THE INFANTE DOM HENRIQUE BRIDGE OVER THE RIVER DOURO: construction method, monitoring equipment and structural control

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Abstract:

Oporto provides the splendid setting for several world-renowned bridges over the river Douro. The last to be built consists of a shallow and extremely thin arch spanning a distance of 280 metres under a stiff box-beam deck. The deck is the stabilizing element of the slender arch. This "Maillart" type of arch bridge is a world record and was built by creating rigid triangular structural systems requiring temporary struts and diagonals to complement those bars provided by the arch and the deck. The equilibrium of the two half-bridges cantilevering over the river at a high of more than 70 metres was achieved by cable anchorages into the granite slopes, together with concrete-ground struts forming rigid triangular structural systems with the deck and temporary diagonals.

The construction of this bridge was understood to be extremely difficult and quite spectacular, but solutions for unexpected problems were non-existent. Therefore, "state-of-art" monitorization equipment was installed in the bridge and its foundation elements for *on line* follow up of the construction method and of several imposed settlements, upwards and downwards, introduced into the structure.

The bridge is affected by the maritime climate of Oporto, and thus several design decisions were taken and a variety of environmental equipment was installed in the bridge. This allows for the follow up of the behaviour of the bridge during its lifetime of at least 100 years without any major repair. The structural behaviour of the bridge is numerically controlled and supervised regularly from the design office. Therefore, a complete understanding of reality ensures timely structural interventions for the rehabilitation of the bridge, if required.

1. Introduction

The public call for tenders for the design and built of the Infante Dom Henrique Bridge, over the river Douro, connecting the cities of Oporto and Gaia and located halfway between Luiz I Bridge and Maria Pia Bridge, demanded a solution that should match the technical and aesthetic qualities of those two bridges, which are both considered great works of engineering. The responsibility in designing such a bridge was raised further by naming the bridge after Infante Dom Henrique, who is one of the most distinguished figures of the city of Oporto and Portugal and who led Europe on the adventure to meet other civilisations.

The project designers understood that these qualities would have to appear in a discrete manner, without fanfare and embellishment. A bridge that, without supports on the riverbed, without supports even on the banks of the river, would fly as if it were a bird over the noble waters of the river Douro, with great transparency and expressing itself in the purest possible way (Figure 1).

Figure 1 – Westwards lateral view of the bridge in the final construction stages

This bridge does not contain any decoration. It does not contain anything that does not comply with the functional requirements. Everything in the bridge has a purpose that is both structural and functional. For this reason it has virtue of simplicity, structural purity and geometric regularity (Figure 2).

Figura 2 – Eastwards lateral view of the completed bridge, seen from the lower deck of Luiz I Bridge

The conception and design of this bridge was developed by a team of Structural Engineers lead by the author of this document, by José Antonio Fernández Ordóñez† and by Francisco Millanes Mato. The design team included Structural Engineers Renato Bastos, Pedro Fradique Morujão and Luís Pedro Moás, of AFAssociados – Projectos de Engenharia, SA, Luis Matute Rubio, Javier Pascual Santos and Arturo Castellano Ortuño, of IDEAM, SA, and, as geotechnical specialists, José António Mateus de Brito and José Manuel Romeiro, of CENOR – Projectos de Engenharia, Lda. On the other hand, the international call for tenders for the "design and built" project was awarded to the construction consortium of contractors EDIFER – Construções, SA and NECSO – Entrecanales y Cubiertas, SA.

2. The structure

The Infante Bridge is composed of two mutually interacting fundamental elements: a very rigid prestressed reinforced concrete box beam, 4.50 m in height, supported on a very flexible reinforced concrete arch, 1.50 m thick, as shown in the elevation and cross-sections in Figure 3. The span between abutments of the arch is 280 m and the rise till the crown of the arch is 25 m, thus with a shallowness ratio greater than 11/1.

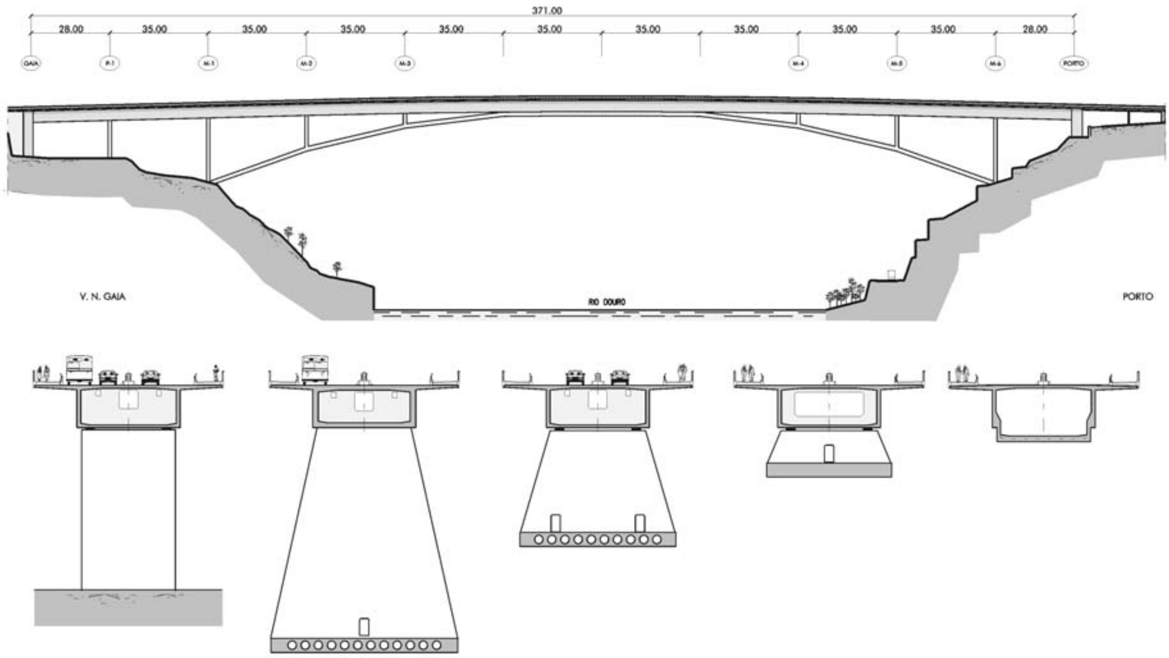


Figure 3 – Elevation and cross-sections of the bridge

In the 70 m central stretch of the bridge, the arch combines with the deck to form a box section that is 6 m in height. The lateral faces of this section bear an indentation that gives the impression of continuity to the masses corresponding to the deck and the arch.

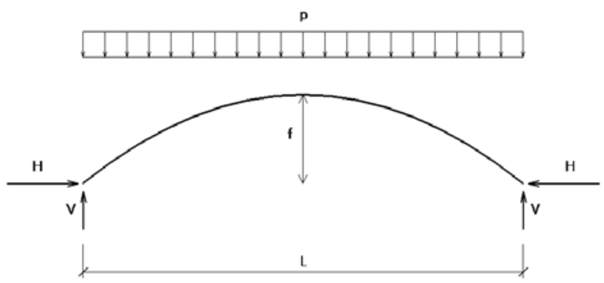
The structural behaviour of this bridge is determined by the interaction that exists between deck and arch. The great rigidity of the deck, together with the high shallowness and low rigidity of the arch, moves the behaviour of the Infante Bridge closer to that of a bridge in which the deck surpasses the span entirely by flexural action, as in a conventional continuous beam bridge, but with the arch providing intermediate elastic supports by means of the columns between the arch and deck.

The shallowness ratio of the arch is not inside the recommended range for bridges, which goes from 5 and 10. Above this maximum, axial stresses on the arch increase rapidly, as well as flexural action caused by moving loads, possible differential settlements of the foundations and thermal and rheological effects. Then, important reductions in the rise of the arch may take place and hyperstatic effects due to second order deformations need to be taken into account.

The arch has constant thickness and responds to the increase in axial forces from crown to abutments by linearly increasing its width, from 10 m in the central span up to 20 m at the abutments.

The Infante Dom Henrique Bridge exhibits high technical and aesthetic qualities and represents an important technological advance in construction, both because of the magnitude of its dimensions and because of the following set of relevant aspects:

- It is the second largest concrete arch in Europe; with a span L = 280 m, it is only surpassed by the Krk Bridge, in Croatia, constructed in 1979 and which, with a 390 m span, held the world record for 18 years, up to 1997;
- It holds the world record for straight segmental arches; with a constant thickness of 1.50 m (approximately L/187), it stands out for being extremely slender in relation to the usual thicknesses used in conventional rigid arch solutions (between L/40 and L/60);
- The rise of f = 25 m means a shallowness (L/f = 11.2) for the arch that has no parallel in the field of large span arch bridges;
- Its "static coefficient" ($L^2/f > 3000$), which is directly proportional to the axial force existing at the crown of the arch (Figure 4), is the largest of any concrete arch built to this date.



$$H = \frac{p}{8} \times \left(\frac{L^2}{f} \right)$$

Figure 4 – Structural response of a "perfect arch"

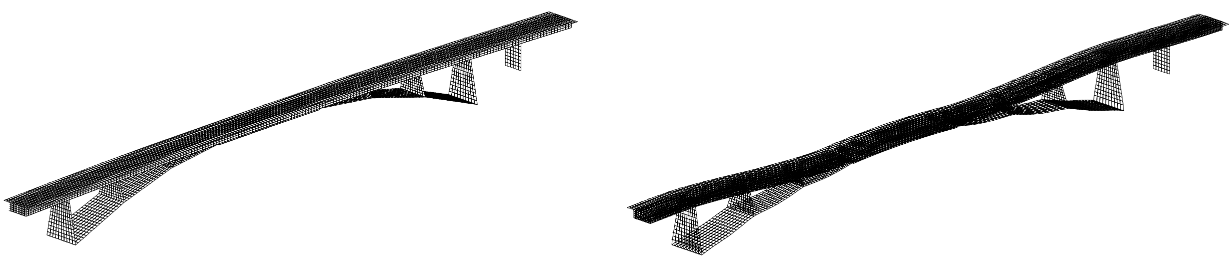
In fact, it may be said that this arch is the most loaded and also the most "delicate" in the world. Notwithstanding the fact that it is a world record holder for slenderness, it possesses the greatest axial force of any arch ever built.

Evidently, it is a bridge inspired by the works of art designed by the brilliant Swiss engineers Robert Maillart and Christian Menn. From the former, it is mentioned the Bridge over the Schwandbach stream, built in 1933 and with a span of 37.4 m. From the latter, the Hinterrhine Bridge, in the Viamala Gorge, and the two bridges with a span of 112 m and built in the second half of the twentieth century over the Moesa stream, on the south slope of San Bernardino’s Pass, are recalled.

3. Analysis of the structural response of the bridge

The structural response of the “flexible arch – rigid deck” combination has the following basic characteristics [1]:

- With the exception of the inevitable flexural compatibility at abutments, the absence of important bending moments in the arch, as a result of its low level of rigidity;
- The reduction in the arch rise due to thermal and rheological action is controlled and distributed by the rigidity of the deck; by hindering the rise loss from attaining significant values, axial force variations suffered by the arch are relatively moderate;
- The deck behaves as if it were a continuous beam on elastic supports provided by columns spaced 35 m apart; in fact, the contribution of the deck towards resisting the applied vertical loads is around 15% for permanent actions and symmetrical live loads; this percentage increases to 20% in the case of asymmetrical live loads; this increase means that the usual high bending moments in the arch under life loads with a pressure line not matching the arch shape are avoided.
- In the arch-deck intersections, the eccentricity between the directrix of the arch and the directrix of the box-beam of the 70 m long central span of the deck implies that the high compression force “arriving” from the arch generates localised high negative bending moments that eliminate the positive bending moments along that central span. Therefore, a convex curvature (Figure 5) in that span that counteracts the deformations that occur in the rest of the structure is guaranteed.
- The high compression force introduced by the arch in the central span of the deck implies also that no prestressing is required on that span after the bridge is finished. Nevertheless, compatibility at the arch-deck intersections implies an increase of positive bending moments in the spans preceding those intersections.



a) b)

Figura 5 – a) Finite element shell model; b) Deformed bridge under dead weight

The option for a single box-beam in the 70 m central span, where arch and deck combine into one single element, was also an important factor in the optimisation of the structure. In effect, the dead weight of the structure per metre (in length) in this span is close to half of the weight per metre of the structure anywhere else on the bridge, where the arch and deck are separated.

