DOUBLING THE WIDTH OF THE PLATFORM OF THE SAN PEDRO BRIDGE (SPAIN)

José A. Torroja, Dr. CE, Torroja Ingeniería–Universidad Politécnica de Madrid, SPAIN
José M. Simón-Talero, CE, Torroja Ingeniería-Universidad Politécnica de Madrid, SPAIN
Alejandro Hernández, CE, Torroja Ingeniería, Madrid, SPAIN

ABSTRACT

The SAN PEDRO BRIDGE is a six span bridge, 750m long, 88m high, 12m wide and curved (R=700m) built in 1993 by the cantilever method. Its superstructure is a prestressed concrete box girder; main spans are 150m. In 2008-2009 the width of the platform was enlarged to 23m, using five movable scaffoldings while keeping the bridge open to traffic under construction. The original platform was widened 6m on each side connecting a new lightweight concrete cantilever to the original upper slab. These cantilevers were supported by inclined steel struts. The tie of the upper-slab was made of new transverse prestressing tendons.

The original superstructure was strengthened to resist new extra loads. An additional new central web and a composite concrete-steel section have been constructed connected to the concrete box and to the central web using vertical high-strength prestressing bars. Also, external prestressing tendons were implemented.

It was also necessary to strengthen the connection of the original concrete box section to the piers. Detailed calculations were made in order to evaluate the load distribution transmitted to the piers by the webs and by the original inclined concrete walls of the box girder. Finally, a detailed second-order-analysis of the complete structure was made to guarantee the resistance of the piers compared to actual loads.

Keywords
Widening, External prestressing, Cantilever bridge, Light concrete, Bridge rehabilitation.

1.- DESCRIPTION OF THE ORIGINAL BRIDGE

The bridge of San Pedro de la Ribera is located in the North of Spain. It was built from 1992 to 1994. The deck of the original bridge is 750 m long and 12 m wide, made of prestressed concrete. It is a continuous deck divided into 5 spans 75 + 4x150 + 75 m. (Fig. 1)

The deck is curved with a radius of 700 m. The elevation profile is a ramp of 3.712 %. The cross slope is constant equal to 3.5 % all over the bridge.

The cross section of the deck is a box section: its depth varies from 3.0 m in the middle of the main spans to 7.5 m in the connection with the piers; the width of the box is 6.5 m; two cantilevers of 2.75 m are located on both sides of the upper slab of the box to make the total width of the deck equal to 12.0 m. This is the platform of the original bridge. The webs are 0.48 m thick; the thickness of the upper slab varies from 0.18 m to 0.35 m. The lower slab is 0.25 m thickness in the middle of the main spans and it grows till 1.5 m near the piers.

The deck is a segmental bridge built using the cantilever method; the length of the voissoirs was 3.75 m for the heaviest voissoirs located near the piers, and 5.00 m for voissoirs of the centre of the spans. The deck is made of prestressed concrete (f_{ck} = 35 MPa). There are two different families of tendons on each span: the upper tendons are the typical prestressing tendons located on the upper slab used for the segmental construction; the tendons located in the lower slab are tensioned once both cantilevers of each span are connected using a central segment.

Fig. 1 General view of the original bridge

The piers are made of reinforced concrete (f_{ck} = 30 MPa). Each pier is made of two parallel walls separated 8.0 m. Each wall is a box section; the thickness is constant and equal to 0.35 m, but the depth and the width vary along the pier. The cross section on the top of the pier is

6.5 m x 1.75 m. The highest pier is 81 m tall. The connection of the deck to the piers is rigid, made by the vertical reinforcing of the piers that goes into the deck, and overlapping with the reinforcing of the deck.

The foundation of piers no. 3 and 4 is made by 16 concrete bored piles each, that reach a resistant stony layer. The piles are 2.0 m diameter and some 20 m long. The pile caps are 23 x 23 x 3 m. The piers no. 1, 2 and 5 are directly founded on a rock layer.

2.- DESCRIPTION OF THE SOLUTION FOR WIDENING THE PLATFORM FROM 12 M TO 23 M

2.1.- BASIS OF DESIGN FOR WIDENING THE DECK

As it has been said, the original platform of the deck of the Bridge of San Pedro was 12 m wide (Fig. 2). In 2005 the design for the new A-8 freeway was finished, and it was proposed to build a new bridge similar to the existing one. So, the freeway would have a bridge 12 m wide for each platform. Nevertheless, before the construction of the new bridge was started, some additional environmental considerations were raised, and the solution of making a new bridge was questioned. So, a new alternative was considered: to widen the existing bridge to a platform of 23 m (Fig. 3), instead of constructing a new bridge 12 m wide.

