VIADUCT ACROSS THE STREAM “LAS PIEDRAS”:
The first high speed railway line steel concrete composite bridge in Spain

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ABSTRACT

The paper describes an innovative solution on steel concrete composite bridges for high speed railway lines. The new design focuses on typical twin plate girder solutions to improve them with strict box girder capabilities. The double composite action frequently used for bending at hogging areas is generalized at the whole length of the bridge to achieve torsional stiffness requirements. The paper also describes additional aspects considered in the design. Specific details to improve patch loading resistance of webs in launching are described, and some considerations about seismic design of the bridge are also included.

Keywords: high speed railway bridge/double composite action/patch loading/isolating seismic device.

1. DESCRIPTION OF THE BRIDGE

The bridge over Arroyo Las Piedras is the first high speed railway line steel concrete composite bridge in Spain. It is located in the High Speed Line between Córdoba and Málaga. The structural form is a continuous beam with spans of 50.4 + 17 x 63.5 + 44 + 35 meters. The main spans are included among the longest in the world with this structural form for high speed railway bridges. The front view of the bridge is shown in figure 1.

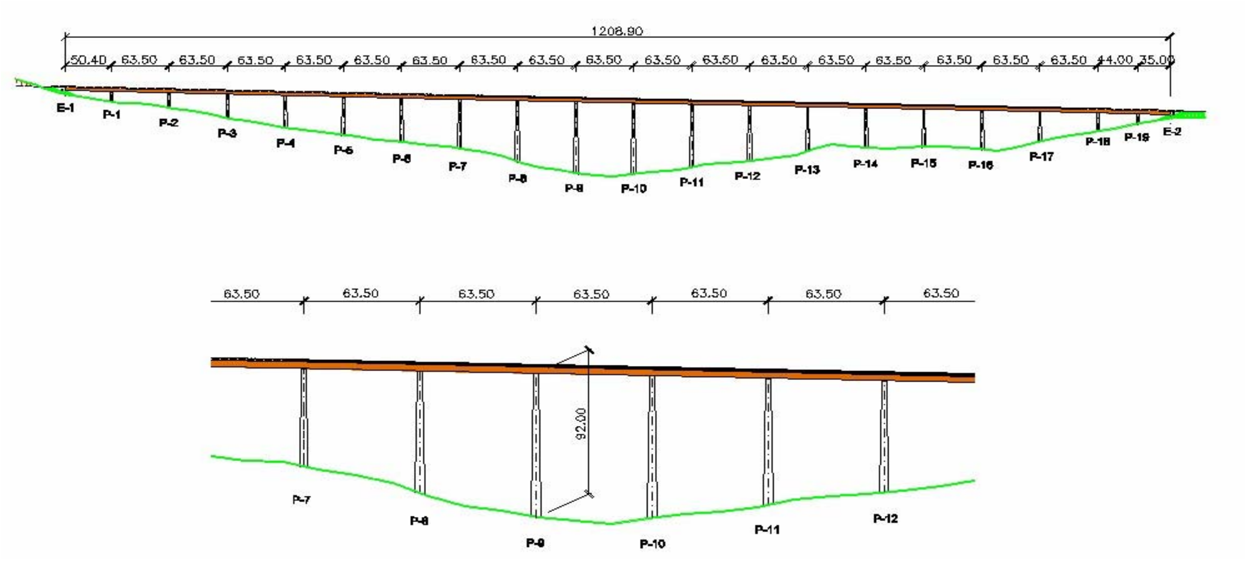

Fig. 1: Front view of the bridge

The height of the piers is remarkable, several of them higher than 90 meters. The bridge is presently under construction by incremental launching.