4. Prestressing

The prestressing cables layout is straight for all families of cables, all of which are located in the deck. This was the correct option given the construction process that was adopted (cantilever advance, for both the deck and the arch, the latter following and hanged from the former). The use of maximum negative eccentricity on the prestressing for this construction phase is then recommended, and that requires the cables to be positioned in the upper slab of the deck. This option was also correct in view of the need to occupy the webs with the provisional diagonals of the arch-deck triangulation.

There are, therefore, two families of prestressing cables: the *"S"* family, housed in the upper flange of the box-beam, and the *"I"* family, housed in the lower flange of the box-beam. The positioning and application of the tension to every cable was meticulously studied in order to control the stresses in the deck during all construction phases.

During construction, before closing the arch, negative bending moments are paramount in the beam-box. Thus, the majority of the *"S"* prestressing cables were placed as the cantilevers advanced. After the arch was closed, the final prestress was established by extra prestressing cables “S” in the deck above the columns coming from the arch abutments, by extra prestressing cables “I” in the deck spans before the arch-deck intersections, and by taking out all provisional prestressing, including the prestressing cables in the central span.

Provisional prestressing was used, always in the upper flange of the box-beam of the deck, in three distinct situations:

- In response to the high negative bending moments above the provisional pillars;
- To build the deck spans before the arch-deck intersections by the cantilever method, in which spans exist the highest positive bending moments in service;
- To build the central span of the bridge by the cantilever method, in which span a high compression exists in service.

Figure 6 shows that the bending moment diagram in the deck of the bridge constructed by phases (phase accumulation) exhibits a translation towards negative moments in comparison with the equivalent moments existing in the same structure built on total temporary supports.

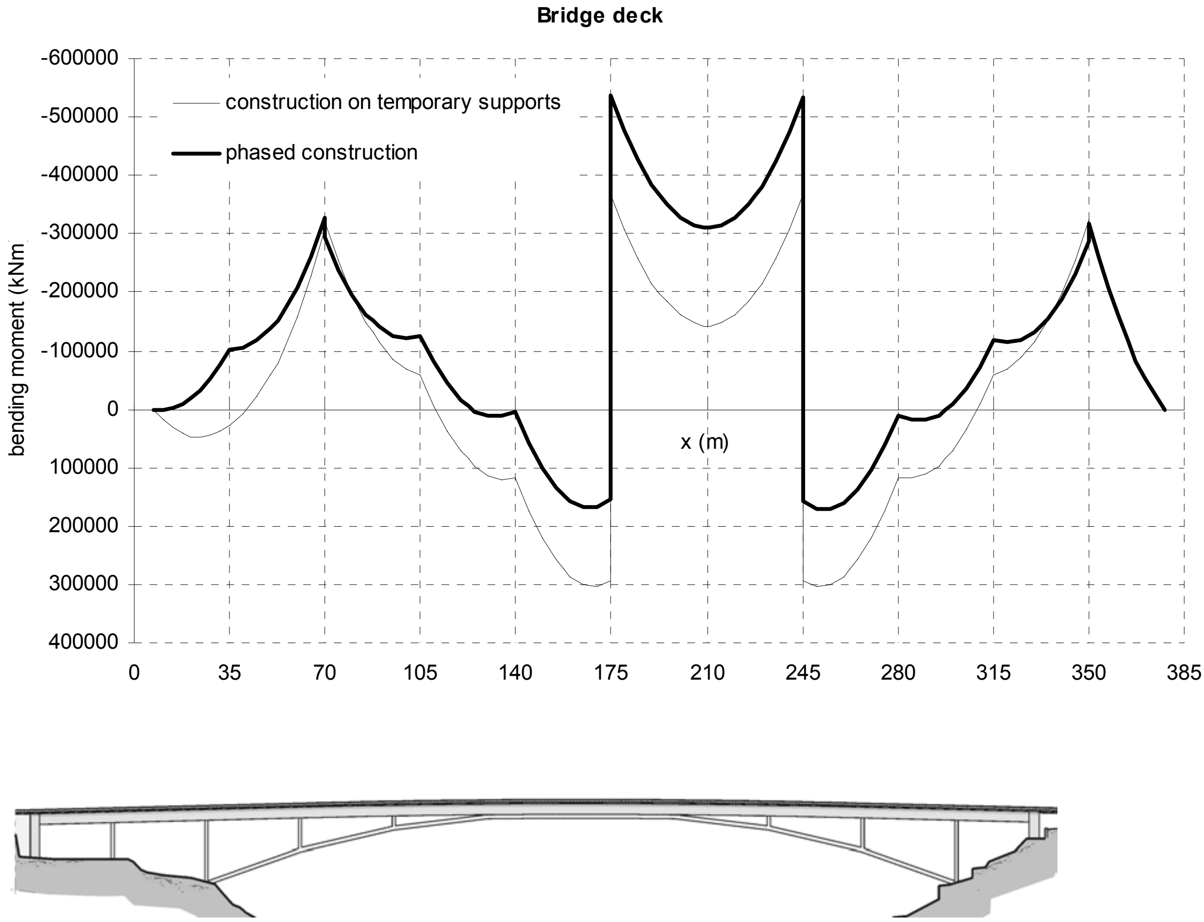


Figure 6 – Comparison of the bending moment diagrams

At first sight, this difference is especially inconvenient in the central span of the structure, where the high compression provided by the arch generates very high negative bending moments. The lower flange of the box-beam could then be subject to compression stresses that would not be admissible, and the “centralisation” of that compression force cannot be achieved through prestressing, since prestressing is not very efficient in reducing compression stresses in tubular cross-sections. On the contrary, hyperstatic effects of prestressing may successfully reduce compression stresses, since the hyperstatic effects generate important flexural forces together with negligible axial forces.

Figure 7 shows the influence coefficient diagram of one of the cross-sections (next to the arch-deck intersection) of the central span in which the negative bending moment is at its maximum. Identification of the most effective prestressing cables in controlling stresses in those same cross-sections is then evident. In relation to prestressing in the lower flange, favourable stresses are induced by hyperstatic effects of prestressing forces introduced, after the arch and deck were finished, between columns M3 or M4 and the corresponding arch-deck intersections. Also, the removal of temporary construction prestressing forces in the upper flange of the central span of the bridge generates significant hyperstatic moments that are highly favourable. Therein, the objective of reducing the negative bending moments in the central span of the bridge is achieved by adding those two effects.

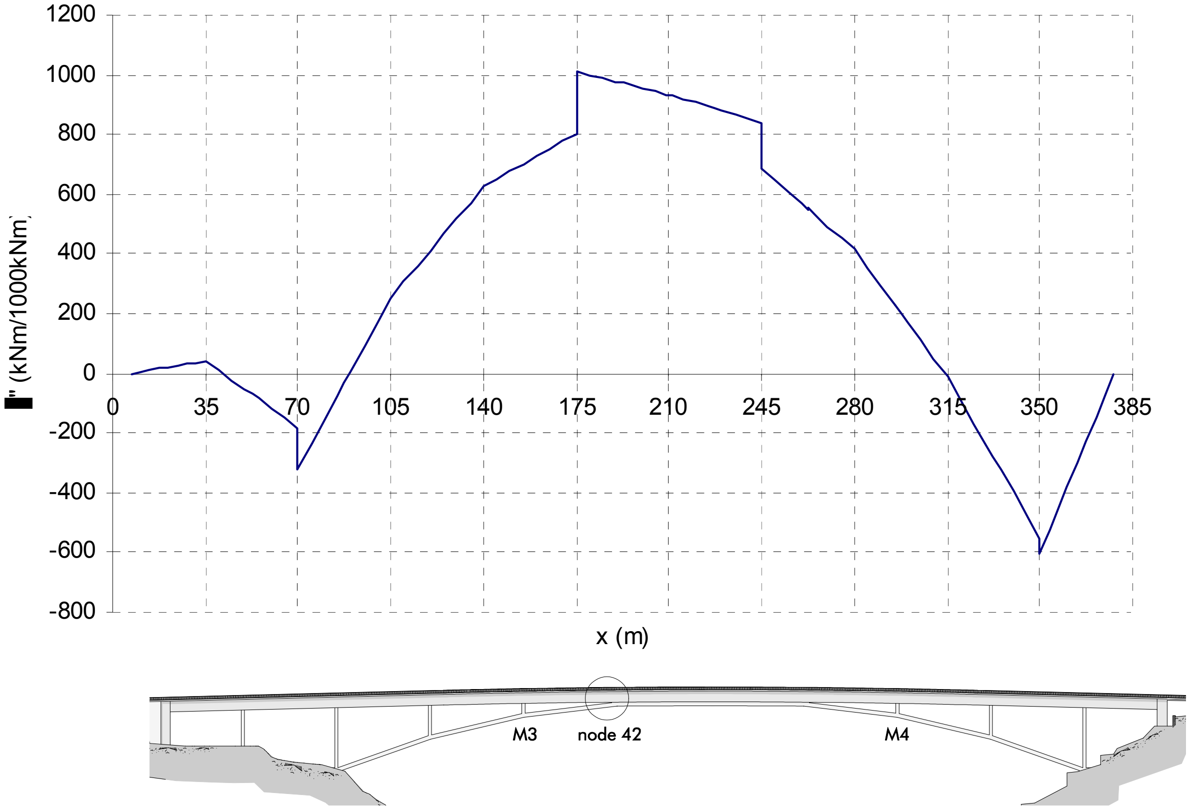


Figure 7 – Coefficient of influence at node 42 of the central span

- The coefficient of influence *(η''_M)* of cross-section *j* upon cross-section *i* is the hyperstatic bending moment (*M)* in cross-section *i* due to a prestressing force *(P)* of unit value applied with a eccentricity *(e)* of unit value along an element of unit length in cross-section *j*. Thus, the total hyperstatic bending moment in cross-section *i* due to a constant prestressing force *P* along length *L* is given by (1):

$$M = \int_0^L P(x).e(x).\eta''_M .dx \qquad (1)$$

5. Construction method

An arch that is so shallow and so slender can only function structurally if in conjunction with the deck. As a matter of fact, this bridge is very much a girder bridge. Therefore, classic construction methods of arch bridges are not the most appropriate. On the contrary, if the bridge is very much a girder bridge, then construction methods for such bridges are the most suitable.

In the tender phase, a construction system was proposed that involved the construction of temporary piers with cables staying both the arch and the deck. Notwithstanding, it was recognized to be illogical to build first a cable stayed bridge to be later transformed into an arch bridge (Figure 8).

