ASEISMIC FOUNDATION DESIGN PROCESS
LESSONS LEARNED FROM TWO MAJOR PROJECTS :
THE VASCO DA GAMA AND THE RION-ANTIRION BRIDGES

Alain PECKER
Géodynamique et Structure, 157 Rue des Blains 92220 BAGNEUX, France

1 INTRODUCTION

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. These aspects will be reviewed and illustrated by comparison of the solutions adopted for two major European cable stayed bridges: the Vasco da Gama bridge in Lisbon (Portugal) and the Rion-Antirion bridge, near Patras (Greece). The foundations solutions for those two structures are described and an attempt is made to pinpoint the major factors that have guided the final choices. The innovative foundation concept finally adopted for the Rion-Antirion bridge is further described in details to highlight how capacity design principles, which are familiar to structural engineers, can be implemented in earthquake resistant design of foundations.

2 ASEISMIC FOUNDATION DESIGN PROCESS

The aseismic design process for foundations is a very broad activity requiring the synthesis of insight, creativity, technical knowledge and experience [1]. Information is required and decisions have to be made at various stages including:

(i) the geological environment and geotechnical characterization of the soil profile;
(ii) the definition of the loads that will be applied to the foundation soil by the facility to be constructed;
(iii) information about the required performance of the structure;
(iv) investigation of possible solutions with evaluation of load capacity, assessment of safety factors and estimates of deformations;
(v) consideration of construction methods and constraints that need to be satisfied (finance and time);
(vi) exercise of judgment to assess potential risks.

Obviously the process described above is not a linear progression. Several iterations may be required, at least from step ii to step vi, before arriving at a feasible, reliable and economic design. But, it must also be realized that some of the above step cannot be necessarily treated independently from the others, in an iterative loop fashion. There may be a close interaction between them which is sometimes overlooked in the state of practice.

For instance, since the early days earthquake geotechnical engineers have focused their attention on the non linear behavior of soils and on the evaluation of the seismic forces and cyclic deformations of foundations. This was clearly dictated by the need for an accurate evaluation of the soil-structure interaction forces which govern the structural response (seismic demand). It is only during the last decade that seismic bearing capacity problems (evaluation of foundation capacity) have been tackled. These studies have clearly been motivated by the foundation failures observed in the Mexico City (1985) and Kobe (1995) earthquakes and by the ever increasing challenge of designing foundations in difficult conditions and for extreme earthquake loads. It is however important to realize that the state of practice for foundation design, as implemented in every seismic building code, consists in uncoupling the seismic demand evaluation from the foundation capacity check. One of the strong assumptions underlying this practice is the independence between the computed design actions and soil yielding. The design actions are computed assuming quasi-linear foundation behavior although it is recognized that partial yielding of the foundation may affect the forces [2]. Recently attempts have been made to couple both phenomena : a solution has been developed for monotonic static loading based on the concept of a macro-element modeling the soil and foundation [3]; this concept of macro-element expressed in global variables at the foundation level has been extensively used in mechanics but seldom applied to soil-structure interaction. It has been extended to seismic loading and has now reached a point of development where they can be implemented in design with different degrees of complexity [2], [4], [5], [6]. It is interesting to note that these concepts have effectively been implemented for the Rion-Antirion bridge, whose foundations would have been otherwise over designed, and possibly not feasible.

Emphasis has been put just above on the computational aspects and the temptation may be great for designers to dedicate most of their efforts to the evaluation of the demand and capacity (steps ii and iv), neglecting the other aspects. This is obviously more comfortable because design tools and methodology are more or less standardized, and previous experience can be easily utilized. However all the other items are equally important and certainly more difficult to deal with because they require considerable experience, judgment and also continuous interaction between the geotechnical and structural designers, the construction company and the bridge owner. Aside from the issue of soil characterization, which is beyond the scope of this paper, probably the most difficult parameters to assess are those listed under items iii and v. The required collaboration between all the entities involved in the design and construction of a bridge is certainly most efficiently achieved within the framework of BOT contracts, as those implemented for the Vasco da Gama and Rion-Antirion bridges. In the sequel we will show for these two bridges, constructed in somewhat similar geotechnical and seismic environmental conditions, eventually present totally different foundation schemes as a result of the due consideration given to steps ii to vi.