The cross section of the deck is made up of two 3,85 m deep twin-plate-girders plus a top slab whose thickness varies from 0,41 m at the longitudinal axis of the bridge to 0.22 m at the end of the cantilevers at both sides. The result is a steel-concrete composite cross-section with constant total depth of 4,26 m. A detail of the cross section at sagging areas of the deck is shown in figure 2a. It is someway similar to typical French twin girder decks frequently used for high speed railway bridges. However, important differences are apparent:

- Plate web diaphragms of the same depth than primary girders are replaced with vertical steel truss bracing with rolled profiles. They are easy to on site assembly, and significant reduction on steel weight and welds are obtained. They are laid all along the bridge every 8 meters approximately. Fillet welds were chosen for joints instead of high resistant bolts, to simplify control processes required. Fatigue response was carefully studied, with satisfactory results.
- The bottom steel truss bracing is replaced with a bottom precast slab made of 2 meters long and 14 cm. deep precast slabs. These are not lengthwise connected among themselves, and only one meter wide at both sides is needed to transfer torsion flows between the main girders and the slabs. Torsional stiffness required is guaranteed, even higher than that from the typical bottom lattice, and no tension stresses from bending moments are imposed because of the transverse joints between slabs. As a result, no relevant cracking occurs. Cast in place lateral strips are also easy to extend to the whole width between girders at hogging cross sections to obtain double composite action, as is described later.
- External triangle shaped cells are substituted for longitudinal stiffeners. They improve compressive web and flanges response against instability in assembly and serviceable states. In addition, the bottom external cell and lateral strip cast in place described before constitute an excellent system to improve patch loading resistance of the webs during launching. The stiffening of the webs is reduced to transverse stiffeners located every 4 meters along the deck.
- Transverse profiles are laid below the upper slab every 2 meters and connected with studs once the slab concrete is hardened. In this way, a steel concrete composite frame acts as a rail support and a relevant reduction in total slab thickness is obtained.

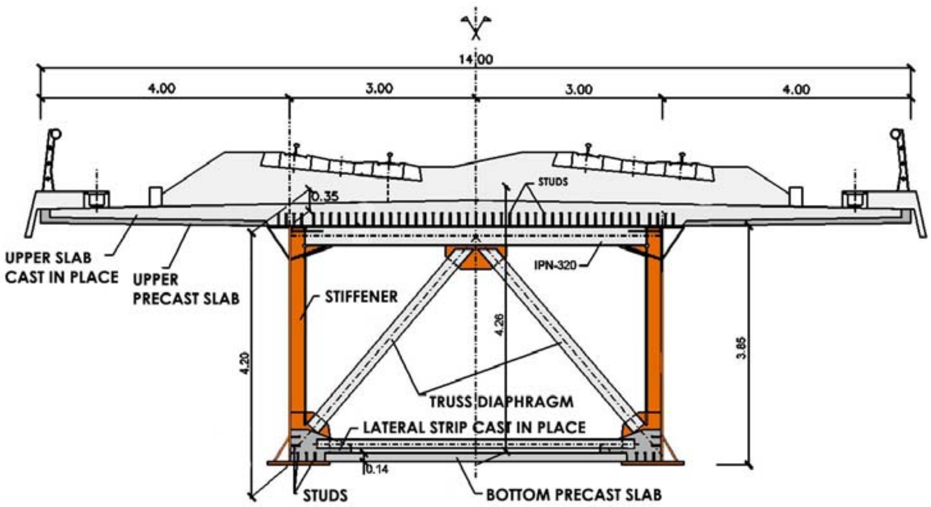

Fig. 2a: Mid-span bending cross section

A hogging cross section at negative bending moment area is shown in figure 2b. It is similar than mid span cross section, but double composite action is obtained by a bottom slab cast in place over precast previous slabs. The bottom slab extends 13.90 meters at both sides from the axis of piers in spans 63.5 meters long, some less in shorter lateral spans. The thickness varies from 25 cm at the end of the slab up to a maximum of 50 cm. located over piers. This slab is connected by studs and reinforcement steel to the main girders, and it allows a maximum thickness of bottom plates of 40 mm, much thinner than classical twin girder solution.

