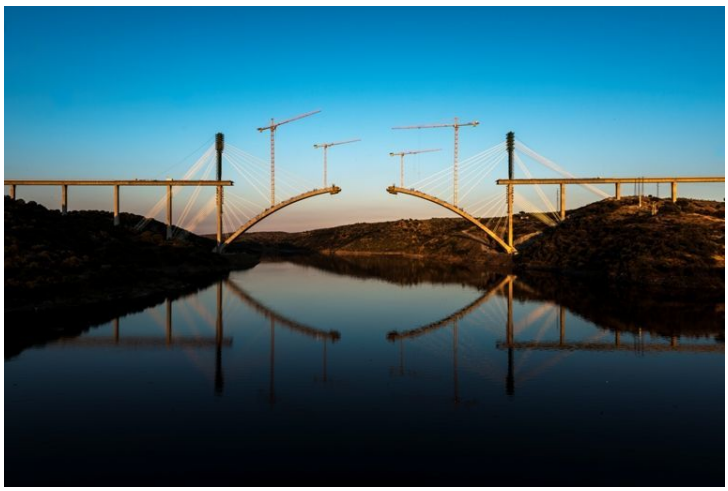
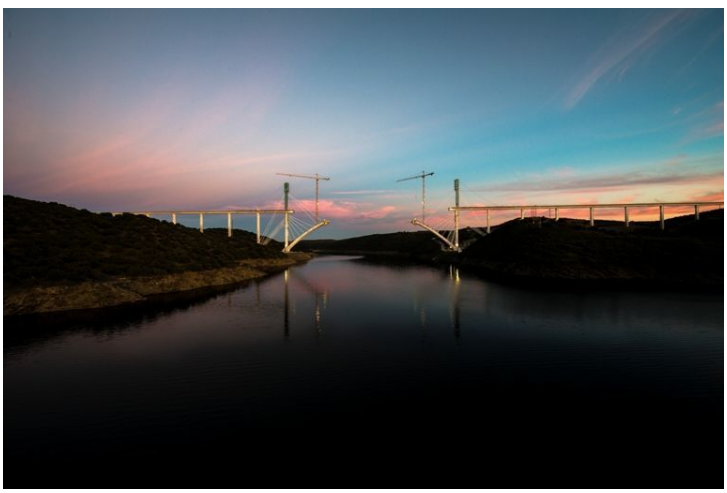
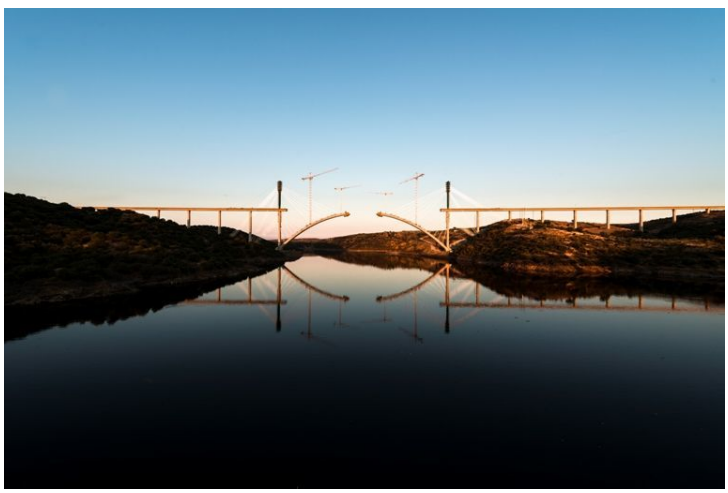
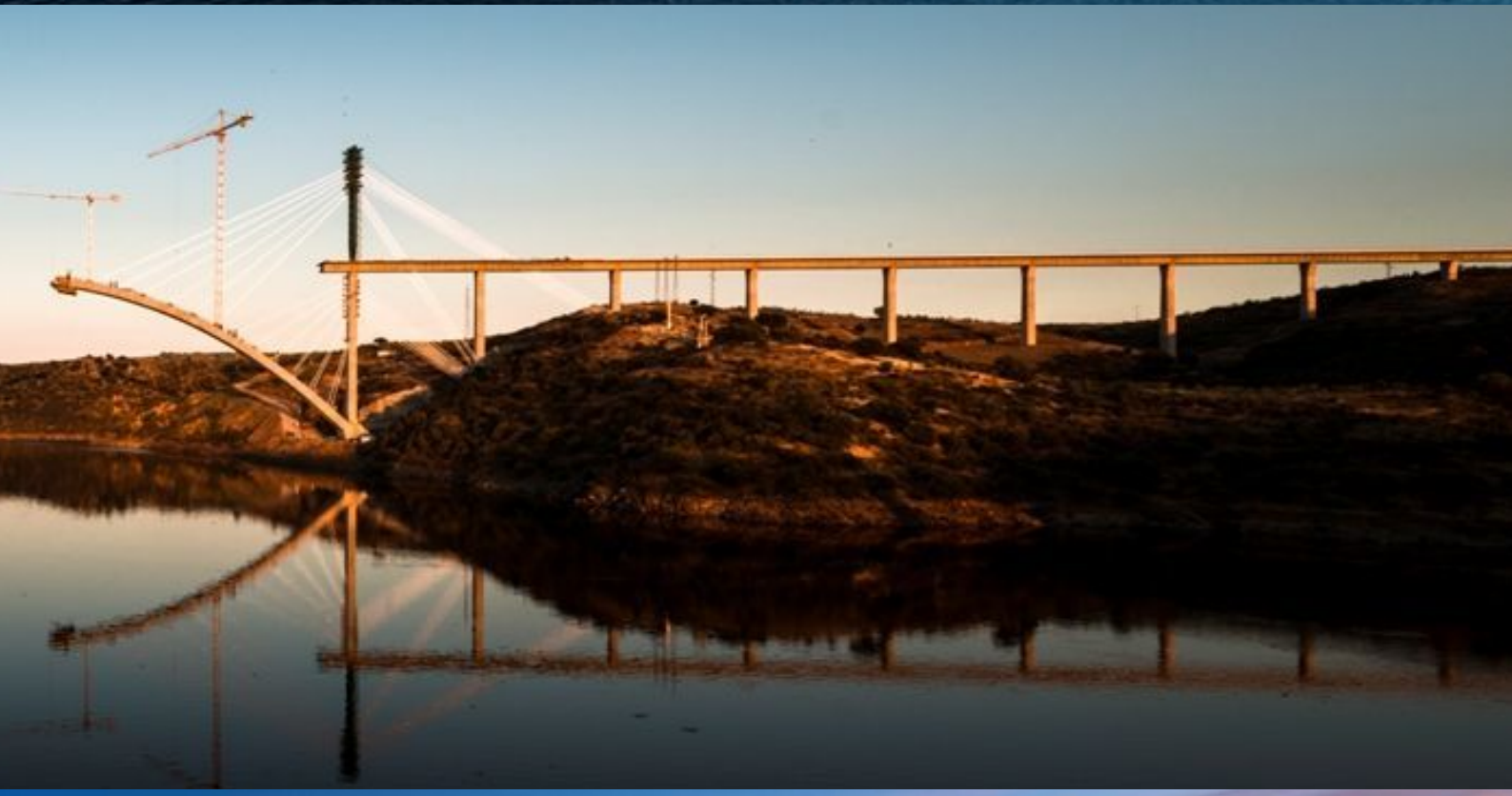
PHOTO GALLERY

