

THE RION – ANTIRION BRIDGE: DESIGN AND CONSTRUCTION

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ABSTRACT

The Rion-Antirion Bridge located in the Gulf of Corinth of Greece, was given to traffic in August 2004. As part of the Transeuropean Network, it links the Peloponnesus to Continental Greece over a stretch of water 2,500m wide. The project was awarded by the Greek government to GEFYRA S.A, a consortium led by VINCI and six Greek construction companies. It was the first concession contract in Greece, based on a financial scheme of build, operate and transfer.

The environment in which the bridge was constructed combines a number of physical challenges and thus makes this project quite complex: deep water (up to 65 m) combined with deep soil strata of weak alluviums, possibility of strong seismic activity, tectonic movements and adverse high wind actions.

The paper presents the innovative designs of the cable-stayed bridge, which include: shallow foundations on reinforced soil; continuous and fully suspended deck for the full length of the bridge (2.252 m); a huge dissipation system; stay cables equipped with special anti-seismic accessories; and offshore construction methods.

Keywords: Cable Stayed Bridge, Inclusions, Seismic Dissipation System, Cantilever Construction

THE ENVIRONMENT

The RION-ANTIRION Bridge is located between the Peloponnese and the continent over the Gulf of Corinth, Western Greece, and is intended to replace an existing ferry system.

Its environment presents an exceptional combination of physical conditions, which makes this project quite complex: large water depth (up to 65m), deep soil strata of weak alluviums, strong seismic activity combined with possible tectonic movements, and adverse high wind actions.

The structure spans a stretch of water of some 2500m. The seabed presents fairly steep slopes on each side and a long horizontal plateau at a depth of 60 to 70m. No bedrock was encountered during soil investigations down to a depth of 100m. Based on a geological study it is believed that the thickness of sediments is greater than 500m. General trends identified through soils surveys are the following:

- a cohesionless layer is present at mudline level consisting of sand and gravel to a thickness of 4 to 7m, except under pier M4, where its thickness reaches 25m.
- underneath this layer, the soil profile, rather erratic and heterogeneous, presents strata of sand, silty sand and silty clay.
- below 30m, the soils are more homogeneous and mainly consist in silty clays or clays.

In view of the nature of the soils, liquefaction does not appear to be a problem except on the north shore, where the first 20m are susceptible of liquefaction.

The seismic conditions to be taken into account are presented in the form of a response spectrum at seabed level given in figure 1. The peak ground acceleration is equal to 0.48g and the maximum spectral acceleration is equal to 1.2g between periods of 0.2 and 1.0sec. This spectrum corresponds to a 2000-year return period, or 5% probability of exceedance for a life of 120 years. In Figure 1 the design spectrum of the Bridge is compared with the one required by Greek Seismic Code (EAK 2000) for the specific seismic zone of the bridge (III), with an importance factor of $I=1.3$.

It is worth mentioning that the Peloponnese drifts away from mainland Greece by a few millimeters per year. Taking into account the 120 years design life of the bridge, contractual specifications require the bridge to accommodate possible fault movements up to 2m in any direction, horizontally and/or vertically between two adjacent piers, in addition with 1/500 vertical tilt of the pylons. The critical accidental seismic load combination consists of the specified earthquake combined with 50% of the tectonic movements.

