Railway Arch Bridge over the Contreras Reservoir on the Madrid-Levante High-Speed Railway Line

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1. Introduction

Part of the Madrid-Valencia high-speed railway line, the Contreras Reservoir – Villargordo del Cabriel stretch represents an example of the use of the state-of-the-art construction systems, instigated by the limitations resulting from a layout that allows the running speed of up to 350 km/h, with large radii bends and gradients of less than 30‰ in area with rugged terrain conditions. It is against this background that the arch bridge over the reservoir stands. This is a reinforced concrete arch bridge with a 261 m span and an upper prestressed concrete deck that on the construction completion date was world record holder for a concrete railway arch bridge (Fig. 1).

Figure 1. View of the arch bridge built over the Contreras Reservoir
2. General Description

The bridge amounts to a total length of 587,25 m. The arch span measures 261 m and the mid-span sag is 36,944 m, which determines a span-to-rise ratio of 1/6,77 (Fig. 2).

![Figure 2. Elevation, plan view and cross sections](image)

The arch is embedded in two large plinths that allow the diffusion of the load over the affected ground by means of direct foundations. It is divided in six parts with a polygonal directrix. That way the non-funicular arch is maintained while reducing the bending moments that would exist in the area between the vertical columns if the arch were perfectly curved.

The cross section is a box girder with a variable depth ranging from 2,80 m at mid-span to 3,40 m at the ends. The box girder width is also variable ranging from 6,00 m in the centre of the arch to 12,00 m at the foundations, which is the width required to resist the great bending moments of the vertical axis produced by the plan curvature of the arch and the crosswind. The box girder walls range from 0,60 to 1,35 m.

The upper deck span distribution is 32,625+12x43,50+32,625 m. The piers P-6 to P-11 are supported on the lower arch structure. The deck is made of a 3,00 m deep box girder (which determines a 1/14,50 span-to-rise ratio), a 5,00 m wide lower slab, a 6,50 m wide upper one, and series of segments that complete the total section width of 14,20 m. The web thickness is 0,50 m. The webs are thickened over the piers until reaching a total thickness of 1,27 m, to allow the anchoring of the service prestressing cables. The lower slab is 0,30 m thick.

The arch has a polygonal curved directrix in the vertical plane, which corresponds to the non-funicular of the permanent loads. In plan the arch is drawn within the circular alignment of 3 875 m radius in order to avoid eccentric forces at the points where the piers are built-in in the arch. It is made of reinforced concrete C-70, due to the great compressive forces it must bear.

The variable height of the piers ranges from 3,53 to 35,38 m. All the piers are generated by one basic pier which has a rectangular box-girder cross section of a 2,60 m constant width and a variable depth ranging from 5,20 m on the upper edge, 3,20 m at the “waist” situated 5,00 m away from the upper edge, and a widening towards the base.

3. Main Structural Features

3.1 Results of the static analysis

The annexed graphics (Fig. 3) represent self-weight forces and dead load in terms of axial forces, bending moments in the horizontal and vertical axes as well as deflections produced. It also shows permanent load forces compared to those due to creep, shrinkage and temperature:
• Sag reaches 150 mm if the arch is built in one go on a centering.
• The great importance of the vertical axis bending moments is shown, due to the plan curvature of the deck, \( R = 3\,875 \, m \), and the arch.

The figure also shows the minimum and maximum forces in the arch and deck under the superimposed live loads of traffic and crosswind. The maximum and minimum bending moments of the deck in the area over the arch end up having a magnitude in the order of twice as large as that corresponding to a deck with the same span but supported on the ground instead, and lacking the effects produced by the vertical deformations of the arch.

![Figure 3. Permanent load forces and imposed movements. Live load forces: traffic and wind](image)

### 3.2 Studies to establish the arch directrix

The arch directrix was obtained taking into account the non-funicular of all permanent loads and not only the dead load of the arch itself. In the non-funicular analysis, significant concentrated loads were introduced on the arch, coming from the piers supported on the arch. The following effects are obtained:

• The arch directrix tends to be polygonal.
• The effect of loads uniformly distributed on the deck provokes a positive bending moment in the ends of the arch, due to the fact that the shape of the arch adopts a disposition that tends to slightly pull towards the frame instead of the two-hinged girder.
• Consequently, the non-symmetrical traffic live loads provoke greater positive than negative forces in the arch ends.

