

THE USE AND DEVELOPMENT OF THE NETWORK SUSPENSION SYSTEM FOR STEEL BOWSTRING ARCHES

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Abstract: The present paper shows the development of the Network Suspension System applied in the project of steel bowstring arch-bridges. After the experience of bridge over river “Deba” in the north of Spain (110 m span), IDEAM has recently designed and constructed “Palma del Río” Bridge over Guadalquivir river, a double bowstring arch, 130 m span, with tubular elements, and closed cables materializing the double Network Suspension System. New “Valdebebas” Bridge (150 m span), in the access to Madrid’s Airport Terminal 4 will become the development of this typology with a singular double ‘diagrid’ or permeable structural mesh from which the deck hangs and which, as a plane of great stiffness, materialises a latticed web of a variable depth beam.

1 Introduction: The network tied arch system

The most relevant feature of the network tied arch system lies in its structural and aesthetic efficiency, resorting to oblique hangers in a network arrangement.

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 turn 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 main limitation in this system comes from the compressions which may appear in some hangers when the live loads/permanent loads ratio is too high, typical in railway bridges and in light structures.

In the 1950’s Professor Eng. Per Tveit (Norway) developed the concept of network bowstring arch bridge. In an article published in the June 1966 issue of The Structural Engineer, he defined it as a system which uses “inclined hangers with multiple intersections on the arch’s plane”. By resorting to a 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 susceptible to be used in extremely light footbridges as well as road and railway bridges.

Steinkjer Bridge (Fig 1), built in Nimega in 1963, with a span of 80 m, was his first project using this typology, which attained a fast development in countries like Norway, Germany, United States and Japan.

The most remarkable example is the beautiful and renowned Fehmarnsund Bridge (Fig 2), in the Baltic Sea, a composite steel-and-concrete bridge for both railway and vehicles and a span of 248 m. Built in 1963 it still holds the world record for this typology.



Fig. 1: Steinkjer Bridge (1963)



Fig. 2: Fehmarnsund Bridge (1963)

2 Structural behaviour

The vertical loads of the deck a bowstring arch are suspended by the tensioned hangers, raising the loads to the upper arch. The vertical component of the arch's compression is transmitted to the extreme supports, and the horizontal component is resisted by the tensioned lower tie [1].

This typology is especially useful when the soil doesn't bear important horizontal reactions.

The structural response against antimetric vertical loads of a bowstring with vertical hangers induces important bending moments on the arch and tie of the bridge (Fig. 3).

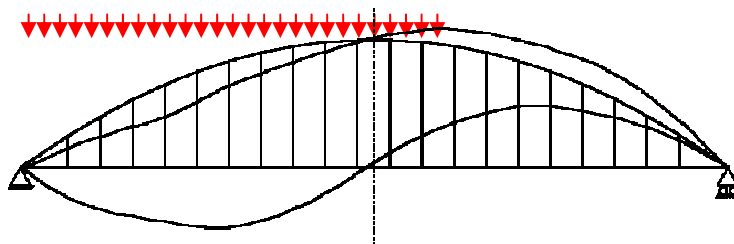


Fig. 3: Structural response against antimetric vertical loads of a bowstring with vertical hangers. Bending moment forces.

The network system allows for a very efficient structural response which leads to a very homogeneous hanger dimensioning –the cross-section is practically the same all along the structure–, as well as to the minimisation of bending stresses in the arch and the tie beams (Fig. 4). It also improves both the in-plane and out-of-plane arch buckling conditions. Both the arch and the deck are subjected practically exclusively to axial forces, thus making it possible to attain high slenderness ratios and great material economy.

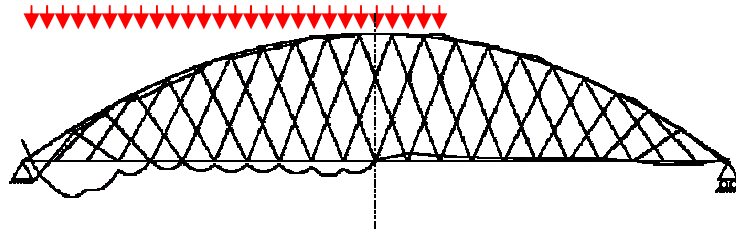


Fig. 4: Structural response against antymmetric vertical loads of a bowstring with a network suspension system. Bending moment forces.

