# 3 launched composite bridges recently designed and built in Spain

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Abstract

*Launching bridges is a well known and usual procedure for erecting bridges where no access is allowed underneath the bridge. Nevertheless, some important details are often ignored while designing and calculating the bridge.*
*The purpose of this paper is to present details of launching of three bridges recently designed and constructed in Spain.*
*The GUADALFEO bridge is a five span continuous steel truss bridge. Central spans are 140 m length and deck width is 24 m. The tallest pier is 85 m high. The girder was launched from the abutment South using a provisional tower for decreasing the 2 m deflection of the tip of the cantilever.*
*The VICARIO bridge is a two span continuous composite bridge. Main span is 87.5 m and deck width is 24 m. The pier is 58 m high. The girder is curved (R=1420 m) and special care has been taken on distribution of reactions on supports while launching because of eccentricity of actions.*
*The ALVARES bridge is a seven span continuous composite bridge. Main span is 102 m and deck width is 27 m. The tallest pier is 66 m high. Procedure for launching the girder involved varying level of bearings taking into account pre-camber deflections of the girder. Ignoring them supposes increasing bending moments and patch loading effects to unsafe limits.*

## 1 THE GUADALFEO BRIDGE

### 1.1 Description

#### *1.1.1 Location*

Road N-323 crosses the said Guadalfeo river near the toes of the Mount Veleta, one of the highest mountains in Spain, where very special conditions of the surroundings can be found, such as:

- The area around Granada is a geologically active area with definite tectonic activity. In Spain a statically equivalent action can be used as an approximation of the dynamic effect produced by seismic actions. In doing so, the code proposes for this area a design acceleration equalling 0.22g.
- A bridge was proposed to be located where the Guadalfeo and Izbor rivers meet. As a consequence, some 15 to 20 m thick alluvial deposits are found. So, a deep foundation using in-situ piers was proposed for the main spans.
- Furthermore, there are some active large landslides on the vicinity of the southern abutment, because of the tectonic activity.
- Central piers are expected to be 90 to 100 m high. The distance between both valley slopes results 600 m, approximately.
- The construction of a new dam is undergoing 2 km down the river from the site of the proposed bridge. So, in the future the depth of the water will reach 60 to 70 m deep near the bridge.

During the preliminary design phases different solutions were considered, but only medium span bridges were selected because of the foundation conditions, using spans 80 to 170 m long. The number of piers was scarce, as the needed deep foundation did not reach unreasonable proportions. Arch bridges, cable stayed bridges and suspension bridges were eliminated because of foundation conditions.
Considering seismic resistance, the material of the deck was analysed. Steel girders were preferred to prestressed concrete girders: concrete decks are approximately 200% heavier than composite girders, and therefore seismic effects are double.

Finally, the existence of a 60 to 70 m deep water body during the construction of the deck was to be taken into account. Different construction methods were studied in connection with selected typologies of the deck. Among them, cantilever construction using temporary wires and launching of the deck were examined.

## 1.2 BRIDGE DESCRIPTION

### *1.2.1 Horizontal and vertical alignment*

The bridge over the Guadalfeo river is a five span bridge with a total length of 585 m. Its platform is 24 m wide, holding a dual carriageway with two lanes in each direction. The horizontal alignment is a very gentle curve, with a radius of 17200 m. Near the northern abutment there is a transition. The cross slope is 8% near the northern abutment; due to the slightly curved horizontal alignment, a cross slope of 2% is used in the rest of the deck. Concerning the vertical profile, nearly all the deck is placed on a slightly descending alignment sloped 0,20 %. The maximum distance from the upper level of the deck to the ground is about 100 m.

Abutments reach a height of 20 m on the Northern access and some 15 m on the south slope.