3 MAIN FEATURES OF THE BRIDGES

The Vasco da Gama bridge, with a total offshore length of 12.3 kilometers, is divided into five sections: the North viaduct, 488 m long, the Expo viaduct, 672 m long, the main bridge which is a one span cable stayed bridge, 820 m long with a central span of 420 m, the Central viaduct, 6531 m long, and the South viaduct, 3825 m long [7]. The bridge is built across the Tagus river in Lisbon, in order to increase the traffic capacity of the Ponte 25 de Abril. The call for tender was launched in 1992, and the contract awarded to Novaponte, a consortium led by the French company Campenon Bernard SGE, now merged into the Vinci group, in April 1994; construction started in December 1994 and the bridge was opened to traffic in March 1998. Only the main cable stayed bridge will be considered in the sequel.

The Rion-Antirion bridge in Greece will span a total length of 2883 m, which includes a five spans cable-stayed bridge 2252m in length and two approach viaducts [8], [9]. When completed in 2004, it will be the longest cable-stayed bridge in the world. Crossing a 2500 m stretch of water, the bridge will consist of a cable stayed main bridge, 2252 m long and two approach viaducts. The main bridge will have two side spans 286 m long and three middle spans of 560 m. The call for tender was launched in 1992, the contract awarded to a consortium led by the French company Vinci construction in 1996 took effect in December 1997. Construction started in 1998 and is scheduled to be completed in 2004. As for the Vasco da Gama bridge we will focus our discussion on the main bridge.

4 ENVIRONMENTAL CONDITIONS

The main features related to the geotechnical and seismic environmental factors are briefly summarized below, limiting the presentation to those parameters required for earthquake design.

4.1 Geotechnical soil profiles

Both sites have 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. Based on the results of these investigations representative soil profiles have been defined at the locations of each bridge.

For the Vasco da Gama bridge, the soil profile consists of the superposition of two main geological units: a Holocene alluvial deposit, 65 m to 80 m thick, composed from the river bed level of soft marine clay (30 m to 40 m), fine to medium sand, silty clay, medium to coarse sand and peebles. These formations overlay the Pliopleistocene bedrock composed of overconsolidated sand and marls. This bedrock dips gently to the east with a dip angle of 5 to 7° (Figure 1). One important feature of the site is its bathymetry: except for the navigation channels, along the major length of the bridge alignment and especially at the location of the two main bridge pylons, the ground surface emerges at low tide.

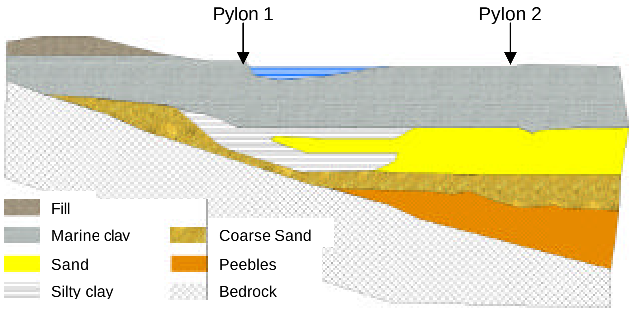

Fig. 1, Vasco da Gama bridge : schematic soil profile.

The undrained shear strength of the marine soft clay varies from a few kN/m^2 at the river bed elevation to 40 kN/m^2 at the base of the layer (Figure 3). The sandy layers are not prone to liquefaction and their friction angles are between 36° and 40°. For the seismic analyses two sets of values are adopted for the elastic shear wave velocities: a lower bound velocity and an upper bound one (Figure 4).

For the Rion-Antirion bridge 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 (Figure 2). No bedrock was encountered during the investigations and from geological studies and geophysical surveys its depth is believed to be greater than 500 m.

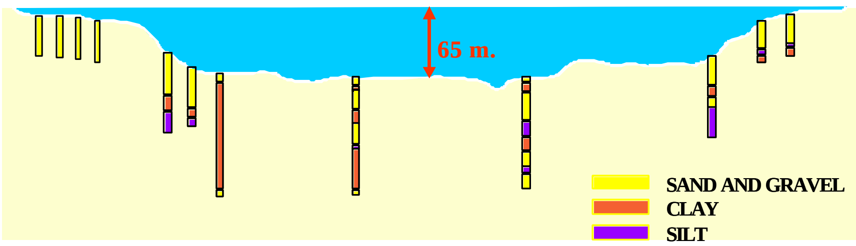

Fig. 2, Rion Antirion bridge : schematic soil profile.

For the seismic analyses, like for the Vasco da Gama bridge, two sets of characteristics (upper bound and lower bound) are defined for the undrained shear strength and the elastic shear wave velocities (Figures 3 and 4). As seen from figures 3 and 4 the soil characteristics are in the same range as those found in the Vasco da Gama bridge site in the top 30 meters. 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. Another important feature of the site is the large water depth, 65 m, found along the bridge alignment.

