OUTSTANDING COMPOSITE STEEL-CONCRETE BRIDGES IN THE SPANISH HSRL

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Abstract: A wide programme for development of HSRL infrastructure is being carried out in Spain. Concrete solutions were always chosen for the first HSRL. Shortly ago, the proposal for a composite deck for the Arroyo de las Piedras viaduct was accepted by ADIF (National Railway Network Administration Office). Finished in 2005, with a standard span of 63.50 m, slightly longer than the 63.00 m in the Orgon viaduct, at the French TGV, being the longest span for this typology. The French twin plate girder solution was modified according to the Spanish double composite action technology. Its success has opened the way for the acceptance of more composite steel and concrete solutions for the Spanish HSRL projects. The new viaduct over the Ulla river is the most representative one of all of them. The Ulla viaduct is 1620 m long, with three main spans of 225+240+225 m in length and several approaching spans of 120 m long each, which means a main span about 20% longer than the current world record, the Nantenbach bridge in Germany. The solution is a composite truss bridge with double composite action at the hogging zone. The depth ranges from 9.15m to 17.90 m in the five main spans, in which the deck is rigidly connected to the four central concrete piers, creating composite frames.

1. "ARROYO DE LAS PIEDRAS" VIADUCT: THE FIRST COMPOSITE BRIDGE IN SPANISH HSRL

Arroyo de las Piedras bridge [Ref. 1, 2] is the first composite steel-concrete high speed railway bridge in Spain, located on the Córdoba-Málaga HSRL. The structural typology is a continuous beam with spans of 50.4 + 17 x 63.5 + 44 + 35 m *(Fig. 1)*. 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 m *(Fig. 2)*.

Fig. 1: View of the bridge

Fig. 2: View of the central piers

1.1. Description of the bridge

The cross section of the deck is made up of two 3.85 m deep twin-plate girders plus a top slab 14 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. 3* 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 m were used instead of full-web diaphragms, thus facilitating on-site assembly and significantly reducing the weight of steel and volume of welding required.

- 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, as described later.

- 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 J2G2W weathering steel was used in the main structure. This steel is very appropriate for the Andalousian atmospheric conditions, whilst a reduction is achieved in the cost of maintaining the carbon steel used in the inner trusses.

The cross section in the hogging areas *(Fig. 4)* is similar to the sagging cross section, but double composite action is obtained by bottom in situ concreting over collaborating precast slabs. The bottom slab extends 13.90 m on both sides of the pier in 63.50 m spans. Its thickness varies from 0.25 to 0.50 m. 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 mm, much thinner than in classical twin girder solutions.

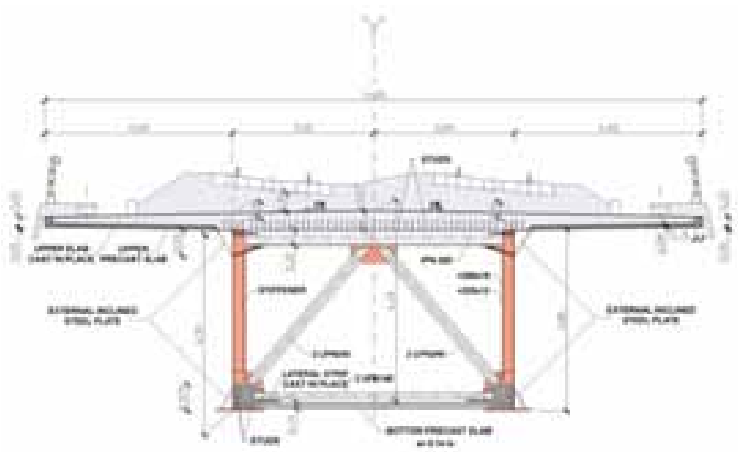

Fig. 3*:* Mid-span bending cross section (Units: m)

