

Design and Construction of Composite Tubular Arches with Network Suspension System: Recent Undertakings and Trends

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Abstract: The use of Network hanger arrangement, a development of the classical Nielsen V-hanger system, in steel bowstring arch bridges allows for important steel saving, with very slender main elements, owing to the remarkable reduction of bending stresses in the arches and tie beams. The present paper describes the main features of the design and construction of several long-span arch bridges of this typology in Spain: the three pedestrian footbridges for the Madrid cycling ring track, with spans of 52, 60 and 80 m, the Bridge over River Deba in Guipúzcoa with a span of 110 m and Palma del Río Bridge over River Guadalquivir in Córdoba, 130 m long. In all cases, two inclined arches linked at the crown were implemented, a very effective disposition to reduce the out-of-plane buckling length. The multiple crossings of the hanger system, consisting of prestressed bars in the case of Deba Bridge and the footbridges, and locked coil cables for Palma del Río Bridge, were dealt with by means of crossing devices which led to a technically satisfactory solution with minimal visual impact. An innovative approach to bowstring arches was introduced in Valdebebas Bridge over M-12 motorway in Madrid, next to the new T-4 Terminal of Barajas Airport, with a span of 162 m, where the hangers are replaced by a structural steel mesh –diagrid– which acts as the web of a simply-supported beam whose compression head is the arch and the tie beam is the deck.

Key words: Bowstring arch, network system, hangers, crossing devices, composite bridge, diagrid.

1. Introduction: The Network Hanger System

In 1926 Octavius F. Nielsen patented the development of the conventional vertical-hanger typology for bowstring arches, by means of oblique steel rods, in a V-configuration, which allowed him to transform the arch into a beam-type structure in which the rods took the shear forces caused by non-antifunicular load distributions, dramatically reducing the bending moments in the arch and the deck. The arch and the deck work under compression and tension, respectively, and, therefore, are highly efficient in structural terms.

The main limitation in the Nielsen scheme stems

from the compression forces, and possible instability, which may appear in one or some hangers when the live loads/permanent loads ratio is too high, typical in railway bridges and footbridges or light structures, where live loads are relevant.

In the 1950's Dr. Eng. Docent Emeritus Per Tveit (Norway) developed the concept of network bowstring arch bridge [1, 2], defined as a system which uses "inclined hangers with multiple intersections on the arch's plane". By resorting to greater complexity and a higher amount of steel in the hanger system, it very notably reduces the risk of the hangers being subjected to compression in non-symmetrical load distributions, which renders this typology liable to be used in the aforesaid typologies [3, 4].

Steinkjer and Bolstadstraumen Bridges (Figs. 1 and 2), built in Norway in 1963, with spans of 80 and 84 m,

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Fig. 1 Steinkjer Bridge (1963).



Fig. 2 Bolstadstraumen Bridge (1963).

were his first projects using this typology, which attained a fast development in countries like Norway, Germany, United States and Japan. Since 1963, the most remarkable example has been the renowned Fehmarnsund Bridge (Fig. 3), in Germany, a composite steel-and-concrete bridge for both railway and vehicles and a span of 248 m. Its world record for this typology was beaten in 2008, when Blennerhassett Bridge (Fig. 4), spanning 267.8 m over River Ohio (USA), was completed.

Structural Response

Fig. 5 shows the parabolic distribution of bending moments along a simple-supported beam, and Fig. 6 displays the typical antifunicular shape of a bowstring arch bridge.

It is well known that the structural behaviour of arch bridges is founded on their geometrical antifunicular shape, which counteracts the uniform



Fig. 3 Fehmarnsund Bridge (1963).



Fig. 4 Blennerhassett Bridge (2008).

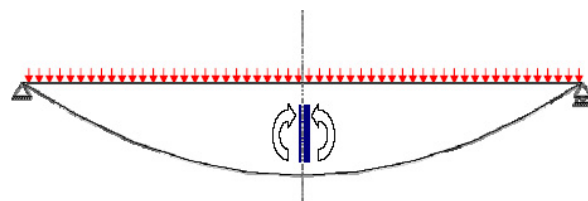


Fig. 5 Bending moments along a simply-supported span.

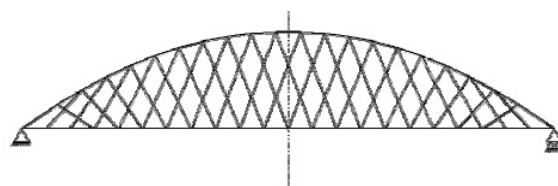


Fig. 6 Antifunicular geometry of a bowstring arch bridge.

vertical loads acting along the deck. For this load configuration, the arch is under compression, with no bending at all. When the bridge's deck is tied to the arch, that is, in bowstring arch bridges, the deck becomes a tensioned tie beam which links the arch's supports and causes the structure to transmit only vertical reactions to the foundations (Fig. 7).

This typology is especially useful when the foundation cannot bear important horizontal forces.

This behaviour is irrespective of the chosen hanger arrangement, whether vertical or inclined. However,

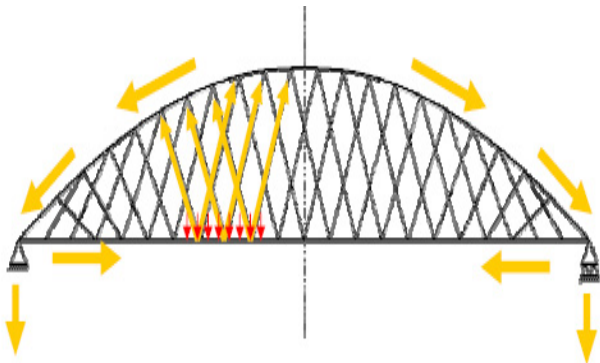


Fig. 7 Structural response of a bowstring arch bridge with Network hanger arrangement.

when vertical loads only act on one side of the deck (longitudinally speaking), the bending moments are not withstood by pure tension in the deck and pure compression in the arch, but these elements are now subjected to bending as well. The arch is no longer the antifunicular structure for the acting loads, and the load transfer from the deck to the supports is attained by means of different structural schemes, depending on the arch-deck link and their bending stiffness ratio.

With vertical hangers (Fig. 8), part of the shear stress generated by the vertical load acting on a non-symmetric position is transferred to the supports as arch compression, while the rest becomes shear force in the arch and tie beam—the share depending on their bending stiffness ratio—, and bending appears in both elements.

With oblique hangers, the load transfer to the supports is more efficient thanks to the hangers' inclined force component. The arch-hangers-deck system works as a beam, whose web is materialised by the hangers, and bending moments in the arch and the deck are notably smaller than those in the case of bowstring arches with vertical hangers. When the hanger arrangement is a mesh or “Network”, with the hangers relatively close to each other, this behaviour is optimal.

The “Network” system guarantees a highly efficient structural response, which allows for a very homogeneous, almost uniform, hanger design along the whole bridge, dramatically minimizing bending stress in the arch and the deck (Fig. 9). This leads to

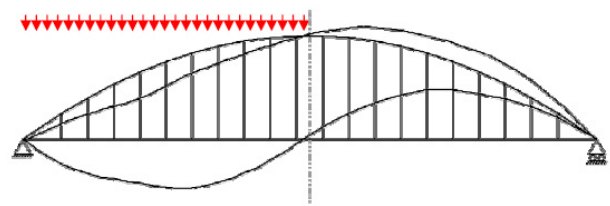


Fig. 8 Bending moment distribution in a bowstring with vertical hangers and non-symmetric live loads.