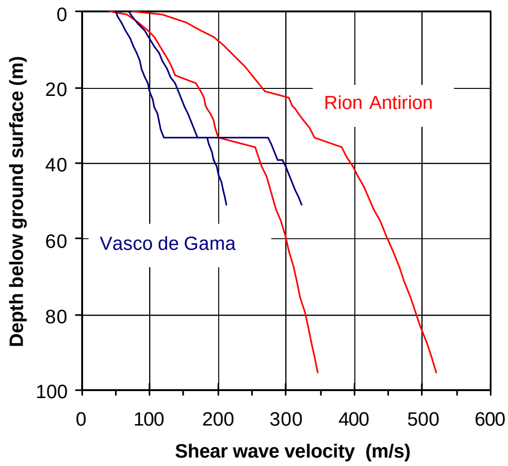

Fig. 3, Shear strength profile : Vasco da Gama (blue); Rion Antirion (red)

It is worth noting that for both sites the use of two sets of soil characteristics, instead of a single set based on characteristic values, is guided by the fundamental necessity to maintain compatibility in the whole design process: the seismic demand is calculated assuming one set of properties (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.

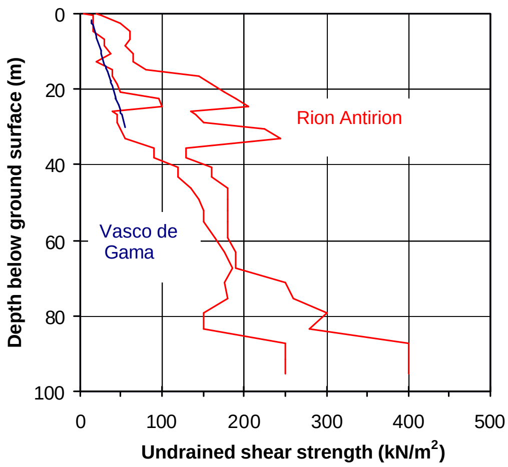

Fig. 4, Shear wave velocity profile : Vasco da Gama (blue); Rion Antirion (red)

4.2 Design earthquakes

In recognition of the influence of the soil characteristics on the ground surface motion, the design response spectra at the river bed, or sea bed, elevation are for both sites defined from specific site response analyses based on actual soil characteristics and an assumed rock motion. For the Vasco da Gama bridge, the design earthquake is governed by a nearby 7.5 surface wave magnitude event. For the Rion Antirion bridge the design earthquake is a 7.0 surface wave magnitude event originating approximately 8 kilometers east of the site. Both 5% damped response spectra are shown in figure 5.

Both design motions have a peak ground acceleration of 0.5g, a similar amplitude at the plateau (1.1g and 1.2 g) but the width of the plateau is significantly larger for the Rion Antirion site. In the long period range, above 1.5 s, both spectra are alike.

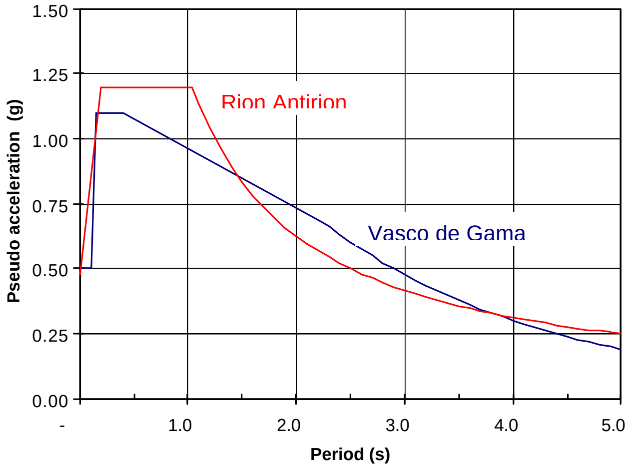

Fig. 5, Design ground surface response spectra : Vasco da Gama (blue); Rion Antirion(red)

5 FOUNDATIONS FOR THE VASCO DA GAMA BRIDGE

Several factors guided the choice of the foundation concept for the Vasco da Gama bridge. The first are of purely technical origin:

- The soil profile presents a rather favorable configuration with a competent rocklike formation at reasonable depth (~70m);
- The site bathymetry, with emergence of the ground surface at low tide, makes the construction of the foundation slab easy;
- The required performance of the foundation (elastic behavior) under earthquake loading precludes the development of permanent displacements or deformations.