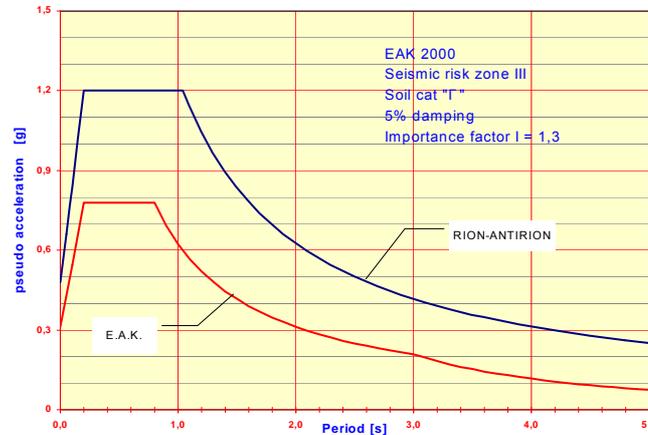


Fig. 1 Design spectrum

Due to the morphology of the surrounding mountains, the Rion-Antirion strait is known for the frequent and strong winds. The reference wind speed (hourly average, 10 meters high, 120 years return period) was estimated, from a large database of wind measurements, to be 32 m/sec. At deck level (57 meter above sea level) the reference wind speed is 50 m/sec. Flexible decks, however, such as cable-stay (i.e. Rion-Antirion Bridge) and suspension bridges are prone to aerodynamic instabilities. Existing standards require that the aerodynamic stability of flexible bridges with span larger than 200 meters be confirmed with aerodynamic tests. For the Rion-Antirion Bridge, the specifications required the critical wind speed for flutter instability to be higher than 74 m/sec, and that vortex shedding does not occur for speeds between 0 to 53 m/sec.

In addition, the bridge is capable to withstand the impact from a 180000 dwt tanker sailing at 16 knots. This corresponds to an equivalent horizontal static load of 28000 tons applied at a height of 67m from the foundation (3m above normal water surface level).

DESCRIPTION OF THE BRIDGE

These difficult environmental conditions called for an original design based on large foundations able to sustain seismic forces and large spans in order to limit the number of these foundations. The bridge consists of:

- the cable-stayed main bridge, 2252m long, built on 4 large foundations with a span distribution equal to 286m – 560m – 560m – 560m – 286m (Figure 2).
- the approach viaducts, 1000m on Rion side (composite deck), the 392m being within the concession area and 239m on Antirion side (pre-stressed simple supported beams).

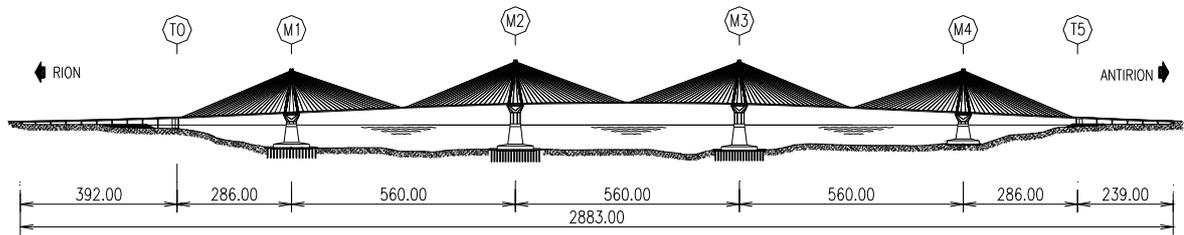


Fig. 2 Bridge elevation

Foundations consist of large diameter (90m) caissons, resting on the seabed (Figure 3). The top 20m of soils are rather heterogeneous and of low mechanical characteristics. To provide sufficient shear strength to these soil strata, which have to carry large seismic forces coming from structural inertia forces and hydrodynamic water pressures, the upper soil layer is reinforced by inclusions.

