THE STAY-CABLE SYSTEM OF RION-ANTIRION BRIDGE

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1. ABSTRACT

The stay cable system selected for Rion-Antirion Bridge is made of parallel seven-wire strands. It satisfies very stringent requirements in terms of corrosion protection, fatigue performance, easy of installation, maintenance and replaceability (with traffic). Special accessories have been implemented on the cable system in order to ensure elastic behaviour during the specified earthquake. It includes wedge blocking devices in case that the cable is unloaded and bending guiding tubes at the anchorage to control the curvature of the cable. In addition, conservatory measures have been taken for possible easy future installation of dampers and secondary crossties, in case that the cables experience any vibration. Moreover, the unilateral behaviour of the cable-elements during earthquakes is also taken into account in a comparative seismic analysis. Recordings of the structural monitoring system and field measurements on the cables and deck, are also presented.

2. INTRODUCTION

The Rion-Antirion Bridge in Greece was open to traffic on August 8, 2004. It is a multispans cable-stayed bridge, 2252 m long with three main spans of 560 m and two side spans of 286 m (See Fig. 1).

Figure 1 Rion-Antirion Bridge

The deck is a composite structure with a width of 27 m, carrying two traffic lanes in each direction. It is continuous and fully suspended for its total length of 2252 meters by means of 368 stay cables, arranged in a semi-fan shape in two inclined planes. The cables at deck level are spaced at a distance of 12.2 m apart.

According to the Greek Code EAK2000, the bridge is located in a seismic zone III. It has been designed to withstand an accidental ultimate limit state earthquake of 0.48g peak ground acceleration (p.g.a.) combined with tectonic movements of up to 2.0 m in any direction between two consecutive pylons. The stay-cable technology has been adapted to meet the severe seismic requirements of the project.

Like any major structure, the bridge is equipped with a continuous monitoring system in order to follow the daily performance of the bridge, as well as the behaviour of the bridge during special events, such as earthquake and strong winds. The monitoring system was used to investigate the condition of the bridge, after a cable failed on January 2005.

3. STAY CABLE SYSTEM

The specifications of the project called for a stay cable system capable to satisfy high level requirements in terms of corrosion protection, fatigue performance, ease of installation, maintenance and replaceability. It is worth mentioning that the bridge has been designed to allow traffic during cable replacement, and to withstand an accidental failure of two consecutive stay cables. Special requirements of the system included wind vibration stability and elastic behavior during the design earthquake.

3.1 Description

The stay cable system commonly used for major bridges since the late 1980’s is formed of a bundle of parallel seven wires strands placed inside a high density polyethylene (HDPE) duct. For the Rion-Antirion Bridge the Freyssinet HD system was selected.

To satisfy the corrosion requirements all wires are hot dip galvanized. Each strand is individually protected with a petroleum wax that fills the inter-wire voids and a HDPE extruded sheath. Finally, the bundle of monostands is contained within an external HDPE pipe, 6mm thick. The outer white color of the pipe, which is obtained
from special mineral additives, reduces the thermal effects on the stays, ensures long term color stability and provides efficient protection against ultraviolet rays.

Figure 2: Freysinet HD cable stay with active anchorage

For the Rion-Antirion Bridge, the stay cables consist of 43 to 75 “seven-wire strands”, 15mm nominal diameter (with a cross section of 150 mm²) each and class Y177057+Z-16-B, according to French regulation SFA 35-035 (Guaranteed Ultimate Tensile Strength of 1770 MPa).

According to the contract design assumptions and specifications, the permissible stresses on the stay cables for the different limit states are as follows:

- Serviceability limit state: $0.50 \, F_{Ed}$
- Construction limit state: $0.60 \, F_{Ed}$
- Ultimate limit state including specified earthquake load combination: $0.70 \, F_{Ed}$

- Fatigue limit state:
  - for strand material: $\Delta \sigma = 300 \, MPa$ for $2 \times 10^6$ cycles
  - for mono-strand with wedges: $\Delta \sigma = 200 \, MPa$ for $2 \times 10^6$ cycles
  - for complete cable system: $\Delta \sigma = 160 \, MPa$ for $2 \times 10^6$ cycles

The connection of the cable to the pylon and deck is made with an active and passive anchorage. Each strand is individually anchored in a high-grade steel anchor block with high fatigue performance wedges resisting stress amplitude of 200 MPa combined with an angular deflection of 0.5 rad, for a minimum of 2 million cycles with no wire or strand breakage. The deck anchorage is passive and fixed. It is connected to a steel gusset plate welded to the main girder web. This gusset was especially designed to meet fatigue and seismic requirements. The pylon anchorage is active stressing end and can be adjusted. The stay adjustment is obtained from a threaded tube and a nut by de-tensioning the anchor head using a huge hydraulic jack.