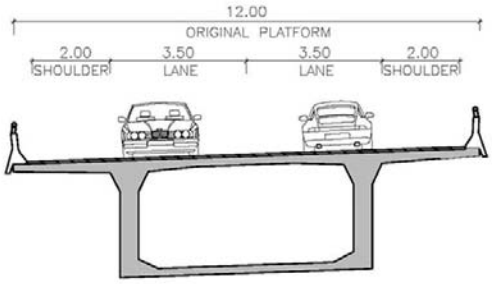

Fig. 2 Cross section of the original deck of 12 m width

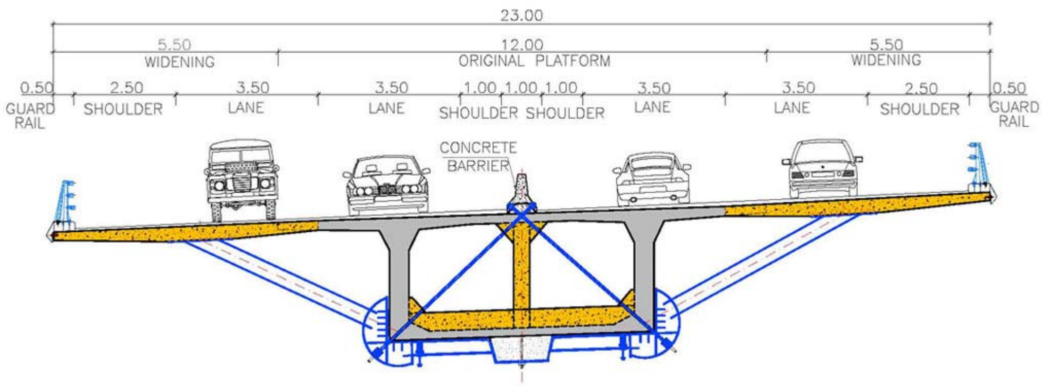

Fig. 3 Proposed cross section of the deck widened to get 23 m width

The basis of design for the widening of the deck were established as follows:

- Traffic should not be interrupted while the operations for widening the deck were in progress.
- The resistance of the widened deck should be enough for loads defined in the new Spanish code for actions on road bridges, approved after the existing bridge had been built.
- Widening the deck from 12 m to 23 m increases the dead load of the structure. The safety factor of the deck and the piers should be equal to the required values defined on the actual Spanish codes.

In addition to that, some other requirements were established for the design:

- The existing foundations of the piers should resist the new loads. It is not possible to make any strengthening operation of the piles, and only minor rehabilitation of the pile caps is possible.
- Obviously, the existing piers should also resist the new loads. Only some changes on the top of the piers are permitted in order to correctly connect the deck.
- The connection of the widened deck to the existing piers is difficult. So, it is desirable that no external additional elements are connected on both sides of the pier tops. It is preferable to connect the deck using the original triangular concrete cell that is the inside of the deck.
- If possible, the cross slope of the deck should be upgraded from the existing 3.5% to 4.5%.

These requirements were all met.

2.2.- POSSIBLE SOLUTIONS FOR WIDENING THE DECK

Two groups of possible solutions for widening the deck were studied, taking into account the basis of design presented above. In the beginning, it was investigated to add a new structure to the existing deck. This new structure should stand the dead load of the widened deck and some of the traffic load of the widened platform. The new structure could be a steel truss (Fig. 4), which is lighter than a steel beam structure or a concrete box section.

Fig. 4 Widening the deck using a steel truss partially connected to the existing deck

A detailed study of this solution was produced, and some remarkable difficulties were found:

- Important relative deflections were induced because of the different stiffnesses of both structures, the existing concrete box girder and the new steel truss.
- Because of the inclination of the steel truss, important horizontal and vertical deflection were produced on the steel truss when the dead load and the live load were considered.
- A preload by means of a “predeflection” of the new steel truss was proved to be necessary. Thus, the absolute and relative deflections were minimized and the extra load supported by the existing concrete deck was reduced.
- The connection of the steel truss to the piers was difficult to build because of the high loads to be transmitted to a existing element and the reduced space available for this connection.

2.3.- GENERAL CONCEPT OF THE PROPOSED SOLUTION

Because of the said difficulties, a new solution was studied. So, it was decided to strengthen the existing deck to support the widened platform and to resist the new extra load. On 2007, the final solution was approved (Fig. 5).

Fig. 5 The San Pedro Bridge. General view after widening the platform.

The proposed solution includes the construction of different elements, as follows (Fig. 6):

- Two cantilevers of 6 m span each one to support the widened platform. These cantilevers were made of lightweight prestressed concrete (γ = 19 kN/m^3).
- Some inclined steel columns to support the cantilevers and transfer the vertical loads to the existing concrete box girder.
- Two pairs of steel diagonal bars connected to the said inclined columns to convey the loads to a new central web.
- A new central web made of concrete and vertically prestressed using high strength steel bars. This central web is used to increase the resistance to shear efforts.
- A composite box section connected to the lower slab of the existing concrete box girder. This new structural element was used to increase the bending resistance of the existing deck both for positive and negative bending moments.
- Two groups of tendons placed in the inside of the box section of the deck. These prestressing tendons were designed to increase the bending resistance and the torsional capacity of the deck.
- The lower slab and the upper slab of the existing concrete box girder were strengthened using steel plates attached with epoxi.
- The connection of the deck to the piers was strengthened. Some new prestressing tendons were added to the existing reinforcing between the deck and the pier.
- The bearings located between the deck and the abutments were replaced and some new reinforcement was designed to increase the local bearing capacity of the deck.
- Two pairs of impact transmission devices were placed connecting the girder to each abutment.
- Finally, the front walls of the abutments were strengthened with high resistance steel bars.

