Stayed Viaduct in Bucaramanga, Colombia

Leonardo FERNÁNDEZ
Ph. M. Sc. Civil and Structural Engineer
Carlos Fernández Casado S.L.
Madrid, Spain
cfcsl@cfcsl.com

Lucía FERNÁNDEZ
M. Sc. Civil and Structural Engineer
Carlos Fernández Casado S.L.
Madrid, Spain
luciafm@cfcsl.com

Guillermo AYUSO
M. Sc. Civil and Structural Engineer
Carlos Fernández Casado S.L.
Madrid, Spain
gayuso@cfcsl.comr

Antonio CANO
M. Sc. Civil and Structural Engineer
Carlos Fernández Casado S.L.
Madrid, Spain
acano@cfcsl.com

Raúl GONZÁLEZ
M. SC. Civil and Structural Engineer
Carlos Fernández Casado S.L.
Madrid, Spain
rgonzalez@cfcsl.com

Celso IGLESIAS
Ph. M. Sc. Civil and Structural Engineer
Carlos Fernández Casado S.L.
Madrid, Spain
ciglesias@cfcsl.com

Summary

The Carrera Novena viaduct in Bucaramanga is situated over a wooded area of great environmental importance which must be respected. The proposed viaduct with a principal span of 292.4m and two compensation spans 129.2m long. The maximum height over the ravine is 75m and the piers of the viaduct have heights of 52m and 72m; for this reason singles piers have been used, over which the towers have been built which stay the deck in its axis. It is 30m wide because 6 lanes of traffic and 2m wide pedestrian walkways were planned. The structure of the deck is formed of a single-cell box girder with a trapezoidal section and a depth of 2.8m. This girder is extended laterally by slabs. Its width creates significant bending moments which are resisted by interior transverse triangulations and exterior struts every 3.4m. It is situated in a zone of high seismic activity and for that reason a structure has been adopted which is encastred in the piers and free in the abutments. It was constructed using successive compensated cantilevers, and concreted in situ.

Keywords: cable stayed bridge, concrete, high seismic activity, central staying, successive cantilevers.

1. General outline of the viaduct

The Carrera Novena viaduct in the city of Bucaramanga passes over the Rosita and Loro ravine to counteract the disruption these ravines cause to traffic flow in the city.

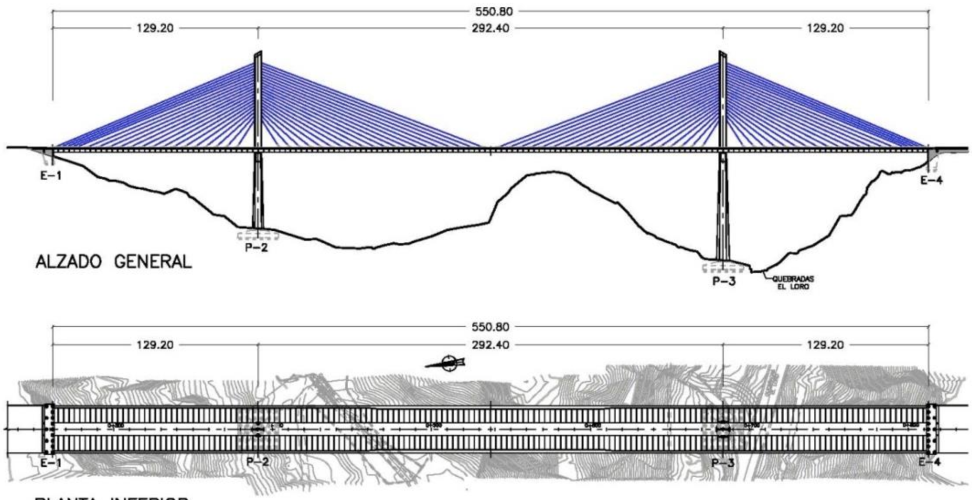

Fig. 1. Lateral and plan view of the bridge

The Carrera Novena passes close to their confluence and for that reason the crossing has been achieved with a single viaduct of 550m in length, the distance between the outside edges of the ravines. Between them is a wooded area with large trees, and great environmental importance which must respected to the highest degree. For this reason the design was for a viaduct of large spans to reduce as far as possible the number of piers in the ravines. Given its length this has mean three spans: a central span of 292.4m, and laterals of 192.2m. These three spans give rise to two piers situated close to the bases of the banks which are the flattest areas and thus those which require the least earth moving to secure the foundations. This has lead to piers of 52m and 72m in height, from the foundations to the surface of the deck.

