DESIGN AND CONSTRUCTION METHOD OF THE NEW COIMBRA FOOTBRIDGE

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Summary

The paper deals with the design, detail and construction method of an innovative and slender footbridge currently under construction at the city of Coimbra, in Portugal.

Keywords: Footbridge; steel structure; arch bridge; construction

1. Introduction

The development of the Mondego Green Park along both banks of the river at the city of Coimbra, Portugal, set the opportunity to construct an innovative pedestrian and cycling bridge [1] that is intended to be a landmark for the city and to contribute to the quality of a new leisure area.

The bridge is 274.5 m long with two 4 m wide ramps coming up from each bank. These straight ramps are inclined 4% and are tangent to a transition parabolic arc at the centre of the river. In structural terms, the bridge is essentially composed by one central parabolic arch and two lateral parabolic half arches defining, together with the deck of those ramps, two continuous triangular cells (Figure 1).

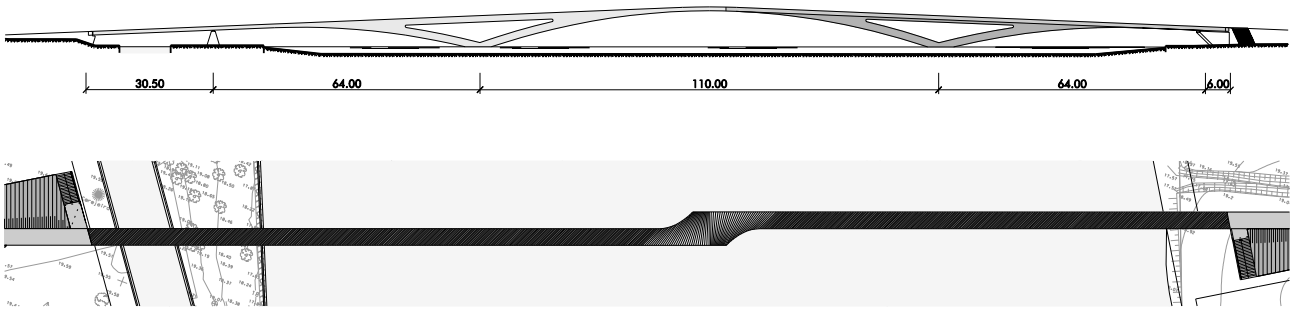


Fig. 1 - Elevation and plan view of the footbridge

2. Bridge geometry

For best structural performance, geometry of arches should match the anti-funicular of permanent loads of the bridge. Simple compression would be generated in the arches under that loading, with bending taking place for other load distributions, especially asymmetrical loads.

Parametric studies of this bridge advise a maximum value of 12 should be considered for the “span over rise” ratio of the central arch, because bending increases rapidly with high shallowness of arches. Water sport requirements led then to a central parabolic arch spanning 110 m and rising 9.362 m. Since the river is 215 m wide, 105 m are left to be covered by

lateral spans. These spans contain half arches springing also from the foundations of the central arch, where horizontal components of their reactions are partially balanced.

The bridge deck crosses the river supported by the central arch and the lateral half arches. Lateral spans are 64 m long, with a further southern span of 30.5 m over a rowing canal. Geometrically, deck and arches meet tangentially. Arches are clamped in their foundations, but supports of the deck provided by the column on the left bank and by abutments allow free longitudinal movement of the deck. In order to control vertical deformations in the lateral northern span, this span is also clamped in the abutment by means of a 6 m extension of the deck into that abutment.

Arch and most of the deck of each half-bridge are positioned anti-symmetrically with respect to the longitudinal axis of the bridge (Figure 2). A complex torsion response of the structure develops but its lateral stiffness increases when compared to a symmetric cross-section system.