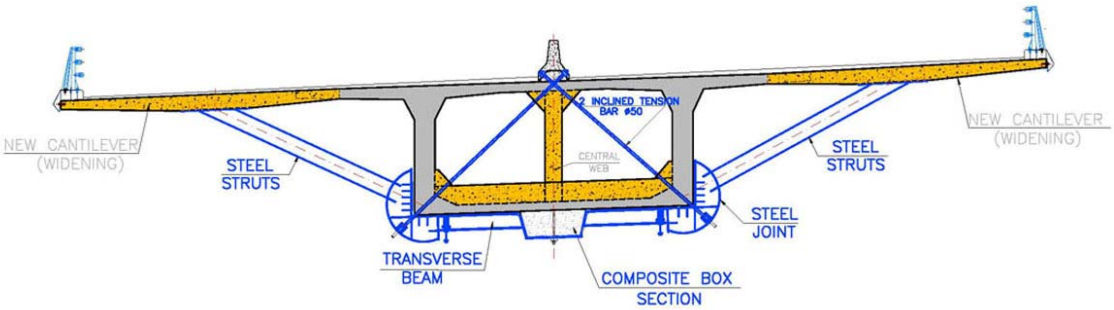

Fig. 6 Selected proposal for widening and strengthening the deck.

In order to increase the cross slope from 3.5 % to 4.5 %, the lateral extensions (in yellow in figure 6) were not exactly aligned with the existing deck, but placed with a different slope than the nominal slope (4.5 %). The final transverse line was achieved using lightweight mortar. A further refinement of the surface was achieved using a variable thickness of the base layer of the pavement.

3.- CALCULATIONS. MAIN RESULTS

3.1.- VALIDATION OF THE EXISTING PIERS

The highest pier of the bridge is nearly 81 m high, so wind forces are the most important action when validating the existing reinforcing of the columns of the piers. In order to get an accurate result of the calculations, wind tunnel tests were made (Fig. 7) both for the piers and for the widened deck. Pressure coefficients were obtained from the wind tunnel tests, showing a remarkable reduction from what is stated on the Spanish Code or on the Eurocode 1 – Part 1.4 (Table 1)

Fig. 7 Wind tunnel tests.

WIND PRESSURE COEFFICIENTES							
DECK				PIERS			
TRANSVERSE WIND		VERTICAL WIND		TRANSVERSE WIND		LONGITUDINAL WIND	
SPANISH CODE	WIND TEST	SPANISH CODE	WIND TEST	SPANISH CODE	WIND TEST	SPANISH CODE	WIND TEST
1.30-1.64	1.06	SIMILAR		1.13-1.19	0.36	2.09	1.93

Table 1. Comparison of results from the wind tunnel tests results and the Spanish Code.

For the validation of the piers a second order analysis was used. A 3D model was implemented including the piers and the deck. The results showed that the existing reinforcing of the columns of the piers was enough for resisting the new loads produced by the widened deck. Then, to validate the foundations of the piers a detailed FEM was used. In using this detailed model, non linear distribution of stresses were considered to determine the exact load distribution on each pile.

3.2.- CALCULATION OF THE EXISTING DECK.

It is obvious that the first calculations to be made were those to quantify the effects of the extra loads produced by the widened platform on the existing deck. So, a detailed 3D model of the whole structure was made. It had to consider the effects of creep and shrinkage produced during the construction and on the 13 years from the end of the construction to the beginning of the works for widening the bridge. The results of these calculations showed that:

- The existing internal prestressing was not enough to prevent cracking due to bending moments on the deck. This meant that the Serviceability limit state (SLS) was not achieved.
- The principal stresses on the existing webs were too high. That is, shear and torsional shear stresses were unacceptable. Again, SLS was not achieved.
- Concerning flexural Ultimate limit state (ULS), the resistance of the centre span section and of the pier section was not high enough.
- Also shear and torsional ULS were not achieved.

4.- STRUCTURAL ELEMENTS PROPOSED FOR STRENGTHENING THE DECK.

As it was expected, it was necessary to strengthen the deck. Therefore, different structural elements were proposed to increase the resistance of the existing deck. A brief description is presented below.

4.1.- LONGITUDINAL COMPOSITE BOX SECTION CONNECTED TO THE LOWER SLAB.