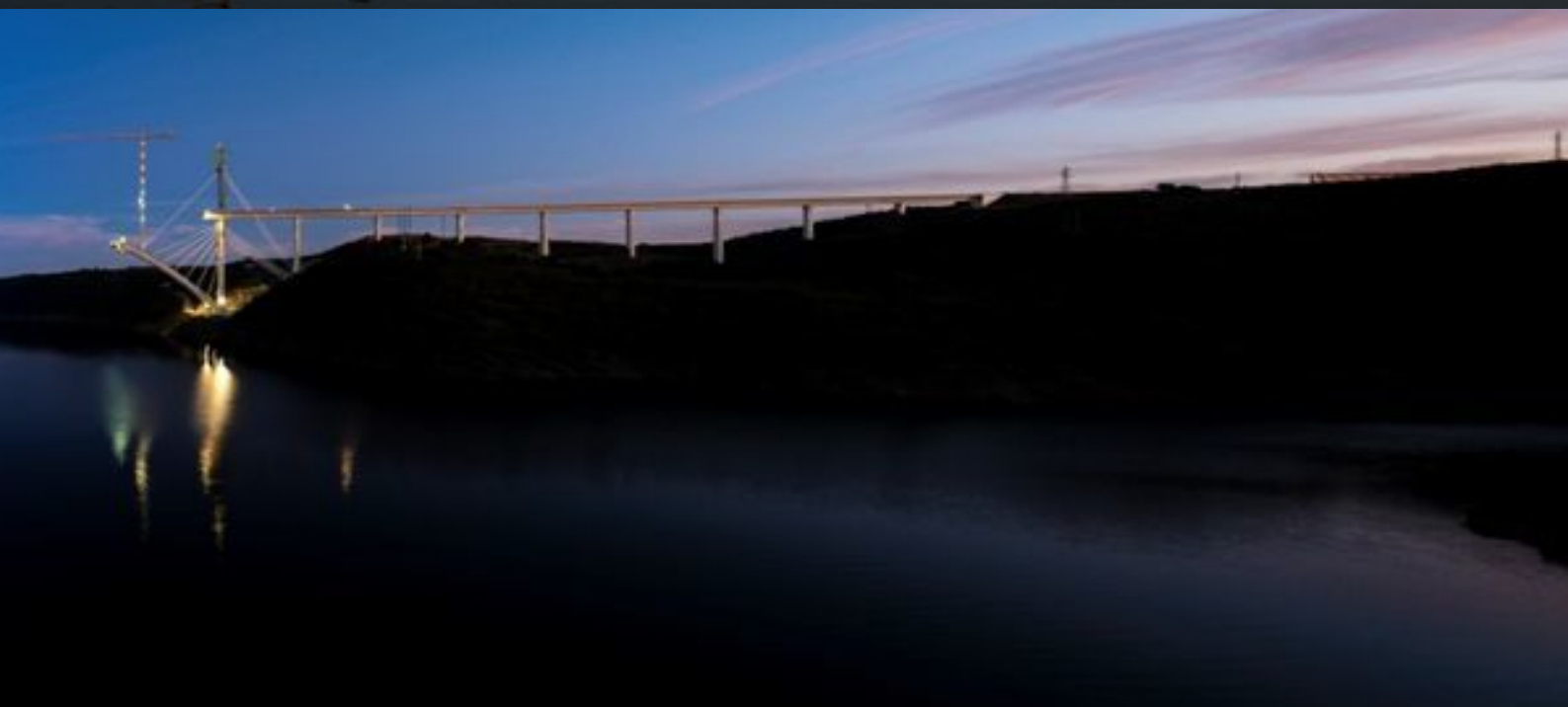












All photos in the photo gallery: Carlos Manterola for Carlos Fernandez Casado

THE RIVER IRWELL NETWORK ARCH BRIDGE

A 21ST CENTURY APPROACH TO CIVIL ENGINEERING INFRASTRUCTURE DELIVERY

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Commencement of works: 2016

Opening of the bridge to traffic: 2017

Type: Twich network railway arch bridge

Length: 89m

Rise:13.57m (max)

Width: 15.62m (max)

Location: crossing River Irwell in North of England as part of the Ordsall Chord – elevated railway viaduct

Promoter: Network Rail

Design: AECOM-Mott MacDonald JV

Contractor: Skanska Bam JV

Steel Fabricator: Severfield

Architect: BDP

SUMMARY

An 89m, twin network arch bridge was designed to span the River Irwell in the North of England. The bridge will carry a twin, bidirectional track arrangement as part of the Ordsall Chord intervention. A number of requirements and constraints were imposed, which posed challenges to both the design and construction teams. Effective use of BIM technologies and processes and the adoption of a collaborative approach by all parties involved, including the early appointment of the Steelwork Fabricator were catalysts to the successful completion of the design phase of the structure. The merits of the wider team collaboration can also be seen during the construction phase.

This is the first network arch railway bridge in the United Kingdom. It is also believed that the arrangement of asymmetric arches, open section longitudinal ties and type of loading constitute a world's first variation of a network arch bridge. The structure is currently under construction and due for completion in 2017.

1. Introduction

The River Irwell Bridge forms part of the Ordsall Chord, which is a new elevated railway viaduct currently under construction in the heart of Manchester in the North West of England. The Chord itself is part of the wider Northern Hub and Electrification Programme, which will improve the capacity of the Railway network and contribute significantly to the economic prosperity of the region. A visualisation of the Ordsall Chord is shown in Fig. 1.

The viaduct is approximately 300m long and connects two existing masonry arch viaducts, namely the Castlefield and Middlewood viaducts. It crosses a site with significant historic value to both cities of Manchester and Salford, which used to be the epicentre of the development of railways in the UK. Specifically, the Viaduct crosses Water Street, Stephenson's Viaduct, which is a Grade II listed Heritage structure, the River Irwell and Trinity Way, which is part of the Manchester and Salford Inner Relief Road. The River Irwell Bridge is a steel network twin arch, which is the first of this kind of bridge in the UK, spanning 89m across the River Irwell.

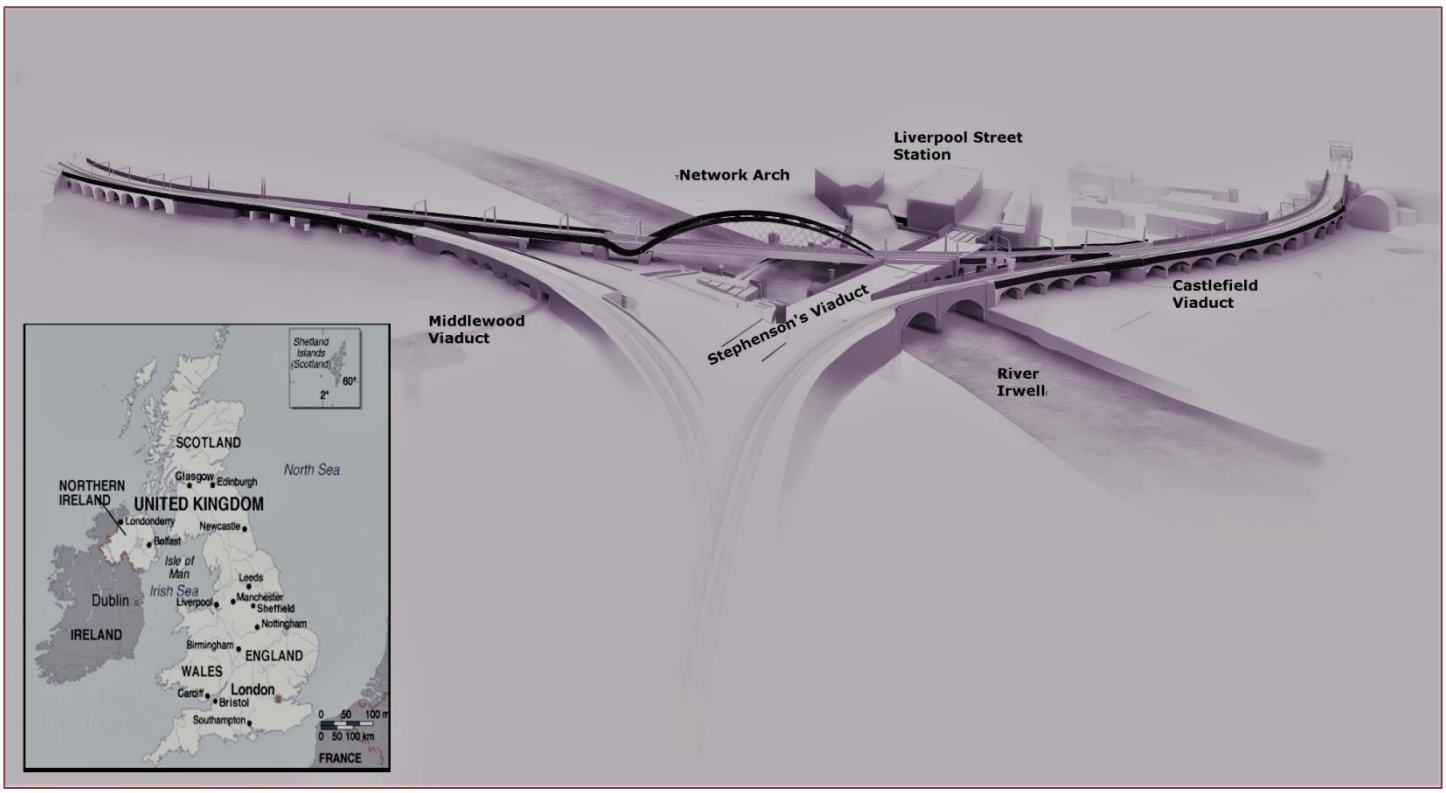


Fig. 1: Visualisation of the Ordsall Chord

The structure comprises a system of two inclined, braced network arches supporting two tracks via a steel and concrete composite deck with transverse members supported from the longitudinal steel tie beams.