The other ones are more related to economical issues and also have a profound impact on the final choice; these factors are:

- The relatively short time between the effective date of the contract and the start of construction that precludes the search for innovative solutions, which would require lengthy studies and validations;
- The short time allowed for construction requires the implementation of well proven and documented techniques.

All these factors naturally oriented the choice towards end bearing piles. However different alternatives are still to be examined: should the piles be open ended driven piles, concreted after driving, or should they be bored concrete piles? In view of the large seismic forces developed at foundation level, should the piles be inclined or vertical? All these issues were carefully examined and for economical reasons and safety issues the following scheme has finally been adopted for the main bridge piers. The foundation consists of vertical large diameter bored concrete piles, cased in the upper soft marine deposits; 44 piles, 2.20 m in diameter, are drilled under each pylon (Figure 6). The rationale behind this choice, [7], is guided considering that a lot of experience exists in Portugal with this construction technique, the large axial forces in the piles call for a high bearing capacity which cannot be easily obtained with driven piles, the casing prevents borehole collapse during drilling and also increases confinement of the concrete in the vicinity of the pile-slab connection where the bending moments and shear forces are larger; finally vertical piles are favored against inclined piles because of the large post earthquake bending moments developed in the inclined piles due to soil settlement and of the lack of ductility of raked piles [1].

Finally, although it does not have a major impact on the choice, the use of a piled foundation simplifies the design and therefore diminishes its duration and cost, and prevents unforeseen hazards that may be faced with less proven foundation layouts: theoretical methods and numerical tools exist for the seismic design of vertical piles groups and were validated beforehand.

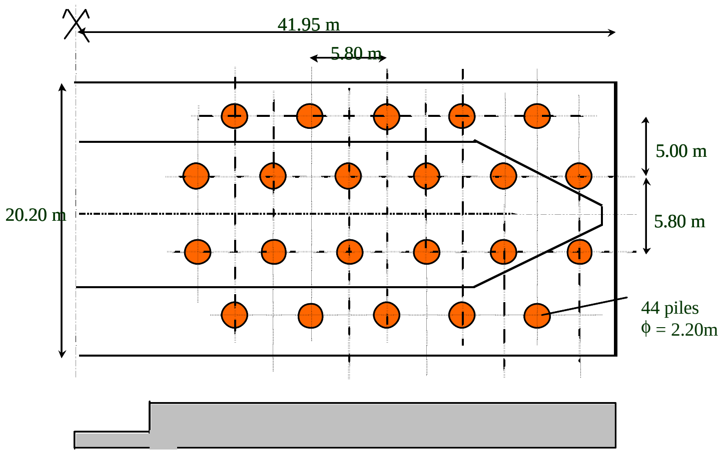

Fig.6, Vasco da Gama bridge: Layout of the piles under one pylon (only half of the foundation is shown)

To enhance the fact that the choice of a foundation scheme is strongly influenced by several factors, it is worth noting that the same scheme is not use along the whole bridge: the main bridge, the Central viaduct (navigation channels) and the South viaduct are founded on bored piles; the Central viaduct (outside the navigation channels), where large construction rates are required, is founded on driven piles.

6 FOUNDATIONS OF THE RION-ANTIRION BRIDGE

When compared to the Vasco da Gama bridge, the geotechnical conditions are far less favorable, with no competent layer at shallow depth, the water depth is larger, typical of depths currently encountered in offshore engineering; on the other hand, the required performance of the foundation is less demanding because permanent displacements are accepted for the design earthquake "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". A factor, which is not obvious at first but turned out to be very important, is the time allowed for the design: 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. It can be stated that 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. These two factors (time constraint and full interaction between all the parties involved) are not technical factors per se, but were certainly of the utmost importance.

6.1 Investigated foundation solutions

After a careful examination of all the environmental factors listed above, no solution seems to dominate. Therefore 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 (Figure 2), 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*.

Fig.7, View of one pylon of the Rion-Antirion bridge

6.2 Description of the foundation improvement

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 consists of a gravity caisson (90 m in diameter at the sea bed level) resting on top of the reinforced natural ground [8], [9], (Figure 7). 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 6500 m^2 to 8500 m^2 depending on the foundation pier. 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, 3.0 m thick, on top of the inclusions just below the foundation raft with no structural connection between the raft and the inclusions heads (Figure 8).