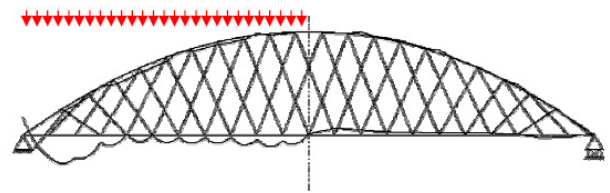


Fig. 9 Bending moment representation of a bowstring arch bridge with Network hanger system under non-symmetric live loads.

designs of high geometric slenderness, low structural steel ratios and remarkable aesthetic quality.

2. Three Arch Footbridges at the Cycling Ring Track in Madrid

2.1 Conditioning Aspects

As elements of a cycling ring path which surrounds Madrid, it was necessary to design three structures [5] that spanned some of the main highways connecting Madrid with the outskirts and the chief cities of Spain: M-500 (Castille Road), N-VI (Madrid-La Coruña) and N-II (Madrid-Barcelona), all of which bear an intense traffic flow.

In this context, the structures not only had to be as light as possible, in order to allow for a simple construction process and a quick put in place, but also aesthetically attractive, since hundreds of thousands of people will see them every day.

The chosen solution was a bowstring arch, with spans of 52 m (M-500), 60 m (N-VI) (Fig. 10) and 80 m (N-II) (Fig. 11) and a deck width of 5.0 m for the M-500 and N-VI footbridges and 6.0 m for that over N-II.

The implementation of a bowstring arch typology, either in a Nielsen arrangement or a network configuration, turned out to be a successful choice. With their remarkable slenderness and lightness, these



(a)



(b)

Fig. 10 Frontal lateral views of the footbridge over N-VI (Nielsen, 60 m span).

footbridges constitute a solution which combines optimal structural behavior, constructive simplicity, cost efficiency and aesthetic quality.

The structural concept behind these footbridges stems from know-how garnered after the design of Palma del Río Bridge, which will be extensively described in this article.

2.2 Description of the Structures

The arches consist of two hollow steel tubes, with a diameter ranging from 508 mm (M-500) to 610 mm (N-II) and a maximum thickness of 25 mm. The slenderness (span/height) ratio is 131 while the rise/span ratio is approximately 1:7, with a maximum rise of 11.50 m in the N-II arch. The arches' inclination with respect to the vertical plane is around



(a)



(b)

Fig. 11 Front and interior view of the footbridge over N-II (Network, 80 m span).

18°, with a transverse separation of 8.50 m at both ends of the footbridge.

The tie beams are two steel hollow tubes with the same diameter as that of the arch and a maximum thickness of 16 mm. Transverse bracing consists of either a stiffened plate at the crown which gradually becomes a K-shaped truss or simply a K-truss (Fig. 10).

The deck comprises a 0.20 m thick slab (0.26 m for the 60 m span Nielsen) poured onto precast slabs which rest on steel transverse girders of variable height (0.50 at mid-span to 0.30 m at the ends) located every 5.0 m.

The hangers are 42 mm diameter, S 460 N steel bars. In the case of the footbridges over M-500 (52 m) and N-VI (60 m) a typical Nielsen lattice arrangement was adopted (Fig. 10), whereas in the N-II arch a network

configuration was implemented (Fig. 11), since the excessive verticality of the hangers would lead to inadmissible compression for non-symmetrical loads.

The adopted solution in the latter case was a system of three families of hangers on the same plane, whose anchorages were placed every 5.0 m along both the tie beams and the arch. The crossing is dealt with by means of a “needle-eye” device which allows for hanger passing through another (Fig. 12).

2.3 Structural Response

Both the Nielsen and the network arrangements lead to an efficient structure in which the main longitudinal elements (arch and tie beams) barely take flexure stresses.

As for the hangers, under non-symmetrical loads and as the rise increases, the Nielsen configuration fails to prevent some hangers from being compressed, even in service conditions, which causes the longitudinal elements to take the flexure in those areas where the hangers cease to function. On the other hand, the network disposition proves to have better behavior since the hangers are in tension in all cases with a wide safety margin.

In the particular case of the 60 m span Nielsen solution, the dead load was so low that it was necessary to use a 0.26 m thick slab in order to gain



Fig. 12 N-II footbridge hanger crossing device.

some weight so that the tension in the hangers was high enough to compensate the compression induced by non-symmetrical loads. This shows that, for high values of rise and a fixed anchorage separation, steep hangers fail to work properly. In the light of this circumstance, the 80 m span footbridge required a network bowstring arch.

With respect to second-order effects, the lattice configuration (both Nielsen and network) minimizes the risk of buckling since the arch is perfectly braced by the hangers. The non-braced zone is the most delicate area, with a buckling length of approximately 15 m for the 80 m span footbridge.

2.4 Erection Process

The three footbridges were erected following the same process:

- (1) Pre-assembly of tie beams and twin arches in 3 pieces each in a metallic works plant. Special attention was placed on the arch-tie beams connection plate.
- (2) Transport of all the structural elements to the construction site.
- (3) Completion of the metallic structure using temporary supports and scaffolding.
- (4) Placing of precast slabs, ranging from 50 to 100% of the platform's surface.
- (5) Adjustment of hangers and tensioning operations.
- (6) Hoist and final positioning, at night (Fig. 13).
- (7) Placing of the remaining precast slabs (the same night) and reinforcement bars.
- (8) Concrete pouring.
- (9) Final touches.

Given the lightness of these structures, it was possible to lift them with the precast slabs and part of the reinforcement bars already placed.

By means of this industrialized process it was possible to minimize the occupation of space as well as the need for on-site metallic works. The hoisting operation, carried out with just one crane, took place at night in less than 5 hours, barely affecting traffic.



Fig. 13 Erection of N-II footbridge.



Fig. 14 Bird's eye view of the bridge and the access viaduct.

3. Arch Bridge over River Deba

3.1 Fitting the Structure in Its Surroundings

The Bridge over River Deba (Fig. 14) is located in the village of Deba, Guipúzcoa [6]. The 680 m long viaduct starts at a roundabout, under which a railway line exists, crosses the river's lowlands and spans the river itself by means of a structure ending at a tunnel (Fig. 15).

The structure caters for road and pedestrian use, thanks to the lateral sidewalks, 2.30 m wide (Fig. 16), which run longitudinally supported on the tips of

impressive lateral cantilever ribs which stem from the deck every 5 meters. The void space between the deck and the sidewalks, 2 m wide, is covered with a lightweight steel grid.

The aforesaid clearance between pedestrian and road traffic, which in the arch bridge is stressed by the psychological isolation created between both domains by the intertwined hanger planes, favouring the pedestrians' immersion in the beautiful surrounding landscape away from the traffic, definitely marked the structure's formal design and its structural response (Fig. 17).



Fig. 15 Arch view from outside the tunnel and from within the tunnel.



Fig. 16 View of the lateral sidewalks.



Fig. 17 Downstream view.

3.2 Description of the Structures

The structure bridges the Deba inlet and its surrounding marshes, linking the roundabout to the tunnel located on the east bank of the inlet.

The structure consists of three clearly defined zones (Fig. 18):

- The roundabout zone, comprising a precast U-girders deck, with irregular elevation geometry, that crosses the railway line.
- The access viaduct, located over the river's lowlands, a continuous steel and concrete composite hollow box girder with 20 + 30 + 30 m long spans.
- A 110 m long span bowstring arch bridge over the river [6].

