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SEISMIC DESIGN FOR THE FOUNDATIONS OF THE RION ANTIRION BRIDGE

Alain PECKER
Chairman
GEODYNAMIQUE ET STRUCTURE
157, Rue des Blains
92220 BAGNEUX - FRANCE

Jean-Paul TEYSSANDIER
Scientific Director
DUMEZ-GTM
32, Avenue Pablo Picasso
92022 NANTERRE - FRANCE

Thierry GUYOT
Chief Engineer
SEEE - Infra
174, Boulevard de Verdun
92413 COURBEVOIE - FRANCE

Jacques COMBAULT
Scientific Director
DUMEZ-GTM International
32, Avenue Pablo Picasso
92022 NANTERRE - FRANCE

SUMMARY

The Rion Antirion Bridge near Patras (Greece) is a four-spans cable-stayed bridge, each span being 560 m long. The project is located in a zone of very difficult environmental conditions characterized by deep soil strata of weak silty clays over depths in excess of 80 m, a high water depth (60 m) and a very strong seismic design motion with a peak ground acceleration of 0.48 g at mudline level. All these constraints have called for an original design of the foundations which minimizes the overall cost of the bridge, has a small impact on the environment and requires advanced design tools to account for the pronounced soil-structure interaction under seismic conditions.

1.0 INTRODUCTION

The difficult environmental conditions encountered for the design of the Rion Antirion Bridge near Patras (Greece) had called for an original design of the foundations which consists in a large soil improvement of the surficial soil layers encountered under the four bridge piers.

After examination of many different foundation concepts, among which piles foundations, deep embedded caissons, substitution, were studied, the final choice consists in large diameter (90 m) pier foundations, resting at mudline level on a volume of improved soil strata. To provide sufficient shear strength to the soil strata, which have to carry the large seismic forces coming from the structural inertia forces and hydrodynamic water pressures, the soil improvement is realized with driven hollow steel pipes, 20 m long, 2 m in diameter, at a regular spacing of 5 m; about 400 pipes are driven at each pier location. The improved soil volume is considered as a homogenized material, strength characteristics of which are evaluated taking into account the interaction between the inclusion and the surrounding soils; the principles of soil nailing which has been the subject of a considerable bulk of studies (references 1, 2) are used.

The dynamic response of the bridge and the verification of the foundation bearing capacity are performed in two steps: evaluation of the dynamic forces transmitted to the foundation using a finite element model with step by step integration; verification of the foundation bearing capacity using the fundamental results of the yield design theory (references 3 to 6) for the homogenized soil volume. A step by step integration was required for the dynamic analysis since foundation uplift was allowed during the earthquake resulting in a significant reduction of the seismic overturning moment. These simplified, uncoupled, methods of analysis were implemented at the preliminary design stage and proved later, by comparison with two dimensional dynamic finite element analyses including the soil layers, to realistically account for the complex soil-structure interaction phenomenon.

Owing to the considerable amount of studies carried out on the project, it has been possible to prove that the proposed, innovative foundation scheme was feasible and results in an economical project in very severe environmental conditions.

The different steps of the design process are briefly reviewed in this paper.

2.0 GEOTECHNICAL SOIL PROFILE

2.1 Existing Soil Profile

Boreholes were drilled across the Gulf of Patras at various locations to let open the choice of the bridge and foundation types. Out of those, eight are relevant to the present project: three under one pier, two under the other two piers and one under the last pier.

From the investigations of the continuous borings, dilatometer tests, cone penetration tests with pore pressure measurements and laboratory tests on undisturbed samples, five main units are identified within the surveyed depths (30 m to 1 000 m): gravel and sandy gravel, sand, silty sand, silty clay and clay. A soil profile along the bridge route is presented in figure 1; no defined continuity exists between the layers from one location to the other: however, the general trends are worth mentioning:

- a cohesionless layer is present at mudline level at each pier location; it consists in gravel and sandy gravel and its thickness ranges from 28 m to 4 m;

- a second cohesionless layer, sand or sandy gravel, is encountered at depths of the order of 18 - 20 m; its thickness varies between a few meters up to 20 m;

- in between both layers, strata of silty sand and silty clay (with plasticity indices of the order of 15%) are encountered; these strata cannot be correlated from one location to another, yielding a rather erratic, heterogeneous soil profile;

- below the second cohesionless stratum, the soils are more homogeneous and mainly consist in silty clays or clays (with plasticity indices of the order of 20%);

- no bedrock has been encountered within the surveyed depth.