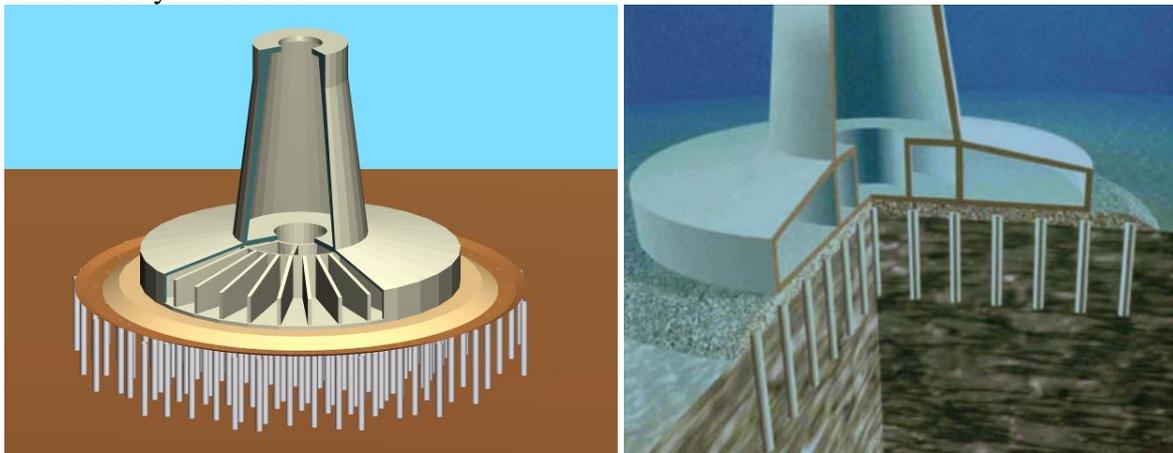


Fig. 3 Foundation and soil reinforcement with inclusions

These inclusions are hollow steel pipes, 25 to 30m long, 2m in diameter, driven into the upper layer at a regular spacing of 7 to 8m (depending on the pier); about 150 to 200 pipes were driven in at each pier location. They are topped by a 3m thick, properly selected and leveled gravel layer, on which the foundations rest. These inclusions are not required under pier M4 owing to the presence of a thick natural gravel layer.

The cable-stayed deck is a composite steel-concrete structure with a width of 27.2m, carrying two traffic lanes in each direction. The steel frame was made of two longitudinal plate girders 2.2m high on each side of the deck with transverse plate girders spaced at 4m and a concrete slab, 0.25 to 0.35m thick (Figure 4). On top, asphalt concrete pavement of 75mm thickness, including special waterproofing membrane, have been placed. The deck superstructure is continuous and fully suspended by means of stay cables for its total length of 2252 meters. In the longitudinal direction, the deck is free to accommodate all thermal and tectonic movements. At its extremities, expansion joints can accommodate the following movements:

- Under service conditions – in the longitudinal direction: opening +1.26m, closing – 1.22m, and
- under an extreme seismic event: +2.81m to –2.2m in the longitudinal direction and ± 2.5 m in the transverse direction.

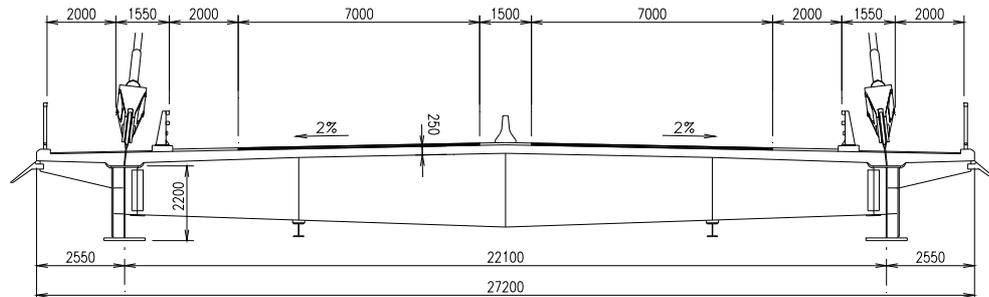


Fig. 4 Typical deck cross section

Each pylon is composed of four legs 4 x 4m, made of high strength concrete, joined at the top to give the rigidity necessary to support unsymmetrical service loads and seismic forces (Figure 5). The pylons are rigidly embedded in pier head to form a monolithic structure, up to 230m high, from sea bottom to pylon top.