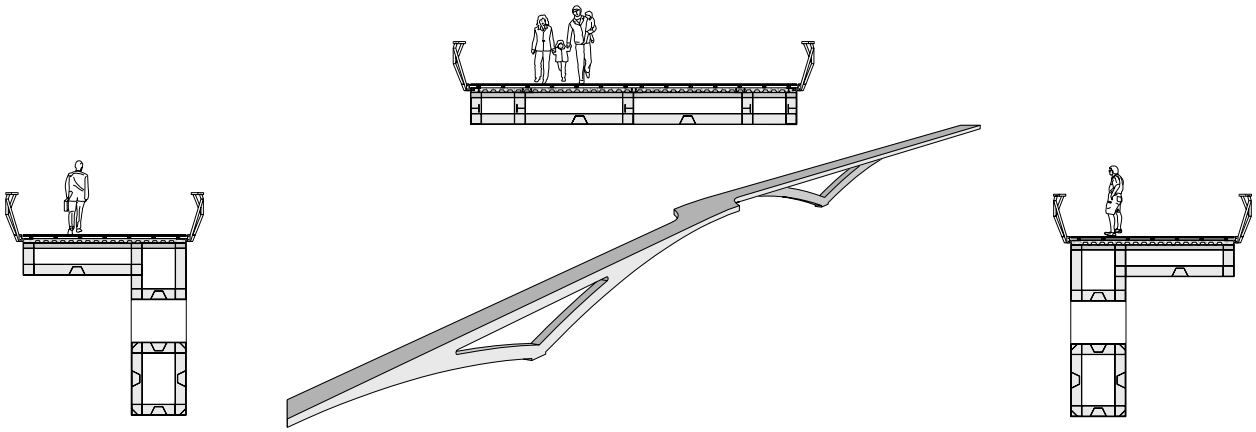


Fig. 2 - Perspective of the footbridge. Cross-sections in the proximity of the arch springs and at mid-span

Cross-section of the arch is a steel box 1.35 m wide and 1.8 m high. Cross-section of the deck is a composite steel-concrete box 4.00 m wide with a concrete slab as upper flange. This cross-section is symmetric and only 0.90 m deep in the middle of the central span and near the abutments, becoming asymmetric and 1.5 m high in spans between central and lateral arches (Figure 3).

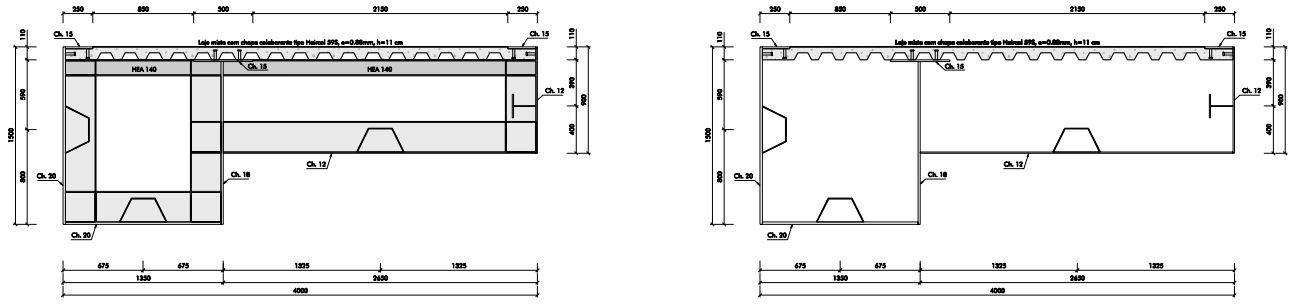


Fig. 3 – Cross-section of the bridge between central and lateral arches: with and without transversal stiffeners

3. Structural response of the bridge

Structural behaviour of the bridge in the longitudinal direction is very much dependent upon the relationship between flexibility of the major triangular cells and flexibility of the foundations of the arches with respect to horizontal displacements.

Due to low resistant capacity of the underground soil layers, the bridge foundations are indirect with 30 m deep piles (Figure 4). For minimum cost, piles are vertical. Therefore, stiffness of foundations is high for vertical forces but low for horizontal forces. This low stiffness means the arch effect is “relaxed” and the global “arch+beam” behaviour generates less compression forces and more bending moments.

