DESIGN AND CONSTRUCTION OF THE RION ANTIRION BRIDGE FOUNDATIONS

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Abstract

The choice of a design concept for a bridge foundation is guided by various factors; several of these factors are indeed of technical origin, like the environmental conditions in a broad sense, but others non technical factors may also have a profound impact on the final design concept. The foundations solutions adopted for the Rion Antirion bridge are described and an attempt is made to pinpoint the major factors that have guided the final choices. The Rion Antirion bridge is exemplar in that respect: the foundation concept combines the simplicity of capacity design, the conceptual facility of construction and enhances the foundation safety. The design of these foundations was a very challenging task which required full cooperation and close interaction with all the parties involved: concessionaire, contractor, designers and design checker.

INTRODUCTION

The Rion-Antirion bridge project is a BOT contract granted by the Greek Government to a consortium led by the French company Vinci Construction (formerly Dumez-GTM). It is located in Greece, near Patras, will constitute a fixed link between the Peloponese and the Continent across the western end of the gulf of Corinth and is intended to replace an existing ferry system (Fig. 1). The solution adopted for this bridge is a multiple spans cable stayed bridge with four main piers; the three central spans are 560 m long each and are extended by two adjacent spans (one on each side) 286 m long. The total length of the bridge, with the approach viaducts, is approximately 2.9 kilometers (Fig. 2). The call for tender was launched in 1992, the contract, awarded to the consortium in 1996, took effect in December 1997. Construction started in 1998 and has been completed in summer 2004 and opened to traffic on the 11th of August, five months ahead of schedule.

The bridge has to be designed for severe environmental conditions (e.g., Teyssandier et al. 2000; Teyssandier 2002): weak alluvium deposits, high water depth, highly seismic area, possible occurrence of large tectonic displacements. Very early in the design process it turned out that design was mainly controlled by the seismic demand imposed to the foundations.

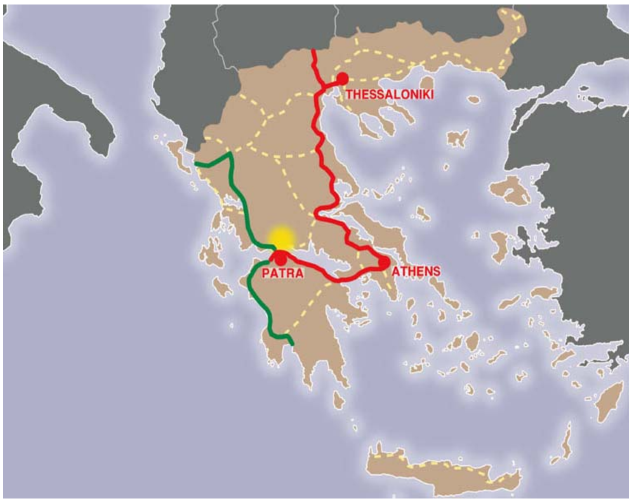

Fig. 1: Location of the bridge

In order to alleviate potential damage to the structure due to the above adverse conditions and to carry the large earthquake forces brought to the foundation (shear force of the order of 500 MN and overturning moment of the order of 18 000 MNm for a vertical buoyant pier weight of 750 MN), an innovative foundation concept was finally adopted which consists of a gravity caisson (90 m in diameter at the sea bed level) resting on top of the reinforced natural ground (Combault et al. 2000). The ground reinforcement is composed of steel tubular pipes, 2 m in diameter, 20 mm thick, 25 to 30 m long driven at a grid of 7 m x 7 m below and outside the foundation, covering a circular area of approximately 8 000 m^2. The total number of inclusions under each foundation is of the order of 150 to 200. In addition, the safety of the foundation is greatly enhanced by interposing a gravel bed layer, 2.8 m thick, on top of the inclusions just below the foundation raft with no structural connection between the raft and the inclusions heads. This concept (inclusions plus gravel layer), in addition to minimizing the hazards related to differential settlements, enforces a capacity design philosophy in the seismic foundation design, (Pecker 1998).

In the following we will focus on the foundations of the main bridge, because the foundations of the approach viaducts (pile foundations) do not deserve much comments.

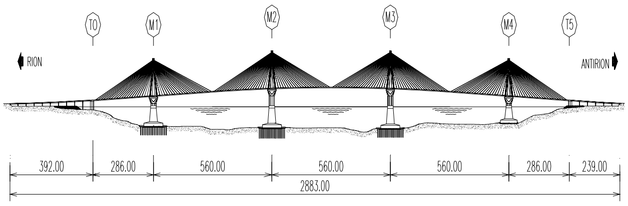

Fig. 2: Bridge elevation

GEOLOGICAL AND GEOTECHNICAL ENVIRONMENT

The site has been subjected to extensive offshore soil investigations performed either from floating barges or from a ship, controlled with a dynamic positioning system; these investigations included cored boreholes, static Cone Penetration Tests with pore pressure measurements (CPTU), Standard Penetration tests (SPT), vane tests and dilatometer tests, seismic cone tests and sampling of intact soil samples for laboratory testing. All the borings reached depths ranging from 60 m to 100 m below the sea bed. Under each of the main bridge pier three continuous boreholes, three CPTU, two seismic cones, one SPT/dilatometer boring have been drilled. Approximately 300 samples have been retrieved and subjected to advanced laboratory testing. Based on the results of these investigations representative soil profiles have been defined at the locations of the main bridge piers and ranges of soil mechanical characteristics have been derived.

The water depth in the middle of the strait reaches 65 m. The soil profile consists of weak alluvial strata deposited in alternate layers, with individual thickness of a few meters, of silty sands, sandy clays and medium plasticity clays. In the top hundred meters investigated by the soil survey, the clay, or silty clay, layers predominate (Fig. 3). No bedrock was encountered during the investigations and based on geological studies and geophysical surveys its depth is believed to be greater than 500 m.

