Outstanding Composite (Steel and Concrete) High Speed Railway Bridges in Spain. Design aspects focused on sustainability

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ABSTRACT: During the last 15 years IDEAM has been involved in the development of new steel and composite (steel and concrete) solutions for several Spanish High Speed Railway Bridges. Viaduct "Arroyo Las Piedras", concluded in 2006, was the first composite HSR bridge in Spain, with a main span of 63.5 m. Another recent experience is "Archidona" Viaduct, concluded in summer 2011, with a 50 m long typical span and only one central fixed point, with an overall length between expansion joints of 3150 m, the longest H.S.R. viaduct in the world without joints or track expansion devices. Viaduct over river Ulla, presently under construction, will feature the longest span in HSR Composite Bridges, with a main span of 240 m, ousting the bridge over river Main in Nantenbach (208 m span).

1 "ARROYO LAS PIEDRAS". THE FIRST COMPOSITE VIADUCT ON THE SPANISH HIGH SPEED RAILWAY LINES

"Arroyo lasPiedras" viaduct (Millanes, F. et al. 2007) is the first composite steel-concrete high speed railway bridge in Spain, located on the Córdoba-Málaga HSR Line (Fig. 1).

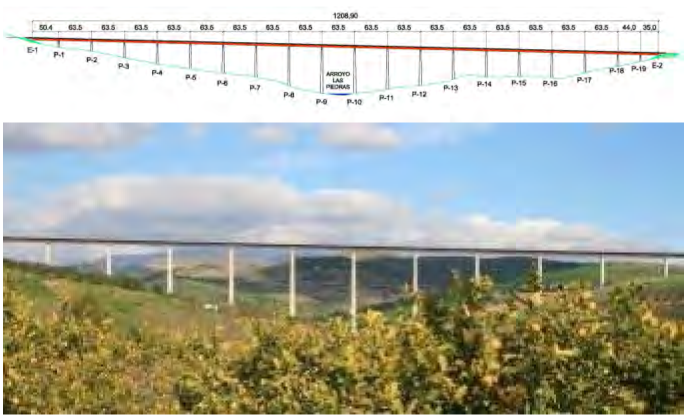

Figure 1. Elevation view of "Arroyo lasPiedras" viaduct

The structural typology is a continuous deck with spans of 50.4+17 × 63.5+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 at the deck's center line to 0.22 m at the edge of the cantilevers. Figure 2 shows the cross section in sagging areas. It is similar to the typical twin girder solutions, although some modifications were implemented:

- 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 strips at the bottom corners of the cross section (Millanes, F. et al. 2007)
- 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 attained by means of an in situ bottom concrete slab poured onto collaborating precast slabs. This slab is connected with studs and passive reinforcement to the main girders, and allows the maximum thickness of steel plates in the bridge to be as little as 40.0 mm, much thinner than in classical twin girder solutions.

The bridge was erected by launching the deck from both abutments, with all the structural steel, the bottom concrete slabs 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).

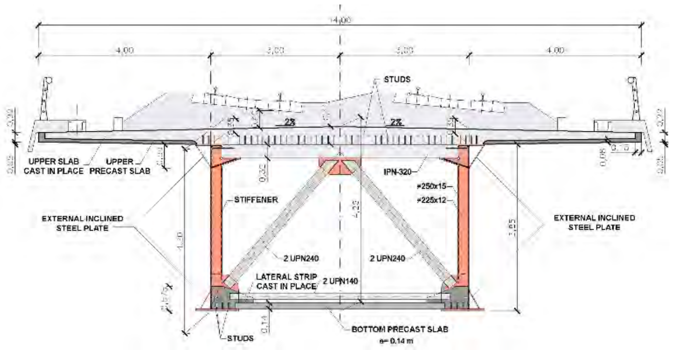

Figure 2.Sagging cross section

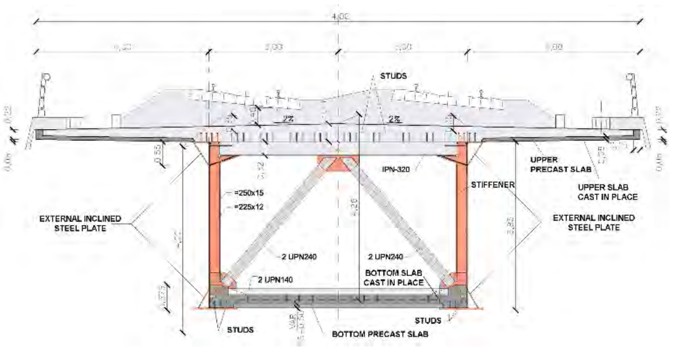

Figure 3. Hogging cross section

Figure 4. Underneath view during launching

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 to usual levels in Spain (0.10 g). Thus, slow displacements due to thermal and rheological actions could take place 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 little 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 required flexibility is entrusted elastic deformation of the piers.