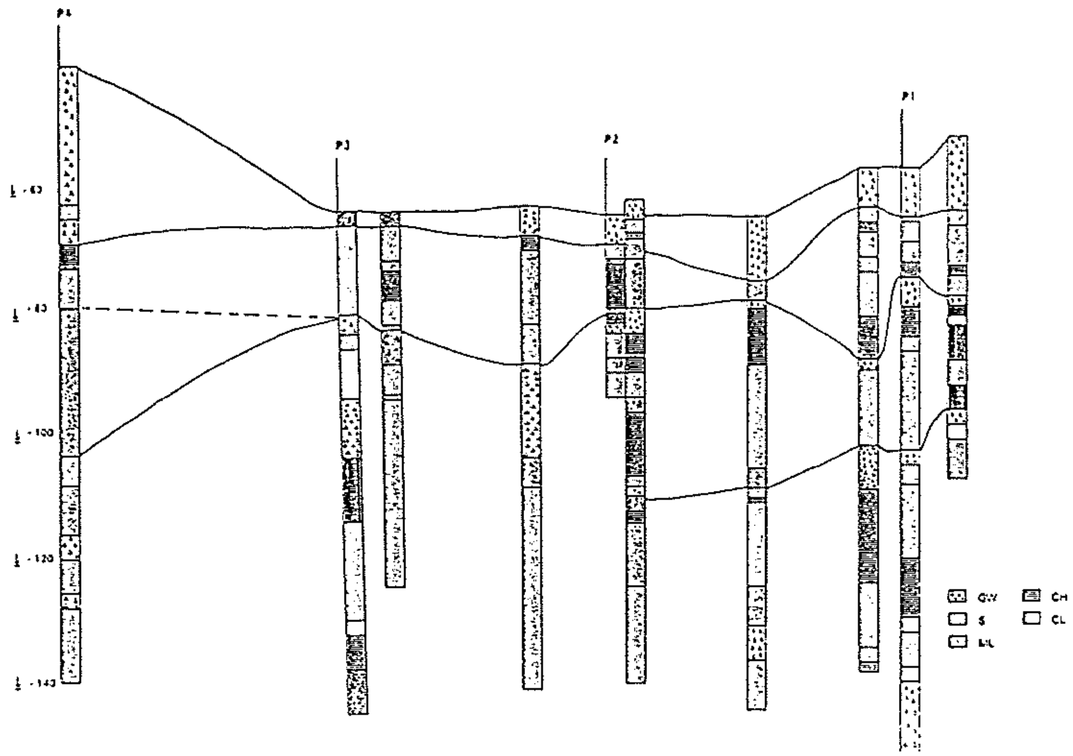

Fig. 1. Soil profile across the Gulf

The mechanical characteristics of the various soil units have been derived from the results of laboratory tests, cone penetration tests and dilatometer tests. The soil strata appear to be slightly overconsolidated in the top 20 m, then normally consolidated in depth. The undrained shear strength of the cohesive units is related to the vertical effective stress through:

$$Cu = 0.32\,\sigma'_v$$

This strength does not appear to be significantly affected by cyclic loading and exhibits a slight increase (20% to 50%) for high strain rates. Both effects compensate each other and no account of either of them is taken in the analysis.

In view of the nature of the soils, liquefaction does not appear to be a problem; large flow slides cannot take place and the top 20 m of soil where sand lenses are encountered, are improved: therefore, large cyclic strains and liquefaction are precluded.

2.2 Reinforced Soil

It has been noticed previously that the top 20 m of soils are rather heterogeneous and of low mechanical characteristics. In order to improve the mechanical characteristics of these layers, the reinforcement scheme described in the introduction is set under each pier and outside the foundation raft, up to a radius equal to 65 m. The improved soil volume is considered as a homogenized material, strength characteristics of which are evaluated, taking into account the interaction between the inclusion and the surrounding soil. The yield criterion for the homogenized material is governed by the condition, among the following ones, which first occurs:

- yielding of the soil with a contact pressure between the inclusion and the soil reaching the ultimate lateral soil pressure;

- yielding of the inclusion under bending stresses;

- yielding of the inclusion under shear stresses.

For the conditions prevailing for the project, the first condition (soil yielding) is the governing one. The computed equivalent shear strength of the homogenized material is equal to Cu = 250 to 300 kPa.

3.0 DESIGN SEISMIC CONDITIONS

The seismic conditions to be taken into account are presented in the form of the mudline response spectrum in figure 2. The peak ground acceleration (zero period) is equal to 0.48 g and the maximum spectral acceleration is equal to 1.2 g between 0.2 s and 1.0 s.