3.2 Vibration mitigation

The vibration of cables under wind action is obviously the most common aerodynamic problem related to the modern cable stayed bridges. The main reason is the very low internal damping level of the vibration modes of the cables, due to the high tension. This low damping, combined with numerous low frequencies of vibration modes result in a potential wind sensitive structure. The longer the cable, the smaller are the frequency of vibration and the structural damping. Therefore, the longer cables are more sensitive to wind vibrations. The stay cables of Rion-Antirion Bridge have lengths ranging from 77 m to 293 m, with an estimated structural damping $\zeta = 0.04$ to the critical of 0.19% to 0.23%, and 1st modes of vibration between 1.0 Hz to around 0.44 Hz, respectively.

The main sources of stay cable vibrations are:

1. Rain and wind induced vibrations, where water rivulets flowing along the stay alter the circular profile of the cable and create instability.
2. Buffeting, i.e., the direct action of gusty wind on the cable.
3. Vortex shedding, i.e., the oscillating forces arising from Von Karman vortices.
4. Parametric excitation, i.e., forced vibration of the cable, induced by the vibration of the anchorages.

The following mitigation measures were considered:

- The external surface of the HDPE is provided with a helical profile that disorganizes the formation of rivulets and, therefore, prevents rain and wind induced vibration. For the Rion-Antirion Bridge, the shape of the helix was derived experimentally with cable vibration tests performed at a wind tunnel.
  - Increase the intrinsic damping, $\zeta$, above 0.5%, using dampers. Depending on the length and the inclination of the cable, Freysinet has developed two types of viscous dampers:
    - Internal hydraulic damper (IHD) located inside a tub rigidly connected to the steel gusset of the bottom anchorage (see Fig. 3a).
    - External hydraulic damper (EHD), placed a few meters away from the bottom anchorage and connected directly to the deck (see Fig. 3b).

  For the Rion-Antirion Bridge all measures for future easy installation of IHD for cables #1 through #10, and EHD for the longer cables #11 to #25 have been taken.
- Provide cross-damping-ties (also called ‘aiguilles’), to prevent parametric excitation by offsetting the stay cable modal frequencies. Although from the analytical aerodynamic study it was concluded that cross-ties are not required, special sleeves have been placed at one third length locations along the cable, allowing future installation of ‘aiguilles’.

Figure 3: (a) CAD view of internal hydraulic damper and guide tube (b) External hydraulic damper on Normandy Bridge
3.3 Seismic protection

The performance-based design of the structure for the accidental ultimate limit state of the specified earthquake load combination HE (0.48g + 50% tectonic movements) consisted of a "control damage" at the following locations: sliding and partial uplift at the foundation raft-soil interface, dissipation of energy by viscous dampers connecting the deck to the pylon and potential formation of plastic hinges at pylons legs. The dynamic response of the structure was estimated with artificial and natural accelerograms matching the design spectrum. This analysis took into account large displacements, hysteretic behaviour of materials, non-linear viscous behaviour of the dissipation system, sliding and uplifting elements at the raft-soil interface and a realistic model for the soil-structure interaction.

For the specified earthquake load combination, the stay cables should maintain their elastic behaviour without any damage at any part of the system (permissible normal stress of 0.70 Fy, ksi). In order to study the dynamic performance of the cable system, a time-history analysis of several cables was carried out by Vinci Construction Design Office using ANSYS software. Each cable was modelled with finite elements and distributed masses. The load input consisted of time histories of displacements, derived from the global time-history analysis of the bridge, imposed at bottom and top anchorages of the cables. The anchorage displacements and the dynamic response of the stays result in axial tension variations combined with simultaneous angular deflections of the cable close to the anchorages. Therefore, a design criterion was necessary in order to combine the axial tension with the secondary bending stresses. Since, inelastic behaviour of the cable is not permitted, the following criterion was adopted:

\[ \sigma_t < \sigma_{xt} \leq f \]

Where,

- \( \sigma_t \) is the normal stress resulting from axial tension
- \( \sigma_{xt} \) is the normal stress resulting from secondary bending
- \( f \) is the 0.1% yield stress of the cable (1593 MPa for class 1770 MPa).

In order to satisfy the above requirement a special device capable of guiding the cable deviation, was developed by Freyssinet and installed at both anchorages (top and bottom). They are made in two parts from polyurethane material with steel inserts, with a curvature of 4 m. The secondary bending stresses are therefore, limited to 250 MPa. These devices, called parametric deviators, which are shown in Figure 4, had to be compatible with other functions of the stay system such as continuity of the corrosion protection, durability of the deviator, ease of cable installation, individual behavior of each monostay, proper functioning of IHD damper (in case it is installed in the future).