The results of the calculations proved that there was a lack of resistance both in the centre of the spans and on the sections close to the connection with the piers, concerning the flexural ULS. So, it was decided to add a new resistant element connected to the lower slab of the existing box girder (Fig. 8). This element should be a composite section because it was tensioned in the centre of the spans but it was compressed when the section was near to the piers.

The composite box section was 1200 x 800 mm. The grade of the steel was S355 and the concrete was a lightweight concrete, f_{ck} = 30 MPa. The connection to the existing deck was made using high strength bars 40 mm in diameter and 0.875 m long, approximately. Because the deck was curved (R=700 m), special care was taken when building the steel section and when making vertical holes in the existing deck, in order to correctly place the steel bars of the connection.

Fig. 8 Composite box section connected to the lower slab.

4.2.- EXTERNAL PRESTRESSING

In addition to the composite box section connected to the lower slab, it was decided to add to the existing deck an external prestressing located into the existing box girder (Fig. 9). Two types of tendons were designed on each span.

- Type no. 1 was made of 6 tendons 31Φ0.6". These tendons were tensioned near the lower slab of the deck some 23 m far from the section of the deck connected to each pier where they were located near the upper slab. These 6 tendons were designed to increase the flexural capacity of the section located over the piers. In addition, these tendons make a big contribution to the shear resistance of the deck, as they take the shear effort of the deck and transmit it directly to the piers because of their inclined geometry.
- Type no. 2 was made by another 6 tendons 28Φ0.6". These tendons were tensioned near the upper slab of the deck and close to the section connected to each pier. These 6 tendons were designed to increase the flexural capacity of positives bending moments of the centre of the spans.

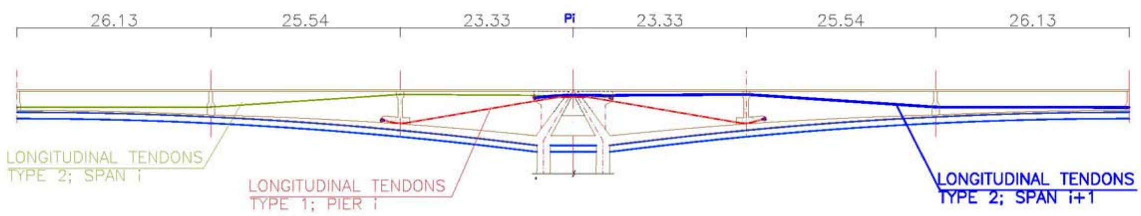

Fig. 9 Definition of the external prestressing

To transfer the forces due to the external prestressing to the existing deck it was necessary to build some deviation walls (Fig. 10). Some difficulties were found for introducing the prestressing forces of tendons no. 1 to the deck. Therefore, some vertical and horizontal high strength steel bars were used to properly connect the deviation wall to the webs and to the slabs of the existing deck. Also, at the bottom of the deck near the abutments it was necessary to use high strength steel bars to transfer the forces of the final anchorages of the tendons of type no. 2 to the webs of the deck.

Fig. 10 Deviation walls and anchorages of the prestressing.

It is remarkable that the geometry of the tendons is not parallel to the axis of the deck. The tendons placed near the upper slab make somehow a "zig-zag" opposite to those located near the lower slab. In doing so, an additional resistance to torsional effects is produced (Fig. 11).

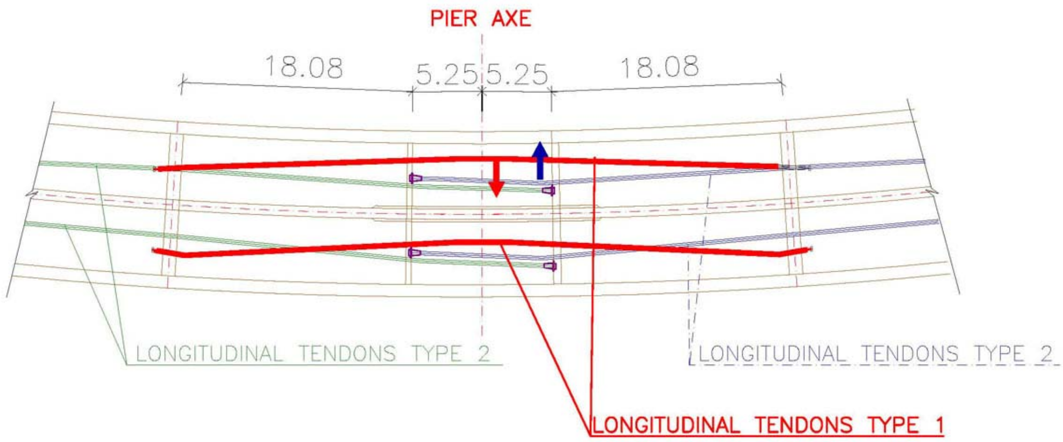

Fig. 11 Plan of the external prestressing

To consider the local effects of the tendons on the deviation walls a FEM was implemented. This model represented one span, 150 m long, and was also used for the detailed study of the distortion of the box section due to eccentric loads.

