CONGRESSO DE
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DEVELOPMENT OF STEEL AND COMPOSITE SOLUTIONS FOR OUTSTANDING VIADUCTS ON THE SPANISH H.S.R. LINES

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Abstract: After the recent experience with the project and execution of “Arroyo las Piedras” viaduct, IDEAM is involved in the development of new steel and composite solutions for the Spanish High Speed Railway Bridges. Viaduct over river Ulla, presently at its first stages of construction, will feature the longest span in H.S.R. Composite Bridges, with a main span of 240 m, ousting the bridge over river Main in Nantenbach (208 m span). Another recent experience on H.S.R. Composite Bridges is “Archidona” Viaduct, with a main span 65 m long with only one central fixed point and a total length between expansion joints of 3150 m, the longest H.S.R. viaduct in the world without joints and expansion devices.

1 “Arroyo las Piedra”. The first composite viaduct on the Spanish HSRL

“Arroyo las Piedras” viaduct [1, 2] is the first composite steel-concrete high speed railway bridge in Spain, located on the Córdoba-Málaga H.S.R. Line (Fig. 1).

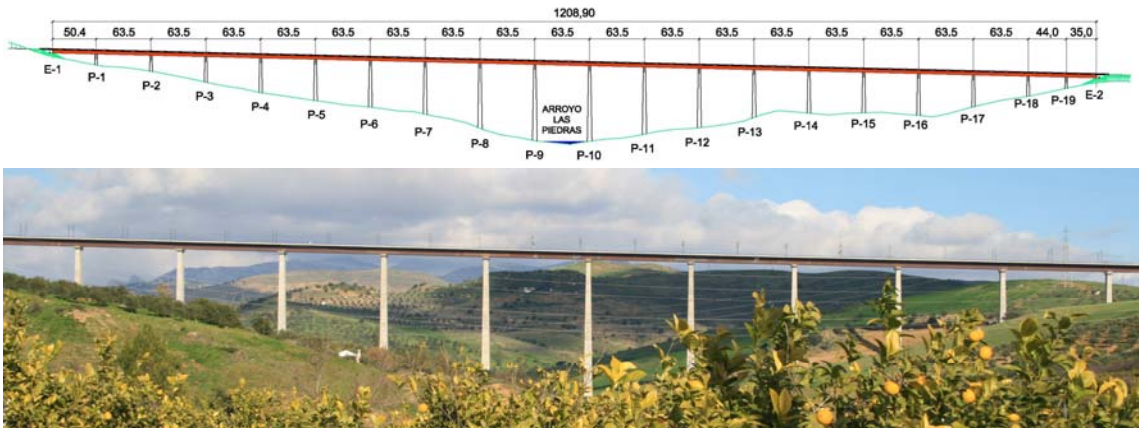


Fig. 1: Elevation view of “Arroyo las Piedras” viaduct.

The structural typology is a continuous beam with spans of 50,4+17x63,5+44,0+35,0 m. When designed and built, it was the longest-span viaduct of its type, 0,50 m longer than the

Orgon viaduct on the French TGV Mediterranée. The piers are remarkably high, since several of them exceed 93,0 m.

The cross section of the deck is made up of two 3,85 m deep twin-plate girders plus a top slab 14,0 m wide, whose thickness ranges from 0,41 m in the deck's longitudinal axis to 0,22 m at the edge of the cantilevers. Fig. 2 shows the cross section in sagging areas. It is similar to the typical twin girder solutions, although some modifications were implemented:

- Cross truss diaphragms situated every 8,0 m were used instead of full-web diaphragms.
- The bottom steel truss was replaced by a series of precast slabs 2 m long each and 0,14 m thick, longitudinally connected to each other only in the lateral cast-in-place concrete strip at the bottom corner of the cross section [1, 2]
- External inclined steel plates, placed at the top and bottom of the steel beams, replace the longitudinal web stiffeners. They improve the stability of compressed flanges and webs both in erection and service conditions.
- S-355 J2W weathering steel was used in the main structure.

The cross section in the hogging areas (Fig. 3) is similar to the sagging cross section, but double composite action is obtained by bottom in situ concreting over collaborating precast slabs. This slab is connected with studs and passive reinforcement to the main girders, and allows the maximum thickness of sheet steel in the bridge to be as little as 40,0 mm, much thinner than in classical twin girder solutions.

