ALMONTE VIADUCT – DETAILED DESIGN

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SUMMARY

Given the scale involved, construction of this bridge necessitated more comprehensive engineering and detailing for some of its components. These tasks were performed by the builder's staff to ensure coordination as full as possible between the design and the construction teams, as described in this paper.

Keywords: *Concrete arch bridge, world record railway arch bridge, temporary cable stayed construction, self-consolidating concrete, high-strength concrete.*

1. INTRODUCTION

The River Almonte Viaduct is located on the high speed rail line running from Madrid to Extremadura in southwestern Spain. At some future time, this railway will be extended to provide high speed transport between Madrid and Lisbon. The stretch of line near Cáceres runs across an area of great environmental value where the Rivers Tagus and Almonte flow into the Alcántara Reservoir.

The main section of the viaduct was consequently designed as a 384 m concrete arch that straddles the river from bank to bank with no intermediate piers (Fig. 1). At the time of completion, the arch held the record for being the longest span on a steel and concrete railway bridge in the world and was among the longest on all types of arch bridges.

Fig. 1. *Overview of bridge.*

Construction on the viaduct began in August 2011 and on the arch in April 2012. The arch was completed in August 2015 and at this writing completion of the bridge as a whole is scheduled for summer 2016.

The rail line is owned by ADIF, Spain's railway infrastructure management company. ADIF commissioned the design of this section of the line from a Spanish consortium whose members are IDOM and Arenas y Asociados. The viaduct was designed by the latter. Construction of the section was awarded to a Spanish-Portuguese consortium in which FCC Construcción, the Spanish member, holds an 85 % share and Conduril the remaining 15 %.

Given the complexity of the project and the challenges inherent in bridge construction, after awarding the project the owner realised that the bridge design, in particular the construction details and the detailed design, would require additional engineering. The construction consortium, in turn, proposed certain adjustments to the detailed design to adapt the project to the resources that were to be used. ADIF entrusted this part of the work and the organisation of all the engineering for the detailed design and details to FCC Construcción's Engineering Department, while the design team was entrusted with supervising both the detailed design and construction process.

2. STRUCTURE

The viaduct's continuous 996 m deck has a straight horizontal alignment and a gentle 45 000 kV vertical transition curve. The deck is a post-tensioned box girder with a constant depth of 3.10 m and a constant width of 14.0 m. Its main section is flanked by approaches on the north and south (Fig. 2).

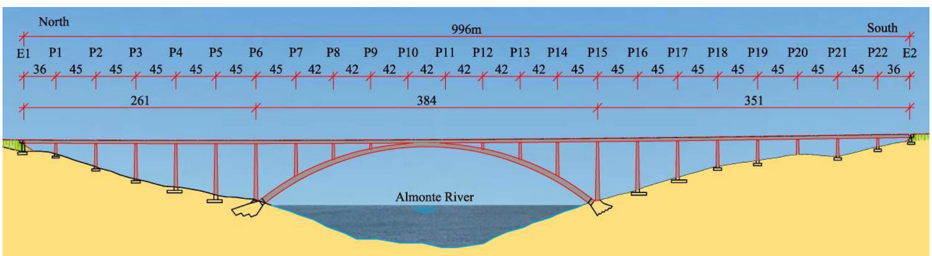

Fig. 2. Elevation sketch of bridge.

In the structural scheme of the bridge, the deck is longitudinally free and transversally restrained. The deck converges on the arch crown to resist the longitudinal forces induced by the braking and starting typical of railway bridges. The arch and deck in fact merge at the crown to form a single member.

2.1. Approaches

The approaches connect the main part of the bridge (the arch) to the abutments. The span arrangement in the 261 m long north approach is 36+5 x 45 m, and in the 351 m south approach, 7 x 45 + 36 m. The continuous post-tensioned deck was built in situ with overhead movable scaffolding system, one on each approach.

2.2. Main bridge

The deck on this part of the bridge rests on eight piers supported by the 384 m span arch. The deck is continuous throughout, i.e., including the approaches and this main area. Its two end spans over the arch measure 45 m each, and the seven inner spans 42 m. The centre span differs from the others in that the middle 17 m merge with the arch.