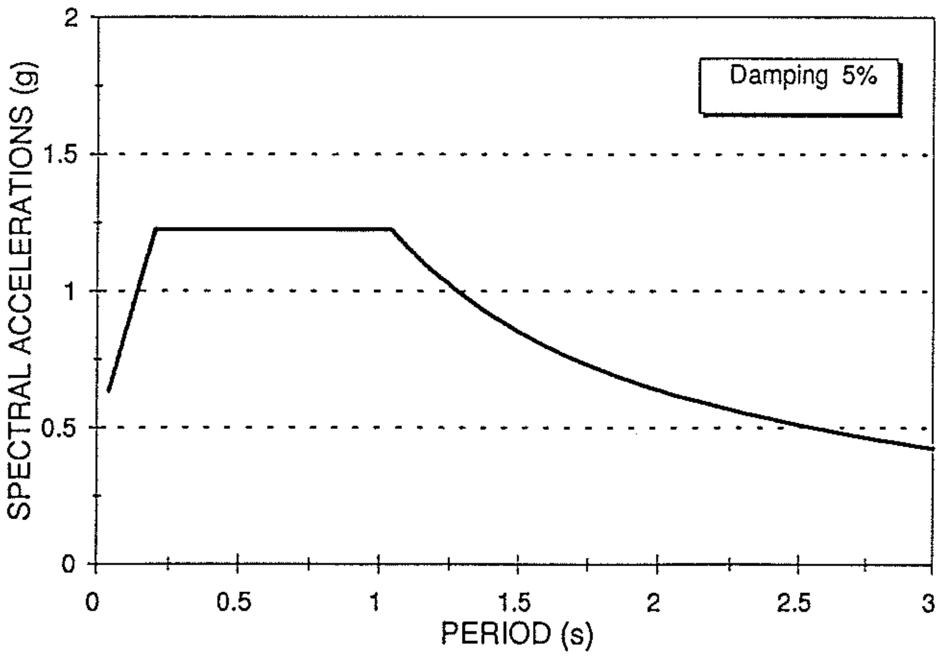

Fig. 2. Design horizontal spectrum

4.0 DYNAMIC ANALYSIS OF A BRIDGE PIER

The dynamic stability of any of the bridge piers is assessed using a multi-steps analysis in which :

- the dynamic impedances of the circular foundation are computed using a dynamic linear finite element model, assuming full bonding between the soil and the foundation;

- the rotational stiffness is modified to account for possible uplift of the foundation; the non-linear moment-rotation curve is derived from a static finite element analysis of the raft;

- a dynamic step by step integration analysis of the pier and its cantilever deck is performed for the horizontal input motion, represented by an artificial time history, which spectrum closely matches the design spectrum;

- a modal spectral analysis is performed for the vertical input motion and its results are combined to those of the horizontal directions;

- the global forces (vertical and horizontal forces, overturning moment), retrieved from the dynamic analysis, are used to check the foundation stability, using limit equilibrium methods.

All these steps, and the results obtained, are detailed herebelow.

4.1 Structural model for the pier

Owing to structural arrangements, each pier with the cantilever deck is independent from the others; therefore, the structural model is composed of the pier itself, the pylon and the cantilever deck. A schematic view of the pier, pylon and deck, is given in figure 3.

The total mass of the model is equal to 209.23 kilotons, the submerged weight is equal to 1 000 MN, which corresponds to an applied pressure of 160 kPa.

