STRUCTURAL RESPONSE OF THE RION-ANTIRION BRIDGE

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SUMMARY

The Rion Antirion Bridge will cross the Gulf of Corinth near Patras, in western Greece. It consists mainly of a very impressive multi cable-stayed span bridge, about 3 kilometres long, which will be the most important bridge of this type in the world (Refer to the article "The Rion-Antirion Bridge – Design and Construction" by Jean-Paul Teyssandier, in these proceedings).

An exceptional combination of physical conditions makes the project quite unusual : high water depth, deep strata of weak soil, strong seismic activity and fault displacements.

The structure is designed in view of challenging the earthquakes and ensuring the every day serviceability of the link as well. Unusual techniques have been developed to solve the critical problem of high degree seismicity in conjunction with a weak soil.

As the in-situ soil had to be improved with stiff and closely spaced inclusions, the yield design theory, used for the evaluation of the bearing capacity of shallow foundations, has been extended to an innovative foundation concept.

As the bridge deck is suspended on its full length, and therefore isolated as much as it can be, sophisticated dynamic calculations have been implemented with the objectives of adjusting the main parameters of the structure, evaluating the behaviour of the bridge under a strong seismic event and taking into account the variability of the input motion.

Finally, as the pylons are one of the most critical part of the structure, several push-over analyses were carried out to evaluate their structural response and have an over view of their behaviour.

INTRODUCTION

The Rion Antirion crossing consists of a main bridge, 2252 m long and 27.20 m wide, connected to the land by two approaches, respectively 392 m and 239 m long, on each side of the gulf.

The main bridge is located in an exceptional environment which consists of a high water depth, a deep soil strata of weak alluvions (the bedrock being approximately 800 m below the sea bed level) and finally a strong seismic activity with possible slow but important tectonic movements.

If all these difficulties could be considered separately, there would be no unusual conceptual problem. But, the conjunction of all these unfavourable conditions leads to a tough design. As the seismic activity is severe, the soil structure interaction is the centre of high forces. As high forces are generated in the weak top layers of the soil, they have to be reinforced and such reinforcement is not an easy task under 60 m of water.

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DESIGN FEATURES

The seismic conditions to be taken into account are given by the response spectrum at the sea bed level which corresponds to a 2000 year return period (figure 1). The peak ground acceleration is 0.5 g and the maximum spectral acceleration is equal to 1.2 g on 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 the subject of small inclinations due to the corresponding rearrangement of the sea bed below the foundations.

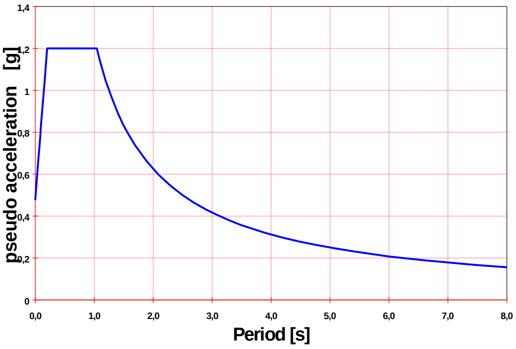

Fig. 1 - Design Spectrum

MAIN BRIDGE CONCEPT

This range of possible disasters lead to reduce as much as possible the number of pylons in the strait and, therefore to an exceptional multi-cable stayed span bridge made of 3 central spans 560 m in length and 2 side spans 286 m long (figure 2).

The corresponding 4 pylons rest on the sea bed through a large concrete substructure foundation, 90 m in diameter, 65 m high, which distribute all the forces to the soil.

Below this substructure, the heterogeneous and weak soil is 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 equal to or more than 7 m.

Fig. 2 - Main Bridge Concept - General view

The top of the steel pipes is covered by a calibrated gravel layer which provides a transition between the structure and the reinforced soil.

The huge foundations support, through octagonal pylon shafts, pyramidal capitals which are the base of 4 concrete legs converging at the top of the pylons and giving them the appropriate rigidity.

The deck is a composite steel-concrete structure 27.20 m wide (figure 3).

It is made of a concrete slab connected to twin longitudinal steel I girders, 2.20 m high, braced every 4 meters by transverse cross beams.