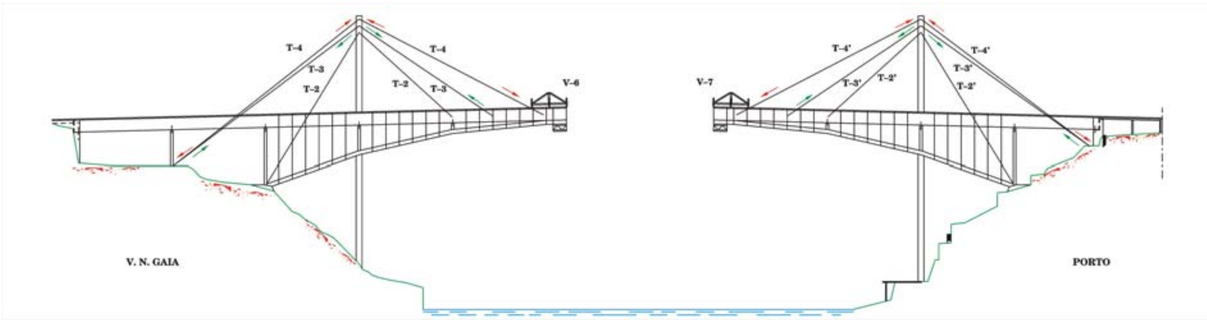


Figure 8 – Construction method at tender phase (1997)

Once the design was awarded, the construction method was revised and the solution described herein was adopted. That is, "only one bridge" was built [2].

In general terms, the construction method was to advance by cantilevering the deck and the arch from each side of the river, as shown in Figure 9. Two provisional pillars were built first in order to reduce the span from 280 m to 210 m, during construction, and trusses were created by adding tensile diagonal bars (provided by provisional stays) and vertical compression bars (provided by the reinforced concrete columns and provisional steel struts) between the arch and the deck. Therefore, two cantilever trusses of considerable height were constructed until deck and arch met. The 70 m central span was built by the traditional *in situ* advance of box-beams, and the bridge was finished when the two "half bridges" were united by the central box-beam segment.

Figure 9 – Construction method

Trusses were created similarly in the slopes outside the arch. They were defined by the deck, abutments, column P1 (on the Gaia side of the river – Figure 10), reinforced concrete struts built on ground and working in conjunction with the rock foundation, and diagonals provided by provisional stays. These trusses transferred to the ground the high longitudinal tensile forces in the deck cross-sections located above the provisional pillars. Diagonals worked as backstays and ensured that the two “half bridges” were tied back to the deck abutments until they were united at the centre.

Tensile forces in all diagonals were applied and regulated in a predefined order to control the structural response of the two “half bridges”, at each phase.

Equilibrium of the advancing cantilevers was secured by inclined ground anchorages and by footings connected together by ground reinforced concrete struts (Figure 10). Geometry of these footings was optimized in order to mobilize the rock foundations in resisting the horizontal components of the construction forces, which meant that forces generated in ground struts were kept under control and stability of the rock slopes was ensured. This ground-structure interaction was studied extensively in an elastoplastic finite element model with Mohr-Coulomb behaviour, demonstrating that a significant alteration to the direction of the lines of force acting on all footings was possible during construction.

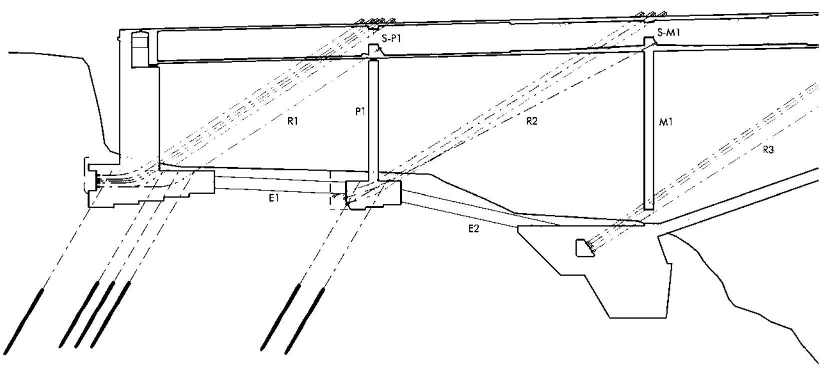


Figure 10 – Backstays, ground struts and anchorages on the Gaia side of the river

The deck above the slopes outside the arch was built on traditional scaffolding. Pairs of advancing formworks for the deck and the arch were used only after columns M1 (Figure 10) and M6 (on the Oporto side) were built. Slenderness of the arch required stays or suspension bars from the rigid and powerful deck to be installed during construction (Figure 11). In fact, axial compression in the arch must be generated before bending moments due to self weight could be resisted. Indeed, one, major advantage of the adopted construction method is the gradual introduction of compression in the arch and its foundations by the truss system, allowing for creep effects to be better controlled.

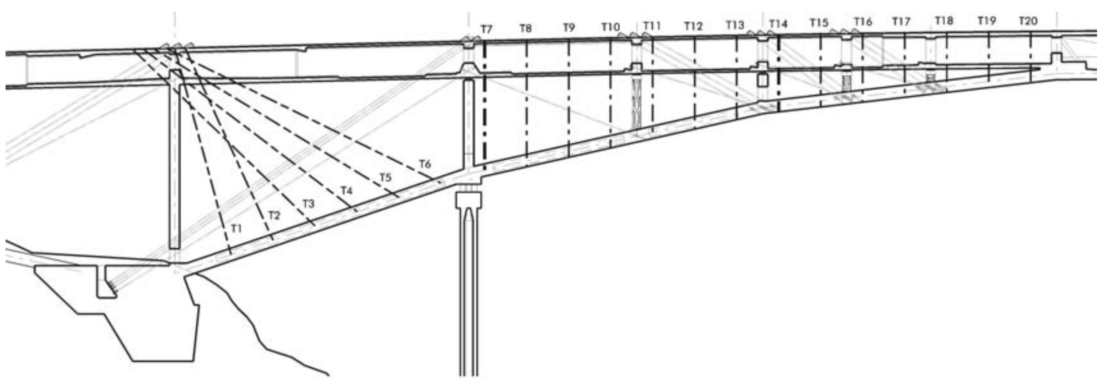
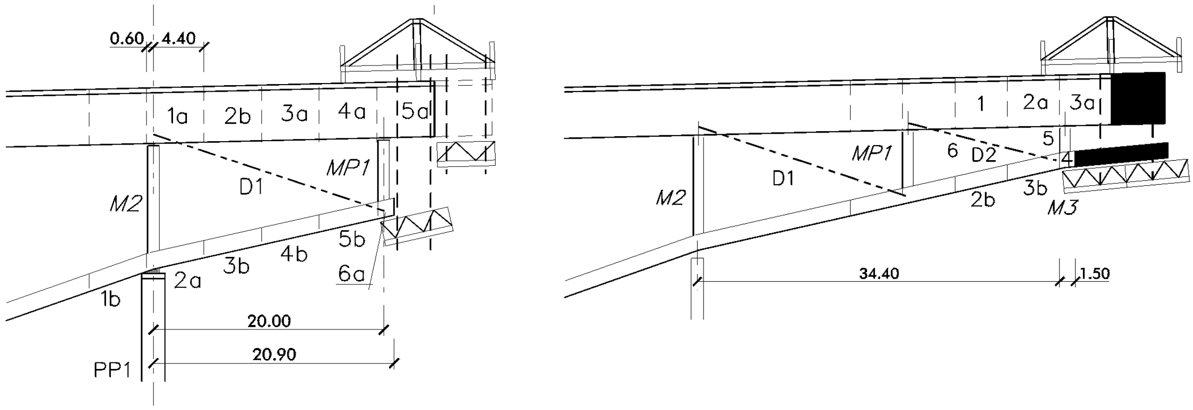


Figure 11 – Stays and suspension bars for the arch

After columns M2 (Figure 12) and M5 (on the Oporto side, symmetrical to M2) were built, a predefined upwards force of 9000 kN was introduced at the top of those columns by a set of jacks and a predefined force of 5000 kN was introduced at the top of the provisional pillars by another set of jacks. These forces produced positive bending moments reducing the negative moments generated by the segmental cantilever method at the deck cross-section above columns M1 and M6 and at the arch spring cross-sections.

Construction of both deck and arch progressed from columns M2 and M5 till 20 m of cantilever spans were built. At this stage, provisional struts MP1 (Figure 12) and MP6 (on the Oporto side, symmetrical to MP1) were positioned and corresponding diagonals D1 and D8 were tensioned to initiate the truss behaviour of each "half bridge".



a) b)

Figure 12 – a) D1 - provisional diagonal; b) D2 - provisional diagonal

This procedure was repeated for each triangular element of the advancing trusses. At predefined instants, tensile forces in relevant diagonals and backstays were adjusted to control internal forces and displacements generated in the two “half bridges”. These adjustments were determined by analytic calculation of the evolving hyperstatic system set up in the axial force influence matrix of all active bars (diagonals and backstays), as explained in 7.1.

Between column M3 (Figure 12) and the arch-deck intersection, two provisional struts MP2 and MP3 were positioned in order to define diagonals D3 and D4 with efficient inclination. Symmetrically, provisional struts MP5 and MP4 and diagonals D6 and D5 were installed between column M4 (on the Oporto side, symmetrical to M3) and the corresponding arch-deck intersection.

Figure 13 shows the bridge in an advanced stage of its construction. The central 70 m of the bridge is formed by a 6 m high box-beam built also by the segmental cantilever method. When construction of the two 35 m cantilever spans was half-way, a downwards settlement of 25 mm was introduced on top of both provisional pillars. There settlements had been programmed at design stage and envisaged a convenient redistribution of internal forces before closing the bridge.

Figure 13 – Construction stage in January 2002

After the two “half bridges” were united by the crown segment (Figure 14), backstays, diagonals, provisional struts and pillars were all dismantled obeying to a very detailed sequence [2].

Figure 14 – General view of the bridge before provisional elements were dismantled

Several techniques and procedures used in the construction of the Infante Dom Henrique Bridge can be considered highly innovative. Moreover, to build a large bridge subject to a geometrical precision criteria never before demanded was, in itself, an enormous challenge for the Contractor. And the erection of an extremely slender and shallow arch over the distance of 280 m lead to the construction of the deck ahead of the "suspended" arch, which is a method that had been used only once before, with Nakatanigawa Bridge, in Japan, where the arch spans 100 m and rises 19 m.

The generalised use of computer controlled and aided systems should be also mentioned, which is a situation that is going to be a feature of construction in the twenty first century. As examples from the construction of the Infante Bridge, the following points are noted:

- The positioning of the support platform for the arch formwork was adjusted to the millimetre before the concreting of each segment of the arch, and that was carried out using two automatic and computerized hydraulic systems.
- The monitoring of the structural behaviour of the bridge during construction was carried out by centralized computer systems that collected data from all important structural elements of the bridge and that automatically stored, managed and processed these data in order to interpret readings supplied by the internal monitoring devices.
- The special operations of upwards and downwards settlements on top of the provisional pillars, as well as the release of the bridge from those provisional pillars, were controlled by those computer systems, with on line follow up of readings in the internal monitoring devices.
- The removal of the provisional pillars weighing 8000 kN was achieved by means of a rotation and transfer system (Figure 15) with on line control and adjustment of forces in the hydraulic jacks that suspended the rotation axis located halfway up the pillar.

Figure 15 – Removal of the provisional pillar on the Oporto side of the river

The segmental cantilever method of construction of both the deck and the arch was performed with a very ingenious double formwork traveller (Figure 16) allowing the simultaneous execution of the arch and deck segments. This operation became very difficult when the distance between them reduced as they approached the arch-deck intersections. And all that was implemented with extremely tight geometric tolerances.

Figure 16 – Double formwork traveller

6. Comparison between construction methods

If the structural solution of the bridge is bold, then it is no surprise that its construction was difficult and complex. This would be the case no matter what construction method was adopted. Amongst the many possibilities studied, it was concluded that the construction process used was the most suitable for a number of reasons, among which the following are emphasized:

- With the construction method used, the rock layers under the arch springs were gradually compressed as the construction advanced; the resulting advantages are evident; this reason is so important that alone could be justify the decision taken.
- With the construction method used, the arch was gradually compressed as the construction advanced, which allowed compensation of its elastic shortening and reduction of shrinkage and creep effects; therefore, the arch functioned just the way it was designed right from the start of its construction, that is, under compression;

internal forces were introduced gradually, as shown in Table 1 for the "half bridge" on the Gaia side of the river.

