PERFORMANCE OF CONCRETE BRIDGES IN PORTUGAL

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The purpose of this paper is to present the author's experience in inspecting, repairing and strengthening relevant 25 to 70 years old concrete bridges and to present a synthesis of the main deterioration problems, structural anomalies and interventions required in these bridges. Recent inspections of the repaired bridges to evaluate the durability of the repair works will also be referred.

1. CHAMINÉ BRIDGE

1.1. Description of the Structure

The Chaminé Bridge, over the River Raia on the EN2, km 474 between Montargil and Mora is a 95,60 m long 3 arched reinforced bridge with 27,20 m spans , 3,40 m high, supporting a ribbed slab deck through 2,57 m spaced reinforced concrete walls (Fig. 1.1). The bridge was designed in 1932 and built in 1934.

Fig. 1.1 - General view

Typical reinforcement details are shown in Fig.1.2.

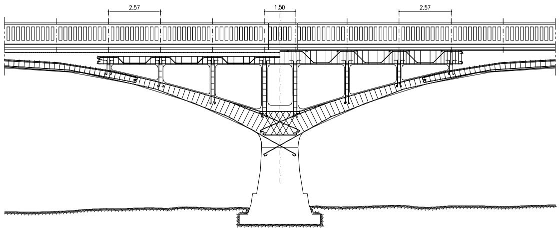


Fig. 1.2 - Reinforcement details

1.2. Inspection and Reasons for the Intervention

The main reason for the intervention was related to the traffic restrictions created by the bridge on the national road EN 2. The objective was to enlarge the deck from 6,75 m to 10,50 m with two 3,50 m lanes and 1,00 m side lanes. At the same time the bridge underwent global rehabilitation works, naturally needed due to the reduced maintenance during its life.

In 1994 the bridge was inspected showing globally a good condition. Nevertheless the following anomalies could be found:

- Significant water penetration through the deck, showing stalactites on the bottom face of the slab (Fig.1 3). When the stalactites were broken, during a dry period, a significant amount of water poured from the deck, showing that the water was retained above and inside the slab (Fig. 1.4).

Fig.1.3. - Effects of the water penetration

Fig. 1.4 - Water leakage from the slab

- Localised reinforced corrosion at regions of low cover (Fig.1 5) and concrete segregation due to compacting defects (Fig.1.6)

Fig 1.5 - Local corrosion

Fig 1.6 - Segregation of concrete

The measurements of the reinforcement cover showed large variations: 11 mm to 21 mm in the slab; 6 mm to 13 mm in beams; 0 to 13 mm in arches and 20 to 89 mm in walls.

- The river bed was not properly cleaned, thus reducing the capacity of the water flow (Fig. 1.1)

The concrete used had 400 kg of cement per cubic meter of concrete and a water / cement ratio of 0,50. The depth of carbonation was generally small (5 - 20 mm). The smooth reinforcement bars were made of mild A235 steel.

1.3. <u>The rehabilitation and the enlargement of the deck</u>

The transverse sections of the bridge before and after the intervention are shown in Fig 1.7.

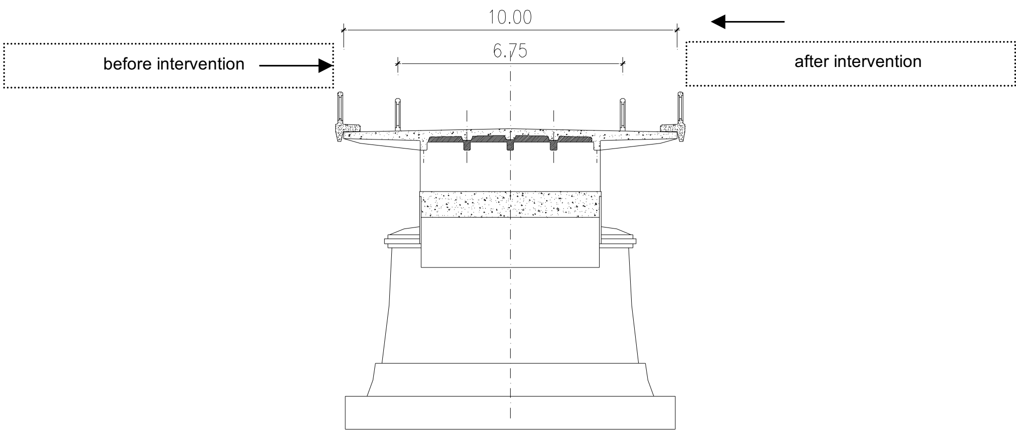


Fig. 1.7 - Transverse section

After removing the 0,20 m thick concrete non structural pavement, the new 0,20 m thick slab reinforced concrete deck was cast over the primitive slab with additional 2,30 m span cantilevers .

A new slab was also cast over the abutments avoiding their widening and giving continuity to the new cantilevers. The abutments were consolidated by injecting cement grout in cracks and construction joints (Fig.1.8).

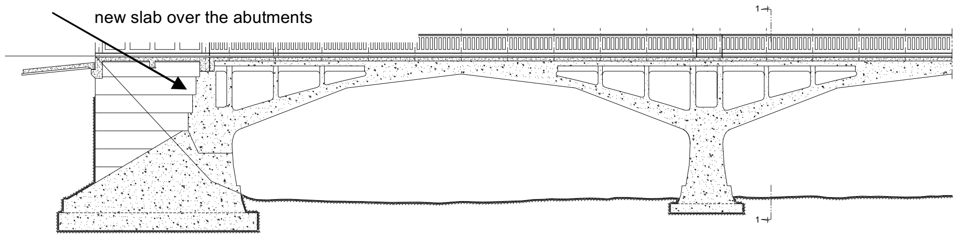


Fig. 1.8 - Longitudinal section

The existing girders were monolithically connected to the new slab increasing its resistance by adding new reinforcement and increasing the total slab thickness (Fig.1.9).

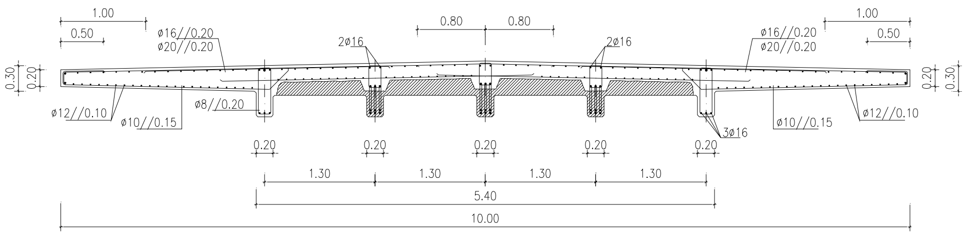


Fig.1.9 - Enlargement and strengthening of the slab

The foundations of the arches in the rock substrate did not require any strengthening. A new bituminous pavement was laid over a new water proof membrane for the deck. New expansion joints, new safety barriers and new approach slabs were introduced. The bridge underwent a general rehabilitation of its surface with protective acrylic paint.

The rehabilitation took place in 1995. The inspection of 2009 did not show any deterioration of the structure.

2. ARCH BRIDGE OVER THE RIVER SADO

2.1. Description of the structure

The "Ponte dos Arcos" over the River Sado was built in 1944. It is a two 33 m span bowstring reinforced concrete deck. The total width is 9,90 m. The arches have a theoretical span of 31,50 m and a height of 5,25 m with a rectangular section of 0,60 m x 1,10 m and ties with

0,90 m x 0,60 m section .The slab has a variable thickness with a maximum of 0,43 m at mid span. The slab loads are transferred to the arch by 8 vertical ties of 0,50 m x 0,30 m section, the central one being connected by a transverse top beam (Fig. 2.1).