With a 67.5 m rise, the arch has a very airy span/rise ratio of 5.7. It features a fairly singular geometry, inasmuch as it has dual members up to a distance of 87 m from the abutments, where they merge and continue as a single section to the crown. Its cross-section is variable (Fig. 3).

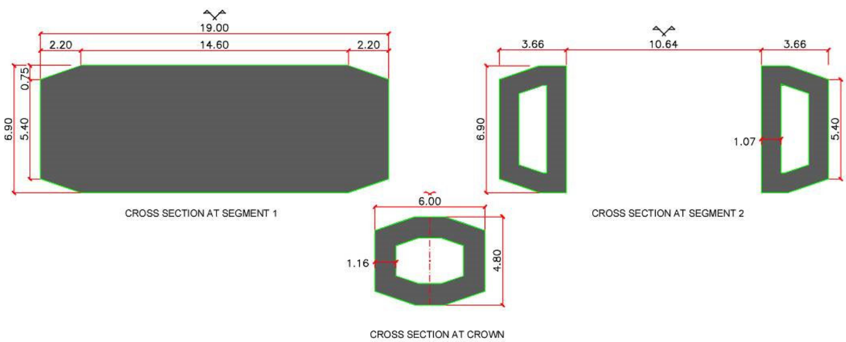

Fig. 3. *Cross-sections on the arch.*

The cross-sections are shaped as follows.

- Double section area: the 4.17 m long base has a solid octagonal cross-section measuring 19 m wide by 6.90 m deep. The two legs that arise from this base have hollow hexagonal cross-sections measuring 6.90 m deep by 3.70 m wide. This configuration changes gradually as the width of each hexagon grows and the two draw nearer until they ultimately merge into a single octagonal member at 87 m from the starting point.
- Single section area: here the hollow cross-section is octagonal with dimensions tapering from 6.09 m deep and 8.37 m wide to a depth of 4.80 m and a width of 6.0 m at the crown. The wall thickness varies from 0.97 m to 1.16 m.

This part of the bridge was built by cantilevering supplemented by a provisional cable-stay system with back-stays anchored in the footings under the two piers adjacent to the main piers at either end of the arch. Provisional towers built on these main piers (piers 6 and 15), in conjunction with the piers themselves, served as temporary pylons.

The two travellers on each side of the river used for cantilevering were converted into one where the two legs merge. The arch was built with high-performance (80 MPa), self-consolidating concrete. Whilst the consistency of this material was fluid enough to fill in all the voids between the rebar with no need for vibrating, it required formwork on all the sides of the cross-section. The travellers were consequently designed with forms that, once sealed, were water-tight to avert leakage.

3. DETAILED DESIGN FOR THE BRIDGE

As noted earlier, part of the original design for the bridge had to be developed more fully or modified. The most significant works about detailed design is discussed below.

3.1. Arch foundations

The shallow foundations for abutments, piers and arch were sunk into rock. Concrete was injected into the terrain under the arch foundations to reduce its deformability and restore any rock that may have been altered during the earthworks. The arch rests on footing shared with piers 6 and 15, the two closest to the reservoir on each bank.

The arch foundations had to be adapted to the actual terrain as surveyed at the outset of the works. The footings were analysed with several finite element models to determine the stresses transferred to the terrain. According to the geotechnical survey conducted, the terrain was able to accommodate the stresses derived from the models: a mean value of 1.2 MPa and a maximum of 2 MPa.

The flow of the principal stresses obtained with finite elements was applied to a several strut and tie models, in turn used to design the reinforcement for the footings.

For the intents and purposes of concrete casting, pier 6 was divided into nine parts and pier 15 into six to avert the risk of insufficient concrete supply and avoid high heat of hydration during setting.

3.2. Double leg-single leg connection

The connection was analysed on the occasion of the detailed design. The problem lay essentially in the fact that two of the inner walls in the dual section (double hexagon) area are discontinued in the single section (octagonal) area (Fig. 4).