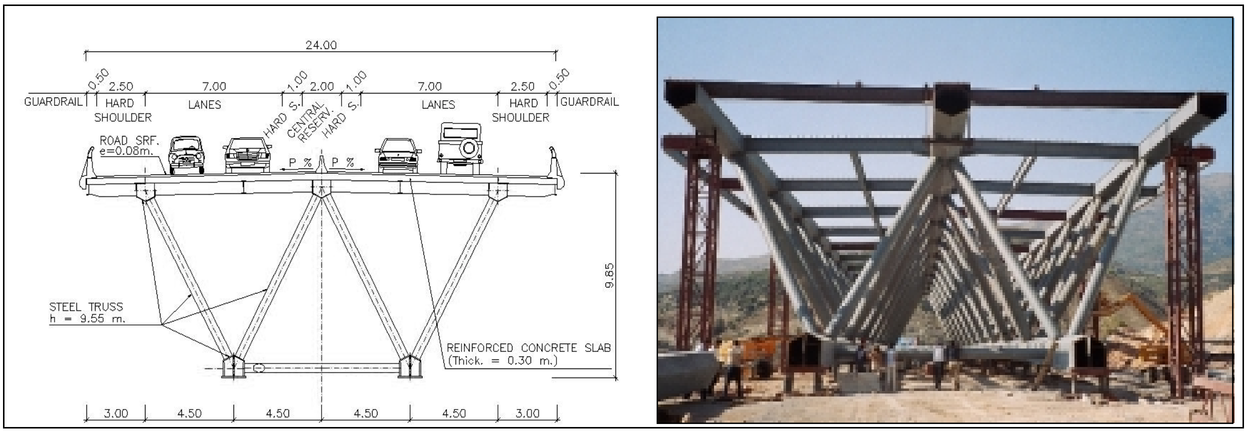


**Figure 1** Cross section of the deck.

### *1.2.2 Deck*

The proposed bridge is a composite truss 585 m long. The deck is a 5 span continuous beam. Spans are 85+140+140+110+110 m.

The cross section of the steel truss is 9.55 m deep. The truss is made of three main beams on the upper level and another two lower main beams. The distance in between main beams is 9 m. The cross section of each bar of the upper level is made of five plates 1000 mm deep and 950 mm wide. Its shape is pentagonal to optimise junction between main beams, transverse beams and diagonals. The cross sections of the two main beams of the lower chord are similar to the above mentioned beams of the upper level, but 1200 mm deep and 1280 mm wide. Additionally, inclined plates were located close to the connection between the webs and the lower horizontal plate. In doing so, a closed partial section was created to distribute the concentrated load from temporary bearings during launching to the webs. By doing so, patch loading effects were diminished. The thickness of webs varies from 15 to 50 mm. Thickness of horizontal plates of main beams varies from 15 to 60 mm.

Groups of four diagonal bars are used for connecting the main beams of the upper level to those of the lower level, with nodes distributed each 10 m. Each diagonal is a circular hollow section (CHS). Their diameter goes from 406 to 609 mm and their thickness is between 6.3 and 35 mm.

Secondary beams are located parallel to main beams of the upper level. An I-shaped section 600 mm deep was proposed. The connection of these longitudinal beams to the main beams is made using transverse I beams located each 10 m.

Therefore, a girder measuring 10 x 9 m is made by longitudinal secondary beams, main beams of the upper level and transverse I beams. On the top of the girder a reinforced concrete slab 30 cm thick and 24 m wide is placed. Nominal resistance of the on site poured concrete is 30 $N/mm^2$.

The steel grade for all beams, CHS´s and auxiliary elements is S355 ($f_y$ = 355 $N/mm^2$).

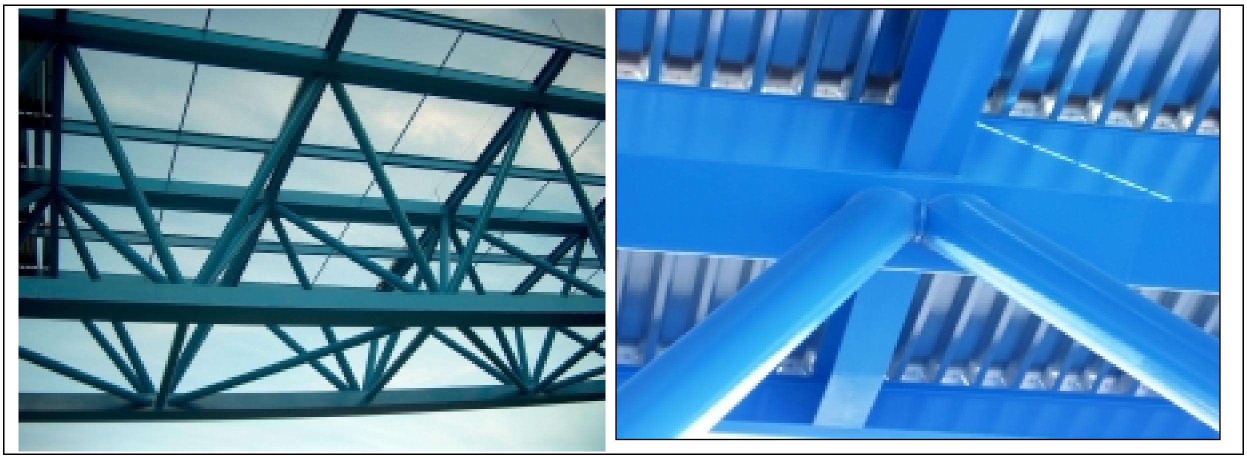

**Figure 2** General view of the steel truss and connection of the main beam of the upper chord with main diagonals.

#### *1.2.3 Substructure*

Piers are made of reinforced concrete with a compression strength of 30 $N/mm^2$. Their cross section is a hollow section measuring 4.5 x 12.0 m at the top. Their transverse dimension increases 2.5% all along the pier, reaching some 16.5 m on the base of pier P3, which is 90 m approximately high. The wall thickness of the piers is 35 to 40 cm.
The lower part of the pier is made of a stand with pyramidal shape, constituting a transition from the pier itself to the pile cap, which measures 28x12x3 m. The deep foundations of pier P2 and P3 are made by 17 concrete piles 2 m in diameter, whereas the foundation of piers P1 and P4 are made with 14 piles 2 m in diameter. As it has been mentioned, the piles cross a thick layer of alluvial deposits until the rock layer is reached. The longest piles are 31 m long.

