Rion-Antirion Bridge, Greece – Concept, Design, and Construction

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Fig. 2: Main bridge concept – general view

Introduction

The Rion Antirion crossing consists of a main bridge, 2252 m long and 27,20 m wide, and two approaches, respectively 392 m and 239 m long, one on each side of the Corinth Strait. The main bridge is located in an exceptional environment which consists of deep water (65 m), deep soil strata consisting of weak alluvium (the bedrock being probably more than 500 m below the sea bed level) and strong seismic activity with possible slow but important tectonic movements. Of course, if all these difficulties could have been taken into account separately, there would have been no problem, but the conjunction of all these unfavourable conditions lead to unusual conceptual problems. As the seismic activity in the area is severe, it is clear that an earthquake will unavoidably lead to high soil structure interaction forces at any bridge support location. As these high forces have to be resisted by weak layers of soil, the foundation of any support under more than 60 m of water was a major point of concern.

Design Criteria

The seismic condition is based on response spectrum at the sea bed level which corresponds to a 2000 year return period (*Fig. 1*). The peak ground acceleration is 0,48 g and the maximum spectral acceleration is equal to 1,2 g over a rather large period range.

As previously mentioned the bridge also has to accommodate possible fault movements which could lead to a 2 m vertical and horizontal displacement of one part of the main bridge with regard to the other part, the pylons being simultaneously subjected to small inclinations due to the corresponding rearrangement of the sea bed below the foundations. In addition, the pylons have to withstand the impact of a big tanker (180 000 t) moving at a speed of 30 km/h.

Main Bridge Concept

Taking into account this range of possible occurrences, the span length of the main bridge had to be adjusted to reduce as much as possible the number of supports in the strait. Clearly, these conditions would have favoured the design of a suspension bridge but a major slope stability problem on the Antirion side excluded such a solution from the very beginning of the conceptual design stage. Instead, an exceptional cable stayed bridge (*Fig. 2*) made of 3 central spans 560 m in length and 2 side spans 286 m long was selected [1].

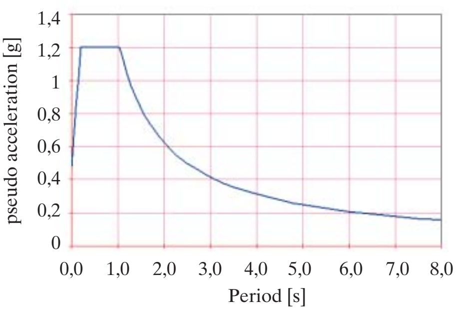

Fig. 1: Design spectrum

The corresponding four pylons rest on large concrete substructure foundations, 90 m in diameter, 65 m high, which distribute all the forces to the soil. Below this substructure, the bearing capacity of the heterogeneous and weak soil was improved by means of inclusions, which consist of 20 mm thick steel pipes, 25 to 30 m long and 2 m in diameter, driven at a regular spacing of 7 or 8 m. The top of the steel pipes is covered by a calibrated gravel layer which provides a transition from the structure to the reinforced soil.

Initially, a concrete block which acts as the base of 4 concrete legs converging at the top of the pylons and giving them the appropriate rigidity was supported by these huge foundations through octagonal pylon shafts, pyramidal capitals and a sophisticated set of bearing devices, post-tensioned tendons and spring dampers. This was absolutely necessary since each pylon supported a symmetrical cantilever 510 m long and each cantilever was connected to the adjacent one or to the approaches by a simply supported deck girder 50 m long. Careful analyses of the behaviour of the reinforced soil and improvements of this innovative concept led to giving up the initial static scheme of the main bridge and to adopt a much more efficient structure with a continuous pylon (from sea bed to pylon head) and a continuous fully suspended deck isolated as much as possible from the pylons. This allowed the depth of the deck and therefore also the wind effects on the bridge to be reduced.

The deck is a composite steel-concrete structure, 27,20 m wide, consisting of a concrete slab, 25 to 35 cm thick, connected to twin longitudinal steel I-girders, 2,20 m high, braced every 4 m by transverse cross beams (*Fig. 3*). It is continuous over its total length of 2252 m, with expansion joints at both ends, and is fully suspended by 8 sets of 23 pairs of cables. In the longitudinal direction the deck is free to accommodate all movements due to thermal and seismic actions and the joints are designed to accommodate 2,5 m displacements under service conditions and movements of up to 5,0 m in an extreme seismic event.