The Carrera Novena is going to be a great avenue with six traffic lanes and with two metre wide pavements on its edges. This, added to the space necessary for the barriers and for the stay anchors has led to a total deck width of 30m which is exceptional in a viaduct of this type.

2. Structure of the viaduct

Fig. 2. Perspective of the bridge

Given the general dimensions of the viaduct it is necessary to explain the type of structure used. The central span of almost 300m means a long span bridge which means that the options available are those which require a structure above the deck. It is true that a 300m beam bridge has been constructed using successive cantilevers but it required a depth over the supports of 15m which would be excessive over the wooded area, even though the viaduct passes at a significant height.

The upper arch solution is not applicable in this case because it would mean making three unequal spans and construction would be complicated. The span of 300m is short for suspension bridge and for that reason it is also not the correct solution for this bridge. This span is clearly within the range of cable stayed bridges, which are cheaper than suspension bridges, and hence this is clearly the best solution in this case.

The type of structure having been chosen, which cable stayed bridges there are two possibilities. Either stayed at the edges with two parallel sets of stays or stayed along the axis of the bridge is one single set of stays. In this case we adopted the solution of one set in the axis because it allowed us to continue the lower piers into the upper towers and make the two one structure from the foundations of the piers to the tops of the towers. The total heights of the piers and towers together are 113m and 133m.

Fig. 3.Panorama of the bridge under construction

The fundamental problem which faced the structure of the viaduct is the seismic effects, given that in this region they can reach great magnitudes which influenced the type of structure which was chosen. In this bridge, once the towers and the deck were specified, a number of studies were made of different types of structures, varying the ways of connecting its different elements and considering whether or not to use dampers. The solution which appeared best, and the most economic, is to make rigid joints in the connections between the pier, deck and tower which lead to a frame extended by the compensating spans which are supported on the abutments. Once this structure was chosen, the possibility existed to use longitudinal dampeners to reduce the seismic effects, but the difference between using dampers or sliding bearings on the abutments was small. The only advantage of the dampers is that with them the longitudinal movements would be slightly reduced; but the slight difference in movement added the saving of the dampers led us to a solution of permitting free longitudinal movement in the abutments and coercing the transverse movement in them and the piers. This configuration leads to a pair of forces on the abutments which produced a pull on one and increases the force of support in the other, a situation which may be symmetrical like all seismic effects.

3. Description of the viaduct

3.1 Foundations

This viaduct has its foundations in flood deposits made up of silty sands, some of great size, and of varying depths, which meant that the terrain necessitated a deep foundation, with piles of 1.50 m in diameter and 18m long. The strong bending moments in the piles due to seismic horizontal forces of led to piles caps of 25.5m x 25.5m with 36 piles in each. The difference in height between the two piers is compensated by their varied flexibility, and for this the foundations have been the same. The presence of boulders of great size complicated the piles constructions.

The abutments were also cemented with piles 1.2m in diameter and 20m long.

3.2 Piers

Fig. 4. Pier 3

The piers of 52m and 72m in height have a hexagonal section, twin-celled in the lower part with a central wall and single celled in the upper part. The transverse section varies its longitudinal and transverse dimensions with its height. Pier 3 of 72m in height varies from a section of 10x8m at its base to 4x4 in the upper part. In the last 6m it broadens again to the 8m of the base of the deck beam. Pier 2 of 52m in height has the same variation. Its section at its base is 7.4m x 7m.

Both piers are reinforced with large quantities of passive reinforcement due to the level of seismic activity they have to withstand. The connection of the principal reinforcements, with diameters of 25mm and 32mm has been achieved using reinforcement bar couplers.

3.3 Towers

The towers are prolongations of the piers through the rigid joints formed by the deck, the piers and the towers. The pseudo-hexagonal form of the piers is maintained but with a constant exterior section and with a central gallery where the anchors of the stays are placed. Their exterior dimensions are 4.4m x 3.2m and the interior gallery is of

2.44m x 1.7m where the anchors are situated and this is reduced to 1.2m x 1.4m in the interior of the tower where they are not present.