The abutments are made of on site poured 30 $N/mm^2$ reinforced concrete. The northern abutment is 23 m tall. Foundation of both abutments is directly on superficial layers of adequate resistance.

As mentioned above, seismic effects equivalent to 0.22g are to be considered in the design and calculations. So, seismic devices were proposed. The two main goals of this design were:
- Eliminating longitudinal loads from the top piers.
- Diminishing global seismic effects to adequate limits.

In order fulfil these two conditions, seismic dampers were proposed. Detailed dynamical calculations were performed for evaluating the forces to be considered for different nominal velocities. Maximum target load was fixed to 15.000 kN per abutment and maximum allowed movement was considered to be $\pm$ 300 mm.

### 1.3 Proposed method of construction

The highest pier is approximately 90 m. So, sliding formwork was used for erecting all. As mentioned above, it was supposed that water will be flowing underneath the structure before the construction of the deck could be initiated. So, it was proposed to launch the deck. When preliminary calculations were made, a 2000 mm deflection was estimated on the tip of the truss. In order to decrease this high deflection, a temporary tower 40 m high was considered. This tower was balanced by two sets of cables with an adequate strength to resist a load of 22000 kN. In doing so, the maximum vertical deflection was reduced to 850 mm and loads on diagonals and main beams during launching were also reduced. It was not feasible to launch the first span near the north abutment due to its geometry. Additionally, access to the site was possible. So, normal construction was proposed for this first span, erecting steel structures using temporary piers and placing parts of the steel structure with cranes.

### 1.4 Structural analysis

Different structural models were used, both for general calculations and for local evaluation of particular effects.

- A finite element model (FEM), including second order analysis, was used for the entire structure. This model was able to consider the possible buckling of steel elements, instability of reinforced concrete sections of piers and contribution of sliding bearings.
- An interactive and time-dependent FEM model was used for studying launching process.
- A general dynamic model was used to estimate the seismic effects on the deck, piers and abutments, considering the contribution of specific implemented seismic devices.
- Different FEM local models were used for studying specific zones of the structure, such as the connections between diagonals and main beams, local bending effects while launching, the connection of the temporary tower to the truss, etc.

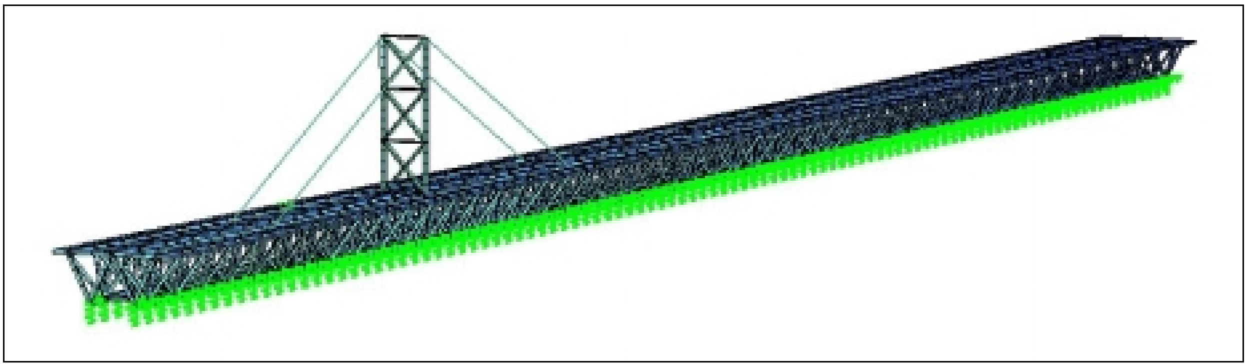

**Figure 3** Structure's general model. Launching stages

While still in the design stage, special care was taken to evaluate local flexular effects on main beams of the lower chords.

While launching the deck, main beams are sliding over temporary launching beams. Loads are applied to the truss not only on nodes but also on intermediate points all along main beams of the lower level. Therefore, local shear and bending moments are generated on main beams, working as a continuous beam supported each 10 m. These effects have been found to be the most severe loads on these main beams, so a careful calculation is necessary.