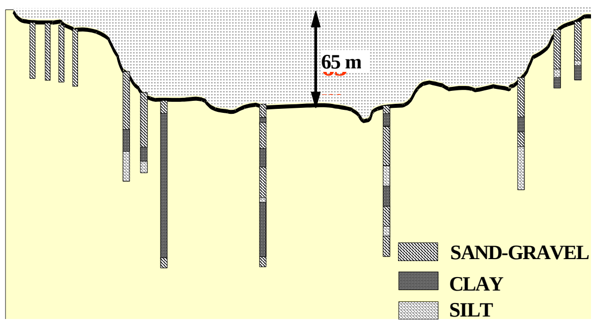

Fig. 3: Soil profile

The mechanical characteristics of the offshore layers are rather poor with undrained shear strengths of the cohesive strata increasing slowly with depth from approximately 30-50 kN/m^2 at the sea bed level to 80-100 kN/m^2 at 50 m depth (Fig. 4). A major difference occurs in the cohesionless strata: some of these layers are prone to significant pore pressure buildup, even possibly to liquefaction, under the design earthquake. Accordingly, the undrained strengths of the cohesionless layers are taken equal either to their cyclic undrained shear strengths or to their residual strengths.
Based on the results of the laboratory one dimensional compressibility tests and of correlations with CPT results or with the undrained shear strengths, a slight overconsolidation, of the order of 150 kN/m^2, of the upper strata was evidenced.
The shear wave velocities are also small, increasing from 100-150 m/s at the ground surface to 350-400 m/s at 100 m depth (Fig. 5).
It is worth noting that for design, as shown in Fig.4 and Fig.5, two sets of soil characteristics have been used instead of a single set based on characteristic values; this decision is guided by the fundamental necessity to maintain compatibility in the whole seismic design process: the seismic demand is calculated assuming one set of properties (successively lower bound and upper bound) and the foundation capacity is checked with the associated strengths (respectively lower bound and upper bound); the capacity is never checked with low properties when the forces are calculated with high properties and vice versa. In addition, given the large foundation dimensions, special attention has been paid to the spatial variability of the soil properties across anyone foundation; this variability

may have an impact on the differential settlement and tilt of a pylon. Fig. 6 gives an example of the variability of the cone point resistance across a foundation, variability which is obviously reflected by variability in the shear strength and compressibility of the soil strata.

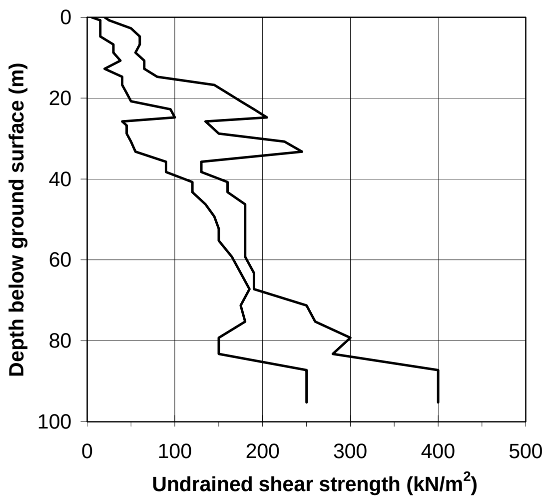

Fig. 4: Undrained shear strength

ENVIRONMENTAL CONDITIONS

The environmental conditions are defined by three major possible events likely to occur during the bridge life time: ship impact on a main bridge pier, occurrence of a major earthquake in the vicinity of the bridge, long term tectonic movements.

Ship impact

The hazard represented by this impact corresponds to 160 000 Mg tanker hitting one pier at a speed of 16 knots (8.2 m/s). This impact induces an horizontal shear force of 480 MN acting at 70 m above the foundation level; at the foundation level the corresponding forces are: shear force of 480 MN and overturning moment of 34 000 MNm.

Earthquake event

The bridge is located in one of the most seismic area in Europe. In the past 35 years three earthquakes exceeding 6.5 on the Richter scale have occurred in the Gulf of Corinth. The 1995 Aigion earthquake took place less than 30 km east of the site. Fig. 7 presents the epicenters of the major earthquakes felt in the Gulf of Corinth along with the major tectonic faults. The contract fixed for the design motion a return period of 2 000 years. A comprehensive seismic hazard analysis has defined the governing event as a 7.0 surface wave magnitude earthquake originating on the Psathopyrgos fault (shown with an arrow in Fig. 7) only 8.5 km east of the site (circle in Fig. 7).

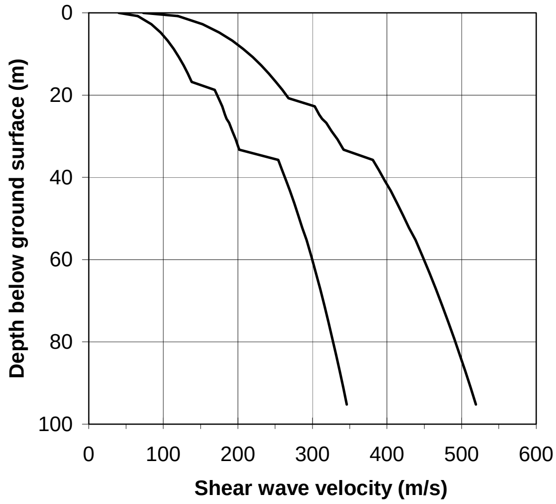

Fig. 5 : Shear wave velocity

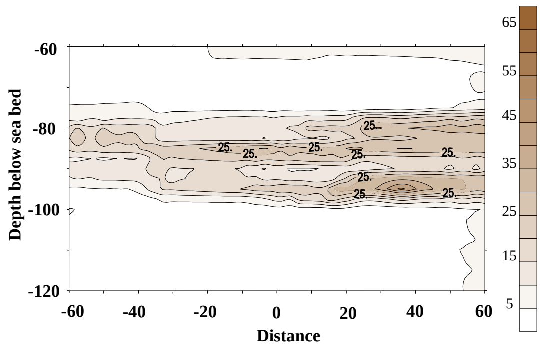

Fig. 6 : Spatial variability of cone resistance

In recognition of the influence of the soil characteristics on the ground surface motion, the design response spectrum at the sea bed elevation is defined from specific site response analyses based on the actual soil characteristics and on the design rock motion defined by the seismic hazard analysis. The 5% damped response spectrum is shown in Fig. 8: the peak ground acceleration is equal to 0.5g, the plateau at 1.2 g is extending from 0.2 s to 1.1 s and at 2 s the spectral acceleration is still high, equal to 0.62 g.