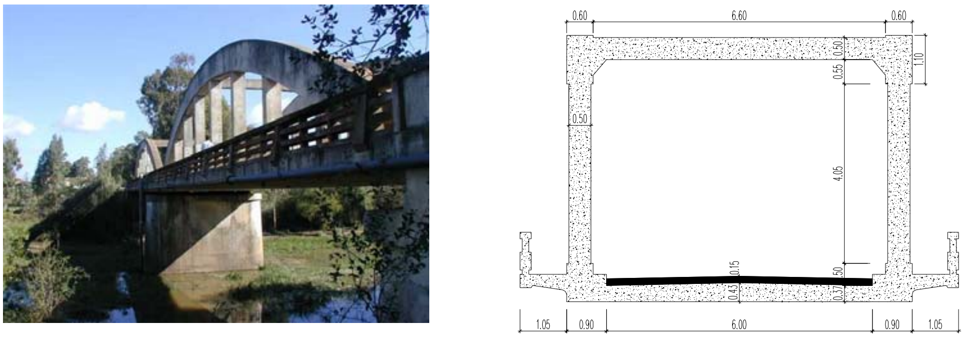


Fig. 2.1 - General view and transverse section of the bridge

The central pier is 5 m high with a rectangular cross section varying from 2,50 m x 10,0 m at the top to 3,00 x 10,50 m at the pile head block foundation, which connects 24 concrete piles of 0,50 m diameter.

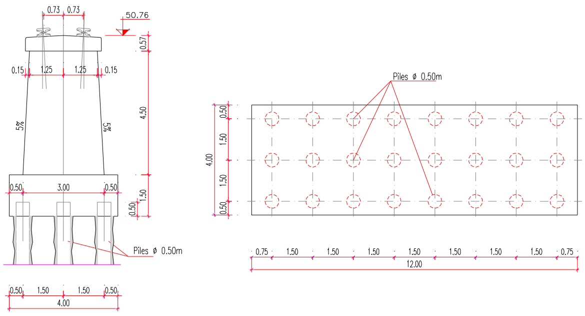


Fig 2.2 - Central pier

The abutments are made of usual front and lateral walls supported on piles. The deck is fixed at the central pier and longitudinally is free to move at the abutments.

2.2. <u>Inspection and rehabilitation</u>

The bridge has had a minor maintenance during its 60 years of service life. The Road Authority decided, in 2005, to access the safety conditions of the bridge according to the actual codes and to execute a general rehabilitation of the structure. The inspection of the bridge showed the following anomalies:

- Local corrosion of reinforcement in the arches, ties and barriers (Fig. 2.3) and slight corrosion of steel bearings (Fig. 2.4).

Fig 2.3 - Local reinforcement corrosion

Fig 2.4 - Bearings at the abutments

- Deterioration of the expansion joints and surface pavement (Fig 2.5) and local concrete removal due to impact of vehicle (Fig 2.6).

Fig 2.5 - View of the expansion joints

Fig 2.6 - Vehicle impact in a transverse beam

The safety assessment led to the conclusions that the pile resistant capacity for the earthquake action load combination is largely exceeded. The 1st the mode of vibration has a frequency of 0,686 Hz.

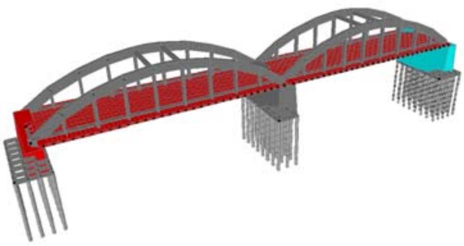


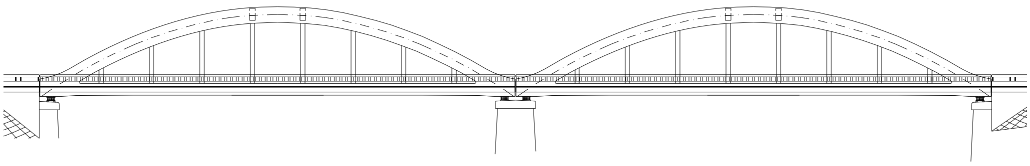
Fig 2.7 - Structural model of the bridge

2.3. The rehabilitation and seismic strengthening design

The decision for the earthquake intervention was the introduction of base isolation of the deck, together with a fuse system to maintain the present behaviour under normal traffic conditions.

This intervention requires the replacement of the present steel bearings by high dissipation rubber bearings HDRBs, to introduce axial continuity of both arches and to introduce a light reinforcement of the foundation of the central pier with new micro-piles. This intervention also requires to provide space between the deck and the abutment and to replace the expansion joints.

A simple design model consisted in introducing the flexibility of these new bearings and to increase the damping ratio from 5% to 10%. In Fig. 2.8 and table 1 a synthesis of the results are presented.



		before intervention	after intervention
Maximum displacement	of the deck	121 mm	157 mm
	of the pier top	121 mm	11 mm
Maximum longitudinal force transferred from the deck	to the abutment	0kN	372 kN
	to the pier	6640 kN	732 kN

Table 1 - Main seismic response parameters before and after intervention

Fig. 2.8 - Longitudinal forces and displacements with base isolation

This solution decreased the horizontal force in the pier from 6640 kN to 732 kN and the longitudinal displacement increased from 121 mm (assuming elastic behaviour) before intervention to 157 mm. This simple model was checked using a nonlinear time dependent analysis with artificial accelerograms to simulate the seismic action. The execution of the general rehabilitation and seismic upgrading started in 2008.

3. VIADUCT DUARTE PACHECO

3.1. Description of the structure

The design of the viaduct dates from 1937 and its construction was finished in 1944. The viaduct has a total length of 355.10 m between supports at the abutments. The width of the deck is 24.00 m. Expansion joints separate the viaduct in 5 main structures (Fig. 3.1):

DUARTE PACHECO VIADUCT IN LISBON

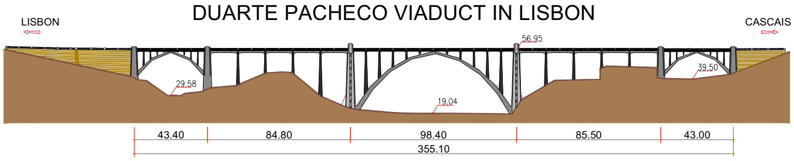


Fig. 3.1 - General dimensioning (dimensions in meters)

- Two lateral overpasses with 3 reinforced concrete parallel 40 m span arches, 3,10 m wide. The deck has 10 longitudinal beams 0,30 m wide and 0,65 m to 0,81 m high, spaced 2,30 m with 10 $\times$ 4,00 m spans. The slab is 0,21 m to 0,34 m thick and has lateral cantilevers of 1,65 m span. The longitudinal beams are supported by transverse walls 2,683m $\times$ 0,40m, hinged at the top and monolitical with the arches.
- Two intermediate viaducts with a total length of 85,80 m. The deck has 10 longitudinal beams 0,50 m wide and 1,20 m to 1,90 m height spaced 2,30 m with 5 $\times$ 16,35 m spans, supported by 4 hinged columns connected by transverse beams. These transverse frames are also hinged (for longitudinal rotations) at mid height with the exception of one frame. The columns have, at the top, a section of 0,697 m $\times$ 1,022 m.
- One main overpass over Av. Ceuta with two reinforced concrete parallel 91,80 m span arches 7,75 m wide The deck has 10 longitudinal beams 0,35 m wide and 0,75 m to 1,15 m height, spaced 2,30 m, with 12 spans of 7.83 m. The longitudinal beams are supported by transverse beams connected through hinges to 4 columns, spaced 6.90 m, with a variable cross section with a maximum of 1,466 m $\times$ 1,641 m. The arches are connected to the foundation of main piers, 43,00 m height. Those piers support the deck of the main overpass and the

intermediate viaducts. These piers have a voided section of 29,00 m × (4,40 m to 6,12 m) and include 8 reinforced concrete slab floors connected by wooden stairs.

The foundations of the viaduct are direct through pads in the limestone rock substrate of the Alcantara Valley.

3.2. <u>Main anomalies and rehabilitation</u>

The visual inspection showed the occurrence of irregular cracking ("craquelet") in the structure, Fig. 3.2, vertical cracking in piers and longitudinal cracks in the arches. The cause of this cracking, typically associated with alcali-silica reaction was confirmed through tests with the electronic microscope (MEV) and mineralogic analysis by DRX. The residual potential for the progression of those reactions was assessed to be very little. The origins were course reactive aggregates in the form of silex and quartz used in the concrete mix.

The cracking due to ASR originate an expansion of the deck, closing the expansion joints and damaging the region of beam supports and the top of the main piers

Fig. 3.2 - Main anomalies

Due to this cracking, and in some cases due to a reduced cover, some corrosion of the reinforcement occurred. As it is common in structures of this period, the scatter of reinforcement cover is high. Carbonation depth varies from 8 mm to 22 mm.