Both the arch bridge and the access viaduct possess lateral sidewalks at either side which ease the pedestrian passage, thus fulfilling the demanded requirements for the solution. The sidewalks consist of 0.20 m thick precast concrete slabs, supported on the transverse ribs located every 5 m.

3.3 Access Viaduct

The access spans consist of a 1.25 m deep continuous steel hollow box, with a trapeze-shaped cross-section and curved bottom. Its webs are leaning inwards in such a way that the width is 5.0 m at the bottom and 4.0 m at the top. These dimensions allow the steel girder to be transported in one piece from the factory to the worksite, thus reducing all the on-site welding operations to the segment-to-segment connection.

Every 5.0 m, coinciding with the transverse stiffening trusses, 7.3 m long haunch lateral ribs are attached to the deck. Their bottom line follows the

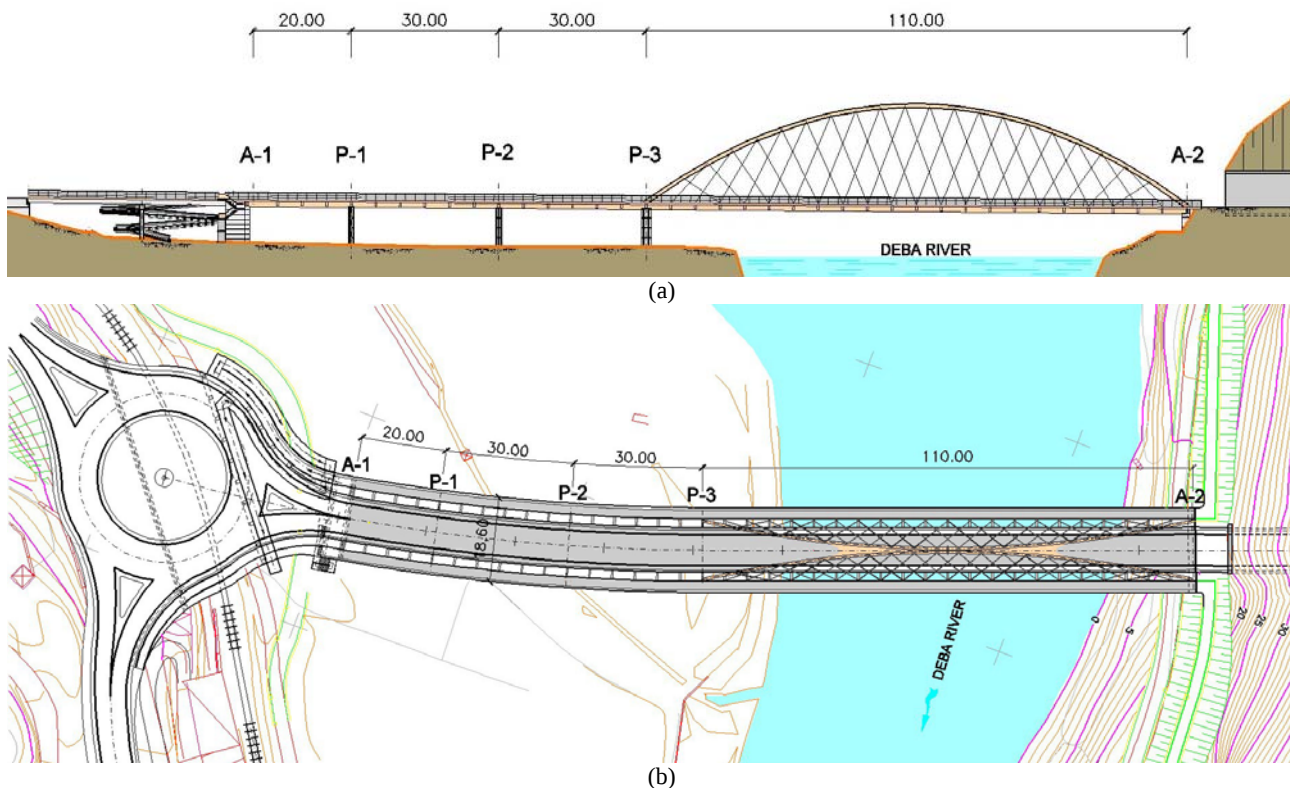


Fig. 18 Roundabout, access viaduct and arch bridge over river Deba: Elevation and plan views.

deck's bottom curve. The 2.3 m wide sidewalks are located at their end (Figs. 19 and 20).

On top of the steel box, 2.5 m long, 10.0 m wide precast concrete slabs are placed, onto which the deck slab is cast. The overall thickness is 0.32 m at the deck's axis.

At either side of the pier sections, a 0.30 m thick concrete bottom slab is cast (double composite action).

3.4 The Arch Bridge

The arch bridge is, without a doubt, the singular piece in the set of structures, not only because of its span, 110 m, but also because of its geometric configuration, consisting of a double tubular arch linked to the deck by means of hangers arranged in a mesh pattern, which confer a series of peculiarities, both to its morphology and to its structural behaviour.

3.4.1 Deck

The deck of the arch bridge has the same depth and the same bottom curve in its cross-section as the access

viaduct. The deck's cross-section (Figs. 21 and 22) comprises 2 hollow box girders whose inner webs are 4 m apart at the top, like in the access spans (Fig. 19).

The arch bridge's hollow box girders are 2.75 m wide each, and 0.95 m deep. Each girder's webs are very different. The outer one is vertical, barely 0.28 m deep, while the inner web, 1.04 m long, is inclined 26.8° (Figs. 21 and 22).

Supported on the steel girders' top flanges are the precast concrete slabs with steel trusses on which the upper concrete slab is cast.

Every 5.0 m, just like in the access viaduct, transverse cantilever ribs are attached to the deck (Fig. 22). The precast slabs which constitute the sidewalks are placed at the ribs' ends. The ribs act as transverse beams which take all loads coming from the sidewalks as well as from the deck and transfer them to the hangers through the anchorages located at the platform's edge, 6.5 m at each side of the deck's centre line.





Fig. 21 View of the arch bridge's deck (foreground) and the access viaduct's deck (background) prior to their erection.