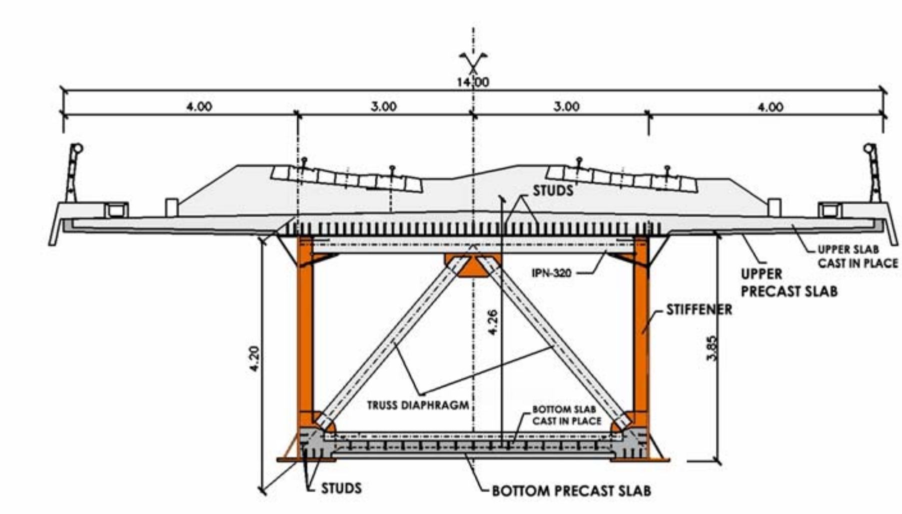

Fig. 2b: Hogging cross section

2. THE DOUBLE COMPOSITE ACTION GENERALIZED FOR TORSION.

The typical twin plate girder solutions for road bridges must improve their torsional stiffness to achieve high speed railway requirements. Transverse diaphragms and bottom lattice between girders are frequently used to limit the bending and torsion dynamic accelerations under the high-speed wagons. However, an alternative solution has been developed for the bridge over Arroyo Las Piedras. Double composite action has been extended to the whole length of the bridge to close torsion flows. A strict box cross section is obtained at mid span areas with bottom discontinuous precast slabs connecting the girders. When eccentric loads are applied to the bridge, torsional flows are introduced by the transverse truss diaphragms. In plane shear and bending response is developed by the bottom slabs acting as Vierendel system, and torsional flow is then closed someway similar than typical hollow closed sections. The thickness of the slabs is 14 cm and depth is 2 meters. They extend between bottom flanges of the main girders, but they are only connected among themselves at both sides by two lateral strips one meter wide cast in place. These strips are enough for torsion flow transmission between the girders and the slabs, which results in shear force in the plane of the slabs. As antissymetric conditions of the load require no bending in the longitudinal axis of the bridge, variable in plane bending of the slab is necessary, maximum in the connection to the girders and zero in the longitudinal axis.

As a result, compression area in one slab due to in plane bending is balanced by tension area in the next one. In plane shear force in the slab equilibrates with forces in studs introducing the torsional flow. All these forces have constant values between successive diaphragms because of the torsional flow is introduced by transverse truss diaphragms. However, it is clear that centroids of forces are not all coincident at the same point, so horizontal bending in the strip is needed for the equilibrium. The figure 3 shows the structural response of the bottom discontinuous slabs between two successive diaphragms submitted to torsion flow.