Tectonic movements

The seismic threat arises from the prehistoric drift in the earth's crust that shifted the Peloponese away from mainland Greece. The peninsula continues to move away from the mainland by a few millimeters each year. As a result the bridge must accommodate a 2 m differential tectonic displacement in any direction and between any two piers.

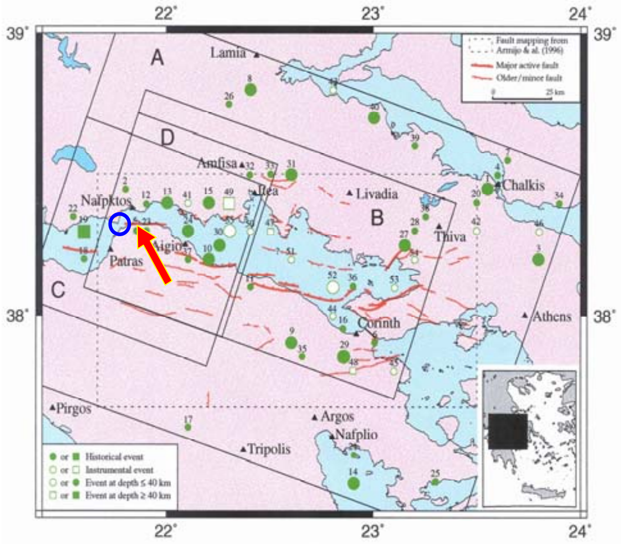

Fig. 7: Locations of major earthquakes

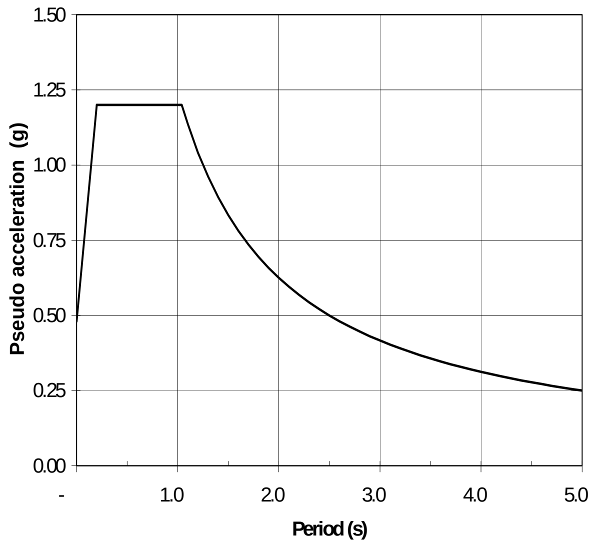

Fig. 8: Design ground surface response spectrum

MAIN BRIDGE FOUNDATIONS

Very soon during the design stage it appeared that the foundations design will be a major issue. On the one hand, the unfavorable geotechnical conditions with no competent layer at shallow depth, the large water depth, typical of depths currently encountered in offshore engineering and the high seismic environment represent a combination of challenging tasks. It was also realized that the earthquake demand will govern the concept and dimensioning of the foundations. On the other hand, the time allowed for design turned out to be a key factor: thanks to the contractor who decided to anticipate the difficulties, the design studies started one year ahead of the official effective date. Advantage was taken of this time lapse to fully investigate alternative foundation solutions, to develop and to validate the innovative concept that was finally implemented. The amount of time spent initially for the development of the design concept was worthwhile and resulted in a substantial saving for the foundation. In addition, the close cooperation that existed from the beginning within the design team between structural and geotechnical engineers, between the design team and the construction team on one hand, and between the design team and the design checker on the other hand, was a key to the success.

Investigated foundation solutions

After a careful examination of all the environmental factors listed above, no solution seems to dominate. Several solutions were investigated: piled foundation, caisson foundation, surface foundation. Piles were quickly abandoned for two reasons: the difficulty to realize the structural connection between the slab and the piles in a deep water depth, and the rather poor behavior of floating piles in seismic areas as observed in Mexico city during the 1985 Michoacan earthquake. Caissons foundations were hazardous due to the presence of a gravel layer at the ground surface (Fig. 3), which may induce some difficulties during penetration of the caisson. Surface foundation was clearly impossible in view of the poor foundation bearing capacity and of the high anticipated settlements. However, it was quickly realized that surface foundation was the only viable alternative from a construction point of view: construction techniques used for offshore gravity base structures are well proven and could easily be implemented for the Rion-Antirion bridge.

The problem posed by the poor soil conditions still remains to be solved. Soil improvement is required in order to ensure an adequate bearing capacity and to limit the settlements to acceptable values for the superstructure. Several techniques were contemplated from soil dredging and backfilling (soil substitution), to in situ treatment with stone columns, grouted stone columns, lime columns. The need for a significant high shear resistance of the improved soil and for a good quality control of the achieved treatment led to the use of driven steel pipes, a technique derived from offshore engineering, to reinforce the soil beneath the foundations. To prevent any confusion with piles foundations, which behave differently than steel pipes, those are named *inclusions*.