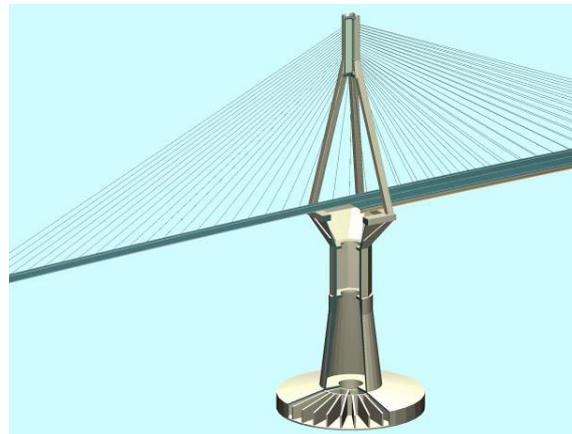


Fig. 5 Pier and pylon

The stay cables, forming a semi-fan shape, are in two inclined arrangements, with their lower anchorages on deck sides and their upper anchorages at the pylon top. They are made of 43 to 73 parallel-galvanized strands individually protected with an extruded layer of HDPE. Finally, all strands are placed inside a HDPE duct. The cable stay system, which is supplied by Freyssinet, is equipped with special accessories related to its behaviour during a major 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 during earthquake oscillations. Following a close monitoring of the behaviour of the cables for wind induced vibrations, in 2006 (two years after operation) it was decided to improve the behaviour of cables, with length higher than 100m, by increasing the damping of the cable

system. This was achieved with the installation of external dampers close to the bottom anchorage (Figure 6).



Fig. 6 Cable stays external hydraulic dampers

Finally, conservatory measures were taken for future installation of internal dampers on the short cables and secondary cross-ties, in case that the cables experience any additional type of vibration.

An innovative energy dissipation system connects the deck to the top of each pier and limits the lateral movement of the deck during the specified earthquake, while dissipating the seismic energy. The seismic protection system comprises fuse restraints and viscous dampers acting in parallel, connecting in the transverse direction the deck to the piers. The restrainers are designed as a rigid link intended to protect (to block) the hydraulic devices by withstanding the high wind loads up to a pre-determined force corresponding to a value slightly higher than the wind load ultimate limit state. Under an earthquake, the fuse restrainers are designed to yield, leaving the viscous dampers free to dissipate the energy induced by the earthquake into the structure. After the seismic event, new fuses can be re-installed in few weeks.

DESIGN CONCEPT

From the beginning it was clear that the critical load for most of the structure is the design seismic loading. The impact from the 180000 dwt tanker, equivalent to a static horizontal force of 280MN at sea level, creates shear forces and overturning moments smaller than seismic loads under the design spectrum and necessitates only local strengthening of piers in the impact zone.

The choice of the present design was made after examination of a wide range of possible solutions in term of span type (suspension spans vs. cable-stayed spans) and foundation concepts. Particularly with regard to the foundations, the bearing capacity was a major concern in these difficult environmental conditions characterised by poor soil conditions,

significant seismic accelerations and large depth of water. Alternative foundation concepts (such as pile foundations, deep embedded caissons and soil substitution) were investigated with their relative merits in terms of economy, feasibility and technical soundness¹. This analysis showed that a shallow foundation was the most satisfactory solution as long as it was feasible to significantly improve the top 20m of soils. The improvement of the shear resistance of the soil was achieved by means of metallic inclusions, as described here above. Although these foundations resemble piled foundations, they do not at all behave as such: no connection exists between the inclusions and the caisson raft, which will allow for the foundation to uplift partially or to slide with respect to the soil; the density of inclusions is far more important and the length smaller than would have been the case in piled foundations. The major concept for the selected shallow foundation is that the forces of the structure will initially transferred through contact (mainly friction) to a gravel bed layer and then to the original soil that is reinforced with the metallic inclusions. The word “inclusion” was selected instead of “pile” in order to mark the difference in the foundation behavior. This type of soil reinforcement is quite innovative and necessitated extensive numerical studies and centrifuge model tests for its validation in the Laboratoire Central des Ponts et Chaussées (France).

Another unique feature of this project lies in its continuous cable-stayed deck, which, in addition to being the longest in the world, is fully suspended. This creates an effective isolation system significantly reducing seismic forces in the deck and allowing the bridge to accommodate fault movements between adjacent piers thanks to its flexibility. In particular the 20 first natural periods of vibration of the deck are from 1.8 sec up to 7.2 sec, while the pier eigen-frequencies are around 0.55 sec. This feature allows low acceleration for the deck mass and thus lower forces on the piers.