Figure 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 2 main spans 65 m long, with only one central fixed point and an overall length between expansion joints of 3150 m. It is currently the longest High Speed Railway Viaduct in the world without joints and track expansion devices. The deck consists of a continuous double composite beam with spans of 35+29 × 50+2 × 65+30 × 50+35 m. The fixed point, materialised by means of a Delta-shaped pier located between the two 65-m long spans, was designed to withstand very high seismic forces.

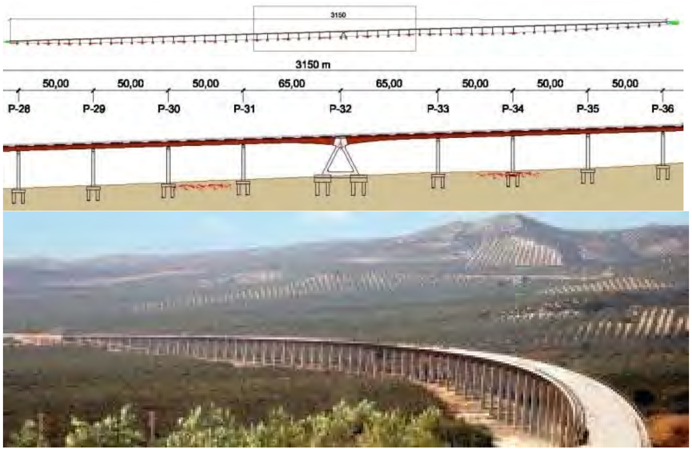

Figure 6. Elevation view of "Archidona" Viaduct, and picture of the Viaduct after its completion

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 the Spanish Railway Administration (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 seismic forces, whose subsequent displacements were not acceptable for the track. Besides, the substructure was penalized when withstanding the maximum design earthquake. Since it was not advisable, owing to maintenance reasons, to place track expansion devices on 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 flow in sagging sections, a concept that had been previously implemented in "Arroyo lasPiedras" Viaduct.

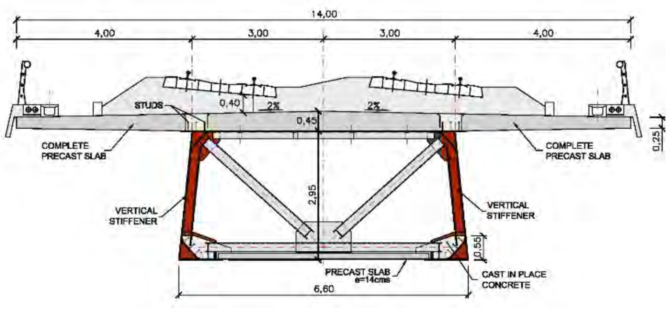

Figure 7. Sagging cross-section

The typical pier was a frame (Fig. 9) with two constant-section reinforced concrete shafts and the same inclination as that of the deck's webs. The trapezium shape renders the necessary stiffness against seismic actions. The deck is transversely linked to the pier by means of seismic stoppers, whereas it is longitudinally free and supported on sliding spherical bearings. The foundations of the piers consisted of 4 piles 2.00 m in diameter and 30.00 m long.

The central pier (Figs. 8 & 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 (ϕ2.0 m and 32.0 m long). The deck, which next to the main pier features a slight depth increase, is embedded in the pier's head.

Owing to the maximum dilatable length, about 1600 m, a continuous composite deck allowed 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 induced by the steel subsection;
- longitudinal creep deformations are inexistent because the deck is not prestressed.