The rehabilitation intervention included the following activities:

- Local repairing of damaged areas due to reinforcement corrosion.
- Crack injection of large cracks ($\underline{\sim}$ 0,50 mm) with cement grout (Fig. 3.3).

Fig.3.3 - Crack injection in columns

In this figure it can be seen that the main structure was protected with a finish mortar. When required this finish mortar was repaired and completed with a cement mortar added with latex.

- General surface protection with a multi-barrier surfacing with an elastic flexible coat covered with an acrylic paint (500 μm total thickness), Fig. 3.4 and deck waterproofing and new surface pavement (Fig. 3.5).

Fig. 3.4 - Surface painting

Fig. 3.5 - Deck waterproofing membrane

- Rehabilitation and replacement of bearings (Fig. 3.6)

Fig 3.6 - Bearing rehabilitation and bearing replacement

- Reconstruction of the slabs of the cantilevers, which had significant deterioration and a slab thickness of 7 cm. They were replaced by a new 10 cm slab (Fig. 3.7). Damaged support regions of the girders were also reconstructed (Fig. 3.8) as well as the top of the main piers.

Fig 3.7 - New cantilever slab

Fig 3.8 - Reconstruction of girder support region

- Strengthening of the cracked abutments with prestressed bars and cement grout injection

3.3. Structure assessment of the existing structure

The safety assessment of the deck showed that, after enlargement of the road, the lateral girders (originally under a pedestrian path) had not enough reinforcement. They were strengthened with CFRP laminates located in the lower region of the webs (Fig. 3.9).

Fig 3.9 - Strengthening of lateral girders with CFRP laminates

The original design did not consider the effect of the earthquake actions. In fact, at the time of its construction, no code provisions considered the effects of earthquakes in bridges.

A three dimensional framed model of the whole viaduct was done (Fig. 3.10) although the 5 structures have, at present, independent responses for the earthquake action.

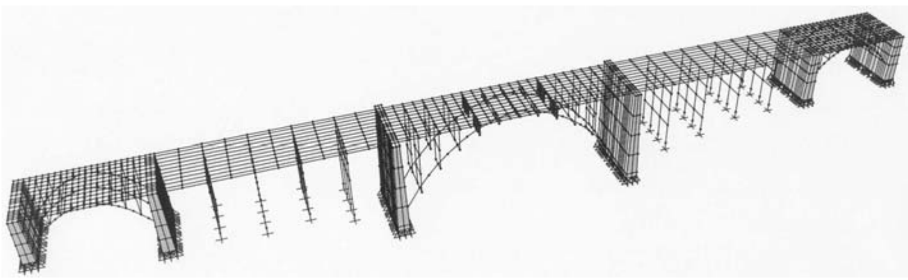


Fig.3.10. - Three Dimensional Model and Restraints

A main characteristic of these 5 structures is its low redundancy due to the various hinges and expansion joints adopted in the design.

The lateral 40 m over passes with the 3 arches behave quite well for an earthquake action due to the rigidity of the arches and rigid connection of the walls that support the deck.

The intermediate viaducts are too flexible in the longitudinal direction. In fact, the deck is supported by sliding bearings at the main piers and by hinges at the columns. For the design earthquake action the displacement of the deck obtained in the calculation is 0,20 m, which shows the flexibility of the structure.

For the central arch the design seismic displacement of the deck, obtained in the model, is 0,15 m.

It was then considered that the intervention should reduce the flexibility of the intermediate viaducts and main overpass and should provide the required strength capacity.

In order to reduce the intervention in the existing structure, the conceptual idea for the seismic upgrading was to connect the intermediate viaducts to the main pier of the central arch structure. Since these piers have a very low reinforcement ratio they need to be strengthened in order to support the longitudinal and the transverse earthquake actions of both the viaduct and main arch. The strength of the main piers was achieved by introducing external vertical prestressing (inside the pier). With the prestressing, not only cracking is reduced but also the bending resistance of the pier is increased.

4. BRIDGE OVER RIVER CAVADO

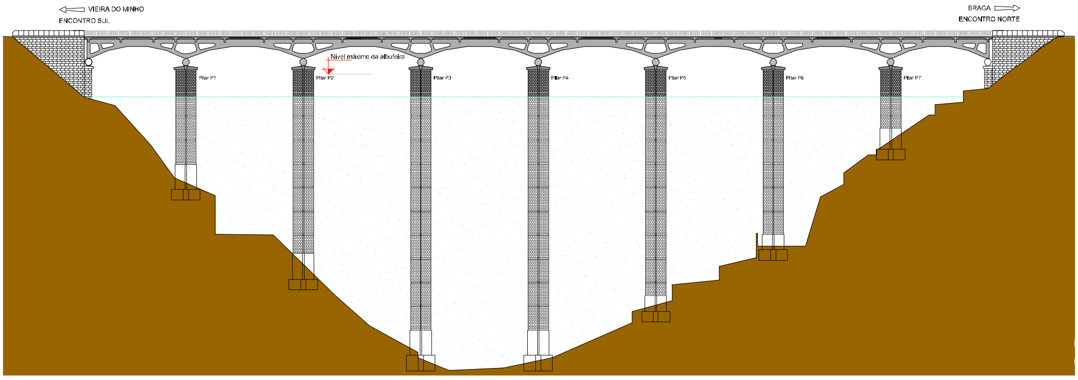
4.1. Description of the structure

The bridge over river Cavado in the EN 304, with a total length of 205,80 m was completed in 1954 and, in 2008, is under repair and strengthening.

The bridge has a special characteristic of having hollow masonry columns that reach 58 m of height with a lozenge cross section with 4 m and 8 m diagonals and a wall thickness of 30cm/40cm, made of granite blocks. At each 4,50 m reinforced concrete diaphragms strengthened the columns (Fig. 4.1).

The deck is a continuous beam with 19 m + 6 x 23 m + 19 m spans, 8,00 m wide with two main reinforced concrete girders of variable depth from 1,60 m in the span to 4,00 m at the supports, joined by a 0,25 m thick slab.

The girders are opened at the support region, having transverse beams at that region. The reinforced concrete bearings are 1,40 m diameter discs, 0,90 m thick. Those bearings perform like pendulums fixed to the deck by shear bars. At the abutments and column P1, P6 and P7 the bearings have also an hinged connection to the columns; at piers P2 to P5 those discs are rigidly fixed.



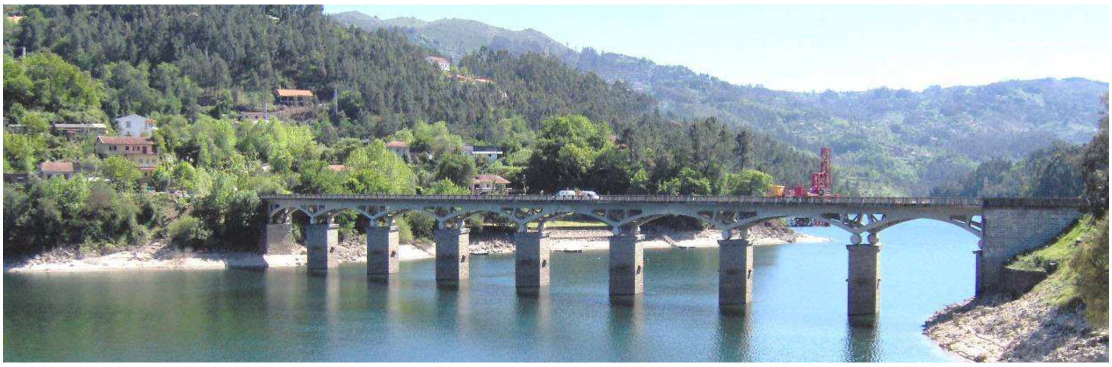


Fig. 4.1 - General view of the bridge

4.2. <u>Inspection and main anomalies</u>

A main inspection took place in May 2005, revealing the following anomalies:

- The main girders are systematically cracked in all spans at 5 main sections, as represented in Fig. 4.2
- Some local delamination occurs in regions of low cover. The cover presents a large scatter varying from 5 mm to 98 mm and the carbonation depth varies from 3mm to 47mm.
  Concrete has a good quality; its strength obtained in cores varies from 36 MPa to 58 MPa.