A number of requirements and constraints were stipulated by the project's Stakeholders that lead into a number of design and construction challenges. These challenges were resolved within the spirit of collaboration, which was promoted at the outset of this project by all parties involved. The solutions to the aforementioned challenges utilized cutting edge technology, not only from the engineering, but also from the project delivery and interface management perspectives. The utilization of Building Information Modelling (BIM) techniques was essential for the design, steelwork fabrication and construction phases. The general arrangement and typical sections of the superstructure are shown on the following pages.

2. The Challenges

The existing Prince's Bridge was being used as a pedestrian crossing of the river. That structure was on the route of the new viaduct and it had to be demolished. The existing public Right of Way will be substituted by a new footbridge, which has been installed in advance of the new network arch bridge. One of the biggest challenges was working in a spatially constrained area and this is illustrated in Figure 2.

Due to the site constraints the bridge had to be constructed "piecemeal", and this is discussed in more detail later.

It is also the aspiration of both the cities of Manchester and Salford to regenerate the area. As the new viaduct is at an elevation, the need for a landmark structure, which will mark this regeneration, was apparent. However, the new viaduct, as a whole

Fig. 2: Site overview

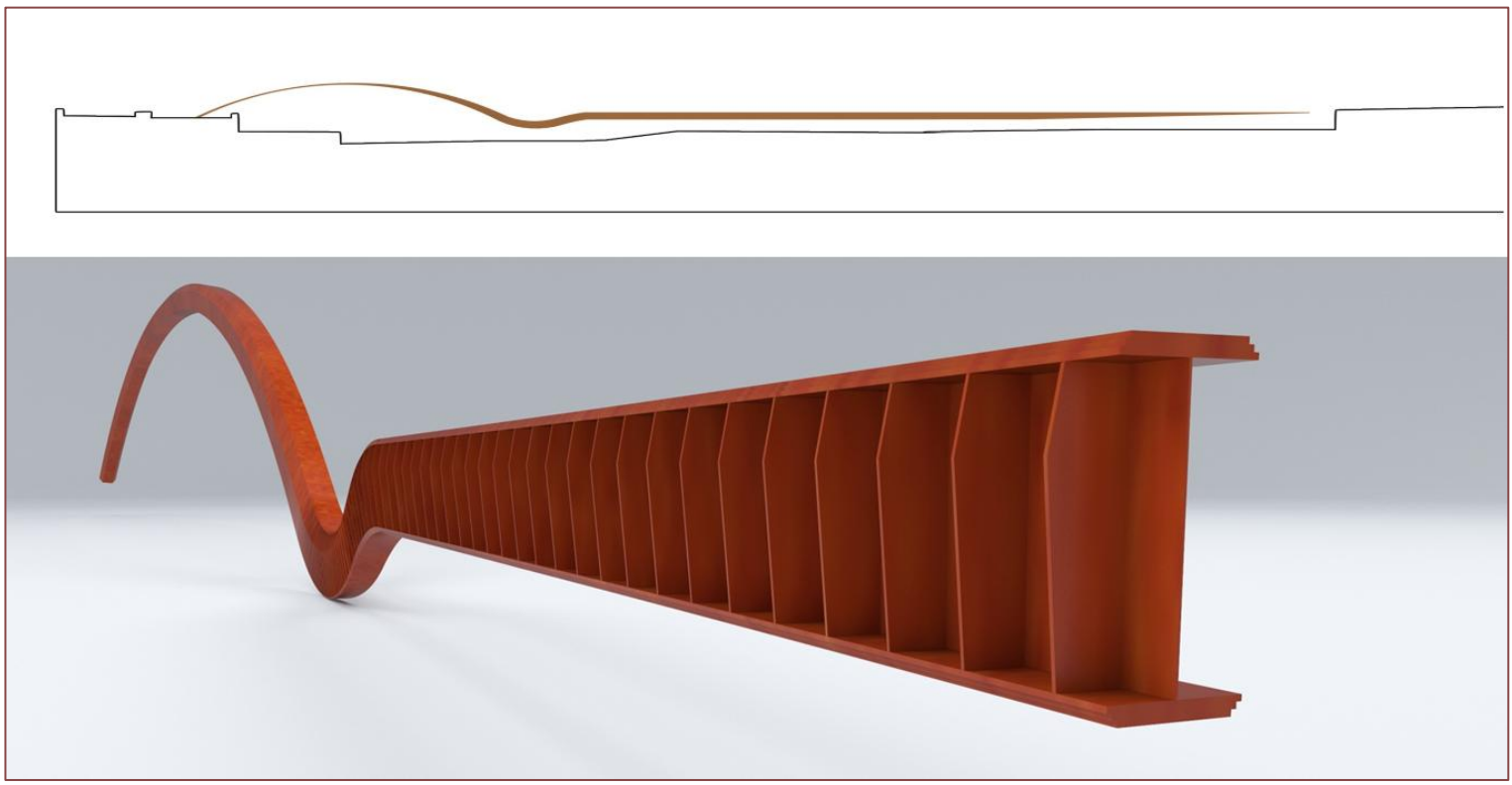


Fig. 3: The Ordsall Chord ribbon

entity, had to be designed to respect the significant heritage of the site. Following extensive public consultations, the aesthetic requirement for a slim and elegant but emblematic structural line has led to the choice of the architectural signature of the Ordsall Chord, a ribbon in weathering steel, connecting and unifying visually the adjoining structures, as shown in Figure 3.

The requirement for visual continuity between the network arch and adjacent parts of the viaduct influenced the shape of the structure. It should be noted that in plan, the viaduct is not on a straight line but is slightly curved at this location. A series of parametric studies using BIM technologies were performed, in order to establish a harmonic relationship between the curved and the straight lines of the ribbon in both plan and elevation. This had a significant influence on the height, shape and inclination of the arch. Too tall an arch would benefit its stiffness but affect the harmony with the adjacent straight line. Too shallow, it would benefit the aforementioned harmony but would not suit a railway bridge, whose stiffness is profoundly important for the safe operation of the railway itself.

Network Rail, as the Promoter, sought a structure, which was robust and straightforward to maintain, but these aspirations had to be balanced with the need for an architecturally very high quality structure at a visually sensitive location. Figure 3 also illustrates a typical arch rib together with the main girder of Trinity Way Bridge and the link structure, and this is the result of the aforementioned parametric studies.

It is worth noting that the behaviour of the structure with open and box sections was also studied. Open sections offer easy access for inspection compared to boxes. However, in order to achieve the targeted appearance a great amount of additional stiffening was required. This additional stiffening would increase the inspection and maintenance effort significantly and deemed this solution impractical. Finally, the project team decided that the main arch and bracings would comprise box sections, with the remainder of the steel structure being fabricated from open steel sections.

The choice of steel type was also important, as it affects both the aesthetics and the maintenance requirements. Weathering steel was chosen for the main arches and bracings and painted steel for the longitudinal ties and transverse girders. The choice of weathering steel, generally, minimises the whole life maintenance costs. However, a separate study was performed to ascertain, how any graffiti could be

successfully removed from the weathering steel without damage to the stabilising patina.

The rise to span ratio was set at 0.1525, which is at the lower range for a network arch. In order to enhance the appearance of the arches a 6 degree inward inclination was prescribed at planning stage. The solution was a good compromise between aesthetic and maintenance requirements and the collaboration between the Planning Authorities, the Architect, the Promoter, the Contractor, the Bridge Designer and the Steelwork Fabricator was instrumental in achieving the final structural configuration.

3. The Erection Sequence

It was recognised from the very early stages of the project that the design of this structure was heavily dependent on the construction methodology to be adopted. The methodology was heavily influenced by the requirement to have a navigable channel in the river for the duration of the construction. Furthermore, in order to reduce the impact of the temporary works on the construction programme, it was decided to use the same temporary works to facilitate the demolition of Prince's Bridge. This effectively led to a "piecemeal" erection approach and necessitated in depth discussions between the Steelwork Fabricator, the Bridge Designer, and the Contractor, in order to determine the exact methodology, from which the design of the structure evolved.

The tie beams will be erected first in sections followed by the transverse girders to form a ladder deck. The alignment of the bridge with respect to the river means that the deck will be installed on skewed temporary towers in the river.

When the tie beam sections are installed, they will deflect and twist as they are asymmetric and the centre of gravity and the shear centre are not coincident. In order to understand the behaviour of the asymmetric tie beam sections better, the deck erection was re-analysed using a shell-element based model for the ladder deck.