6.3 Role of the inclusions

The concept (inclusions plus gravel layer) enforces a capacity design philosophy in the foundation design [10]. The gravel layer is equivalent to the "plastic hinge" where inelastic deformation and 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. It therefore fulfils one of the objective listed in paragraph 2, i.e. it permits a sound assessment of the potential risks.

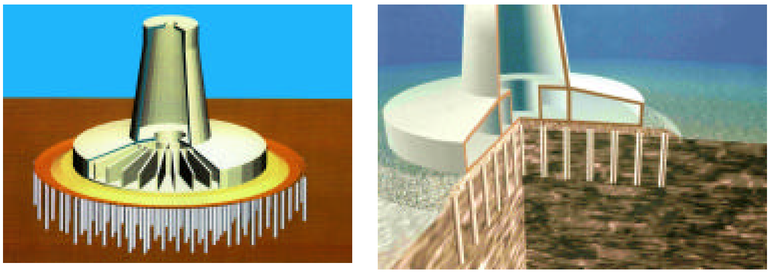

Fig.8, Rion-Antirion bridge improved foundation

6.4 Justification of the foundation

Because of the innovative concept and therefore 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:

(a) 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.

(b) 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.

(c) Experimental verification of the design tools developed in (a) 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 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 [16]. 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.

6.4.1 Foundation ultimate capacity

The design tools are based on the Yield Design theory [11], further extended to reinforced earth media, [12]. The kinematic approach of the Yield Design theory applied to mechanisms of the type shown in figure 9, [13], 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.

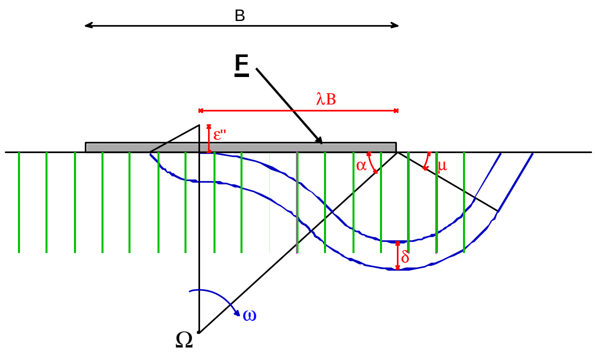

Fig.9 Kinematic mechanism

Figure 10 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 on 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 figure 10 along a straight line passing through the origin, assuming that the vertical force is constant; it eventually reaches the bounding surface defining foundation failure. The inclusions layout is then determined in a way that this point be located on the vertical ascending branch of the bounding surface.

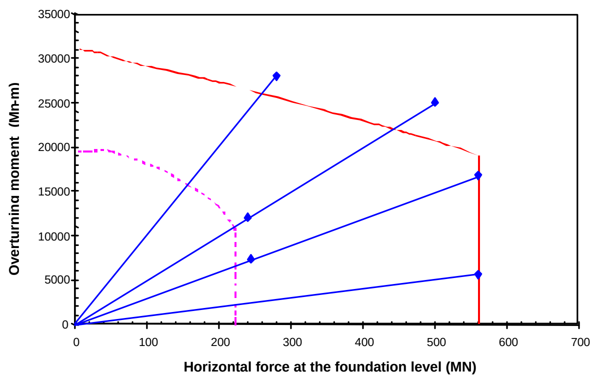

Fig.10 Cross section of the bounding surface:

Doted line without inclusions; solid line with inclusions

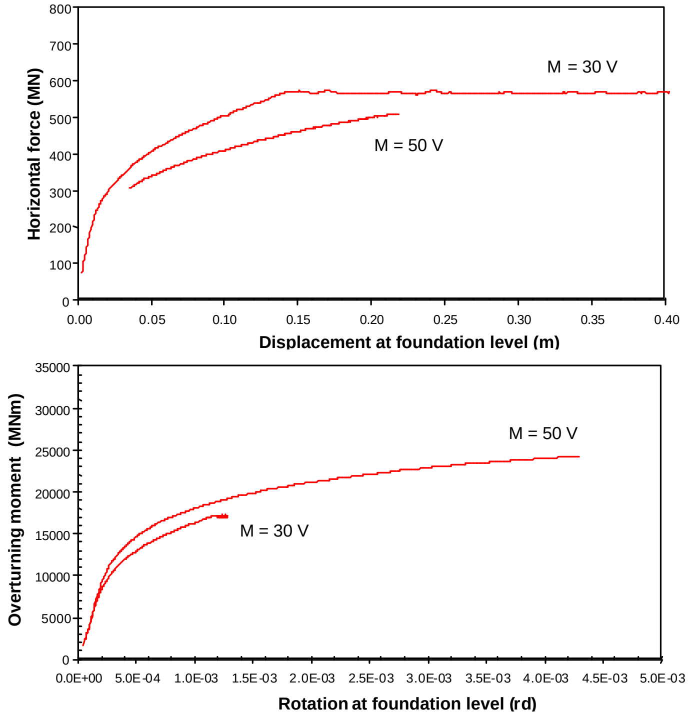

Fig.11 Finite element analyses: force-displacement curves

6.4.2 Finite element analyses

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 (figure 11). Results are compared to those obtained from the Yield Design theory in figure 10; 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 figure 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.