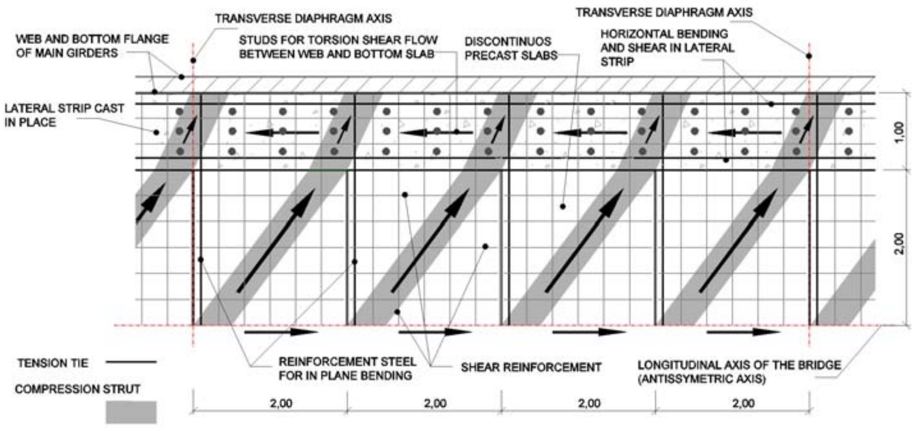

Fig 3: Torsional response of bottom discontinuous precast slabs and lateral strips cast in place

The incidence of the bottom discontinuous precast slabs in the torsional stiffness of the strict box girder was studied by means of a finite element analysis. Finally, a reduction at 66% from the stiffness with continuous bottom slab was established. In addition, the results were actually conditioned by possible cracking in the bottom slabs. The design of the bridge was made under the conservative hypothesis of total cracking obtained by a tie-and-strut analysis. In this case the torsional stiffness obtained is approximately 18 % of the uncracked one, which corresponds to the 12 % of the equivalent uncracked one with continuous bottom slab. Static and dynamic analysis with these values guaranteed the adequate behaviour of the bridge. Even in this extreme state of cracking the equivalent thickness obtained for the bottom slab was about 1.4 mm, similar than that obtained with the classical horizontal steel lattices, which have an equivalent thickness around 0,8/2,0 mm.

The bottom slabs have also a highly controlled bending cracking at the sagging area. The absence of transverse connection between them out of the lateral strips greatly prevents from the development of a tension collaborating effective width in the closure concrete and allows for low levels of cracking under longitudinal bending. Tensile stresses at the bottom flange are lower than 1300 kp/cm2 under dead loads and 3000 kp/cm2 under live loads. Connection between steel and lateral strips and longitudinal reinforcement placed on site in the concrete lateral strips are enough to control crack width in this area.

As it was mentioned before, the bottom lateral strips cast in place at midspan areas are extended to the whole width between girders at the hogging area to obtain a classical double composite action. Compression stresses from bending maintain uncracked the bottom slab, so bending and torsional stiffness in this area are clearly higher than that with just steel bottom solutions. Double composite action drastically improves the inertia and dynamic stiffness conditions both for bending and torsion.

But undoubtedly the main advantage from double composite action affects the ultimate load behaviour of the bridge. Class 1 or 2 cross sections are obtained all along the bridge both for positive bending at midspan and negative bending over piers. The figure 4 shows the position of the plastic neutral axis along a span 63.5 meters long, in comparison with limit border values for class 2. It is shown that cross sections of the deck are class 2 all along the bridge. In this way, instability problems in the ultimate limit state are avoided, not only in bottom flanges because of connection to concrete, but webs are compact as well, due to the low position of neutral axis at ultimate limit state.

The figure 5 shows the bending moment–curvature relationship at hogging cross section located over piers. At ultimate bending resistance a longitudinal strain of about 0,28% is reached at the most compressive fibre and tensile maximum strains of about 0,7 % at the top flange. The ultimate curvature is then 3.40 times the elastic curvature, so an important reliable ductility for positive and negative bending is obtained in the ultimate limit state. As a result, a safety and economical design at ultimate limit state with global elastic internal loads and elastoplastic resistance of sections for positive and negative bending can be applied. There is even enough capacity for reaching situations of the structure close to the global plastic ULS by means of an adequate control of the elastoplastic rotations and without any risk of brittle plates instability phenomena. This turns to be an irrefutable structural advantage of the strict box solution with double composite action regarding the classical twin-plate-girder alternatives.