Loads coming up from piers and abutments are applied to the truss using temporary bearings. The dimensions of these bearings are 1500x500 mm. There are two launching bearings per section, able to support a maximum vertical load of 11250 kN. These bearings are neoprene – teflon bearings 30 mm thick. The longitudinal dimensions of the bearings are 1/8 of the total calculation span and bearing thickness is not very high. So, it is not reliable to suppose that the bearing reaction will be constant. It is necessary to take into account the longitudinal local flexural deflection on main beams, as well as the different deformation of the bearing along a distance of 1500 mm. Additionally, the local transverse distribution of the loads between both webs of the cross section is to be considered.

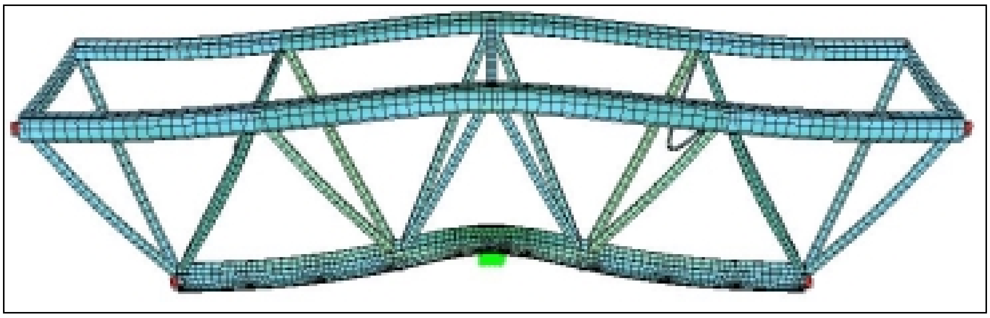

**Figure 4** FEM model for evaluating local flexural effects

The evaluation of all these structural phenomena is not easy. So, a detailed FEM model was prepared, in order to quantify importance of every effect. Figure 9 shows the local model used for calculations of local effects on the segment of the main beam of the lower chord.

Figure 5 shows the distribution of reactions on bearings on section SI2. The applied load was 6224 kN, and the maximum vertical compression stress was 200 N/mm$^2$, but quickly decreasing to some 96 N/mm$^2$.

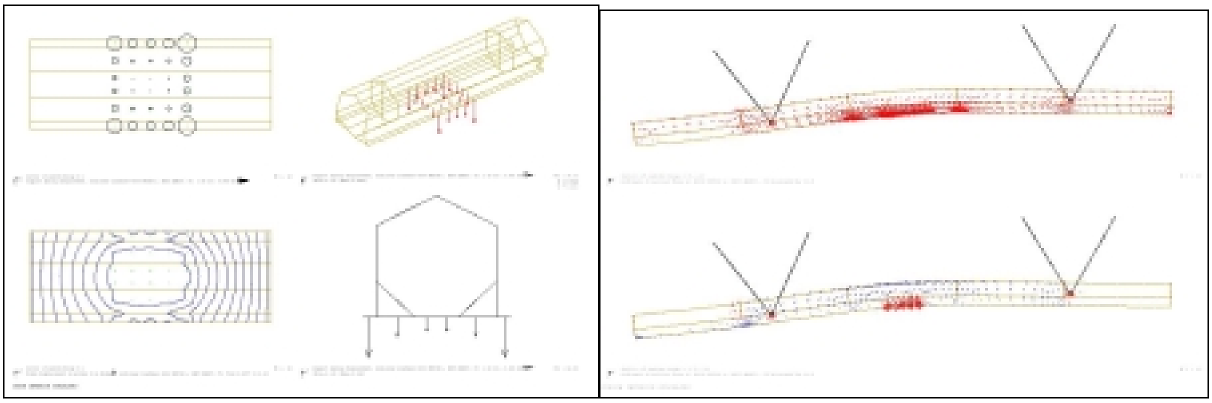

**Figure 5** Distribution of reactions on bearing section SI2and maximum principal stresses on section SI2 under load equal to 6224 kN.

Figure 5 also shows the maximum principal stresses on section SI2. The maximum vertical compression stress is due to patch loading effects. It reaches a value of 103 N/mm$^2$. The maximum transverse horizontal stress is produced by the distribution of loads from bearings to the inclined plates and to the webs. Its value is only 122 N/mm$^2$. Finally, the maximum longitudinal horizontal stress is produced by local flexural effects when distributing the load from the centre of a 10 m span to both nodes located 5 m far. The stress reaches 318 N/mm$^2$.

## 1.5 Manufacture, erection and launching

### *1.5.1 Manufacture and erection*

Approximately, 5300 tons of steel have been used to manufacture the steel truss. A total of 3200 tons were used for the main beams of chords, 1400 tons were used for diagonals and 700 tons were used for horizontal bracings. In addition, the temporary tower weights 150 tons and some 80 tons were used for different auxiliary elements. Finally, 460 tons of steel grade S235 were used in a profiled steel sheeting used as formwork for the on site poured reinforced concrete slab.