6.4.3 Centrifuge tests

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:

(a) To validate the theoretical predictions of the ultimate bearing capacity of the foundation under monotonically increasing shear force and overturning moment,

(b) To identify the failure mechanism of the foundation under these combined loads,

(c) To assess the behavior of the foundation under various cyclic load paths.

Four tests have been performed in the 200 g-ton geotechnical centrifuge at the LCPC Nantes center, [14]. 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 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 figure 12 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.

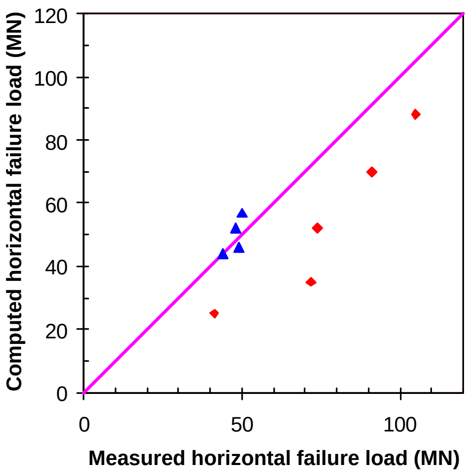

Fig.12 Computed versus measured failure loads in the centrifuge tests: Preliminary tests (red diamonds); final tests (blue triangles)

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 [15]. 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 (e.g. Figure 9), or with those computed from the non linear finite element analyses (Figure 13).

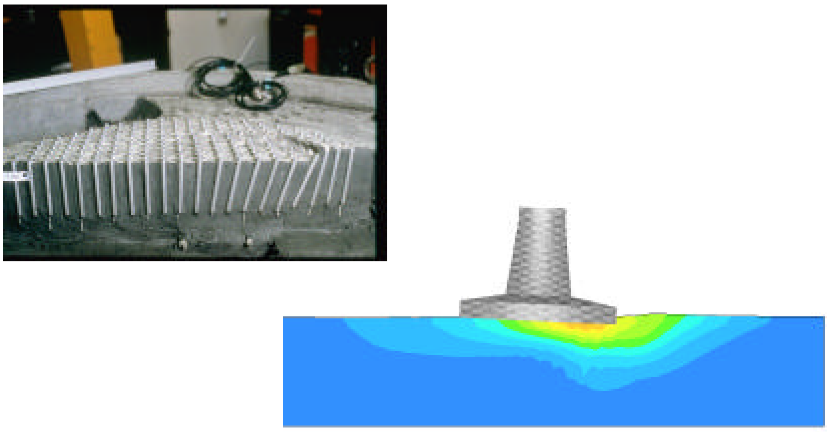

Fig13. Failure mechanisms: centrifuge test (upper left), FE analysis (lower right)

6.5 Soil structure interaction analysis

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 [1]. 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, [2], [4], [5], [6], throw some light on this assumption and clearly show that when 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 [5], [6]. The concept of macro-element is based on a partitioning of the soil foundation into (Figure 14):

- 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) [5]. 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 figure 14; 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 (Figure 11). For unloading-reloading a Masing type behavior is assumed; the validity of this assumption is backed up by the results of the centrifuge tests [14] and of cyclic finite element analyses [15]. 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. Figure 15 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.