Table 1 – Evolution of the axial forces in the arch spring cross-section on the Gaia side.

Phase	Axial force (kN)
On reaching column M2	12280
On reaching column M3	29770
On reaching the arch-deck intersection	174470
After uniting the two "half bridges"	269280
After removing all backstays, diagonals and arch suspension	278650

- With the construction method used, the highest compression force in the arch occurred when the diagonals were removed, therefore when the structure was fully built, with a small increase in that force from the instant the arch was closed. On the contrary, the temporary cable stayed method proposed at tender stage would have implied the transfer of most of the total axial force of 278650 kN with the release of the stays. Such a sudden application of force to the arch would certainly not be a good solution, especially in relation to an arch as slender as this one.
- The construction method of temporary cables staying the arch bridge would add the uncertainties resulting from hyperstatic redistribution of thermal effects between the concrete bridge and stays to the complexity of the geometric control; in such a case, there would be a greater risk of error in the computation of internal forces and stresses in the deck.
- With the adopted construction method, internal forces in the deck, arch, diagonals and struts are much less susceptible to redistribution, which, it should be underlined, is difficult to evaluate and control.
- With stays from provisional towers, during construction both the arch and the deck would be dead weight supported by the stays and not performing any structural function. This is not the case with the construction method that was adopted, which is thus a lot more efficient.

7. Structural adjustment

7.1. Adjustment forces in active bars

The adjustment of internal forces in active bars (backstays and diagonals) meant the modification of internal forces and displacements generated in the structure.

It is important to note that once the internal forces and displacements are "compensated" up to any construction phase, through forces applied in active bars in previous phases, variations of the forces in active bars for the next construction phase were dependant only upon forces and/or displacements at that same phase. This is true but for the effects of the rheological behaviour of concrete, which were not relevant during the construction phases.

Therefore, "compensated" variation values from all previous phases add up to "compensated" accumulated values (internal forces and displacements) at each phase.

During construction, it has already been stated that the deck carries out the function of tension flange of the trusses. But it should be added that each “half deck” under construction was “fixed” to the abutment in order to avoid any horizontal displacement of the “half deck” until the two “half bridges” were united at the centre. Advantage was taken from the flexural rigidity of the abutments, which were greater than the axial rigidity of the backstays. Therefore, deck abutments withstood flexural moments in order to balance the remaining axial force developed in each “half deck”. But with the successive adjustments to the forces acting on the backstays, the flexural moment in the corresponding abutment was also subject to variation, either because the advancing cantilever was generating an increase of the axial force in the deck or because backstays forces were increased and thus were balancing more of the axial force in the deck. Notwithstanding, flexural moments in each abutment were always towards the slope because backstays were always pulling the “half deck” towards the abutment.

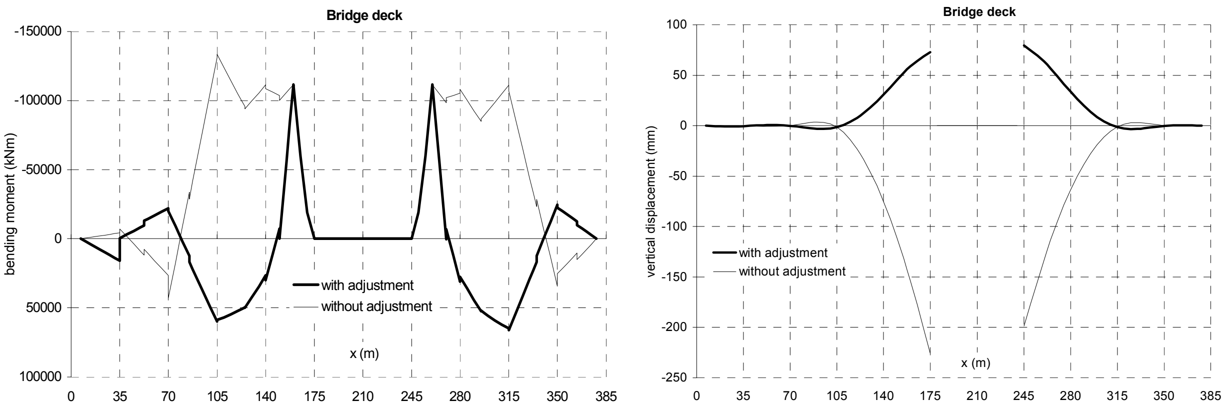
Distinctly, the only function of the adjustment of the forces acting on diagonals was to control the flexural forces on the deck, simultaneously modifying its displacements. The objective was to compensate the negative bending moments resulting from the cantilever construction, thus avoiding higher negative moments in the deck.

7.2. Analytical calculation of adjustment forces

Internal forces produced by the gravitational loads applied in each construction phase were calculated for that phase. Forces corresponding to an imposed deformation on each one of the active bars (backstays and diagonals) were also calculated. Forces at each phase were then linear combinations of all previous forces. The combination coefficients were determined by imposing as many conditions as there are active bars in that phase, as explained in 7.3.

To better recognize the need for this procedure, the effect of casting the concrete in the deck and arch between the last provisional struts (MP4 and MP5) and their corresponding arch-deck intersections is taken as an example. Figure 17 displays the bending moment and vertical displacement diagrams in the deck with and without adjustment of forces in the active bars.

Therefore, the successive application of gravitational loads in all construction phases generated internal forces and displacements that were “compensated” by the adjustment of forces installed in the “active bars”.



a) b)

Figure 17 – a) Bending moment diagrams; b) Vertical displacement diagrams

7.3. Formulation of the adjustment of tension in the active bars

The adjustment of forces applied by active bars was carried out according to an internal force criterion, for which the following equations were established:

$$f_j^{pp} + \sum_{i=1}^{n} f_{ji}^{Ti} x_i = b_j \quad \text{ou} \quad \begin{bmatrix} f_1^{pp} \\ \vdots \\ \vdots \\ \vdots \\ \vdots \\ \vdots \\ f_m^{pp} \end{bmatrix} + \begin{bmatrix} f_{11}^{Ti} & \cdots & \cdots & f_{1n}^{Ti} \\ \vdots & & & \vdots \\ \vdots & & & \vdots \\ \vdots & & & \vdots \\ \vdots & & & \vdots \\ \vdots & & & \vdots \\ f_{m1}^{Ti} & \cdots & \cdots & f_{mn}^{Ti} \end{bmatrix} \times \begin{bmatrix} x_1 \\ \vdots \\ \vdots \\ \vdots \\ x_n \end{bmatrix} = \begin{bmatrix} b_1 \\ \vdots \\ \vdots \\ \vdots \\ \vdots \\ \vdots \\ b_m \end{bmatrix} \qquad (2)$$

where

- n - no. of active bars (bars to be post-tensioned)
- m - no. of bars of the tension adjustment structural model *(>n)*
- $n+1$ - no. of load cases of the tension adjustment structural model for
- i - load case *i*
  Case *1* – applied gravitational loads; Case *2* – applied force in active bar 1; ... ;
  Case *n+1* – applied force in active bar *n*
- f_j^{pp} - force in bar *j* due to applied gravitational loads
- f_{ji}^{Ti} - force in bar *j* due to the tensioning of active bar *i*
- x_i - scale factor, or combination coefficient, affecting the load case corresponding to the post-tensioning of active bar *i*
- b_j - value (null or not) of force in bar *j*

Imposing values in *n* lines of vector b_j ($m\times1$), the combination factors x_i ($n\times1$) of the load cases referring to the application of tension to the active bars were calculated.

The force applied in each active bar *i* results from the multiplication of factor x_i by the force generated in that bar in the load case corresponding to its tensioning.

$$\text{Applied force on active bar } i = f_{ki}^{Ti} x_i$$

where *k* is the corresponding line to active bar *i* in the influence matrix (2).

The adjustment force, that is, the variation of the internal force acting in active bar *i* at the end of each phase does depend on its applied force and also on the response of that same bar to the tensioning of all the other active bars.

$$\text{Adjustment force in active bar } i = \sum_{i=1}^{n} f_{ji}^{Ti} x_i$$

Finally, the internal force generated in a general bar (active or not) j at the end of each phase is given by

$$\text{Final force in general bar } j = f_j^{pp} + \sum_{i=1}^{n} f_{ji}^{Ti} x_i = b_j$$

The vertical displacements of all the nodes q of the structure are obtained in the same way:

$$u_q^{pp} + \sum_{i=1}^{n} u_{qi}^{Ti} x_i = u_q$$

where

u_q^{pp} - vertical displacement of node q due to applied gravitational loads

u_{qi}^{Ti} - vertical displacement of node q due to the tensioning of active bar i

x_i - scale factor, or combination coefficient, affecting the load case corresponding to the post-tensioning of active bar i

u_q - final value of the vertical displacement at node q

8. Mathematical models used in the analysis of the construction process

8.1. Evolutive analysis

First, a linear elastic analysis of the bridge was carried out using the software ROBOT [3] to calculate forces acting on backstays and diagonals at every construction phase of the bridge. This analysis allowed the design of all structural elements of the bridge, as well as the fine-tuning of the construction process.

The criterion for the adjustment of forces installed in the active bars is expressed in the influence matrix described in 7.3. Strict limits on bending moments in the abutments and the maximization of deck length under positive flexural action were the adjustment restrictions. With these forces applied in active bars, instantaneous deformations in every construction phase were also determined.

Afterwards, two evolutive calculations were performed, apart from one another, which consider the time-dependent behaviour of concrete through the correct modelling of the viscous-elastic properties of materials. These mathematical models were coded into the DIFEV [4] and FASES [5] softwares, both confirming the results of the adjustment criterion used in the linear elastic model.

The calculation with the DIFEV software considered 58 phases. This model is less detailed than the model implemented with the FASES software, where more than 1000 phases were defined, with all execution steps inherent to the execution of each segment of every structural element introduced separately. The modelling of the viscous-elastic behaviour of concrete is different in the two softwares – DIFEV follows the MC78 (additive model) [6] and FASES follows the MC90 (multiplicative model) [7]. This difference provides an evaluation of the sensibility of the structure to long-term effects, necessarily distinct in the two models.

Parallel to the FASES evolutive model, a linear elastic model incorporating all steps and construction phases considered in the FASES code was developed with ROBOT. This model was used to generate the influence matrices of the active bars, step by step, and provided the countercheck of the FASES model output. It provided also the analysis of the responsiveness of the bridge at any construction step with respect to any alteration in the geometry or in the applied forces and to the effects of differential thermal variations.

8.2. Geometric control criteria

The geometric control criteria for the construction of the Infante Dom Henrique Bridge were the following:

- Geometric conditions of the road:
    i) Grade line of the road;
- Geometric conditions related to structural behaviour:
    ii) Resistant configuration of the bridge as a whole, in particular the rise of the arch and its angles of deviation under the columns, given that any loss in rise would decrease the structural efficiency of such a slender arch and, in the event of major deviations, would generate non-negligible second order effects.
    iii) The shape of the arch, which was built with cambers distant from the fundamental buckling modes in order to obtain additional safety factors in relation to instability caused by the bending of such a slender and heavily compressed element.

Geometric corrections introduced into the structure are “instantaneous actions”, but decision on the value of any geometric correction was always provided by an evolutive calculation. Thereafter, values of actions (modification of forces in diagonals) to be implemented in order to achieve that correction were obtained from the influence matrices referring to that instant, that is, with linear elastic behaviour for the reinforced concrete, for which knowledge of the values of the elasticity modules of the concrete at that instant was required.

Therefore, decisions on any correction and on corresponding actions to be implemented were based on analyses performed with both linear elastic and evolutive mathematical models, but, as construction progressed, those models were updated with data that was collected continuously from the bridge internal and external monitoring system.