Fig 4.2 - Main structure cracking in the girder deck

4.3. <u>Structure assessment and interventions</u>

A 3D linear elastic model and a nonlinear analysis (Athena software) showed that, for the permanent loads, cracking in the referred 5 sections was to be expected (Fig. 4.3). The extent of the cracking was due to a misjudgement of local effects in the structure. Safety requirements were not also guarantee, thus a strengthening intervention was required.

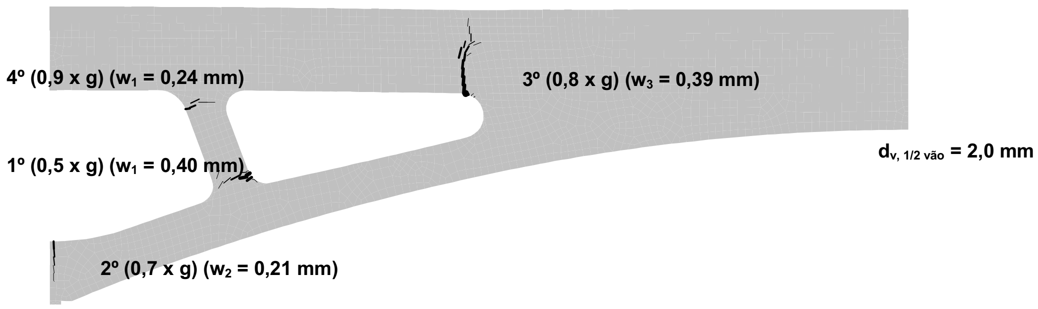


Fig. 4.3 - Non linear analysis. Cracking

An external prestressing was introduced at the top flange girder, with 2 x 7 strands per girder (Fig. 4.4). This is an active strengthening technique that presents several advantages. The ultimate capacity assessed with the nonlinear model increased from 1,35 PL + 0,5 LL to 1,35 PL + 2,0 LL. Together with this main intervention the general rehabilitation of the bridge is under progress.

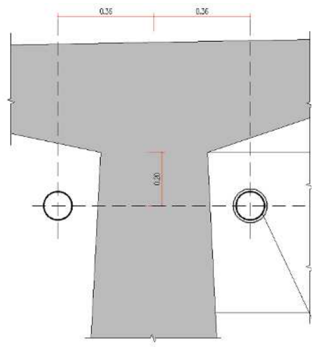


Fig. 4.4 - External prestressing

5. ALHANDRA VIADUCT

5.1. Description of the structure

The Alhandra viaduct on the A1 Motorway has a total length of 276 m and its construction ended in 1961.

Longitudinally the structure comprises a south abutment with 19,50 m, 16 span girders (14,05 m + 14 $\times$ 15,00 m + 14,01 m) and a north abutment with 18,10 m.

The width of the deck is 26 m with two continuous 0,15 m thick slabs connected at the middle by a central simply supported 0,12 m thick slab (Fig. 5.1).

The slab is supported by 2 × 9 precast prestressed girders with a 0,18 m × 1,00 m section, spaced 1,44 m. The slab is connected to the longitudinal beams by transverse prestressing. The longitudinal beams are also interconnected by prestressing cables to transverse beams located over the supports, at mid span and at quarters of span.

Each span is simply supported in a transverse beam connecting two columns with a Y shape and approximately 11m high. The two columns transfer their loads to cast in situ 2 × 6 piles 0,50 m diameter connected by a foundation beam.

The columns are hinged at the base (allowing the rotation associated with longitudinal horizontal displacements) and since the deck is also supported on the columns through hinges, the structure is only fixed at the north abutment, for longitudinal movements. The Y columns are connected to the top transverse beams by hinges that allow the relative transversal rotation. The rigid abutments include a grid slab deck and walls, supported on piles.

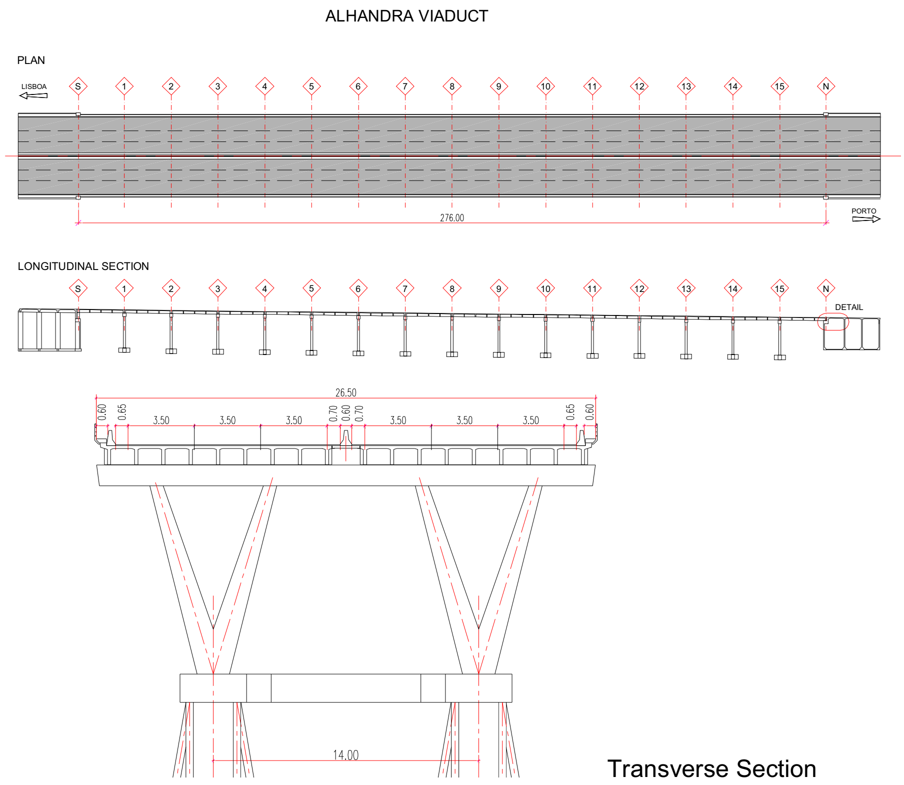


Fig. 5.1 - General dimensioning (dimensions in meters)

5.2. Inspection, main anomalies and rehabilitation

The inspection took place in 2000. The main anomalies were the following:

- Local reinforcement corrosion, mainly in the external beams and in low cover regions
  It is to be referred that the web reinforcement was placed outside the stirrups leading to horizontal cracking and corrosion of these horizontal bars (Fig. 5.2).

Fig 5.2 - Horizontal web reinforcement exposed due to corrosion

- Countless water penetrations from the deck creating deterioration areas under the deck (Fig. 5.3)

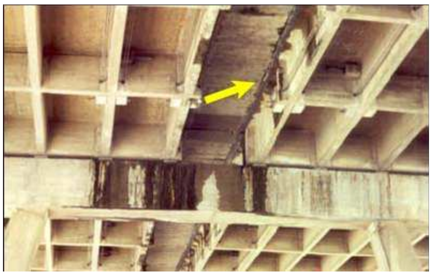


Fig 5.3 - Water penetration from the deck

- Deterioration of the anchorages and of the mono-strand plastic ducts of the external prestressing system (Fig. 5.4).
- Damages in concrete blocks serving as girders bearings.
- Deterioration of expansion joints

The rehabilitation works included the local repairing of all damages, a new concrete surface protection and the waterproofing of the deck central region where main leakage occurs.

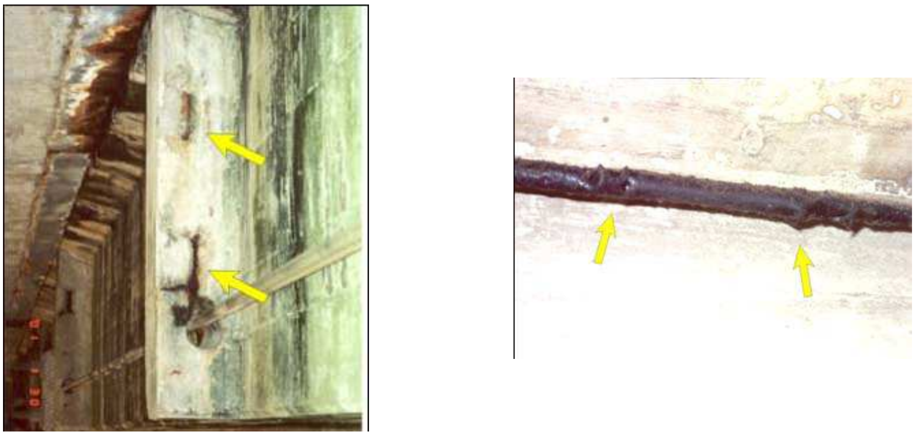


Fig. 5.4 - Damages in concrete anchorage regions and in the plastic duct of the mono-strands

5.3. Assessment of the seismic resistance of the existing structure

At the date of the design studies, no earthquake action was prescribed for viaducts in Portugal. For the earthquake action, acting in the longitudinal direction, the deck is fixed to the north abutment through 48 ϕ 32 mm shear bars, which were not designed for the earthquake action.

