VIADUCT OVER RIVER ALMONTE – CONSTRUCTION PROJECT ANALYSIS

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SUMMARY

The High Speed Railway (HSR) link Madrid - Extremadura crosses over River Almonte with a great arch viaduct of high - performance concrete. The main span of this structure is 384 meters.

This paper explains the exceptional techniques and structural analysis outside the usual engineering work that have been developed to reach its design and construction. These studies include the selection of the antifunicular arch axis taking into account construction process and train loads, geometric and material nonlinear analysis, dynamic analysis and aerolastic behaviour.

Keywords: *Arch bridge, high speed railway, high performance concrete, cantilever, instrumentation.*

1. INTRODUCTION

The Viaduct over River Almonte at the Alcántara Reservoir is an arch bridge with a main span of 384 m and a total length of 996 m. It will become, once completed, the longest span in a high-speed railway and the third longest arch in concrete in the world. Its design and construction includes many special features and demand complex analysis methods that are unusual, but in this case become crucial.

2. SPECIAL FEATURES OF THIS STRUCTURE

The specifications on a bridge for high speed rail are greater than those of a road bridge. For example, bigger dynamic effects by passing convoys, significant horizontal loads or fatigue. All these facts cannot be disregarded.

All these specifications must be combined with strict functional considerations, in order to obtain a service level of the structure that shall not be limited at any time: small deflections and accelerations and a length between expansion joints limited for technological reasons. Given these characteristics and limitations, the spans above 100 m have been unusual in HSR bridges, but this structure falls within an exceptional span for the topographic features of the site.

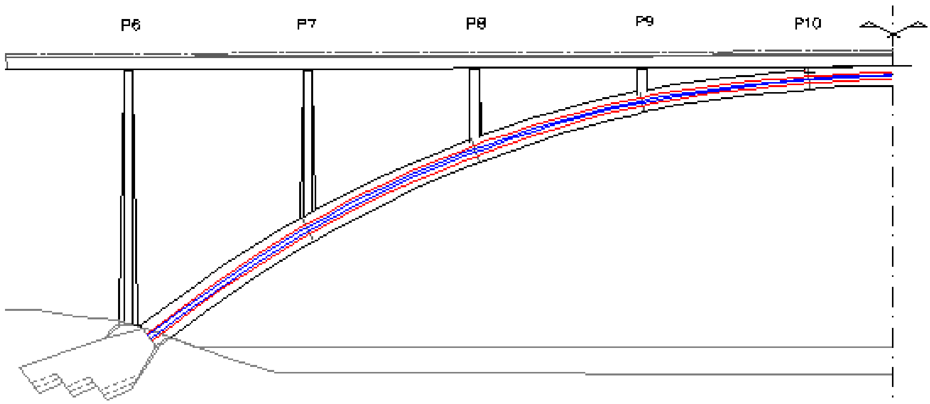

Fig. 1. Pressure lines (blue) in central area (red) of the cross section is the key of the design.

Despite the uniqueness of the main span, this bridge is formed with a typical railway box deck with maximum spans of 45 m and 3.10 m depth along its 996 m. These spans are constructed using a movable formwork system on piers and deck, so it is the arch element and its zone of structural influence, where it has been necessary to implement more elaborated construction methods. Specifically, the arch is constructed by a cantilever method with temporary stay cables from two temporary steel towers.

As a consequence of the service conditions, the nature of the structure itself and the construction method, the structural analysis includes:

- Detailed analysis of construction stages.
- Finding of an anti-funicular arch axis.
- Analysis of service limits state and nonlinear forces.
- Detailed dynamic analysis.
- Other special studies: confirmation of the aeroelastic behaviour of the bridge in its environment through wind tunnel tests.

In the following pages, the most significant points of some of these calculations are shown. They focus on aspects related to the construction project: shape of the arch, analysis of nonlinear Ultimate and Service Limit States and deformability of the deck.