3 Two metal bowstring arch bridges with network suspension system.

We have had the occasion to design and construct the first big two arch bridges of this typology in Spain, Deba and Palma del Río, of extreme slenderness.

The arch bridge over Deba River [1], with a 110 m span (Fig. 5), was designed with two inclined steel arches with 800 mm of diameter and 45 mm of maximum thickness near the supports, and 25 mm of minimum thickness at mid span. The arches are inclined $17,3^\circ$ with respect to a vertical plane, and rise 20 m, with a ratio rise/span 1/5,5.



Fig. 5: Elevation view of the bridge over Deba River

3.1 Bridge of “Palma del Río” over Guadalquivir River

The bridge of Palma del Río, over Guadalquivir River [1,2], constitutes a remarkable development of this typology with a bowstring that spans 130 m (Fig. 6), and two inclined arches that rise 25.0 m with a tubular cross section of 900 mm of diameter and 50 mm of maximum thickness (Figs. 7, 8). The deck consists of two simple lateral ties of 900 mm of diameter 40 mm of maximum thickness, and one intermediate platform supported by transverse composite concrete-steel beams of variable height spaced every 5,0 m (Figs. 7, 8).

The extreme slenderness of the arches and ties ($H/L=1/144,4$) is obtained thanks to closed cable network suspension system (Figs. 6, 8), which drastically minimizes the bending moments on the arches and ties, and reduces the buckling effects on the arches, as it has been described before.

The deck's platform is 16 m wide, and the deck consists of an upper slab of 0,25 m in thickness. This slab is materialized by the inferior precast slabs supported on the transverse

steel beams of the deck placed every 5 m, and the upper reinforcement and cast-in-place concrete which completes the total thickness of the slab.



Fig. 6: Aerial View of “Palma del Río” Bridge over Guadalquivir River

The transverse beams that materialize the deck span 20,4 m between the tubular lateral ties of the bridge (Fig. 14). The hanger's anchorages are located right where the transverse beams meet the lateral ties (Fig. 17). This way the lateral ties aren't subject to the effect of the punctual loads that act on the deck.

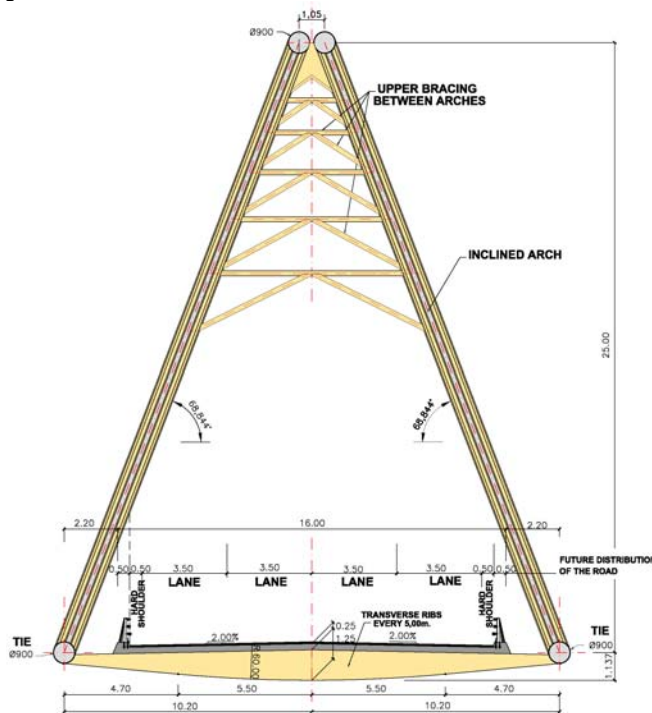


Fig. 7: Cross section



Fig. 8: Aerial view of the bridge

The deck's transverse composite steel-concrete beams have variable height, and the beam's lower plate follows a circular curve with 60 m of radius in the central part, and with a linear variation in the rest. The maximum height is 1,25 m, and the cross section is an “I” beam.

The steel used in both arches, ties, and the deck is S-355-J2G3.