The total weight of the structure is 77473 kN. For an acceleration of a = 4.0m/s^{-2} the equivalent static horizontal force transferred to the abutment would be 30989 kN. The capacity of the shear connectors is only 48 $\times$ 60KN = 2880 kN.

Since the abutment and its foundation are not capable of sustaining the earthquake action of the viaduct and the shear connectors are not capable of transferring the earthquake action either, the need for an intervention was justified.

For the transverse direction, the earthquake resistance is provided by the columns and the transverse beams acting as frames.

The conceptual idea for this problem had the following goals:

- to affect as little as possible the existing structure
- to control the seismic action by introducing damping devices between the deck and a new supporting structure, adjacent to the abutments To achieve this, we had to eliminate the rigid connection of the deck to the north abutment and to introduce new sliding bearings between the deck and that abutment and to redesign the expansion joint. We also had to design a new supporting steel structure and its foundations. Axial continuity of the deck was also introduced

The intervention consists in the introduction of 4 viscous damping devices connecting the existing deck to a new steel structure, executed near the north abutment, supported by new pile foundations.

Fig. 5.5 shows the structure model of the deck – damper – new supporting structure.

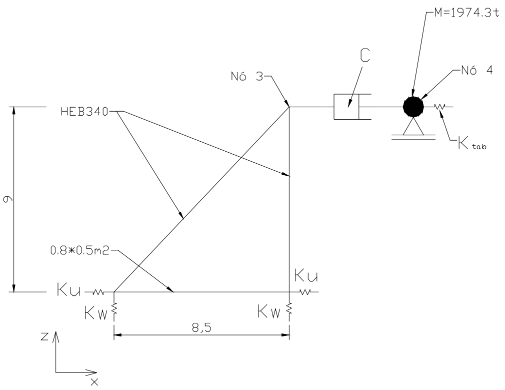


Fig. 5.5 - Structural model

The non linear time dependent analysis was performed considering a set of 10 artificial accelerogramms chosen to simulate the action of the design earthquake, as defined by the Portuguese Code and Eurocode 8.

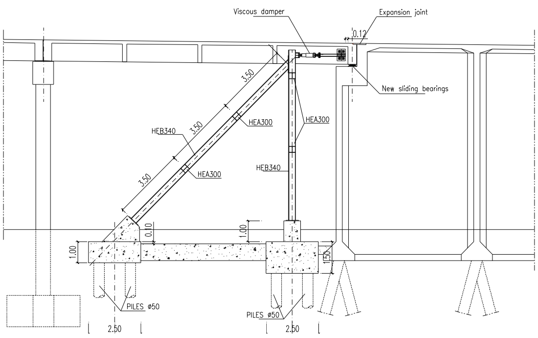
To choose the appropriate values of C and α, a parametric study was performed varying C from 250 to 1500KN/(s/m)$^{\alpha}$ and α = 0.1; 0.3.

From this study the following value were chosen:

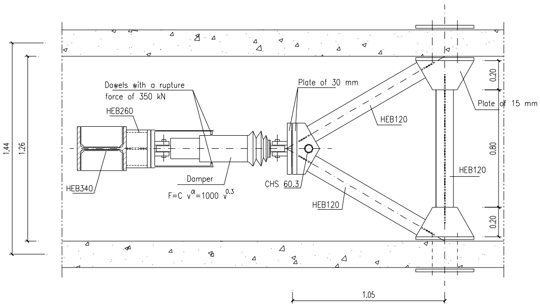
C = 1000KN/(m/s)$^{0.3}$

α = 0.3.

Due to the flexibility of the supporting system in this case, lower values of α were not as efficient as the one chosen. Fig. 5.6 shows the new steel supporting structure and the connection of the dampers to the deck.



Section A-A'



Plan - Detail of the force transmission system

Fig. 5.6 - Details of the new supporting structure and viscous dampers

6. ARRÁBIDA BRIDGE

6.1. Description of the structure

The Arrabida bridge, finished in 1963, is an arch bridge over the River Douro at Oporto, near the sea coast. The inspection was carried out in 1997 in order to promote its general rehabilitation.

The Arrabida Bridge has a total length of 493,20 m with an arch of 278,40 m span and 52,00 m height. The deck with 21,20 m spans has a total width of 25,00 m with 12 beams Fig. 6.1).

The beams have a variable depth from 1,80 m to 1,10 m and the slab between beams has a thickness of 0,18 m and spans of 2,00 m. The deck is supported through transverse beams in 4 rectangular columns 1,20 m x 1,20 m, in each support alignment. Over the river two twin arches 8,00 m wide with a double box section support the columns. The structure is full continuous between abutments.

Because the bridge has had no significant inspection or maintenance during its service life the Road Authority decided to promote a special detailed inspection and a detailed structure safety assessment according to the new codes and new service conditions, since the number of lanes were increased from 4 to 6.

6.2. Inspection

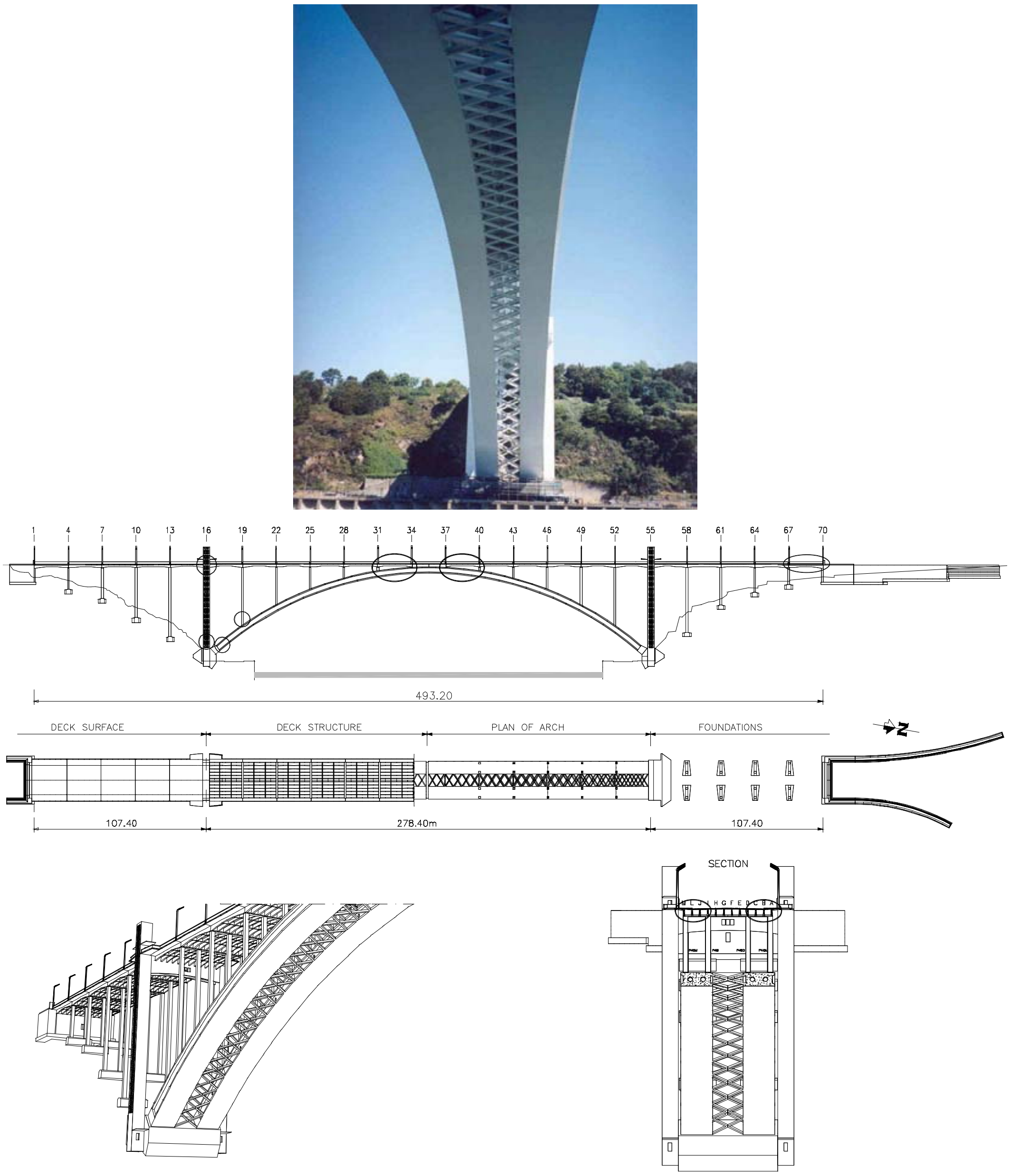
A preliminary visual inspection was carried out in order to establish the objectives, to plan the inspection and define the type, number and location of the tests. (Fig. 6.1). The selected zones for testing include areas in the upstream and downstream sections and in the various types of structure elements: beam and slab deck, columns and arches.

