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A Recent Difficult Foundation Problem: The Case Of The New Tagus Bridge

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TEJOPROJECTO

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ABSTRACT

A brief description of the New Tagus Bridge is presented.

The main site geological conditions and the field and laboratory tests are described.

The seismic studies related to the design spectra, the liquefaction potential assessment and the seismic pile analysis are presented.

The pile load tests (both static and dynamic) carried out on trial piles are described.

The objectives of reception tests for piles and monitoring during the construction phase and the long term are presented.

KEYWORDS

Bridges; Foundations; Field and laboratory tests; Seismic studies; Liquefaction; Pile tests; Monitoring.

INTRODUCTION

This lecture is divided into four parts. In the first part a brief description of the new Tagus Bridge is presented.

In the second part the main geological conditions are described. The field and laboratory tests are referred.

In the third part the analyses to derive the design free field surface spectra are described. The liquefaction potential assessment is performed. Some considerations about seismic piles analysis are presented.

The results of pile load tests carried out on trial piles are described.

The fourth part presents the reception tests for piles and the objectives of monitoring during the construction phase and the long term.

Some final considerations are presented.

Part 1

“Step after step the ladder is ascended”

Jacula Prudentum

George Herbert (1640)

BRIEF DESCRIPTION OF THE BRIDGE

With the construction of the new 18 km Tagus bridge Lisbon, capital of Portugal, will double the road access across the Tagus river into the city.

Concession company Lusoponte was awarded the contract to design, build, finance and operate the new bridge by the client Gattel, the Portuguese Ministry of Public Works, Transportation and Communication.

The leader of Lusoponte is Trafalgar House and the other members of the group are Campenon Bernard of France and six Portuguese contractors: Bento Pedroso Construções, Mota & Companhia, Somague - Sociedade Construções, Teixeira Duarte - Engenharia & Construções, Sociedade de Construções H. Hagen and Edifer - Construções Pires Coelho & Fernandes.

The bridge is composed by a number of structures. From the Sacavém interchange on the west bank of the estuary traffic will cross the North viaduct, the Expo viaduct, the Cable stayed bridge, the Central viaduct and, finally, the South viaduct (Fig.1).

All design work was carried by Tejoprojecto composed by COBA, leader of the consortium, Trafalgar House Technology, Europe Etudes GEC and Proponte.

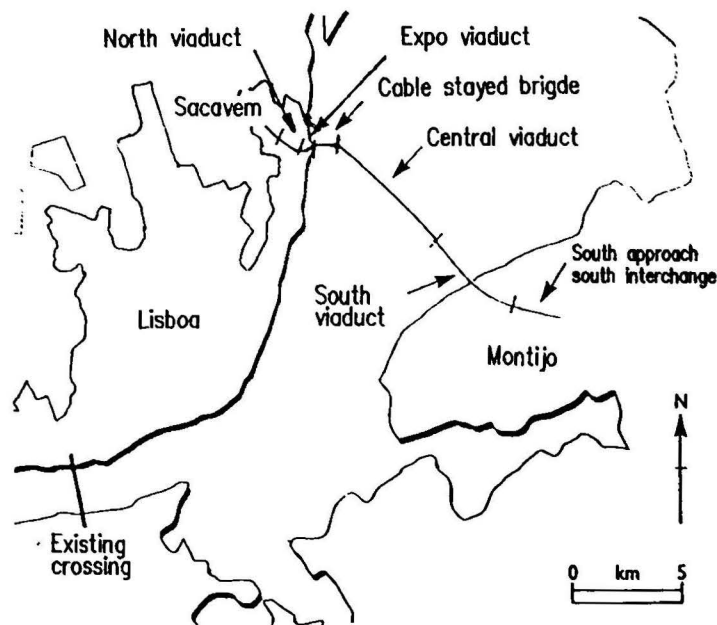


Fig.1 New Tagus crossing site

Foundations for the bridge structures are a mixture of bored and driven piles.

The central viaduct, with 6.5 km long, will be supported on 648 driven piles up to 60 m long. Eight piles with a diameter of 1.7 m will be installed below each pier on all the central viaduct piers except those next to shipping channels.

Three channels pass below the Tagus crossing; the main thoroughfare under the cable stayed bridge, and two smaller channels under the central viaduct. Piles supporting these piers are 2.2m in diameter, to protect against possible ship impact.

Driven piles were installed by joint venture subcontractor Volker Stevin - Ballast Nedam. Large barge mounted cranes were used to drive each pile as one piece. A handling capacity around 58 t was necessary by the cranes and the hammer to drive the piles into position.

Foundations on the cable stayed bridge with 0.83 km long and the south viaduct, with 3.9 km long, were bored piles installed by Italian contractor Trevi. Some 148 piles with 2.2 m diameter were used on the cable stayed bridge and on the south viaduct there will be 60 piles with 2 m diameter and 280 with 1.8 m diameter.

For the north viaduct, with 1.4 km long, and for the Expo viaduct with 0.7 km long, bored piles, with 1.8 m diameter installed by Teixeira Duarte, were used.

One of the most important considerations for designers is the risk of earthquakes since Lisbon was wiped out by an earthquake in 1755 of 8.5 of Richter magnitude. In the event of serious seismicity activity the new Tagus bridge will be the main access for emergency vehicles crossing the estuary.

Part 2

“The weapons which your hands have found
Are those which Heaven itself has wrought
Light, truth and love; your battle - ground
The free, broad field of thought”
Whities
To the Reformers of England

MAIN GEOLOGICAL CONDITIONS

Introduction

Taking into account the geological data obtained from two site investigation programmes (the first one executed in 1992 by

GATTEL (LNEC, 1993)), the second in 1994 by TEJOPROJECTO (1993a), the ground is composed by the following two main units (Fig. 2):

- a) Alluvial deposits (A1), aged Holocene and Pleistocene;
- b) The bedrock, under alluvial deposits, formed by Plio-Pleistocene and Miocenic materials.

Alluvial deposits

The maximum observed thickness of this unit is around 78 m. In average, its thickness varies between 60 and 70 m.

Five sub-units were defined, named a_0 , a_1 , a_{2a} , a_{2b} and a_3 . The a_0 to a_{2b} units show the common geological structure of alluvial deposits, with lenticular or interstratified layers, with some lateral variations sometimes even inside each sub-unit.

At the bottom of the alluvial deposits there is a gravel layer (a_3), made of fine to coarse gravel, with sand, cobbles and occasionally boulders. The coarser elements (cobbles and occasionally boulders) appear scattered or concentrated in some areas, making in this last case difficult the drilling equipment to go through the a_3 layer.

In the following paragraphs will be presented the general description of each type of the differentiated alluvial deposits.