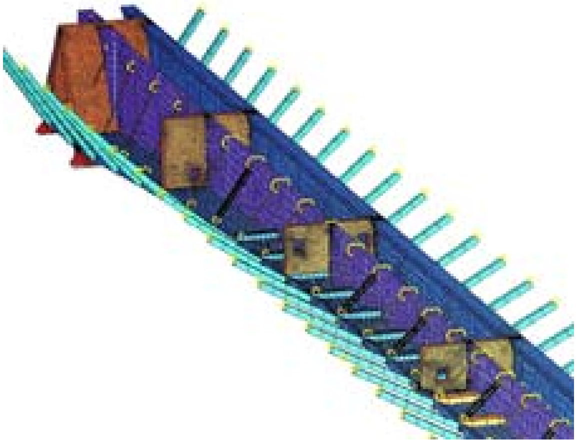

Fig. 12 3D model for the calculation of the local effects of the prestress

4.3.- NEW CENTRAL CONCRETE WEB

A new central web was constructed all along the deck (Fig. 13). This web is made of lightweight concrete, f_{ck} = 30 MPa and it is connected to the upper and lower slab with high strength steel bars 40 mm in diameter and 0.875 m long This new web is used to increase the shear capacity of the existing deck. As it will be presented next, a large proportion of the shear effort produced by the extra load due to the widening of the platform is transferred to this new web. So, the lateral existing webs are responsible for resisting the torsional effect produced by the new loads, but they were not subject to an extra shear effort.

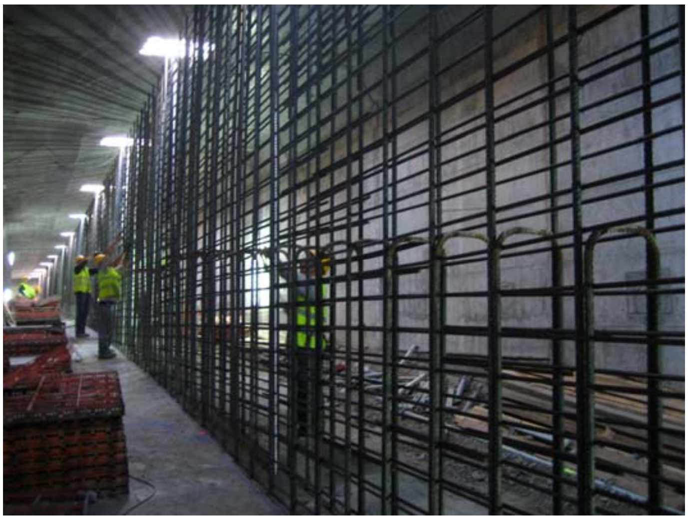

Fig. 13 New central web.

4.4.- TRANSVERSE BENDING MOMENT RESISTANCE ELEMENTS

The existing concrete box section was 6.5 m wide and the original platform was 12 m width. Widening the platform to 23 m meant that the span of the cantilevers of the upper slab should be increased from 2.75 m to 8.25 m. Obviously, the existing reinforcement of the cantilevers was not enough to resist the new loads. Thus, a new type of structural mechanism had to be implemented, and so some inclined steel columns were placed, spaced between 4 and 5 m, approximately, resting 4.65 m from the lateral web of the existing box section (Fig. 14)

Fig. 14 Inclined steel columns to support the cantilevers.

These inclined columns are the compressed elements of the "strut and tie" system proposed to resist the transverse bending moment produced by the new cantilevers (Fig. 15). The upper slab is then tensioned, so a transverse prestressing is proposed. This prestressing is made of 10Φ0.6" strands for each inclined column. Cables are placed into a flat duct, 30 mm thick.

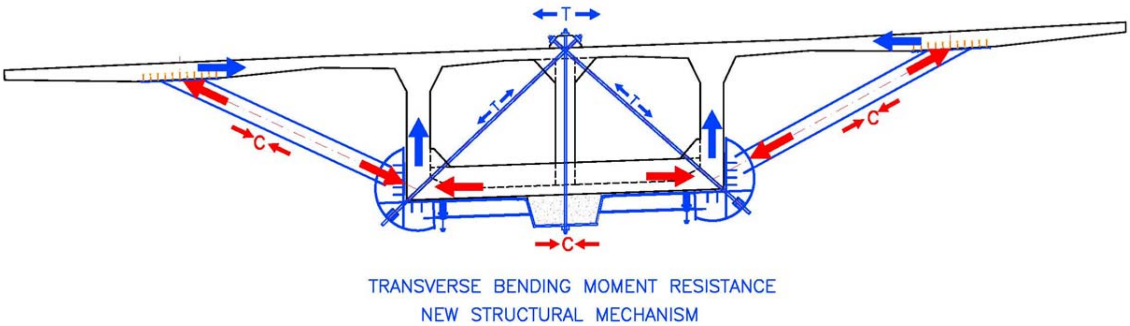

Fig. 15 "Strut and tie" system for resisting the transverse bending moments effects.