One main observation was the extensive delamination, spalling and reinforcement exposure in many beams over the arch, mainly concentrated in the soft corners or near construction joints. Other areas with low cover or concrete defects showed significant corrosion of the reinforcement. The surface paint almost disappeared over the river, the most exposed surfaces of the bridge.

Figure 6.2 shows a deterioration mapping and photos illustrating some of the visible defects.

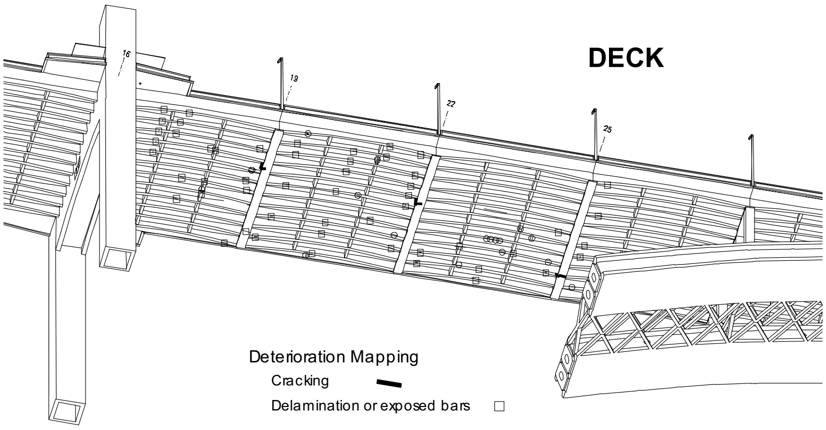
The variability of the concrete properties, cover and penetration processes are very important to access the deterioration process. Figure 6.3 shows the variability of the cover measurements in the main girders of Arrabida Bridge.

The assessment of the chloride penetration is illustrated in Fig. 6.4, where the chloride concentration at different levels is presented. These results were obtained after 35 years of service and from samples taken in the construction joints, the location where deterioration is more relevant. It is also important to stress the fact that these results will vary from element to element due to the heterogeneity of concrete, eventual use of different mixes, different compaction and curing, different exposure conditions, ...



Selected areas for testing

Fig 6.1 - Arrabida Bridge dimensioning and selected areas for testing



Exposed bars in construction joints and low cover regions

Fig. 6.2 - Visual inspection. Main pathologies

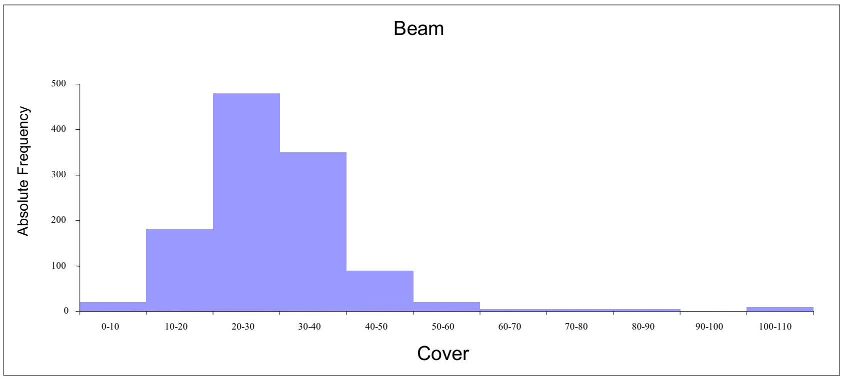
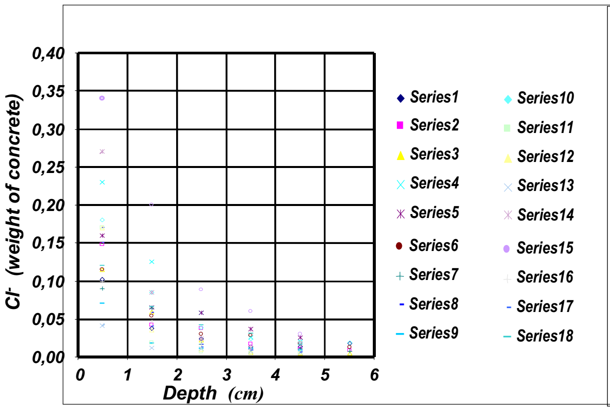


Fig. 6.3 - Variation of cover in the main girders



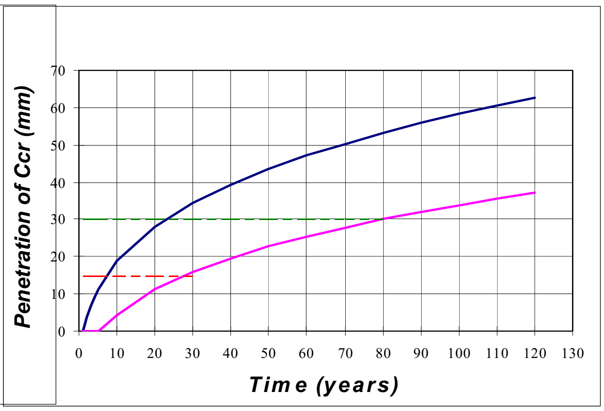


Fig. 6.4a - Chloride penetration near construction joints in the girders

Fig. 6.4b - Modelling of chloride penetration

From these results and assuming a diffusion of chlorides governed by the modified Fick's law and considering the variation in time of the diffusion parameters it is possible to obtain an interpretation of the penetration process and a prediction of its evolution. In Fig.6.4b the relation between the time needed for the chloride content to achieve a level of 0.05 % by weight of concrete is presented. The envelope between the two curves reflects the variability of the critical chloride penetration.

From these curves it is possible to understand that for the average cover depth in the girders of 27 mm the time needed for the critical chloride content to be reached at this level varies between 20 and 70 years. Then depassivation occurs, corrosion may start and the corrosion rate variation can be assessed.

It is important to refer that in areas other than those of the construction joints or concreting defects, the chloride contamination was still not relevant due to the good quality of the concrete. This is thus an example of the importance of good execution for durability of concrete structures.

6.3. Structure safety assessment

The codes used at the time of the design dated from 1935 and safety criteria for reinforced concrete were based in stress allowable limits for steel and concrete which were compared with acting stresses for service loads calculated in the cracked state, when appropriate.

Due to the relevance of this bridge, the Designer and Road Authority decided to adopt live loads much higher than those of the existing codes (similar to those of the present EN 1991-2). Even the earthquake action was considered (a = 0.5 ms^{-2}) although Oporto, at the time, was not considered in the codes a region with seismic risks.

Analytical models and experimental tests in scaled models were done by the designer for this high hyperstatic structure with a complex geometry. At that time the elastic analysis of the full 3D structure would not be an easy (or possible) task, the scaled models were very accurate (in the elastic regime) and Edgar Cardoso was an expert in this area. However, very detailed analytical models were also done for the deck (influence surfaces for the slab and influence lines for the beams). The results from analytical and experimental models were compared and adopted, for dimensioning, the most unfavourable ones.