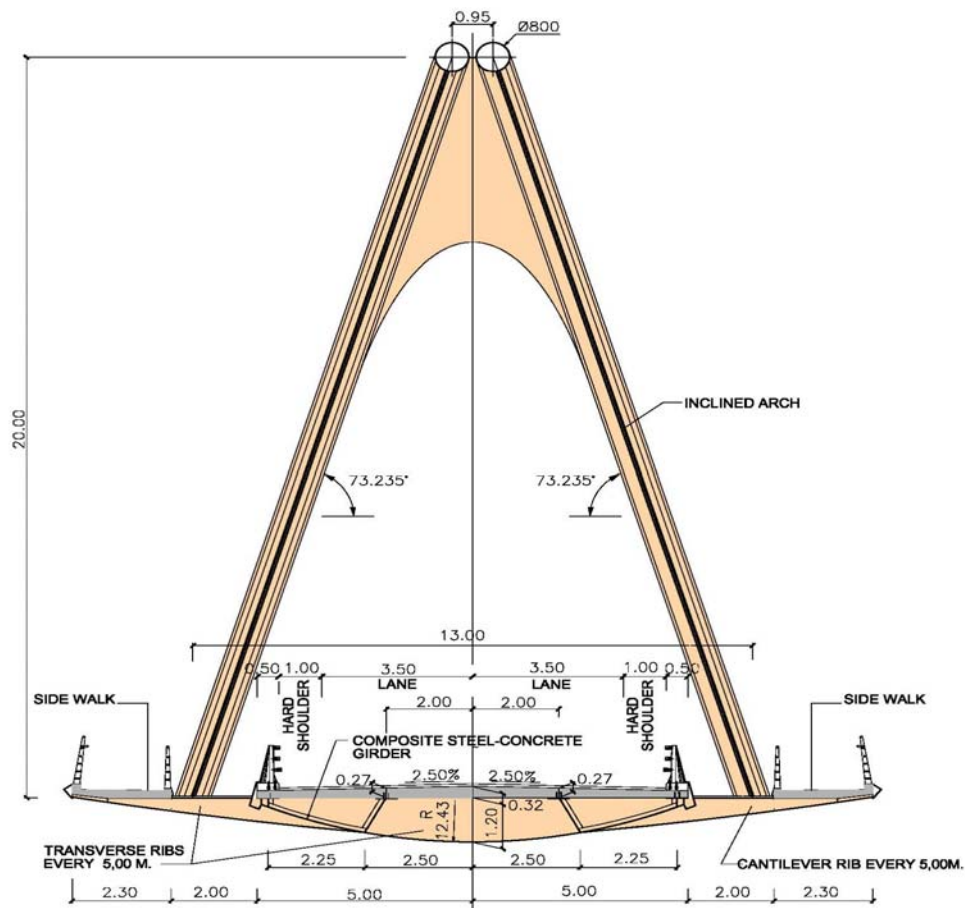


Fig. 22 Deck cross-section at the arch.

3.4.2 Arch

The arch consists of two circular tubes 0.8 m in diameter, made of S-355-J2G3 grade steel, 35 mm thick at the springings and 20 mm thick at the crown. Both arches lean inwards at an angle of 18° with the vertical plane. The arches' springings are 13 m apart, while at the crown the tubes are almost tangent, with a minimal clearance of 0.15 m. Each arch's axis is a parabola with a rise of 20 m.

A 20 mm thick steel plate, ending in an elliptic edge, braces the two arches together (Fig. 23). It is aimed at guaranteeing a joint response from both arches to transverse wind actions and limiting the arches' buckling length.

3.4.3 Hangers

Lying on the arches' planes, the hangers are circular solid bars 56 mm in diameter made of S-460 grade steel. A Network arrangement was chosen leading to a latticed mesh with multiple crossings. Each plane of hangers contains two families, each of them parallel to one direction, where every hanger crosses two of the opposite family. The anchorages are 5.0 m apart both along the arch and the tie beams (Fig. 24).

In order to prevent interference between hangers with different inclination, it was necessary to solve their crossing detail, which required a special piece being devised, in the shape of a needle eye (Fig. 25), which solved the hanger crossing. Besides, since the individual hangers were supplied in 12 m long units,



Fig. 23 View of the arch bridge from abutment 2 (tunnel).

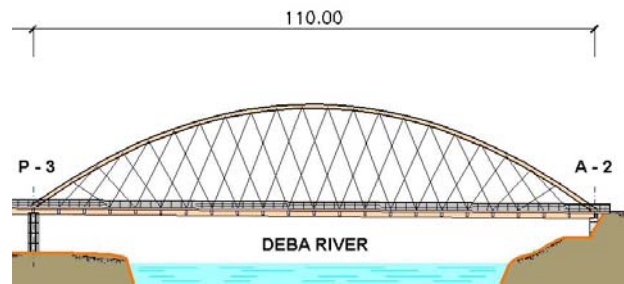


Fig. 24 Arch bridge elevation view: Network mesh scheme with anchorages every 5 m.



(a)



(b)

Fig. 25 (a) Hanger crossing devices. (b) Hanger crossing device close-up.

An elastic disc-shaped piece was implemented (Fig. 25b) so as to prevent the hangers from colliding with one another due to transverse deflections caused by wind or by any other vibratory effect. It prevents direct contact between the metallic pieces and consists of two halves, fastened together by screws, which allows for easy assembly and adjustment right at the hanger crossing. The hangers are anchored to the deck the very piece was used as a coupler between bars to

attain hangers as long as 21 m at the transverse ribs, 1.5 m from the deck and 0.5 m from the sidewalks. A hot-rolled IPE-300 stringer was laid along the whole bridge, aimed at taking the stresses caused by hanger load disequilibrium at each anchorage. This profile was embedded within the sidewalk's inner edge and hidden by the railing's kerb's lower flap.

The hangers are anchored to the ribs by means of two cylindrical tubes, one at each side of the rib's web. They are welded to the rib with a plate parallel to their axis and two horizontal cap plates at the tubes' top and bottom ends securing them (Fig. 26).

The hangers' stressing anchorage is the bottom one. It comprises, as shown in Fig. 27, a bearing plate on the tube's bottom face, a spherical hinge, a washer and two nuts fastening the bar. On the upper face of the anchorage tube a rubber ring is laid acting as a damper in order to reduce parasite bending moments originated at the hanger's anchorage.

3.4.4 Erection Process

Even though the erection of Bridge over River Deba was planned according to a conventional method suitable to this typology [5], some main features should be pointed out:

- (1) Assembly of the steel deck from full-width segments, including the transverse ribs (Fig. 28), supported on temporary shoring piled in the river bed;
- (2) Deck slab concreting onto collaborating self-bearing precast slabs supported on the steel substructure;
- (3) Erection of arches with intermediate temporary props supported on the composite deck, which remains shored (Fig. 29);
- (4) Arch props removal after closure;
- (5) Fitting of the double-family Network hanger system;
- (6) Load transfer to the bridge in three stages:

Stage 1: Hanger initial stressing

An initial tensile load was applied to all the hangers, excepting the outermost hangers, which were not fitted during the shoring removal operations. Being short bars, and close to the structure's supports, any slight



Fig. 26 Hanger lower anchorage close-up.



Fig. 27 Lower (hanger-deck rib) anchorage.



Fig. 28 Erection sequence of a steel deck segment.



Fig. 29 View of the arch and deck while shored.

deviation in the parameters used in the structural model might induce important modifications in their theoretical load, so it was decided not to connect them to the structure until after the shoring withdrawal and stress them afterwards controlling their tensile force.

Stage 2: Deck shoring removal

The deck shoring removal comprised the simultaneous, controlled descent of all the deck's points resting on the temporary struts. A computerised console was used. It allowed for independent monitoring of the 6 couples of jacks located on the top of the struts, making it possible to know their load and displacement in real time.

A step-by-step descent sequence was planned for the deck, aimed at preserving a geometric configuration homothetic to that achieved by the shoring.

Stage 3: Final load adjustment in hangers

Hanger stressing is a very delicate operation since the system is statically redundant to a high degree, and stressing a hanger means modifying the loads in the adjacent ones. Given the large number of independent variables, the structure is very susceptible to any possible difference between the model and reality.

With these factors in mind, it was set as a goal to perform as few hanger stressing operations as possible. After the shoring removal, final adjustments were made to the six central hanger couples and the four outermost couples. Eventually, a final check-up was performed, with two other couples having their tension adjusted.

(7) Finally, the pavement, the sidewalks and the rest of dead loads were laid.

4. Bowstring Arch Bridge over River Guadalquivir in Palma Del Río

4.1 Description of the Undertaking's Structures

The new Bowstring Arch Bridge over River Guadalquivir is on the Palma del Río detour road (Córdoba). The project was supposed to come up with a singular solution over River Guadalquivir, prioritizing aesthetics and landscape integration.