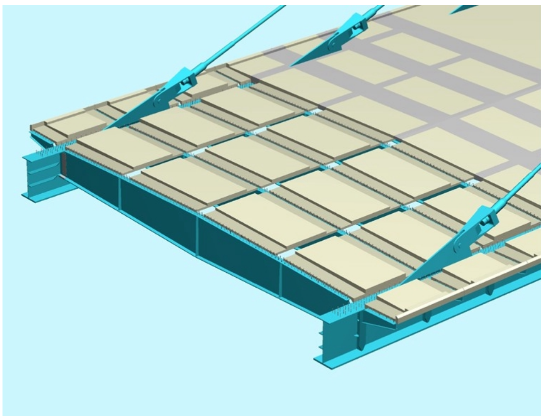

Fig. 3 - Composite Deck Concept

Careful analyses of the behaviour of the reinforced soil and improvements of this innovative concept lead to the design of a continuous deck fully suspended and therefore isolated as much as it can be.

This made also possible to reduce the height of the deck girders and therefore to reduce the wind effects on the bridge.

DESIGN

According to the previous general presentation of the Project, it is clear that the design of the Main Bridge is mainly governed by the capability of the whole structure to resist the major seismic events including a possible fault movement.

But both of these actions are generated by the motion of the ground and the corresponding displacements of the foundations which implies the flexibility of the whole structure, the motion due to the earthquake being temporary when the displacement due to the fault movement is a new permanent action.

As a matter of fact, substantial progress has been made with time in the seismic design approach. Meanwhile, it does not allow the engineers to make a clear differentiation of the various steps of the design process to be adopted in that case and it is not always easy to keep in mind that the structure has first to fit for purpose.

Actually, as long as the force-based design is definitely considered to be not appropriate from most of the world experts, several design philosophies, including Capacity Design, Performance-based Design, Displacement-based Design, appeared in many publications and could generate confusion in the design teams.

Indeed these design philosophies cover the same fundamental aspect of the problem which is that more flexible is the structure, better will be its behaviour.

This means that the structure has first to be designed to resist what will be the main actions during its span life, i.e. for the classical Serviceability Limit States and the corresponding Ultimate Limit States. This is the only process which will produce the most flexible structure and will lead to the most favourable concept from a seismic behaviour point of view. Then, the main components of the structure will be adjusted to the demand during a given design earthquake in terms of acceptable damage.

Seismic Evaluation of the Foundations

The foundations are a typical example of a major part of a structure where the Performance of the concept has 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 be the subject of 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 (figure 4) consist of two separate parts :

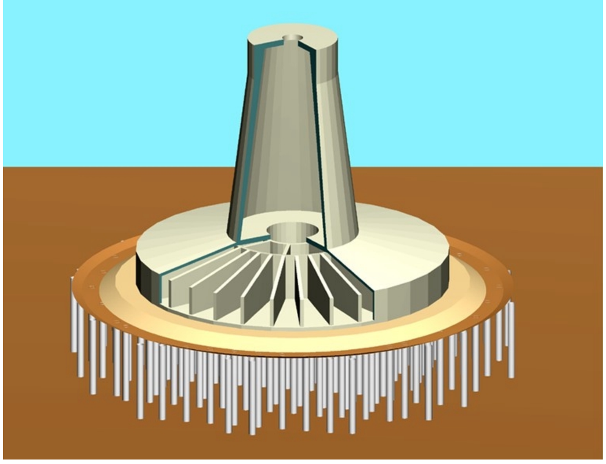

Fig. 4 - Reinforced Soil and Foundation Concept

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

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

Although the foundation looks like a piled foundation, it does not at all behave as such : no connection exists between the inclusions and the raft. The foundation is therefore allowed to uplift or to slide with respect to the reinforced soil.

The capacity design philosophy, introduced in foundation engineering for the evaluation of the seismic bearing capacity of shallow foundations through the yield design theory, had then to be extended to this innovative foundation concept in seismic areas.

Using extensively the yield design theory, through a set of appropriate kinematic mechanisms (figure 5), it was possible to derive an upper bound estimation of the global bearing capacity of the reinforced soil (figure 6).

For this purpose the reinforced soil was modeled as a two-dimensional continuum appropriately connected to beams simulating the stiff inclusions.

Consequently, the calculations included the contribution of the inclusions to the overall resistance of this new concept

.The simplicity of such calculations allowed to optimize the size and the spacing of the inclusions.

A set of centrifuge tests was run to validate the concept and the theoretical approaches.

Then, non-linear finite element analyses could be carried out. They lead to the constitutive laws of the reinforced soil which are used in the general calculations of the structure.