The aim of the lateral ties of the bridge, tubes of 900 mm of diameter, is mainly to equilibrate 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 beams of the deck.

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 were designed in an inclined network suspension system linking the ties to the arches, with spaces of 5 m between their extreme anchorages (Fig. 9&10).



Fig. 9: Frontal view of the arches and the upper bracing.



Fig. 10: Anchorages every 5 m

Furthermore, as the number of hangers increases, it allows us to use small units easy to fit.

The main bearing structure of the bridge are the two arches, which are inclined $21,20^\circ$ with respect to a vertical plane, and they arise from the extremes of the lower part of the deck converging with the lateral ties. Both arches meet at the crown of the bridge and rise 25,0 m, with a ratio rise-span 1/5,2.

All along the arches, and every 5 m, are arranged the upper extreme anchorages of the hangers. Bracing the inclined arches, we designed a K truss with tubular elements as it can be seen on figure 9, which reduces the out-of plane buckling length of the arches.

The distance between anchorages for the network suspension system is reduced to 5 m, fulfilling multiple objectives:

- Reducing the buckling length of the arches (Fig. 11)
- Reducing the bending moments of the deck.
- Simplifying the arches-hangers and deck-hangers anchorages, because of the use of small units of hangers.
- Achieving a great efficiency in the distribution of punctual loads on the deck.

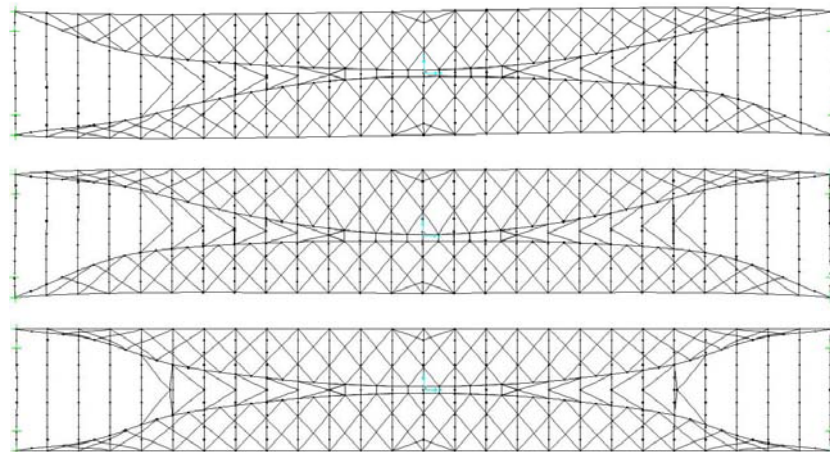


Fig. 11: Plan view of the first three out of plane buckling modes

At the initial point of the arches and lateral ties of the deck, is arranged a transverse rib, similar to the normal transverse beams of the deck, which has to resist the horizontal transverse forces due to the inclination of the arches. These ribs have two intermediate bearings so as to coerce the torsional turn of the ties and constrain the out of plane buckling of the arches in their beginning.

The multiple crossings between hangers were resolved with an original device, which enables the hangers' arrangement and minimizes the visual impact of these crossings.

The hangers are closed cables with 45, 40, or 37 mm of diameter, which compose the network suspension system of each inclined arch, are located on two different parallel plans, each one of them with a slight eccentricity of 6 cm with respect to the arch's plane, that is the reason why the hangers cross but don't cut one another. At the end of the crossing hangers there was a device that allowed the fastening of the cables as well as the free turn of one cable with respect the other (Fig. 12).

In Deba Bridge all the hangers were steel rods disposed on the same inclined plane, what required a device similar to a yoke, as can be seen on figures 24 and 25, which lets a rod cross through the other (Fig. 13)



Fig. 12: Palma del Río's cable hanger crossing device



Fig. 13: Deba's rod hanger crossing device

4 Development of the network suspension system. Valdebebas bridge.

IDEAM's design for Valdebebas bridge won a tender for the design of a singular bridge which is to become an urban landmark, Valdebebas' urban concept's calling card, and also the best access to Madrid's Airport Terminal 4 (Fig. 14).



Fig 14: Render view of the bridge in the access to Madrid's Airport New Terminal 4.

Among the tender's specifications were 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.