As it has been mentioned, diagonals have a circular hollow section (CHS). Their diameter goes from 406 to 609 mm and their thickness varies from 6.3 to 35 mm. The steel grade used for all elements is S355, and it has been manufactures and assembled in Spain.

Welded connections were made both in shop and on site. Full penetration butt welds were used for connecting different plates of the five side sections of the main beams of upper and lower chords. Joints between main beams and diagonals were made using fillet welds.

**Figure 6** General view during erection of the steel truss on the access to the South abutment

### *1.5.2 Launching*

Concerning the construction method, the first span was erected using temporary piers (see figure 13) and also using final bearings on abutment 1 and pier 1 for supporting the steel truss. The rest of the steel truss was launched from the Southern abutment. Because of the length of the main spans, the deflection of the tip of the truss reaches some 2000 mm while launching. To decrease this value, a 40 m tall provisional tower was implemented (see figure 14). This tower was balanced by two sets of prestressed cables. The capacity of these cables was 25000 kN. By doing so, the final deflection of the truss was reduced to 1200 mm, and bending moments and axial forces on the longitudinal beams were also reduced. Cables were tensioned to 45% of its maximum failure load when a 75 m cantilever is reached. Once the pier is reached, tension of cables is decreased to 20% of the initial load.

Profile steel sheeting is located onto main beams of the upper chords all along the width of the deck (24 m) before launching. At the back of the truss, some extra weight was placed during some stages of launching to counterbalance the structure.

### *1.5.3 Control of launching*

The tension on the cables supporting the auxiliary tower was controlled using a load cell near the anchorage of the cables at the top of the tower. In addition to that six strain gauges were located on the base of each leg of the tower to measure the normal force on it.

Two different sections of the truss were also controlled, trying to measure the real stresses on each stage of the launch. In order to do so the main beams and diagonals were studied. On each of the said two truss sections, 19 strain gauges were located: two on each of the three main beams of the upper chord, three on each of the two main beams of the lower chord and seven on the four diagonals of the section. Also, clinometers were located on each pier, and some temperature probes were located all along the truss.
Finally, the pulling force on launching system was monitored using another load cell connected to the pulling system.

**Figure 7** General view while launching

## 2 THE VICARIO BRIDGE

The Vicario bridge is located near the Guadalfeo bridge on road N-323, Granada, Spain. It is a two span bridge; length of spans is 85 m and the deck is 24 m wide. The pier is some 65 m high. Originally, the bridge was a four spans bridge of the same length than the proposed one that is 170 m long. But access to slopes was very difficult, so the Contractor proposed to eliminate lateral piers, but increasing the length of the spans. The deck is completely fabricated and launching is expected to be made on April 2006.

The cross section of the deck is a steel box 4.20 m high and 8.50 m wide. Each 4.25 m two twin systems made each one by a horizontal tie and an inclined compression profile is located. So, upper level of the steel structure reaches 24 m wide. The steel grade for all plates and profiles is S355 ($f_y$ = 355 N/mm$^2$). On the top of the steel box girder a steel sheeting is located and a reinforced concrete slab 37 cm thick is poured on site all along the width of the deck (24 m) after launching. The deck is launched from northern abutment. No temporary tower nor nail on the tip of the cantilever is to be used for launching but at the back of the beam, some extra weight has to be placed during launching to counterbalance the structure. Maximum expected vertical deflection is some 2100 mm,

Concerning structural analysis, detailed FEM modelizations have been used for studying patch loading effects produced while launching when heavy loads are aplicated to thin webs. Figure 9 shows local stresses produced on a thin web 12 mm thick by a load of 2666 kN , distributed by an elastomeric-TFT bearing of 1500x500x30 mm. Three different longitudinal stiffners were implemented for reducing local buckling.

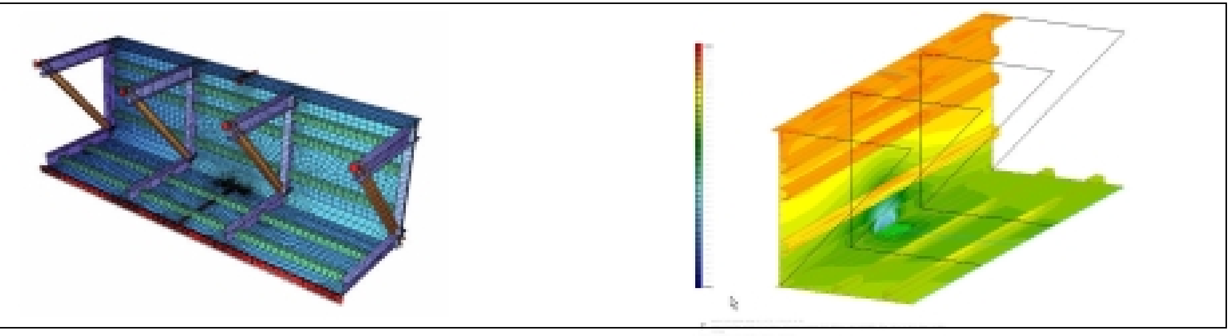

**Figure 8** Patch loading effects on thin web

# 3 THE ALVARES BRIDGE

## 3.1 Design criteria

Alvares bridge is located on the industrial town of Avilés, in the North of Spain. It allows motorway A-8 crossing over a river, a local road and two railway lines. The particular location of all this artificial and natural accidents and the fact that the western slope is very steep, have important influences on the configuration of the bridge; spans are 49+49+102.08+49+49+49+36.75 m. Total length of the bridge is 383.83 m and the highest pier is about 65 m high.