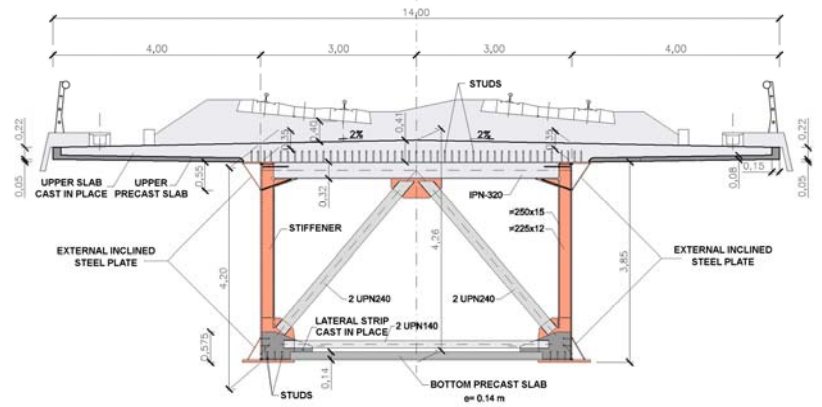


Fig. 2*:* Mid-span bending cross section

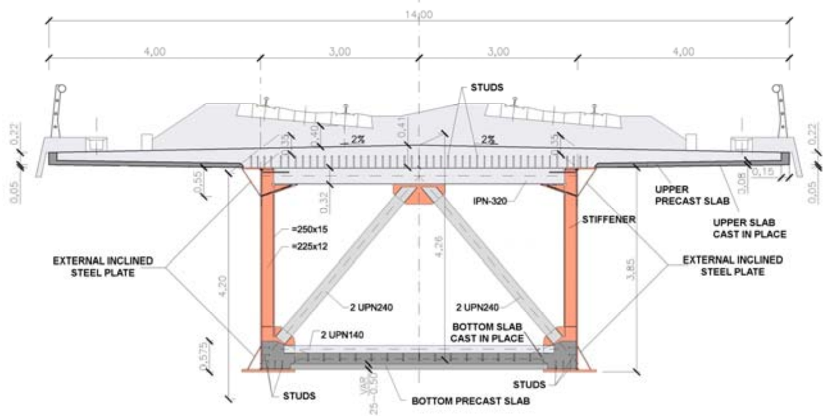


Fig. 3: Hogging cross section

The bridge was built by launching the deck from both abutments, with all the structural steel, the complete bottom concrete and the top precast slabs being present, however not yet connected to the deck. The top reinforcement steel is also placed in its final position, leaving the bridge ready once closed, to concrete the top slab (Fig. 4).

For the first time on HSRL in Spain, impact transmitters (Fig. 5) were fitted on both abutments, incorporating shock absorbing devices against seismic force, the site's acceleration was high compared with usual levels in Spain (0,10 g). Thus, slow displacements due to thermal and rheological actions were able to develop with hardly any resistance whilst, at the same time, the deck's horizontal forces due to railway stock braking are transmitted to both abutments with hardly any displacements. Besides, the force transmitted to the substructure during an earthquake is restricted to controlled values. The seismic isolation system also includes guided, sliding supports in P1 to P7, P12 to P19 and abutments, and fixed supports in two directions in the highest piers, P-8 to P-11. No isolators were arranged in the cross direction, where the flexibility called for is entrusted to the piers' elastic deformation [1, 2].

Fig. 4: Underneath view during launching **Fig. 5**: Longitudinal viscous damper-shock transmission

2 "Archidona" Viaduct.

Another recent experience of the development of high speed composite bridges is "Archidona" Viaduct (Fig. 6) located on the Córdoba-Granada H.S.R.L., with a main span 65 m long, with only one central fixed point and a total length between expansion joints of 3150 m. It is currently the longest H.S.R. Viaduct in the world without joints and expansion devices. The deck consists of a continuous double composite beam with spans of 35+29x50+2x65+30x50+35 m. The fixed points, materialised by means of a Delta-shaped pier located between the two 65-m long spans, was designed to withstand very high seismic forces.