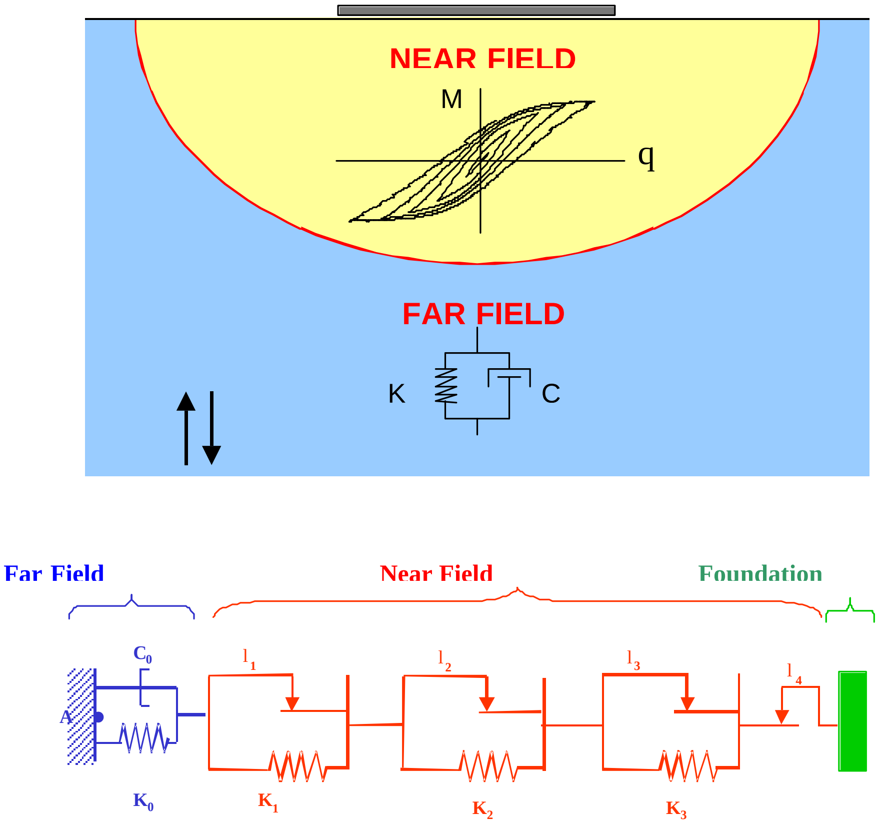

Fig.14 Macro-element for soil structure interaction

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; figure 16 shows, on the right, the displacement of the foundation center and, on the left, the variation of the forces in the (N-V) plane; sliding in the gravel bed layer occurs at those time steps when $V = N \tan f$.

7 CONCLUSIONS

The design of the foundations of two major bridges, located in highly seismic areas, has been used to illustrate that earthquake foundation design is not a single process that starts from one end and finishes at the other. Many factors, either technical or economical, have a marked influence on the design process and require several back and forth iterations to fully account for all of them. Most notably, it has been shown how the time constraint, a non technical factor, may have a profound impact: a short time allocated for design precludes the development of original solutions, whereas advantage may be taken of a longer time lapse to investigate new areas. The Rion Antirion bridge is exemplar in that respect: the extra year, ahead of the contractual starting date, was fully employed to develop and to validate a very innovative foundation concept, without the pressure coming from the construction site. This 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.