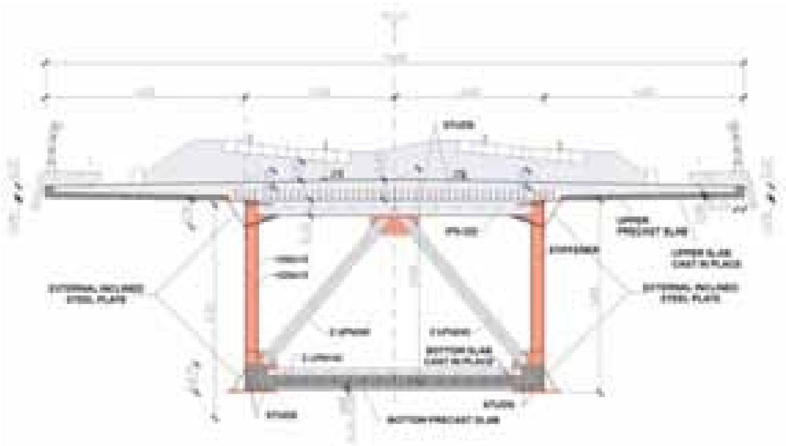

Fig. 4: Hogging cross section (Units: m)

1.2. The double composite action generalized for torsion

The twin girder solution for road bridges had to be improved by increasing the torsional stiffness in order to meet HSRL requirements. The double composite action was extended to the whole length of the deck to allow the torsion flow to be closed. The box cross section is obtained in sagging areas with the use of discontinuous precast slabs connected to the girders. When eccentric loads are applied the bottom discontinuous precast slab acts as a Vierendeel system, developing shearing and bending in its plane and allowing the torsional flow to close.

The slabs are 0.14 m thick and 2 metres long, and are only connected to each other at the lateral 1m wide cast-in-place concrete strip, in order to transfer the torsion flow between girders and slabs. (*Fig. 5)* shows the structural response of the discontinuous slab system between two successive diaphragms subjected to torsion. Finally, a 66% reduction of the bridge’s torsional stiffness was achieved compared to what a continuous slab would provide. The design of the bridge was drawn up under the conservative hypothesis of total cracking of tension areas. In this case the torsional stiffness obtained is approximately 18% of the crackless state, which corresponds to 12 % of the stiffness for the crackless continuous slab. A static and dynamic analysis of the bridge using these values guaranteed an adequate deck response. Even under this extreme cracking hypothesis, the equivalent thickness obtained for the bottom slab was about 1,4 mm, similar to that obtained with the classical bottom steel trusses, usually between 0.8 and 2.0 mm.

The bottom slabs also respond well in the sagging areas. The absence of a transverse connection between them, beyond that provided by the lateral cast-in-place strip, prevents the development of a relevant effective collaborating tensile width. Tensile stresses in the bottom flange are less than 130 N/mm^2 under dead loads and 300 N/mm^2 under the worst live loads and the connection between bottom flanges and the lateral cast-in-place strip and the bottom longitudinal reinforcement located in the latter are enough to control cracking in this area.

The lateral cast-in-place strip extends over the whole width of the section in hogging areas to provide the double composite action. Compression stresses from bending keep the bottom slab crackless, so bending and torsional stiffnesses in these areas are noticeably higher than those classically obtained with steel sections. Double composite action greatly improves the deformational and dynamic response both to bending and torsion.

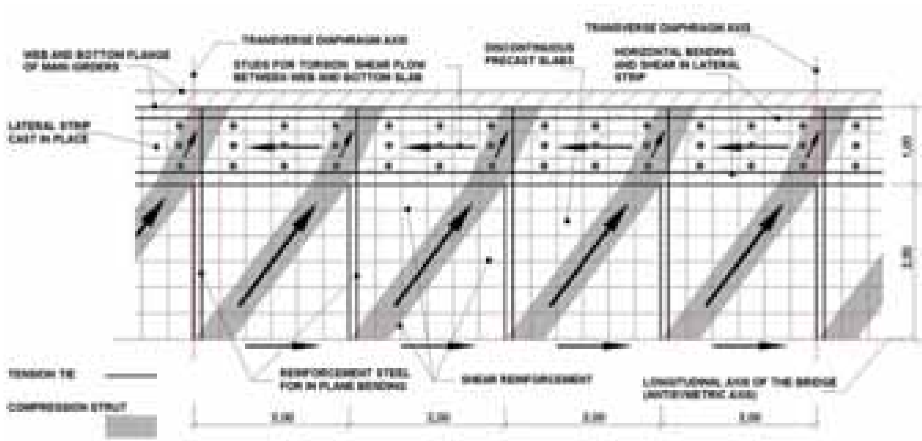

Fig. 5: Torsional response of bottom discontinuous pecast slabs and lateral strips cast in place

1.3. Description of the construction process

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 *(Figs. 6* and *7)* 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. 6: View of the upper precast slabs

Fig. 7: View of the bottom concrete slab

The deck could be visited even during the launching phases to facilitate the delivery of auxiliary materials to the piers in a safe, simple way inside the deck. The first span only included structural steel with no top or bottom prefabricated slabs in order to prevent excessive stresses during launching in the cantilever stages. In a single phase up to 127 m of deck was launched each time from each abutment *(Fig. 8)*.