The design issue to be solved was that in the dual section area, each leg transfers a factored axial force of 250,000 kN to the merged section. Therefore, a way had to be found to transfer to the outer walls the forces of approximately 120,000 kN acting on the inner walls in the dual section area.

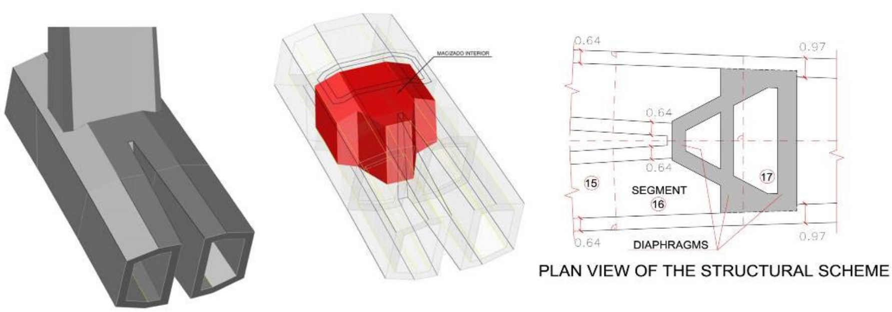

Fig. 4. *Structural scheme, plan view and detail.*

The mechanism devised consisted in building two inner vertical walls forming a 'V' in which the vertex would be positioned on the inner walls of the dual legs, while the arms, which would connect into the top and bottom of the box girder, would end at the outer walls of the single section. The load flow is as follows:

- Axial load transferred directly to the lateral walls: 40 % of the load on the inner walls with a transverse brace to prevent load deviations
- Shear transferred to the top and bottom slabs: 60 % of the load on the inner walls.

The need for transverse bracing to transfer the loads from piers 8 and 13 to the arch rendered joint engineering even more complex. In the solution ultimately adopted, to avert the need for inner hollowing that would have hindered construction, the joint was cast as a solid, although the reinforcement was distributed as modelled.

3.3. Deck-arch connection

The deck merges with the arch in the central 17 m over the crown. This connection was carefully engineered in the original design as a gradual convergence of the two members, arch and deck, into one.

This connection is instrumental to bridge functionality, for it transfers all the longitudinal stresses from the deck to the arch across it. The origins of the stresses are:

- External loads: train braking and starting and unbalanced friction on the PTFE bearing pads in the rest of the bridge
- Internal loads: primarily the axial loads induced by deck prestressing and the differential shrinkage between the two members, as the deck concrete was cast in a second stage, much later than the arch.

The total shear force passing across the joint at ULS was estimated to be 60,000 kN on each side of the key. Moreover, bending moments and shear stresses are transferred from the deck to the arch in this area, where the deck node converges with the arch.

These loads were analysed with a simplified finite element model, from which a strut and tie model was deduced. The latter was built to resemble an elastic model as closely as possible that could be used to design effective reinforcement both for the uncracked, in-service structure as well as for ULS conditions.

The shear connection between the deck webs and the arch webs, which is essential to this mechanism, required intentional roughening of the top side of the arch and the placement of continuing reinforcing steel to join the two members. Mechanical couplers were used to ensure continuity in the latter.

4. DETAILED DESIGN OF THE CONSTRUCTION PROCESS

The construction system envisaged in the original design was construction in cantilever supplemented with provisional stay cables. Whilst that approach was maintained in the revised design, it had to be adjusted to the construction facilities that were to be actually used and to slight changes in criteria affecting a number of members. The most significant re-designing is discussed below.

4.1. Segment size

Due to the complex shapes envisaged for the arch, the travellers to be used could not be clearly defined during the design phase. With the typical segment length of approximately 3.30 m envisaged, each cantilever would comprise 52 segments.

A smaller number of segments was ultimately stipulated in the detailed design to reduce the number of traveller movements, enhancing construction procedures and safety. The traveller typology accorded with the supplier called for segments measuring approximately 6.40 m long, which lowered the number of segments per cantilever to 32, for a total of 65, counting the key segment. That increase in segment size raised the traveller weight from 1,350 kN to 2,400 kN.