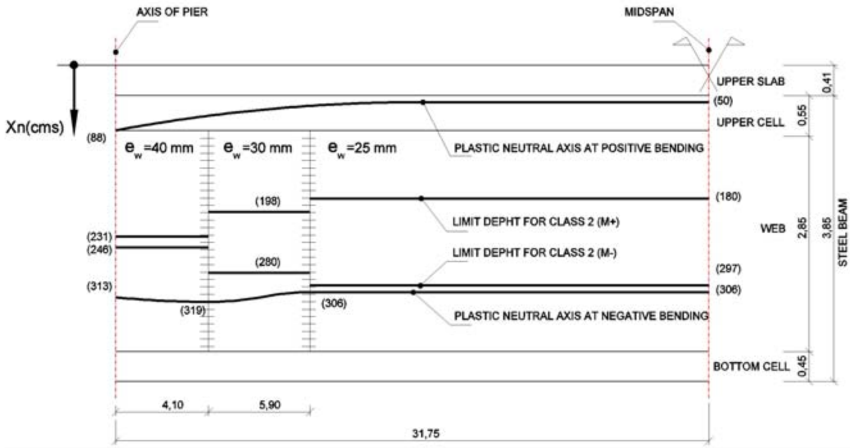

Fig 4: Position of plastic neutral axis along the span

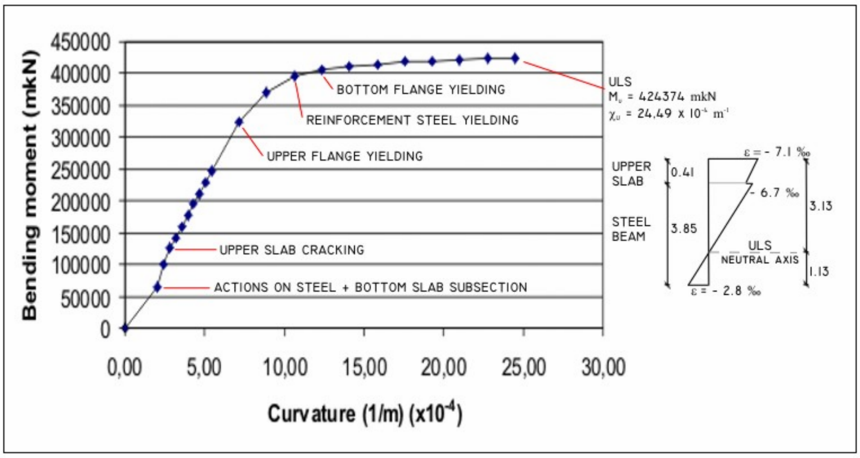

Fig 5: Bending moment-curvature relationship at hogging cross section located over piers

3. PATCH LOADING RESPONSE OF WEBS DURING LAUNCHING

As it was mentioned before, the bridge is presently under construction by incremental launching. The cross section in launching (figure 6) includes the whole steel and bottom concrete, and upper precast slabs without connection to the main girders during launching. Cross section remains accessible during the whole launching process. Once the deck reaches the final position the upper slab will be completed with cast in place concrete, and connection will be developed. As exception, the first span only includes steel section (figure 7) without bottom or upper concrete in launching, to avoid excessive forces acting on the cantilever system. Launching process is made simultaneously from both abutments.

Fig 6: Cross section in launching

Fig 7: Steel twin girder system

Concentrated forces on webs during launching reach 4250 kN. This value clearly exceeds those frequently obtained in launching of conventional steel bridges. The thickness of the web plate is 25 mm along an extensive length of the deck, and stiffeners are located every 4 meters. An exhaustive study has been carried out about patch loading response of webs during launching in order to guarantee safety conditions without increasing unnecessarily the total steel weight.