IDEAM's proposal, which won the tender, consisted of a steel tied arch (bowstring) with a 130 m long span and composite deck [7].

The two most remarkable structures, worthy of a longer description, are the arch bridge over River Guadalquivir and the South access viaduct, described in the following lines.

4.2 The South Access Viaduct

The South Access viaduct has a 436 m long continuous deck with a span distribution $26.0 + 12 \times 32.0 + 26.0$ m, materialized by a post-tensioned slab. The viaduct crosses River Guadalquivir's flooding plain (Fig. 30).

The deck is 11 m wide. The bridge's design was conceived bearing in mind a forthcoming widening operation, with a final width of 16 m, the same as in the arch bridge (Fig. 30).

The deck slab is divided in two symmetric ribs 4.5 m wide each linked together by a 2 m long intermediate slab, 0.25 m thick at its meeting with the ribs and 0.27 m thick at the deck's centre line (Fig. 31a).

The slab's bottom is curved, with a maximum depth, measured at the centre line, of 1.5 m, which yields a span/depth ratio of 21.6.

The ribs are hollow cored, filled with four expanded polystyrene cylinders in each rib running along the whole structure, except at the pier and end diaphragms.



Fig. 30 Bird's eye view of the South Access viaduct (foreground), the arch bridge, the North Access viaduct and the roundabout.

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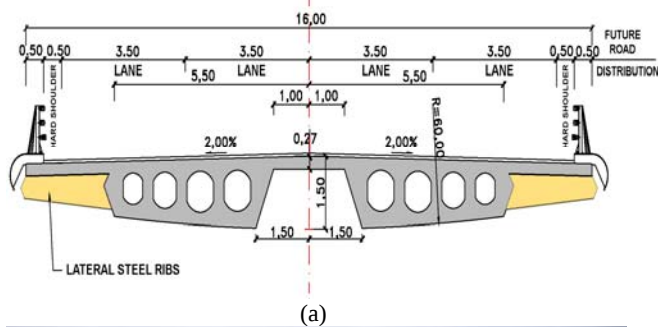


Fig. 31 Slab cross-section after widening and view of erection with falsework.

The deck was erected span by span with travelling falsework (Fig. 31b).

4.3 The Main Structure: the Arch Bridge

The typology chosen for the main span was a double symmetric 130 m span steel arch. The arches are at an angle of 68.8° with the horizontal plane and lean on each other at the crown (Fig. 32). This solution was aimed at bestowing the structure with special dynamism and originality and reducing each arch's buckling length with as few transverse bracing elements as possible.

4.3.1 Arches

The arches' cross-section (Fig. 33a) consists of constant-section tubular profiles 0.90 m in diameter aiming at formal and constructive simplicity. The number of construction and welding operations is minimal. The thickness ranges from 50 to 25 mm.

Each arch's axis is a parabola (antifunicular line of permanent loads) with a rise of 25 m, which yields a rise/span ratio of $1/5.2$ (Fig. 33).



Fig. 32 Bird's eye view of the arch bridge over River Guadalquivir.

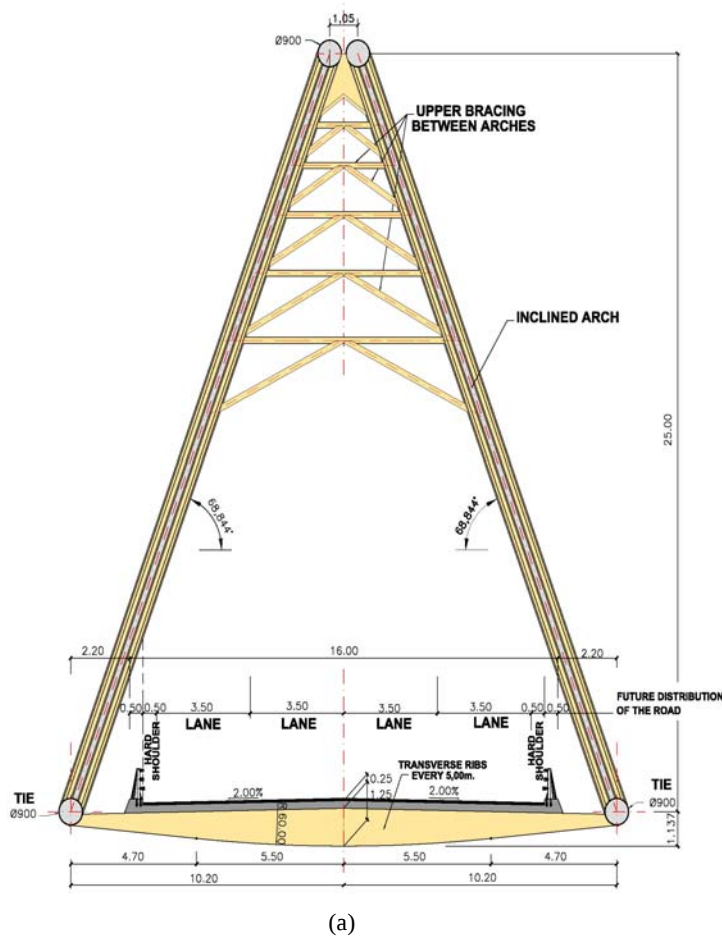


Fig. 33 Cross-section and front bird's eye view.

The arches were linked together by a “K” truss with tubular elements, which helped shorten the arches’ buckling length (Fig. 33b).

4.3.2 Deck

The deck is 16 m wide and consists of a 0.25 m thick slab with a 2% outward gradient. It is connected to the steel transverse girders located every 5 m. These transverse girders span 20.4 m between the tie beams (Fig. 33a). The hanger anchorages are located right where the transverse girders meet the lateral tie beams (Fig. 34). By acting this way the longitudinal tie beams are not subjected to concentrated loads acting on the deck.

The transverse girders are haunched (Fig. 35), matching the geometry of the access viaducts, with a maximum depth of 1.25 m. The girders’ cross-section is a double-tee.

In order to create a transverse composite

steel-concrete beam, connection studs are welded to the transverse girders’ upper flange, connecting it to the slab.

The lateral longitudinal tie beams are 10.2 m from the deck’s centre line. They are 0.90 m diameter steel tubes and 130 m long, with a slenderness (depth/span) ratio of 1/144. The transverse girders are supported on them and the hangers are also anchored to them. The aim of the lateral ties of the bridge is mainly to counteract the horizontal component of the arches in the extremes, avoiding the transmission of the horizontal reaction to the foundations, as well as the previously mentioned lateral support for the transverse deck girders.

All the metallic elements of the bridge, arches, tie beams and girders, were made of S 355 J2G3 grade steel.



Fig. 34 Details of hanger anchorage and transverse girders ends at 5 m intervals along the bridge.

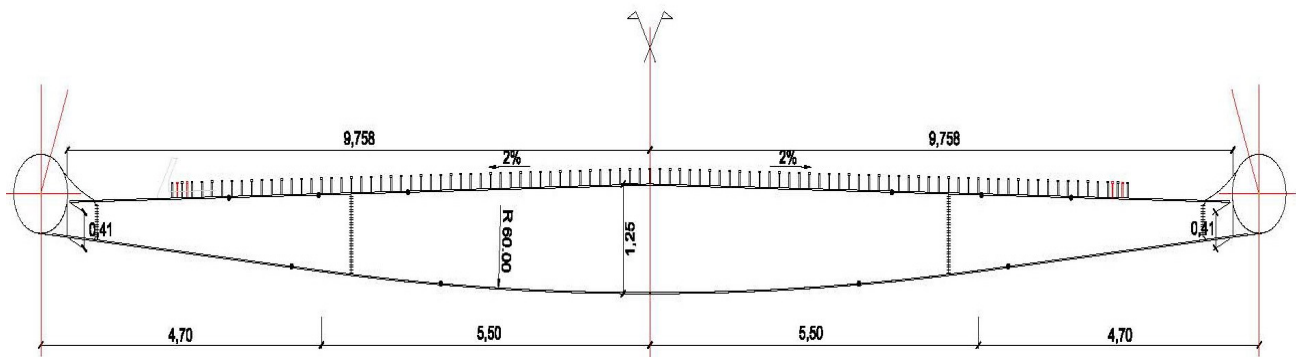


Fig. 35 Detail view of a transverse girder.