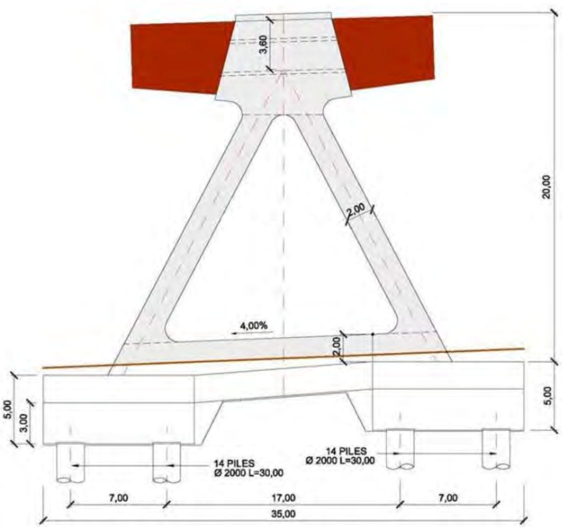

Figure 8. Central pier elevation

Figure 9. View of the central and typical piers of the Viaduct

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 the holes for stud groups are cast on site. The slab in the hogging sections consists of traditional transversely ribbed precast slabs, on top of 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.

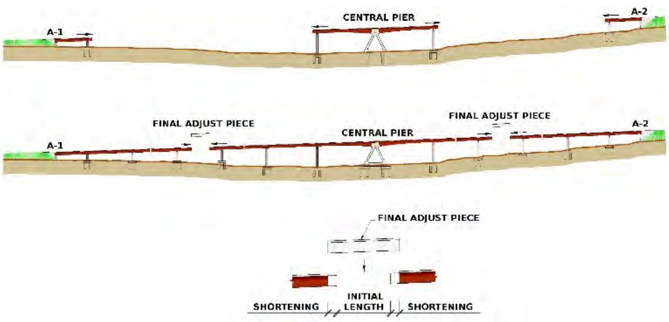

Figure 10. Shrinkage deformation reduction by adjustment of erection process

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

The central pier (Fig. 9) was designed as the only longitudinal fixed 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, soil studies were carried out in order to determine its properties. A geophysical study of the soil around the central pier was performed, so as to define the seismic parameters of the ground more accurately, as well as the resisting parameters of the piles, and to analyze ground collapse when it is subjected to seismic force. Dynamic and pseudostatic finite element models were created to calibrate the good correlation between the conventional spring models used at the foundation pre-sizing stage and more accurate methods.

It is important to emphasize that the composite deck's relative axial flexibility, somewhat higher than that of conventional concrete box girders, moreover if the additional flexibility induced by the eventual cracking of the reinforced upper slab under seismic forces is considered, together with the flexibility due to the axial elongation of each lateral stretch, 1575 m long, allows 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, leads to a result about 20% of the maximum value (Fig. 11). Composite decks turn out to be, therefore, more suitable than concrete deck solutions when responding to this type of situations in long viaducts located in highly seismic zones.

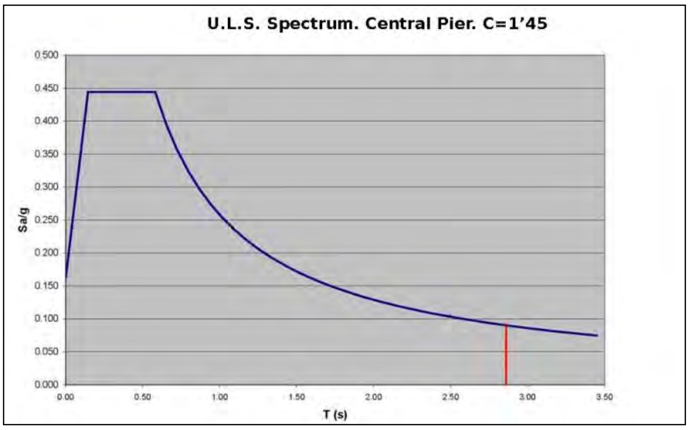

Figure 11. Reduction of the seismic force for a longitudinal seismic action

3 VIADUCT OVER RIVER ULLA

3.1. *Description of the bridge*

Viaduct over river Ulla constitutes the boldest and high-profile undertaking in the High Speed Atlantic Railway Line in Galicia, in northwestern Spain (Millanes, F. et al. 2008). It is currently under construction and will be finished in summer 2014. Its location, close to the firth of Ulla, a landscape of outstanding natural beauty and strong environmental constraints, was the object of a project tender among the most reputable Spanish specialists. The proposed alternative presented in this paper was finally chosen. The project constraints focused especially on the following aspects:

– The outstanding nature of the project, which required serious consideration of the aesthetic qualities and viaduct integration.
– The reduction of the number of piers in the water course, minimizing the impact on the riversides.
– The erection procedures, being suitable to the works scale, had to be kept as independent as possible from the inlet's course, to avoid environmental damage as much as possible.
– Visual transparency and minimal bridge interference with the surrounding landscape.