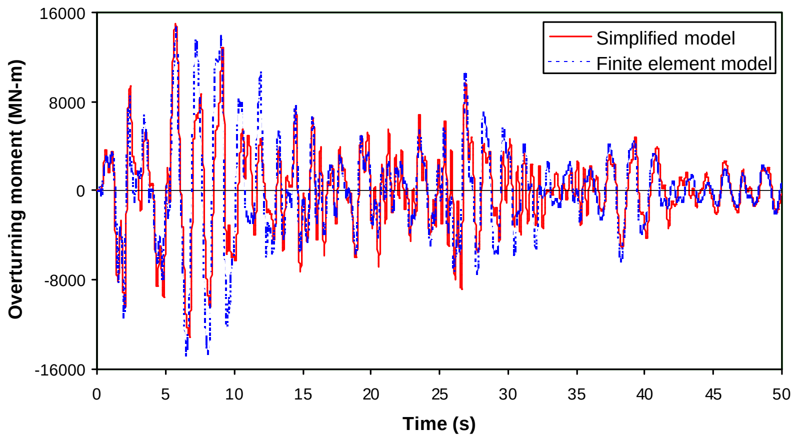

Fig15. Time history of foundation overturning moment

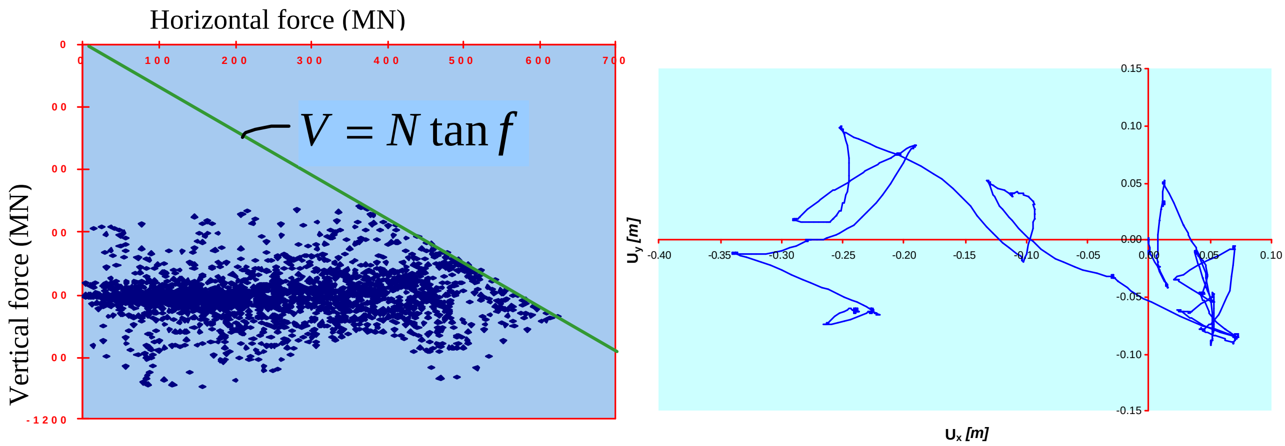

Fig16. Time history of foundation displacement

REFERENCES

[1] Pecker, A., Pender, M.J. (2000). Earthquake resistant design of foundations : new construction. *GeoEng2000* **(1),** Invited paper. 313-332.

[2] Paolucci, R. (1997). Simplified evaluation of earthquake induced permanent displacement of shallow foundations. *Journal of Earthquake Engineering,* **(1)**, 3, 563-579.

[3] Nova, R., Montrasio, L. (1991). Settlements of shallow foundations on sand. *Geotechnique* **(41)**, 2, 243-256.

[4] Pedretti, S. (1998). Nonlinear seismic soil-foundation interaction: analysis and modeling method. PhD Thesis Dpt Ing Structurale, Politecnico di Milano.

[5] Cremer, C., Pecker, A., Davenne L. (2001). Cyclic macro-element for soil structure interaction - Material and geometrical non linearities. *Numerical Methods in Geomechanics,* **(25)**, 1257-1284.

[6] Cremer, C., Pecker, A., Davenne L. (2002). Modelling of non linear dynamic behaviour of a shallow strip foundation with macro-element. *Journal of Earthquake Engineering,* **(6)**, 2, 175-212.

[7] Wastiaux, M., Ducroq, J., Corbetta, F. (1999). Les pieux maritimes du pont Vasco de Gama. *Revue Française de Géotechnique* **(87)**, 27-33.

[8] Teyssandier, J.P.(2003). The Rion-Antirion bridge, Design and construction. *ACI International Conference on Seismic Bridge Design and Retrofit.* La Jolla, California.

[9] Teyssandier, J.P.(2002). Corinthian Crossing. *Civil Engineering,* **(72)**, 10, 43-49.

[10] Pecker, A. (1999). Capacity Design Principles For Shallow Foundations In Seismic Areas *Keynote lecture, Proceedings XIth European. Conference. on Earthquake Engineering*. Balkema.

[11] Salençon, J. (1983). Introduction to the yield design theory. *European Journal of Mechanics A/Solids* (**9**), 5, 477-500.

[12] de Buhan, P., Salençon, J. (1990). Yield strength of reinforced soils as anisotropic media. *in Yielding, Damage and Failure of Anisotropic Solids;* EGF5, *Mechanical Engineering Publications*. London.

[13] Pecker, A., Salençon, J. (1999). Ground Reinforcement In Seismic Area. *Proceedings of the XI Panamerican Conference on Soil Mechanics and Geotechnical Engineering*. Iguasu, 799-808.

[14] Pecker, A., Garnier, J. (1999). Use of Centrifuge Tests for the Validation of Innovative Concepts in Foundation Engineering. *Proceedings 2nd International Conference on Earthquake Geotechnical Engineering;* Lisbon, 433-439.

[15] Dobry, R., Pecker, A., Mavroeidis, G., Zeghal, M., Gohl, B., Yang, D. (2003). Damping/Global Energy Balance in FE Model of Bridge Foundation Lateral Response. *Journal of Soil Dynamics and Earthquake Engineering*, **(23)**, 6, 483-495.

[16] Pecker, A., Prevost, J.H., Dormieux, L. (2001). Analysis of pore pressure generation and dissipation in cohesionless materials during seismic loading. *Journal of Earthquake Engineering*, **(5)**, 4, 441-464.