In the context of the rehabilitation project safety was re-evaluated and it was concluded that the actual safety requirements are complied with. The single remark refers to the forces obtained in the short columns over the arch, which in the design were not properly assessed.

In fact, full continuity between these columns, the deck and the arch was introduced in the construction. However, in a 3D model the effect of the arch deformability generates (even in an elastic model) high bending moments in these columns. A main cost of this intervention was the execution of various movable platforms. A monitoring scheme was introduced in the bridge, namely to access the evolution of the resistivity of concrete.

7. BRIDGE OF FIGUEIRA DA FOZ

7.1. Description of the structure

The Figueira da Foz Bridge over the River Mondego (Fig. 7.1) has a total length of 1421 m, including a 405 m long cable stayed bridge and two approach viaducts with 630 m (left bank) and 315 m (right bank). The bridge was built in 1982.

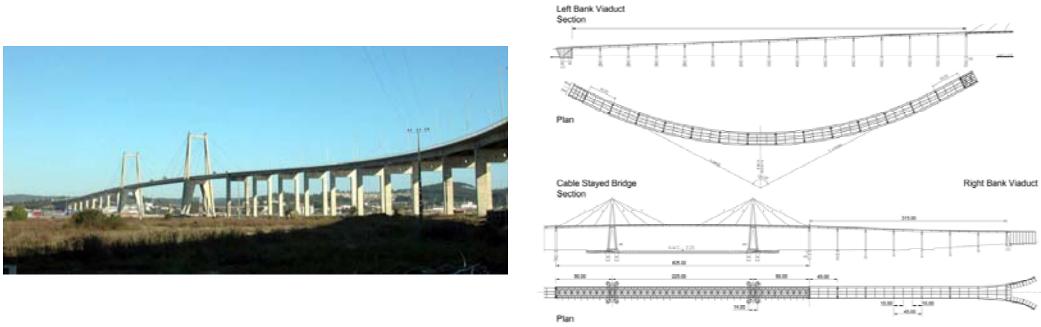
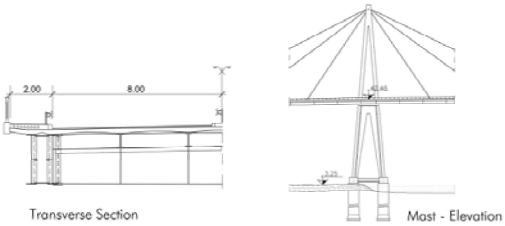


Fig. 7.1 - General view and dimensioning of the bridge

The cable stayed bridge has a central span of 225 m and lateral spans of 90 m. The 85 m above the water level masts include four hollow concrete inclined elements. The stays, spaced 30 m at the deck, are made of galvanised wires passing through saddles at the top of the masts. The steel deck has two main beams, where the cable stays are fixed (each one made of two 2 m high I beams) interconnected by transverse beams spaced 10 m. These transverse beams support longitudinal girders spaced 8,20 m which in turn support a reinforced concrete slab with a variable thickness of 0,13 m to 0,20 m (Fig. 7.2).



Deck cross section

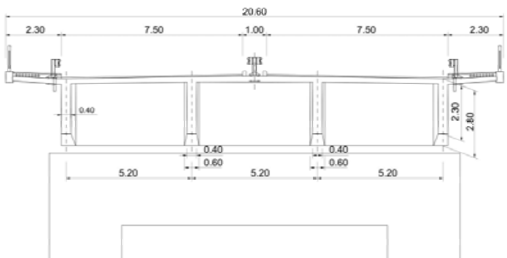


Fig. 7.2 - Dimensioning of the cable stayed bridge

Fig. 7.3 - Transverse section of the approach viaduct

The deck of the approach viaducts have a slab 0,18 m (span) to 0,22 m (over the girders) thick supported by 4 longitudinal beams, spaced 5,20 m with 45 m spans. The prestressed concrete girders have a section of 0,40 m $\times$ 2,00 m at mid span and 0,60 m $\times$ 2,50 m at the supports. The slab is prestressed in the transverse direction (Fig. 7.3). The deck is continuous for each viaduct. The girders are fixed to the transverse beams of the columns by dowels and plumb bearings. Only in the transition pier to the cable stayed bridge the support of the deck allows relative horizontal movements. The longitudinal beams are also interconnected by transverse beams spaced 15 m. Each support alignment has 2 hollow rectangular columns 3 m $\times$ 1,60 m connected at the top by a hollow rectangular beam 4,00 m × 1,60 m, with a thickness of 0,25 m.(Fig 7.3). The abutments are apparent. The deck is fixed to the abutments.

7.2. Inspection and Assessment

A detailed inspection and assessment of the safety of the structures according to the new codes showed that the bridge and the viaduct had suffered significant deterioration and that a seismic strengthening was also required.

The main conclusions were the following:

- The quality of execution was low. Concreting defects such as voids in prestressed concrete beams and low reinforcement cover were frequent. The painting of the steel structures was also of poor quality (Fig 7.4).

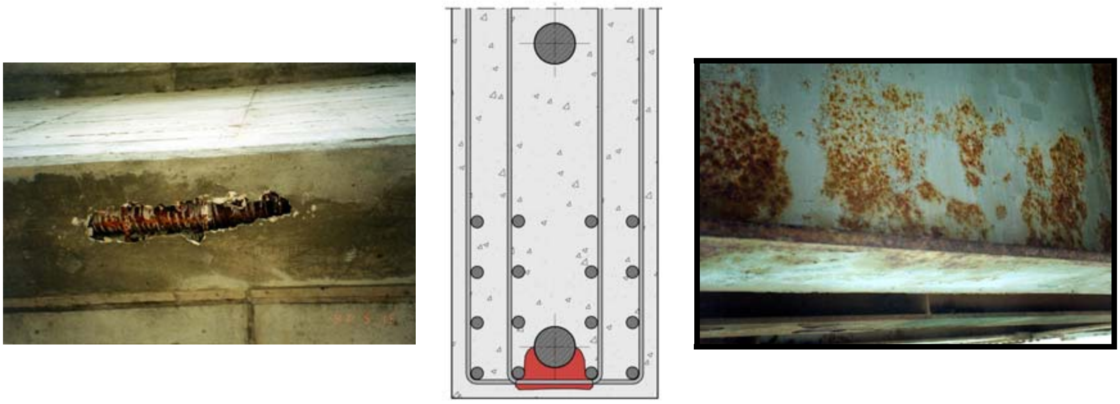


Fig 7.4 - Concrete defects in prestressed girders and corrosion in main steel beams of the cable stayed bridge

- Due to the quality of the concrete and to the defective execution, reinforcement corrosion represented a significant anomaly.
- Locally alcali-silica reactions and sulphate attacks were identified (mainly in the foundations of the towers)
- In the approach viaducts the prestressed longitudinal girders were not conceived to avoid cracking in service conditions and cracks of 0,2-0,3 mm were frequent at mid span.
- In the cable stayed bridge there is a permanent tensile reaction at the transition piers, requiring a redundancy of the anchorage system for safety and reliability.

The rehabilitation of such a construction requires proper planning and the introduction of working platforms (Fig. 7.5) which represent a significant cost of the works.

Fig. 7.5 - General view of the bridge under repair and working platforms

The general rehabilitation of the cable stayed structure included the saddles, the repairing and addition of a new full protection of the steel elements as well as the local repairing and concrete surface protection of the masts (Fig. 7.6).

Fig. 7.6 - General rehabilitation of the saddles

The general rehabilitation of the prestressed concrete approach viaducts included local repairing and a concrete surface protection.

When local repairing involved a thickness greater than 6 cm, micro concrete was used (obtained from a mixture of 70% weigh of pre-packed grout with 30% of weight of 9 mm aggregates).

7.3. Assessment and Strengthening

For the assessment of the seismic resistance of the viaducts, a three dimensional model was used. This model showed that the design acting forces in shorter columns exceeded its strength capacity.

The connection of the deck to the abutment presented two problems. The dowel bars do not have the required strength capacity and the connection to the beam is also not robust enough to guarantee the transfer of the seismic action to the abutment. The estimated force transferred to the abutments is 11,268 kN and the strength capacity of 6 Ø 40 dowels per beam leads to an estimated resistance of 4×605 kN = 2420 kN.

