BOWSTRING FOOTBRIDGES IN THE CYCLING RING ROAD IN MADRID

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Summary

Three new bowstring footbridges have been built recently in Madrid, as part of the ‘Cycling Green Ring’, a cycling route which skirts the metropolitan area and must, therefore, cross the main highways leading in and out of the city.

The spans of the arches – 52, 60 and 80 m– were conditioned by the width of the existing roads. The project was developed after serious consideration of the following factors:

1) Conception of a unique, aesthetically outstanding design which would become a formal reference for the ‘Green Ring’.
2) Necessity of spanning a long distance without middle supports and with very stringent clearance requirements.
3) Allowance for quick project completion and easy execution and construction process given the tight schedule.
4) Capability of placing the structure in its final position in less than 5 hours, at night, thus minimising the hindrances to traffic, since the affected roads are key to Madrid´s access network.

Keywords: Footbridge; aesthetic; bowstring; Nielsen configuration; network arrangement; hangers

1. The network tied arch system

The three footbridges were designed in response to the above mentioned restrictions, resorting to steel tied –or bowstring– arches with hollow tubes as their main structural elements, that is to say, the double leaning arches and their respective tie beams.

The deck consists of a series of transverse composite steel-and-concrete girders of variable height –fish-belly shape– at regular intervals of 5.0 m and hinged at the tie beams.

The most relevant feature of this design lies in its structural and aesthetic efficiency, resorting to oblique hangers, either in a Nielsen configuration (spans of 52 and 60 m) or in a Network arrangement (80 m span).

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 absorbed the shear stress 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 Fehrmarnsund Bridge (1963)

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. 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.

We have had the occasion to project and construct the first big arch bridges of this typology in Spain: Deba and Palma del Río. Of extreme slenderness, special care was placed on the design of those elements with a greater visual and aesthetic impact: geometrical order of the two leaning families of hangers on each plane, arch and tie beam anchorage details and, especially, the specific design of anti-friction devices at the crossing of hangers (Figs 3 to 5).

Fig.3 Palma del Río Bridge. Hanger anchorage detail (2008)

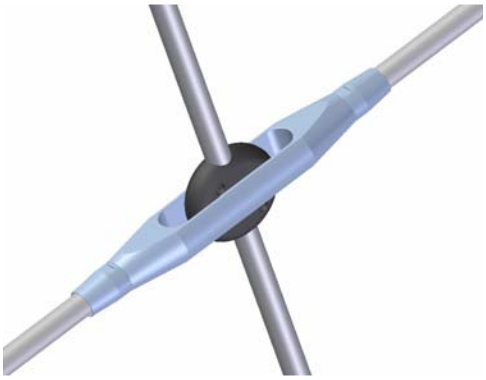

Fig.4 Deba Bridge. Hanger crossing device (2006)

Fig.5 Palma del Río Bridge. Hanger crossing device (2008)

1) Bridge over river Deba (Figs 6 & 7), with a span of 110 m and a vertical rise of 20.0 m; a tubular hollow section arch of 90 cm in diameter, a box cross-section deck of 1.30 m in height and rod hangers.

Fig.6 Deba Bridge. Front view (2006)

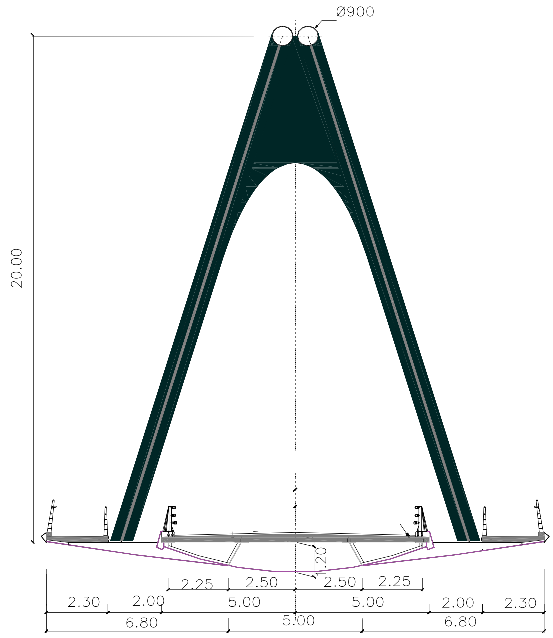

Fig.7 Deba Bridge. Cross section (2006)

2) Bridge over river Guadalquivir in Palma del Río (Figs 8 & 9), with a span of 130 m and a vertical rise of 25.0 m; tubular tie beams of 90 cm in diameter and cable hangers. This bridge constitutes the embryo of the 'Green Ring' footbridges.