Specific regulations for the patch loading behaviour are given in several codes such as SIA, BSI or Eurocode 1993:1-5. However, in our opinion neither of them provides useful design criteria for the project when the cross section differs significantly from the classical double T girder. The external triangle shaped bottom cells and the longitudinal lateral strips cast in place near the bottom of the webs (figure 8) clearly improve patch loading response more than can be obtained with those codes. Both systems together provide great additional bending stiffness to the bottom flange consequently increasing in plane membrane stiffness of the web. In addition, bottom cell and lateral strip constitute a stiff longitudinal beam-like member, greatly able for distributing concentrated loads over a longer length of resisting web. The bottom flange with the bottom cell and lateral concrete strip system constitute also a truly beam capable of spanning the distance between transverse stiffeners. These improvements in the design for patch loading response of the webs are not at all considered in current codes regulations, so a specific non linear analysis using finite elements has been carried out. This analysis has shown significant increases in patch loading resistance of webs in comparison with that obtained with the proposal of codes adapted to the specific dispositions of the bridge.

Some important results obtained from the non linear finite element analysis are summaryzed in figure 8. With initial length of the load of 1.50 meters provided by the launching bearings, the final distribution of vertical stresses on the bottom side of the web just over the bottom cell reaches more than 3.50 meters, clearly higher than that obtained with the web alone without bottom cells. In addition, a significant part about 35 % of the total load is directly transferred by shear in the bottom cell to the adjacent vertical stiffeners as beam in longitudinal bending. The global improvement in patch loading ultimate resistance is about 70 % regarding to standard calculations of the conventional double T cross sections.

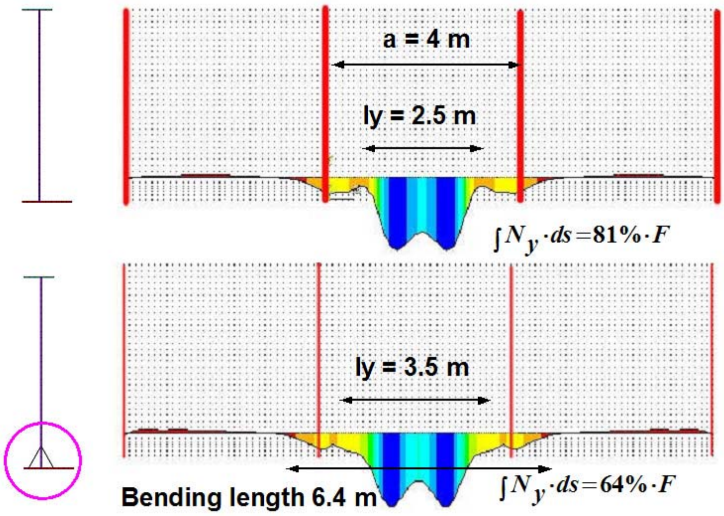

Fig 8: Patch loading improvements with bottom system.

4. SEISMIC DESIGN

The adoption of the longitudinal restraints of the static and dynamic model was the main problem in the design. Conditions were really severe in our case:

- Design seismic acceleration was high for Spanish usual levels (0.10 g).
- The total length of the bridge was too high for just one fixed point at one abutment, because of the total movement accumulated at the opposite one.
- Height of piers and foundation conditions were inappropriate for intermediate fixed points.

For the first time in high speed railway lines in Spain longitudinal viscous damper-shock transmission devices were designed at both abutments, to solve simultaneously the fixed points/expansion joints structural response. Low speed movements are then allowed without resistance, but horizontal serviceable forces are achieved without relevant displacements. In addition, transferred force to substructure is limited when an earthquake takes place. The seismic isolation system also includes longitudinal guided PTFE sliding bearings on piers P1-P7, P12-P19 and abutments, seismically acting as isolators in the longitudinal direction only, and bidirectional fixed bearings on piers P8-P11 to introduce restoring forces through rebound elastic force. No specific isolation is disposed in transverse direction. Horizontal flexibility is obtained by piers' elastic deformation.