Such an analysis was required to evaluate the stability of the tie beam sections in the temporary case since the cross section does not have an axis of symmetry. It was also required, in order to understand the build-up of rotations about the longitudinal axis of the ties. The open section has relatively low torsional stiffness, which meant that the twist generated during erection may lead to problems during the installation of the hangers. This risk was mitigated by incorporating transverse props and ties that assist in the installation

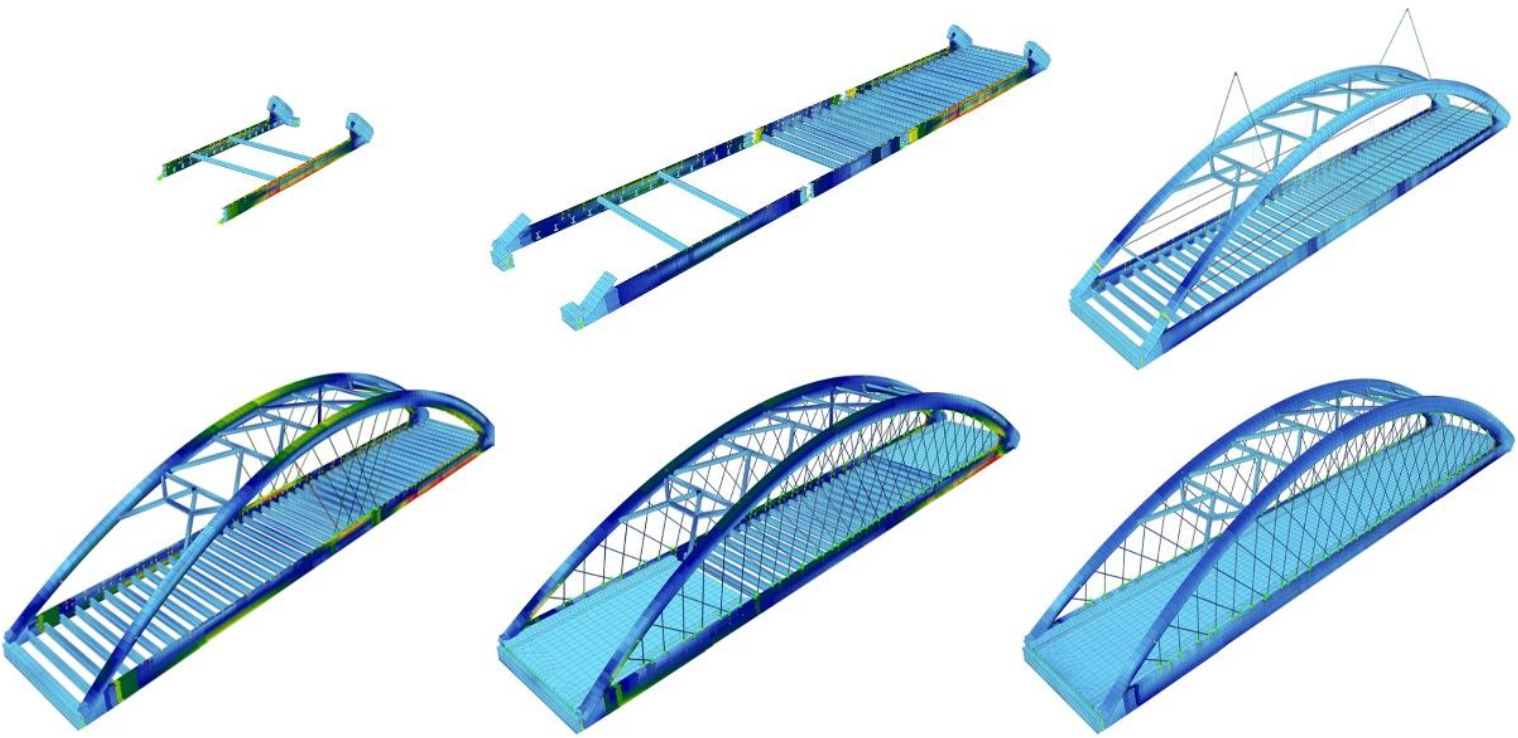


Fig. 4: Construction sequence

of the transverse girders. Grade 460 high strength steel was adopted for the clevis plates and hemi-spherical bearings were also incorporated to enhance the out-of-plane rotational capacity, thus providing greater installation tolerance. The installed north bank arch nodes, the ties and the temporary props are shown in Figure 5.

At the same time the ladder deck is assembled on the temporary supports, the arches and bracing will be assembled on the south river bank. Following completion of the installation of the ladder deck steelwork, the arches and the plan bracing will be installed via a tandem lift by two large cranes from the river bank. The arches will be offered to the end nodes in a prescribed sequence. Temporary tie cables will be used to control the shape of the arches to facilitate accurate placement. Once the site joints are welded, the hangers will be installed and stressed in a prescribed sequence.

The "piecemeal" erection methodology induces bending moments in the tie beams that are generally avoided in network arches. In order to minimise the magnitude of these moments, the stressing of the hangers will commence as soon as the structural steelwork is erected, before the installation of the permanent formwork and deck.

This first phase of hanger stressing will relieve the temporary works and it will transfer the gravity loads in the arches and the tie beams. The hanger installation and stressing pattern is aimed at avoiding "slack" hangers in working conditions. The stressing will commence from the centre outwards so that the temporary towers are relieved after the first few stressing operations.

Once the first phase of stressing is complete the concrete deck will be constructed in sections. Following the initial cure of the deck slab a second phase of hanger force tuning will be applied so that the target in-service performance of the hanger network is achieved.

It is worth noting that the construction sequence was refined on a number of occasions to suit various site parameters and this has led to extensive discussions between the designers, the fabricator and the main contractor and their cooperation was key in the successful and practical step by step construction methodology.

Fig. 5: Current construction stage

4. The hanger network

Each network is formed of 46 proprietary tension assemblies, comprising solid steel bars and cast fork anchorages. The layout of the hanger network is generally based on the theoretical concept of directing the resultant network forces radially with respect to the arch axis. This is achieved by aligning the intersections of the hangers radially towards a common focal point.

The layout of the hanger network was further developed through a series of studies investigating its efficiency in terms of maximum and minimum hanger forces and the resulting bending moment profiles in the arches and the tie beams.

In order to facilitate the replacement of hangers and their stressing during construction, each network was split into two parallel planes. The theoretically derived layout had to be adjusted to provide sufficient space for the development of the anchorages and, in the case of the short hangers, to allow the installation of the stressing equipment. The resulting layout is shown on the general arrangement drawing.

5. Arch geometry

The main arches have a continuously varying cross section that forms a “crease” line, which is visually continuous from the tip of the arch to the end of the approach viaduct. The shape of the hexagonal cross section for the arch was carefully designed so that out-of-plane curvature, or “warping”, of the plates is avoided.

This was achieved by maintaining constant inclination of the upper and lower plates with respect to the true vertical plane perpendicular to the axis of the bridge. The cross section definition is also shown on the typical section drawing.

The “ribbon” theme for the Chord resulted in a very deep arch cross section towards the North end of the bridge. The material distribution along the arch compensates for the increase of section by reducing the plate thickness. This reduction was balanced against the requirement for introduction of longitudinal stiffeners since the latter in combination with the hanger anchorages would lead to congestion within the box.

6. Deck geometry

The aspiration for ease of maintenance was most influential in the development of the deck geometry. Although a closed box would have been a more convenient structural form, an open section geometry was adopted so that touching distance inspection could be undertaken for all deck steelwork not encased in concrete, without the need for confined space inspections. In order to avoid unnecessary eccentric effects, the arch and tie axes intersect over the centreline of the abutment bearings. In order to improve the load path from the hangers into the tie beam, the web was aligned with the outer plane of the hanger network. The bottom flange of the section was maintained truly horizontal to facilitate site installation.

The deck was also designed to incorporate the Promoter’s requirement for a connection that provides a secondary load path for the vertical loads and this is formed by bolted shear key end plate connections for the transverse girders. These were positioned within the footprint of the tie beam bottom flange so that the visual aspiration for “clean” soffit is achieved.

The prominence of the exposed arch surfaces means that their interruption is undesirable since the visual continuity will be compromised. With that in mind the junction between the arch and the tie beam was a defining interface with respect to the cross section geometry. This interface is shown for the North End node in Figure 5.