![Figure 4 CAD drawing of 'parametric deviator']

In addition, the global time-history dynamic analysis of the bridge revealed that during the specified earthquake, there are instances of possible de-stressing of the short cables. To avoid any de-tensioning of the individually anchored strands, a special blocking device has been developed by Freyssinet and installed at the anchor head that holds all individual wedges of the monostays.

4. STRUCTURAL MONITORING

The structural behavior of the bridge, especially for wind and earthquake actions, is monitored with a real-time structural monitoring system and with a geometrical control of the profile of the deck performed upon request. In addition, a calibrated 3-D model of the bridge is used to analytically confirm the recordings from the monitoring system and the geometrical control surveys. Table 1 summarizes the type and quantity of the sensors of the monitoring system. In total 110 sensors were installed, with a total number of 296 channels recorded by four data acquisition systems placed in the four pylons.

<table>
<thead>
<tr>
<th>Type of sensors</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road temperature</td>
<td>4</td>
</tr>
<tr>
<td>Metro station</td>
<td>2</td>
</tr>
<tr>
<td>Load cell on toe restraint</td>
<td>4</td>
</tr>
<tr>
<td>3D accelerometer on deck for wind and earthquake</td>
<td>13</td>
</tr>
<tr>
<td>3D accelerometer on deck for wind</td>
<td>3</td>
</tr>
<tr>
<td>3D accelerometer on pylon for earthquake</td>
<td>12</td>
</tr>
<tr>
<td>3D accelerometer for earthquake ground characterization</td>
<td>2</td>
</tr>
<tr>
<td>Joint movement measurements</td>
<td>2</td>
</tr>
<tr>
<td>Water detection</td>
<td>4</td>
</tr>
<tr>
<td>3D accelerometers on stay cables</td>
<td>16</td>
</tr>
<tr>
<td>Load cells on monostays</td>
<td>16</td>
</tr>
<tr>
<td>LVDTs on lower cable stay anchorages (during construction only)</td>
<td>16</td>
</tr>
<tr>
<td>Strain gages on gussets (during construction only)</td>
<td>16</td>
</tr>
</tbody>
</table>

More specifically, 16 stay cables have been permanently instrumented with 3D accelerometers and load cell on one strand at the upper anchorage, in order to follow their static and dynamic behavior.
4.1 Case study

On January 27, 2005, the South-West outer stay cable of pylon M1 (C1S23W), having a length of 292.6m, caught fire and broke just above the upper collar (93.4m below the top anchorage). The adjacent cable (C1S22W) appeared to have also been damaged.

The recordings from the monitoring system and the geometrical control check were used in order to evaluate the damages and check the structural integrity of the bridge, so that the bridge can be re-opened to traffic during the cable replacement period.

Records from Structural Monitoring System

From the structural monitoring that was in place during the event some important information could be found (see Figure 5).

- During the event, the south expansion joint showed a progressive closure of 20mm.
- From the load cell on cable C1S18W we could see a progressive transfer of load from 93 kN to 94.1 kN (with a sudden transfer at the moment of cable failure).
- From the load cell C1N23E we could see a progressive unloading of the cable (from 85.5 kN to 84 kN).

A possible explanation is the following: As fire progressed, individual strands started to lose their strength, reducing the overall tension capacity of cable and increasing the amplitude of vibration. The load is progressively transferred to the other stays (i.e. instrumented stay #18) and to the transition pier. The total compressive load (from the horizontal component of the cable force) of the south side of M1 cantilever decreases creating an unbalanced load between south and north of M1, which forces the deck into a south movement, as recorded by the movement of the expansion joint.

The above sequence of event and order of magnitudes were confirmed by the analytical model of the design office.

Field (non-destructive) measurements

In order to evaluate the structural integrity of the bridge it was decided to perform a geometrical control of M1 cantilever (North and South) and tension force measurements of adjacent stays of the south cantilever of M1. The tensions were estimated from frequency field measurements. Table 2 gives the frequency measurements with the corresponding tensions, and compares them with the predictions from the analytical model (with one missing cable).

![Figure 6 Geometrical control - Comparison with analytical predictions](image)

In particular for cable C1S22W it was decided to double check the tension on the cable with lift-off measurements on individual strands, performed with the monostrand jacks. In total eight strands were ‘weighted’, four from the burned side of the stay and four from the non-damaged side. The results are presented in Table 3.