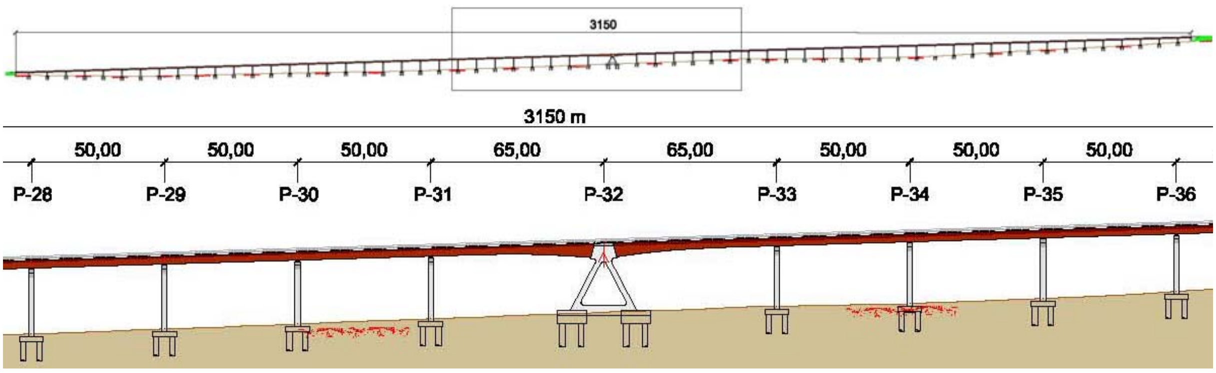


Fig. 6: Elevation view of "Archidona" Viaduct.

This project's restrictions were certainly singular:

- important seismic actions (basic seismic acceleration: 0,11 g; design seismic acceleration: 0,18 g);
- average pier height around 25 m;

- prevention, as far as possible, of placing of expansion joints within the deck, respecting the maximum longitudinal displacements of 1200 mm at the expansion joints, as defined by ADIF.

A possible solution consisting of multiple isostatic spans, generally appropriate for long viaducts with short piers, was ruled out because of the excessive deformability of the [piers+foundations] system against braking and service seism forces, whose subsequent displacements were not acceptable for the track. Besides, the substructure was penalized when withstanding the maximum design seism. Since it was not advisable, owing to maintenance reasons, to place track expansion devices within the deck, the fixed point could only be located in the middle (approximately) of the 3150 m-long deck. Therefore, the resulting lengths subjected to expansion were around 1600 m, something that could not be solved resorting to concrete deck solutions, because of the tolerable displacement range of expansion devices. Consequently, a composite deck solution helped solve this technical problem.

The deck's cross section consists of two steel twin girders (Fig 7) 2,95 m deep, with an upper concrete slab 0,40 m thick connected to the webs. The webs were 6,00 m apart at their top, slightly leaning outwards reaching 6,60 m at the bottom. A lower concrete slab closes the section and fulfils a double purpose: it provides a compression head (double composite action) in the hogging sections and it closes the torsional flux in sagging sections, a concept that had been previously implemented in "Arroyo las Piedras" Viaduct.