All these factors guided the solution to a haunched steel-concrete composite lattice, with double composite action at the hogging zone, three main spans 225+ 240+225 meters long, and several 120 m long approach spans. The main span is 20% longer than the current world record, the Nantenbach bridge in Germany, with a main span of 208 m.

Figure 12. View of Viaduct over river Ulla

The resulting viaduct is 1620 m long with a span distribution of 50 + 80 + 3 × 120 + 225 + 240 + 225 + 3x120 + 80 metres (Fig. 12).

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

The deck was designed as a haunched lattice in the five main spans (Fig. 12), their depth 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 architectonic shapes (Fig. 13), are rigidly connected to the lattice deck creating composite frames which bestow the required stiffness upon the three central spans in order to withstand the stresses arising from loads acting on alternate spans within the stringent deformation limitations in HSRL bridges.

Figure13. Lateral view of the main span of Viaduct over river Ulla

The lateral piers P-5 and P-8 (Fig. 15) were 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 allow for temperature and shrinkage imposed displacements.

The structure's design, preserving the structural orthodoxy, 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 appearance over the Ulla river's course. The colour choice, pearly grey for concrete and green for the lattice, enhances the effect.

Figure14. View of the two inner piers P-6&P-7

Figure 15.View of pier P-5 while erecting a segment

Piers distinctly show a double typology. Firstly, the four main piers are rigidly connected to the deck, configuring a frame which increases the structure's stiffness and improves its behavior regarding horizontal forces. These calyx-shaped piers are formed by a trapezium head 17.5 m high and 11.00 m to 16.80 m wide, and a shaft 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 was optimized in order to restrain deck rotations at the pier section but preventing that bending moments taken by the pier itself (and then transmitted to the foundations) from becoming a decisive design constraint.

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

Lateral side span piers P1 to P4 and P9 to P11 are of a more conventional design. Their hollow-box cross section, with a 0.30 m thick wall and a 3.50 m x 8.50 m cross-section at the top, 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 haunched lattice deck, with a total depth ranging from 9.15 m at the midspan section (Fig. 16) to 17.90 m at the pier section (Fig. 17). The lattices, modulated in 15 m long segments, are 6 m apart, measured between the upper flange midpoints, featuring a 1H/17.5V outward slope. The adjacent spans giving access to the haunched main ones were designed with constant depth.

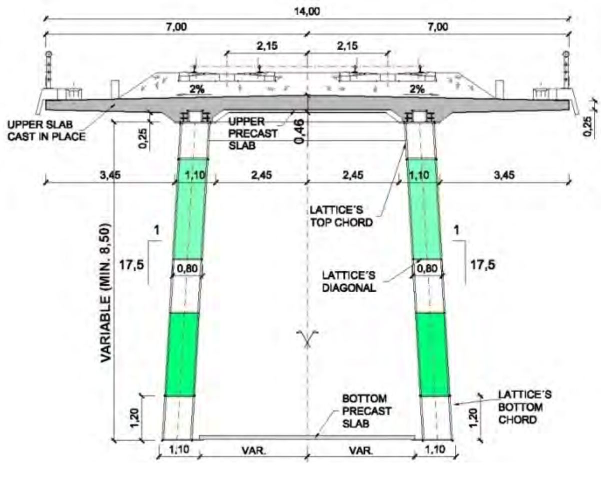

Figure 16. Sagging cross sections

Both the upper and lower members' cross-sections are parallelogram-shaped girders, 0.80 m wide, 1.00 m deep the upper chord and 1,20 m deep the lower one. Diagonal members are also parallelograms (0.8 m wide and 1.00 m deep).

The upper member features a box-like head embedded in the concrete slab lodging 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.

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 thickness is 0.46 m at the center line and 0.25 m over the steel upper chords. The slab, made of cast-in-place C35/45 concrete, is poured on precast concrete slabs spanning the space between the upper chords of both lattices. The lateral cantilevers of the slab are cast using a movable formwork.