In the transverse direction it is connected to each pylon with 4 hydraulic dampers of 3500 KN capacity each and a horizontal metallic strut of 10 000 KN capacity.

The stay cables are arranged in two inclined planes in a semi-fan configuration. They are made of 43 to 73 parallel galvanised strands individually protected by an HDPE sheath.

Design Phase

According to the previous general presentation of the project, it is clear that the design of the main bridge has mainly been governed by the ability of the structure as a whole to resist the major seismic events including a possible fault movement. This means that the structure had first to be designed to resist what will be the main actions during its design life (i.e. for the classical serviceability limit states and the corresponding ultimate limit states). Then the capacity of the main components of the structure was adjusted to accommodate the demands during the design earthquake without exceeding the acceptable damage. This was the best way to get the most flexible structure and therefore the most favourable concept from a seismic point of view.

Since the signing of the contract was delayed by the banks and the bridge was really an enormous undertaking, it

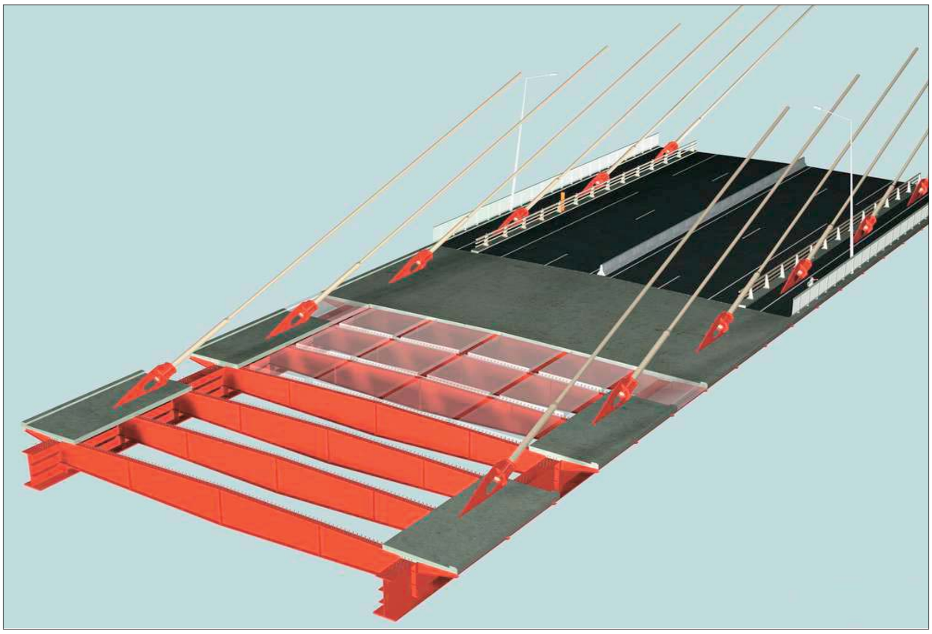

Fig. 3: Composite deck concept

was decided to spend about one year carrying out sophisticated parametric studies with a view to optimising the concept and the structure as well.

Reinforced Soil and Foundation Concept

The foundations are a typical example of a major part of a structure where the performance of the concept had to be evaluated through the capacity of the soil, to resist the soil-structure interaction during the earthquake event, and the ability of the structure to accommodate the exceptional displacements (generated by the ground motion) with a controlled damage considered as acceptable.

In the case of the Rion-Antirion main bridge, the foundations of the structure consist of two separate parts (*Fig. 4*):

– the reinforced soil, which is a clay-steel composite 3D volume
– the pylon bases, which are rigid bodies not subject to any unusual strength problems

These parts are made partially independent by the gravel layer, which was designed to transfer a range of horizontal forces compatible with the strength of the reinforced soil, the global stability of the structure and the acceptable permanent displacements of the pylons.