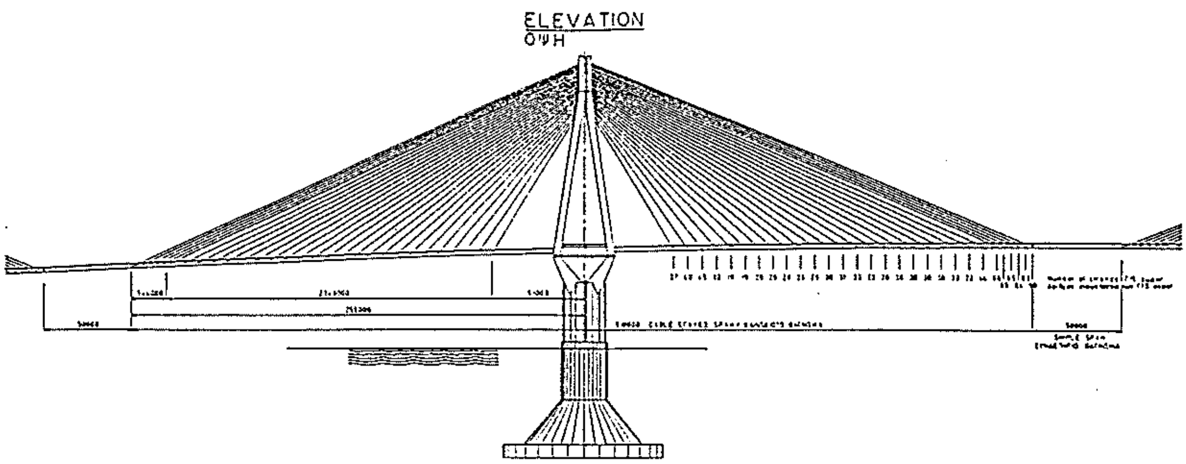

Fig. 3. Elevation of a pier

To the structural masses, additional masses are applied on the underwater part of the model; these masses represent the so-called added masses which account for the hydrodynamic water pressures developed during an earthquake. They were numerically computed for the actual pier geometry and depth of water, with a finite element computer program. Neglecting the free surface effect and the water compressibility, which is justified in the frequency range of interest, the masses are frequency independent. They are equal to:

- translational horizontal mass: 50.27 kilotons
- translational vertical mass: 109.71 kilotons
- rotational mass: 44 200. kilotons.m^2

The translational mass represents ~ 50% of the mass of displaced water.

4.2 Soil Foundation Interface

The evaluation of the seismic response of the bridge requires the knowledge of the so-called dynamic impedances of the pier foundations. These impedances which represent the effect of the underlying soil layers, depend on the foundation geometry and on the dynamic soil characteristics which, in turn, due to the non-linear soil behavior, depend on the level of the seismic excitation. These impedances have been computed for the four modes of vibration (vertical, horizontal, rocking, torsion) in the frequency range of interest for the analysis (0.0 to 10. Hertz). In view of the important heterogeneities in the soil properties from one location to another, large parametric studies were performed to cover the whole range of uncertainties. The dynamic soil characteristics of interest for the analyses (elastic shear wave velocities) were derived from the in-situ tests (seismic cone testing) and from the laboratory tests (resonant column tests). The variations of the shear moduli and damping ratios with shear strain were taken from the results of the laboratory tests. Based on these data, the constitutive model for the soil behavior is the equivalent viscoelastic model developed by Seed et *al.* The calculations involve two steps:

- a free field ground response analysis to assess the strain compatible moduli and damping ratios for the various layers comprising the soil profile,

- an impedance analysis to compute the frequency dependent impedance functions for the four degrees of freedom of the circular foundation. These analyses were performed with a finite element program taking into account the actual soil profiles.

An example of the dynamic impedance (real part and imaginary part) is given in figure 4 for the horizontal vibration mode.

In the frequency range of interest, these impedances, one for each degree of freedom, turn out to be almost frequency independent and can conveniently be modeled by constant springs and dashpots.

These springs and dashpots assume a perfect bonding between the soil and the foundation. However, in order to reduce the seismic overturning moment, it was deemed necessary to allow the foundation to uplift during the maximum design earthquake. Therefore, constructive arrangements are taken to prevent tensile connections between the reinforcing pipes and the raft; however, a shear connection is provided to safely transmit the shear force, even in the case of uplift.