The inclined columns go from the cantilever of the upper slab to the lower corner of the original box section. There, a steel joint is proposed to split the inclined force from the column into two different forces: the horizontal force is then transferred to the lower slab using elastomeric bearings. Nevertheless, the vertical force of the joint should not be transferred directly to the adjacent lateral web of the original concrete box section, because it would increase the load to be resisted by those webs, which have a limited shear resistance. So, two pairs of diagonal connect the steel joint to the upper part of the new central web. These diagonals were made of high strength steel bars and were prestressed, in order to eliminate relative deflections when the total load was applied and to reduce the shear effort of the lateral webs.

As a result of the described resistance model, some high transverse compression forces are applied to the lower slab of the original concrete box section. In the sections close to the centre of the spans the lower slab is not thick enough to avoid instability. Therefore, a new beam had to be designed.

4.5.- CONNECTION OF THE DECK TO THE PIERS

The connection of the deck to the piers was originally made using a "triangular cell" located into the deck and connected to the walls of each pier. A reinforcement of this concrete triangular cell of the deck went into the piers and so the vertical load and the bending moment of the deck were transferred to the top of each pier. The loads to be transmitted from the deck to the piers are greatly increased when the platform is widened from 12 m to 23 m. In consequence, it was necessary to strengthen the said connection; furthermore, some reinforcing was affected when making some holes on the existing triangular cell for the external prestressing to go through the concrete elements.

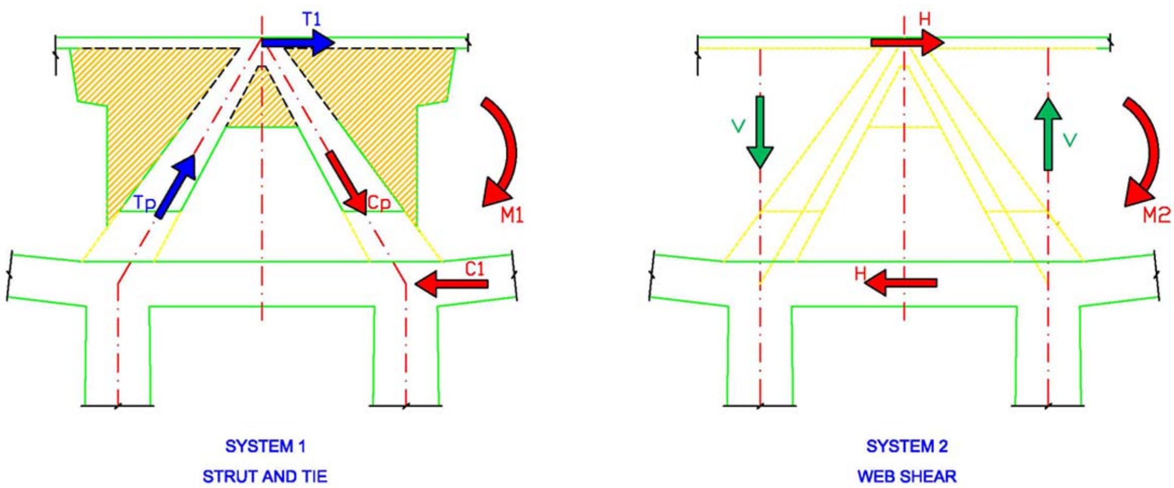

Fig. 16 Resistance models of the connection of the deck to each pier

Some detailed calculations of the said connection were made using linear FEM. The goal was to know the portion of the loads that were transmitted to the pier by the original "strut and tie" system, and the load that was transferred using the mechanism of "shear distortion" of the webs (Fig 16). The results of the calculations showed that nearly 55% of the total load was brought to the piers using the first model (strut and tie system) and the other 45% was connected using the webs.

5.- SOME COMMENTS ON THE CONSTRUCTION

In August 2007 some works concerning the improvement of the soil close to the foundation of pier no. 5 were started. In April 2009 the widened bridge was opened to traffic. It must be mentioned that different tasks were undertaken simultaneously. So, once certain activities were finished on certain spans, another construction works were initiated on these spans; the initial activities were then started on the adjacent spans, and so on. A very complex space and time dependent construction process was proposed. So, all along the works, almost all the activities were made at the same time but in different spans, sometimes working into the inside of the original box section of the deck and also making some external works on the platform at the same time. Undertaking different activities simultaneously was decisive to reduce the period of execution of the works. Finally, the original requirement of not interrupting the traffic (Fig 17) while the construction was in progress was fundamental in the planning phase of the works, except for some exceptional cases.