7. Conclusions

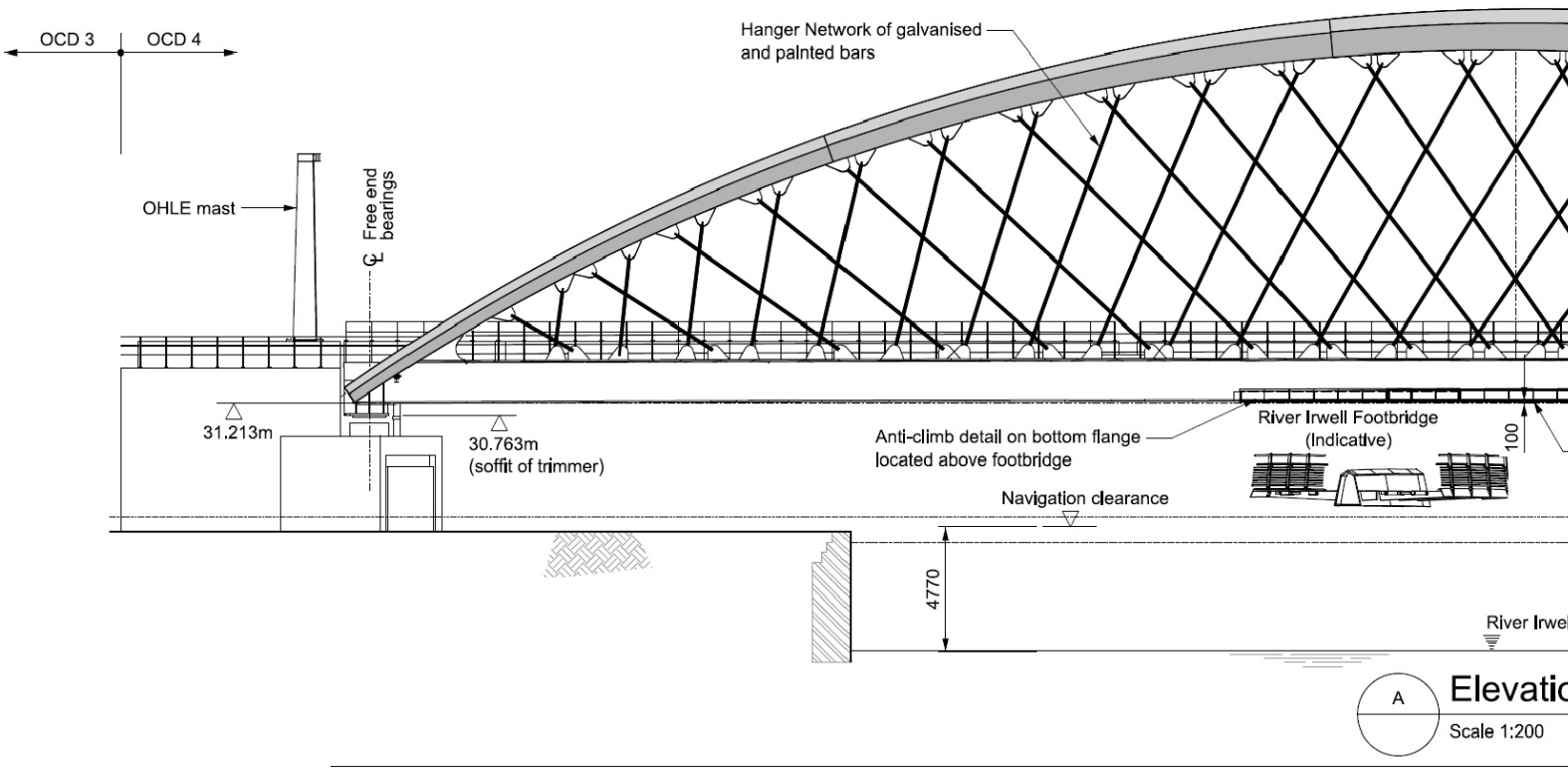
The River Irwell Bridge is the first railway network arch bridge in the UK. At the time of writing, the structure is in the construction phase and is due to be completed in summer 2017.

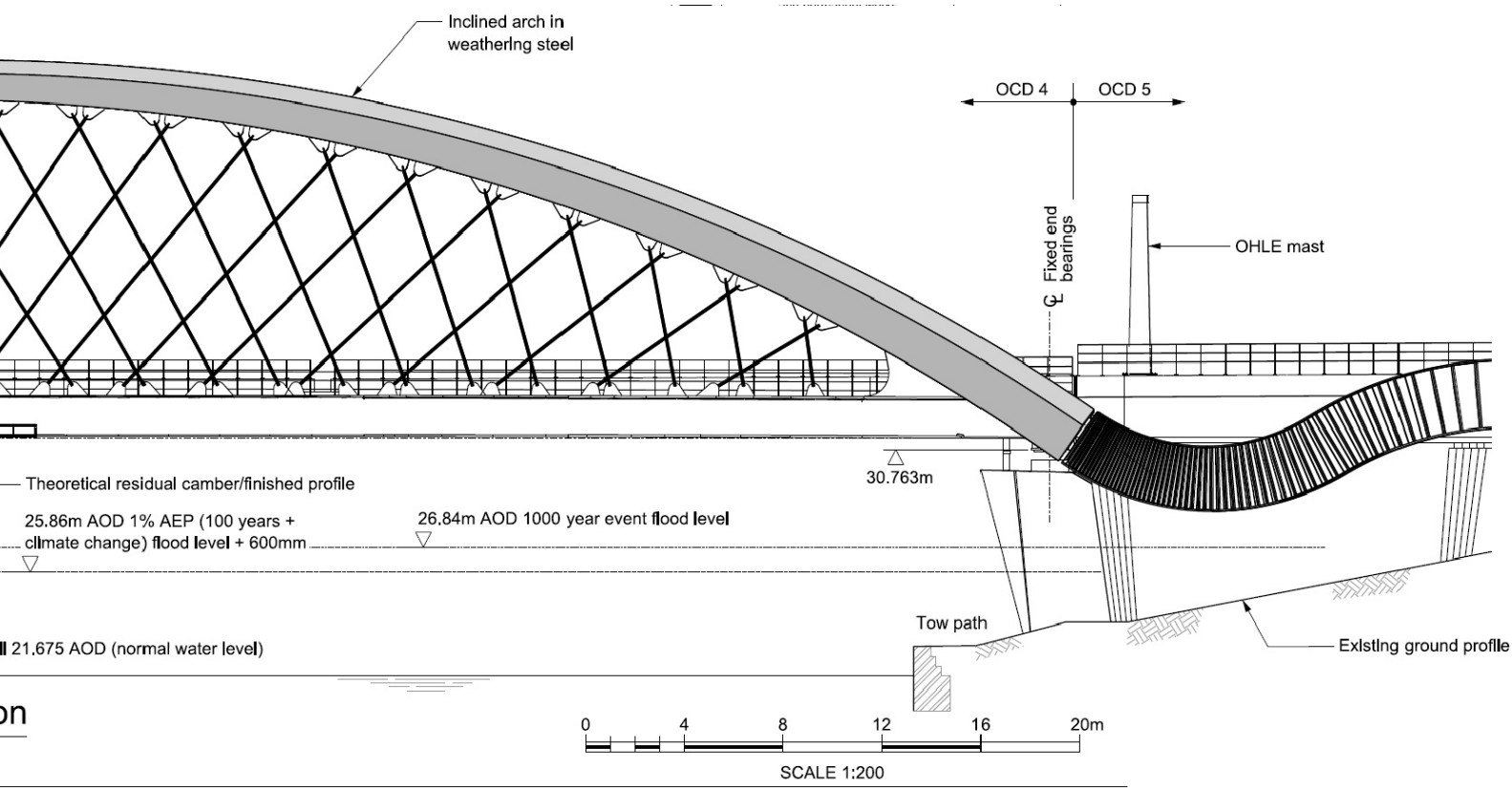
Its complex configuration and the construction sequence adopted, which was tailored to the particular site constraints, presented a number of challenges to both the design and construction phases. However, the collaboration and ingeniuity of all the professionals involved has been key to the delivery of this structure.