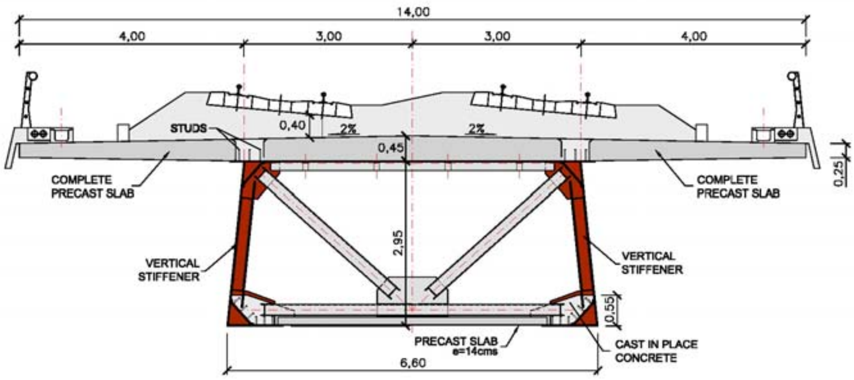


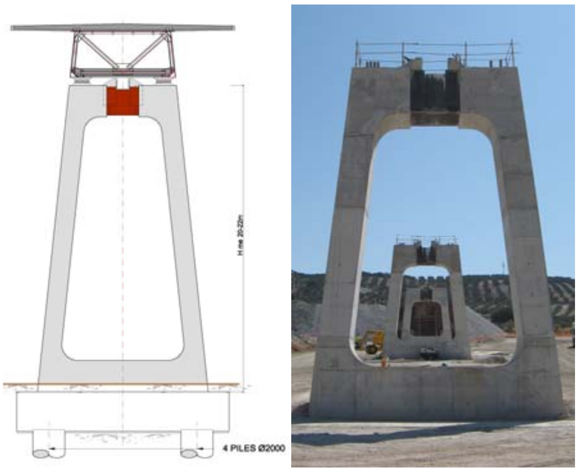
Fig. 7: Sagging cross-section

The typical pier was a frame (Fig 8) with two constant-section reinforced concrete shafts and the same inclination as that of the deck's webs. The trapeze shape renders the necessary stiffness against seismic actions. The deck is transversely linked to the pier by means of seismic stop dices, whereas it is longitudinally free and supported on sliding spherical bearings. The piers' foundations consisted of 4 piles 2,00 m in diameter and 30,00 m long

The central pier (Fig 9), which constituted the only longitudinally fixed point, was designed as a triangular cell comprising two leaning typical piers which met at the apex. The triangle's base linked the shafts' starts together as well as their pile rafts, with 14 piles each (ϕ 2,0 m and 32,0 m long). The deck, which next to the main pier experiences a slight depth increase, is embedded in the pier's head.

Owing to the maximum dilatable length, about 1600 m, continuous composite decks allow us to solve a technical problem beyond the prestressed concrete technology, since a 30-40% joint displacement reduction is attainable.

- thermal displacements barely exceed those reached in concrete solutions by 10%;
- shrinkage effects are reduced by approximately 50% as a consequence of the restraint created by the steel subsection;
- longitudinal creep deformations are inexistent because the deck is not prestressed.



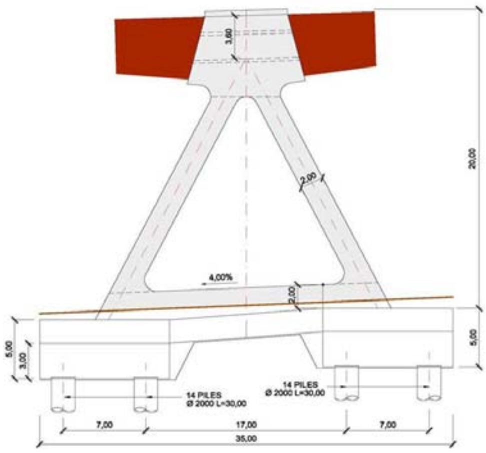


Fig. 8: Typical pier front view. **Fig. 9:** Central pier elevation.

The maximum displacements at the expansion joints located at each abutment are +594 mm gaping (deck contraction) and -386 mm closing (deck dilatation). In order to better counteract the deck's shortening (gaping joint) the following design measures were taken:

- Use of full-section precast slabs in sagging sections. By casting them in advance, shrinkage strain is reduced and it is also possible to take advantage of prefabrication in such a long structure. Only the precast slabs' joints and studs holes are cast on site. The slab in the hogging sections consists of traditional transversely ribbed precast slabs, on top which the rest of the section was cast on site.
- Reduction of rheological deformations by means of an adjustment dowel. The deck's erection was conceived in four stages, two starting from the abutments and two from the central pier. The segments connection (and, at the same time, the release of the temporary restraints at the abutments) takes place at about 750 m from both the central pier and the abutments. By erecting an intermediate adjustment span (Fig. 10), longer than the typical one, erection shrinkage can be neutralised. Total shrinkage displacement, about 270 mm, turns out to attain only 66 mm at the expansion joint.

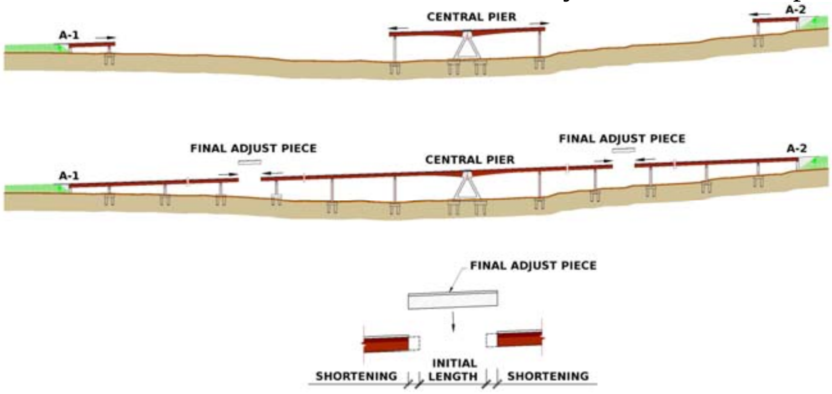


Fig. 10: Shrinkage deformation reduction by adjustment of erection process.