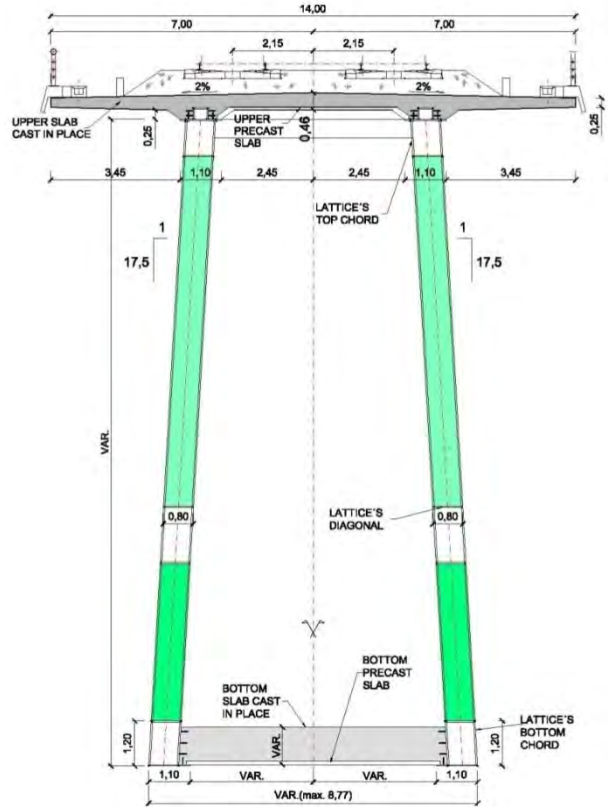

Figure 17. Hogging cross section in the variable depth central zone

Along the hogging zone, a C50/60 bottom concrete slab is arranged between members, thus allowing for 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 visually closed using thin precast concrete slabs, with no structural role but to create a path to allow for extremely easy inspection and maintenance operations.

3.2. *Description of the constructive process*

The chosen procedure to construct the viaduct shall conjugate minimal river affection (always reversible) and erection means suitable to the bridge magnitude.

The foundation of the piers P-5, P-6 and P-7 are located on the river (Fig. 18), and they have being built with a huge double enclosing sheet pilling circular wall (the exterior one has a diameter of 68 m and the interior one of 48 m) to allow the drain construction of the piles and the pile cap in P-5 and P-6, or the shallow foundation of the pier P-7.

So as to access to the foundation of the 3 piers located on the river, a provisional steel access bridge has been built, supported in temporary driven piles placed each 6 m. The construction of this provisional access (Fig. 18) was carried out respecting the natural course of the river avoiding any possible contamination or affection to the protected local fauna. That simplifies the works with road access from both sides of the river, avoiding the need of boat special resources.

Figure 18. Aerial view of the construction of the foundations of the three central piers

Once the foundations have been completed the piers are erected by means of a climbing formwork. When the shaft of the piers P5 to P8 is finished, the zero steel segment (with “W” shape) was assembled in horizontal position at the bottom of the piers, and once both side truss segments are finished, they were lifted and fixed in their position on site over the piers. Each of these huge segments weights around 375 t each, and their dimensions are 35 m length per 17.5 m depth (Fig. 19).

Figure 19. View of the erection of the zero “W” steel segment over one of the central piers

Completed the concrete part of the head of the piers, the steel truss of the central spans with variable depth will be simultaneously erected by a successive cantilever method, from the pier section to the closing segment at midspan. This method ensures independent work at the bridge from the marshes, river and surrounding vegetation.

The welding of the different elements that constitute one segment: nodes, chords, diagonals and transverse bracings, are done at one of the three steel workshops located near the edges of the river. Once the segment is completely finished, it is transported to the pier base in modules that measure 15 m long by means of a special platform with multiple axes accessing.

Once the segment 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. This procedure already begun (see Figs. 15 and 20) will be finished before summer 2014.

The constant depth spans of both sides are built by different procedures due to the different inferior crossing conditions. The side near the abutment A1 has several local road crossing and a local railway crossing, so the constructive process is by launching in three different phases (Fig. 20). As there is not enough free space behind the bridge to prepare a launching yard as it would be conventional, due to a tunnel very near to the end of the abutment, the launching yard has been established between abutment A1 and pier 2 (50+80 m), over temporary props.