Moreover, corrections modified the construction sequence defined in the evolutive mathematical models; therefore, the new sequence had to be introduced into those models and only at the end of the evolutive analyses of the modified models, when all provisional structural elements are disassembled, effects of that particular intervention could be fully evaluated.

Total shrinkage and total creep of the concrete are considered to take place at 20000 days of age of the bridge. Therefore, the intended final geometry for the bridge was established at that age and time-dependent effects incorporated in the evolutive mathematical model were then considered in defining the construction geometry of each segment of the bridge.

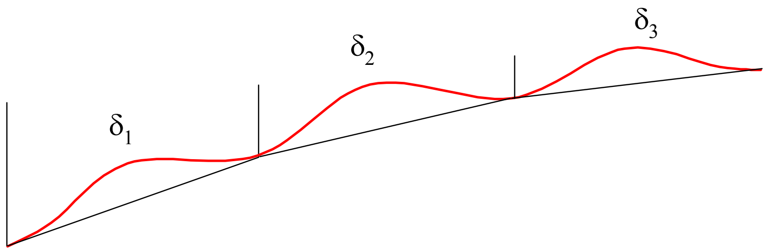
It must be mentioned that redistributions of stresses due to change in the resistant structural system were less important than those resulting from difference in age of the concrete of distinct structural elements.

9. Control of the construction

9.1. Geometric control

Structural efficiency of the Infante Bridge both at construction phase and at service phase demanded a very strict control of all parameters. In particular, geometric conditions ii) and iii) in 8.3 were detailed with the utmost care.

For example, the arch was built with cambers distant from the fundamental buckling modes, as indicated in Figure 18. These cambers had to be constructed with maximum precision (see Figure 19, where the curves "designed" and "real" are shown) and high performance concrete (C60/75) was used for the first time in Portugal.

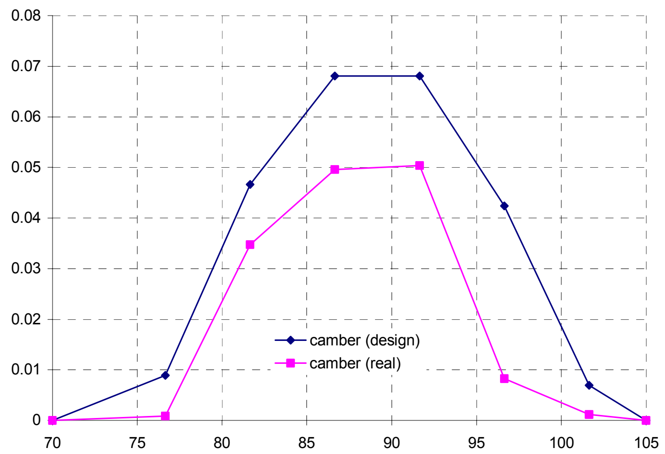


$\delta_{1,t\infty} = 0{,}10$ m

$\delta_{2,t\infty} = 0{,}15$ m

$\delta_{3,t\infty} = 0{,}10$ m

Figure 18 – Definition of the cambers of each half arch



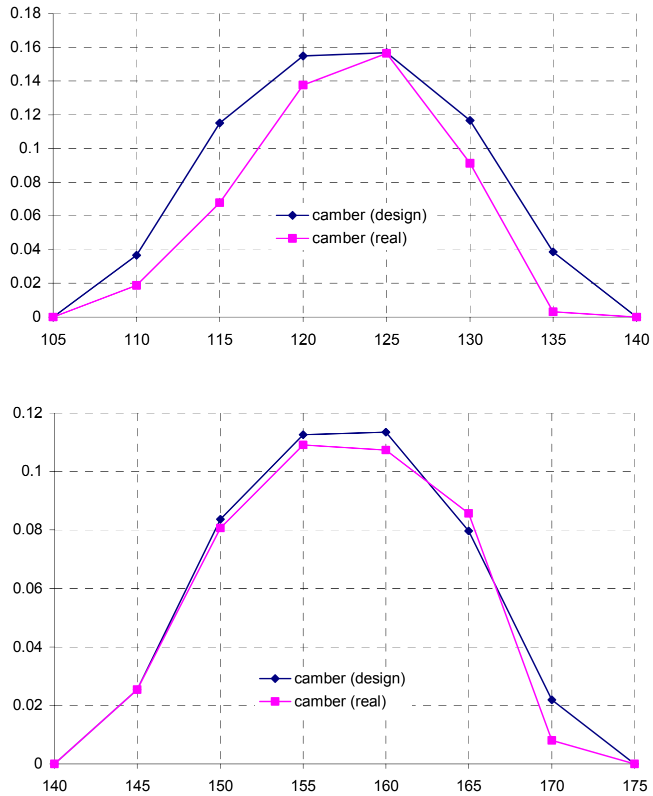


Figure 19 – Comparison between the designed and real cambers

9.2. Monitoring during construction

Construction of this bridge was a major achievement requiring a highly efficient monitoring system, capable of assessing physical quantities of different types, namely support reactions, axial forces, bending moments, rotations and temperatures at particular sections of the arch, deck and provisional struts and pillars, and axial forces in the temporary stay cables (backstays and diagonals).

The monitoring of the construction of the bridge was performed by three separate instrumentation systems, one for the granite slopes on each side of the river, another for the foundations and another for the concrete elements and temporary stay cables (Figure 20).

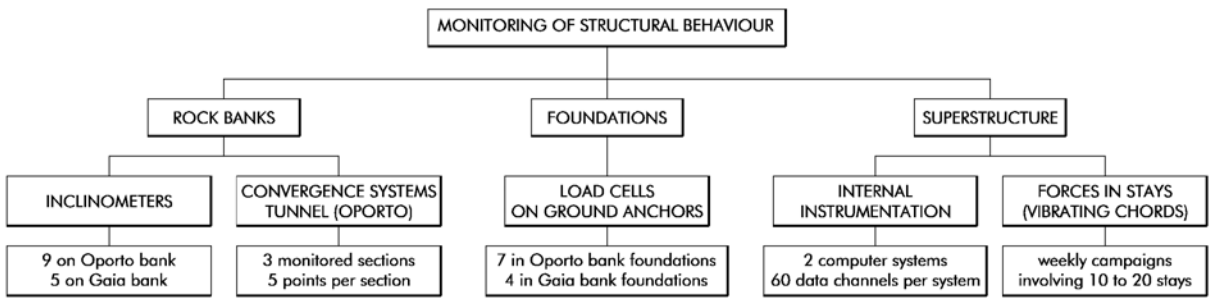


Figure 20 – Monitoring system implemented during construction

9.2.1. Instrumentation of the rock slopes and foundations

A total of 14 inclinometers, going 40 m deep into the granite slopes on both sides of the river (9 on the Oporto side and 5 on the Gaia side), were installed. In addition, inside the old tunnel located near the footing of the arch on the Oporto slope, three transverse sections were monitored with optic convergence systems, as shown in Figure 21.

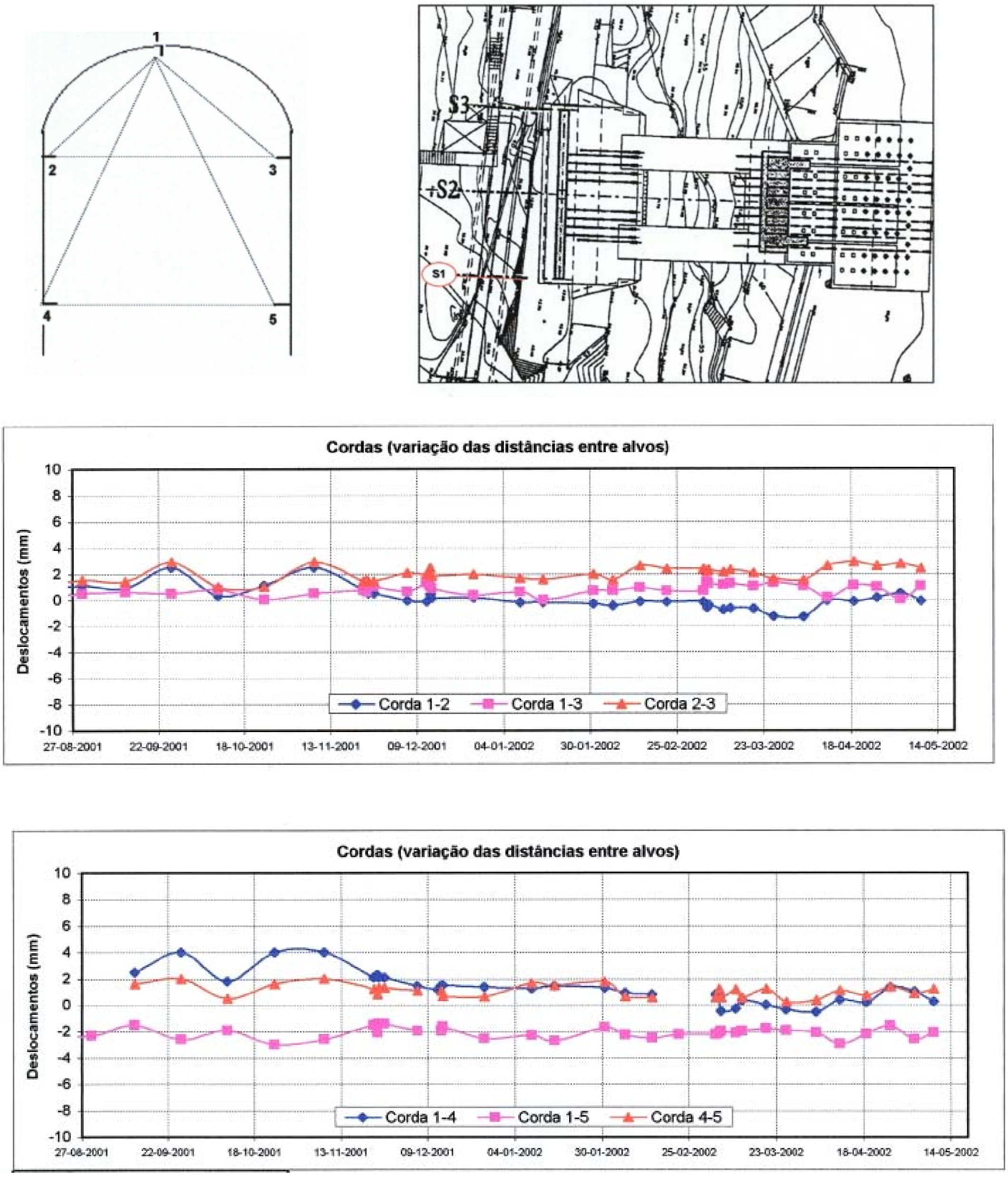


Figure 21 – Optic convergence system in the old tunnel

In both sides of the bridge, a total of 11 ground anchorages were provided with load cells (7 in the Oporto side and 4 in the Gaia side). Figure 22 shows the graphical evolution of the forces in the monitored ground anchorages of the Oporto abutment of the deck.