- A contrast study between the design temperatures and displacements put forward in design codes and the actual ones measured in "Arroyo las Piedras" Viaduct was performed, in order to control the design hypotheses considered when proportioning the expansion joints.

The central pier was designed as the unique longitudinal central fix point of the deck, and has to resist the whole longitudinal seismic reaction, with a value about 100.000 kN. That force is resisted with a foundation of 28 piles ϕ 2,0 m. Due to the structural importance, and the high magnitude of the forces, studies for characterizing the terrain were carried out for defining their properties. A geophysical study of the terrain around the central pier was developed, so as to define more accurately the seismic parameters of the ground, as well as the resisting parameters for the sizing of the piles, and the verification of ground collapse when it is subject to seismic force. Dynamic and pseudostatic finite element models were done to calibrate the good correlation between the conventional spring models used at foundation pre-sizing stage.

It is important to emphasize that the composite deck's relative axial flexibility, sensibly higher to conventional concrete box girder's, even though if the additional flexibility contributed by the eventual upper reinforced slab's cracking under seismic forces is considered, together with the flexibility due to the axial elongation of each one of the lateral stretch with 1575 m length, allow to reduce sensibly the seismic longitudinal force in comparison to the maximum value of the design Spectrum. For the fundamental longitudinal period (T=2,86 s), the seismic force obtained from the design spectrum in the central fixed point, 100.000 kN, lead to a result about 20% of the maximum value (Fig. 11). Composite deck characteristics' result, therefore, much more adapted than concrete deck solutions' so as to respond to this type of situations in long viaducts located in high seismic zones.

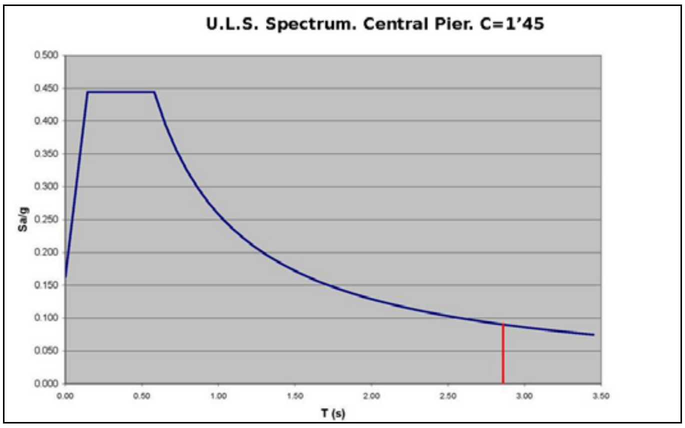


Fig. 11: Reduction of the seismic force for a longitudinal seismic action.

3 Viaduct over river Ulla.

Viaduct over river Ulla [2,3] constitutes the most audacious and high-profile intervention in the High Speed Atlantic Railway Line to Galicia and Spain's northwest regions. It is currently on its first stages of construction. Its location, close to the firth of Ulla, in a landscape of outstanding natural beauty and strong environmental constraints, was the object of a project tender among the most renowned Spanish specialists. The proposed alternative presented in this paper was finally chosen. The project constraints focused specially the following aspects:

- The outstanding nature of the project, which required serious consideration of the aesthetic qualities and viaduct integration into the landscape.
- The reduction of the number of piers in the water course, within the limits of HSR bridges, minimizing the impact on the marshes and riversides.
- The erection procedures, that being suitable to the works scale, shall be kept as independent as possible from the river watercourse, to avoid environmental damage as much as possible.

- Visual transparency and minimal bridge interference with the surrounding landscape.

All these determining factors guided the solution to a haunch steel-concrete composite lattice, with double composite work at the hogging zone, three main spans measuring 225+ 240+225 meters long, and several approaching spans measuring 120 m long, which means a main span about 20% longer than the current world record, the Nantenbach bridge in Germany, with a single 208 m long span.

The resulting viaduct is 1620 m long with a span distribution of 50 + 80 + 3x120 + 225 + 240 + 225 + 3x120 + 80 metres (Fig. 12).