All these calculations, adequately combined with a global dynamic analysis, allowed to check that the effect of the coupled gravel layer and soil reinforcement was to improve the bearing capacity of the whole foundation system while controlling the failure mode:

- The fuse provided by the gravel layer limits the maximum shear force at the interface, dissipates energy by sliding and forces the foundation “to fail” according to a mode which 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 failures modes, like rotational failure which would compromise dangerously the global stability of the structure, and dissipates an important amount of energy as it could be anticipated from the Force-Displacement Diagram (figure 7).

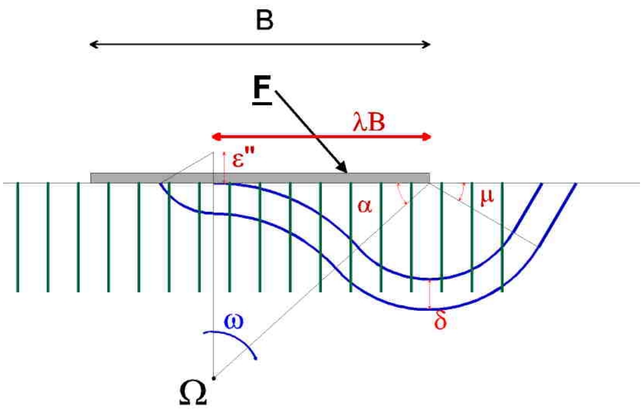

Fig. 5 - Kinematic Mechanism

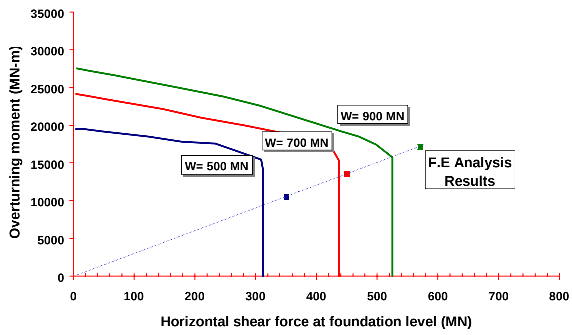

Fig. 6 - Reinforced Soil Interaction Diagram

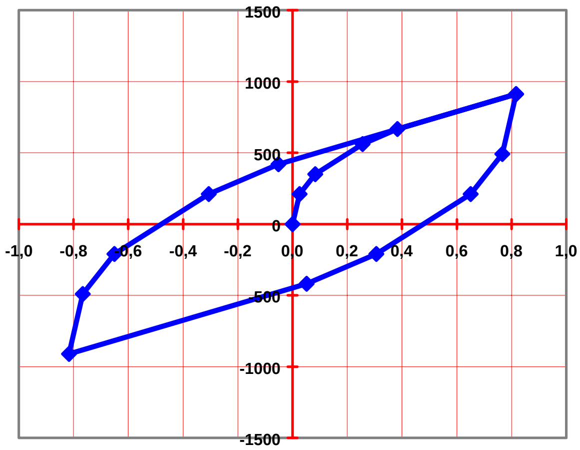

Fig. 7 - Soil Horizontal Force-Displacement Diagram

General Calculation of the Bridge (Dynamic Analyses)

All the previous calculations and results are used to carry out detailed and careful 3D dynamic analyses of the whole structure.

Thanks to the development of a certain number of calculation tools on the basis of an existing powerful computer software, the following very important properties are taken into account :

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

Twelve 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 %) are used.

From these calculations, the way the reinforced soil behaves and the bases slide can be carefully checked.

Behaviour of the Pylons

The general calculation of the Bridge confirms the very good behaviour of the fully suspended deck which is isolated as much as it can be. The relative displacement of the pylon bases with respect to the gravel layer evidences some sliding which remains nevertheless acceptable and if, for any reason, this sliding could not occur it has been checked that this was not a major point of concern.

Because the stability of the fully suspended multi cable-stayed span deck is secured by the stiffness of the pylons which consist therefore of four legs converging at mid height of the anchorage zone, the pylons are the most critical parts of the structure.

The dynamic analyses evidence that 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 show that the extreme shakes generate various crack patterns, distributed along the legs, coming from both bending and tension. On the one hand, it can 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 non acceptable damages). On the other hand, it is 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 impressive. Time steps being 0.02 s - i.e 2500 steps for a 50 second event – 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 face this voluminous quantity of information, the option is to check for the duration of the earthquake that the strains of the materials (concrete and steel) in each cross-section are not exceeding the acceptable limits which guarantee a controlled damage of the pylons while the general consistency of these sophisticated calculations through the corresponding deflection shapes of the legs, axial shear forces and bending moments generated in each cross-section, can be verified for time history peak values of those parameters.