Adopted foundation concept

In order to alleviate potential damage to the structure due to the adverse environmental conditions and to carry the large earthquake forces brought to the foundation (shear force of the order of 500 MN and overturning moment of the order of 18 000 MNm for a vertical buoyant pier weight of 750 MN), the innovative foundation design concept finally adopted (Fig. 9) consists of a gravity caisson (90 m in diameter at the sea bed level) resting on top of the reinforced natural ground (Teyssandier 2002, Teyssandier 2003). The ground reinforcement (Fig. 10) is composed of steel tubular pipes, 2 m in diameter, 20 mm thick, 25 to 30 m long driven at a grid of 7 m x 7 m below and outside the foundation covering a circular area of approximately 8 000 m^2 The total number of inclusions under each foundation is therefore of the order of 150 to 200. In addition, as shown below, the safety of the foundation is greatly enhanced by interposing a gravel bed layer, 2.8 m thick, on top of the inclusions just below the foundation raft with no structural connection between the raft and the inclusions heads (Fig. 10). The reinforcement scheme is implemented under three of the four piers: M1, M2 and M3. Under pier M4, which rests on a 20 m thick gravel layer, soil reinforcement is not required and the caisson rests directly on a 1 m thick ballast layer without inclusions.

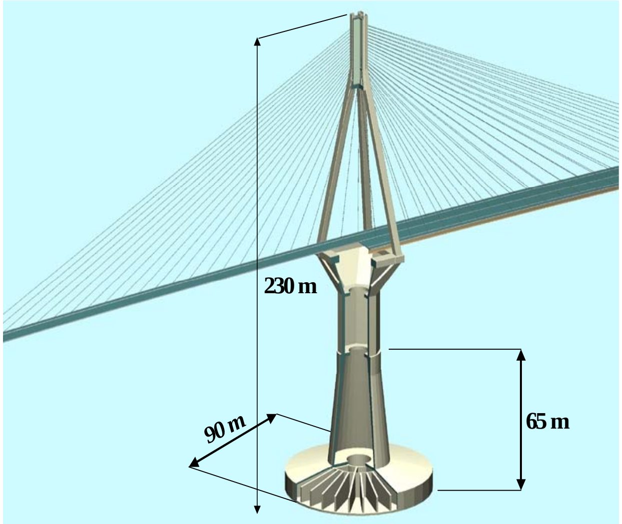

Fig. 9: View of one pylon of the Rion-Antirion bridge

The concept (inclusions plus gravel layer) enforces a capacity design philosophy in the foundation design (Pecker 1998). The gravel layer is equivalent to the "plastic hinge" where inelastic deformation and energy dissipation take place and the "overstrength" is provided by the ground reinforcement which prevents the development of deep seated failure mechanisms involving rotational failure modes of the foundation. These rotational failure modes would be very detrimental to the high rise pylon (230 m). If the design seismic forces were exceeded the "failure" mode would be pure sliding at the gravel-foundation interface; this "failure mechanism" can be accommodated by the bridge, which is designed for much larger tectonic displacements than the permanent seismically induced ones. The concept is also somehow similar to a base isolation system with a limitation of the forces transmitted to the superstructure whenever sliding occurs.

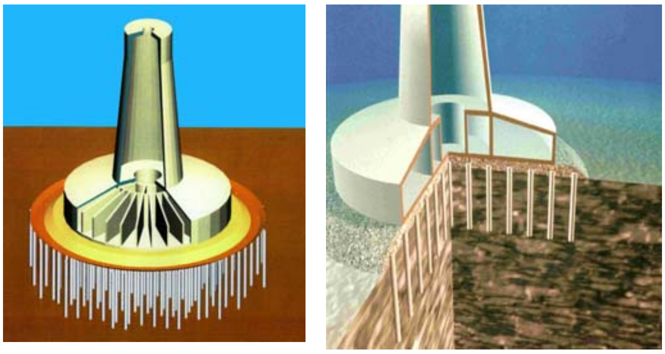

Fig. 10: Foundation reinforcement

JUSTIFICATION OF THE FOUNDATIONS

The foundations have to be designed with respect to static loads (dead loads, live loads and tectonic differential movements) and with respect to dynamic loads (ship impact and earthquake). With regards to the static loads, the main issue is the settlements evaluation: absolute settlement and differential settlements inducing tilt, to which the 230 m high pylon is very sensitive. With regards to the earthquake loads, several problems have to be solved: evaluation of the *seismic demand*, i.e. the forces transmitted to the foundation during an earthquake, and check of the *foundation capacity*, i.e. calculation of the ultimate bearing capacity and earthquake induced permanent displacements.

Static loads

The immediate settlement of the foundation is not a major concern since approximately 80% of the total permanent load (750 MN) is brought by the pier below the deck level. More important are the consolidation settlements of the clay strata which induces differed settlements. The settlements are computed with the classical one dimensional consolidation theory but the stress distribution in the soil is computed from a three dimensional finite element analysis in which the inclusions are modeled. It is found that the total load is shared between the inclusions for approximately 35 to 45 %

and the soil for the remaining 55 to 65 %. The average total settlement varies from 17 cm to 28 cm for Piers M1 to M3. For Pier M4 the computed settlement is much smaller, of the order of 2cm. The figures given above are the maximum settlements predicted before construction; they were computed assuming the most conservative soil characteristics, neglecting for instance the slight overconsolidation of the upper strata. Consolidation settlements were estimated to last for 6 to 8 months after completion of the pier.

The differential settlements are evaluated on the basis of the spatial variability of the compressibility characteristics assessed from the variability of the point cone resistances (Fig. 6). The computed foundation tilts range from 7. 10^{-4} to 1.8 10^{-3} radians, smaller than those that would have been computed without inclusions: the inclusions homogenize the soil properties in the top 25 m and transfer part of the loads to the deeper, stiffer, strata.