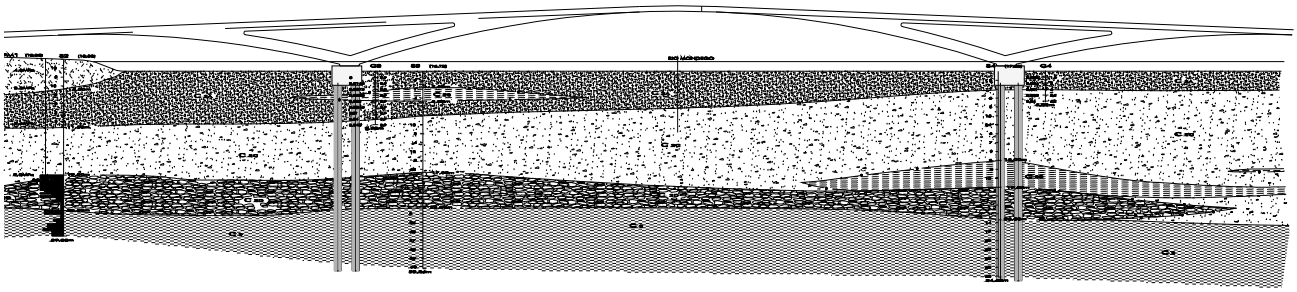


Fig. 4 – Longitudinal cross-section of the bridge and of the underground soil layers

As expected, decrease of the "arc effect" mobilizes the two triangular cells, converting partially the behaviour of the bridge into a continuous beam with global negative bending moment over central supports taken by compression in the arches (struts) and tension in the deck (ties), as shown in Figure 5.

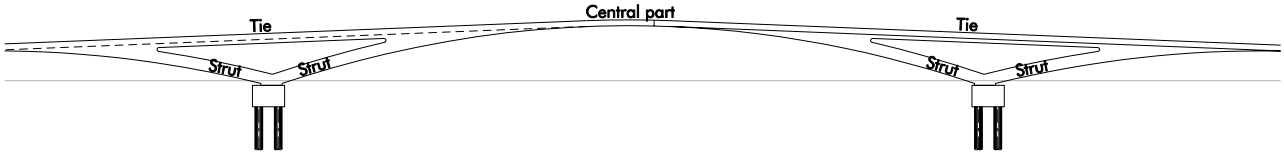


Fig. 5 – Structural elements of the bridge

Equilibrium of horizontal forces is not achieved in each pile cap because force transmitted by the central arch is always higher than the force transmitted by lateral arches. This disequilibrium is resisted by soil reactions along pile lengths. Therefore, the behaviour of this bridge is dependent upon the real properties of the soil layers in which it is founded. A rigid soil makes the structure more rigid, more hiperstatic, with higher compressive forces on the arch and lower bending moments in the central part of the deck. On the contrary, a more flexible soil implies lower compressive forces on the arch and higher bending moments in the central zone of the bridge.

A detailed analysis of the soil-structure interaction effects was fundamental for the study of the structural behaviour of this bridge. Sensibility parametric studies of stresses and deformations in the bridge were performed for several dimensions of piles and properties of the underground soil layers.

Horizontal load tests were performed during the execution of foundations to obtain the real stiffness of the assemblage pile/foundation soil.

4. Numerical models of the bridge

The analysis of the structural response of the bridge was performed by two distinct numerical models. The first is a bar finite element model (Figure 6) and the second is a shell finite element model (Figure 8). These two models were tested and compared, providing similar results.

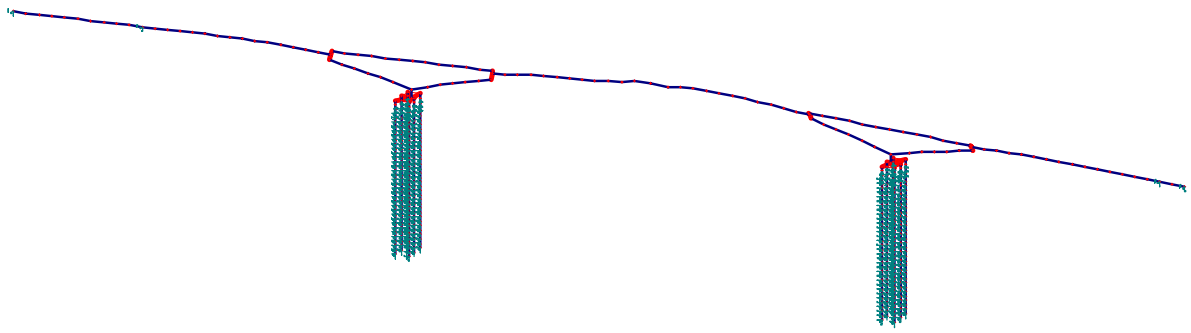


Fig. 6 – Bar finite element model (with piles)

The bar finite element model includes the piles and the soil interaction of the underground soil layers is modelled by nonlinear supports. As shown in Figure 7, the latter are characterized by elastoplastic behaviour laws with distinct stiffness and limited resistance for each soil layer and depth.