Push-Over Analyses of the Pylons

Under these conditions, it makes sense to carry out a push-over analysis of the pylons to evaluate their global behaviour and compare their performance to the demand, in terms of displacements, during the extreme seismic event.

It can be pointed out that such a push-over analysis has become usual. Moreover it is extremely simple for a high 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 not that simple anymore when the pier has become a pylon group made 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 consists in reproducing 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 – i.e. when forces, bending and displacements are the most severe.

This approach allows to assess the displacement demand on the pylon as well as its displacement capacity as estimated from the 3D dynamic analysis.

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

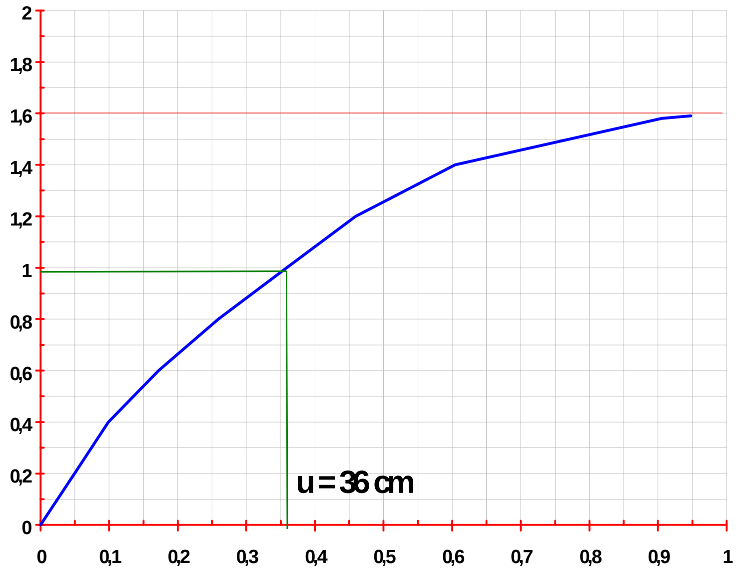

Fig. 8 – Displacement at top of pylon legs / Magnification factor

The diagram showing the displacement D at the top of the pylon legs versus the magnification factor A (figure 8) allows to make a clear differentiation of the various steps characterising the behaviour of a whole pylon group. 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 shows that the displacement demand (D = 0.36 m for A = 1) is far under 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 case of an extreme event and also that any deviation with regard to the input motion should not have any bad consequences.

Another way to assess a realistic push-over analysis consists in using the classical approach and evaluating the stucture's modal response as proposed by the Independent Checker team (Buckland and Taylor and Seismic Experts N. Priestley, F. Seible and M. Calvi).

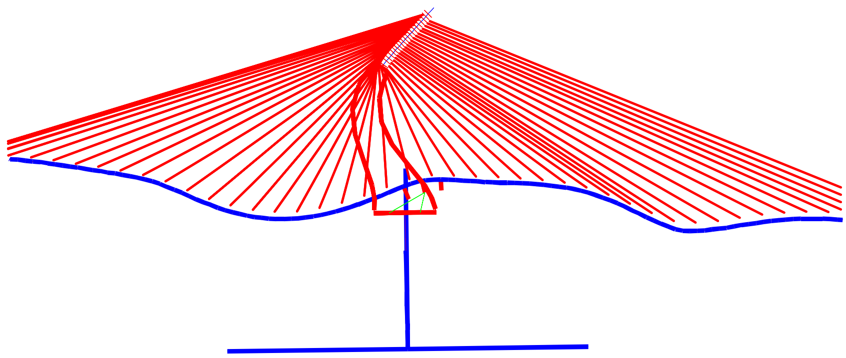

Fig. 9 - Typical Deflection Shape of the Pylons

Such an approach could seem to be a priori unpracticable, the question being how to simplify enough the loading of the pylon to model it by an horizontal pushing force and where to push. Nevertheless, it can be shown from a modal analysis of the structure that the deflection shape of a pylon for a certain number of significant modes always involves simultaneously bending moments and axial loads (either tension or compression) in the various legs (figure 9).