Due to the elastic shortening of the arch, which produces positive moments at the ends and negative ones at the key, a zero bending moments diagram cannot be directly obtained. To offset this fact, the following possibilities were studied:

1. Leaving as the diagram of permanent load the one provoked by the elastic shortening.
2. Tensioning the last stay in excess to offset the negative bending moment provoked by the elastic shortening of the arch.
3. Unloading the stays used for the construction by incremental launching at the key, using jacks to tension the arch. The axial forces used can only be those provoked by the arch dead
load. This is why, finally, only a rather flat diagram of bending moments can be achieved, although it is entirely displaced from the source.

4. In order to offset the displacements due to the diagram of bending moments provoked by the previous diagram, thrust at the key can be applied by installing the jacks eccentrically.

3.3 Long term analysis of the concrete

The effect of creep due to permanent axial load and due to creep is homothetic and similar to that produced by an elastic shortening of the arch. This implies negative bending moments at the arch ends and positive ones at the key. However, the effect of the bending moments of permanent load is the opposite. For this reason, creep provokes quite reduced effects at the arch ends, precisely due to the bending moments of the initial permanent load of the arch. In other words, if there were no such diagram at the start, due to the long term behaviour of the concrete, unduly large negative moments would appear at the ends and positive ones at the key.

Regarding option 4 of the previous point, the long term effect of the eccentric thrust at the key cancels 2/3 parts of the bending moments obtained at the start. This is congruent with the analysis using Dischinger model of a split second action, according to which $M_f = M_0 \cdot e^{-\lambda}$, $M_f$ being the final effect and $\lambda$ the creep coefficient with $\lambda=1,10$. Such a low coefficient is consistent with the fact that the average age of the arch concrete is of approximately 120 days at the moment of loading the arch in order to connect it.

The conclusion to be drawn is that in this case a diagram of bending moments of the permanent load provoked by the axial shortening of the arch is not harmful, since it helps to offset the unbalance between the maximum positive and negative bending moments of the ends of the arch and reduces the long term effects of creep and shrinkage of the concrete at the ends. Between option 3 and 4 it is the second one which has a beneficial effect since it helps even out the positive and negative moments in the spring of the arch, although creep cancels a large portion of its effect rendering its application needless.

3.4 Non-linear analysis

The hypotheses verified for this analysis are:

1. Maximum live load on both tracks, loading the entire deck on the arch.
2. Maximum live load on both tracks, loading half the deck on the arch.
3. Same as the first case only loading just on one track.

When evaluating the transverse effects, we verified that the case 1 provoked greater transverse moments. This means that the arch curvature is more important than the centred load. Case 3, however, produced greater torsional moments in the arch.

A structure functioning comparison was carried out based on the following hypotheses:

- Wholly linear behaviour
- Wholly linear behaviour, with geometrical matrix of continuous loads.
- Non-linear behaviour, with geometrical matrix corresponding to live loads considered.
- Non-linear behaviour, with geometrical matrix corresponding to live loads considered and cracking of the arch segments. The geometrical matrix of permanent loads considerably affects the behaviour. It worsens the strength mechanism, which results in an increase of loads on the deck and the arch. The behaviour of the geometrical matrix of factored loads obtained through iteration does not vary greatly in comparison with the previous point.

In conclusion, the results obtained through a wholly linear analysis with the geometrical matrix of continuous loads are sufficiently precise, thus validating the analysis carried out for the general behaviour of the bridge.