Fig.8 Palma del Río Bridge. Front view (2008)

Fig 9 Palma del Río Bridge. Transverse view (2008)

2. Description of the structures

As elements of a cycling ring path which surrounds Madrid, it was necessary to design three structures that spanned over 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.

Table 1. Project Organization

Owner	Madrid City Council
Works Director	Javier Maestro
Construction company	Acciona Infraestructuras
Steelworks Plant	Talleres Centrales
Structure Designers	Francisco Millanes, Luis Matute, Mónica Alonso, Jorge Nebreda (IDEAM)

The chosen solution was a bowstring arch, with spans of 52 m (M-500), 60 m (N-VI) and 80 m (N-II) 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 (Figs 10 to 17).

Fig.10 M-500 Footbridge (Nielsen, 52 m). Front view (2007)

Fig.11 M-500 Footbridge (52 m). Transverse view (2007)

Fig.12 N-VI Footbridge (Nielsen, 60 m). Front view (2007)

Fig.13 N-VI Footbridge (60 m). Transverse view (2007)

Fig.14 N-II Footbridge (network, 80 m). Front view (2007)

Fig.15 N-II Footbridge (80 m). Hangers detail (2007)

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

Fig.16 N-II Footbridge (80 m). Lateral view (2007)

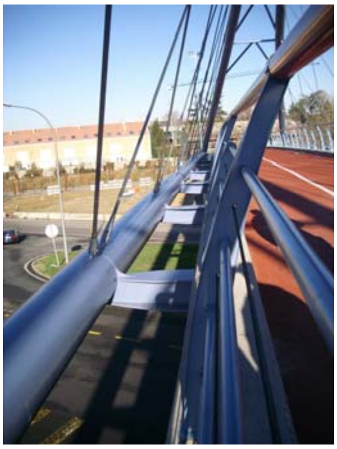

Fig.17 N-II Footbridge.Lateral detail (2007)

The arches are longitudinally braced by means of two steel hollow section 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 18).

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 located every 5.0 m, whose variable height (0.50 to 0.30 m) smoothens the longitudinal view and reduces the tie beams' height perception (Fig 19).

The total weight of steel does not exceed 1300 kN in the worst of cases, which represents a ratio of 270 kg of structural steel per square meter of deck.

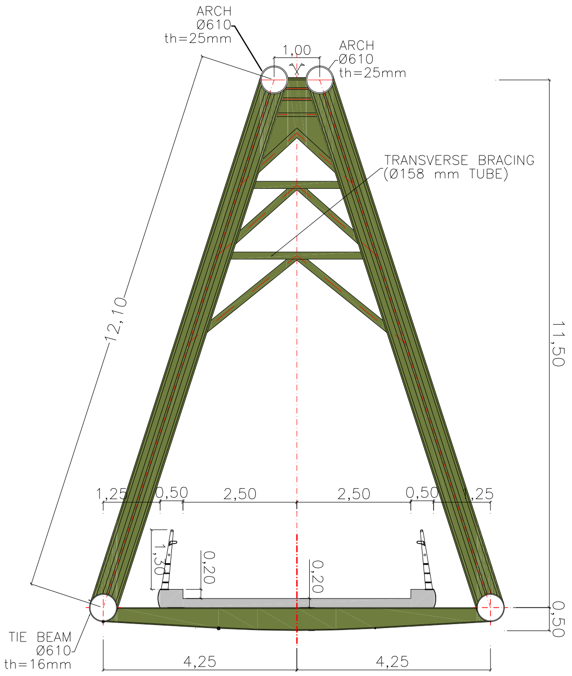

Fig.18 N-II Footbridge (80 m).Cross section

Fig.19 N-VI Footbridge. Bottom view

The longitudinal link between the arch and the deck is materialised by means of 42 mm diameter, S 460 N steel rods. In the case of the footbridges over M-500 (52 m) and N-VI (60 m) a typical Nielsen lattice arrangement was adopted (Fig 20), whereas in the N-II arch a network configuration was implemented (fig 21). The need to withstand the shear stress by means of the hangers in order to attain an appropriate slenderness in the arch and the deck as well as to avoid compression in the hangers does not allow for a Nielsen typology in the latter arch, since the excessive verticality of the hangers would lead to inadmissible compression for non-symmetrical load cases.