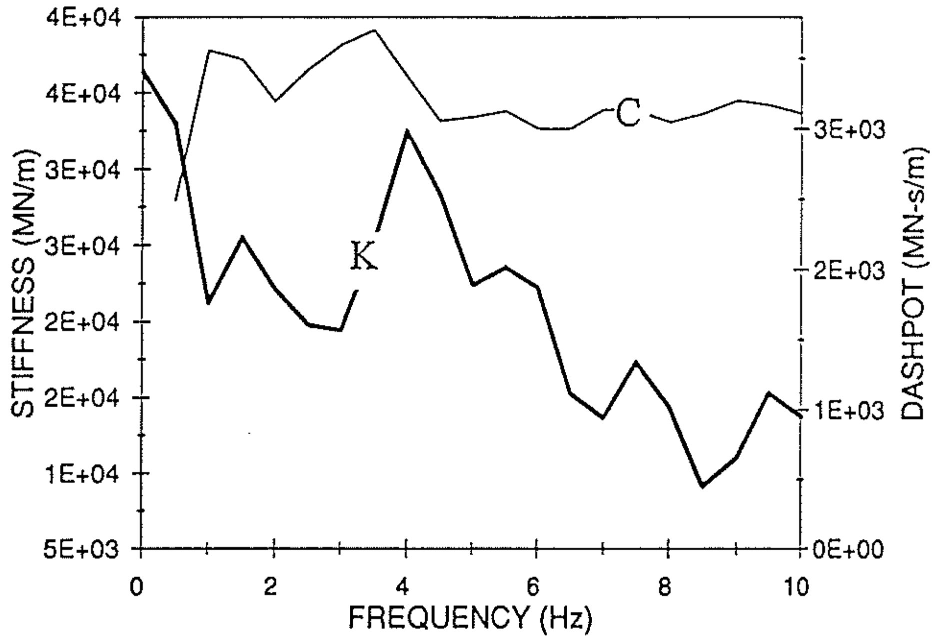

Fig. 4. Horizontal dynamic impedance

To account for this non-linearity in the dynamic analysis, the linear rotational spring is replaced by a non-linear rotational elastic spring, reflecting the non-linear moment-rotation curve of the foundation. This non-linear moment-rotation curve was established from a static finite element analysis with special contact elements and an elastoplastic constitutive model for the soil. This curve is presented in figure 5.

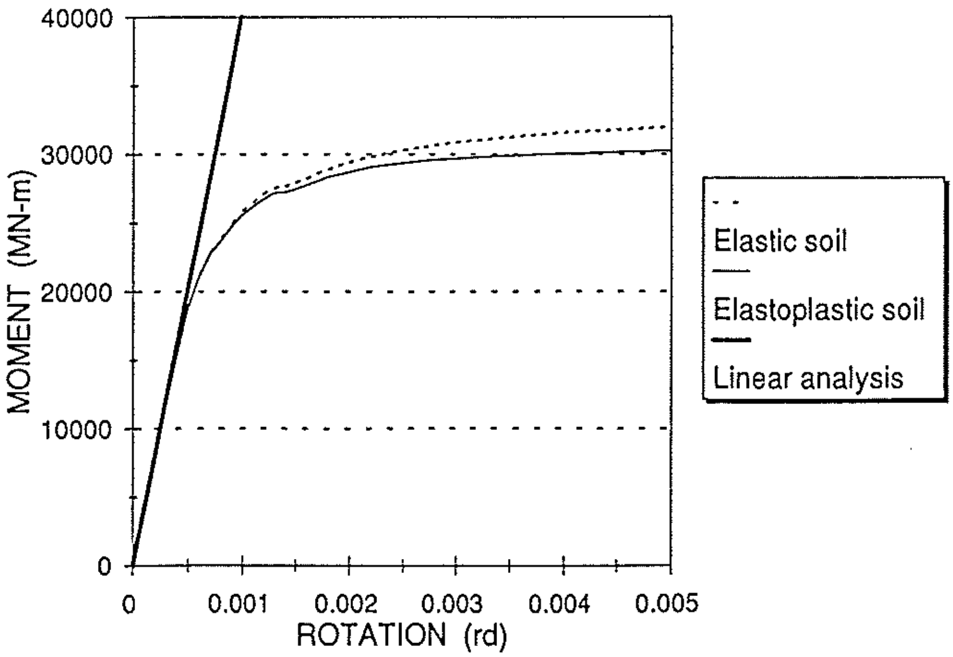

Fig. 5. Non-linear moment-rotation curve

4.3 Dynamic Analysis

Because of the non-linear rotational spring, the dynamic response of the model to a horizontal seismic input motion was computed from step by step history analyses. For these analyses, artificial time histories with response spectra matching the design spectrum of figure 2 were used. Each horizontal excitation was treated independently and both were then combined with the vertical direction, according to the rules set forth in the Eurocode 8 Part 2 (Bridges), i.e. the full value in one direction and 0.3 times the values in the other two directions. For a vertical seismic input motion a more classical modal spectral analysis was used. Parametric studies were conducted to cover the uncertainties in the soil characteristics.

Among the numerous results available from these analyses, the global forces transmitted to the foundation are of interest for the verification of the bearing capacity; one of the major superiorities of the time history analysis over the classical modal spectral analyses is its ability to provide simultaneous compatible force values. In other words, the bearing capacity is checked for the following sets of values:

- the maximum horizontal shear force and the simultaneous overturning moment;
- the maximum overturning moment and the simultaneous horizontal shear force;

Finally the total (static + dynamic) forces combinations for which the foundation bearing capacity is checked are:

	Case n° 1	Case n° 2	Case n° 3	Case n° 4
Vertical Force (MN)	1 000 + 130	1 000 - 130	1 000 + 130	1 000 - 130
Horizontal Shear Force (MN)	1 190	1 190	1 000	1 000
Overturning Moment (MN-m)	14 450	14 450	23 000	23 000

Tab. 1. Static + dynamic forces at foundation level

The maximum amount of uplift of the foundation, which is not a design criterion, is equal to 50% of the total area, associated with a rotation of the order of 10^{-4} radians. Would foundation uplift be prevented, the maximum overturning moment could be of the order of 40 000 MN. Therefore, this non-linear geometric effect results in a significant reduction of the applied forces.