3. SELECTION OF ARCH AXIS

The geometry choice of the axis line in arch bridges is a key point in the design. When axis line is correct, the structure works subject only to compressive stresses in any of its points. This is an anti-funicular axis line according the existing loads. This approach to the problem assures that the material of the arch is free of tractions, not cracking the concrete. It ensures durability with minimal maintenance. The total steel reinforcing bars in the arch is only engaged for response to Ultimate Limit State (ULS).

In the case of HSR arches, the finding of an anti-funicular axis line imposes a new reality in the methodology: the railroad load results in significant deformation and therefore considerable bending forces.

For this reason the decision whether cracking of arch is accepted becomes an important part in the design. This cracking, despite being almost inevitable for Ultimate Limit State (ULS) with factored loads and resistances, in normal use cracks degrade the stiffness and imply greater arch movements. To limit movements for a suitable railway, using a greater arch's inertia would be necessary, i.e. it would be necessary to increase the amount of material compared as if it could avoid cracking. This is why, in arch railway bridges, a criterion against cracking is recommended as a new Service Limit State (SLS).

Once the influence of cracking is defined as an input, the design will be directed by the proper selection of the depth of the arch or, in other words, its flexural rigidity: the depth that controls curvatures in the plane of the arch.

The study of this flexural rigidity is related to the identification of possible load anti-funicular lines to which the structure is subjected given its axis line. The procedure is simple: for each configuration of specific loads it is possible to find one axis line and define stiffness, i.e. an arch depth, so that the pressure line is optimally contained in the central area of each arch cross-section.

The complexity is that the main loads are trains. They can be from several typologies and may or may not be present in varying degrees by either or both tracks on the deck. Therefore, treatment for this wide pattern-type loading should be identical to an envelope.

Fig. 1 shows an elevation of a half arch viaduct where it can be seen the dynamic pressure line of a particular type of train within the lines that ensure the complete absence of cracking in service conditions.

4. BEHAVIOUR OF THE STRUCTURE AGAINST WIND LOADS

In structures the size of this bridge, the behaviour against wind loads is a key factor in the design. The depth of the arch should be as small as possible with the purpose of having a limited area exposed to wind forces. In addition, the shape of the cross-section should be optimized based on the knowledge of wind flow properties with an effective profiling to establish a compact arch opaque to wind.

In order to optimize the behaviour of the structure under wind loads, the project phase of the Viaduct over River Almonte has considered, as a key design parameter, to reduce as much as possible the edge of the cross section of the arch, providing section chamfers that would reduce its drag coefficient.

This section should cause minimal disturbance to the airflow, reducing and optimizing their depth and also possess enough inertia to deal with concomitant bending moments from railway loads, as explained in the previous section of this paper.

The outcome of the above analysis defines in the bridge a hollow section, thus avoiding the unnecessary weight of a solid section that meets the requirements of compactness and therefore less disruption of air flow. The section is elongated and four pronounced chamfered edging profile for horizontal wind direction are added.

Wind tunnel tests and analysis on a reduced model validated the design.

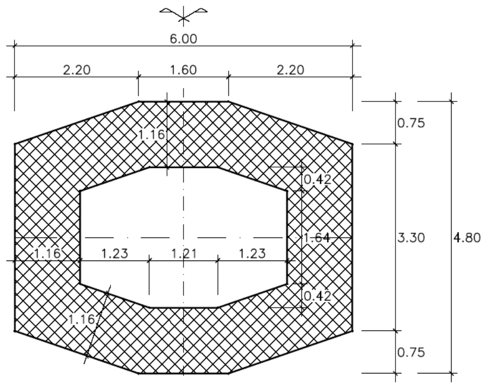

Fig. 2. *Typical cross section of the arch. Relationship between depth and chamfers.*

5. NONLINEAR ANALYSIS OF SLS AND ULS

Nonlinear calculation takes into account the geometric nonlinearity (caused by the change of geometry due to deformation), and the nonlinearity of the material properties, including the effects of cracking in concrete and the nonlinearity of stress-strain curves in both arch and deck.