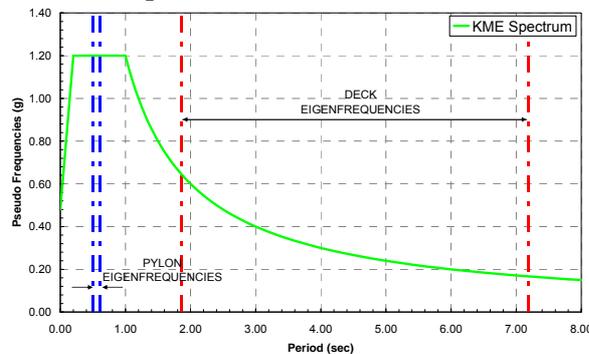


Fig. 7 Structure eigen-frequencies and KME spectrum

In the transverse direction the deck will behave like a pendulum and its lateral movements must be buffered. This is achieved with the use of 4 hydraulic dampers at the pylon locations and two at the transition piers, having a capacity of 3500 kN each, operating both in tension and compression. They are able to limit the relative lateral displacements between the deck and the pylons and dissipate large amount of energy during a seismic event. The dynamic relative stroke between the deck and a pylon during an extreme seismic event will be in the order of 3.50m, with velocities up to 1.6m/sec. The constitutive law of the dampers is non-

linear viscous, with a velocity exponent of 0.15 ($F=C.V^{0.15}$, where $C=3$, V in m/sec and F in MN).

The dissipation system (4 dampers, 2 on each side) installed between deck and a pier is illustrated in the next figure. In order to avoid deck transverse movements due to wind, a special metallic strut, called “fuse restrainer”, connects the deck and each pier. This lateral restrainer incorporates a mechanical device that fuses at $\pm 10500\text{kN}$. The function of this device is to allow transversal deck motion only after a seismic event of low occurrence enabling the dampers to go into action.



Fig. 8 Dissipation system between deck and piers

A prototype test for these dampers was performed in the CALTRANS testing facility at the University of California San Diego² (Figure 9).



Fig. 9 Prototype testing of hydraulic dampers at SRMD Facility (UCSD)

The good performance of the dissipation system of the bridge was confirmed after a seismic event of a moment magnitude of $M_w=6.5$, that took place on June 08, 2008 with an epicenter located approximately 36km SW of the bridge and a focal depth of around 30km. The recorded peak ground acceleration was 0.13g, while the maximum estimated under the piers was 0.18g. The fuses yielded and the dampers operated with a maximum estimated velocity of 0.28m/sec and a maximum stroke of 15cm.

An expansion joint with special fuse elements was designed by Maurer Söhne to allow the maximum design movements of the main bridge to the approach viaducts. While the expansion joint accommodates a service movement of 1.22m closing, 1.26m opening in longitudinal and +/- 0.10m in lateral direction, it is designed for a maximum opening of 2.20m closing, 2.81m opening in longitudinal and +/- 2.50m in lateral direction for the specified design earthquake (Figure 10). During this seismic event fuse elements within the expansion joint will open to minimize the forces induced to the bridge structure and keep the expansion joint with minor damages. Several tests for these fuse elements were performed on a shaking table of the Enel.Hydro S.p.A. – B.U. ISMES in Bergamo (Italy).



Fig. 10 Installation of MAURER expansion joint

The capacity design of the structure for the extreme seismic event consisted of a “control damage” approach at the following locations: sliding at the foundation raft-soil interface, dissipation of energy by viscous dampers at pylon locations and potential formation of plastic hinges (with a minor damage of concrete spalling) at pylons legs. The dynamic response of the structure was estimated with artificial and natural accelerograms matching the design spectrum based on a non-linear time history analysis. 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 rheological model for the soil-structure interaction¹.