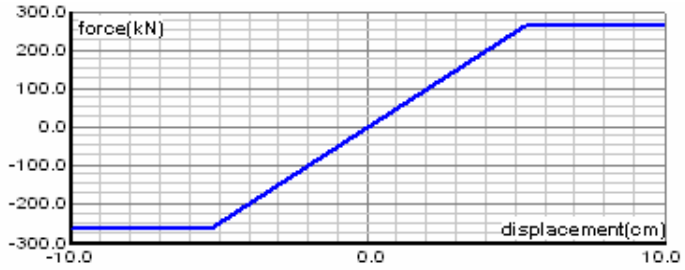
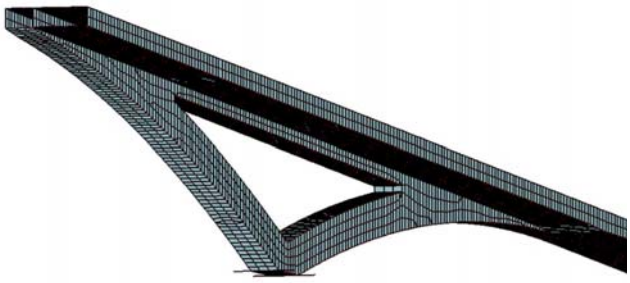


Fig. 7 – Elastoplastic behaviour law of a "soil support"

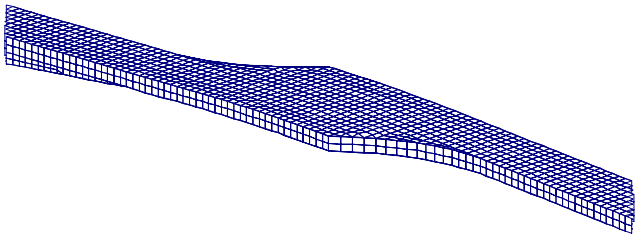
Both deck and arches are modelled by bar finite elements with the geometrical and mechanical properties referred to the centroid of the cross sections. Therefore, shear centres of the cross-sections are neither centroids nor geometrical centres of the bridge pavement. In order to take this into account, self weight of steel plates was applied at the centroid of each cross section as distributed and concentrated vertical loads complemented with distributed moments resulting from the eccentricity between the shear centre and the centroid of that cross section. Similarly, self weight of the composite slab, other dead loads and live loads were applied at the centroid of the section by distributed and concentrated loads complemented with distributed moments to account for distance between shear centre and geometrical centre of the pavement.

This model was used to study the interactive effects between soil and structure, which were known to be fundamental for the structural response of the bridge. Results from this model led to "equivalent elastic constants" characterizing the behaviour of piles and soil layers to be used both in the dynamic analyses and in the shell finite element model analyses.

Although the bar finite element model is considerably easier to use than the shell finite element model and is a good representation of the behaviour of the bridge, three-dimensional effects of the structure are better understood using a 3D model. The design process also becomes easier by the direct checking of maximum stresses in each steel finite element.



a) arch support zone



b) central zone

Fig. 8 – Shell finite element model (details)

The exact thickness of every steel plate was introduced in the shell finite element and a reduced thickness of the composite slab was considered for a long term analysis in order to simulate the effects of creep in concrete. The transversal stiffeners are located every 2.50 m and were modelled as bar elements. Generated stresses were calculated and the checking of the thicknesses of plates on each cross section of the bridge was done directly.

5. Design of foundations of central arch

Foundations of the arches are provided by groups of bored 1.2 m diameter vertical piles about 30 m long. These piles will mobilize the soil layers in order to equilibrate resultants of unbalanced horizontal components of forces arriving from the arches. These resultants are always orientated towards the river margins due to the higher forces on the central arch (Figure 9).

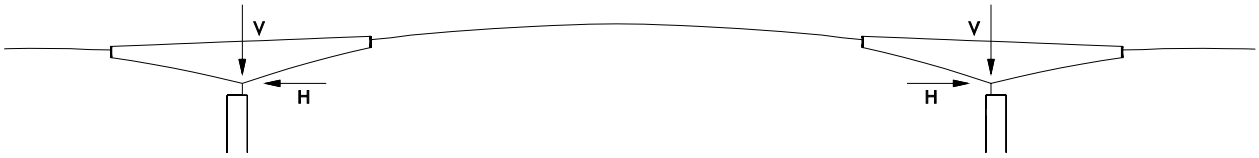


Fig. 9 – Forces acting in the pile caps

As shown in Figure 10a, eccentricity between resultant (H) of unbalanced horizontal forces on each pile cap and resultant of horizontal reactions of underground soil layers on piles generates a binary that can only be equilibrated by a binary of vertical forces (Δv) in piles. Axial forces increase on the "exterior" piles and decrease on the "interior" piles.