The verification methodology for the Ultimate Limit State of instability is guided by Eurocode 2, which is the reference document for its level of development, in terms of nonlinear calculations. The arch's behaviour analysis has considered all the next featured effects:

- Concrete cracking.
- Shrinkage and creep of concrete.
- Nonlinearity of the constitutive equations ($\sigma - \varepsilon$) of steel and concrete.
- Effects of the construction stages and assembly sequences in the final safety factor of the finished structure.

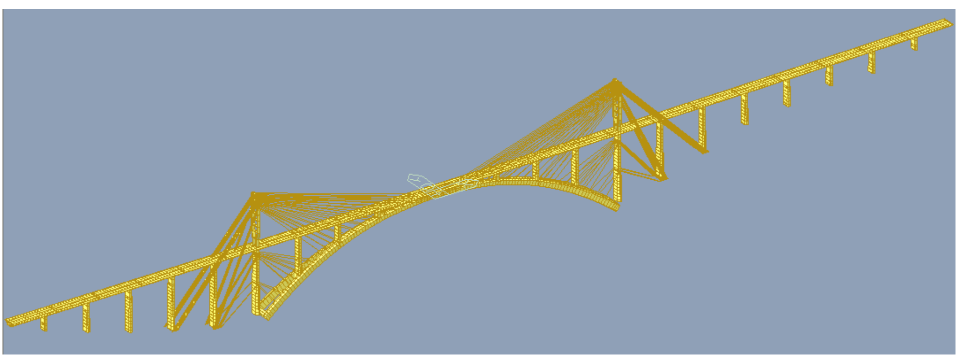

Fig. 3. *Overview of the calculation model.*

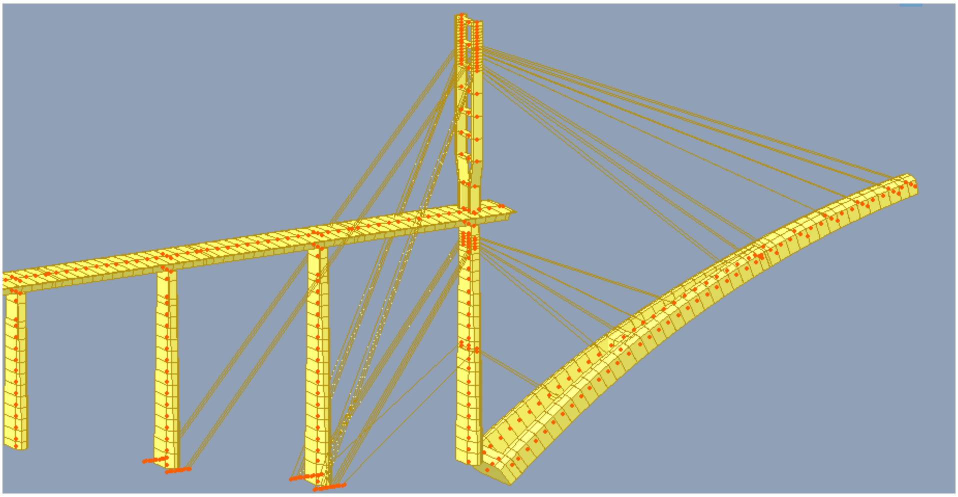

Fig. 4. Close-up view of an intermediate stage of the construction process.

5.1. Analysis steps

The safety of the structure against Ultimate Limit State is the core of the verifications. This has been developed for a nonlinear model with an incremental loading, in which the balance and strain compatibility for every stage of the process is integrated.

As reflected in the EN1992-1-1 and the Model Code 90, the analysis is based on 2nd order model analysis, with average strength and stiffness. Afterwards, the capacity of sections with factored materials is checked.

Calculation stages consisted of:

1) Nonlinear analysis of all construction stages until the final permanent loads, including geometric imperfections.
2) Nonlinear creep and shrinkage analysis, with geometric and material redistributions stresses of the previous step.
3) Nonlinear geometric and material analysis of thermal loads.
4) Global nonlinear analysis, step by step, with geometry and stresses obtained at the end of the previous section.

   Permanent loads are included from their characteristic values to its factored values, and live loads are introduced by incremental steps, from values of paragraph 3, so that the values of ULS are reached in the same step.