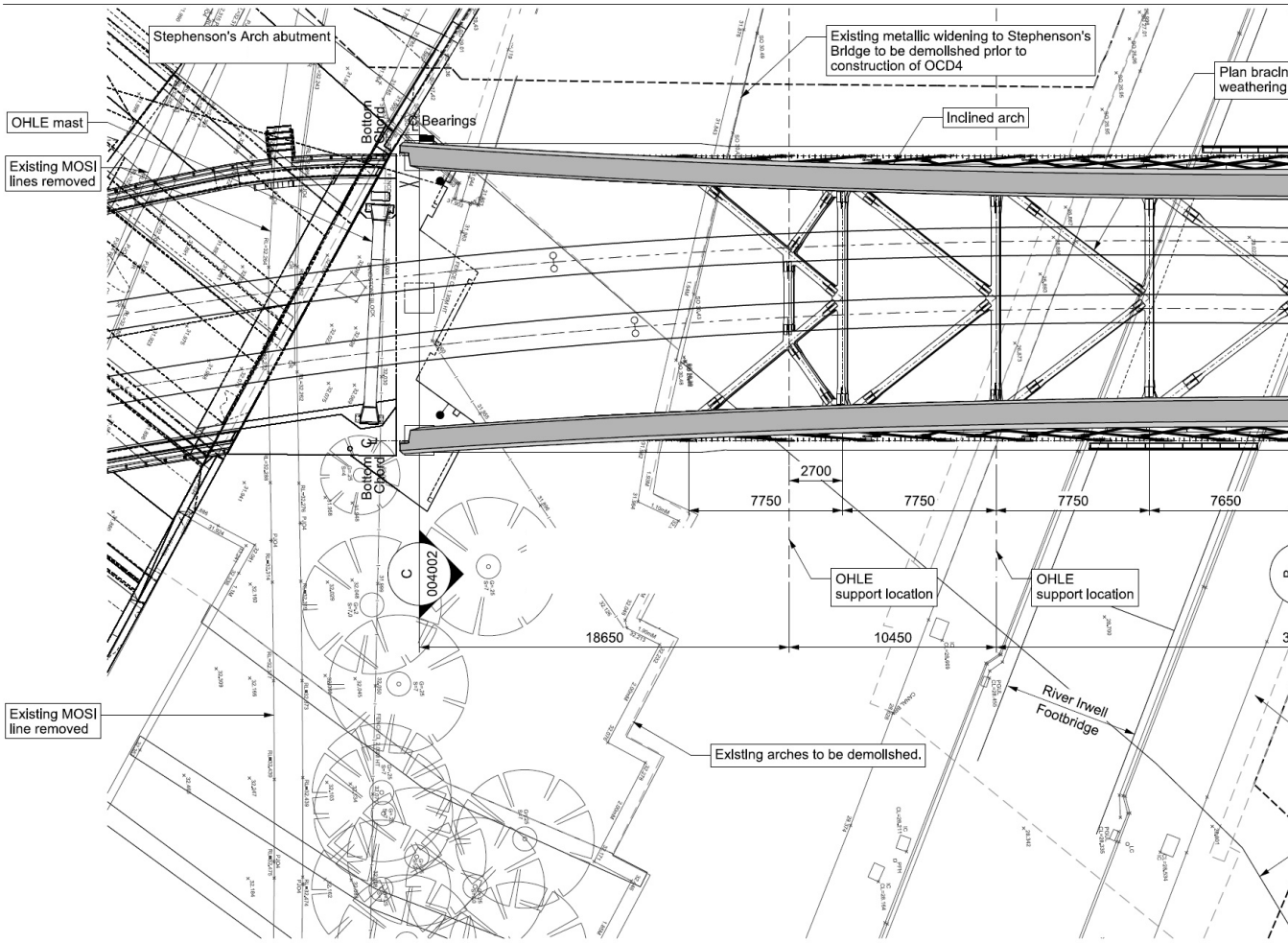
8. Acknowledgements

The authors would like to express their gratitude to the wider team of professionals from Network Rail (Promoter), Skanska Bam JV (Contractor), BDP (Architect), Severfield (Steelwork Fabricator) and the AECOM-Mott MacDonald JV (Designer), who collaborated closely to deliver this landmark structure.

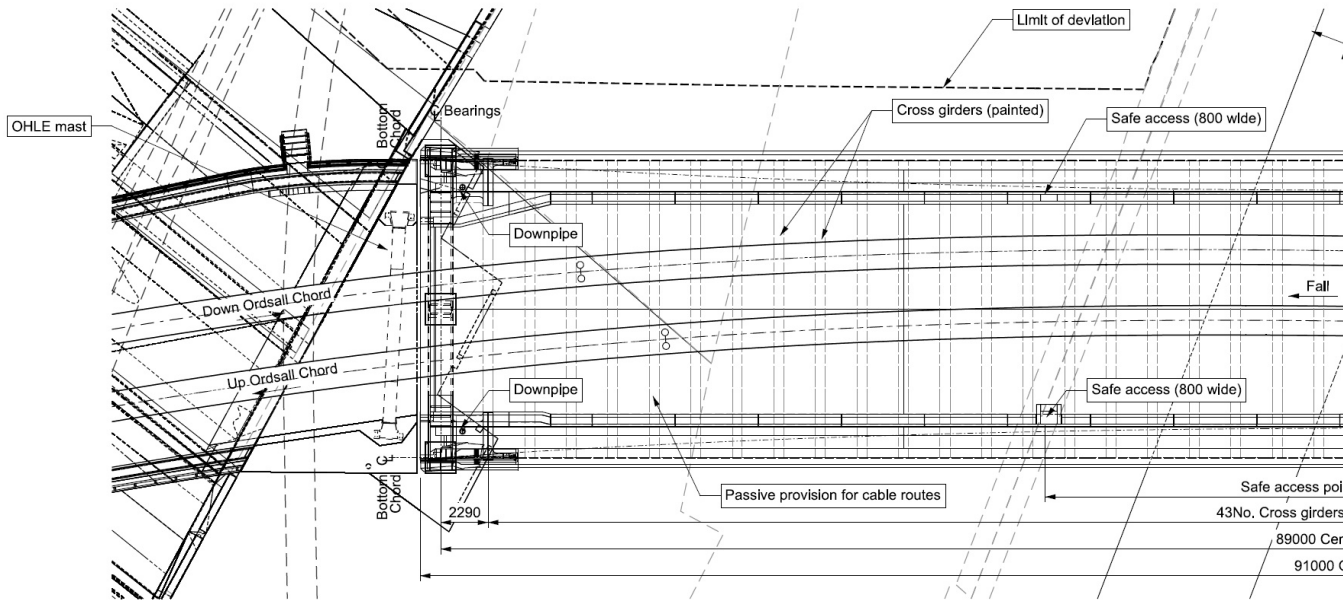
GENERAL ARRANGEMENT AND TYPICAL SECTIONS



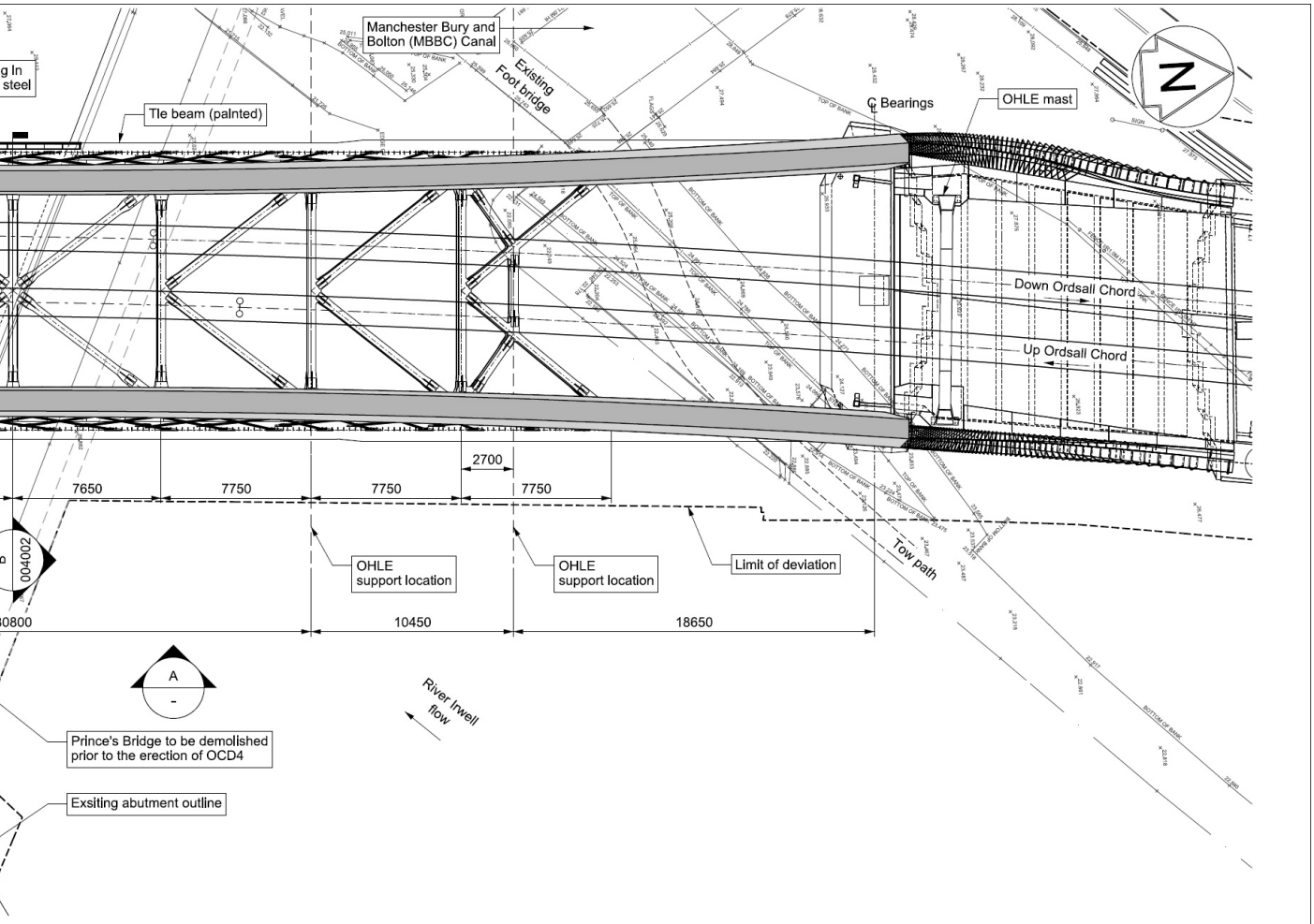

Inclined arch in weathering steel
OCD 4
OCD 5
Fixed end bearings
OHLE mast
30.763m
Theoretical residual camber/finished profile
25.86m AOD 1% AEP (100 years + climate change) flood level + 600mm
26.84m AOD 1000 year event flood level
Tow path
21.675 AOD (normal water level)
Existing ground profile
0
4
8
12
16
20m
SCALE 1:200