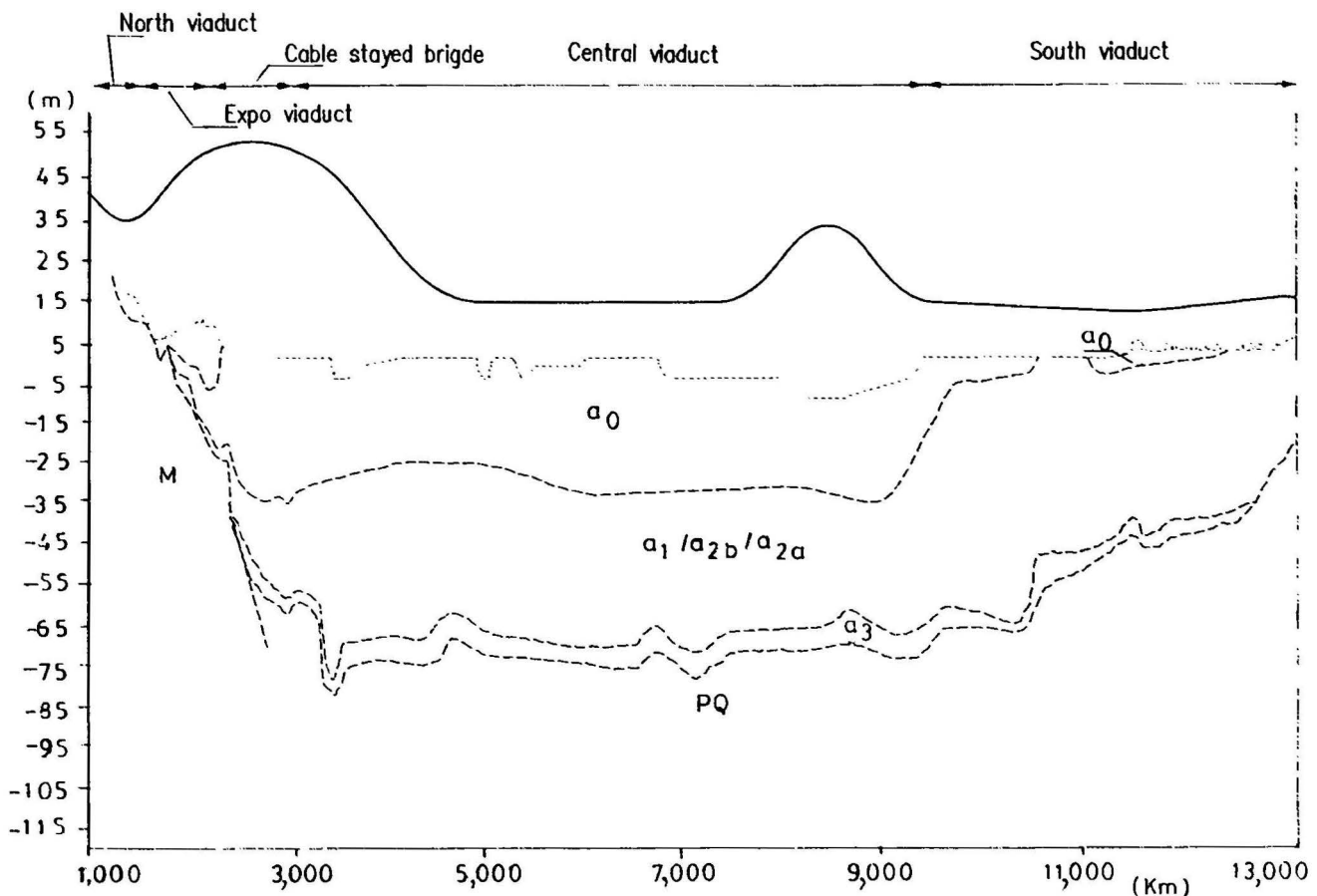


Fig. 2 Simplified geological profile

- a₀ This unit is formed by silty to very silty clay (mud), dark grey, locally greyish brown, sometimes with fine to medium sand bands and lenses, some shells and shell fragments.

Usually, a₀ is the overlying alluvial unit. Its maximum thickness is around 35 m.

- a₁ Fine to medium (occasionally coarse) sands, usually slightly muddy to muddy, dark grey to dark brown, with shells and shell fragments.

- a_{2a} Silty clay to clayey silt, occasionally with sand bands and lenses, dark grey to yellowish brown, sometimes with some scattered fine to coarse gravel. Occasionally black organic matter and shell fragments presence.

- a_{2b} Yellowish brown to grey medium to coarse (occasionally fine) sand, with fine to coarse gravel (rounded to angular). Sometimes with shells and shell fragments. Usually, there is an a_{2b} layer that makes the transition to the a₃ layer.

- a₃ Fine to coarse gravel, rounded to angular, with sand, cobbles and occasionally some boulders, with brown colour.

This unit makes transition of the deposits to the Plio-Pleistocene materials of the bedrock.

Its maximum thickness reaches around 10 m, with an average thickness between 4 and 7 m.

Bedrock (PQ)

The bedrock under the alluvial deposits consists in Plio-Pleistocene and Miocenic materials.

The Plio-Pleistocene materials include basically very dense fine to coarse sands, slightly cemented, sometimes slightly silty and/or clayey, usually with fine to coarse gravel. Some layers of clay to silty and sandy clay can also appear.

The geological structure of this unit is characterised by a more or less regular stratification, gently dipping towards East, around 3-5 degrees.

FIELD INVESTIGATION

The distribution of field tests divided by tender documents and additional site investigation is shown in Table 1. (Tejoprojecto, 1995 a, 1995 b, 1995 c, 1995 d, 1995 e.)

For the river work three jack-up drilling systems "Skate II" with four legs, equipped with a marine drilling mast, which

allow in - situ testing equipment were used. A general view of jack-up drilling system is shown in Fig.3.



Fig. 3 General view of jack-up drilling system

The responsibility for the total soil investigation was taken by Seacore. The piezocone penetrometer tests, vane tests and cone pressuremeter tests were performed by Fugro. The self boring pressuremeter testing and crosshole testing were carried by LNEC and the drilling, coring and sampling on land were performed by Teixeira Duarte.

Hundred fourteen boreholes were performed with a depth between 54 m and 83 m. Standard penetration tests (SPT) were carried out in all boreholes at 1.5 m to 4 m intervals.

Seven boreholes with undisturbed sampling, with a depth between 77 and 84 m, were performed.

Nineteen sets of self boring pressuremeter tests were performed with a depth varying from 3.1 m to 27.3 m (LNEC, 1994 b). From these tests the following parameters were obtained:

- (i) in situ horizontal stress;
- (ii) unload - reload modules G ;
- (iii) undrained shear strength S_u .

Eighteen sets of vane shear tests with depths varying from 2.0 m to 26.7 m were performed. The peak strength values as well as residual values were determined. A view of vane shear equipment is shown in Fig.4.

Eleven crosshole tests with depths varying from 66 to 78 m were performed, (LNEC, 1995 c).