4.4 Verification of the foundation bearing capacity

The verification of the foundation bearing capacity was carried out following the principles of limit analysis methods (references 4 to 6). To this end, the actual circular geometry of the foundation is replaced by an equivalent rectangular foundation with the same area and same moment of inertia around a horizontal axis. The load combinations given in table 1 were used with a partial load safety factor equal to 1.0 because the seismic situation is treated as an accidental one. The calculations consist in determining the material safety factor, value by which the soil undrained shear strength must be divided by to bring the foundation to an incipient state of failure. For the four cases of table 1, the computed material safety factors are given in table 2.

	Case n° 1	Case n° 2	Case n° 3	Case n° 4
Material Safety Factor	1.34	1.34	1.55	1.56

Tab. 2. Material safety factors

A minimum material safety factor equal to 1.30 was deemed necessary to ensure a safe behavior of the foundation. The values associated with cases n° 1 and 2 correspond to a pure sliding failure mode, as can easily be checked from the maximum horizontal resisting force equal to the foundation area times the undrained shear strength :

$$F_{max} = S_u \,.\, A = 0.250 \times 6360 = 1590 \text{ MN}$$

As a conclusion, the analyses carried out have shown that adequate safety is ensured for the main piers under earthquake loads. Under these loads, the most critical failure surface develops within the reinforced soil block at the interface between the foundation and the soil.

5.0 NON LINEAR FINITE ELEMENT ANALYSIS

Each of the aforementioned steps of the multi-steps approach involves some degree of approximation; moreover, since a material safety factor of at least 1.30 was looked for, the analysis does not provide any information on the amount of post-earthquake settlements of the foundation. It can only be concluded from the above analysis that, if the minimum required safety factor is reached, the permanent post-earthquake displacements should be small and acceptable for the bridge pier. In order to validate the multi-steps approach and to have an order of magnitude for the permanent displacements, a global two-dimensional dynamic finite element analysis has been carried out. In this analysis, the soil, the foundation and the bridge pier with its associated cantilever deck are modeled. The global model is subjected, simultaneously, to a vertical and a horizontal motion specified at an hypothetical rock outcrop. A non-linear constitutive model is used for the in-situ soil layers and for the reinforced soil block. Special contact elements are used for the connection between the raft and the soil; these elements do not transmit any tensile force, thereby allowing for separation between the soil and the structure. Finally, transmitting boundaries are provided at the bottom mesh to simulate the presence of an elastic halfspace and prevent total reflection of the waves impinging the boundary.

The most important features of the global model are detailed herebelow.

5.1 Soil constitutive model

One of the major purposes of the 2D finite element analysis being the verification of the foundation stability and the evaluation of the permanent, post-earthquake, displacements, the constitutive model of the soil must adequately reflect the non-linear, hysteretic, soil behavior. To this end, an elastoplastic constitutive law is used. The model (reference 7) is based on the concept of multi-yield surfaces. Each yield surface is represented by a cylinder in the principal stress space (Von Mises criterion) and is dragged along, during loading-unloading, with the stress point. The outer external surface plays the role of a failure surface and is fixed in space. To each yield surface is attached a plastic, constant, modulus. The stress-strain curve under simple shear takes the form of a hyperbolic curve. By using as many yield surfaces as needed, any degree of soil non-linearity can be modeled. The choice of the yield surfaces is dictated by the fact that the present analysis is a total stress analysis, carried out under undrained conditions, and by the nature of the soil layers (silty clay and clay): the Von Mises criterion is therefore appropriate for such an analysis.

The shear moduli and undrained shear strengths of the various layers are identical to those used in the multi-steps approach. The soil bulk moduli take two different values:

- during the static initialization phase under the soil stresses and pier weight, the bulk moduli are equal to 1.7 times the shear moduli (Poisson's ratio $\nu = 0.30$);

- during the dynamic phase of the analysis, the bulk modulus is assigned a very large value, thereby simulating a constant volume condition under undrained conditions (Poisson's ratio close to $\nu = 0.50$).