   At each step an adaptation of the stiffness of each of the model sections is arranged according on the stress-strain curves proposed in Eurocode 2 for both steel and concrete, which consider cracking of concrete in sections where it occurs.

5) Finally the sections are checked with total factored loads, adopting conventional models to calculate sections' security with factored material strengths.

A further check of the structure was carried out based on the new nonlinear security analysis formulations for concrete structures held in Eurocode 2, listed in part 2, and the Model Code 2010.

This formulation uses average characteristics for steel and factored average value for concrete strength of 0.84·fck, in order to "homogenize" partial coefficient of steel security and obtain a global safety factor of the structure, increasing the load until the structure reaches its capacity or until a global failure of the structure exists.

With EN 1992-2 security formulation, the load is increased above the Ultimate Limit State load, up to the collapse of the structure, either by reaching the ultimate strength of a single section or global failure due to instability of the structure (buckling).

5.2. Load combinations

The nonlinearity in the behaviour of the bridge eliminates the possibility of using the classical superposition of loads. It is necessary establishing the parameters to calculi and analyse their combinations.

In this case, based on linear forces, a finite wide number of critical sections in the arch have been defined and the loads combinations for both maximum and the minimum bending forces are determined.

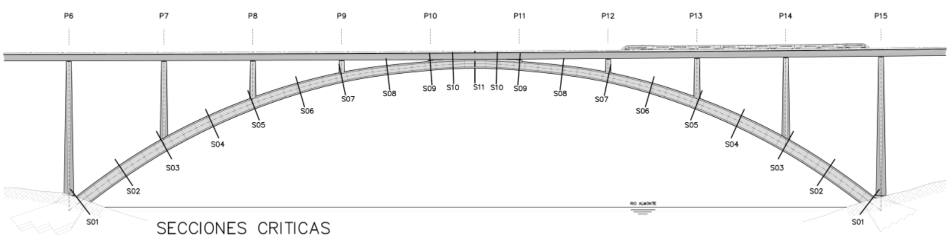

Fig. 5. *Study of critical sections.*

The combinations taken into account have been developed with the usual loads included at railway load instructions:

- Dead loads.
- Creep and shrinkage load.
- Forces from the deck prestressing.
- Friction on bearings.
- Temperature and thermal gradient in piers, deck and arch.

- Live loads of railway tracks (trains UIC -71, loop, braking and acceleration).
- Live loads of lateral sidewalks.
- Longitudinal and transversal wind.

5.3. Geometrical non-linearity. Initial imperfection

The nonlinear analysis should be performed taking into account the adverse effects of possible deviations in the geometry of the structure. For concrete structures, such imperfections are included in section 5.2 of Eurocode 2. In the case of arch bridges, an idealization of the initial geometric imperfection by a sinusoidal function according to first buckling mode with an amplitude equivalent to a sectional elastic eccentricity of value e_0 is recommended.

The calculation method introduces the equivalent geometric imperfection in the arch axis focusing a staged computation starting with the pre-deformed geometry of the arch.

The initial imperfection to be considered is therefore different for each load case, always considering a pre-deformed homothetic to the first buckling mode for each of the load cases. Fig. 6 shows the buckling mode considered for one of the analysed cases.