Hundred twelve piezocone penetration tests (PCPT) with a depth from 40 to 67 m were performed. A Fugro electrical cone friction sleeve and porous ceramic filter stone located at the conical tip was used, (Fugro, 1994).

Six seismic cones with depths from 46 to 55 m were performed.



Fig. 4 View of vane shear equipment

LABORATORY TESTS

The distribution of laboratory tests divided by tender documents and additional site investigation is shown in Table 2. (Tejoprojecto 1995a, 1995b, 1995c, 1995d, 1995e).

Two hundred thirty one identification tests, consisted on sieve analyses as well on determinations of liquid limit, W_L , and plastic limit, W_P , were performed. Determinations of natural water content, W_n , were also done.

Sixty oedometre tests with the determination of the values of water content (W_n), degree of saturation (S_r), pressures, compressibility volumetric coefficients (a_v), consolidation coefficients (c_v) and permeability coefficients (k), were performed (CEBTP, 1994).

Fifty eight triaxial tests for the definition of the strength in terms of cohesion (c) and friction angle (ϕ) were done (LNEC, 1994 a).

The curves ($\sigma_1 - \sigma_3$) versus axial strain (ϵ_1), σ_1/σ_3 versus ϵ_1 , variation of pore pressure (u) versus ϵ_1 , and volumetric variation versus ϵ_1 , as well as the stress path and the Mohr-Coulomb envelopes were obtained.

Thirteen direct shear tests for the definition of the strength in terms of cohesion (c) and friction angle (ϕ), were performed.

Twenty-four permeability tests were done.

Twelve chemical tests related with sulphates content, carbonates content and pH values were performed.

Twelve cyclic simple shear tests were done and the values of the dry density (γ_d), water content (w_n), consolidation stress

(σ_{vo}), initial height of the sample (H_0), axial strain (ϵ_1) during consolidation, the ratio τ/σ_{vo} (τ shear stress), the ratio $\Delta u/\sigma_{vo}$ (Δu variation of pore pressure) and γ (shear strain) were obtained (LNEC, 1994c).

The curves G (shear modulus) versus γ (shear strain), \sqrt{G} versus γ , β (damping ratio) versus γ and γ versus τ/σ_0 were obtained.

The curves G (shear modulus) versus γ (shear strain) and β (damping ratio) versus γ (shear strain) for the resonant column tests were also obtained (Fugro, 1994).

Six cyclic triaxial tests, identification tests, the data related with the initial state, the consolidation state and the curves G (shear modulus) versus γ (shear strain) and β (damping ratio) versus γ (shear strain) for different consolidation pressures were done (Mecasol, 1994).

Three torsional shear cyclic tests were performed (LNEC, 1995 a).

Also twelve particle density tests were performed.

Table 1 Distribution of field tests

TESTS	LNEC/GATTEL	ACE/TEJOPROJECTO	TOTAL
BOREHOLES	23	91	114
UNDISTURBED SAMPLING	0	7	7
SELF BORING PRESSURE-METER	2	17	19
VANE SHEAR TESTS	4	14	18
CROSSHOLE	1	10	11
PCPT	4	108	112
SEISMIC CONE	0	6	6

Table 2 Distribution of laboratory tests

TESTS	LNEC/GATTEL	ACE/TEJOPROJECTO	TOTAL
IDENTIFICATION	25	206	231
SIEVE CURVES	25	204	229
OEDOMETRE	4	56	60
TRIAXIAL	6	52	58
CYCLIC SIMPLE SHEAR	0	12	12
DIRECT SHEAR	0	13	13
PERMEABILITY	0	24	24
CHEMICAL	0	12	12
RESONANT COLUMN	0	6	6
CYCLIC TRIAXIAL	0	6	6
TORSIONAL SHEAR CYCLIC	0	3	3
PARTICLE DENSITY	0	12	12

GEOTECHNICAL CHARACTERISTICS

After a balance between the results of the field tests and laboratory tests the following geotechnical characteristics were considered (Tejoprojecto, 1995a, 1995b, 1995c, 1995d, 1995e).

Alluvial material (a_0)

Thickness = from 17.5 to 33 m

Unified classification CL, CH

% passing sieve # 200 (ASTM) 68 to 99%

Liquid limit 31 to 74%

Plastic limit 21 to 33%

Natural water content 38 to 71%

Triaxial tests (U.U.):

Cohesion strength (C_u) 8 to 41 kPa

Triaxial tests (C.U.):

Cohesion (effective) $c' = 0$ to 14.2 kPa

Friction angle $\phi' = 12.3$ to 36.5°

Triaxial tests (C.D.):

Cohesion (effective) $c' = 0$ to 45 kPa

Friction angle $\phi' = 15$ to 29°

Oedometer tests:

a_v (compressibility volumetric coefficient) = 8 to $39 \times 10^{-7} \text{ m}^2/\text{N}$

c_v (consolidation coefficient) = 1.2 to $54 \times 10^{-8} \text{ m}^2/\text{s}$

k (permeability coefficient) = 1.2 to $230 \times 10^{-11} \text{ m/s}$

The results of cyclic simple shear tests are summarised in Table 3.

Table 3 Cyclic simple shear tests

γ (%) (shear strain)	0.1	0.5	1	5
G (MPa) (shear modulus)	14 to 18	5 to 8	4 to 6	1.0
D (%) (damping coefficient)	15 to 17	14 to 22	19 to 25	23 to 31

The crosshole tests have given the following results:

Shear wave velocities V_s from 53 to 208 m/s

Longitudinal wave velocities V_p from 665 to 1526 m/s

The variation of V_s with depth is shown in Fig. 5.

SPT results were between 0 and 4 blows, with a large frequency of 0 values and the higher values related with silty materials.

PCPT tests, with measurement of pore pressures, have given point resistances between 0.15 and 1.0 MPa, with an increase with depth. This trend is illustrated in Fig. 6.

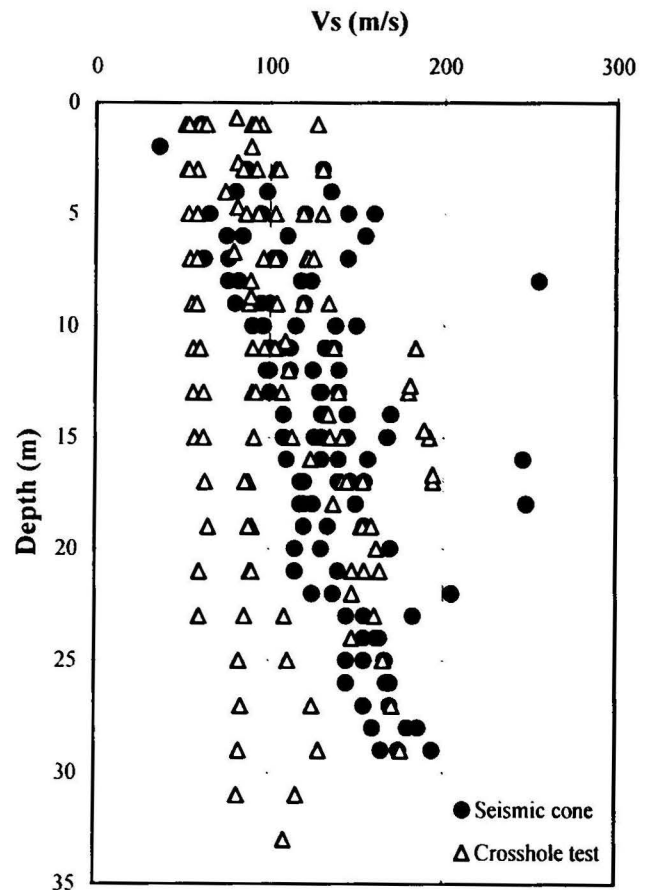


Fig. 5 Variation of V_s with depth

Pore pressures values have allowed the identification of material, higher values were related with mud materials.