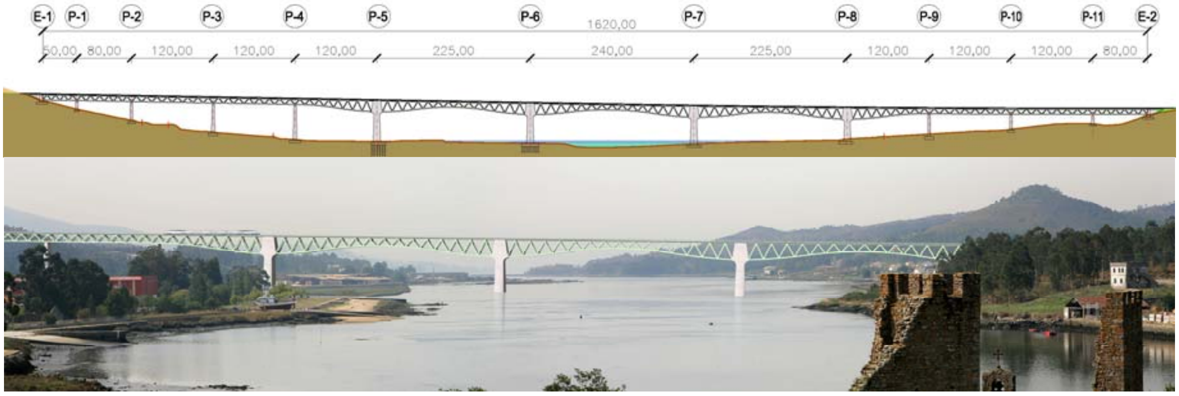


Fig. 12: View of Viaduct over river Ulla.

The structural solution of steel lattice with double steel-and-concrete composite action adequately solves the previously stated conditions.

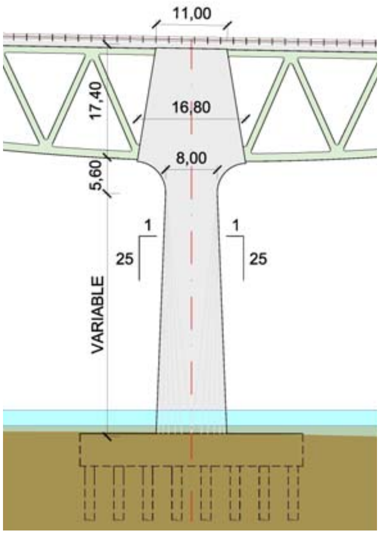
The deck was designed as a haunch lattice in the five main spans (Fig. 12), ranging from 9,15 m to 17,90 m, and as a 9,15 m constant depth lattice in the approach spans.

The four central piers, with architectonical shapes (Fig. 13), are rigidly connected to the lattice deck creating composite frames which bestow the required stiffness upon the three central spans (Fig. 14) in order to withstand the stresses arising from loads acting on alternate spans within the stringent deformation limitations in HSRL bridges.

Fig. 13: Lateral view of the main span of Viaduct over river Ulla.

The lateral piers (Fig. 15) are designed with a lighter cross-section consisting of two separate concrete shafts embedded in the deck and in the foundation. This allowed to preserve some degree of stiffness against alternate loads as well as the necessary flexibility to permit of temperature and shrinkage imposed displacements.

The structure's design, preserving the structural orthodoxy, has placed special care on the integration of shapes and geometry between the concrete piers and the deck's steel lattice. The smooth depth variation along the deck, with an upward concavity, confers a serene visual integration over the Ulla river's course. The colour choice, pearly grey for concrete and green for the lattice, enhances that effect (Fig. 13).



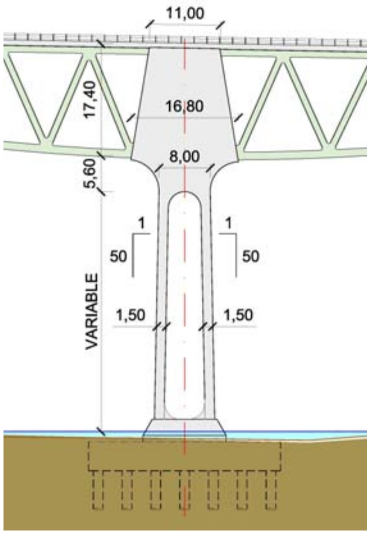


Fig. 14: View of the two inner piers P-6&P-7 **Fig. 15**: View of the two outer piers P5&P-8

Piers show a double typology well differenced. Firstly, the four main piers are rigidly connected to the deck, configuring a frame which increases the structure stiffness and enhances its behaviour regarding horizontal forces. These calyx-shaped piers are formed by a trapezium head measuring 17,5 m high and 11,00 m to 16,00 m wide, and a shaft measuring 8,00 m wide, growing with a 1H:25V slope in piers P6 and P7 (Fig. 14) and a 1H:50V slope in piers P5 and P8 (Fig. 15). The average height of the piers, measured up to the lattices' lower member, is about 42 m (60 m up to the crowning point).