Fig. 8: Underneath view of the viaduct during

The theoretical deflection of a cantilever arriving at a pier in a typical 63.5 m span is 1.03 m, as opposed to actual measures at the pier's arrival of 0.98 m and 0.94 m, with errors of less than 10%.

1.4. Patch loading response of webs during launching

Reaction forces on sliding supports during launching reached 4250 kN per support. These values exceed those frequently obtained in launching steel bridges. The web thickness is 25 mm at sagging areas and the vertical web stiffeners were arranged every 4 m.

Standards give different regulations for the patch loading response. However, none of them provides useful design criteria when the cross section differs from a classical I beam deck. The bottom inclined steel plate and the lateral cast-in-place concrete strip *(Fig. 7)* clearly improve the patch loading response compared to the standards.

Both systems together provide additional bending stiffness to the bottom flange, consequently increasing the web's membrane stiffness. Besides, the concentrated loads spread over a greater length along the web.

The bottom flange with the inclined corner plate and the lateral concrete strip cast in place *(Fig. 3, Fig. 4)* constitute a longitudinal beam with a bending capability between the web's transversal stiffeners. A significant percentage of the vertical load, around 35 %, is directly transferred by shear stress in the bottom plate to the adjacent vertical stiffeners, as a beam bending in a longitudinal direction. The global improvement in ultimate patch loading resistance is about 70% as against the standard calculations of an "I" section. These design improvements are not taken into consideration in the aforesaid standards, so a non linear finite element analysis had to be carried out.

Additionally, some on-site checking of the actual web response with concentrated loads was performed before the beginning of the launching process. A safe response of the web during launching was confirmed, much better than that obtained by analytical methods.

1.5. Seismic design

The deck's longitudinal support condition was a fundamental problem in designing the bridge. Conditions were truly highly restrictive:

- The site's seismic acceleration (0.10 g) was high compared to usual levels in Spain.

- The bridge was too long (1208 m) for just one fixed point in one abutment, because of the total displacement accumulated in the opposite one.

- The pier height and foundation conditions were inappropriate for intermediate longitudinal fixed supports.

For the first time on HSRL in Spain, impact transmitters were fitted on both abutments, incorporating shock absorbing devices against seismic force *(Fig. 9)*. 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.

Fig. 9: Longitudinal viscous damper-shock transmission

Under railway service forces, the shock absorbers act as impact transmitters to longitudinal forces and are the bridge's main supports in this direction, so their design forces correspond to the longitudinal force due to braking and longitudinal wind. Four shock absorbers are fitted in each abutment with 2200 kN of maximum force each. In our case, devices capable of absorbing a displacement of ± 350 mm due to rheological, thermal and seismic origin were provided.

In a transverse direction, the maximum seismic force transmitted by the deck to the head of one pier reached 4500 kN. Anti-earthquake stops were designed under the deck on the pier heads to transmit transverse forces while allowing longitudinal displacement. In addition to this, bearings were designed to resist one third of the transverse force due to the design earthquake, equivalent to an in-service earthquake.

2. THE ULLA RIVER VIADUCT

The new viaduct over the Ulla river [Ref. 3] constitues the most audacious and high-profile intervention in the High Speed Atlantic Railway Line to Galicia and Spain's northwest regions. Its construction will begin after the 2008 summer. 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 HSRL 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.

2.1. Structural concept

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. 10*), ranging from 9.15 m to 17.90 m, and as a 9.15 m constant depth lattice in the approach spans.

Fig. 10: View of Viaduct over river Ulla

The four central piers, with architectonical shapes (*Fig. 11*), are rigidly connected to the lattice deck creating composite frames which bestow the required stiffness upon the three central spans (*Fig. 12*) in order to withstand the stresses arising from loads acting on alternate spans within the stringent deformation limitations in HSRL bridges.

Fig. 11: Lateral view of the main span of Viaduct over river Ulla

The lateral piers (*Fig. 13*) 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.