Vane shear tests have given for undrained strength the following results:

peak values - 12.5 to 51 kPa

residual values - 4 to 20.3 kPa

The variation of these values is shown in Fig. 7.

Alluvial material (a_1)

Thickness = variable

Unified classification SM, SC and SW

% passing sieve # 200 (ASTM) 4 to 34%

Liquid limit - none to 32%

Plastic limit - none plastic to 24%

Natural water content 16 to 36%

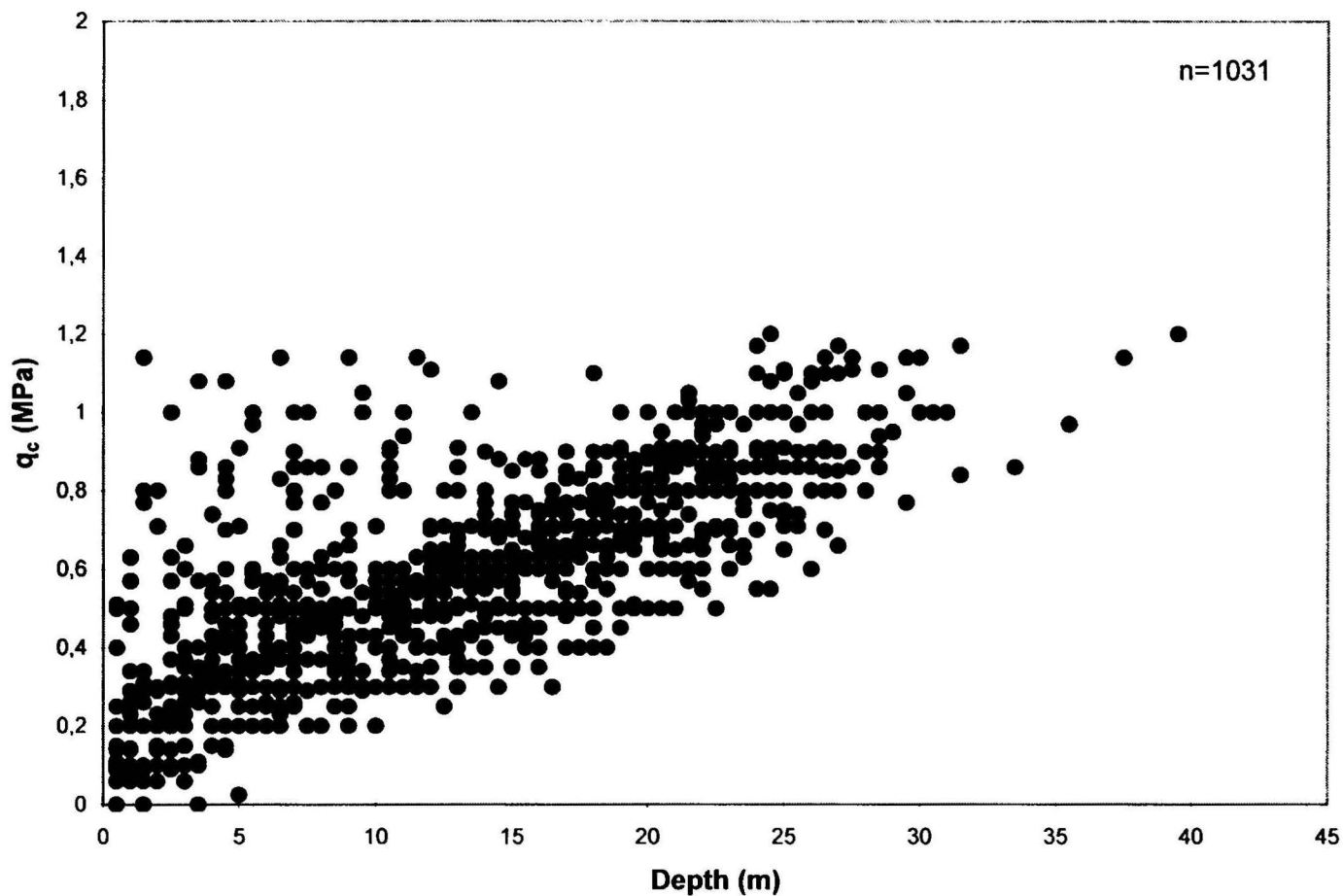
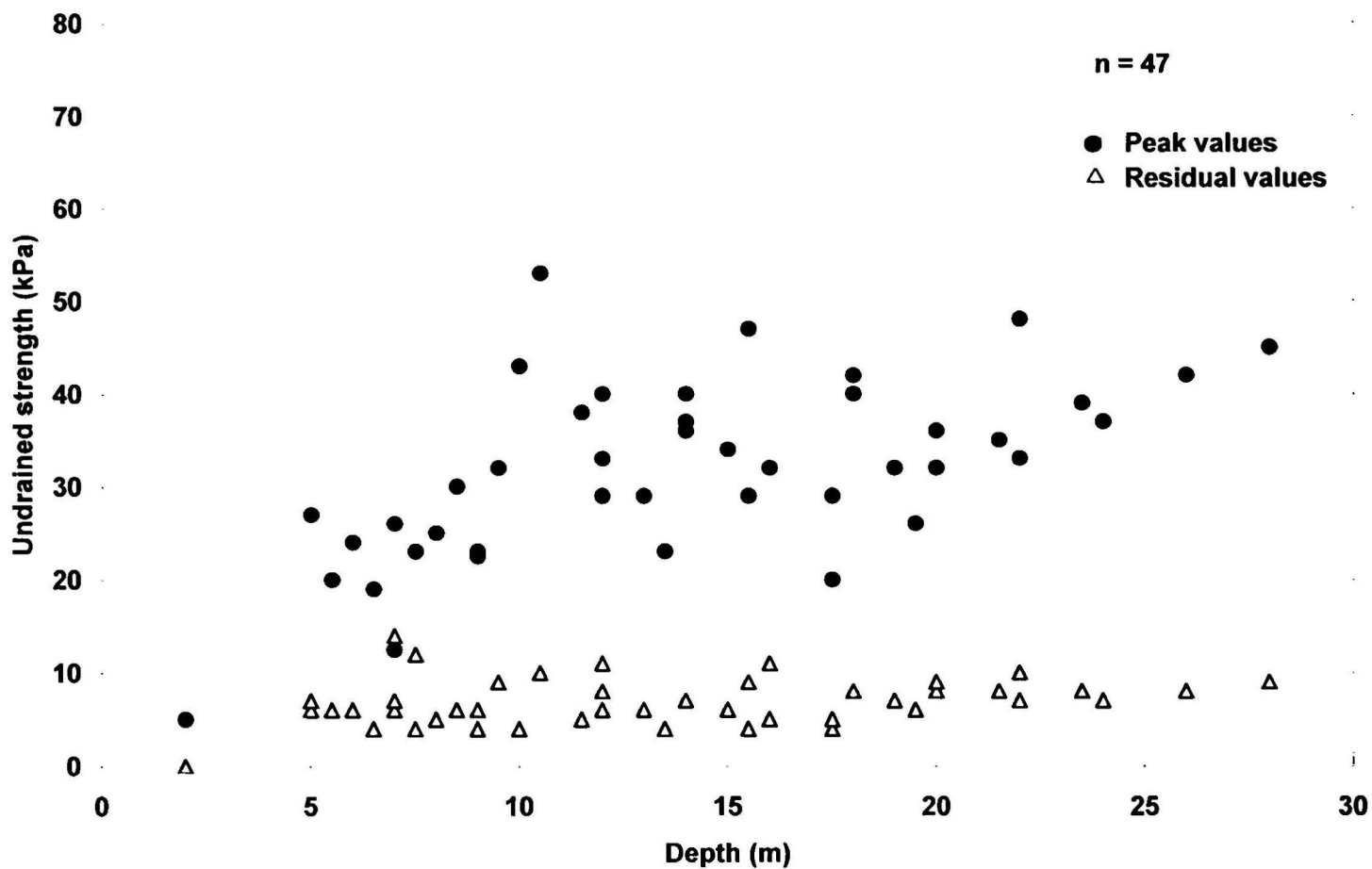
Fig. 6 Variation of q_c values with depth