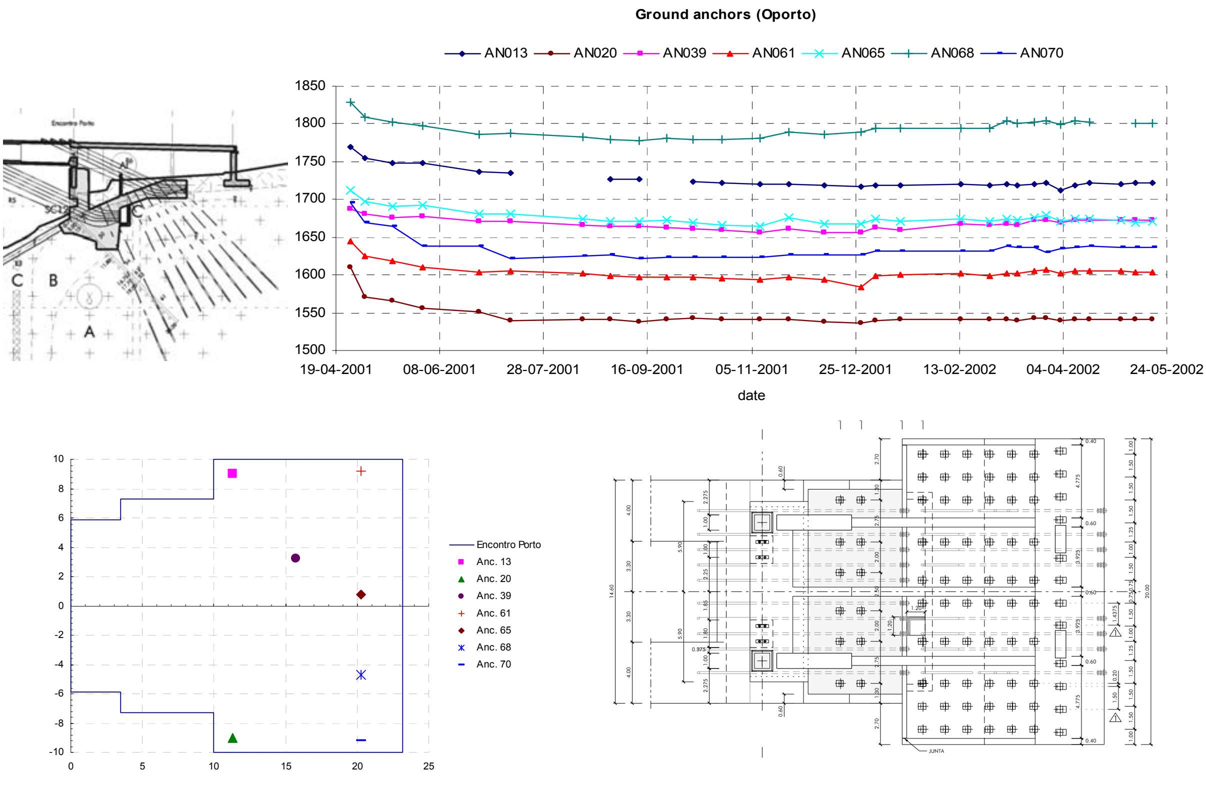


Figure 22 – Location and evolution of forces in monitored ground anchorages in the Oporto side

9.2.2. Internal instrumentation of the superstructure

The internal instrumentation of the bridge superstructure was especially important [8]. It was controlled by two computer systems that were located inside the box-beam of the bridge (one in each half bridge). They worked independently and collected data were transmitted by modem to other workstations (in the construction yard office, in the supervisory agents' office, in the designers' offices – AFAssociados, in Oporto, and IDEAM, in Madrid – and in Kinesia - Ingeniería de Auscultación Company, who were in charge of the system). The system provided details on a number of different parameters, such as reactions in supports and bending moments, axial stresses, rotations, and temperatures in selected sections of the arch, deck, columns and provisional struts and pillars. A total of 120 sensors were installed (strain gauges, tiltmeters, thermometers and load cells), as indicated in Figure 23 and Table 2.

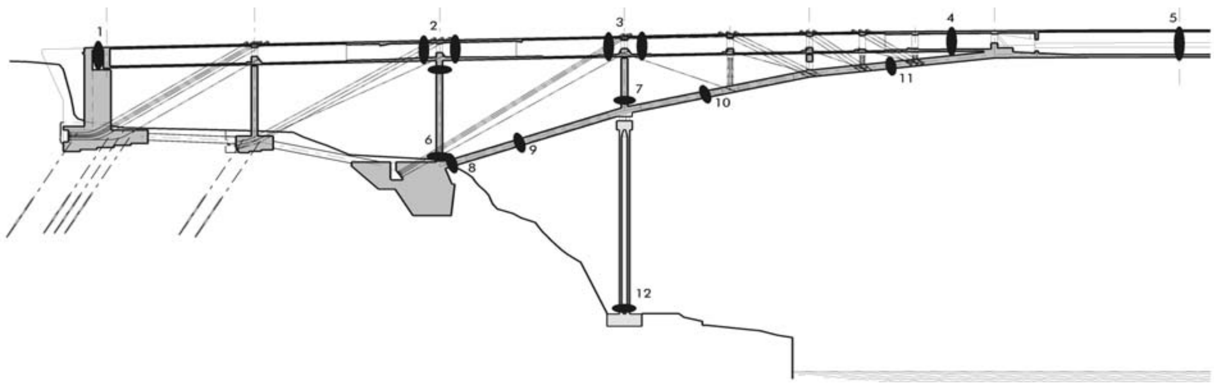


Figure 23 – Location of the sections containing instrumentation in the Gaia half bridge

Table 2 – Sensors in the Gaia half bridge

Section	Sensor	Type of measurement
1	Load measuring bearings	Axial force in deck
2	Tiltmeters Thermometers	Rotation of deck and column, mean curvature of deck (bending moment) Temperature of deck upper and lower slabs
3	Tiltmeters Strain gauges	Rotation of deck, mean curvature of deck (bending moment) Axial force and bending moment in deck
4	Strain gauges Thermometers	Axial force and bending moment in deck Temperature of deck upper and lower slabs
5	Strain gauges	Axial force and bending moment in deck
6	Strain gauges	Axial force and bending moment in column
7	Strain gauges	Axial force and bending moment in column
8	Strain gauges	Axial force and bending moment in arch
9	Strain gauges Thermometers	Axial force and bending moment in arch Temperature of arch upper and lower fibre
10	Strain gauges	Axial force and bending moment in arch
11	Strain gauges	Axial force and bending moment in arch
12	Strain gauges Thermometers	Axial force and bending moment in provisional pillar Temperature in opposing faces of the provisional pillar

9.2.2.1. Adjustment of the structural behaviour models

The system incorporates an "intelligent" statistical correlation module that allows adjustments to readings of sensors and variation intervals of the thermal environmental parameters and mechanical properties of structural materials (creep, shrinkage and elasticity module). That feature was most helpful for the design team when controlling and interpreting data provided by the system. Typical difficulties associated with the deterministic reading of that data were overcome and statistic methods were used to obtain reliable and "mechanically meaningful" measurements that could be compared with values established at design stage, thus supporting a rapid decision-making process.

Another important feature is the capacity to "identify" parameters of the mechanical behaviour of the bridge. For example, creep of the actual structural concrete cast in the bridge could be "measured".

Figure 24 shows an example of correlation of readings from one strain gauge at the base of column M1 (Figure 10) with readings from one thermometer inside the concrete. The upper right hand graph in Figure 24 displays that correlation incorporating automatically the effects of the rheological behaviour of the concrete and the loading history of the structure.

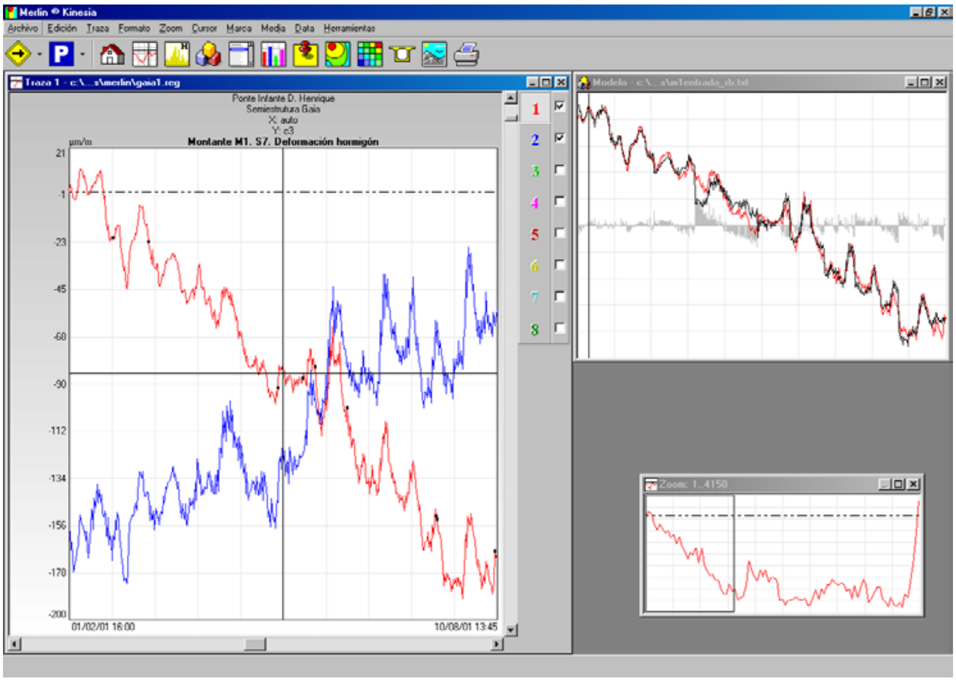
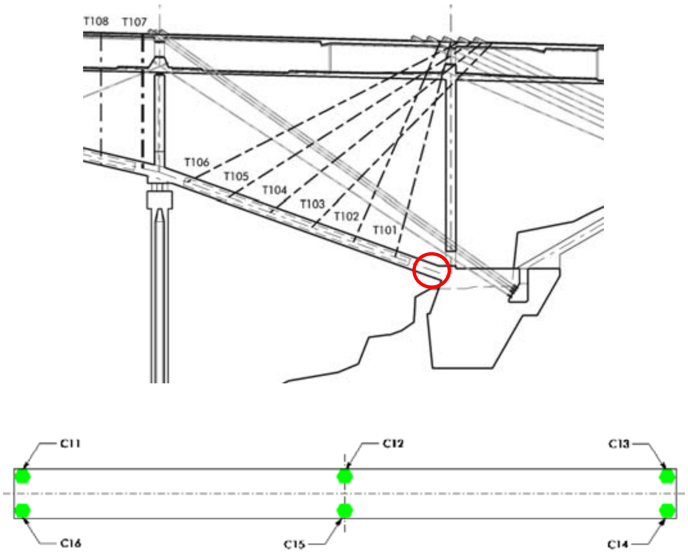


Figure 24 – Example of statistical processing of data provided by monitoring sensors

9.2.2.2. Identification of the need for structural corrective operations

Figure 25 provides an example of readings from the monitoring system that lead to a "structural correction". Graphs show the development of strains in the upper and lower fibres of the spring cross section of the arch on the Oporto side. The arch is clamped to the abutment and the progressive "gap" between the readings on the two fibres signifies negative bending moment at that section was increasing, because of insufficient tension applied to the suspension stay cables during the construction of the first span of the arch. This situation was corrected by re-tensioning the stays before the provisional pillar became a support for the advancing structure.