Fig 15: Illuminated night view of the scale model



Fig 16: Render view for the tender

The aforesaid restrictions led us to consider, as the best possible solution, an arch bridge with lower deck (Figs. 15&16). This bridge features a very peculiar geometrical character, almost aeronautical, 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.

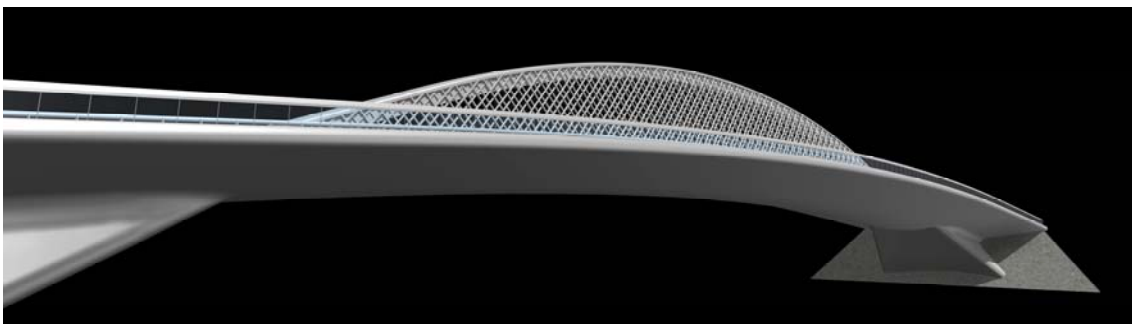


Fig 17: Render inferior view.

The design's most relevant and singular feature consists of the double 'diagrid' or permeable structural mesh (Figs. 17, 19), from which the deck hangs and which, as a plane of great stiffness, materialises a latticed web of a variable depth 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 quadruple offset mesh.

Tour main structural elements should be highlighted:

- Arch: the whole deck is suspended from the arch, with a structural 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, which indicated this is a shallow arch. The arch's cross section (Fig. 18) is almost rectangular, with two ledges jutting out at the top in order to attach the

‘diagrid’ planes. The arch has a constant depth of 1,50 m and variable width ranging from 4,0 m at the start to 2,0 m at the crown. It is made of S355J2 steel, with maximum thickness at the crown and minimum at the start. Seen in elevation, the arch follows a circular curve of approximately 150 m of radius and continues at a tangent up to the start. Below the deck, it goes on up to the supports as it becomes wider and remains linked to the deck, creating a volumetric triangular spandrel, of great formal incidence. The spandrel’s geometry contributes to distributing the compression stress transmitted from the arch to the supports.

- Deck: It consists of a multicellular steel hollow box girder 3,0 m deep (at its center) on top of which a 0,25 m thick concrete slab rests (Fig. 18). S355J2 steel is used, with a yield stress of 355 N/mm². The section’s bottom follows a circular curve 30,00 m of radius which goes on at a tangent up to the abutments. The deck’s cross section comprises 5 cells by means of using 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.

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.

The deck’s twisting stiffness was evaluated, in a safety side, taking into account different cracking hypothesis due to the tie’s tension that constitutes the composite deck of the bridge.