Fig 7 Variation of undrained strengths with depth

Triaxial tests (C.U.):

Cohesion (effective) $c' = 0$ to 1.5 kPa
Friction angle $\phi' = 22.8$ to 40°

Triaxial tests (C.D.):

Cohesion (effective) $c' = 0$ to 59.3 kPa
Friction angle $\phi' = 33$ to 37.8°

The results of cyclic simple shear tests are summarised in Table 4.

Table 4 Cyclic simple shear tests

γ (%) (shear strain)	0.1	0.5	1	5
G (MPa) (shear modulus)	22 to 40	15 to 20	4 to 6	1 to 2
D (%) (damping coefficient)	16 to 18	19 to 21	23 to 25	29 to 31

The crosshole tests have given the following results:

Shear wave velocities V_s from 159 to 223 m/s
Longitudinal wave velocities V_p from 946 to 1696 m/s.

SPT results were between 4 and > 60 blows, and no correlation with depth was obtained.

PCPT tests with measurement of pore pressures have given point resistances between 3 to 28 MPa.

A comparison between the experimental values of G/G_{max} and damping ratio and the proposals of Seed et al. (1984, 1970) is presented in Figs. 8 and 9.

Alluvial material (a_2)

The following results were obtained (a_{2a}):

Triaxial tests (U.U.):

Undrained strength (C_u): 40 to 42 kPa

Triaxial tests (C.U.):

Cohesion (effective) $c' = 0$

Friction angle $\phi' = 40$ to 42°

Triaxial tests (C.D.):

Cohesion (effective) $c' = 0$

Friction angle $\phi' = 27^\circ$

The crosshole tests have given the following results:

Shear wave velocities V_s from 162 to 310 m/s

Longitudinal wave velocities V_p from 1201 to 1674 m/s.

For the materials a_{2a} clay and silty clay, the SPT results were, in average, between 6 and 27 blows. The PCPT tests have given point resistances between 1 to 6 MPa.

The crosshole tests have given the following results for the cohesionless materials (a_{2b}):

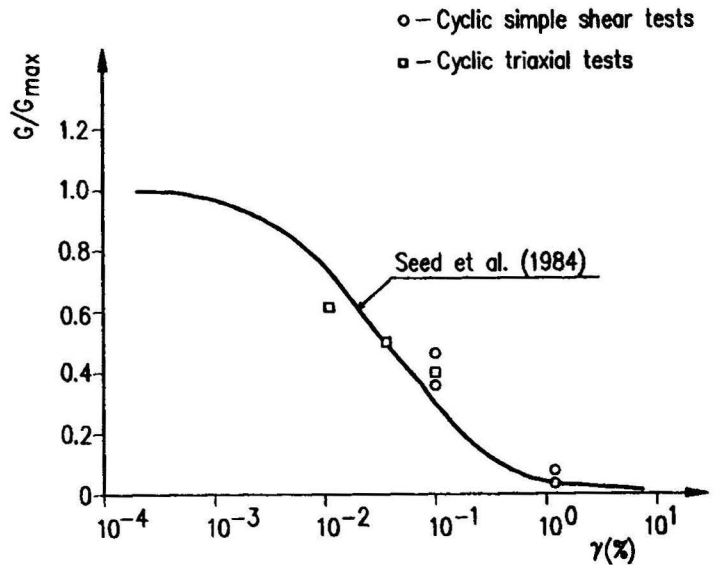


Fig. 8 Experimental values of G/G_{max}

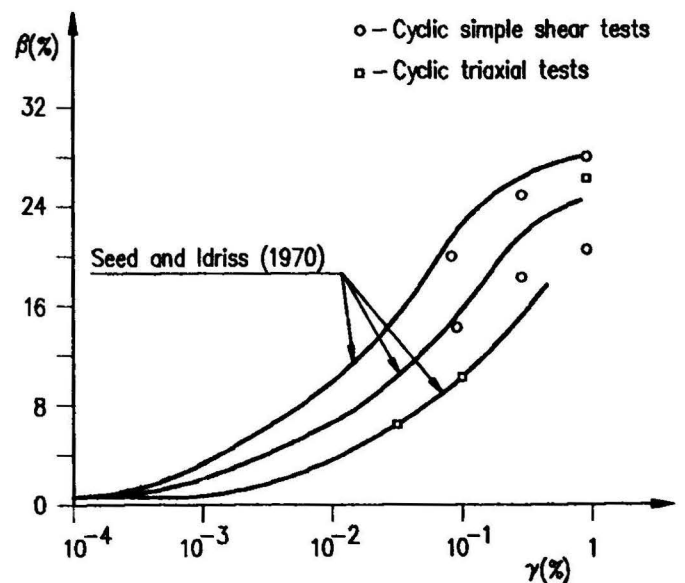


Fig. 9 Experimental values of damping ratio

Shear wave velocities V_s from 152 to 329 m/s
 Longitudinal wave velocities V_p from 1123 to 2195 m/s.