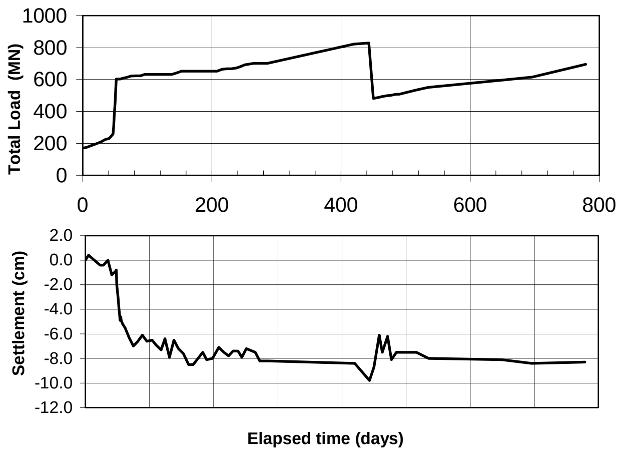

Fig. 11: Settlement versus time – Pier M3

Although the previous figures were acceptable, it was decided, in order to increase the safety of the structure, to preload the pier during construction. This preloading was achieved by filling the central part of the cone pier (Fig. 9) with water during construction to a total weight slightly larger than the final pier weight. Deballasting was carried out as construction proceeded, maintaining the total weight approximately at its final value. The beneficial effect of this preloading is readily apparent in Fig. 11, which shows the recorded settlement and applied load versus time during construction. The final settlement amounts to 8 cm as compared to 22 cm originally anticipated, and occurred more rapidly. This is clearly a beneficial effect of the overconsolidation, as back calculations have shown. More important the foundation tilts are almost negligible, less than 3 10^{-4}.

Seismic loads

The evaluation of the seismic behavior of the foundation requires that the seismic demand and the seismic capacity be calculated. However, as for any seismic design, it is important to first define the required performance of the structure. For the Rion Antirion bridge this performance criterion was clearly stated in the technical specifications: "The foundation performance under seismic loading is checked on the basis of induced displacements and rotations being acceptable for ensuring reusability of the bridge after the seismic event". In other words, it is accepted that the foundation experiences permanent displacements after a seismic event, provided the induced displacements remain limited and do not impede future use of the bridge. Full advantage of this allowance was taken in the definition of the design concept: sliding of the pier on the gravel layer is possible (and tolerated) but rotational mode of failures are prevented (and forbidden).

Seismic capacity

Because of the innovative concept and therefore of its lack of precedence in seismic areas, its justification calls for the development of new design tools and extensive validation. A very efficient three stages process was implemented to this end:

- Development of design tools based on a limit analysis theory to estimate the ultimate capacity of the foundation system and to define the inclusions layout: length and spacing. As any limit analysis method, this tool cannot give any indication on the induced displacements and rotations.
- Verification of the final layout with a non linear two, or three, dimensional finite element analysis. These analyses provide non only a check of the ultimate capacity but also the non linear stress strain behavior of the foundation that will be included in the structural model for the seismic calculations of the bridge.
- Experimental verification of the design tools developed in step 1 with centrifuge model tests.

As shown below all three approaches give results which are within ±15% of each other, which increases the confidence in the analyses performed for design.

The design tools are based on the Yield Design theory (Salençon 1983), further extended to reinforced earth media, (de Buhan and Salençon 1990). The kinematic approach of the Yield Design theory applied to mechanisms of the type shown in Fig. 12, (Pecker and Salençon 1999), permits the determination, for a wrench of forces (N vertical force, V horizontal force and M overturning moment) applied to the foundation, of the sets of allowable loads; this set defines in the loading space parameters a bounding surface with an equation:

$$\Phi(N,V,M)=0 \quad (1)$$

Any set of loads located within the bounding surface can be safely supported by the foundation, whereas any set located outside corresponds to an unsafe situation, at least for permanently imposed loads.

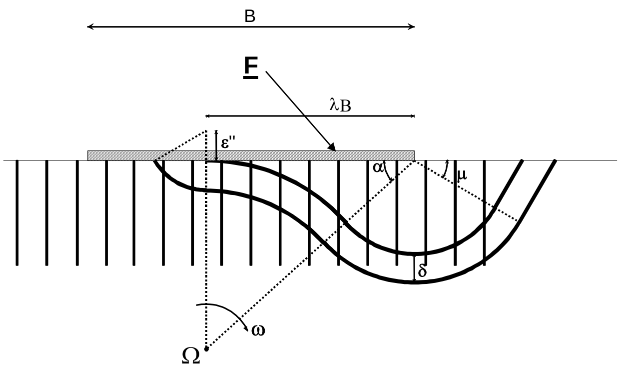

Fig. 12: Kinematic mechanism

Fig. 13 presents a cross section of the bounding surface by a plane N=constant, corresponding to the vertical weight of one pier. Two domains are represented in the figure: the smallest one correspond to the soil without the inclusions, the largest one to the soil reinforced with the inclusions. The increase in capacity due to the inclusions is obvious and allows the foundation to support significantly larger loads (V and M). Furthermore, the vertical ascending branch on the bounding surface, on the right of the figure, corresponds to sliding at the soil-foundation interface in the gravel bed layer. When moving on the bounding surface from the point (M=0, V=560 MN), sliding at the interface is the governing failure mechanism until the overturning moment reaches a value of approximately 20 000 MN; for larger values of the overturning moment, rotational mechanisms tend to be the governing mechanisms and the maximum allowable horizontal force decreases. The height of the vertical segment, corresponding to a sliding mechanism, is controlled by the inclusions layout and can therefore be adjusted as necessary. The design philosophy is based on that feature: for a structure like the pylon with a response governed by the fundamental mode, there is proportionality between M and V, the coefficient of proportionality being equal to the height of the center of gravity above the foundation. When V increases the point representative of the loads moves in the plane of Fig. 13 along a straight line passing through the origin, assuming that the vertical force is constant; it eventually reaches the bounding surface defining foundation failure.