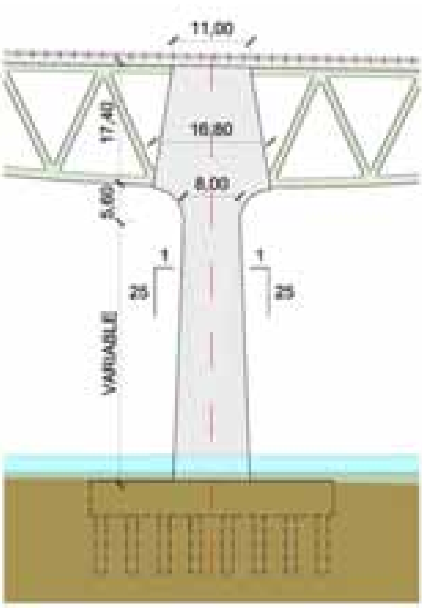

Fig. 12: View of the two inner piers P-6 and P-7

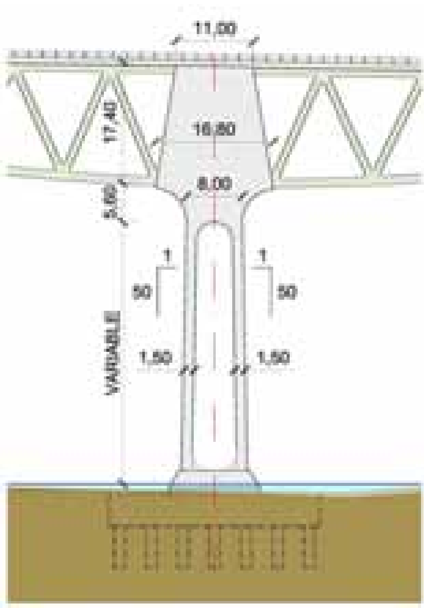

Fig. 13: View of the two outer piers P-5 and P-8

2.2. Description of the bridge

The resulting viaduct is 1620 m long with a span distribution of 50 + 80 + 3x120 + 225 + 240 + 225 + 3x120 + 80 metres. (*Fig. 14*)

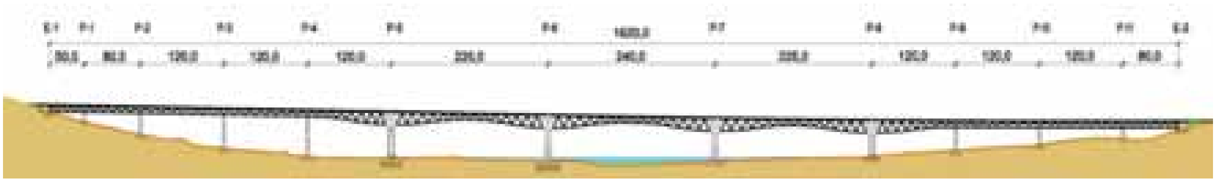

Fig. 14: Elevation view of the viaduct

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. 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. (*Fig.15*).

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, with main dimensions of 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.

Plate thickness is variable and, as a rule of thumb, plates thicker than 80 mm have been avoided. The steel quality is S-355-J2 for the approach spans and S-460-M and ML for the three main spans.