For the materials a_{2b} , medium to coarse sand with gravel, the SPT results were between 29 and > 60 blows. The PCPT tests have given point resistances q_c between 9 and 33 MPa. A correlation between SPT values and q_c values is presented in Fig. 10.

Alluvial material (a_3)

The crosshole tests have given the following results:

Shear wave velocities V_s from 267 to 469 m/s
 Longitudinal wave velocities V_p from 1531 to 2637 m/s.

In general SPT values were higher than 60 blows and have shown some dispersion.

A correlation between V_s and SPT blows for a_1 , a_{2b} , a_3 and PQ is given in Fig. 11.

Part 3

“Life is the art of drawing sufficient conclusions from insufficient premises”
 (Samuel Butler)

DESIGN SURFACE SPECTRA

Introduction

To derive the design free field surface spectra a very comprehensive analysis was performed. Five sites were selected, for the soil material upper and lower bound values were considered and for the seismic action both deterministic and stochastic approaches were used. Linear and non-linear analyses were performed.

Seismic action

The seismic action was based on the Portuguese Code (RSA, 1983) and defined by a stochastic gaussian stationary vectorial process (two horizontal orthogonal components and one vertical component). The Portuguese territory is affected by two seismotectonic sources: (i) near source which represents a moderate magnitude earthquake at a short focal distance with a duration of 10 seconds; (ii) far source which represents a higher magnitude earthquake at a longer focal distance with a duration of 30 seconds.

For the deterministic approach ten artificial time histories of acceleration were produced. For the computation of these accelerograms the validation criteria of EC8 (1994) was considered.

For the stochastic approach power spectral density functions based on RSA (1983) were used.

The mean value of those spectra were factored by 4.5 as required by GATTEL specifications to correspond to the ultimate limit state of collapse event.

Response Analyses

The soil profile was modelled by a vertical column of plane strain finite elements and by a rectangular finite element mesh.

Two approaches were assumed (P&V, 1994 a): (i) a rigid base condition - the motion is directly imposed at the boundary nodes; (ii) a rock outcrop motion - with the impedance of the elastic half space - imposing the boundary condition.

Five soil profiles (SR1, SR2, SR6, SR7 and SR10) were studied in order to represent the geotechnical conditions for the Expo viaduct, Cable stayed bridge, Central viaduct and South viaduct.

The upper and lower bound properties of the materials were assumed.

For the SR2 soil profile, corresponding to the left bank pylon of the cable stayed bridge, the following dynamic properties were assumed based on in situ tests, as the results of laboratory tests were not considered reliable, due the unavoidable remoulding during sampling (Geodynamique et Structure, 1994a). The geotechnical properties for different materials are summarized in Table 5.

Table 5 Geotechnical properties for soil materials

Layer	Bulk Density (kN/m ³)	Maximum Average Shear Modules (MPa)	Damping Ratio	Poisson Ratio
a_0 - mud	16.2	20 - 40	5 - 25	0.4 - 0.45
a_1 - silty sand	19.2	80 - 200	6 - 23	0.3 - 0.4
a_3 - sandy gravel	19.6	140 - 300	6 - 22	0.25 - 0.30
bedrock	20.6		4 - 6	0.2