The stiffness of these piers has been optimized in order to restrain deck rotations at the pier section but avoiding that bending moments taken by the pier itself and then transmitted to the foundations were a decisive design constraint.

In this way, piers P5 and P8 (Fig. 15) at the sides of 225 m spans, have been designed with two detached shafts from base to head, in order to avoid the excessive bending moments arising from the decompensation of a 225 m span next to a 120 m span, and those produced by the temperature and shrinkage displacements, both of them greater than in central piers due to their further distance to the neutral displacement point.

Side span piers P1 to P4 and P9 to P11 are of a more conventional design. Their box girder cross section with a 0,30 m wall thickness and a 3,50 m x 8,50 m head section, varies in depth both transversely and longwise. The pier height ranges from 52 m to less than 20 m.

The main spans are designed with a double haunch lattice deck, with a total depth ranging from 9.15 m at the midspan section to 17,90 m at the section over pier (Fig. 16). The lattices, which are modulated in 15 m long segments, are separated 6 m, measured between the upper flange midpoints, showing a 1H/17.5V outward slope. The adjacent spans giving access to the depth-varying main ones have been designed with constant depth.

Both the upper and lower members' cross-sections are parallelogram-shaped girders, measuring 0,80 m wide and 1,00 m deep the upper chord and 1,20 m the lower one. Diagonal members are also parallelogram-shaped (0,8 m wide and 1,00 m deep).

The upper member shows a boxed beam head embedded in the concrete slab which lodges the connection, allowing a shear transference closer to the center of gravity of the composite upper chord and minimizing the appearance of local forces and moments in the connections.

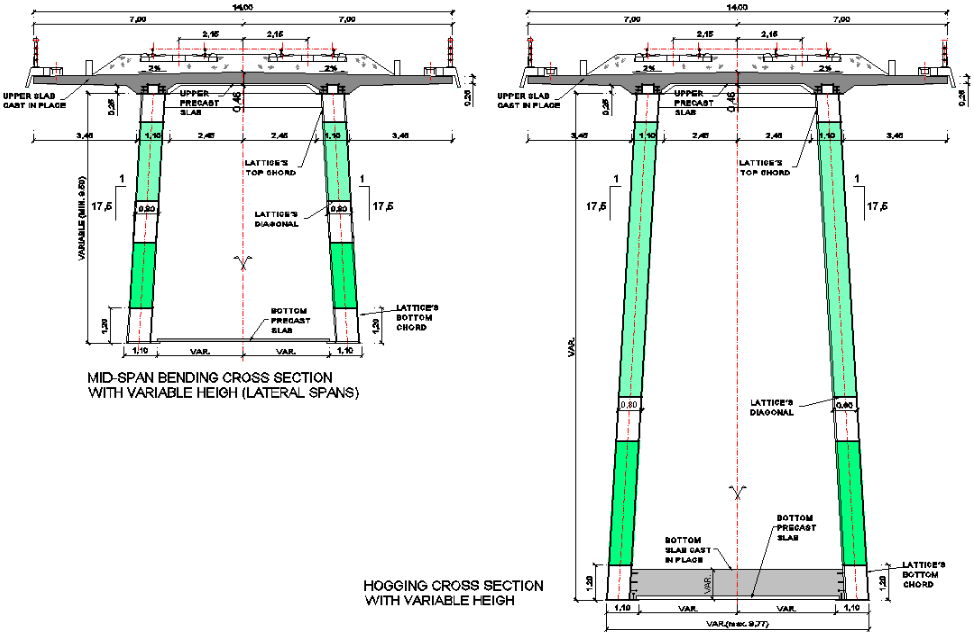


Fig. 16: Cross sections.

The steel grade is S-355-J2+N and K2+N for the approach spans and S-460-M and ML for the three main spans.