With these two changes, construction as a whole had to be re-engineered and the ancillary stay cables re-distributed. The number of stay families was raised from 23 to 26, although the total quantity of cables remained as provided in the design.

4.2. Analysis of the arch during construction

In the new design arch performance was calculated at both SLS and ULS throughout construction, taking both permanent and live loads into consideration.

The loads primarily considered were:

- self-weight
- wind
- deviation of stress on stay cables
- thermal variation in the arch
- differential thermal variations in the stay cables.

The SLS calculations consisted in verifying that the tensile stress on the arch did not exceed the mean tensile strength (5.2 MPa) under frequent combination. With that approach, the arch could be ensured a good stress state and the cracking that shortens a structure's service life could be averted.

4.3. Precambers analysis and geometry control

Inasmuch as geometry is essential to the structural scheme of an arch, it must be monitored during construction to ensure bridge functionality.

The detailed design stipulated the precambers to be used to erect each segment so that upon completion of the arch and 10,000 days thereafter, it would be aligned to its theoretical position.

Precamber was defined to include the correction of the elevation attained by positioning the traveller and the movement induced by tensioning the stay cables prior to casting each segment. The precamber calculated for the last segment was over 600 mm. That value declined during deck construction over the arch, from 300 mm at the outset to naught upon completion.

The theoretical calculations were performed using concrete parameters deduced from stiffness and creep tests conducted on samples of the concrete that was to be used.

Segment construction typically consisted in the following steps.

1) The reinforcing steel was set in place and the traveller forms were sealed.
2) Anchorage blocks were built to fix the stay cable to the preceding segment.
3) The stay cables were tensioned.
4) The segment was poured.
5) The traveller was advanced and re-positioned.

Two types of geometric monitoring were used to control the fluctuations in arch movements.

1) Traveller monitoring: prisms were installed on the traveller to establish the proper initial elevation and verify its position after tensioning and after pouring the concrete. With these prisms were possible verify the actual behaviour of the arch and cable-stay system.
2) Completed sections of arch: targets were set on all the inter-segment joints to verify the position and evolution of the whole arch after each segment was cast.

4.4. Cable-stay system

The stay cables consisted of bundles of 150 mm^2, 1860 MPa waxed steel strands sheathed individually in high density polyethylene. As they were provisional, they were not wrapped in an outer sheath, although they were grouped with steel bands to prevent the strands from vibrating individually.

The design called for cables comprising from 20 to 53 strands and bearing capacities of 5,700 kN to 15,000 kN (Fig. 5).

Fig. 5. *Cable-stay system*

The arch had two types of stay cables:

- Forward stays: anchored at the top to piers 6 and 15 or to the temporary tower and at the bottom to a hinged system of anchor frames attached to anchorage blocks in the arch. This system is also adjustable by means of prestressing bars.
- Back-stays: anchored at the top to piers 6 and 15 or to the temporary tower and at the bottom to a same system of anchor frames attached to anchorage blocks in the back-stay footings.

The stay cables were not assembled strand by strand but cut and pre-assembled. They were then hoisted with cranes to the anchorage points and tensioned by tensioning the prestressing bars that adjust the system of anchorage frames.

Geometrical corrections were made by re-tensioning the stay cables at two construction stages.

- Intermediate: cable 17 was re-tensioned while segment 23 was being cast.
- End: the last two cables, 25 and 26, were re-tensioned to adjust key segment geometry.

4.5. Temporary towers

The provisional towers were also re-designed during the detailed design stage.

The steel towers were made of two vertical profiles, each one of which was fitted with the anchorages for a family of stay cables. The towers, one on each bank, rested on the top of the deck over piers 6 and 15, transferring their entire load to the transversal diaphragm of the deck and bearings for these piers. The total axial load on each steel tower came to approximately 150,000 kN.