It is to be referred that an experimental evaluation of the frequencies was done in the viaduct in order to access its behaviour and accuracy of the analytical model.

The conceptual idea was then to control the force transferred between the deck and the abutment through the use of viscous damping devices. With this concept not only the problem of the connection between the deck and the abutment was solved but also the seismic effects in the columns were reduced.

To introduce the viscous dampers (one in each of the four beams) the structure of the abutment had to be modified to accommodate the dampers and new sliding bearings were introduced between the girders and the abutment (Fig. 7.7).

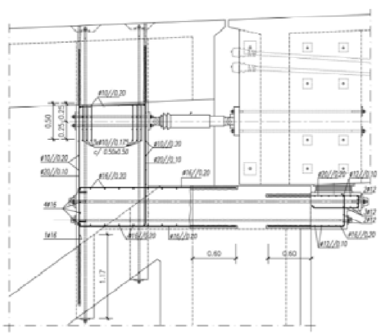


Fig. 7.7 - Details of the changes in the abutment to introduce the dampers

A three dimensional non linear time dependent dynamic analysis was performed in two orthogonal directions considering a set of 10 artificial accelogramms chosen to simulate the design earthquake action as defined by the Portuguese Code and Eurocode 8 for Seismic Region C, Soil type II and Earthquake Type 2.

From the results we chose the characteristics of the damper $F = 1500\ v^{0.1}$ and we can emphasize the gain we obtained with the use of these devices:

- Yielding at the columns for the design earthquake was avoided.
- The displacement at the abutment was reduced from 79,90 mm to 28,90 mm and at the transition pier from 88,60 mm to 35,90 mm, when compared to a solution of free longitudinal displacement at the abutment.

For the cable stayed bridge the main interventions were the following:

- Strengthening of the transverse top beam of the towers for the seismic action by adding an external prestressing system (Fig. 7.8);

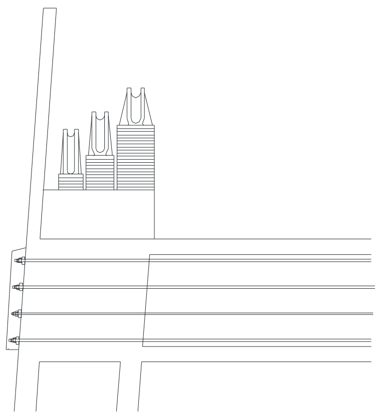


Fig. 7.8 - Strengthening of the transverse top beam of the towers

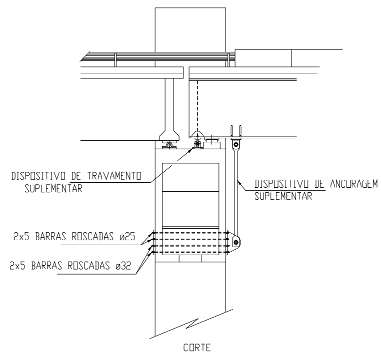


Fig. 7.9 - Replacing and strengthening the anchorage system of the deck to the transition piers

- Replacement and strengthening of the anchorage system of the deck to the transition piers. The initial conceptual design led to the transfer of tension forces from the deck to the transition pier which was guaranteed by prestressing bars .This system was duplicated in this intervention (Fig. 7.9);

For the approach viaducts the strengthening of the main girders by external prestressing was also introduced (Fig. 7.10). This work was required due to the fact that the live load adopted in the design were lower than the recent code values as well as to the damages observed in the beams (cracking, concrete defects).

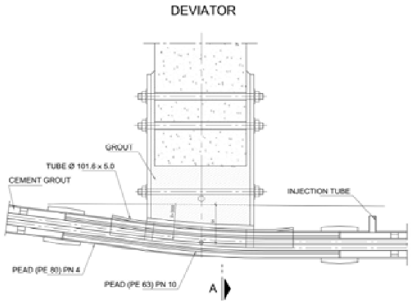
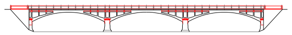
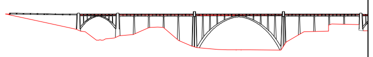
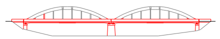
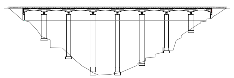
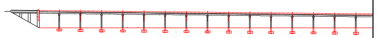
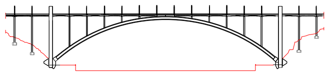
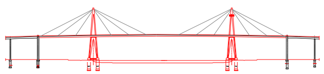



Fig. 7.10 - External prestressing of the approach viaducts and details

8. SYNTHESIS AND MAIN CONCLUSIONS

The following table presents a synthesis of the characteristics, main anomalies and type of interventions in the 7 bridges described in the paper. The following conclusions can be pointed out:

- Although the maintenance policy was almost inexistent concrete bridges behave quite well in an average of 50 years of service.
- The main deterioration problems are reinforcement corrosion due to an enormous scatter of reinforcement cover, concrete defects and deficient construction details.
- Alcalis-silica expansive reaction is also a relevant cause of deterioration.
- Most bridges do not comply with the new seismic design requirements.
- A general rehabilitation cost of the bridges varying from 400 to 800 euros/m² of deck represents 30% to 50 % of the cost of a new structure.

Name	Type	Dimensions L-total length (m) ℓ-max span (m) A-deck area (m²)	Date of Construction Type of Exposure	Date of Inspection Date of Rehabilitation	Main Anomalies	Structure Assessment	Rehabilitation Works	Strengthening Works	Cost €/m² area deck referred to 2008
Chaminé Bridge Albufeira de Montargil	Reinforced Concrete - 3 Arches 	L = 81,60 ℓ = 27,20 A = 550	1934 XC	1994	Lack of maintenance, typical anomalies	No problems	General rehabilitation	Associated with the deck width increase from 6.75 m to 10.5 m	49.000 € 755 €/m²
Duarte Pacheco Viaduct on Alcântara valley in Lisbon	Reinforced Concrete Arches and Viaducts 	L = 355,10 ℓ = 90 A = 8522	1944 XC	1994 2001	Alcalis-silica reaction Cracking and deck expansion	External girders in need of strengthening. Seismic resistance insufficient	Local repairs of concrete, crack injection, general protection of surface	External girders strengthened with CFR laminates Seismic upgrade	5.000.000 € 586 €/m²
Arcos Bridge River Sado	Reinforced Concrete - Bowstring 	L = 66 ℓ = 31,50 A = 600	1944 XC	2005 2008	Lack of maintenance	Seismic resistance insufficient	General rehabilitation	Base isolation for earthquake resistance	392.200 € 653 €/m²
Cavado Bridge Albufeira da Caniçada	Reinforced Concrete Continuous Girder 	L = 176 ℓ = 23 A = 1408	1954 XC	2005 2008	Extensive and systematic structural cracking of main deck girders	Service and ULS safety not guaranteed	General rehabilitation	External prestressing of main girders and local strengthening	1.117.500 € 790 €/m²
Alhandra Viaduct A1-Lisboa-Porto	Prestressed Concrete precast girder deck 	L = 276 ℓ = 15 A = 7176	1961 XC	2000 2003	Extensive local corrosion	Seismic resistance insufficient	Local repairing. Water proofing of part of the deck	Earthquake retrofitting with viscous dampers and introduction of deck continuity	968.000 € 234 €/m²
Arrábida Bridge over the Douro River- in Oporto	Reinforced Concrete Arches 	L = 493 ℓ = 270 A = 12500	1963 XS1, XC	1997 2002	Local corrosion of concrete defective regions due to chlorides	No problems	Local repairs. New protection of surface Monitoring	None	5.000.000 € 400 €/m²
Figueira da Foz Bridge over Mondego River	Cable Stayed Bridge with Steel Deck  Prestressed Concrete Approach Viaducts 	L = 405 ℓ = 225 A = 8340 L = 945 ℓ = 45 A = 19470	1982 XC XS	1997 2005	Steel deck with extensive corrosion. Local corrosion of reinforced concrete. Cracking and construction defects	Seismic resistance insufficient. Deck of approach viaducts requiring strengthening	Local repairing General surface protection of steel and concrete elements	Strengthening of the masts. External prestressing of deck of approach viaducts. Seismic upgrade with viscous dampers	9.000.000 € 324 €/m²