3.5 Dynamic analysis

Complete dynamic analysis was carried out with all HSLM trains and the Talgo AV. The number of
modes was 150, including all the vibration frequency modes lower than 30 Hz. The speed scope ranged from 120 to 420 km/h with a $v_0 = 5$ km/h. The modal damping was $\xi = 2\%$ for all modes. Analysis was also carried out for the quasi-static speed of 20 km/h. The time pace was $t = 0.03$ seconds, measuring the centre of the span and the support of the first span, from the middle of the arch and from the end spans, with the trains circulating from abutment E1 to abutment E2 along the track closest to the reservoir dam.

The conclusions of the analysis are the following: the dynamic behaviour of the bridge under actual high-speed trains is not decisive; the maximum accelerations are always lower than the code-established limits of 3,50 m/s$^2$ and the impact coefficient resulting from dividing the maximum results obtained from a UIC typical train and those obtained from actual trains at high speeds do not surpass in any case the impact coefficient used for the UIC typical train (Fig. 4).

![Figure 4. Results of the dynamic analysis at the arch key](image)

4. Construction Process

The construction method eventually chosen for this bridge and given its situation with respect to the water of the reservoir and the land was the cable-stayed free cantilever launching of the two semi-arches embedded in their foundations (Fig. 5).

A slight modification was taken into account. The proposal was to build the arch by cable-stayed incremental launching as well, only it was to be launched from the first pier of the arch, which was to be extended until reaching the ground where it was then appropriately. At the beginning of the construction, after a particularly favourable hydrological year for this purpose, the reservoir water level was such that the foundations of the temporary pier were above water level for months on end.

The construction process was carried out by first executing the approach viaduct and the deck piers using a climbing formwork for the piers and scaffolding truss for the deck.

The first section of each semi-arch, between the foundation and temporary piers, is built upon a centering supported on the ground. Once the centered arch section is built, piers P-7 and P-10 are executed over the arch, to allow the advance of the scaffolding truss towards these piers.

The centering is then dismantled and the advance of the semi-arches is initiated using cable stayed free cantilevers. To this end metal pylons were placed on the deck, following the vertical line of the temporary piers. From this moment on, the semi-arches advanced in free cantilevers while casted in situ using form traveller. To enable such procedure, we placed nine successive bundles of stay cables on each semi-arch.

One very important matter to consider is whether the use of jacks at the key in a bridge built using temporary cable staying should or should not be applied. Jacks at the arch key are aimed at eliminating the forces and strains produced in the arch as a result of the deformation provoked by shortening of the directrix due to axial compression.

In this case, a large part of the axial force is produced once the arch has already been completed. Consequently, the elastic shortening and the corresponding forces in the bridge thus produced may be eliminated, if desired, by placing jacks at the arch key. By contrast, if the cable-staying is placed on the pier situated over the foundations, the axial forces produced in the arch during construction are much larger than those produced when the cable-staying is carried out from the first pier. Therefore, the axial force that remains to be received by the arch once connected is smaller, which
also reduces the need for placing jacks at the key. In any case, this conclusion is minimised if we take account of the fact that the cable-staying and its compressions only produce values in the order of one half of the axial forces of the self-weight of the arch alone.

In this case, it was decided against placing jacks at the key, since nothing is gained from the structural point of view while it complicates the execution. This by no means implies that such a decision is to be generally applied in all cases.

**Figure 5.** Cable-stayed free cantilever launching. Drawing, model and reality.
5. Conclusions

At the time of its completion this was the longest span concrete arch railway bridge in the world, L=261m.

It presents an interesting balance in all its variables:

- The sag/span ratio is low, yet not too low: 1/6,77
- Depth at midspan = 2,80 m → L/93
- Depth at the feet = 3,20 m → L/76,2

The upper viaduct presents no discontinuity in its spans neither regarding the terrain nor regarding the arch l=43,50m. The fact that the span of this viaduct is not constant throughout its length is not understood quite so well.

The bridge plan is curve as is the arch plan. The clothoid is developed between a 3.000 m arch and a 4.000 m one, which greatly increases the stresses in the arch produced by the vertical axis bending moments.

The arch is slightly polygonal in order to make it perfectly funicular.