Because of the distance from the valley to the upper level of the deck and because of the existence of the river and of the two railway lines under the bridge, it was decided to launch the bridge from both abutments.

## 3.2 Bridge description

As it has been said, the bridge has a total length of 383.83 m, divided on seven spans. Main span is 102.08 m long. The platform is 27 m wide, holding a dual carriageway with two lanes in each direction. Vertical alignement is sloped descending 1.646 %.

The cross section of the deck is a steel box 4.485 m high and 8.50 m wide. Each 4.083 m two twin systems made each one by a horizontal tie and a inclined compression profile is located (figure 10). So, upper level of the steel structure reaches 27 m wide. The steel grade for all plates and profiles is S355 ($f_y$ = 355 N/mm$^2$). On the top of the steel box girder a reinforced concrete slab 26 cm thick is placed.

**Figure 9** General view while launching

Piers are made of reinforced concrete of a compression strength of 30 N/mm$^2$. Their cross section is a hollow section measuring 3.5x9.0 m at the top. Their smallest dimension increases 1.5% all along the pier, reaching some 5.5 m on the base of pier P2, which is 66 m approximately high. The wall thickness of the piers is 35 cm.

The foundations of pier P2 and P3 are made by 9 concrete piles 2 m in diameter, whereas the foundation of piers P1, and P4 to P6 is made directly on the rock layer. The longest piles are 40 m long.

Western abutment is made of on site poured concrete. It has 9.2 m wide. Sidewalls are 9 m long and follow the same alignement as the webs of the box girder. So, systems of tie and compression profile, as there are on the girder, are proposed for reaching a platform 27 m wide. On the top of this steel structure a cast in situ concrete slab, 26 cm thick, is placed.

Eastern abutment is a poured on site concrete structure. It has a 11 m high front wall and two sidewalls 18 m long.

Foundation of both abutments is directly on superficial layers of adequate resistance.

**Figure 10** Cross section view while launching

### 3.3 Proposed method of construction

The box girder was manufactured on workshop and then it was carried on site using some heavy lorries. In order to make transport easier, the box girder was divided in the middle and so pieces were 4.5 m high, 4.25 m wide and 12 m long. Weight of each piece was some 35-40 tons. Once each piece was on site full penetration butt welds were used for connecting different steel pieces. Once the box girder was completed the systems tie-compression profiles were connected also using full penetration butt welds. Nearly 2400 tons of steel were fabricated.

Once the box girder was assembled, it was launched: 234.79 m from western abutment and 149.04 from eastern abutment. Then, concrete on the bottom of the steel box girder, close to piers, was poured, and the top slab was built. A 10 cm precast concrete slab was placed and, then, a 16 cm concrete slab was poured on site.

Concerning piers, creeping formworks were used for all piers.

### 3.4 Structural analysis

While still in the design stage, special care was taken to face different particular problems:

- Instability of the piers
- Loads on temporary elements and bearings
- Evaluation of deflections and loads while launching, specially where these effects conditioned the dimension of the element
- Local flexural effects on the webs and evaluation of patch loading effects

### 3.4.1 *Instabily of piers*

Buckling of piers has been studied using a FEM global model including all relevant structural elements: steel beam, reinforced concrete slab of the deck, bearings and piers themselves. Reinforcement, non linear material behaviour and cracking of concrete have been considered on the analysis.

Figure 11 shows the results obtained from two different models:

– Case A is the result of a "traditional calculation" used very often for this kind of problems. That is, obtaining loads on the top of each pier using a first order analysis on a global model. Then, these loads are applied to a simplified model and a second order analysis of the single pier is performed.
– Case B is a global calculation using a FEM model able to consider second order effects and also the interaction between the deck, the bearings and the piers.