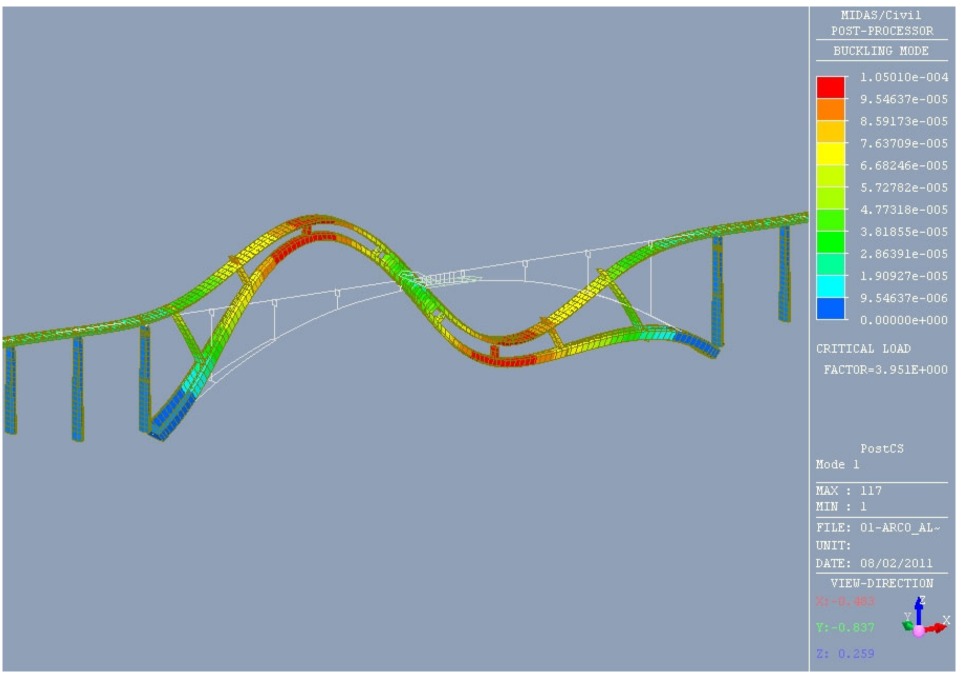

Fig. 6. Buckling mode corresponding to the hypothesis called S08 – Min.

5.4. Modelling of nonlinearity of materials for Ultimate Limit State checking

In this analysis, dead loads are applied from their characteristic values to its factored values, and live loads are simultaneously introduced in short increments so that ELU values are achieved in the same step.

It is developed, at each incremental stage an adaptation of the stiffness for each section of the model according to the stress-strain relations proposed in Eurocode 2 for both steel and concrete, where concrete cracking is considered.

It is therefore an incremental - iterative process. At each stage, the stiffness values are corrected according to the stress-strain diagram of the EC-2. The procedure is then to recalculate forces and check that stiffening corresponds to pressure levels.

This process is repeated until the variation in stiffness between the value adopted for the calculation of forces and the value that obtained from the moment-curvature diagram is negligible.

This incremental procedure needs the tangent stiffness of moment-curvature diagram at the point corresponding to previous incremental step of load as an input.

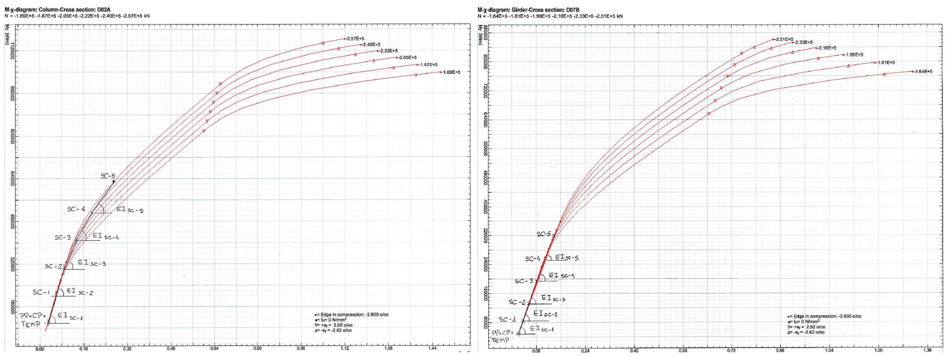

Fig. 7. Moment-Curvature Diagrams for Arch Segment 2 and Segment 7.

Fig. 7 shows the moment-curvature diagrams for different levels of axial force in arch Segment 2 (cracked in the load increment SC-3) and Segment 7 (non-cracked throughout the process for this hypothesis worksheet) in each of the load increments. Bending forces and stiffness considered in the calculation in the last iteration for the hypothesis are indicated.

Both computation model and moment-curvature diagrams have employed a concrete stress-strain diagram adapted to the nonlinear calculation. For the sectional calculation, the parabolic-rectangle diagram was used.