Pla
Scale



Plan At D
Scale



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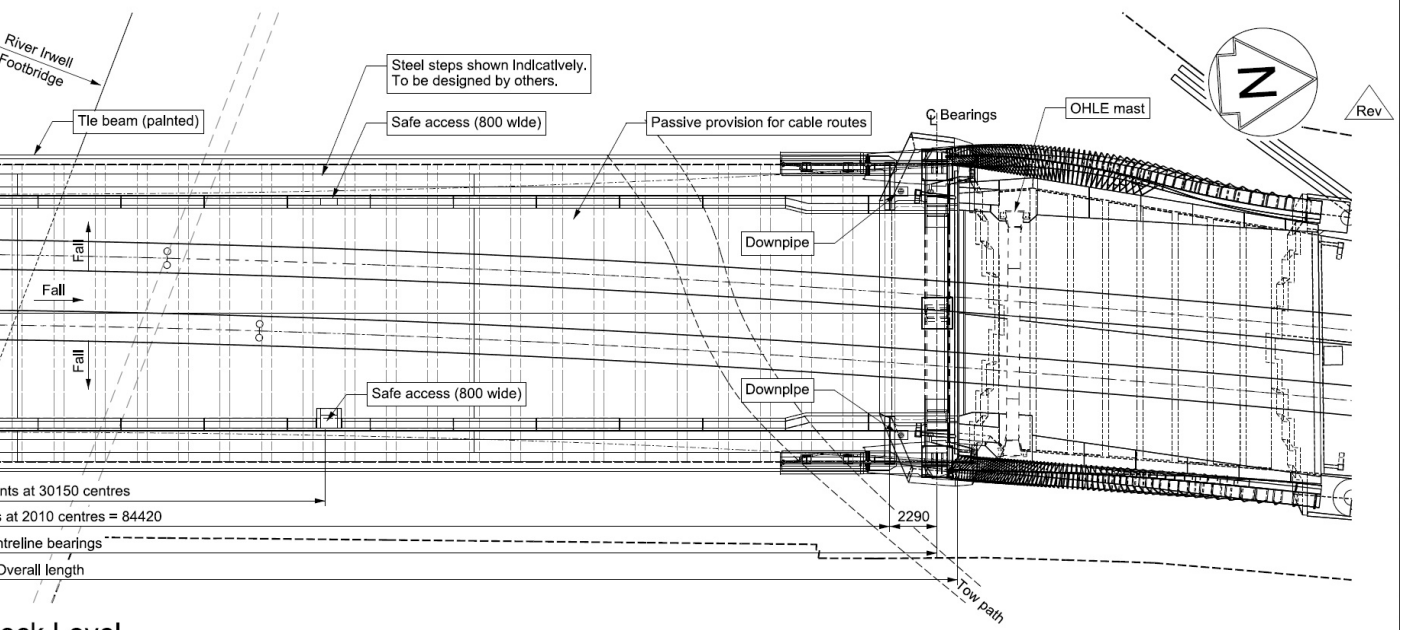

eck Level
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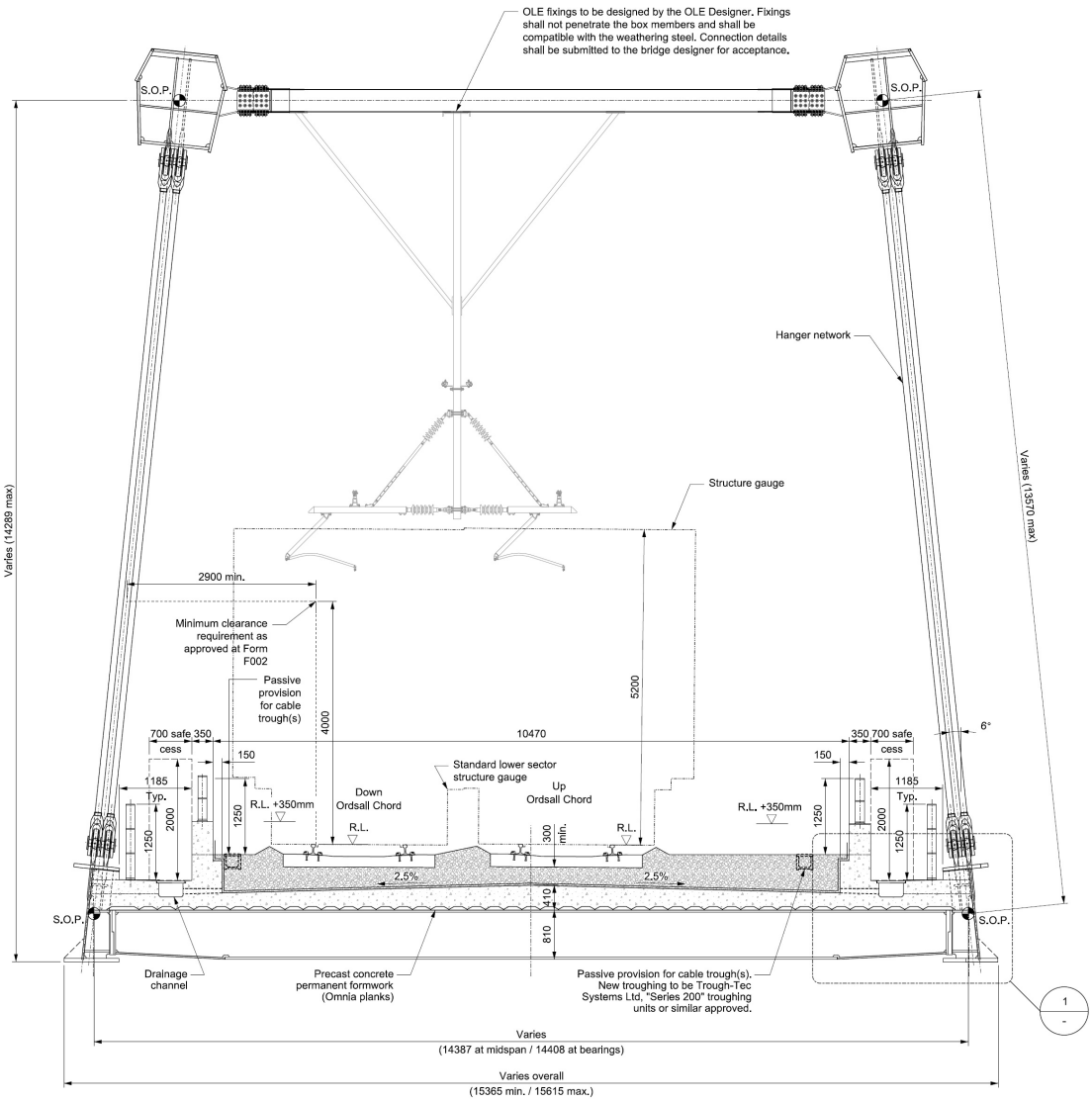
⚠	Unforseen loading situations due to lack of intergration between temporary and permanent works.	CDM-OC4-008



B 004001 **Typical Section**
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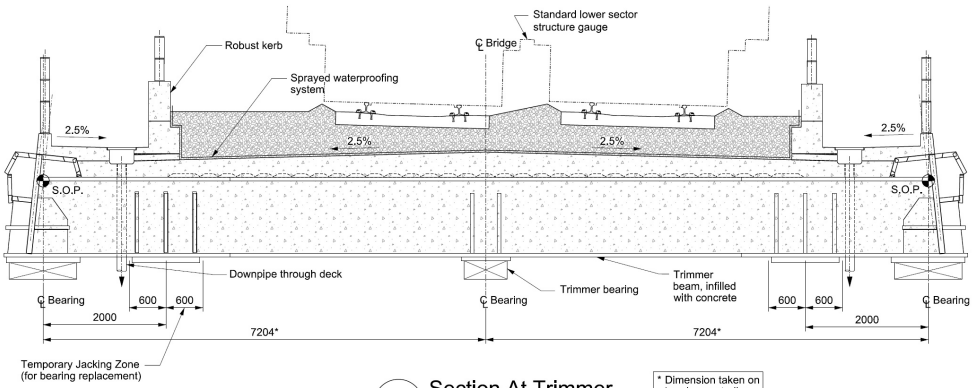