The I-profiles were fitted with strong transverse braces. The fixed head anchorages were secured with a pin that accommodated vertical movement. Each tower weighed approximately 4,500 kN.

Although the towers were temporary, they were essential to bridge functionality during construction. Of particular note in their design was the plate thickness, with values of up to 180 mm, and the high quality of the steel, up to 690 MPa. Their most distinctive feature, however, was their connection to the deck which was hinged rather than restrained, with one pin under each profile.

A hinged solution was chosen for a number of reasons.

- The provisional pylons comprised a pier and a steel tower, with the deck positioned at the connection between the two. The result was not a conventional stiff pylon, but rather a pylon with a discontinuity at the centre subject to longitudinal deck movements.
- With such a configuration, all the components interacted intensely and even slight changes in construction procedures could induce substantial changes in the stress on the tower.
- While more common, restrained connections would not stiffen the cable-stayed system.

For the foregoing, the structural functionality of the temporary towers were separated from the rest of the system. The advantages of this approach include the following.

- The towers behaved independently of the rest of the system.
- As doubly hinged members, the towers worked almost separately from the rest of the system and irrespective of the number of stay cables, i.e., of the stage of construction.
- With their hinged connections, the towers could be rotated vertically into place. Consequently, they could be pre-assembled horizontally on the deck rather than vertically piece-by-piece, rendering pre-assembly safer and faster. They also required less bolting, for they could be divided into fewer, heavier sections.

The south pylon was hoisted into position over pier 15 in September 2014 and the north pylon over pier 6 in December 2014 (Fig. 6). These operations took approximately 20 hours each. Once in position, the towers were provisionally restrained until the first pair of stay cables was installed, stabilising the system, when the provisional restraints were removed.

Fig. 6. *Hoisting the south tower into position.*

4.6. Arch closure

This was one of the most complex operations, for it entailed a drastic change in the structural configuration of the bridge, from two highly flexible 190 m long cantilevers supported by a cable-stay system to an arch with very different structural behaviour.

For reasons of construction, closure was performed in August 2015, one of the warmest and sunniest times of the year. The arch was consequently closed at 34°C, rather than at the reference temperature (16ºC) for which the camber schedule was designed. Since the cooling of the structure generates greater downward movement when the arch is closed than when it is open, closing the arch at a much higher temperature than reference temperature could have induced an incorrigible error in the arch elevation of around 100 mm.

The design decision adopted was, then, to assume a reference temperature of 16°C and install the last stay cables with a slight increment of capacity to be able to adjust the crown geometry if necessary. Re-tensioning these cables would not greatly alter the arch stresses, for the cantilever would be at its longest and the structure consequently very flexible. That was the operation ultimately performed before closure: the two families of stay cables were re-tensioned and each cantilever was raised approximately 100 mm.

In August, thermally-induced daily movements were at their greatest. Provisional restraint had to be provided with an ancillary structure to cancel out the relative movements between the two cantilevers and cast the closing segment. These daily movements ranged as follows.

- Daily stay cable heating was at its most intense. Thermal daily variations of up to 24°C were recorded in the cables, generating downward movements of up to 120 mm.
- Daily transverse solar radiation on the cantilever was also at its most intense, inducing horizontal movements of around 60 mm between 9:00 AM and 11:00 AM.

A steel structure consisting in four longitudinal profiles positioned in the key segment near each corner of the cross section of the arch was designed to reduce these relative movements. It was received with high-strength, quick-setting mortar early in the morning before the effects of solar radiation appeared. After the mortar hardened, the connection was prestressed.

This structure was set in place on 4 August 2015 and the closing segment was cast on 6 August 2015. The operation was successful and the relative error between the two cantilevers at the key before they were joined and the absolute error after that operation were both under 10 mm.

5. CONCLUSIONS

Erecting an arch of this scale for a bridge in which the structural scheme during construction differs widely from its in-service functionality is a complex endeavour.

The extra detailing and designing required to address this complexity were performed by FCC's Engineering Department, which also provided worksite support. The authors of the design verified all the work conducted in this stage of the project, ensuring full cooperation among all the agents involved.