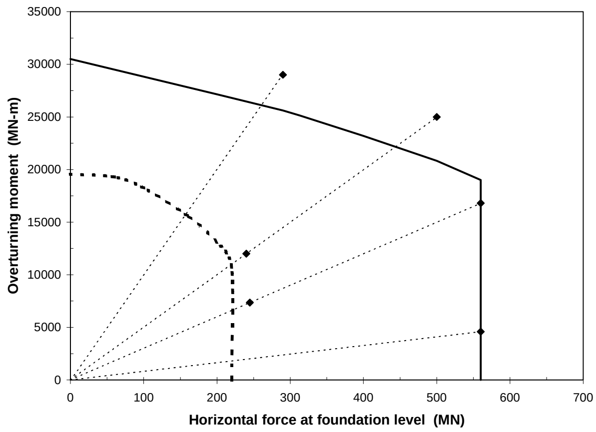

Fig. 13: Cross section of the bounding surface: Doted line without inclusions; solid line with inclusions

The inclusions layout is then determined in a way such that this point be located on the vertical ascending branch of the bounding surface.

The previous analysis provides the ultimate capacity of the foundation but no information on the displacements developed at that stage. These displacements are calculated from a non linear finite element analysis. Most of the calculations are performed with a 2D model, but some additional checks are made with a 3D model. For those calculations, the parameters entering the non linear elastoplastic soil constitutive model are determined from the laboratory tests carried out on the undisturbed soil samples; interface elements with limiting shear resistance and zero tensile capacity are introduced between, on one hand, each inclusion and the soil and, on the other hand, the raft and the soil. These models are loaded to failure under increasing monotonic loads such that M/V=constant (Fig. 14). Results are compared to those obtained from the Yield Design theory in Fig. 13; a very good agreement is achieved with differences of the order of ±12% for all the calculations made for the project. However, it is worth noting that the finite element analyses, because of the large computer time demand, could not have been used to make the preliminary design; the Yield Design theory is, in that respect, a more efficient tool: to establish the full cross section of the bounding surface represented in Fig. 10 requires only 10 to 15 minutes on a PC; a non linear finite element analysis, i.e. a single point of the curve, requires more than 4 hours of computing in 2D and 15 hours in 3D on a workstation with 4 parallel processors.

This totally innovative concept, at least in seismic areas, clearly calls for extensive theoretical analyses and experimental validation. As sophisticated as they can be, the theoretical and numerical tools do not have the capacity for modeling all the details of the behavior of this complex scheme during an earthquake. Centrifuge model tests were therefore undertaken with a three-fold objective:

- To validate the theoretical predictions of the ultimate bearing capacity of the foundation under monotonically increasing shear force and overturning moment,
- To identify the failure mechanism of the foundation under these combined loads,
- To assess the behavior of the foundation under various cyclic load paths.

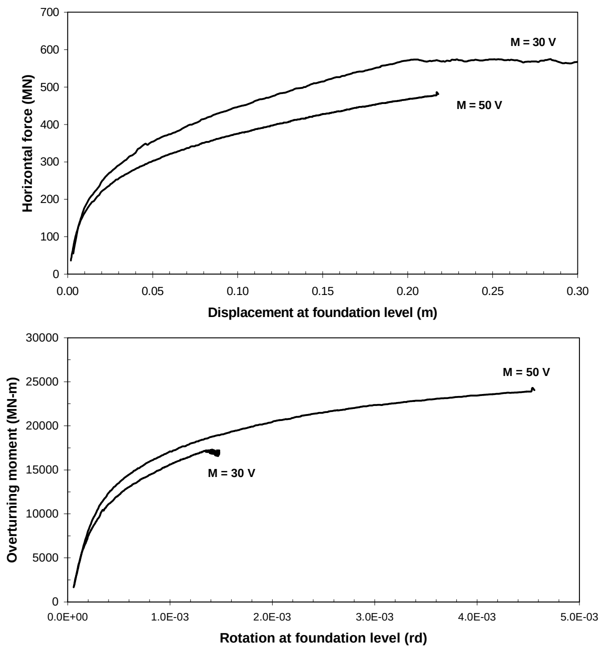

Fig. 14: Finite element analyses: force-displacement curves

Four tests have been performed in the 200 g-ton geotechnical centrifuge at the LCPC Nantes center, (Pecker and Garnier 1999). It is designed to carry a 2000 kg payload to 100 g accelerations, the bucket surface is at a radius of 5.5 m and the platform has a working space of 1.4 m by 1.1 m. All tests have been carried out at 100 g on models at a scale of 1/100. The dimensions of the corresponding prototype are as follows:

- radius of the circular footing: B_f = 30 m,
- inclusions length and diameter: L = 8.5 m and B=0.67 m,
- Wall thickness t = 6.7 mm (steel), Stiffness EI = 158 $MN.m^2$,
- Thickness of the ballast layer: 1.2 m.

The soil material has been sampled at the location of pier N17 of the Antirion approach viaduct and sent to the laboratory where it was reconsolidated prior to the tests to reproduce the in situ shear strength profile. The loads applied to the foundation consist of a constant vertical force and of a cyclic shear force and overturning moment; at the end of the tests the specimens are loaded to failure under monotonically increasing loads with a constant ratio M/V. The main findings of the test are summarized in Fig. 15 which compares the theoretical predictions of the failure loads (class A prediction according to the terminology introduced by Whitman) to the measured failure loads. Disregarding the preliminary tests that were carried out with another equipment (CESTA centrifuge in Bordeaux), all four tests yield values within ±15% of the predictions obtained with the Yield Design theory.

The centrifuge tests not only provide the ultimate loads but also valuable information on factors that either could not be easily apprehended by the analysis or need experimental verification. Of primary interest is the fact that, even under several cycles of loading at amplitudes representing 75% of the failure load, the foundation system does not degrade; no tendency for increased displacement with the number of cycles is noted. The equivalent damping ratios calculated from the hysteresis loops recorded during the tests are significant with values as high as 20%; it is interesting to note that these values have been confirmed by numerical analyses and can be attributed, to a large extent, to the presence of the inclusions (Dobry et al. 2003). Finally, a more quantitative information is given by the failure mechanisms observed in the centrifuge, which compare favorably either with the mechanisms assumed a priori in the Yield Design theory (Fig. 12), or with those computed from the non linear finite element analyses (Fig. 16).