5.2 Main characteristics of the finite element model

The global finite element model, including the soil and the superstructure, is presented in figure 6. It is composed of 1 418 nodes having either two or three degrees of freedom and of 1 300 quadrilateral solid elements, 37 beam elements, 2 truss elements and 21 contact elements; the total number of degrees of freedom is equal to 2 795.

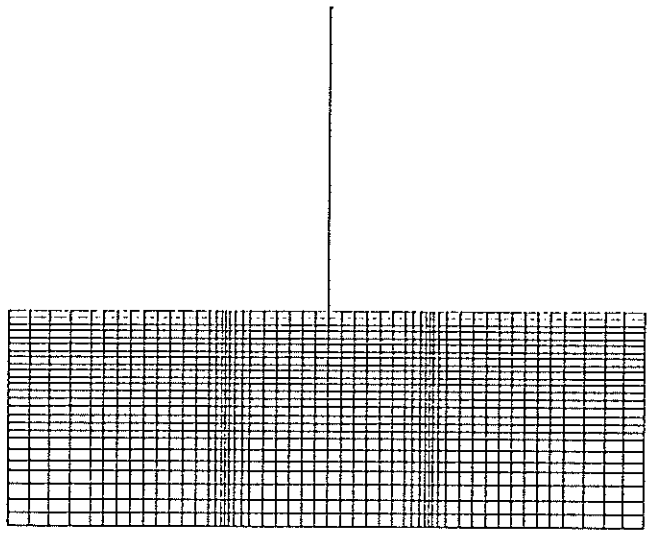

Fig. 6. Finite element model

5.3 Dynamic analysis

The dynamic analysis must be preceded by a static analysis in which the submerged dead weight of the structure is applied. As explained previously, this phase is conducted under drained conditions; however, the soil undrained shear strength ($S_u = 0.32\ \sigma'_v$) is not updated to account for the consolidation of the soil layers under the pier weight.

The dynamic analysis is performed with the computer code DYNAFLOW (reference 8) using a time step of 0.02 seconds during 40 seconds.

The time history of the horizontal shear force at the foundation level is presented in figure 7. The maximum horizontal shear force is reached a significant number of times and is equal to 8.0 MN/mℓ; for the actual 3D geometry, the corresponding value is 656 MN. Alike the shear force, the maximum value of the overturning moment is reached a significant number of times; it amounts to 170 MN/mℓ, i.e. for the 3D geometry to 13 940 MN-m. These values have to be compared to those given in table 1 obtained by the multi-steps approach ; they are smaller by a factor of approximately 2. However, in the dynamic global analysis there is no control on the seismic motion at the mudline level which results from the wave propagation through the soil layers; therefore, the multi-steps approach was redone with the actual time histories of motion computed at the mudline level within the finite element mesh. Similar results to those indicated above were obtained, thereby validating the multi-steps approach.