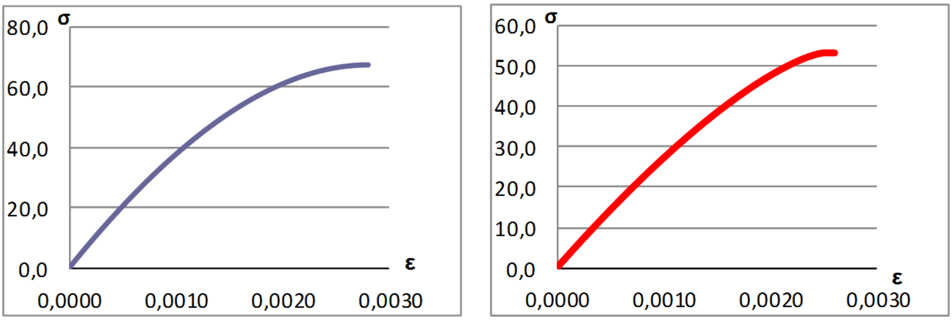

Fig. 8. *σ-ε diagrams of concrete HA-80 for the sectional nonlinear calculation.*

6. VERTICAL DEFORMATION OF DECK

A key point in the final functionality of the structure is the fulfilment of minimum deformational limitations for the track in order to allow a safe and comfortable pass of the train.

This type of viaduct needs a special validation because of the span's length in relation to convoy loads. The verification of the vertical curvature radius of the track has to be compatible with the HSR line's design speed. This requires a dynamic analysis of all possible trains and the correction of the vertical curve parameter in order to allow the maximum slope of the track with the maximum speed for each train.

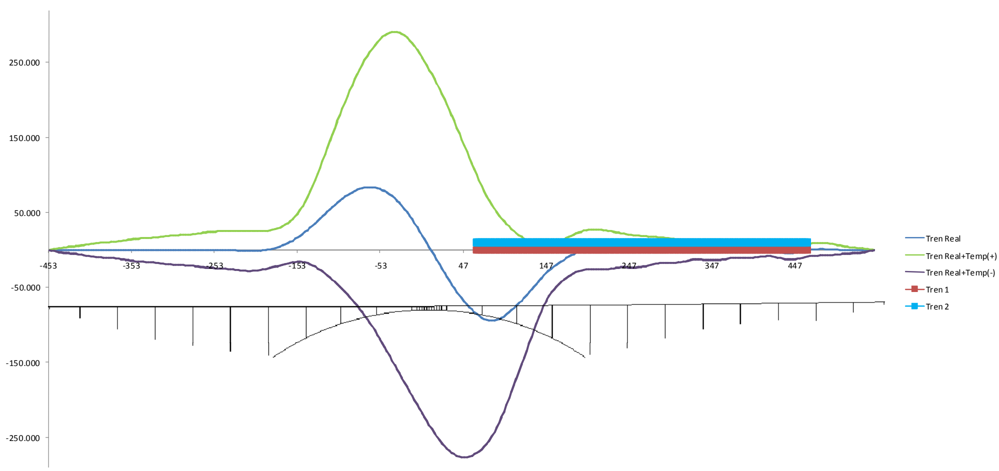

Fig. 9. *Deformed deck for a specific case of two HSML trains at 300 km/h.*

These trains' running over the bridge has been studied with all possible hypotheses including thermal effects:

- One train running in one direction.
- Two trains running in parallel to the same direction and speed.
- Two trains running in opposite directions, crossing in most unfavourable position.

7. CONCLUSIONS

This paper reports the analyse considerations that have allowed overcoming the span of Almonte Viaduct: new high-performance materials, modern analysing tools, aeroelastic modelling and regulatory treatment of semi-probabilistic safety formulations.

Making use of these instruments, it has been possible to refine the criteria for the arch dimensioning in order to take into account the influence of the dynamic amplification of HSR live loads. It can be concluded that nonlinear material and geometry analysis is the only way that allows to accurately assessing safety levels for this type of structures with high slenderness, great span and an important live load level.