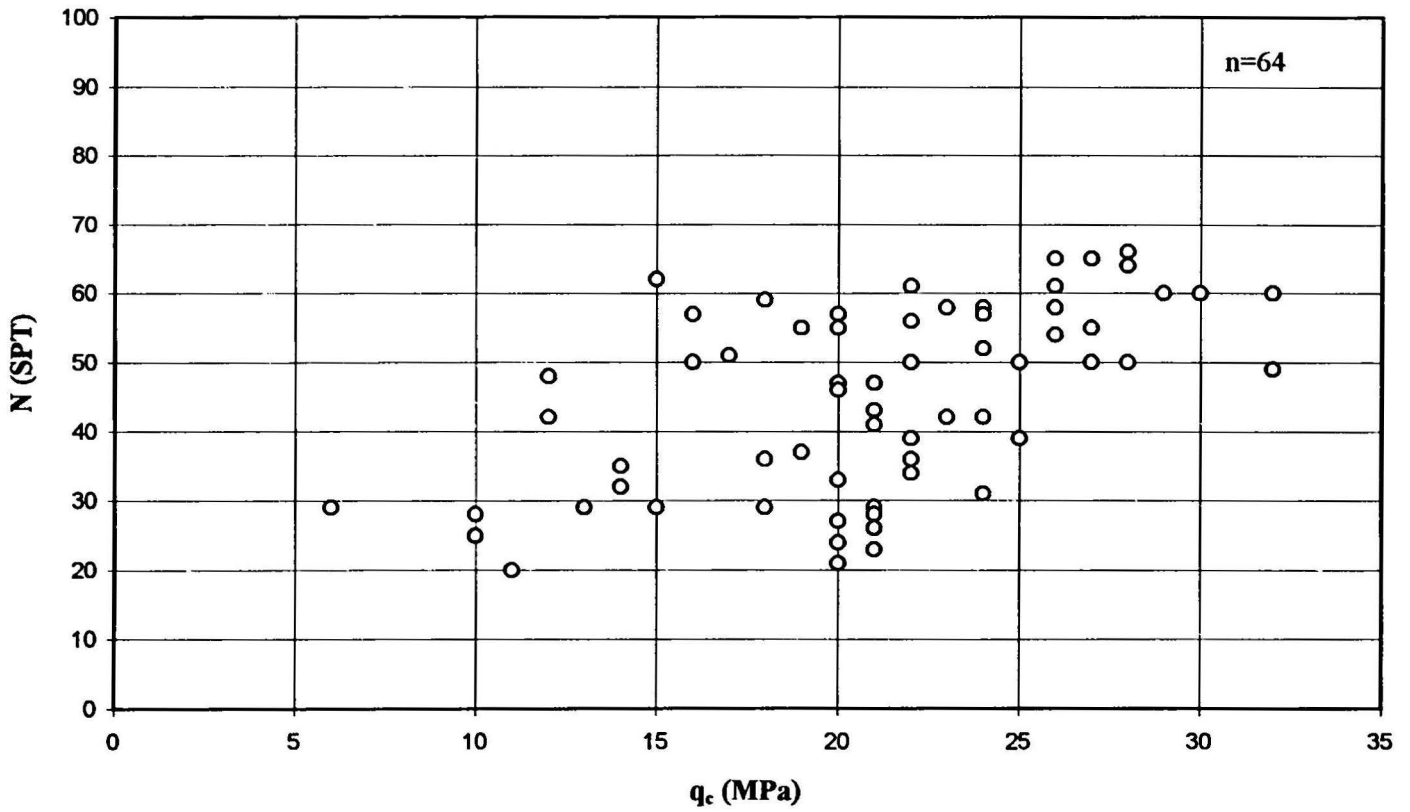


Fig. 10 Correlation between N(SPT) values and qc values

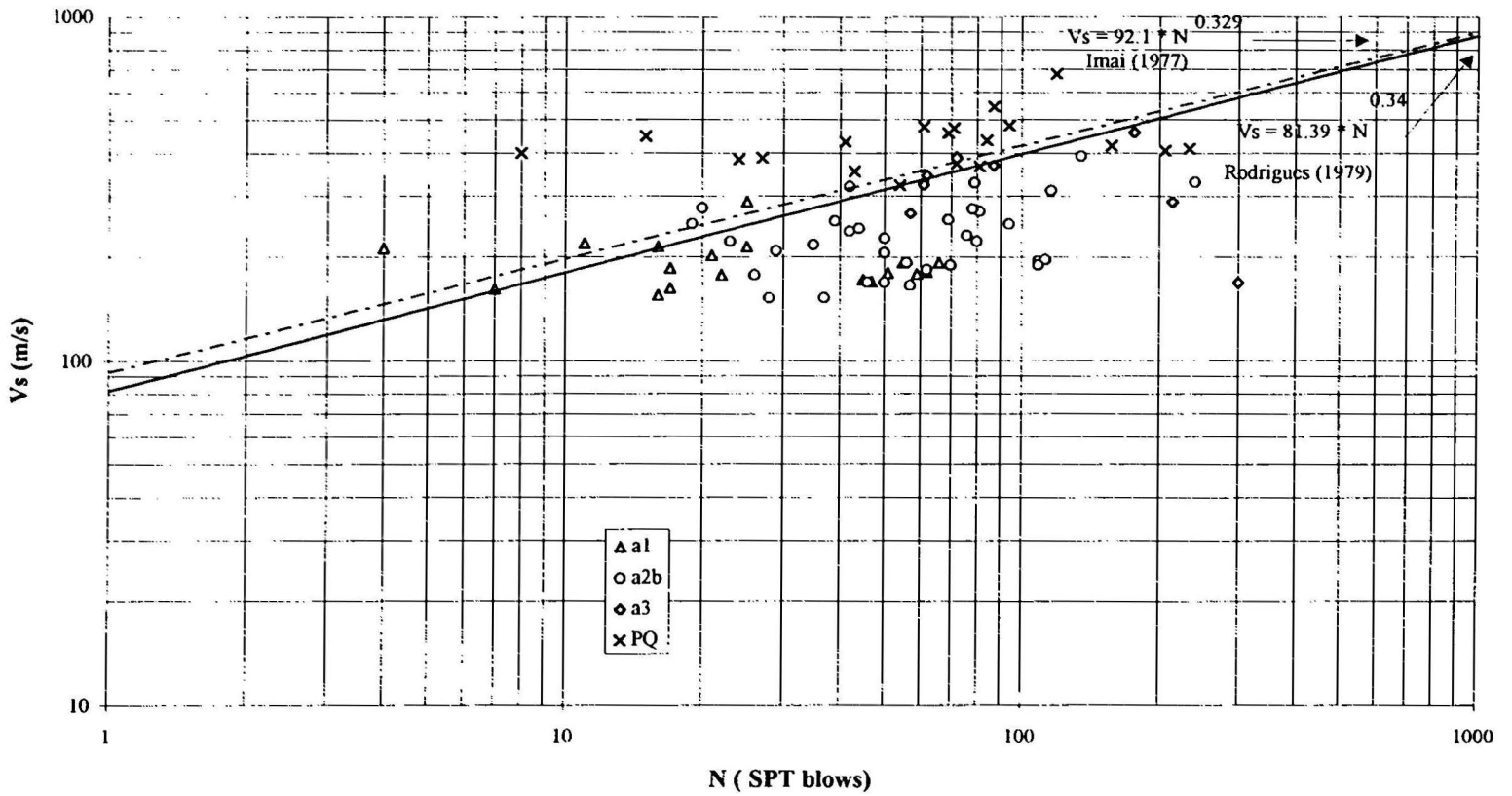


Fig. 11 Correlation between Vs and SPT values

To evaluate the influence of the direction of incident seismic movement the results of analyses considering only horizontal movements and both horizontal and vertical movements are summarized in Fig. 12.

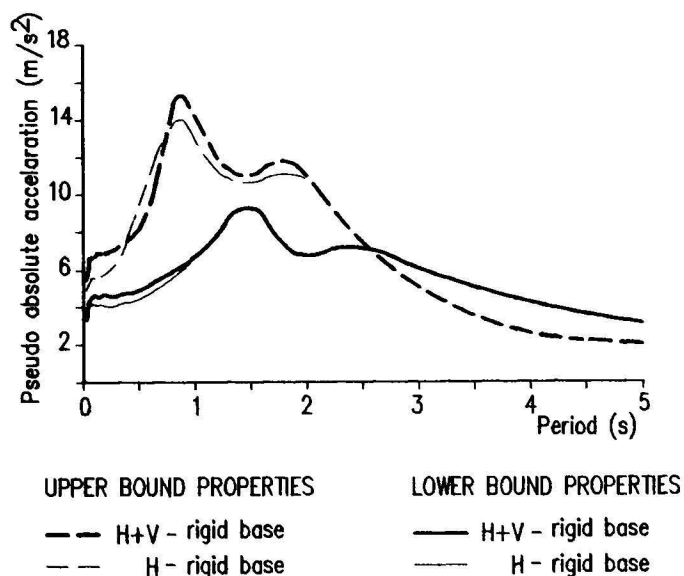


Fig. 12 Computed pseudo absolute acceleration by stochastic analyses for rigid base (adopted from P & V , 1994a)

The results of column stochastic analyses considering both rigid base and boundary condition with impedance are shown in Fig. 13.