If piles in each pile cap were centred with the corresponding arch spring, vertical resultant (V) would be divided equally into the two alignments of piles, but equilibrating binary forces Δv would result in different total forces (V1 and V2) in each alignment of piles. However, if pile caps are built to be eccentric in relation to the corresponding arch spring, total forces V1 and V2 can be made equal for permanent loads (Figure 10b).

a) equilibrating binary of forces Δv *b) eccentric pile cap*

Fig. 10 – Equilibrating binaries and eccentric pile cap

The linear elastic analysis of the bridge with the bar finite element model has shown that perfect continuity of deformations between piles and caps generates high bending moments near the top of piles. However, this perfect continuity is not true due to the cracking of pile's concrete. Therefore, redistribution of those internal forces was introduced into the model. The analysis was repeated with lower inertia of piles near the top, for which the "bending moment - curvature" diagram of the pre-designed cross-section was calculated first. This was an iterative process because generated forces depend on the strength of interacting structural elements.

Forces on piles were determined at ultimate resistant state, but complementary calculations under service loads were also performed in order to verify cracking safety and ensure durability of piles.

6. Design criteria

6.1 Ultimate limit state verification

For double stress states, verification of the ultimate limit states in the steel plates consists in satisfying condition (1) at every point of the structure:

$$\sigma_{Sd,ref} \leq \sigma_{Rd} \tag{1}$$

with σ_{Rd} equal to f_{yd} and $\sigma_{Sd,ref}$ equal to a conventional reference stress calculated as the following function of the acting normal and shear stress design values:

$$\sigma_{Sd,ref} = \sqrt{\sigma_{Sd,x}^2 + \sigma_{Sd,y}^2 - \sigma_{Sd,x} \times \sigma_{Sd,y} + 3 \times \tau_{Sd,xy}^2} \quad (2)$$

This definition respects the Von Mises-Hüber plasticity criteria.

6.2 Longitudinal and transversal stiffeners

To avoid local buckling of the steel plates, longitudinal stiffeners are placed along webs and flanges (Figure 11).

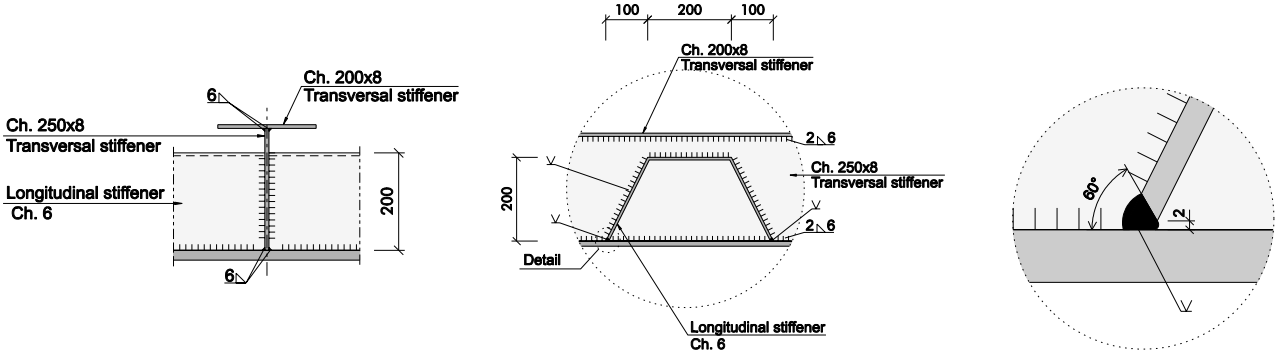


Fig. 11 – Transversal and longitudinal stiffeners (details)

Dimensions and distance between longitudinal stiffeners (the majority of them are of trapezoidal cross-section) were defined so that the entire bridge cross section becomes effective according to Eurocode 3 [2], thus allowing the elastic design of the section.