Although the foundations look like pile foundations, they do not at all behave as such: no connection exists between the inclusions and the pylon rafts. The pylon bases are therefore allowed to experience uplift or to slide with respect to the reinforced soil. The

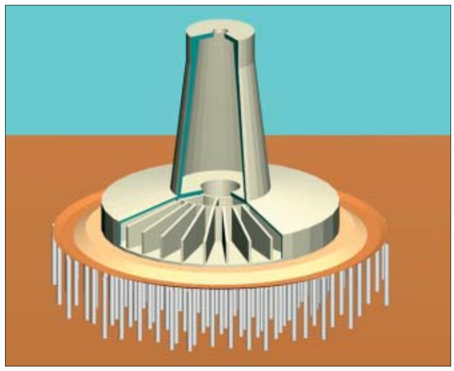

Fig. 4: Reinforced soil and foundation concept

capacity design philosophy, introduced in foundation engineering for the evaluation of the seismic bearing capacity of shallow foundations based on the yield design theory, was then extended to this innovative foundation concept in seismic areas [2]. Using the yield design theory, through a set of appropriate kinematic mechanisms (*Fig. 5*) it was possible to derive an upper bound estimate of the global bearing capacity of the reinforced soil (*Fig. 6*).

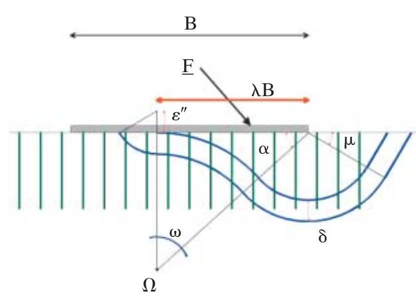

Fig. 5: Kinematic mechanism

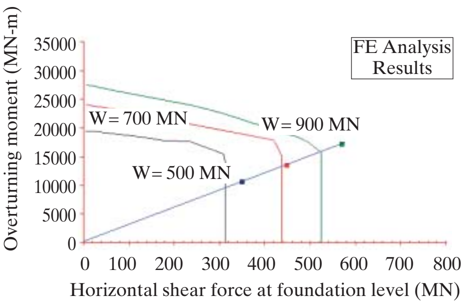

Fig. 6: Reinforced soil interaction diagram

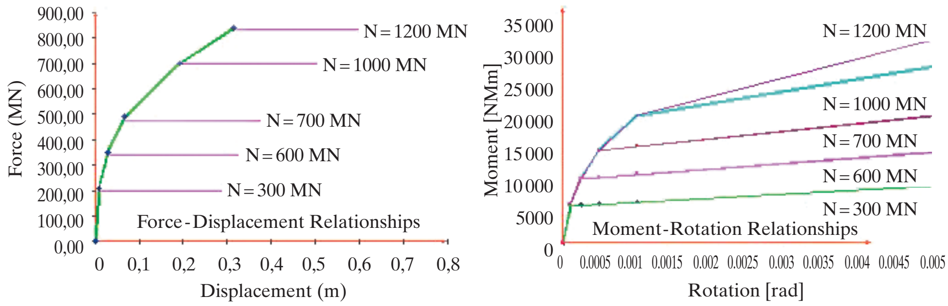

Fig. 7: Behaviour curves of reinforced soil

For this purpose the reinforced soil was modelled as a two-dimensional continuum appropriately connected to beams simulating the stiff inclusions. Consequently, the calculations took into account the contribution of the inclusions for the overall resistance of this new concept. The simplicity of such calculations allowed optimizing the size and the spacing of the inclusions. A set of centrifuge tests was run to validate the concept and the theoretical approaches.

Analyses of Reinforced Soil

Nonlinear finite element analyses were then carried out. They lead to the constitutive laws for the reinforced soil, which were used in the general analysis of the structure (*Fig. 7*).

All these calculations, adequately combined with a global dynamic analysis, demonstrated that the coupled gravel layer and soil reinforcement improved the bearing capacity of the whole foundation system while controlling the failure mode:

- The transition provided by the gravel layer limits the maximum shear force at the interface, dissipates energy by sliding and forces the foundation "to yield" according to a mode that is compatible with an acceptable behaviour of the structure.
- The stiff inclusion reinforcement increases the strength capacity of the soil in order to eliminate undesirable failure modes, like rotational failure which would compromise dangerously the global stability of the structure, and dissipates an important amount of energy as was anticipated from the Force-Displacement Diagram (*Fig. 8*).

Dynamic Analysis of Bridge

All the previous calculations and results were used to carry out detailed and carefully executed 3D dynamic analyses of the whole structure [3].