C 004001 **Section At Trimmer**
Scale 1:50

* Dimension taken on bearing centreline

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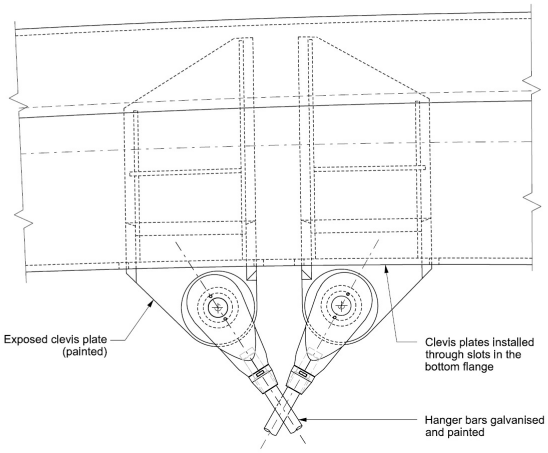
0 0.4 0.8 1.2 1.6 2.0m — SCALE 1:20
0 200 400 600 800 1000mm — SCALE 1:10



Arch Details
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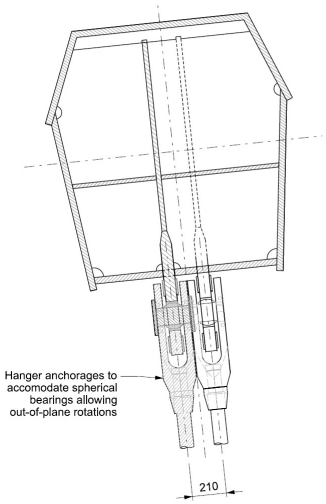

Arch Details
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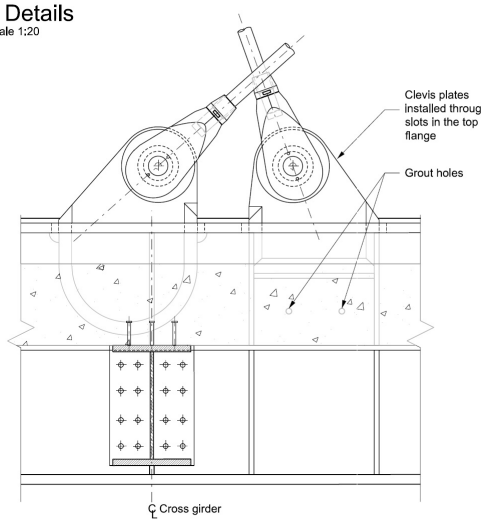

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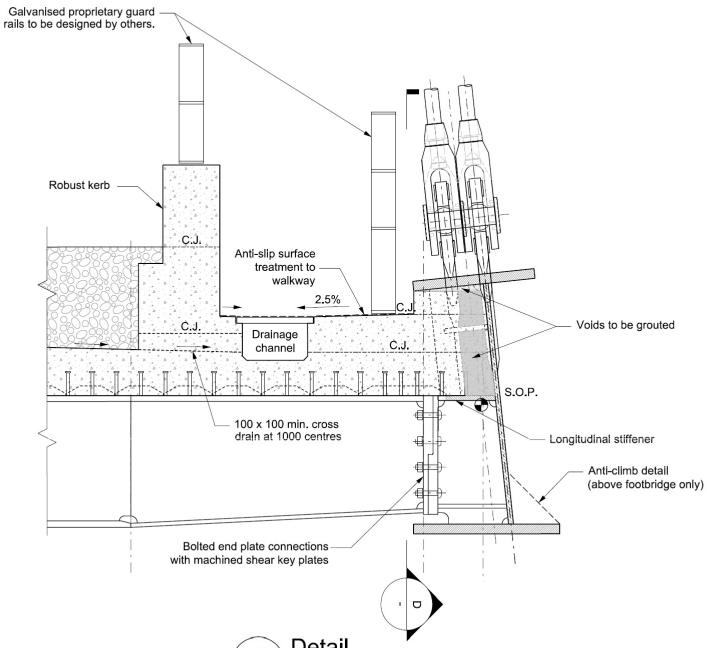
Tie beam details
Scale 1:20



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Detail
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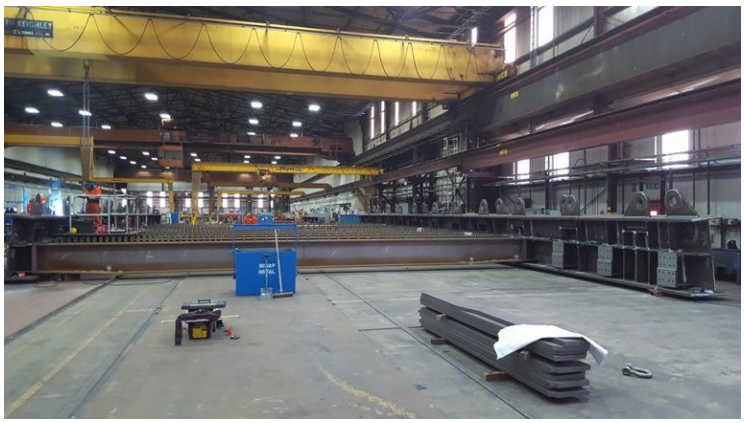
PHOTO GALLERY

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8th International Conference on Arch Bridges

Arch Bridge over the Vistula River in Toruń

General information

The 8th International Conference on Arch Bridges (ARCH 2016) was organized by Wrocław University of Technology on 5th – 7th October 2016 with the general topic: „Arch Bridges in Culture“, following previous seven ARCH Conferences (UK 1995, Italy 1998, France 2001, Spain 2004, Portugal 2007, China 2010 and Croatia 2013).

More information on:

http://arch16.pwr.edu.pl/#/general

Aim of the ARCH conference

The main idea of the cyclic ARCH conference is the international meeting of scientists, experts, designers, contractors and all those who are interested in problems of arch bridge structures aimed at effective exchange of experiences and dissemination of specialist knowledge and information in this field. Considered topics are devoted to a wide range of problems of arch structures, including not only technical issues, but also their presence in common life and in culture. Covered subjects are related to various structures: from historical ones, through those designed and constructed contemporarily, up to the latest and future solutions and concepts.

The conference took place in the Congress Centre of Wrocław University of Technology

Committees:

Prof. Jan Bień

Chairman of the Scientific Committee

Prof. Jan Biliszczuk

Chairman of the Organizing Committee

Paweł Hawryszków, PhD

Co-Chairman of the Organizing Committee

Tomasz Kamiński, PhD

Co-Chairman of the Organizing Committee

More information on:

http://arch16.pwr.edu.pl/#/committees

Conference topics

- Arch bridges in culture and national heritage,
- Historical arch bridges and construction techniques,
- Theoretical analysis of arch structures,
- Experimental studies of arch structures,
- Assessment, maintenance & exploitation of arch bridges,
- Repair, strengthening & reconstruction of old arch structures,
- New materials and techniques applied in construction of arch bridges.

Our magazine „e-mosty“ was a medial patron of the conference

Almonte River Viaduct being presented by Mr David Arribas from FCC Construccion

The River Irwell Network Arch Bridge being presented by Mr Rusi Rusev and Mr Thanos Bistolas from Mott MacDonald

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CZECH LAW FIRM

KGS LEGAL

After you enter the Czech market, our company KGS legal offers you in connection with your activities the following services:

- Legal counselling in the field of corporate law
- Negotiation and drafting of contracts
- Public procurement counselling
- Labour and employment law counselling
- Representation in court and other state bodies

KGS legal provides assistance and guidance to foreign entities and individuals wishing to enter Czech market regardless whether they are a new company or a subsidiary having its parent company abroad.

We offer our clients analysis of certain business area of the Czech market, advisory services relating to entering Czech market in the relevant area. In this regard, we provide our clients with support in legal and commercial matters relating to their business activities, as a secondary activity, we are able to assist them with accounting and tax issues.

KGS legal specializes mainly in acquisitions of small and medium-sized enterprises and investment groups doing their business in all branches of public procurement, civil and light engineering, energetics, renewable resources and medical technologies.

ABOUT US

Our services are most of all based on a personal approach to each client; tailor-made solutions based on their requests and possibilities, on long-term cooperation, their trust, our diligence and maximal flexibility so that we are available whenever the clients need our services. We always aim to understand our client´s needs and wishes, and endeavour to give their ideas the expected shape. We are always searching and considering all possible ways that lead to our goal – the most complex solution for a particular client. We always consider all future aspects of such solution as well. We provide our clients with complex view without legal technicalities so that it is brief but clear and fitting.

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