The approach explained above takes care of the justification of the inclusions. Another important issue is the behavior of the gravel bed layer; a fundamental requirement for this layer is to exhibit a well defined and controlled shear resistance, which defines the ultimate capacity for the sliding modes of failure. This requirement can only be achieved if no pore pressure buildup occurs in the layer during the earthquake; the development of any excess pore pressure means that the shear resistance is governed by the soil undrained cyclic shear strength, a parameter highly variable and difficult to assess with accuracy. The grain size distribution of the gravel bed layer (10-80 mm) is therefore chosen to ensure a fully drained behavior (Pecker and *al* 2001). Furthermore, the friction coefficient between the slab and the gravel has been measured on site with friction tests using a concrete block pushed on top of the gravel bed layer; a rather stable value (0.53 to 0.57) has been measured for that coefficient.

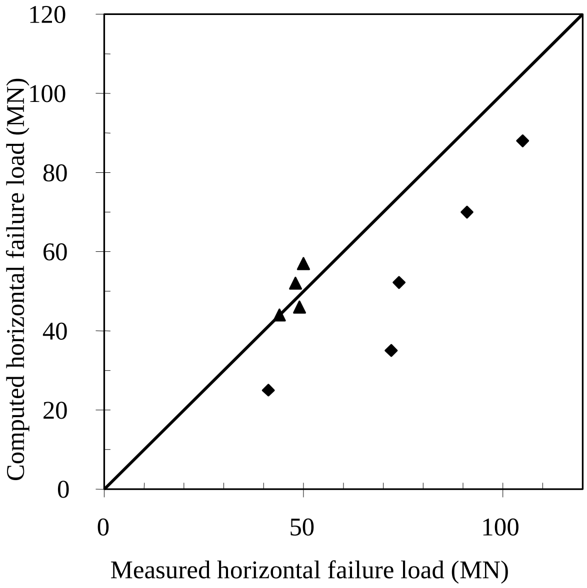

Fig. 15: Computed versus measured failure loads in the centrifuge tests:
Preliminary tests (diamonds); final tests (triangles)

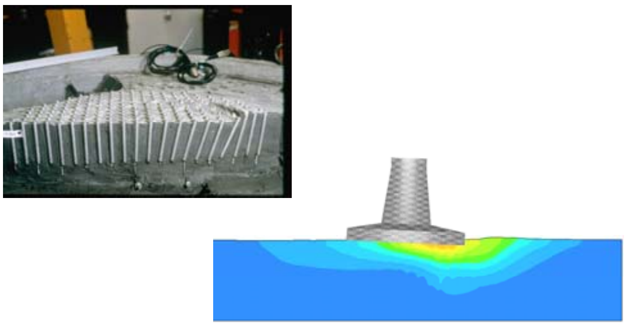

Fig. 16: Failure mechanisms: centrifuge test (upper left), FE analysis (lower right)

Seismic demand

For the seismic analyses it is mandatory to take into account soil structure interaction, which is obviously significant given the soft soil conditions and large pier mass. In the state of practice, the action of the underlying soil is represented with the so-called impedance functions, the simplified version of them consisting, for each degree of freedom, of constant springs and dashpots. In the calculation of these springs and dashpots the soil modulus is calibrated on the free field strains, neglecting any further non linearity developed in the vicinity of the foundation (Pecker and Pender 2000). Such an approach may be inadequate because it implicitly assumes that the forces generated on the foundation are independent of its yielding. Therefore there is some inconsistency in checking the foundation capacity for those forces. Recent studies, (e.g., Paoluci 1997, Pedretti 1998, Cremer et al. 2001, Cremer et al. 2002), throw some light on this assumption and clearly show that close to the ultimate capacity it is no longer valid. This is typically the situation faced for the foundation of the Rion-Antirion bridge. Therefore it was decided to implement a more realistic approach which closely reflects the physics of the interaction. Since numerous parametric studies are necessary for the design, a dynamic non linear soil finite element model is not the proper tool.

In the same spirit as the impedance functions, the action of the soil (and inclusions) below the foundation slab is represented by what is called a macro-element (Cremer and *al* 2001, Cremer and *al* 2002). The concept of macro-element is based on a partitioning of the soil foundation into (Fig. 17):

- A near field in which all the non linearities linked to the interaction between the soil, the inclusions and the slab are lumped; these non linearities are geometrical ones (sliding, uplift of the foundation) or material ones (soil yielding). The energy dissipation is of hysteretic nature.
- A far field in which the non linearities are governed by the propagation of the seismic waves; the energy dissipation is essentially of a viscous type.

In its more complete form the macro-element model couples all the degrees of freedom (vertical displacement, horizontal displacement and rotation) (Cremer et al. 2001). For the design studies of the bridge a simplified version of it, in which the degrees of freedom are uncoupled, is used. Conceptually this element can be represented by the rheological model shown at the bottom of Fig.17; it consists of an assemblage of springs and Coulomb sliders for the near field and of a spring and a dashpot for the far field. One such model is connected in each direction at the base of the structural model; the parameters defining the rheological model are calibrated on the monotonic force-displacement and moment-rotation curves computed from the non linear static finite element analyses (Fig. 14). For unloading-reloading a Masing type behavior is assumed; the validity of this assumption is backed up by the results of the centrifuge tests (Pecker and Garnier 1999) and of cyclic finite element analyses (Dobry et al. 2003). The adequacy of the macro-element to correctly account for the complex non linear soil structure interaction is checked by comparison with few dynamic non linear finite element analyses including a spatial modeling of the foundation soil and inclusions.

Fig. 18 compares the overturning moment at the foundation elevation calculated with both models; a very good agreement is achieved, not only in terms of amplitudes but also in terms of phases.