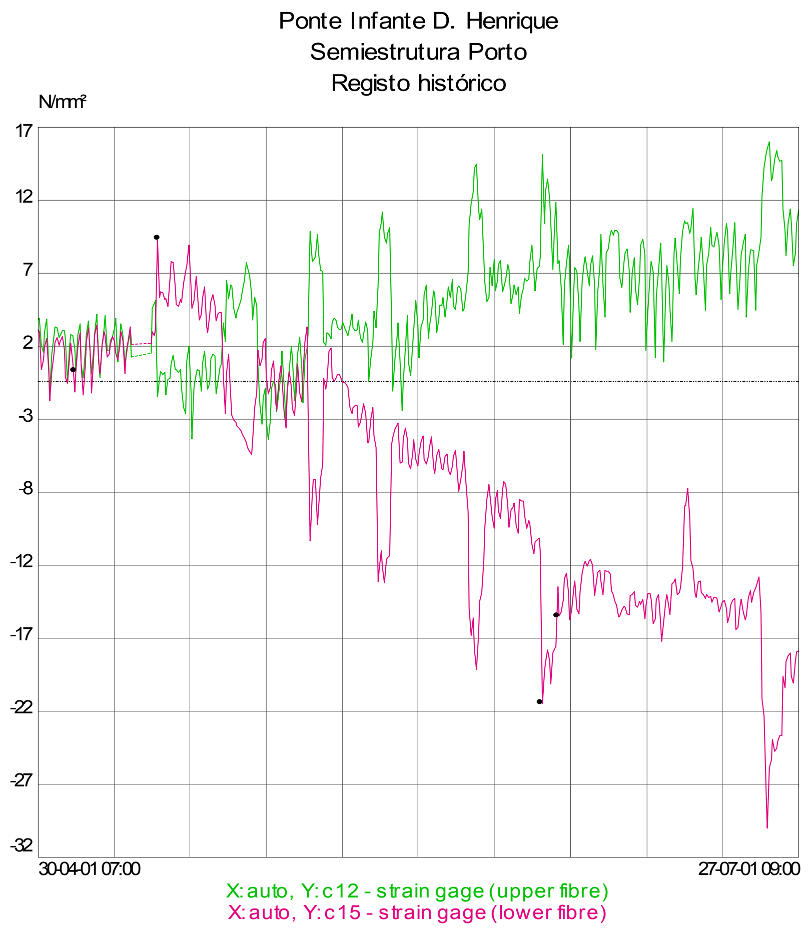


Figure 25 – Development of strains in the upper and lower fibres of the spring cross section of the arch

9.2.2.3. Follow up of the structural response during special operations

The construction method involved several special operations defined at design stage. The first consisted in the application of predefined upwards forces at the top of columns M1 and M6 and at the top of the provisional pillars, as explained before (in Section 5). Mention was also made of downwards settlements introduced on top of both provisional pillars when the two cantilevers of the central span were half-way in their construction.

Another special operation took place at the end of the construction of the bridge, when supports provided by the provisional pillars to the closed bridge were taken away simultaneously on both sides of the river. This process involved the implementation of very precise and controlled partial settlements with the help of hydraulic jacks with a total capacity of 100000 kN for each side. Arch and deck above those provisional pillars descended 90 mm in this extremely delicate operation. An operation controlled in real time by the monitoring system, which provided the continuous checking of data in relation to expected values, previously calculated. Figure 26 shows the comparison between expected and measured values for the upper and lower fibres of the deck cross section above the provisional pillar on the Oporto side, during the jacking operation.

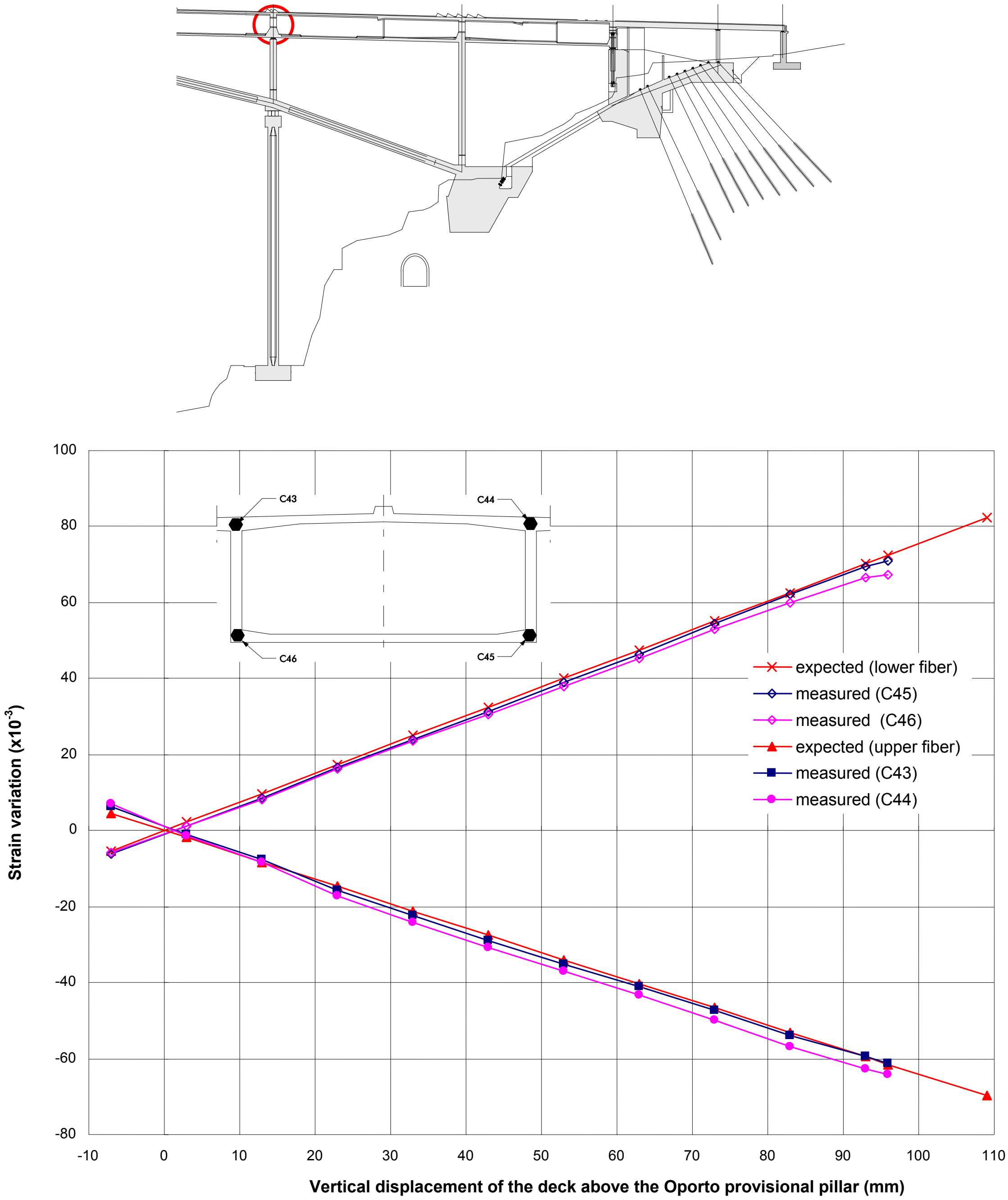


Figure 26 – Strains in the deck cross section above the Oporto provisional pillar

9.2.2.4. Follow up of the structural response during load testing

The structural response of the bridge during load testing was monitored directly and instantaneously for a variety of load cases of both static and dynamic nature.

As an example, Figure 27 shows the “measured values” of the lines of influence of strains in the upper and lower fibres of three cross sections of the deck, obtained with the passing of four 30 ton trucks with a constant speed of 5 km/h.

In this figure, the high level of precision and reliability of the readings can be confirmed.

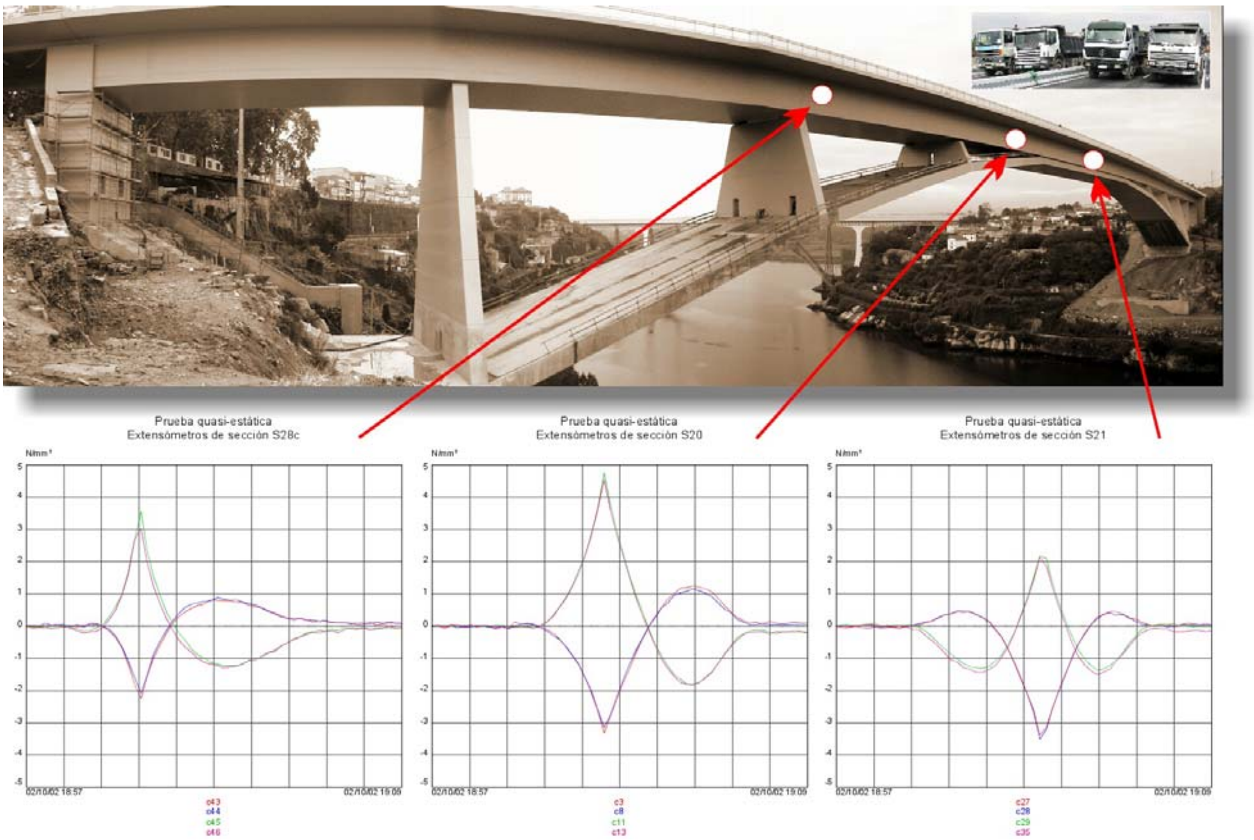


Figure 27 – Lines of influence of strains in three cross sections of the deck

9.2.3. Monitoring of forces in provisional stay cables

A large number of sensors would be required for direct measure of tensile forces on all provisional stays (backstays and diagonals). Moreover, load cells or other permanent devices are very expensive and thus it was decided to measure cable forces indirectly from their vibration frequency, according to the vibration chord theory [9]. An independent team from the Oporto Faculty of Engineering performed that job with portable equipment. On average, a weekly campaign involving 10 to 20 stays was carried out [10].

Freyssinet C-Range system cables with multiple 15.7 mm diameter parallel strands were used in all suspension stays of the arch, diagonals and backstays. Strands were sheathed individually to secure their durability during construction. The number of strands in each cable was 37 in diagonals and backstays, and from 9 to 15 in suspension stays of the arch. Cables were pulled to 60% of f_{ptk}, always from the deck. Length of individual cables varies from 12 to 52 m.

Indirect estimation of the cable tension through measurement of the corresponding vibration frequencies, combined with the use of portable equipment, has proved to be an accurate, simple, and rapid form of analysis for the majority of stay cables.

9.2.3.1. Equipment and test procedures

Measurement of cable natural frequencies was performed by a small high sensitivity piezoelectric accelerometer with a magnetic base attached to the stay cable, as shown in Figure 28. A portable Fourier Analyser based on a laptop and a PCMCIA card collected the pre-amplified ambient vibration signal and provided average power spectral density estimates using 6 to 10 time records, each with an approximate length of 40 s.

(a) (b)

Figure 28 – (a) View of instrumented cables (b) Measurement of vibration on a suspension cable of the arch

Figure 29 shows the average spectral estimates of the ambient response of one suspension cable of the arch during two distinct stages of tensioning. The marked peaks are representative of the cable harmonics, corresponding natural frequencies showing an increase that is associated with an increase of axial force, in line with the vibration chord theory. Since the tension estimate depends on the value of these single frequencies, an adequate frequency resolution is required in order to minimize the error associated with the tension estimate. This was achieved by a selective choice of the sampling rate. For the present case, this rate could vary from 19.3 to 250 Hz, producing an error in the tension estimate of less than 1%.