Thanks to the development of a certain number of calculation tools interfacing with a powerful commercially available computer software, the following very important properties were taken into account:

- Nonlinear hysteretic behaviour of the reinforced soil
- Possible sliding of the pylon bases on the gravel beds precisely adjusted to the accompanying vertical force
- Nonlinear behaviour of the reinforced concrete of the pylon legs (including cracking and stiffening of concrete due to confinement)
- Nonlinear behaviour of the cable stays
- Nonlinear behaviour of the composite bridge deck (including yielding of steel and cracking of the reinforced concrete slab)
- Second order effects (or large displacements, if any).

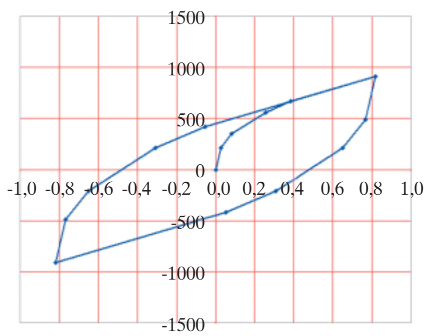

Fig. 8: Soil horizontial force-displacement diagram

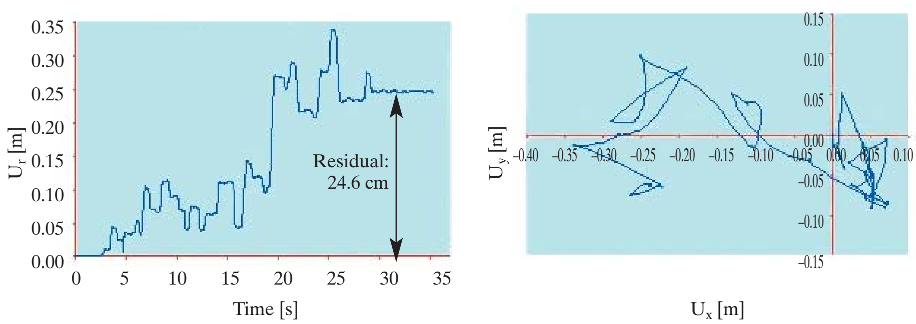

Fig. 9: Controlled response of structure

Several sets of independent artificial accelerograms conforming to the seismic design spectrum for the 3 components of the ground motion (the vertical one being scaled to 70% of the horizontal) were used. From these calculations, the way the reinforced soil behaves and the bases slide could be carefully checked.

Behaviour of Reinforced Soil

The overall analysis of the Bridge, including a lumped parameter model of the reinforced soil, allowed checking the results obtained with the various software components specifically developed for this bridge. The results were consistent with the assumptions. They showed that the forces and overturning moments applied to the soil always remain within the bounding surface. They confirmed the very good behaviour of the fully suspended deck, which is isolated as much as possible. The relative displacement of the pylon bases with respect to the gravel layer evidenced some sliding, which nevertheless is acceptable and if, for any reason, this sliding could not occur it has been checked that this is not a major point of concern. Under the most severe seismic event, the substructure of the bridge will slide (*Fig. 9*); the pylon bases will rotate slightly; but all this will happen without any detrimental effect on the structure of the bridge as the fully suspended and flexible deck is able to align automatically and can be adjusted subsequently to an acceptable geometry by re-tensioning the stay cables.

Behaviour of Structure

Because the stability of the fully suspended multi cable-stayed span deck is secured by the stiffness of the pylons, these were the most critical part of the Structure. The stiffness is achieved by having the four legs converge at mid height of the anchorage zone. The dynamic analyses showed that the pylons and shortest cable-stays are indeed heavily loaded during the earthquake event. Clearly, from this point of view, there is a contradiction between what is required for the normal operation of the bridge and the demand when a severe earthquake occurs. Indeed, the pylons are too stiff and the shortest cables as designed for serviceability are not flexible enough.

Dynamic calculations have shown that the extreme vibrations generate various crack patterns, distributed along the legs, due to both bending and tension (*Fig. 10*). On the one hand, it could be observed that this cracking is favourable as it generates the necessary flexibility of the legs without leading to unacceptable strains in the materials (i.e. unacceptable damage). On the other hand, it was not an easy task to get a global view of the behaviour of the pylon as the information produced by a sophisticated analysis is too voluminous. With time steps of 0,02 s – i.e. 2500 steps for a 50 second event – and the number of cross-sections in the model of one pylon leg being 13 – this means that there would be 130 000 configurations of reinforced concrete cross-sections to be checked for each pylon in order to evaluate the global behaviour of the structure at any time.