4.3.3 Hangers

The hangers act as the linking element between the deck and the arches, and transmit the vertical loads from the former to the latter. The hangers arrangement was a latticed mesh or Network [2, 3], with two overlapping planes linking the ties to the arches.

The Network hanger system with multiple crossings makes the bridge behave like a simply-supported beam whose depth is the rise. It is also capable of transmitting the shear forces and reducing bending moments in the arch and the deck under non-symmetric distributed loads, when compared to a

vertical hanger solution. This arrangement also confers a great distribution capability of concentrated loads along the bridge, which reduces bending stresses in the deck to a minimum.

Besides, increasing the number of hangers allows for the use of smaller units, more available and easier to install.

The anchorage points are spaced 5 m along both the tie beams and the arch, fulfilling multiple goals:

- Reducing the arches' buckling length.
- Reducing bending moments in the deck.
- Simplifying the arches-hangers and deck-hangers

anchorages, because of the use of smaller units.

- Achieving great efficiency in the distribution of concentrated loads on the deck.

The hangers are 45, 40, or 37 mm diameter locked coil cables creating a Network inclined mesh with multiple crossings. Each mesh comprises two planes per arch, each plane having a 50 mm offset with respect to the arch's plane (Fig. 36), causing the hangers to cross, not to intersect, each other.

At the crossing of every two hangers, a crossing device consisting of 3 pieces is installed (Fig. 37). It allows for the free rotation of each hanger and prevents hanger impact under transverse wind actions.

The hanger's ends are pinned to the arches and the tie beams, which permits free rotation of the cable.

4.3.4 Erection Process

The analysis of the erection process made it clear that great advantage could be taken of the use of steel. The arches can be assembled from smaller segments, which allow for easier erection with fairly simple auxiliary elements and in a shorter time than a concrete solution.

The use of steel as the main material also leads to smaller reactions and, therefore, cheaper foundations.

In order to erect the steel structure, shoring elements were set up on temporary embankments made on both banks (Fig. 38). The foundations of the shoring were piled so as to prevent possible scouring caused by the overflow during the erection.

The steel structure erection begins by the transverse girders being welded (on site) to the longitudinal tie beams (Fig. 38), divided in 5 segments along the deck, which are hoisted by cranes (Fig. 39).

The horizontal trusses on the deck (Fig. 39a) acted as bracing elements against wind actions during the erection, since after completion the slab transmits all transverse horizontal forces (wind, braking, earthquake) through the connection studs to the end diaphragms, which transfer them to the supports.

After the deck's steel structure is finished, the arches are erected, supported on four double props. The transverse arch bracing tubes are welded on site and afterwards, each arch segment is hoisted by cranes and fixed onto the props (Fig. 40a).

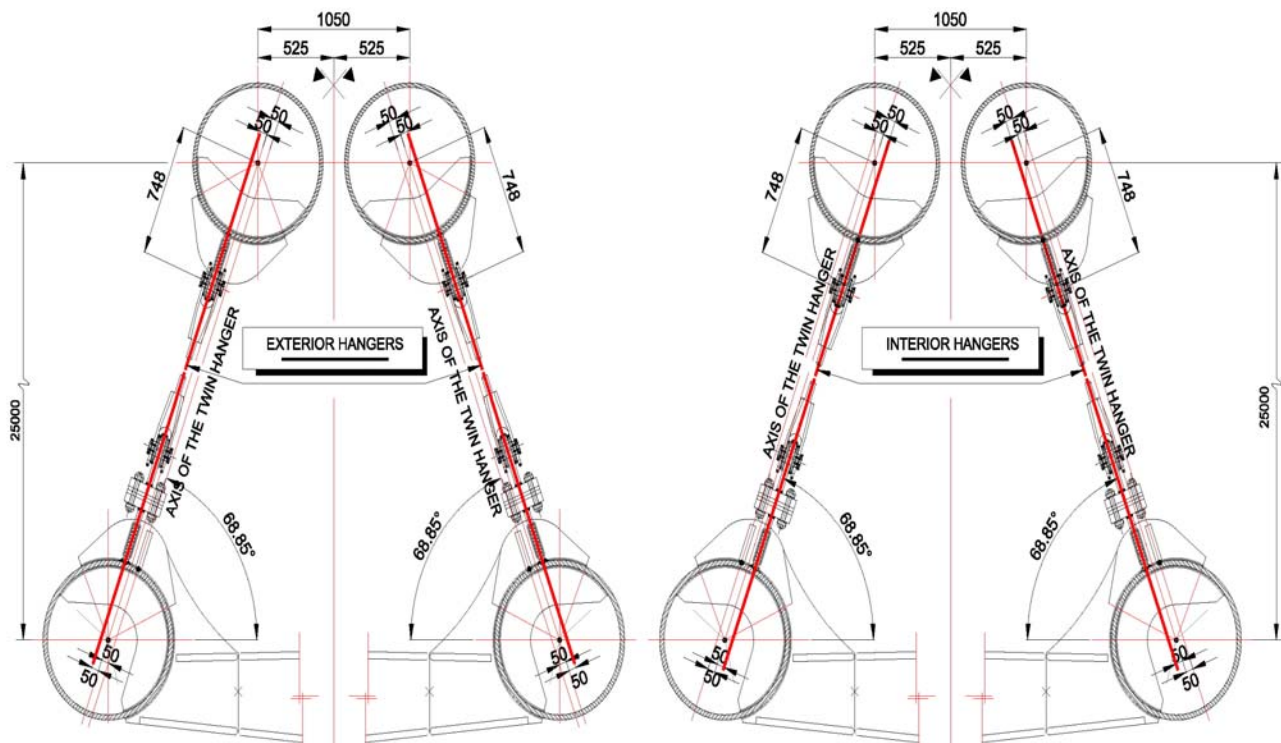


Fig. 36 Hanger layout on parallel planes with 50 mm offset in each arch.



Fig. 37 Detail of hangers on two parallel planes with the crossing device.



Fig. 38 Artificial embankments and steel deck assembly operations.

Finally, all the arch segments are welded together and the arch springing is welded to the end piece, after which the arch shoring is removed, while that under the deck still remains (Fig. 40b).

After that, the hangers are fitted and tensioned according to the process defined in the project. At this moment, the deck is suspended from the hangers and the shoring under the deck can be removed since the steel structure is self-bearing.

With the deck suspended on the hangers, the precast slabs are placed by cranes (Fig. 41a), after which all the slab reinforcement is assembled (Fig. 41b) and the whole deck slab is poured in a single stage.

After this, the final hanger load adjustments are made, the final details are finished and the load test is carried out.



(a)



(b)

Fig. 39 Hoist of deck segment and view of the deck shored after completion.