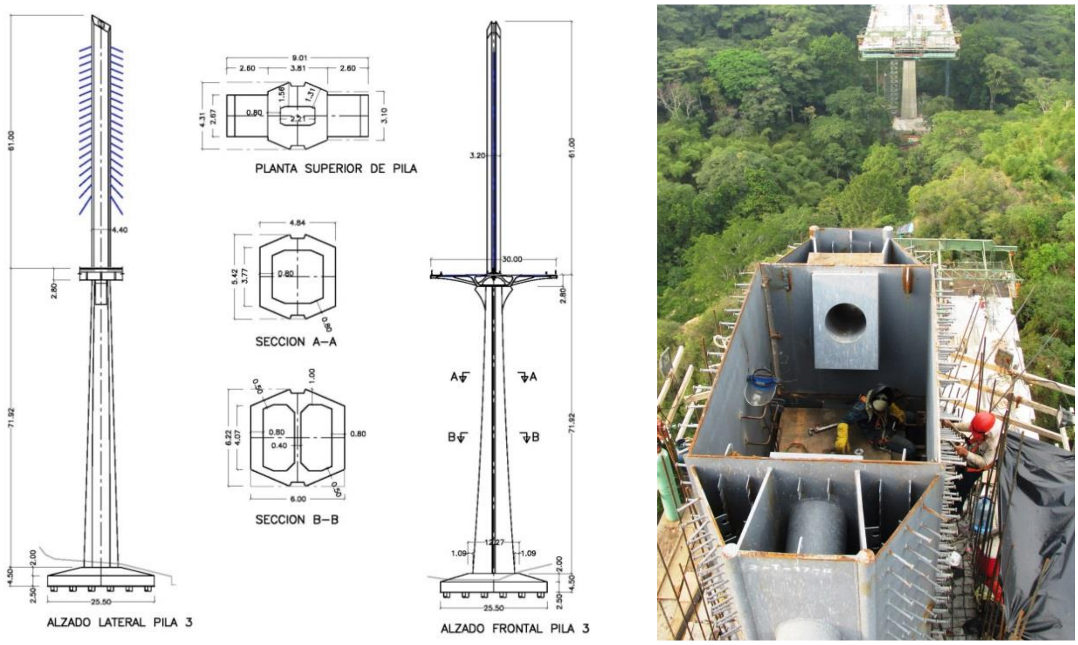

Fig. 5. Plan of pier and tower 3 and metal girder

The fundamental problem of these towers in common with other bridges of this type, is the effect of the anchorages of the backstays and the forestays, situated on the opposite faces of the interior gallery, because due to the fact that the stays must be in the same plane they cannot cross in the tower. This creates a tension on the side faces of the towers, and bending in the front faces, because the anchors are supported on their axis. These effects have necessitated the use of an interior metal girder on the edges of the gallery which resists the tension and the ones which the stay anchors produce on transmitting this force in the axis of the girder. These girders are connected to the concrete of the towers and incorporate the anchor plates which are stepped on the front faces. This girder is continuous throughout the length of the tower which means it also carries vertical forces.

3.4 Deck

Fig. 6. View from below of the deck

The most singular element of the bridge is the deck due to its 30m width and the fact that it is stayed along its axis. This has been achieved using a trapezoidal single cell box girder of 8m at its base edge and 11m on its upper and 2.8m in depth determined fundamentally by the transverse bending of the deck and the torsion due to asymmetrical loads because it is stayed along its axis. These dimensions result in the need for lateral cantilevers of 9.5m and a 9m span of the slab between the web of the boxes, which means that the section must be triangulated. These are fixed using lateral struts and interior diagonals set every 3.4m. The stay anchors are placed every 6.8m, that is to say every two triangulations. For this reason the behaviour of the internal diagonals are different in the sections which have stays and those which do not.

In the lines which have a stay the anchor is placed underneath the upper slab and has to resist the loads of the deck which are transmitted through their webs. For this reason the diagonals have to transmit this load on the base of the webs to the anchor through a tension in them, which is resisted by means of a pre-stressed cable. The intersection joint between the stays and the diagonals are completed by means of horizontal pre-stressing of the slab.

In those which do not have stays, the behaviour is the opposite: its function is to shift the loads on the upper slab to the webs, so there is a compression throughout them.

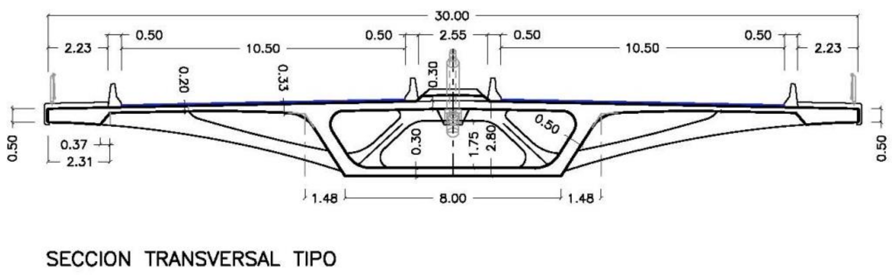

Fig. 7. Deck cross section

The deck is reinforced by longitudinal pre-stressing which is used in the construction process. As well as the prestressing in the upper and lower slabs of the girder, which is extended as the construction advances, there is additional pre-stressing in the lateral beams of the upper deck slab at its edges, which serves to resist bending in the horizontal plane due to transversal seismic forces.

