ABSTRACT: The Figueira da Foz Bridge includes a 405 m long cable stayed bridge with a steel deck and a main span of 225 m and two prestressed concrete approach viaducts with 45 m spans and a total length of 945 m. The rehabilitation included the strengthening of the towers and the replacement of the anchorage system of the deck of the cable stayed bridge to the transition piers. The rehabilitation of the approach viaducts included the strengthening of the deck main girders by external prestressing and the introduction of new dissipating devices between the deck and the abutments. The general rehabilitation also included local repairing and new surface protection of steel and concrete elements.

1 INTRODUCTION

The Figueira da Foz Bridge over the River Mondego (Fig. 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 designed by Prof. Edgar Cardoso and built in 1982. The bridge was subjected to a general rehabilitation and strengthening.

Figure 1. General view and dimensioning of the bridge

2 DESCRIPTION OF THE STRUCTURE

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 deck is a steel construction with 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 trans-
verse 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. 2).

![Deck cross section](image)

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 × 2 m at mid span and 0.60 m × 2.50 m at the supports. The slab is prestressed in the transverse direction (Fig. 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 × 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. The abutments are apparent. The deck is fixed to the abutment.

3 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.
- Due to the quality of the concrete and to the defective execution, reinforcement corrosion represented a significant anomaly.
- Locally alcalis silica reactions and sulphate attacks were identified (mainly in the foundations of the tower)
- 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.
- 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 seismic design did not guarantee the seismic resistance required by the new codes and for the approach viaducts the decks were fixed to the abutments, generating considerable seismic forces.

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 per beam is estimated in 4 m × 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 vibration modes have frequencies of 0.737 Hz (longitudinal – 79.4 %) and of 0.941 Hz (36.4 % – transversal); 1.562 Hz (23.6% – transversal) (Fig. 4).

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

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. The model includes 418 node, 408 bars and 164 shell elements.

In the columns a non linear moment-curvature relation was considered adopting $EI_{\text{effect}} = 0.08 EI_{\text{uncracked}} + M_y/X_y$ where $M_y$ and $X_y$ are calculated for the axial load associated with the permanent loads. The ageing effect was considered in the values considered for $E(t)$, $f(t)$. The viscous damper was defined by its constitutive law: $F = c v^\alpha$. A viscous damper was considered in each of the four beams. The design parameters were obtained from a parametric study varying $C = 1000; 1500 \text{kN/(s/m)}^{\alpha}; \alpha = 0.1; 0.2$.

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.
- The force transferred to the abutment was reduced from 11,268 kN (rigid connection of the deck to the abutment) to $4.00 \times 1206 = 4824 \text{kN}$. 

![Figure 4. Vibration 1st, 2nd Modes in the 3D Model](image-url)
4 REHABILITATION WORKS

The rehabilitation of such a construction requires proper planning and the introduction of working platforms (Fig. 5) which represent a significant cost of the works.

Figure 5. General view of the bridge under repair and working platforms

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

a) Strengthening of the transverse top beam of the towers for the seismic action by adding an external prestressing system (Fig. 6);

Figure 6. Strengthening of the transverse top beam of the towers

b) Replacing and strengthening 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 (Fig. 7);
c) General rehabilitation of the structure, including 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. 8).

For the approach viaducts the main interventions were the following:

a) Strengthening of the main girders by external prestressing (Fig. 9). This work was due to the fact that the live load adopted in the design was lower than the recent code values as well as to the damages observed in the beams (cracking, concrete defects).
The prestressing system, by VSL, included 4 cables for each beam (with a total of 16 strands for each internal beam and 14 strands for each external beam). A trapezoidal layout was adopted with deviators under the two existing transverse beams in each span. The strands are located inside a Ø 75 mm PEAD duct which was injected with cement grout. A double PEAD duct was used at the deviators. Near the active anchorage a free unbounded length was provided to enable the cable replacement (on that length the strands are protected by individual ducts and grease).

At the deviators and at the anchorages the PEAD duct is inserted in a Ø 88.90 mm steel pipe (Fig. 9).

The deviators include a Ø 101.60 mm steel curved plate connected to a steel box fixed to the main girders and filled with grout to provide a rigid support to the cable.

A prototype test was done to check the behaviour of the deviators and of the cables. The anchorages are standard VSL type EC 6-4 active anchorages with galvanised steel caps. The anchorages are fixed to anchor concrete blocks which are fixed to the transverse beams and deck (Fig. 9).

The cables were stressed at 65% of its ultimate strength to achieve the effective prestressing force of 2480 kN for the internal beams and of 2170 kN for the external beams.

b) Introduction of new dissipating devices between the deck and the abutments. 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. 10).
c) General rehabilitation of the concrete structures, including local repairing and a concrete surface protection (Fig. 11).
When local repairing involved a thickness greater than 6 cm, micro concrete was used (obtained from a mixture of 70% of pre packed grout with 30% of weight of 9 mm aggregates).
Fig. 11 also shows the repairing of the towers of the bridge affected by sulphate attack.

5 GENERAL INFORMATION

Owner: Estradas de Portugal E.P.E.
Design: Edgar Cardoso
Rehabilitation Design: Armando Rito / Julio Appleton
Contractor: Soares da Costa, SA

Prestressing steel: 76.2 ton
Surface protection
Steel structure: 22,500 m²
Concrete: 93,500 m²
Concrete repair: 11,300 m²
Reinforcing steel A500: 35 ton
Replacement of steel elements: 25 ton
Total cost: 9,000,000 €
Service date after rehabilitation: June 2005

The cost of the intervention was about 35% of that of the construction of a new bridge.
Void under a cable

Repairing of the prestressed beams of the viaduct

Repairing of the foundations blocks of the bridge tower

Figure 11. Deterioration and repairing of concrete structures