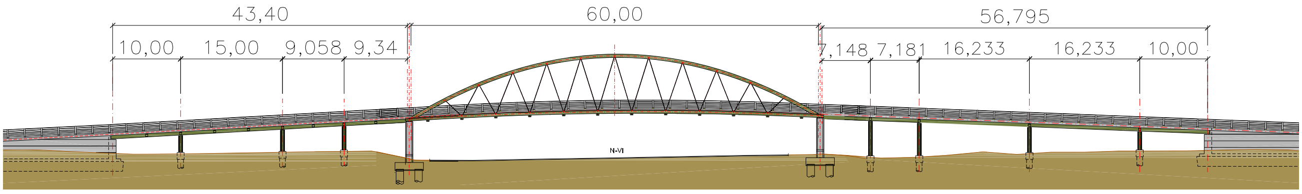

Fig.20 N-VI Footbridge (60 m).Front view

The adopted solution in this case was a system of three families of hangers on the same plane, whose anchorages were placed every 5.0 m both at the tie beams and at the arch. The crossing is dealt with by means of a 'needle-eye' device which allows for the passage of a hanger through another (Fig 22). The end anchorages of the hangers do not differ from those with a Nielsen typology.

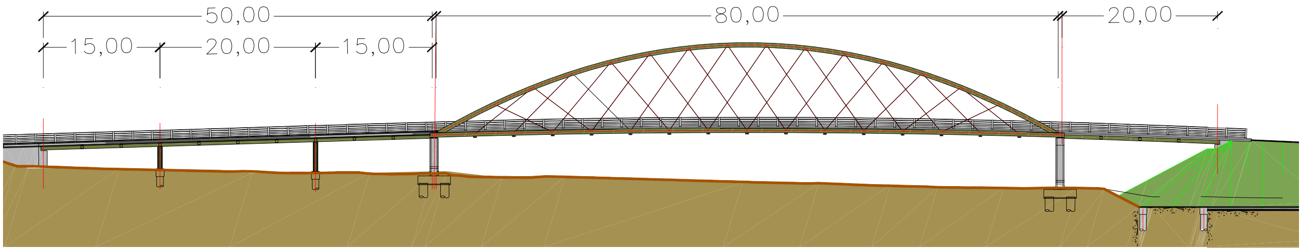

Fig.21 N-II Footbridge (80 m).Front view

The local connection between the arch and the tie beam consists of an intermediate plate (Fig 23) which, through tangential tension, transfers the compression in the arch to the tie beams.

Fig.22 N-II Footbridge (80 m). Hanger crossing device

Fig.23 N-VI Footbridge (60 m).Arch-tie beam connection

The bearings are conventional, made of reinforced rubber. The piers are inverted reinforced concrete hollow trapezes, aiming at the most visual permeability.

3. Structural behaviour

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. The maximum compression in Ultimate Limit State is 4600 kN (located at quarters of the span) and a concomitant bending moment of 200 m·kN. The worst bending moment is 570 m·kN (at the end of the arch) with a concomitant compression of 4380 kN (ULS).

As for the hangers, it is concluded that, for 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 behaviour since the hangers are in tension in all cases (axial forces of 31 kN to 142 kN in service and 13 kN to 350 kN in ULS), with a wide safety margin (ultimate load 509 kN).

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 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, the hangers' verticality does not allow them to work properly. In the light of this circumstance, it was then indispensable to resort to a network bowstring in the 80 m span footbridge.

With respect to second-order effects, the lattice configuration (both Nielsen and network) minimises 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.

4. Construction 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 (Figs 24 to 26).



Figs 24, 25 & 26 M-500 Footbridge (52 m) and approach viaducts .On-site metal works

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 27).

Fig 27 N-II Footbridge (80 m).Hoist

7) Placing of the remaining precast slabs (the same night) and reinforcement bars.
8) Concrete pouring.
9) Final touches.

The hangers tensioning was studied in order to attain the ideal state of a simply-supported arch subjected to its own weight (including the slab) and the dead load. It was concluded that the best process was to act symmetrically and starting from the middle of the structure. The applied forces ranged between 28 and 57 kN. After the first couple of operations the tie beams became detached from the temporary supports.

Given the lightness of these structures, it was possible to lift them with the precast slabs and part of the reinforcement bars already placed (in the case of the 80 m span footbridge, only a half of the slabs was placed at the moment of hoist).

By means of this industrialised process it is possible to minimise 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.

5. Conclusions

The implementation of a bowstring arch typology, either in a Nielsen arrangement or a network configuration, has been a successful choice for these three footbridges belonging to the ‘Cycling Green Ring’. With their remarkable slenderness and lightness, they constitute a solution which combines optimal structural behaviour, constructive simplicity, cost efficiency and aesthetic quality.