To evaluate this vast quantity of information, it was decided to check, for the duration of the earthquake, that the strains in the materials (concrete and steel) in each cross-section do not exceed the acceptable limits that guarantee a controlled damage of the pylons. The general consistency of these sophisticated calculations can be verified for time history peak values of these parameters by checking the corresponding deflection shapes of the legs, axial shear forces and bending moments generated in each cross-section.

Push-Over Analyses of Pylons

Under these conditions, it made sense to carry out a push-over analysis of the pylons to evaluate their global behaviour and compare their capacity to the demand, in terms of displacements, during the extreme seismic event. It should be observed that such a push-over analysis is common-place nowadays. Moreover, this analysis is extremely simple for a tall pier of a bridge which behaves as a single degree of freedom system and is therefore loaded by a shear force acting at the level of the centre of gravity of the bridge deck. It is no longer simple when the pier is a pylon group made up of four legs converging in a zone where a large number of cables are generating many forces at various levels. In this case, one way of performing such a push-over analysis is to reproduce the state of equilibrium at a stage of the dynamic analysis which can be considered as the most unfavourable situation during the 50 second event –

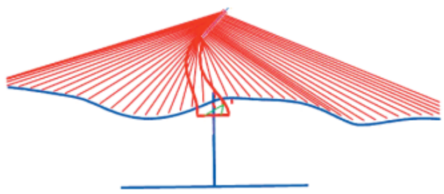

Fig. 10: Typical deflection shape of pylons

i.e. when forces, bending moments and displacements are the most severe. This approach allowed assessing the effect of the displacement demand on a pylon as well as its displacement capacity as estimated from the 3D dynamic analysis.

In a static analysis on an exact model of the pylon, inertial forces coming from the deck through the cables and from the pylon concrete mass acceleration were gradually increased by a magnification factor while gravity or initially applied forces (permanent loads) were not.

The diagram showing the displacement D at the top of the pylon legs versus the magnification factor A allowed a clear differentiation of the various steps characterising the behaviour of a whole pylon group (*Fig. 11*). As the displacement is mainly diagonal, these steps are as follows:

- Step 1 ($0 < A < 0{,}4$) – Elastic behaviour $0 < D < 0{,}10$ m
- Step 2 ($0{,}4 < A < 1{,}2$) – Axial cracking in the tension leg, hinges forming at the top of this leg then at the top of the middle legs ($0{,}10$ m $< D < 0{,}45$ m)
- Step 3 ($1{,}2 < A < 1{,}4$) – Yielding of steel in the tension leg ($0{,}45$ m $< D < 0{,}60$ m)
- Step 4 ($1{,}4 < A < 1{,}6$) – Hinge forming at the top of the compression leg ($0{,}60$ m $< D < 0{,}90$ m).

Such a push-over analysis showed that the displacement demand ($D = 0{,}36$ m for $A = 1$) is far below the displacement capacity of the pylon legs which is of the order of 0,90 m at maximum and, therefore, either that the damage should be limited in the case of an extreme event or that any deviation with regard to the input motion should not have any bad consequences.

Construction

The main bridge concept underwent a spectacular evolution, which took into account all the aspects of project costs and was the result of the close interaction between design and the study of realistic construction methods.

Unusual Aspects

As already mentioned, the construction of the main bridge faced the major difficulty of great water depth, which reaches 65 m for central piers, and poor geotechnical properties of the seabed. As a result, foundation works, including not only dredging and steel pipe driving but also exceptional works like the precision laying of an 8000 m² gravel bed, presented a formidable challenge, requiring unusual skill and equipment. To successfully complete this task a combination of the latest technologies available in the construction of concrete off-shore oil drilling platforms, immersed tunnels and large cable-stayed bridges was extensively used.