5. Second Order Analysis of the Arches

5.1 Buckling Response

For the accurate determination of the stresses acting on the arches of Deba and Palma del Río bridges, elastic, geometrically non-linear analyses were



(a)



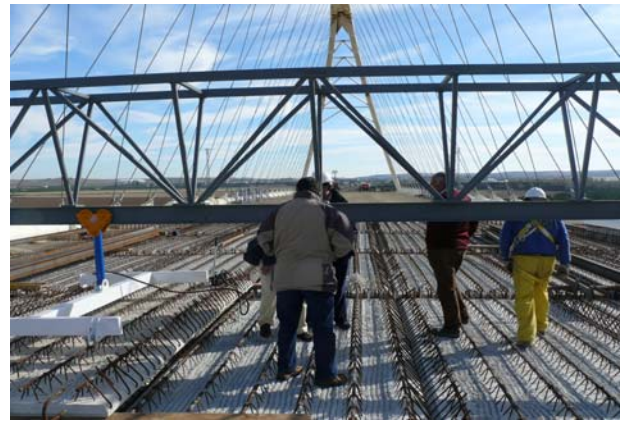
(b)

Fig. 40 Views of arch segments erection and arch crown closure.

performed, starting from a geometric configuration equivalent to the first buckling shape and a maximum deflection as specified in Eurocode [8]. According to this code, it is possible to apply an equivalent imperfection to the arches' geometry in order to account for the possible effects of geometric imperfections and in-built stresses in the steel tubes and carry out a non-linear analysis.

The equivalent geometric imperfection was 8.0 cm in Deba Bridge and 6.6 cm in Palma del Río Bridge, which represents $1/260$ and $1/360$, respectively, of the first-mode buckling length, that is, 20.60 m and 23.60 m.

Each structure's buckling modes were obtained (Fig. 42a, Deba, and Fig. 42b, Palma del Río) firstly. The first two modes correspond to global buckling in the arches. More specifically, the first mode implies the lateral displacement of each semi-arch in opposite directions, while in the second mode both semi-arches move alike.

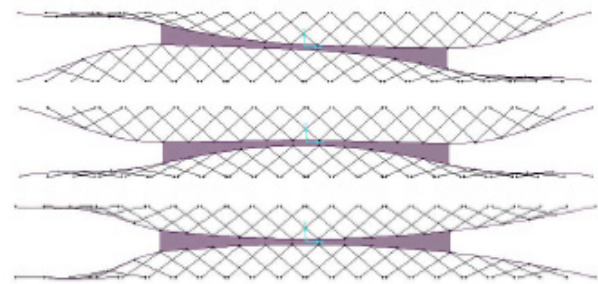


(a)

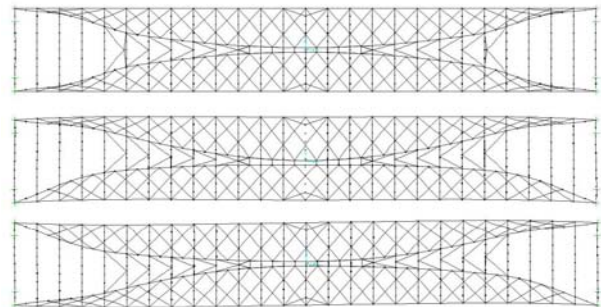


(b)

Fig. 41 Views of the precast slabs placing and of the deck after the reinforcement being assembled.



(a)



(b)

Fig. 42 Plan view of the first three out-of-plane buckling modes. Deba bridge and Palma del Río bridge.

This displacement takes place along the normal direction to the hangers' plane, which is approximately equivalent to a rotation of the semi-arches around a horizontal axis located at the crown.

From the third mode on, local buckling modes of the unbraced arch segments come into effect, with one or more deflection waves. The third buckling mode would be equivalent to that of a straight support of the same length with embedded ends.

It is also important to point out that while in the first and second modes the equivalent buckling length turned out to be, in both cases, 75% of the arch's free length between the springing and the bracing elements—estimated as one fourth of the arch's total length, in the third mode it was 61%. This fact indicates that a pre-sizing of the buckling lengths based on isolating the unbraced segment and assimilating it to a support with embedded ends is not possible, since the governing modes correspond to global displacements of the whole structure and only after the third mode can buckling shapes be likened to those of isolated supports.

On the other hand, restraint provided by the hangers causes in-plane buckling modes not to be conditioning, also helping reduce the out-of-plane arch buckling lengths by 8%.

5.2 Arch-to-Arch Binding

5.2.1 Deba Bridge

The arches were braced together by a 20 mm thick plate across the whole gap between the arches and stretching along 50% of the arches' total length.

The plate's key mission is to act as a transverse bind between the arches. The system constituted by the plate—a web working on its own plane—and the tubes is one of great bending inertia, turning the crown into a monolithic piece (Figs. 22 and 23). Apart from counteracting the weight of the inclined arches, it helps limit the buckling length, as seen previously. In the first two buckling modes (Fig. 42a), the governing ones, the central segment barely experiences any deformation

and practically rotates as a rigid body. Since both semi-arches at the same side of the crown move in the same direction, stresses caused in the plate are moderate. In the third and subsequent modes, since each semi-arch's leg moves in opposite directions, the stress level is more important. A FEM model was implemented in order to verify that the stress state under these possible load configurations did not exceed the steel's yield limit.

5.2.2 Palma del Río Bridge

In the bridge over River Guadalquivir in Palma del Río the upper bracing system consisted of a visually permeable "K" truss made from tubular profiles and a plate near the crown (Fig. 43), which helped shorten the buckling length as commented previously.



(a)



(b)

Fig. 43 View of the bracing systems in Palma del Río bridge.

6. Valdebebas Bridge

6.1 Conceptual Design

IDEAM's proposal for Valdebebas Bridge [3] won a tender for the design of a singular bridge which is to become the new urban plan's calling card, and also the best access to Madrid's Airport Terminal 4 (Fig. 44).

The tender specifications comprised very stringent clearance conditions, both below (M-12 road traffic) and above the bridge (aeronautical clearance), as well as the need to span around 150 m.

The aforesaid restrictions led us to consider, as the best possible solution, an arch bridge with lower deck (Fig. 45). This bridge features a very peculiar design, whose shape stems from an inverted T variable section, associated to a shallow bow-string arch typology, a structural configuration which allows to span the desired length, 150 m, without transferring horizontal forces to the foundations.

The most relevant and singular feature of the structural design consists of the double "diagrid" or permeable structural mesh (Fig. 45), from which the deck hangs and which, as a plane of great stiffness, materializes a latticed web of a haunch beam, that is to say, the bowstring arch itself. The "diagrid" was designed as a double plane at each side of the structure's symmetry plane, constituting a four-layered mesh.

The structure's novelty and its formal and aesthetic singularity, inspired in aeronautic industry designs, are enhanced by the choice of metal grey paint matching the roof cladding of the neighboring T-4 Airport Terminal as well as by the use of ornamental lighting within the diagrid generating a suggestive and subtle game of dim lights and shadows.

6.2 Description of the Structure

6.2.1 Arch

The whole deck is suspended from the arch, with a rise of 10.30 m and a span of 124 m (considering only the arch above the deck), yielding a 1/12 rise/span ratio,



Fig. 44 Lateral view of the Valdebebas Bridge (render).