Under non seismic conditions the viscous dampers act as shock transmission units for longitudinal forces. They constitute the main longitudinal restraint of the bridge, so design forces of the devices are enough to withdraw the longitudinal ULS of the bridge. Four devices with 2200 kN each one were designed at each abutment. Constitutive law of the device for high velocity movements is in the form $F = C \times V^{\alpha}$. So as to maintain maximum force developed fairly constant for a high range of velocity movements during seismic attack α must be less than 0.015. In this way it can be simply assumed $F = C = 2200$ kN, and force is almost velocity independent during a seismic attack.

Displacement capability of the devices has to be carefully taken into account in the design. Rheological, thermical and seismic displacements must be considered. In our case, a total maximum displacement of ± 350mm .was required in the device. Once the total movement is known it is actually important to obtain the maximum displacements of the deck under service braking forces. They may seriously affect the strength and stability of the track. In our case, a maximum displacement less than 8 mm was obtained, clearly admissible when expansion joints in the track are disposed at both abutments. It is interesting to point out that this displacement is drastically affected by the total movement capacity of the devices. Increasing this value we obviously improve the capability under seismic attacks even higher than considered in the design, but service response under longitudinal traffic forces turns worse. Therefore, excessive magnification of the seismic displacements in the design in order to an intended oversafety in the system is not appropriate.

In transverse direction the maximum seismic force at the top of the piers reached 4500 kN, clearly excessive for typical bold connections of bearings to piers. Specific devices shown in figure 9 were designed at the top of the piers and below the deck to transmit this transverse force while allowing longitudinal movements. In addition, typical bolds for bearings were designed for a seismic attack about the third from the design value, considered this as a service seismic attack.

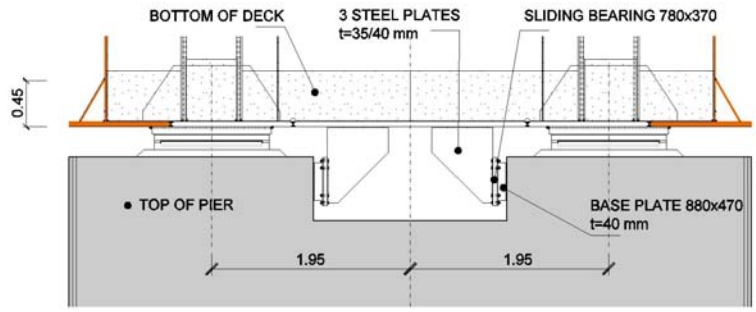

Fig 9: Transverse device for seismic attack at top of the piers

5. CONCLUDING REMARKS

An innovative design for high speed railway bridges has been developed focusing on the classical twin girder solutions with bottom lattice and transverse diaphragms. The new design generalizes the double composite action for bending frequently used in Spain at hogging areas to easily include torsional requirements at the whole length of the deck for high speed railway bridges.This design has been applied to the first high speed railway steel concrete composite bridge in Spain, presently under construction.

Some additional modifications from the classical twin girder solutions have also been included to simplify both execution and control processes. These modifications affect transverse diaphragms, top slab configuration and stiffening systems as described in the paper. The final design is in our opinion simpler than typical solutions, but maintains excellent structural response and great performance qualities.

Specific dispositions in the design have been developed to improve patch loading resistance of webs during launching. These details are not easily considered by current regulations and non linear finite element analysis is necessary. Steel bottom cells and lateral strips contribute to drastically increase ultimate load of webs submitted to concentrated loads.

Seismic design has been carefully taken into account. Longitudinal viscous damper-shock transmission devices constituted and excellent tool to solve the duality longitudinal movements/fixed points required by the specific conditions of the bridge. The length of the bridge, the height of the piers, foundation conditions and seismic emplacement not only turned this solution the best, but we think that the only solution as well.