Together, plates and stiffeners withstand forces developed in the exterior steel plate and resist local buckling between crossbeams (transversal stiffeners).

Transversal stiffeners are 2.50 m apart and support the composite beams of the deck, account for the deviation forces of the cross-section parabolic flanges and limit the distortion and warping of the bridge tubular section. They also provide support for the longitudinal stiffeners, thus limiting their buckling length.

Transversal stiffeners were designed as frame systems instead of triangular systems, which are usually more efficient, because of the dimension of cross-sections. In the present case, the angle of diagonals would be too low.

In the shell finite element model, transversal stiffeners are beam elements. Gravity loads located symmetrically and asymmetrically in the transverse direction, together with the bridge anti-symmetry, generate internal forces in the beam elements that include directly the limitation of distortion and warping in cross-section. These internal forces were used in the design of the crossbeams. Minimum stiffness criteria imposed by international rules was also verified.

6.3 Global buckling effects

According to the item 5.2.4.3 of "Eurocode 3: Design of steel structures – Part 2: Steel bridges" [2], system imperfections shall be used in the global analysis when second order effects cannot be neglected, as is the case of arch bridges.

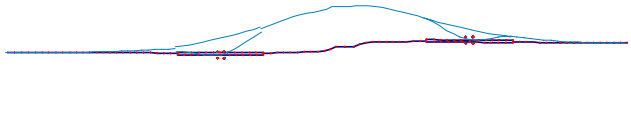
The second order analysis of compressed members is made considering the effects of practical imperfections. These imperfections include:

- Geometrical imperfections, usually L/1000, with L equal to the length of the member;
- Other imperfections, namely the welding residual stresses

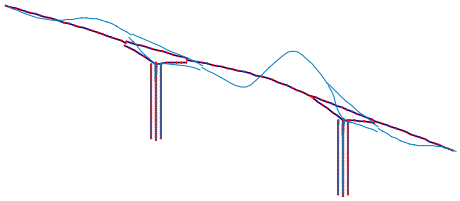
Suitable equivalent geometric imperfections may be used, with values that reflect the possible effects of all types of imperfections, unless these effects are included in the resistance formulae for member design.

The assumed shape of system imperfections can be derived from the elastic buckling mode of the structure in the plane of buckling. Using this method, both possibilities of in and out of plane buckling with symmetric and asymmetric buckling shapes should be taken into account.

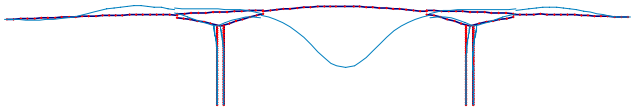
In the present bridge design, global buckling effects were calculated using a second order analysis of the beam model, beginning with a pre-deformation homothetic to the fundamental buckling modes (Figure 12) and maximum amplitude of L/300, where L is the effective buckling length of the correspondent mode shape.



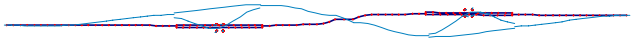
Mode 1 (lateral)



Mode 2 (vertical)



Mode 3 (vertical)



Mode 4 (lateral)

Fig. 12 – Buckling modes

"Equivalent additional stresses" due to global buckling were then estimated for each part of the structure. These values were added to the stresses obtained in the linear elastic shell finite element model. Table 1 summarises the results of the second order analysis.

Table 1 –Equivalent additional stress due to global buckling

Element	Stress
Lateral arch	40 MPa
Tie	25 MPa
Central arch	50 MPa
Deck lateral spans	20 MPa

7. Dynamic studies

Dynamic analysis performed in the beam model and in the shell finite element model show that the bridge is prone to human induced vibrations. Furthermore, a simplified time-history analysis proved that pedestrian action could lead to vibration levels that, although not associated with any unsafe situation, may produce discomfort in pedestrians.

The first lateral vibration mode has a frequency of 0.7 Hz and several global vertical modes have frequencies between 1.6 and 3 Hz (Figure 13).

Mode 1 (lateral) – f = 0.7 Hz

Mode 5 (vertical) – f = 1.64 Hz

Fig. 13 – Vibration modes

Detailed numerical studies were then performed [3] and it was decided to add up to eight tuned mass dampers to control

the relevant vibration modes.

8. Monitoring system

During construction and in service the bridge will be monitored on its static and dynamic behaviour. The monitoring system measures temperatures, stresses, rotations and accelerations in the most relevant cross-sections of the bridge. The information collected will be compared with calculated parameters in the design phase.