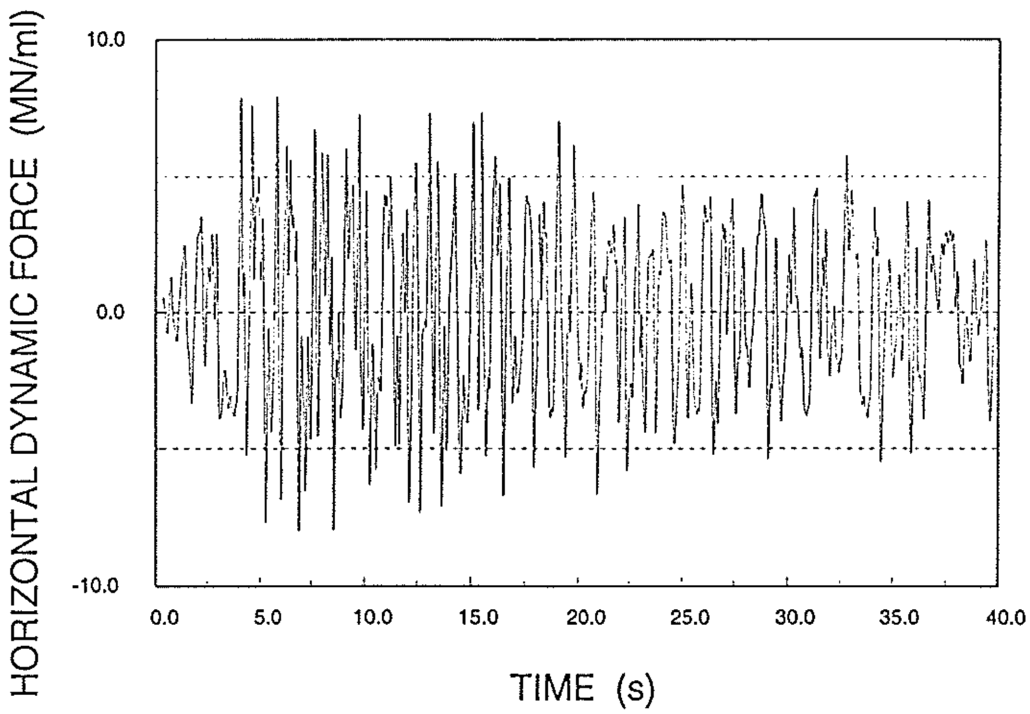

Fig. 7. Dynamic shear force at foundation level

The permanent displacements resulting from the non-linear soil behavior are presented in figure 8 for the horizontal, vertical displacements and rotation. Since due to the high level of the seismic input motion significant non linearities develop in the free field, the displacements are evaluated after subtraction of the free field permanent displacements ; they reflect the additional non-linearities induced by the soil-structure interaction. At the end of the earthquake the displacement is equal to 1.5 cm which is a small value compared to the 25 cm obtained in the free field. The foundation undergoes also a vertical settlement of 13 cm which takes place during the strong phase of the motion (5 to 10 seconds). This settlement is also small when compared to the anticipated static settlements which were estimated to amount to 30 to 70 cm, depending on the pier. Finally the maximum rotation is attained during the seismic excitation and is equal to 7 10^{-4} radians with a permanent rotation of 4 10^{-4} radians at the end of the earthquake.

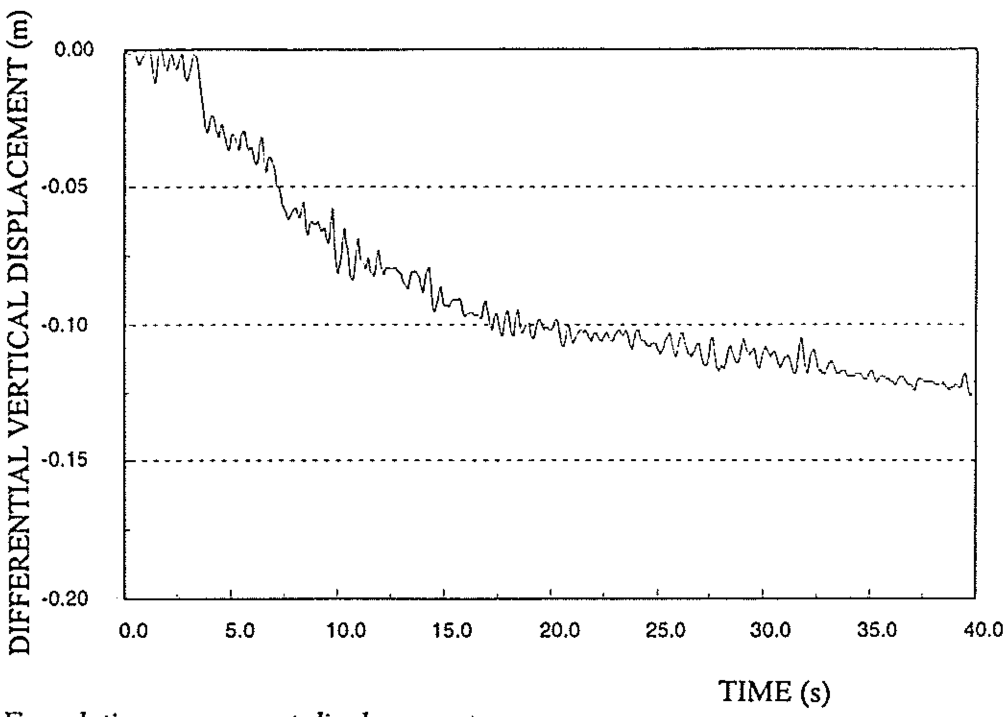

Fig. 8. Foundation permanent displacement

6.0 CONCLUSIONS

The seismic design of the foundation piers of the Rion Antirion bridge was performed in a multi-steps approach in which the forces applied to the foundation are computed from a dynamic analysis in which the soil is represented by linear and non-linear springs; these forces are used to check the foundation bearing capacity using limit equilibrium methods. To ensure a safe design it was estimated that a material safety factor higher than 1.30 was necessary. In order to check the validity of the proposed approach and to evaluate the permanent, post earthquake, displacements a global finite element analysis has been carried out with a non linear constitutive model for the soil and the soil-foundation interface. Results were essentially similar in both approaches, therefore validating the more easy-to-use multi-steps approach.

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