Each 120 m span are assembled on site over the launching yard by welding the segments over temporary supports, and once finished it was launched.

Figure 20. View of the current state of the works near abutment A1 (December. 2013)

The second launch operation moved two complete spans of 120+120 m length, and finally the lateral side spans 1 and 2 (50+80 m) are welded on site over the temporary props by erecting each segment with the use of cranes (Fig. 20).

The four lateral approaching spans near abutment A1 will presumably finish before end of winter 2013.

The approaching spans on the side of the abutment A2, do not have the same restrictions as the ones on the side of the abutment A1. As there are no inferior interferences, a complete span lifting procedure has been designed.

The complete span are welded on site by assembling each segment propped on the ground (Fig. 21), and finally each span will be vertically liftedand welded to the previous one giving continuity to them.

Figure 21. Approaching spans near abutment 2 (120+120+80 m) ready for vertical lifting.

Once the assembly of the steelwork has been completed, the lower precast concrete plates will be placed and the subsequent lower slab concrete casting done. The upper concrete slab is poured over precast concrete plates bridging the space between steel upper chords of both trusses. The lateral cantilever part of the slab is cast using a movable formwork.

3.3. *Manufacture of the deck´s metallic structure*

The singularity and complexity of the bridge´s metallic structure, as well as the huge amount of structural steel, nearly 20,000 tons, made it necessary to divide the manufacture in 4 groups of workshops, 3 of which located in the north of Spain and 1 in Portugal.

In order to handle, manufacture and ship the steel elements to the worksite, the deck’s trusses are broken down into the following simple elements (Fig. 22): upper nodes, upper chords, lower nodes, lower chords, horizontal struts and cross bracings.

Once each module’s individual elements are ready, and prior to being shipped to the worksite, they are welded in larger subsets, depending on the case: node+chord or node+chord+node. The subsets and the rest of simple elements (diagonals, horizontal struts and bracings) are shipped and, later on, assembled in the on-site shops, thus making up the modules of the truss.

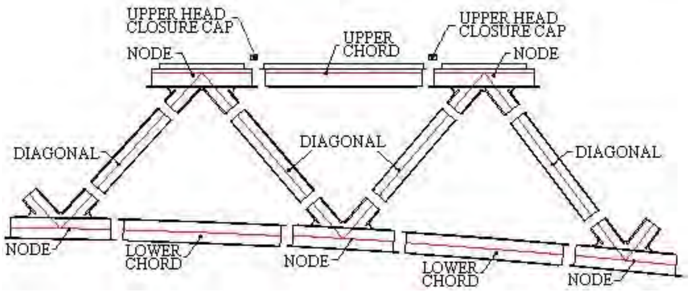

Figure 22. Break-down into simple elements for shop manufacture

The assembly of individual pieces and pre-welded subsets required setting up remarkable permanent facilities on both banks of the river. It was also necessary to arrange large storage and assembly yards both below the viaduct’s vertical projection between pier P-9 and abutment E-2 and in a nearby expanse with access to the river through a wharf.

The outstanding size of the steel modules, 8.75 m high in the constant depth segments and up to 17.5 m high in the haunch spans, made it forceful to set up assembly workshops as large as actual steelwork plants, with over 20 m clearance, nothing to do with temporary facilities.

The complexity of the metallic structure, has required an important amount of beforehand work in studies and development of a series of very repetitive details in order to do the assembly drawings.This way, the assembly drawings solve every encounter, welding, transition and specific detail, avoiding future executing problems.

The 131 drawings of the project, that define the metallic structure in detail, are developed in more than 5000 workshop drawings and nearly 18000 broken down drawings, defining with absolute accuracy each one of the plates, encounters and weldings of the bridge. This important engineering effort is essential to ensuring the correct design of all details, which have to accomplish very strict requirements related to fatigue resistance due to its being a composite bridge for high speed trains.

4 REFERENCES

Millanes F. et al. 2007. “Arroyo lasPiedras Viaduct: The first Composite Steel-Concrete HighSpeed Railway Bridge in Spain”. *Structural Engineering International*, Vol. 17, Nr. 4 (Eds.: IABSE).

Millanes F. et al. 2008. “Viaduct over the Ulla river in the HSRL Eje Atlántico in Spain. An outstanding structure in the field of Composite Steel-Concrete HSRL Bridges”. *Proceedings of Eurosteel 2008*. Graz, Austria.