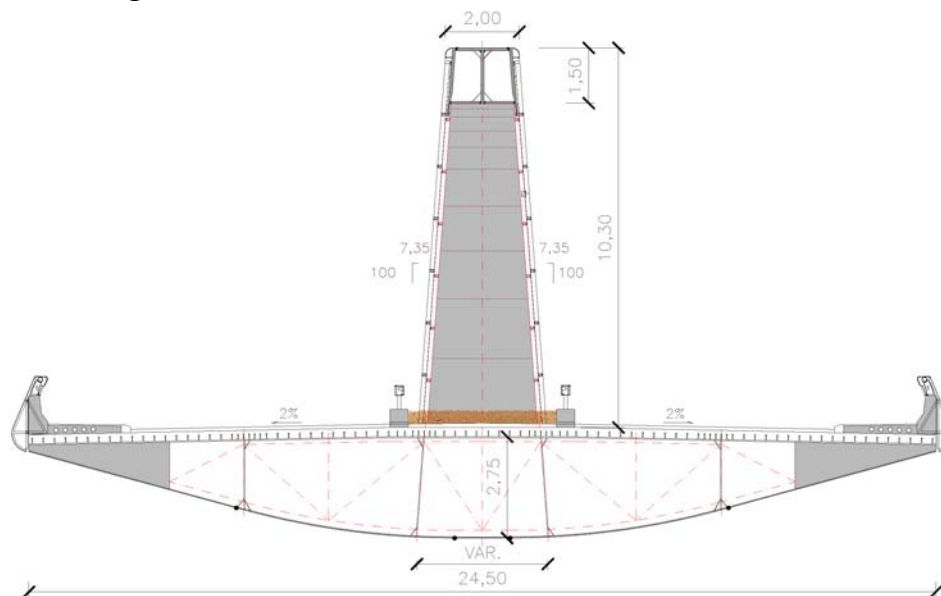


Fig 18: Cross at the arch’s crown

- Deck suspension system or diagrid (Fig. 19). The deck is linked to the arch by means of a mesh or lattice of S355J2H steel tubes termed “diagrid”. Its structural mission is to transfer the vertical loads acting on the deck from it to the arch. Therefore, it basically responds under tension, since the shear stress which might have to be withstood acting as a web of an inverted T-section is dramatically reduced by the arch’s compression inclined component as well as by the deck’s (or tie beam’s) flexural stiffness. 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.

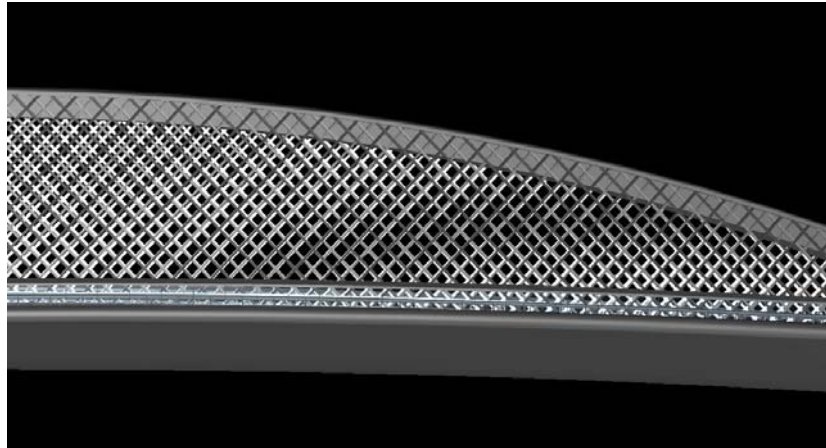


Fig 19: ‘Diagrid’ detail.

- Abutments and counterweights: The bridge is in equilibrium at its end by placing a concrete counterweight across the deck’s width. Equilibrium is achieved at the top of the counterweight. 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. Besides, a massive node is created at the counterweight’s head.

The deck’s upward lift is withstood with a stopper and 6 spherical bearings (or similar devices). Since abutment 2 is longitudinally fixed, stoppers are also placed along that direction.

Despite the proposal’s innovative formal design and its architectural approach, uncommon in the field of bridges, this 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 bearing elements design.

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 suspension hangers are inclined, like in the Network type bowstring arches, but in this case, they are replaced by a quadruple ‘diagrid’ mesh arranged on two planes with conventional rectangular hollow sections (RHS) 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 or cantilevers, 19 meters from the support lines. The reaction can be projected in two directions: a compression strut along the spandrel's lower face (which must be properly connected to a concrete inner bottom head, in double composite action) 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.

- Alternatively, 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.

Even though both structural approaches to the design's conceptual explanation are correct, the second one (embedded beam) is better to describe the bridge's behaviour against live loads. The behaviour as bowstring arch with network hangers between the zero-moment points and a deck supported on triangular cells assumes that the arch-deck structure is articulated at their intersection or that the zero-moment point of the three-span beam ($22.5+162+22.5$) is located right there. The arch and the deck are connected and even though the zero-moment point of a variable stiffness beams is located at the weakest section for uniform loading, live loads alter that situation. This peculiar aspect does not appear in conventional Nielsen bowstring arches, which behave as simply supported beams. This feature enhances the structure's beam behaviour.

In order to optimize the structure's behaviour 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 obtaining the redistributions forces and reactions after an infinite period of time.

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