Thirty static sensors and six dynamic sensors will be installed with the following characteristics: 23 strain gauges in the steel structures, 6 thermometers, 1 tiltmeter and 6 uniaxial high sensitivity accelerometers.

9. Construction method

The construction method that is currently being used is the traditional propping of the entire bridge on top of provisional hydraulic in-fills built in the river (Figure 14).

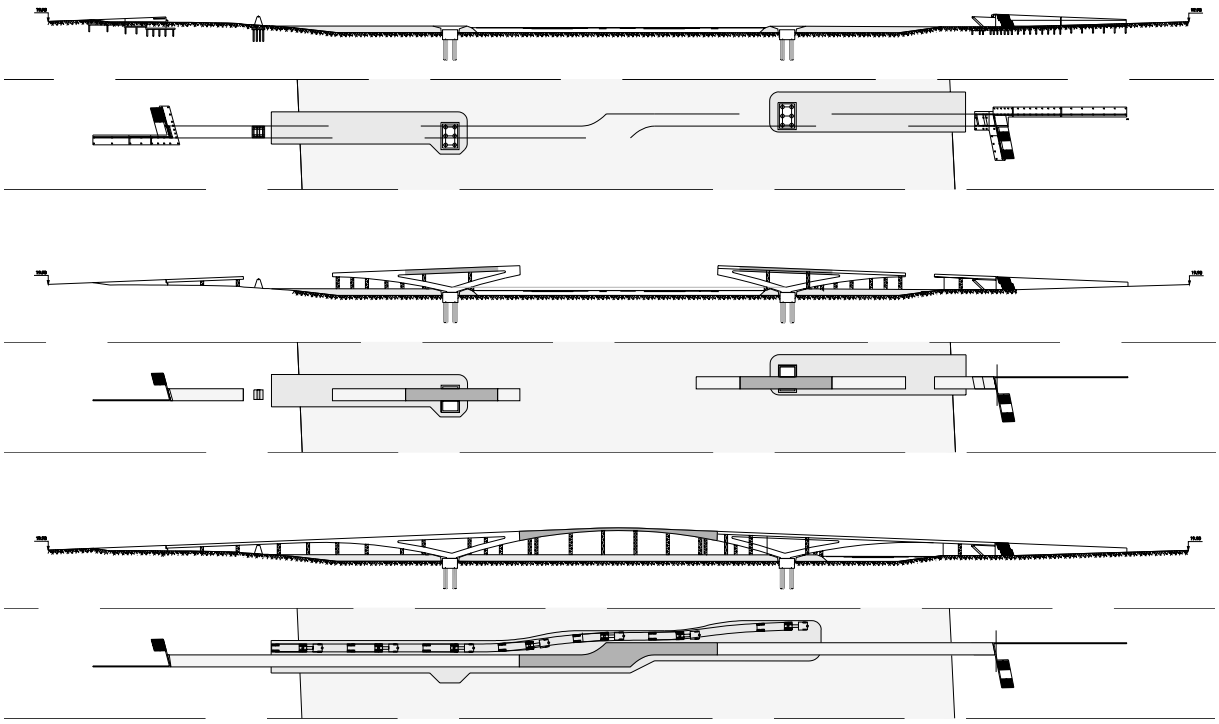


Fig. 14 – Construction sequence

Due to the asymmetry the bridge, correct location of temporary props is very important for the structure stability during construction (Figure 15).

Fig. 15 – Bridge under construction (October 2005)

Removal of the propping elements will be done in a pre-defined sequence, with the structural response of the finished bridge followed closely by the static monitoring system and by topographic measurements.

Acknowledgements

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References

[1] ADAO DA FONSECA, A. & BALMOND, C., *Conceptual design of a footbridge over river Mondego, Coimbra*, Footbridge 2005 – 2nd International Conference, 6-8 December, Venice.

[2] ENV 1993-2:1997, "Eurocode 3: Design of steel structures – Part 2: Steel bridges", CEN, 1997

[3] CAETANO, E., CUNHA, A., ADAO DA FONSECA, A., BASTOS, R. & ADAO DA FONSECA Jr, A. (2005) *Assessment and control of human induced vibrations in the new Coimbra footbridge*, Footbridge 2005 – 2nd International Conference, 6-8 December, Venice