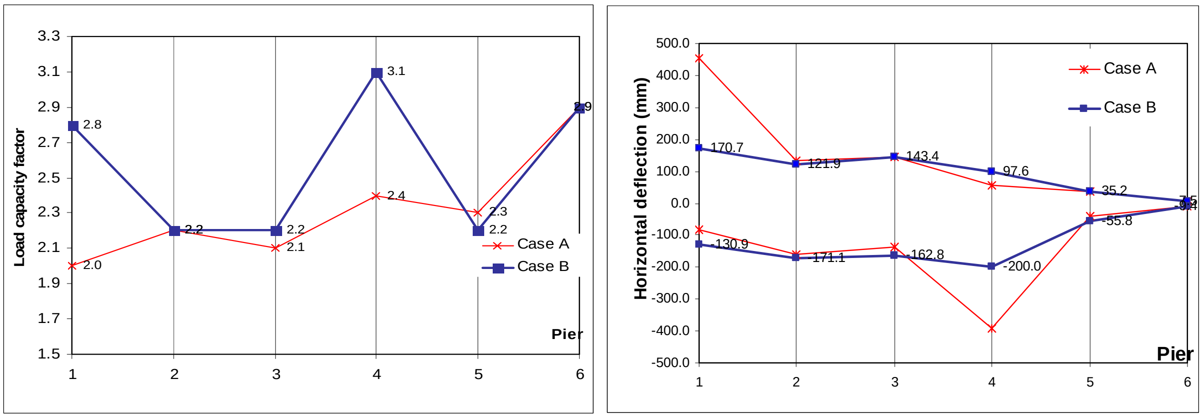


**Figure 11** Instability of piers. Comparison of results of different models

Results are presented for the relevant design combination, that is, maximum longitudinal effects. These results are presented in terms of strength (load capacity factor) and of deflections of the top of the pier. As it can be deduced from the figure, usage of global second order calculations could be very effective for reducing amount of reinforcement on slender piers, but, in some cases, not to decrease the need for the level of structural capacity of the structure.

### 3.4.2 *Influence of precamber deflections on flexural effect while launching*

The box girder was fabricated considering expected deflections due to self weight and dead loads. So, pre-camber on centre of span 3, 102 m long, was 380 mm. Spans 2 and 4 have a precamber on centre of spans equal to –48 mm, that is, opposite to the main span. Using a FEM structural model, the maximum loads on temporary bearings were estimated for each phase of launching. Figure 12 shows the maximum vertical load calculated for each section, considering three different models:

– Case A: it is supposed to have jacks placed on each temporary bearing. So, level of supports can be adjusted and launching is performed as if the girder would be in a "horizontal" position, with no influences of precamber deflections.
– Case B: it is supposed that no jacks are available. Consequently, precamber deflections of each section produce some flexural effects on the beam.
– Case C is the proposed method, where some adjustments can be made on particular temporary supports during specific stages of launching.

Figure 12 also includes the "patch loading nominal resistance" of webs. Results show that some sections are very sensitive to precamber deflection. That is, under certain circumstances, precamber deflections cannot be neglected for estimating loads on temporary launching bearings. Consequently, it is sometimes necessary to use jacks to maintain the loads on bearings under certain limits. Figure 12 also shows minimum bending moments considering on the said three cases. As it can be deduced from the figure, neglecting the existence of precamber when calculating bending moments leads to unsafe results. On the other hand, if no additional measures were adopted, it would be necessary dimensioning the beam for bending moments greater than those produced if all bearings could be adjusted to completely eliminate the effects of precamber.

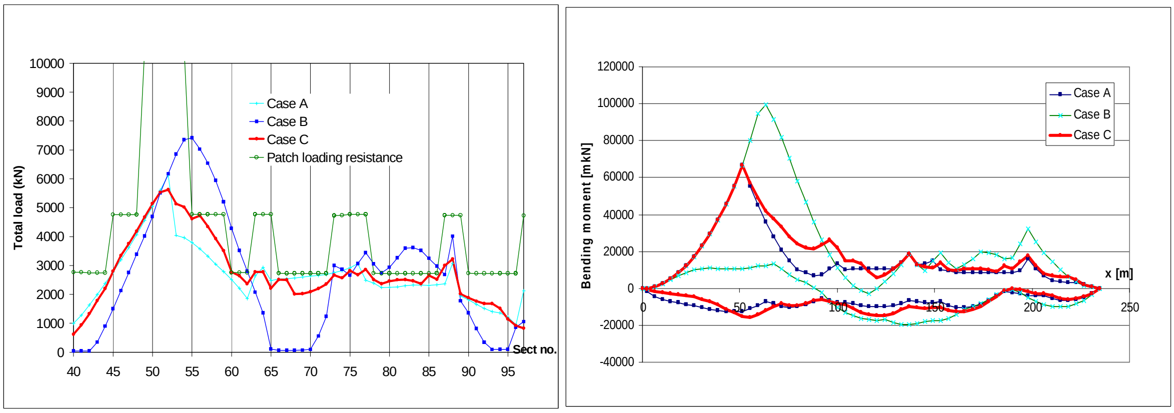


**Figure 12** Influence of precamber on reactions and on bending moments when launching

### 3.5 Manufacture, erection and launching

The total weight launched from western abutment was 1450 tons. From eastern abutment total weight was 940 tons. Considering a target friction coefficient equal to 7-9 % and considering the slope equal to 1.646%, pulling forces necessary for launching were expected to be 1500 kN for western abutment and 750 kN for eastern abutment.