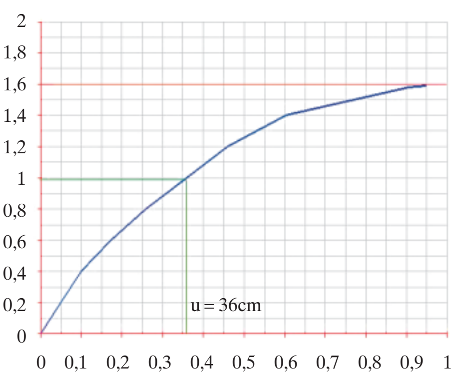

Fig. 11: Displacement at top of pylon legs/magnification factor

Pylon Base

Pylon bases were built in two stages near Antirion: the footings were cast first in a dry dock 230 m long and 100 m wide; the conical shafts were completed later in a wet dock with sufficient water dept.

In the dry dock two cellular pylon footings were cast at a time (*Fig. 12*). In fact, two different levels in the dock provided 12 m of water for casting a leading footing and 8 m for the next one. When the first footing, including a leading 3,2 m lift of the tapering shaft, was complete, the dock was flooded and the nearly 17 m tall structure was towed a few hundred metres to deep water. A very original idea allowed saving much time in the production cycle of the pylon bases. Before the first tow-out, the dry dock was closed using a typical sheet pile protection dyke, which then had to be completely removed.

Clearly, building and removing such a dyke again would have been extremely time-consuming. In fact, as long as the less advanced second footing was floated forward into the deeper dock and sunk by flooding, it was possible to use it as a gate, providing that everything was designed to do so. Instead of building a dyke again, temporary steel walls around the base top slab and sheet piles projecting from its sides could easily seal the dock mouth, allowing it to be de-watered.

As work resumed in the dock, the conical shaft of the leading base anchored with chains in the wet dock was progressively cast with standard jump forms on top of the footing. Cells in the base were progressively flooded to sink the structure and maintain a constant height above water (*Fig. 13*). When a pylon base was tall enough to stand a few meters above water, tugs took it to its prepared bed where it was ballasted and placed at its final location. It was then pre-loaded by filling with water, to speed up and anticipate settlements (between 20 and 30 cm) during pylon shaft and capital construction, thus allowing a correction for differential settlements when erecting the pylon legs.

Foundations and Tension Leg Platform

Foundation work began in October 1999 by dredging the seabed at pylon locations, laying a 90 cm thick sand layer, driving the inclusions and leaving them projecting 1,5 m above the sand to be finally covered by a 1,6 to 2,3 m thick layer of rounded river gravel and a 50 cm thick layer of crushed gravel. Gravel was laid in parallel berms, 2 m wide, separated by V shaped cuts about 30 cm deep to provide some flexibility when placing the pylon bases.

All these marine works were performed, step by step, from a 60 m long and 40 m wide tensioned leg platform anchored with adjustable chains to movable concrete blocks. Equipment for driving soil reinforcing tubes and preparing the seabed was mounted on submersible pontoons anchored to one end of the platform with steel arms. A movable steel tube, reaching nearly to the sea floor, guided piling equipment and deposited sand and gravel on the pre-dredged bed. This equipment permitted the necessary works to be performed on a 14 m wide, 28 m long area. The platform then had to be moved from one area to the next by a barge equipped with a dynamic positioning system. Permanent sonar scanning of the finished gravel bed allowed precise checking of the achieved foundation level from the platform and showed it was generally within a 5 cm tolerance. To prepare the sea bed under each pylon base it was necessary to place the platform, at forty different locations, over the course of about five months.

Upper Part of Pylons

For the remaining sections of the pylons, all materials, concrete, reinforcement, post-tensioning and equipment were provisioned by a support barge, used as a fixed base, and a roll-on roll-off barge transporting the truck mixers and the reinforcement from the shore to the pylons. The octagonal shafts of the pylons were cast in place using self climbing formworks.

The huge inverted pyramidal capitals are key elements of the pylon structures; they have to withstand the tremendous forces coming from the legs, mainly during a seismic event, and to transfer them to the shafts. This is the reason why they are heavily reinforced and prestressed. The construction of these components, which were cast in place, took seven months and required 4000 m^3 of concrete, 1750 t of steel reinforcement and 30 000 m^2 of external

Fig. 12: Construction of pylonbases in dry dock

Fig. 13: Towing out a pylon base

forms as well as sophisticated equipment.