Fig. 8. Bridge crane on a cantilever

3.5 Abutments

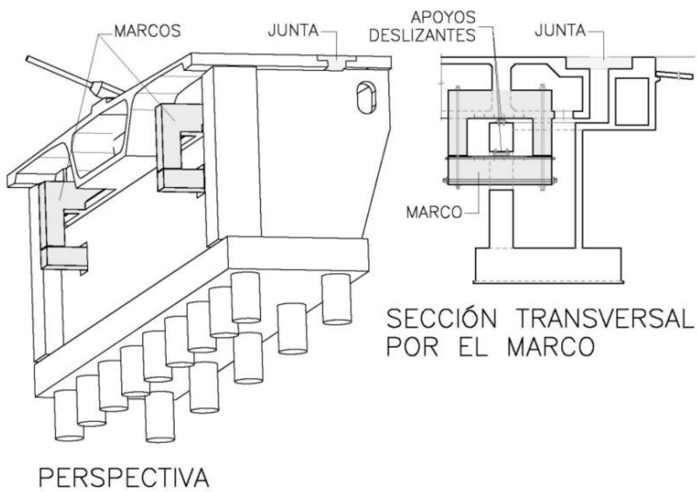

Fig. 9. Abutment frames

The abutments serve to support the deck and as we have seen must allow the free longitudinal movement of the deck and impede on the transverse movements, having to resist the transverse forces originated by seismic activity.

The fundamental problem of the abutments of this bridge is that as well as permitting the free movement of the deck, which results in significant movement, in the order of 1m and 1.5m total for seismic action, it must resist the vertical forces of push and pull compatible with this movement. This has been resolved by means of a frame joined to the deck, which penetrates the abutments wall and transmits the force to it by means of a support which can slide the upper piece if it is a push force and through a support of the lower piece which penetrates the stirrup wall if it is a pull. This frame is held together vertically by pre-stressing, which can be disarmed if there is need to replace supports.

4. Special studies

One fundamental problem in the calculation of the piers of a tall viaduct is the ductility coefficient to apply. These coefficients are laid down by construction codes and these set out that to apply them the ductility of the structure must be tested. In the study of this viaduct a calculation based on capacity was carried out to evaluate what this coefficient should be in function of the structure geometry, the sectional geometry and of the reinforcement of the cross sections.

The concept of a structure’s capacity allows to evaluate with surety the seismic forces when behaviour of the structures ceases to be linear. This way to design seismic-resistant structures also permits the designer to locate damage in particular chosen sections in such a way that the inspection and repair may be simpler. In those sections plastic hinges are produced which improve the ductility of the overall structure. The structure which have undergone great seismic activity respond to this in a non-linear way. This response has great influence on the effects of the forces generated by the seismic activity.

The procedure consists of obtaining the capacity diagram of the structure, taking into account cracking and the confinement of the sections in such a way that the intersection of this curve with the demand diagram will give us the demand point of the structure. And from such a value it is possible to obtain the ductility coefficient of the structure.

With this calculation method the aim is avoid the production of brittle breaks in the structure maintaining the elements of the structure at point below their bending moment resistance. This is achieved in the structure using fusibles which protect the rest of the elements, concentrating the damage in those chosen sections. These sections must undoubtedly be capable of maintaining their resistance throughout the seismic activity and for this reason it is necessary they behave in a ductile manner.

5. Construction Process

The construction process followed in this bridge is the classic process for viaducts of this type.

In the first place the piers were constructed to the level of the deck and on this was raised the metallic platform raised from the ground to construct the segment of pier of 17.9m in length on which were raised the bridge cranes to construct the deck in compensated cantilevers.

Fig. 10. Cantilever under construction

The deck and the towers were constructed simultaneously.

To construct the towers it was necessary to raise the sections of internal metal girders and these were joined to the previous section by welding and once the joints were finished the formworks were put in place to concrete each phase of the tower.

The deck was constructed using symmetrical cantilevers compensated using segments of 6.8m in length, which is the distance between the stay anchors in the deck. Once the segments had been constructed over the pier, the bridge cranes were raised to construct the joint symmetrically. Given their weight, of the order of 300t, it was necessary to concrete them simultaneously with a maximum difference of 50t, that is to say 16% of the segment weight. When the lateral cantilevers reached the abutments they were fixed to these and the construction of the central cantilevers was continued until close in key using one of the two cranes.