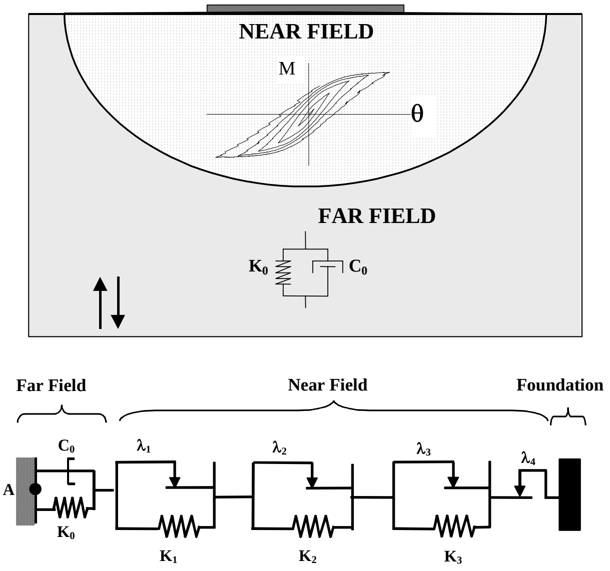

Fig. 17: Macro element for soil structure interaction

This model has been subsequently used by the structural design team for all the seismic analyses of the bridge. It allows for the calculations of the cyclic and permanent earthquake displacements; Fig.19 shows the displacement of the foundation center (bottom) and the variation of the forces in the (N-V) plane (top); sliding in the gravel bed layer occurs at those time steps when $V = N \tan\phi$.

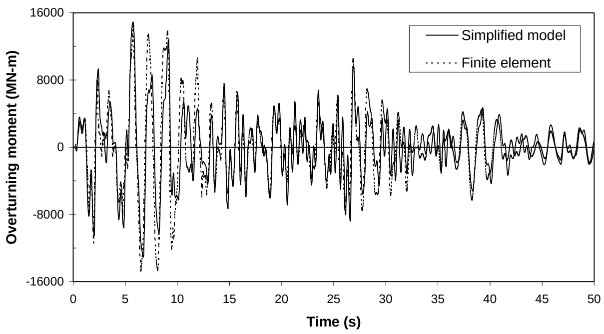

Fig. 18: Time history of foundation overturning moment

CONSTRUCTION METHODS

The construction methods for the foundations, described in details by Teyssandier (2002 and 2003), are those commonly used for the construction of offshore concrete gravity base structures:

- construction of the foundation footings in a dry dock up to a height of 15 m 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 its final position.

However some features of this project make the construction process of its foundations quite exceptional.
The dry dock has been established near the site. It was 200 m long, 100 m 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.

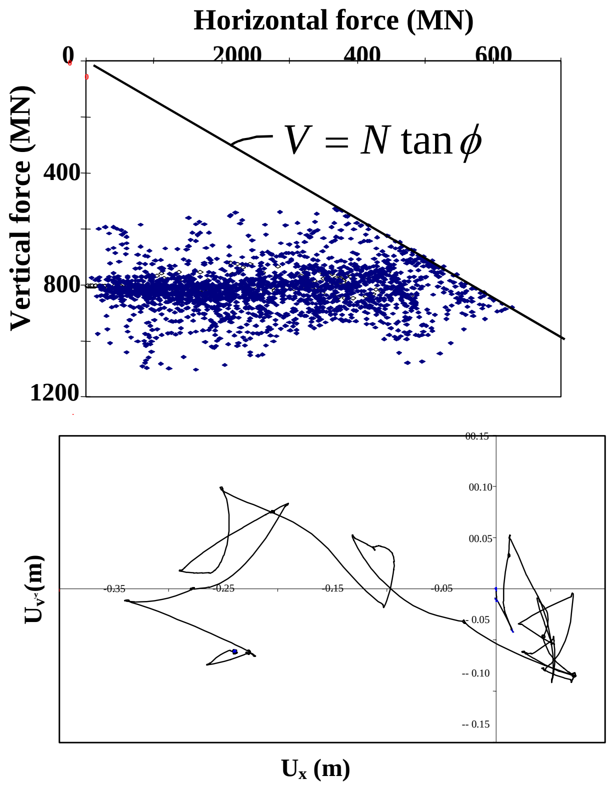

Fig. 19: Time history of foundation displacement
Top – variation of forces in the (N-V) plane
Bottom - displacement of foundation center

Dredging the seabed, driving of inclusions, placing and levelling the gravel layer on the top, with a water depth reaching 65 m, 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 (Fig. 20). 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 fixed 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 foundations are towed then sunk at their final position. Compartments created in the footings by the radial beams can be used to control tilt by differential ballasting. Then the foundations are filled with water to accelerate settlements. This pre-loading was maintained during pier shaft and pier head construction, thus allowing a correction for potential differential settlements before erecting pylons.

The deck of the main bridge is erected using the balance cantilever technique, with prefabricated deck elements 12 m long comprising also their concrete slab (another unusual feature).

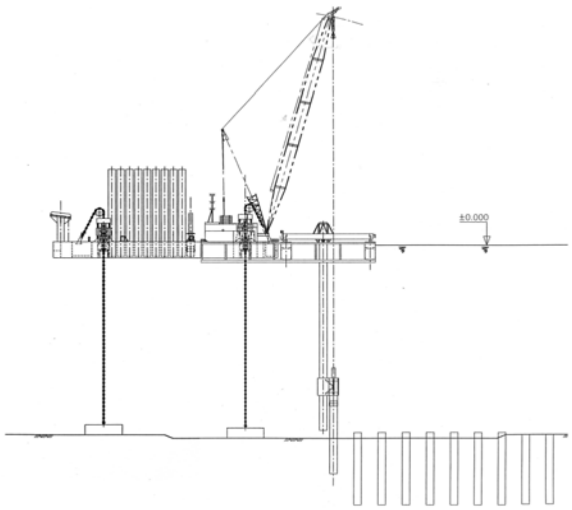

Fig. 20: Tension-leg barge

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More information on the project can be found on the web site www.gefyra.gr.

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