This is indeed a consequence of the fact that the vertex of the legs has been initially designed to be at the centre of gravity of the horizontal forces generated by the cable-stays in the pylon heads in order to reach the maximum stiffness under unfavourable live loads (traffic).

In addition to that it could be pointed out that the relative horizontal displacement between the deck and the pylons due to the horizontal motion of the ground should generate an extra tension of the longest stay cables which should then reduce the action of the bending moments in the legs.

But it happens that the conjunction of all the characteristics of the fully suspended bridge deck systematically leads to a modal deflection shape of the deck which tends to generate a detention of the longest cable-stays which should be over-stressed by the pylon deck relative displacement. In other words, the favourable effects of the bending moments in the legs are amplified by the favourable action of the cable-stays and this is a very important result as far as the seismic evaluation of these stiff pylons is concerned.

It can be therefore anticipated that a force F has to be applied step by step at a distance d below the vertex (figure 10) which was finally considered to be 10.5 m after due consideration of the forces and bending moments generated at the bottom of the pylon legs.

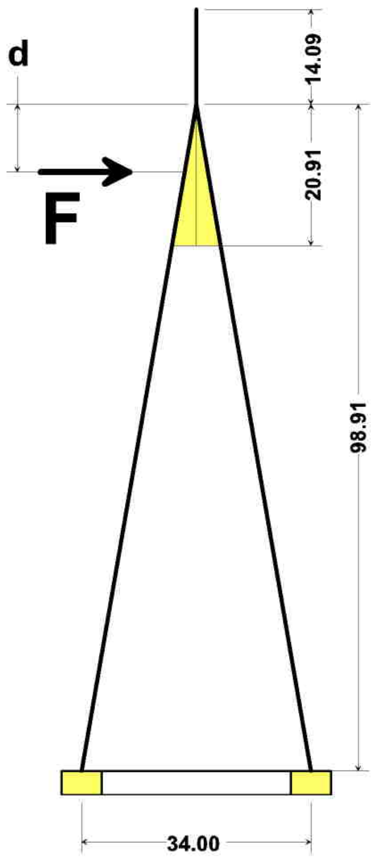

Fig. 10 - Push-Over Principles

Comparison of results from both push-over approaches with the results of the non linear full structural model time-history approach reveals a very good consistency and then show the reliability and accuracy of this last approach which, by the way, gives many other fruitful results for all parts of the structure the other approaches cannot give.

Meanwhile, it must be emphasised that the second approach allows evaluating the seismic behaviour and performing sensitivity analyses of an unusual reinforced concrete structure in a simple way. Used in an appropriate manner, it gives a force-deflection curve similar to the one obtained from the peak time-history results and can be run quickly to study the influence of the most significant pylon parameters (axial load and steel reinforcement ratios).

CONCLUSION

It appears that permanent displacements of the reinforced soil are not significant when the sliding of the pylon bases is of the order of 10 cm only.

Although the horizontal force is limited by this sliding, the pylons can globally rotate and the permanent rotation of the reinforced soil can lead to a displacement of 85 cm of the pylon top.

All these displacements are acceptable as long as provision has been made for the whole structure to be adaptable even if some adjustment has to be done after the major seismic event.

During the earthquake, the dynamic analysis shows that three of the four legs of the pylons are under tension. In this case the maximum axial load ratio in the leg under compression is 0.40 and the maximum strains in the materials are respectively 2.5 %o for the concrete (under combined compression and bending) and 6 %o for the steel reinforcement. These values are therefore in a range which should guarantee that no uncontrolled damage may happen.

In addition to that the pylon behaviour has been checked through a Push-Over Analysis which evidences a safety factor of 1.6 with regard to the maximum effects generated by the design seismic event.

Finally, if the number of strands in most of the shortest cable stays is governed by the forces generated during the earthquake it has to be pointed out that the deck is flexible enough to behave normally without any damage.

REFERENCES

[1] Combault J.and Morand P., The exceptional structure of the Rio-Antirion Bridge in Greece, Proceedings of the IABSE Symposium in Kobe, Japan, September 1998.

[2] Pecker A., Capacity Design Principles for shallow foundations in Seismic Areas, 11th European Conference in Earthquake Engineering, Key Note Lecture, Paris la Defense, September 1998.

[3] Teyssandier J.P., The Rion-Antirion Bridge, Proceedings of the International Conference on New Technologies in Civil Engineering, Lisbon, July 1997