The upper slab depth is 0,46 m at mid-span and 0,25 m over the steel upper chords. The slab, made of cast in place C35/45 concrete, is poured over precast concrete slabs bridging the space between steel upper members of both lattices. The lateral cantilever part of the slab is cast using a movable formwork.

Along the hogging zone, a C50/60 concrete slab is arranged between members, allowing a double composite action. The thickness of this lower slab ranges from 0,30 m to 1,10 m. Along the sagging zone, the deck's lower face is closed visually using precast concrete plates, with no structural role but to create a path to allow for extremely easy inspection and maintenance.

4 "Abroñigal" Viaduct.

Certain circumstances advice that steel solutions be used, for example with skewed lower crosses without enough clearance. That happens in "Abroñigal" Viaduct, near Atocha HSR Station in Madrid, where there are multiple crossings. That laid the design of a double bowstring arch bridge with a Nielsen suspension system. The arch bridge spans 91 m, with a rise of 14.50 m (Figs. 17, 18). The hangers' anchorages at the tie beams are spaced about 10.10 m (which is also the separation between the transverse girders). Both the arches, tie beams, hangers and transverse beams are made of S-355 structural steel, with S-460 steel for the arch-tie beams joint webs. The transverse girders are composite and connected to the concrete floor.

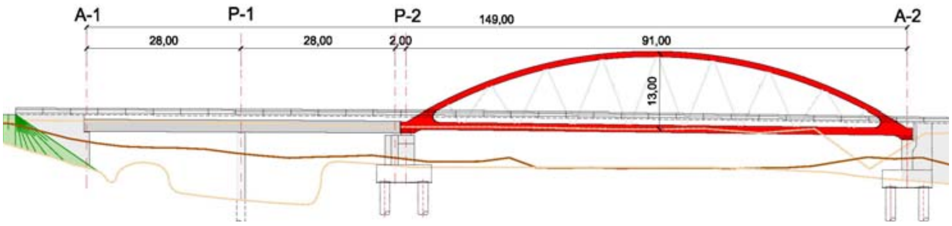


Fig. 17: Elevation and cross section of the Viaduct.

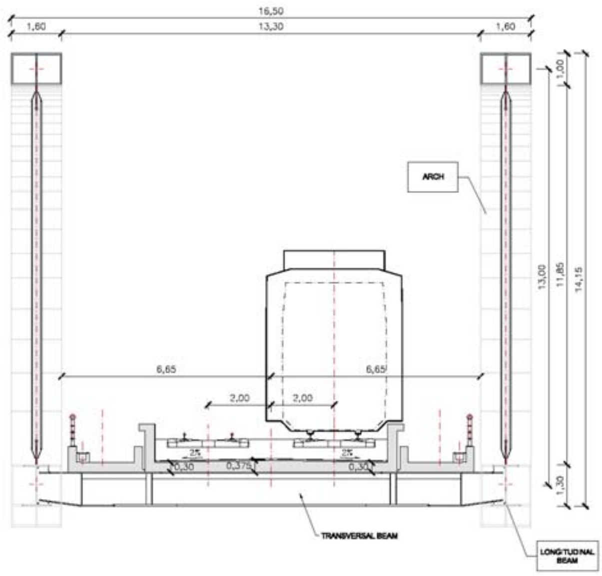


Fig. 18: Elevation and cross section of the Viaduct.

One of the most remarkable features is the chosen solution for the deck’s superstructure, conceived as an alternative to the usual subsystem of longitudinal steel girders where the precast slabs rest, and on top of which the slab is cast. Precast U-shaped beams were used instead. Those used for the track infrastructure and railway traffic are 8,0 m wide, while those for the sidewalks are 2,345 m wide.

Both kinds of beams have holes located at the support section, right where the transverse girders’ studs are. The concrete beams are erected isostatically, but those for the track are linked together by means of a steel tube welded to a plate at the support section. The total dead load is similar to the case of a slab across the whole width.

This system avoids the use of longitudinal steel between the transverse girders and increases the separation between the latter. However, monolithic response as a composite beam must be developed in a very short distance. Most of the load is carried through the concrete web of the track U-beam. Therefore, the maximum bending moment (almost constant along the transverse girder) takes place very near the intersection of the transverse beam and the tie beams. This leads to very high tangential stresses, which affect several design aspects: stud layout, concrete strength, concrete-stud transfer reinforcement and transverse girder working as a structural steel subsection alone.

Concerning the traffic retaining, this function has not been trusted to the U-beam’s webs, but to a third rail or counter-rail.

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