Some horizontal jacks were used for launching from western abutment the two first sections of the box girder and for launching the whole section of the steel beam from eastern beam. Jacks have a capacity of 400 kN, and a maximum stroke of 750 mm. Actually, two jacks were used on abutment east for “pushing” the steel beam and another two jacks “pulled” the box girder from abutment west. For the final section launched from abutment east, weight of the beam was too heavy, and a couple of high strength steel tendons (7T15) were used for launching.

**Figure 13** Pushing jacks and pulling system made of high strength steel tendons

Concerning temporary bearings for launching, elastomeric-TFT bearings were used. Capacity of bearings were 3000 kN and maximun stroke reaches 400 mm. Adjusting level of bearings allows varying forces on temporary bearings to reach the reaction expected on calculations.

For launching and for changing bearings from temporary ones to the final bearings it was necessary to design different auxiliary elements such as: anchorages of the pulling jack to eastern abutment, lanes for launching, lateral temporary supports of abutments, elements for connecting the pulling tendons to the steel beam, connections of the temporary bearings to the piers and lateral wind supports.

In addition to that, connection of central section of the main span P2-P3 was made using high strength bars for levelling both parts of the central section of the main span. Then a “narrow” piece of steel, 300 mm long, was welded to both sides of the central section and so, the steel beam of the main span was finished.

Finally, for changing from temporary bearings to the final bearings, hydraulic jacks were used. When doing so lateral stability was assured using lateral temporary supports. When temporary bearings were removed holes made in the piers were filled with on site concrete.

### 3.6 Control of launching

Bending moments, shear and vertical deflections are governed by both the length of the cantilever span and the level of temporary bearings on each stage of launching. As it has been mentioned above, temporary bearings with vertical jacks has been located on piers and on abutments. So, measuring loads on bearings and their vertical level was proposed for controlling launching. Figure 14 shows expected and measured loads on temporary bearings on each stage of the launching. Mentioned figure also shows vertical deflection on the tip of the cantilever.

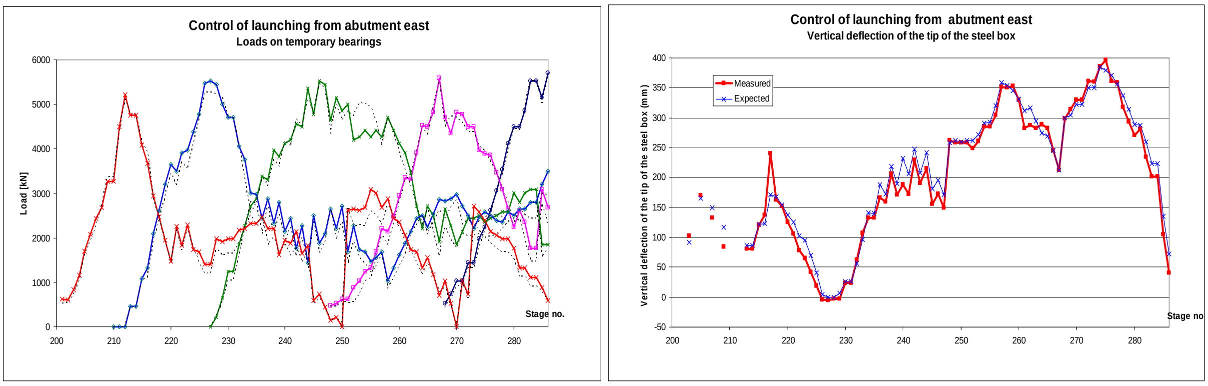


**Figure 14** Control of launching. Loads on bearings and vertical deflection of the tip of the deck

Concerning vertical deflection it is important to say that calculation of expected deflections is a hard job beacuse of the different parameters that are involved on these calculations. Vertical deflection produced by the dead load applied on the cantilever beam is the first one. But it has also to be considered the flexural deflection produced on the deck because of bending moments due to the fact that the level of bearings does not follow exactly precamber of the deck. Finally, rigid solid movement is evaluated for taking into account correct level of the bearings.In the end.

As it can be deduced from figure 14, relationship between expected and measured structural effects was very accurate.

Concerning friction coefficient on temporary bearings, total load and pushing or pulling force were measured. Slip factor varied depending on different phases of the launching: when beginning each launching slip factor varied from 5.5% to 11.4% and when the steel beam was in motion slip factor coefficient decreased to 2.4% to 4%.

**Figure 15** General view when closing central span

# 4 ACKNOWLEDGEMENTS


## 4.1 Guadalfeo bridge and Vicario bridge

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