In addition to the 3D finite element analysis, a non-linear 3D push over analysis was performed for the 4 leg pylons in order to estimate their remaining capacity after the formation of plastic hinges and to confirm their ductile behaviour of the highly confined pylon legs. Special concrete confinement tests were also performed at the University of California San Diego on high strength concrete (C60/75) used for pylon legs in order to define its strain-stress curves (especially the reduced slope of the descending branch of the curves) for various confinement ratios.

CONSTRUCTION METHODS

Construction methods for the foundations were those commonly used for the construction of offshore concrete platforms:

- construction of the foundation footings in a dry dock up to a height of 15m in order to provide sufficient buoyancy;
- towing and mooring of these footings at a wet dock site;
- construction of the conical part of the foundations at the wet dock site;
- towing and immersion of the foundations at final position.

However, some features of this project made the construction process of its foundations quite exceptional.

The dry dock has been established near the site. It was 200m long, 100m wide, 14m deep, and could accommodate the simultaneous construction of two foundations. It had an unusual closure system: the first foundation was built behind the protection of a dyke, but once towed out, the second foundation, the construction of which had already started, was floated to the front place and used as a dock gate.



Fig. 11 Construction in dry dock

Dredging the seabed, driving 500 inclusions, placing and leveling the gravel layer on the top, with a depth of water reaching 65m, was a major marine operation, which necessitated special equipment and procedures. In fact, a tension-leg barge has been custom-made, based on the well-known concept of tension-leg platforms but used for the first time for movable equipment. This concept was based on active vertical anchorage to dead weights lying on the seabed (Figure 12).

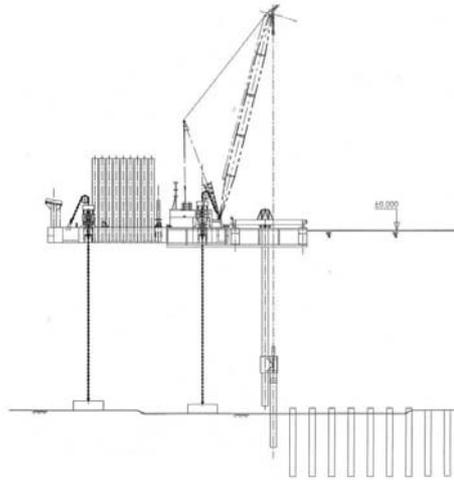


Fig. 12 Tension-leg barge

The tension in these vertical anchor lines was adjusted in order to give the required stability to the barge with respect to sea movements and loads handled by the crane disposed on its deck. By increasing the tension in the anchor lines, the buoyancy of the barge allowed the anchor weights to be lifted from the seabed, then the barge, including its weights, could be floated away to a new position.

As already stated, once completed the pier bases were towed then sunk at their final position. Compartments created in the footings by the radial beams were used to control trim by differential ballasting. Then the pier bases were filled with water to accelerate settlements, which are significant (between 0.1 and 0.2m). This pre-loading was maintained during pier shaft and pier head construction, thus allowing a correction for potential differential settlements before erecting pylons and deck superstructure.

The deck of the main bridge was erected using the balanced free-cantilever technique, with prefabricated deck elements 12m long comprising also their concrete slab. The total weight of a prefabricated segment is 340 tons and was placed with a floating crane (TAKLIFT 7). The rate of erection of completed deck structure was up to 60m per week (Figure 13).



Fig. 6 Installation of a prefabricated deck segment

CONCLUSIONS

The Rion – Antirion Bridge had to overcome an exceptional combination of adverse environmental conditions: water depth up to 65m, deep soil strata of weak alluviums, strong seismic activity and tectonic movements.

This resulted in a unique multi-span cable-stayed bridge with a continuous deck for the full length of the stretch (2252m), fully suspended from four pylons. Foundations consist in large diameter (90m) caissons resting on the seabed. The top 20m of soils are reinforced by means of metallic inclusions, based on an innovative concept.

The design and construction of this € 800 million project have been undertaken under a private BOT scheme, led by the French company VINCI. The whole project was completed on summer 2004, ahead of the contractual deadline by several months (Figure 14).

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Fig. 7 The Rion-Antirion Bridge close to completion (spring 2004)