<table>
<thead>
<tr>
<th>Table 2 – Comparison of frequency measurements with Design JV calculations</th>
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<tbody>
<tr>
<td>Cable</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>C1S22W</td>
</tr>
<tr>
<td>C1S21W</td>
</tr>
<tr>
<td>C1S20W</td>
</tr>
<tr>
<td>C1S19W</td>
</tr>
<tr>
<td>C1S18W</td>
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<tr>
<td>C1S23E</td>
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<tr>
<td>C1S22E</td>
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<tr>
<td>C1S21E</td>
</tr>
<tr>
<td>C1S20E</td>
</tr>
<tr>
<td>C1S19E</td>
</tr>
<tr>
<td>C1S18E</td>
</tr>
</tbody>
</table>

(1) Fcuts = 1770 Mpa. In reality, the strand installed was with Fcuts = 1860 Mpa.
(2) DJV model was run with permanent loads and missing cable C1S23W at 10°C.
Table 3 – Monostrand measurements on cable C1S22W (in kN)

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>Average</th>
<th>Estimated stay force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-damaged side</td>
<td>95.9</td>
<td>98</td>
<td>97.5</td>
<td>97.5</td>
<td>97.2</td>
<td>5.832</td>
</tr>
<tr>
<td>HDPE burned side</td>
<td>98</td>
<td>95.9</td>
<td>98.1</td>
<td>96.8</td>
<td>97.2</td>
<td></td>
</tr>
<tr>
<td>Comparison with frequency measurements</td>
<td>5.832 / 5.990 = 0.97</td>
<td></td>
<td></td>
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<tr>
<td>Comparison with DIV analytical values</td>
<td>5.832 / 6.104 = 0.96</td>
<td></td>
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</tr>
</tbody>
</table>

From the non-destructive measurements performed on the cables it can be concluded that:

(i) The tensions measured from the vibration tests are in good agreement with the values calculated by the designers.

(ii) The cables that are in the vicinity of the missing cable 23 have tensions that are less than 38% of the guaranteed ultimate tensile strength of 1770 MPa.

(iii) Cable C1S22W is carrying the design load (0.38 F_{Guts}). Despite the damage of the external duct of C1S22W, the tested strands appeared to be in good condition satisfying the isolation requirement (equal tension).

(iv) For the geometrical control, the field measurements are in good agreement with the analytical predictions.

Analytical Study

The design office of VINCI performed analytical study to evaluate the consequences of the failure of cable C1S23W. The calculation was repeated with the assumption that the adjacent cable C1W22W was also broken. The calculations were performed for permanent loads and full traffic load. The objective of the analysis with the permanent loads was to confirm the field measurements (Fig. 6, Tables 2 and 3).

The analysis with the full traffic load (four lanes loaded) was performed to ensure that the bridge meets the serviceability design requirements (i.e. allowable stresses on the cables for S.L.S. 0.50 F_{Guts}).

The analysis concluded that, even for the case of two missing cables (C1S23W and C1S22W) the stresses on the deck and cables under full traffic loads do not exceed the serviceability design requirements.

Comparative Earthquake Study

A comparative seismic analysis using the non-convex approach of inequality mechanics has been also carried out and is still in progress. This inequality approach had been initialized in Structural Engineering by the late Professor for Steel Structures in Thessaloniki University, P.D. Panagiotopoulos [4], who unfortunately passed away (1998). In this study the unilateral behaviour of the cable elements during earthquakes (or strong winds) is taken strictly into account. So the problem formulation and numerical solution, based on double discretization, in space by the Finite Element Method and in time by the Houboul method, see e.g. [5], arrives eventually to an Linear Complementarity Problem of the form

\[ \mathbf{y} \geq 0, \quad D\mathbf{y} + \mathbf{d} \leq \mathbf{0}, \quad \mathbf{y}^T(D\mathbf{y} + \mathbf{d}) = 0. \]

By \( \mathbf{y} \) is denoted the unknown vector of unilateral quantities, and \( D \) and \( \mathbf{d} \) are known quantities, computed by the incremental methodology of [5]. The obtained numerical results agree satisfactorily with those obtained by the other incremental approaches discussed in the previous sections.

5. CONCLUSIONS

The stay cables of Rion-Antirion Bridge are made of parallel galvanised strands and satisfy high level requirements in terms of corrosion protection, fatigue performance, ease of installation, maintenance and replaceability. In addition, special devices necessary to prevent wind vibration and withstand high seismic loads have implemented on the stay cable system.

Very good agreement was observed between field measurements and the numerical models, both the analytical and the numerical inequality ones, in terms of deck geometry and cable forces, as recorded during the investigation of the failure of one cable.

As the comparative dynamic analysis has shown, the bridge can safely withstand full traffic load or specified earthquakes with one or even two consecutive missing cables.

It has also been confirmed in practice, that ones the different parts of the stay cable system have been procured, a cable can be replaced in few days with a minor restriction to the traffic.

6. REFERENCES