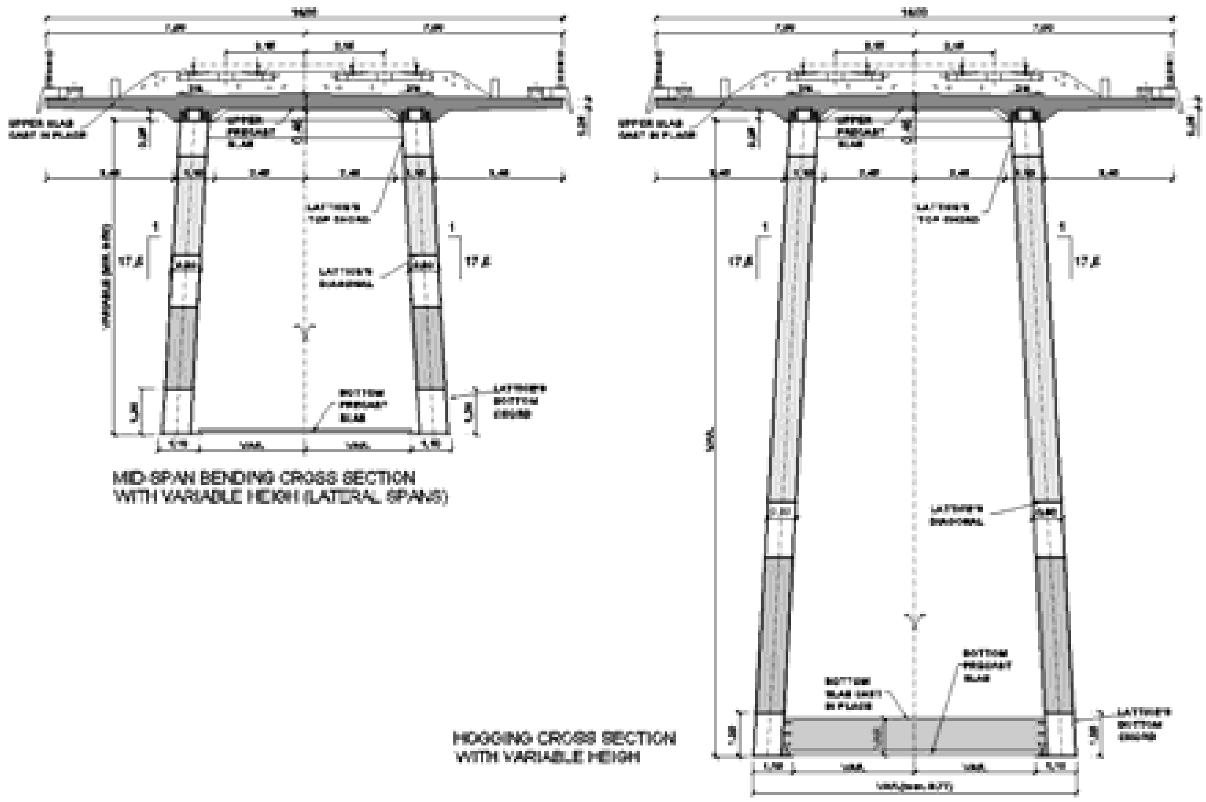

Fig. 15: Cross sections

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.

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 and a 1H:50V slope in piers P5 and P8. 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 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 transversally and lenghtwise. The pier height ranges from 52 m to somewhat less than 20 m.

The latter piers and both abutments are crowned with two spherical bearings, totally free one of them and transversally restrained the other. All the bearings will be disposed with the sliding surface horizontal, excepting the A1 abutment ones, which shall be disposed following the i=-1.8% deck slope, in order to avoid undesired displacements at the expansion join devices. At the A2 abutment, due to the weak slope, the sliding surface will be kept horizontal.

Piers foundation is supported by the existing granite substratum by direct foundation, excepting piers P5 and P6 where the alluvial deposit thickness prevents the use of footings and forces the use of a 1.5 m diameter pile foundation.

2.3. Erection procedure

The chosen procedure to construct the viaduct shall minimize the river and environmental affection. P5 pier is located in the tidal range of the firth, close to the Tellería islet but outside it. A precast hollow caisson based access shall be constructed, allowing a water flow trough it. Once the works have been completed, the caissons shall be withdrawn leaving the firth in its original conditions.

P5 to P7 pier foundations are planned to be built with an enclosure perimeter made up of bulk rockfill and whose interior shall be filled with granular material in order to provide a dry and stable platform. As piers P6 and P7 shall be accessed by means of boats, a docking facility must be built using a sheet piling curtain.

All the works carried out in the firth for the pier P5 to P7 foundations and the access path between P4 and P5 shall use a protective suspension particle isolating barrier so that all potential damage to the water course is avoided.

Simultaneously the steel lattice wil be built at each one of the three assembly yards. The steel will arrive from the workshop in small pieces (box members and joints) and the transportation could be made by conventional means.

The constant depth spans will be built by lifting segments, using temporary piers, and only occasionally by means of cranes in those cases where the access will be difficult. The segment lenght to be lifted ranges from 25.00 to 40.00 m, depending upon the position of the temporary piers. The lifting will be done simultaneously on both lattices, so once the final height is reached a simple transverse displacement puts the deck piece on its final position.