Fig. 17 Traffic on the bridge when strengthening operations were in progress using movable platform no.1

It is also to be mentioned that there were three different types of movable platforms that were specially built for this bridge (Fig 17 and 18).

Fig. 18 General view during construction (3 movable platforms operating at the same simultaneously)

Concerning the external prestressing, some difficulties were found when crossing the tendons with the diagonal bars located between the lower corner of the original deck and the upper part of the new central web. Also, the curved shape of the deck (700 m radius) was to be considered. When it was impossible to avoid the said interference, a special device was designed (Fig 19) in order to allow the longitudinal tendon to pass through.

Fig. 19 Special device because of the interference of the external longitudinal tendons with the diagonal steel bars.

The main activities for the construction of the new platform and for strengthening the deck are briefly presented below. They are sorted in the starting sequence that they followed.

a) Strengthening of the upper slab

- Demolition of the original pavement and substitution of the original steel guard rails.
- Carving of transverse paths, 5 cm deep, on the upper slab to place the transverse prestressing cables.
- Strengthening of the upper slab using normal reinforcement and steel plates attached to the concrete with epoxi.
- Thickening of the upper slab using 3 cm of fibre reinforced concrete.

b) Strengthening of the original concrete box section (Fig 20)

- Boring of the holes on the upper and lower slab for crossing the steel bars that will connect the original concrete with the new web and with the lower composite box.
- Construction of the new central web using lightweight self compacting concrete
- Manufacture and placement of the composite box section connected to the lower slab. Each piece of the longitudinal composite section was erected jointly with the transversal beams and a pair of lateral joints making something close to a “cross shape section”. Movable platform no. 1 was used. It weighs 68 t and was 23 m high.
- Pouring the concrete of the lower composite longitudinal box section using lightweight self compacting concrete
- Demolition of the original pavement and substitution of the original steel guard rail section.

Fig. 20 Composite box sited with platform no. 1

c) External prestressing

- Construction of the deviation walls inside the original deck. This deviation walls must be made after the diagonals that connect the steel lateral joints to the central web are tensioned. This is to avoid disturbing the distortion of the original concrete box section when the diagonals are tensioned.
- Strengthening the connection of the deck to the piers.
- First stage of the tensioning of the tendons of the external prestressing that is made prior the upper slab is widened.
- Second phase of the tensioning of the tendons of the external prestressing. This operation must be made once the upper slab has been widened.

d) Construction of the transversal structure

- Tensioning of the vertical bars connecting the longitudinal composite box section with the original deck.
- Tensioning of the diagonals that connect the original concrete with the new central web. These operations were made using two using two movable platforms no. 2 (Fig 21). They weight 6 t each.
- Manufacture and placement of the steel inclined columns.
- Construction of the new cantilevers (span equal to 6.5 m) for widening the upper slab. This was made using two movable platforms no. 3 (Fig 22). They weight 60 t each.
- Construction of new transversal beams in the lower slab of the original deck, to resist a potential instability due to the transverse compression transmitted by the inclined columns to the existing lower slab,
- Tensioning of the transversal prestressing of the upper slab.

Fig. 21 Platform no. 2

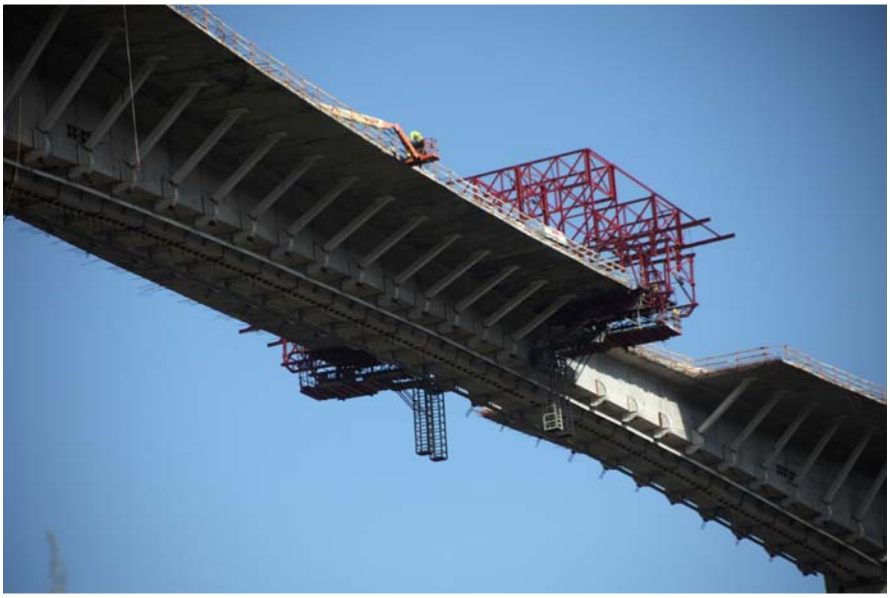

Fig. 22 New cantilevers built with platforms no. 3

e) <u>Finishing</u>

- Placing of the waterproofing device.
- Building of the guard rails.
- Paving
- Load test (dynamic and static load test)