The construction of the pylon legs progressed step by step, in 4,8 m long sections, up to the point where they merge to support the cable anchoring zone. Heavy temporary bracing provided seismic resistance during construction (*Fig. 14*). The steel core of the pylon head was made of two units placed at their final location by a huge floating crane able to reach a height of 170 m above sea level.

Deck

The construction method of the composite steel – concrete deck was similar to the one successfully used on the second Severn crossing. Deck elements, 12 m long and including the concrete slab, were prefabricated at a pre-assembly yard. They were placed in their final location by the floating crane and bolted to the previously assembled segments using the balanced cantilever erection method (*Fig. 15*). Only small joints providing enough space for an appropriate steel reinforcement overlapping had to be cast in place.

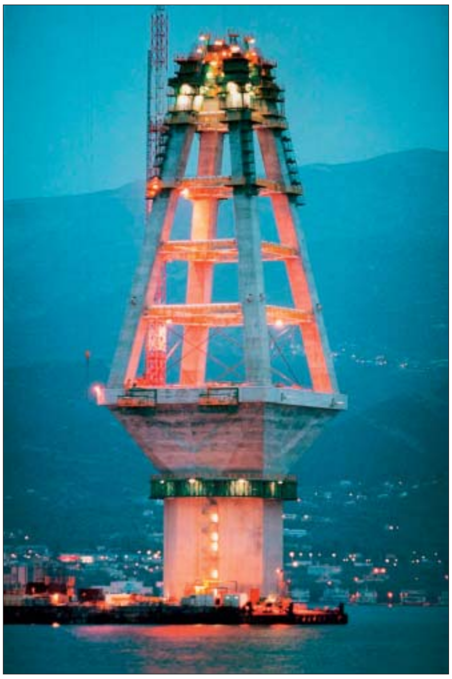

Fig. 14: Construction of pylon legs

Fig. 15: Deck as per April 2004

Conclusion

The Rion-Antirion Bridge is a major and impressive link when compared to other major cable-stayed bridges such as the second Severn Bridge and even to the Normandy Bridge. The design and construction of this $ 750 million project undertaken under a private BOT (build-operate-transfer) scheme could overcome an exceptional combination of adverse environmental conditions thanks to the choice of an appropriate concept and seismic design philosophy. The pylons are founded directly on a gravel layer placed on the sea bed allowing them to undergo controlled displacements under the most severe earthquake and, based on an innovative concept, the top 20 m of soil located under the large diameter bases (90 m) of the pylons are reinforced by means of steel inclusions to resist high soil-structure interaction loads. The 2252 m long deck of the cable-stayed bridge is continuous, fully suspended and therefore isolated as much as possible from the worst seismic motions. If small damage is experienced in the pylon legs after the big seismic event, the whole bridge will be safe and still opened to emergency traffic if necessary. Completed in August 2004, the Rion-Antirion Bridge was opened to traffic 4 months before the contractual deadline.

Acknowledgements

The BOT contract has been signed by the Ministry of Environment and Public Works and the concessionaire which consists of the same companies as the contractor.

References

[1] TEYSSANDIER, J.P.; COMBAULT, J.; MORAND, P. The Rion-Antirion Bridge Design and Construction. *12th World Conference on Earthquake Engineering.* Auckland, New Zealand, 2000.

[2] PECKER, A. A Seismic Foundation Design Process, Lessons learned from two major projects: The Vasco da Gama and the Rion-Antirion Bridges. *ACI International Conference on Seismic Bridge Design and Retrofit.* La Jolla, California, 2003.

[3] COMBAULT, J., MORAND, P., PECKER, A. Structural Response of the Rion-Antirion Bridge. *12th World Conference on Earthquake Engineering.* Auckland, New Zealand, 2000.

SEI Data Block

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Ministry of Environment and Public Works, Athens, Greece

Architect:
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Structural Engineers:
VINCI Construction Grands Projets, Paris, France
INGEROP, Paris, France
DOMI, Athens, Greece
Buckland and Taylor, Vancouver, Canada
DENCO, Athens, Greece

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J & P –AVAX, Athens, Greece
Athena, Athens, Greece
Proodeftiki, Athens, Greece
Pantechniki, Athens, Greece

Concrete (m^3):	210 000
Reinforcing steel (t):	57 000
Structural steel (t):	28 000
Stay cables (t):	3800
Total cost (EUR million):	750
Service date:	August 2004