The voussoirs of the haunch spans will be assembled at the building site with the pieces arriving from the workshop, in complete modules measuring 15.00 m long and comprising both lattices. The transportation to its final placement will be made by means of a special platform with multiple axes, accessing the pier base. The voussoirs of piers in water course P6 and P7 are transportated similarly shipping them in a boat.

Once the voussoir has arrived to the pier base, a gantry crane picks up the module close to the pier shaft, translating it to its final position and lifting it to be welded in place. So far, the steel lattices are erected by a succesive cantilever method, from the pier section to the closing key section at midspan. This method ensures independent work at the bridge from the marshes, river and surrounding vegetation.

Once the assembly of the steelwork has been completed, the lower precast concrete slabs will be placed and the subsequent lower slab concrete casting done. The next stage will be the temporary piers removal from the side spans and a simultaneous and controlled downward displacement of 0.25 m of the deck at piers P4 and P9 sections.

The upper concrete slab 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.

Finally, the bulk rokfill protective embankment of piers P6 and P7 will be removed, replacing it with local material from the affected sourroundings. The access path built with concrete caissons will also be removed and the corresponding corrective measures will be taken.

3. OTHER COMPOSITE STEEL AND CONCRETE HSRL VIADUCTS

Nowadays several composite steel-and-concrete solutions have been chosen over the more conventional concrete alternatives in Spanish HSRL. IDEAM itself is now involved in the project stage of several outstanding steel-and-concrete composite bridges, some of which are to become the most emblematic in HSRL in Spain.

- Kinatoi Viaduct (Basque Y): 65 m long spans and 95 m high piers, similar to “Arroyo Las Piedras Viaduct” before described.
- Ibaizábal Viaduct (Basque Y): 83.50 m long spans (*Fig. 16*).

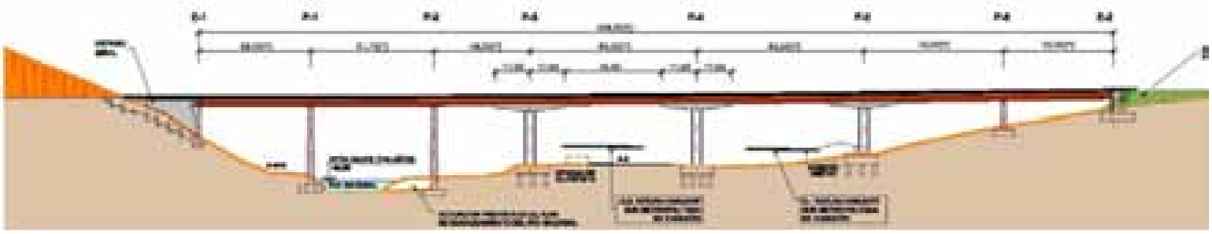

Fig. 16:Elevation of Ibaizábal Viaduct

- Bergara Viaduct (Basque Y): main span 180 m long (*Fig. 17*).

Fig. 17: View of Bergara Viaduct.

- Archidona Viaduct (Cordoba-Granada HSRL): main span 80 m long with central fixed point and a total length between expansion joints of 3115 m (*Fig. 18*).

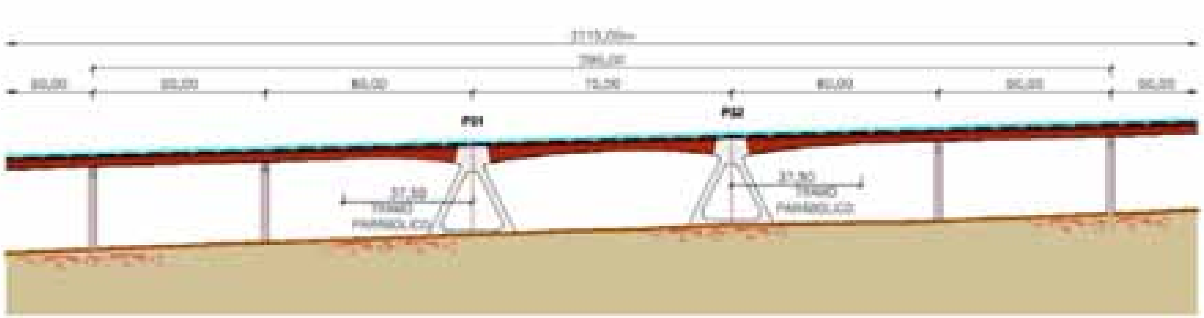

Fig. 18:Elevation of Archidona Viaduct

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