Fig. 23 General view of the widened bridge during load test

6.- MONITORING

As it has been said, the construction of the new cantilevers, of the new central web and of the lower composite box section increased the original loads. Additionally, some new supporting elements were built, mainly the already mentioned central web and the lower composite box. The distribution of new loads among all the resistant elements was evaluated using detailed calculations, but it was decided to monitor some elements in order to validate the supposed distribution. So, different monitoring devices such as stress gauge, inclinometers and temperature sensor were installed on the bridge to validate the following:

- Distribution of vertical loads among the three webs.
- Loads transferred to the lower longitudinal composite box section.
- Structural behaviour of the transversal resistant system (distribution of loads of the joint located in the lower corner of the original concrete box section).
- Loads transferred to the piers.

All the devices were connected to a computer and so the measures were sent on real time to the Consultant in order to check the behaviour of the bridge, compared to that expected on calculations. Following (Fig. 24) a figure including the measured rotation of the top of pier no. 3 compared to the calculated rotation is presented. The accuracy of results can be clearly observed.

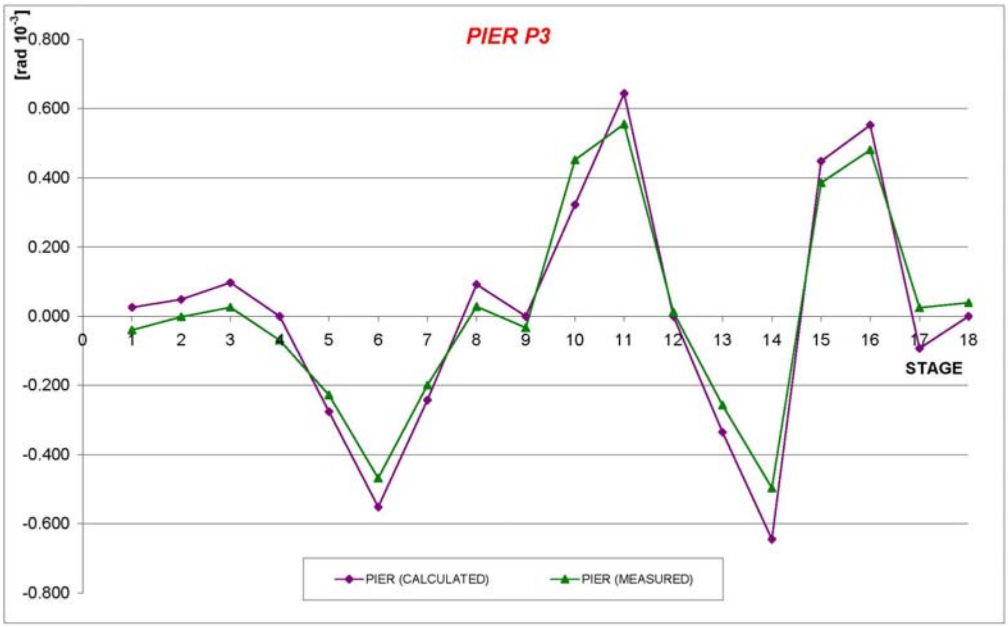

Fig. 24 Comparison of measured and calculated rotation of the top of pier no. 3

7.- CONSTRUCTION SCHEDULE, MAIN QUANTITIES AND TOTAL COSTS

The Final detailed design was completed on September 2006. In August 2007 some works concerning the improvement of the soil close to the foundation of the pier no. 5 were started. In April 2009 the widened bridge was opened to the traffic.

The total expected costs for all the construction works come to nearly 13 million Euros (19 million USD).

The quantities of the most important materials used for the construction were:

- Lightweight self compacting concrete ………….. 6 700 m^3
- Reinforcing steel ……………………...…………. 940 000 kg
- Prestressing steel ………………………………… 320 000 kg
- High strength steel bars …………………………. 190 000 kg
- Structural steel …………………………………... 1 390 000 kg

8.- ACKNOWLEDGEMENTS

Bridge owner…….. MINISTRY OF PUBLIC WORKS – SPAIN
Construction design. TORROJA INGENIERÍA S.L. (Madrid - SPAIN)
José A. Torroja, José M. Simón-Talero, Alejandro Hernández
Contractor ………. CUDILLERO JOINT VENTURE - SPAIN
Antonio Navarro, Arturo Bahamonde
Technical assistance during the construction works
MINISTRY OF PUBLIC WORKS – SPAIN
Ignacio García–Arango, Jesús Villameriel
TORROJA INGENIERÍA S.L. (Madrid - SPAIN)
José A. Torroja, José M. Simón-Talero, Alejandro Hernández
AEPO (Madrid – SPAIN)
Manuel Serralta, Manuel Juliá