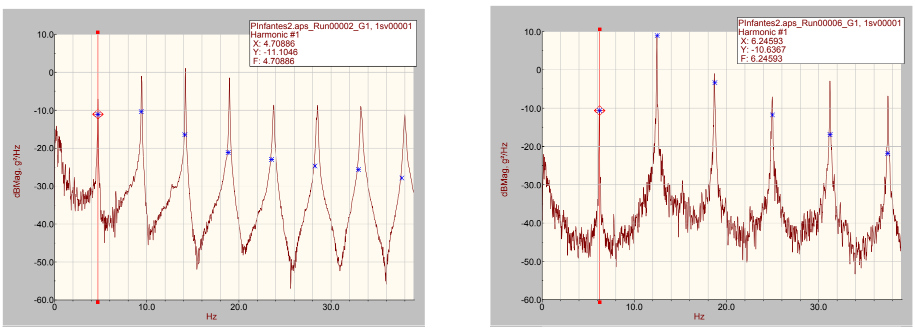


Figure 29 – Average spectral estimates of the ambient response of one suspension cable of the arch during two distinct stages of tensioning.

9.2.3.1. Results

Figures 30, 31 and 32 display the evolution of axial forces in some suspension cables of the arch, backstays and diagonals, respectively. Additional curves shown in those Figures refer to the expected variation of the same forces obtained from the mathematical models at the relevant construction stages of the bridge.

If relevant construction operations are considered in the analysis of these figures, good agreement is found between measured and expected forces. Discrepancies that were identified were also present in the monitoring system of the deck and arch, and they were explained by occasional deviations at the site from the construction schedule.

For example, axial forces in suspension stays T104, T105 and T106 in Figure 30 increase at every concrete casting operation until 18-08-2001. The weight of the reinforced concrete was to be balanced by adequate tensioning of each pair of new stays, with minor modification of tensile forces in stays previously installed. Because new stays were tensioned to lower values, previous stays were taken an extra burden. As explained before, pairs of strain gauges in the arch also detected a progressive bending of the arch (Figure 25) and the subsequent re-tensioning of stays was implemented.

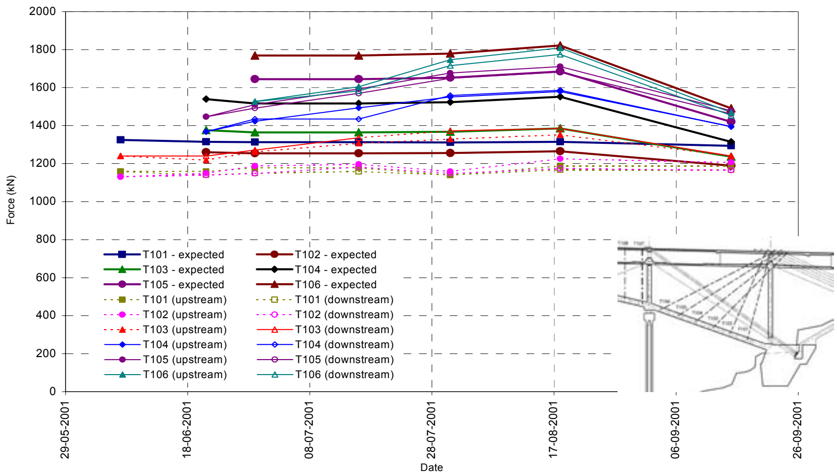


Figure 30 – Variation of axial force on suspension cables of the arch (measured *versus* expected).

Figure 31 shows that no variation greater than 5% was measured in backstay cables. This is in agreement with the expected constant values, because the deck is fixed to its abutment and no relevant vertical displacement was taking place at the cross sections of the deck where the backstays are anchored.

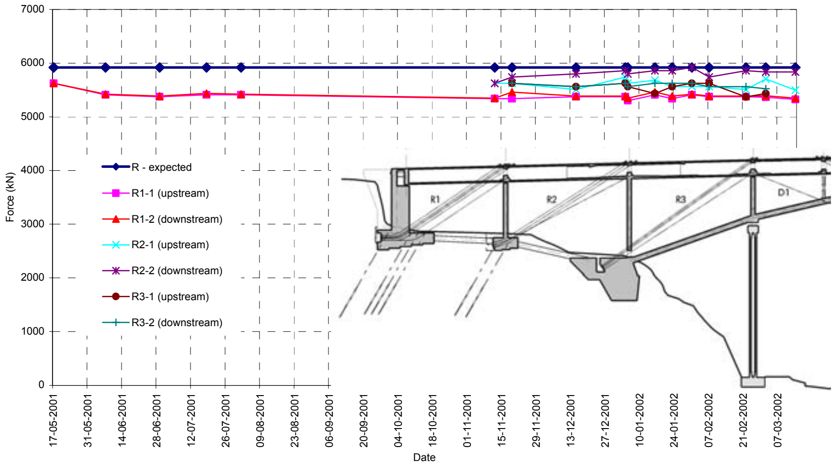


Figure 31 – Variation of axial force on backstay cables R1, R2 and R3 (measured *versus* expected).

Variations of the axial force in diagonal D1 are shown in Figure 32. These variations are of passive nature and they are due to the global flexibility of the structure under construction. The segmental cantilever method of construction and tensioning of new diagonal cables can be identified in the diagram. Casting of segments implies increase of cable tension and tensioning of new diagonals implies decrease of cable tension. The advance of the structure and, consequently, the increase of distance between the construction front and diagonal D1 signifies that variations in tension are progressively smaller, as shown in Figure 32.

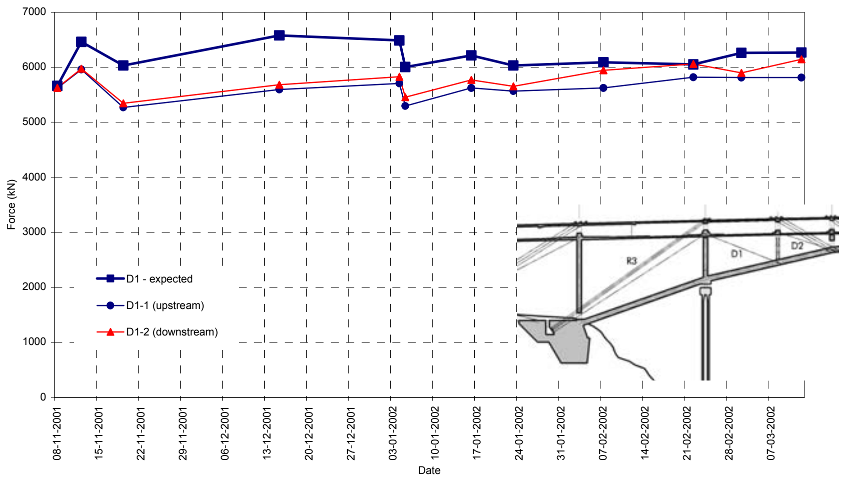


Figure 32 – Variation of axial force on diagonal cable D1 (measured *versus* expected).

Topographic campaigns during construction of the bridge are most important and a wide range of correlations were made, controlling the entire construction process in geometrical terms and providing reliable and counterchecked data for decisions to be taken, as explained before.

10. Control of the bridge in service

The internal monitoring system used during construction was kept operative and is now following the structural behaviour of the bridge during its lifetime.

Continuous information about the "in situ" time-dependent effects of the cast concrete is provided and associated redistributions of internal forces are identified through readings in sensors.

The methodology of analysis of the measured data includes the identification of the section deformation due to temperature variations and the statistical evaluation of the rheological properties of concrete, as it was made during construction.

At design phase, the prediction of concrete creep and shrinkage is always very difficult because it is associated with the variability of many parameters, namely those depending of the local environmental conditions.

The monitoring system will permit the identification of the time-dependent behaviour of the bridge and its contrast with the assumptions made during the design phase.

Figure 33 shows the comparison between the expected and measured values of stresses in the reinforcement at the central cross section of the bridge.

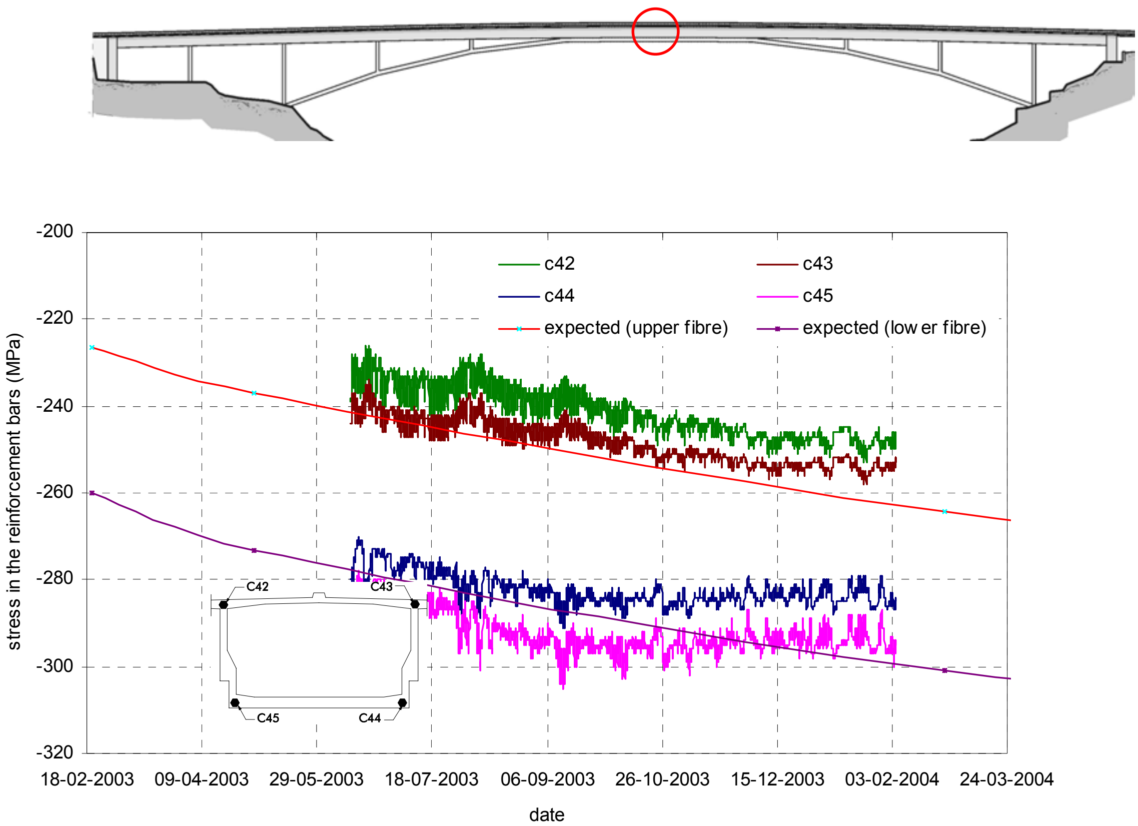


Figure 33 – Stresses in the reinforcement bars at the central cross section of the bridge

(expected *versus* measured)

11. Final remarks

Designers and Contractors were aware that the construction of the Infante Bridge was extremely difficult, but it was always carried out in conditions of total safety and control of its structural behaviour, largely because of the reliability and capacity of the installed monitoring system.

Therefore, the Maria Pia and Luiz I Bridges welcome the Infante Dom Henrique Bridge in their vicinity, as shown in Figure 34.

Figure 34 – Aerial view of the Maria Pia, Infante Dom Henrique and Luiz I Bridges

12. Acknowledgements

Acknowledgements are due to our colleagues at AFAssociados and at IDEAM for their contributions to the discussions in which many of the final solutions were found.

13. References

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