Fig. 45 Front view of the Valdebebas Bridge (render).

that is, a shallow arch. The arch's cross section (Fig. 46) is almost rectangular, with two ledges jutting out at the top where the "diagrid" planes are attached. The arch has a constant depth of 1.50 m and variable width ranging from 4.0 m at the springing to 2.0 m at the crown. It is made of S355J2 steel, with maximum thickness at the crown and minimum at the start. The arch follows a circular line of 150 m in radius and continues at a tangent below the deck. The lower part becomes wider and remains linked to the deck, creating a massive triangular spandrel, of great formal incidence.

6.2.2 Deck

It consists of a multicellular S355J2 steel hollow box girder 3.0 m deep (at its center line) on top of which a 0.25 m thick concrete slab rests (Fig. 46). The deck's bottom follows a circular curve 30 m in radius which goes on at a tangent up to the abutments. The deck's cross section comprises 5 cells by means of 4 webs. In order to easily handle the section's elements and to

attach the “diagrid” to the inner webs, the latter have the same inclination as the diagrid planes.

Within the section and every 5.0 m, transverse trusses are located so that torsional effects derived from eccentric actions can be transferred to the “diagrid”, which acts as a plane of inclined hangers which divert the loads along the arch. Torsional moment, quite important owing to the bridge’s width, is withstood in the intermediate 124 m thanks to the high torsional stiffness of the deck’s composite section.

6.2.3 Diagrid

The deck is linked to the arch by means of a mesh or lattice of S355J2H steel tubes termed “diagrid” (Fig. 47). Its structural mission is to transfer the vertical loads acting on the deck to the arch. Therefore, it basically responds under tension. Each “diagrid” mesh consists of two coplanar, mutually perpendicular families of tubular profiles arranged at angles of 45° with respect to the horizontal plane. There are two “diagrid” planes at each side of the arch, one in each direction.

Diagrid’s stiffness within its own plane suppresses any in-plane arch instability problem. Likewise, the two “diagrid” planes at each side of the arch are braced together by means of trusses in order to prevent any wind-induced vibrations. These trusses are parallel to the “diagrid tubes” direction, thus preventing visual and inner lighting interference.

6.2.4 Abutments and counterweights

The bridge is in equilibrium at its end by placing a concrete counterweight across the deck’s width. The acting forces are the deck’s tension, the counterweight’s vertical force and the compression along the concrete strut which links the counterweight to the arch’s support section. In order to facilitate the stress transmission, the deck’s final meters, hidden within the abutments, were conceived in prestressed concrete, in such a way that the steel deck (where the prestressing cables are connected) transfers its tension adequately.

The deck’s upward lift is withstood with a buffer and 6 spherical bearings.

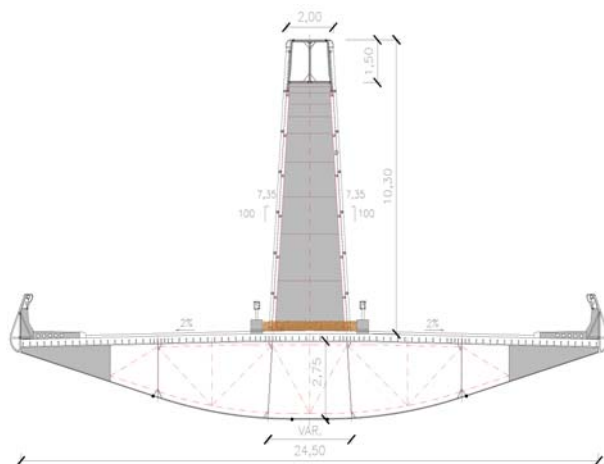


Fig. 46 Mid-span cross-section of the Valdebebas Bridge.

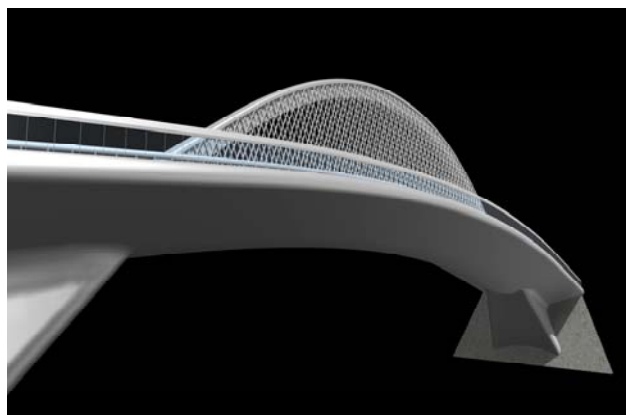


Fig. 47 Valdebebas Bridge: Diagrid close-up (render).

6.3 Structural Behaviour

The longitudinal structural scheme can be interpreted in two ways which only represent two different approaches to the same structural concept:

- The 156 m apparently (or visually) long span, equivalent to 162 m between the abutments’ hidden supports, is dealt with by means of a 124 m long bowstring steel arch with composite deck (or tie beam). The inclined suspension hangers (Network type bowstring arch), are replaced by a quadruple “diagrid” mesh arranged on two planes with conventional rectangular hollow sections at angles of 45° creating a stiff suspension plane which prevents in-plane arch buckling.

The bow-string’s vertical reactions act at the end of the spandrels, 19 meters from the support lines. The reaction can be projected in two directions: a

compression strut along the spandrel's lower face (bottom compression head) and a horizontal tension force withstood by the steel upper box girder. This scheme is self-balanced in a traditional triangular cell arrangement: the upper tension force goes on by means of a 25 m long prestressed concrete tie anchored at the end of the composite deck and hidden within the abutments. This force can be split into two: an upward vertical force compensated by the counterweight (anchored at the end of the abutment), and an inclined compressed concrete strut which, heading to the main span support, closes the cell transmitting the vertical resultant force to a deep foundation.

- The structural scheme can be explained from a different angle: a 162 m long doubly embedded span is designed. The positive bending area stretches 124 m and is withstood by a variable depth beam with a latticed web (diagrid). The arch acts as the compression chord and the deck, as the tension tie. In the negative bending areas, 19 m long, next to the supports, the composite deck carries the upper tension and the spandrel, with double composite action, the lower compression. The bedding moment, thanks to the triangular cell scheme, is provided by a short hidden compensation span, whose extreme negative reaction is balanced by a counterweight.

In order to optimize the structure's behavior during the erection process, once the bridge is continuous and before placing the dead loads, jacks will be applied to readjust the reactions. By doing so, it is possible to control the position of the zero-moment point of the three-span beam, taking it to the intersection of the arch and the deck (approximately) and reducing bending in the structure's weakest sections. A time-dependent analysis was also made to obtain the long-term force and reaction redistribution.

7. Conclusions

This article has given an in-depth insight on the bowstring tied arch and its application in several singular long-span bridges.

By using inclined hangers, either in a Nielsen or in a Network arrangement, bending stresses in the arches and tie beams are minimal when compared to vertical-hanger solutions. All elements attain great structural performance, which renders slender structures with very low steel quantities.

The double leaning arch system with crown bracing not only bestows the aforesaid bridges with a strong special character, but it also becomes an important structural aid, be it with the central bracing plate –Deba bridge– or the “K” truss –Palma del Río bridge and Madrid footbridges–, because it reduces the out-of-plane buckling length, improving their structural response and leading to an optimization of the steel quantities.

Furthering these concepts, Valdebebas Bridge's design is the aftermath of an orthodox structural conception in which, as explained, all its elements come from an optimized structural scheme, and where any ornamental or superfluous elements have been eliminated. This proposal's formal incidence and architectural character stem from the order and treatment of the structure itself.

In all cases, original, innovative solutions were come up with, which satisfactorily met both the structural and aesthetic requirements.

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