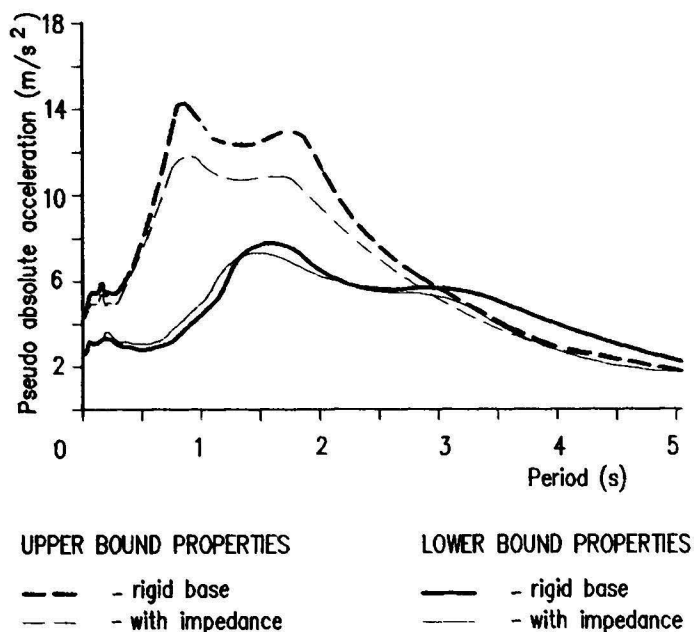


Fig. 13 Computed pseudo absolute acceleration by stochastic analyses for rigid base and with impedance (adopted from P & V , 1994a)

Analyses using the program SHAKE (Schnabel, 1972), and the program DYNAFLOW (Prevost, 1993) were performed by Geodynamique et Structure (1994 b). Also non linear analyses with the code SIREN were performed by Ove Arup & Partners (1994).

A comparison between linear and non linear results is shown in Fig. 14 (EEG, 1994 a).

A comparison between the results obtained by the SIREN and DYNAFLOW programs is presented in Fig. 15 (EEG, 1994 a).

A comparison between the results obtained by P & V and G S is shown in Fig. 16 (EEG, 1994b).

Conclusions

From the performed analyses the following conclusions can be taken:

- (i) A good agreement between stochastic and deterministic results were obtained.
- (ii) The boundary condition has a non negligible effect on the results, particularly for the upper bound parameters.
- (iii) The vertical input has a non negligible influence in the upper bound soil parameters.
- (iv) The surface response spectra calculated for the ultimate limit state by the program SIREN are in reasonable agreement with the values obtained by the program DYNAFLOW.

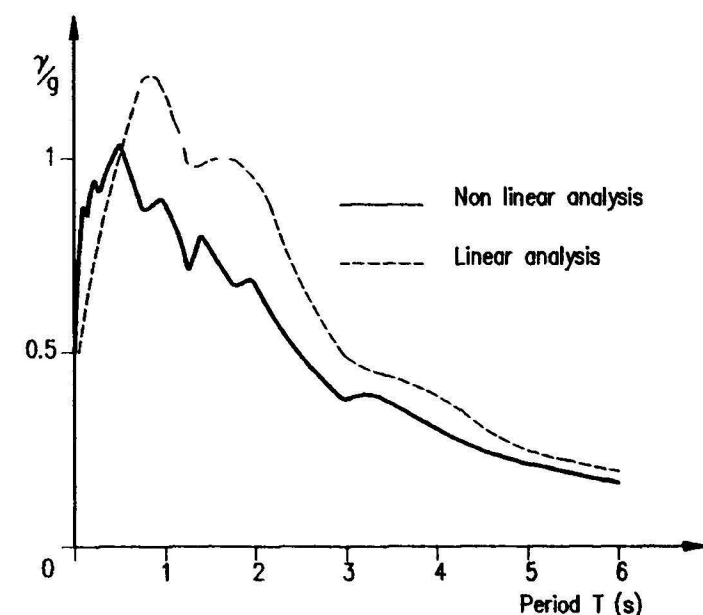


Fig. 14 A comparison between linear and non linear analyses (adopted from EEG, 1994a)

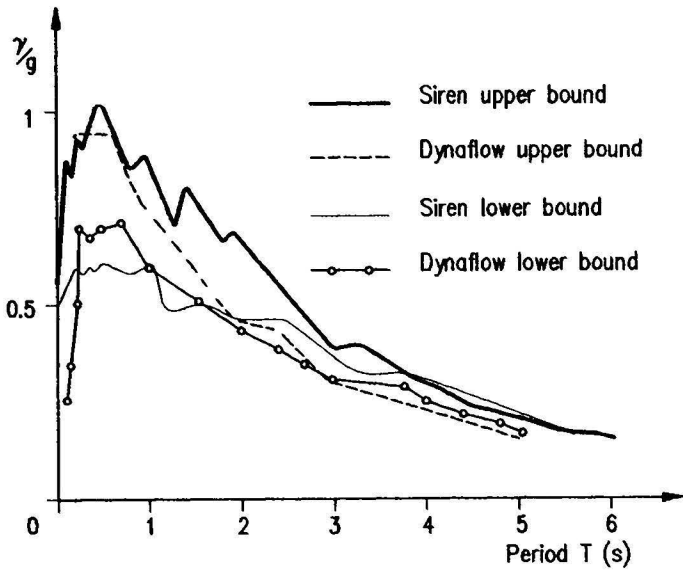


Fig. 15 A comparison between non linear analyses performed by SIREN and DYNAFLOW codes (adopted from EEG, 1994a)

(v) The linear equivalent model has given more conservative results than the non linear model.

LIQUEFACTION ASSESSMENT

The liquefaction potential evaluation was performed only by field tests taking into account the disturbance that occurs during sampling of sandy materials.

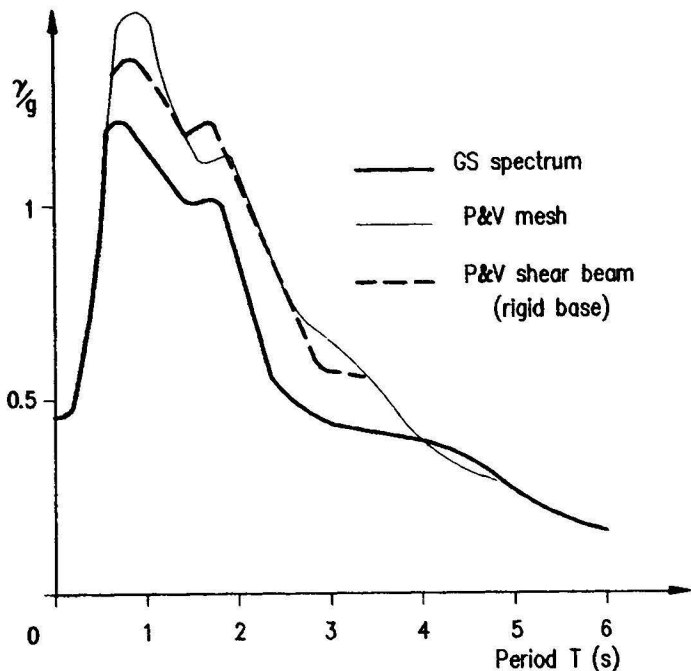


Fig. 16 A comparison between the analyses performed by P&V and GS (adopted from EEG, 1994b)

In this analysis attention was drawn for SPT and CPT tests as the seismic tests have only been used when soil contains gravel particles (Tejoprojecto, 1995 f).

In a first step the shear values were computed from a total stresses model, that gave results on the conservative side using the program "SHAKE", for a second step it was performed an analysis in effective stresses using the computer program DYNAFLOW. The geotechnical profile of SR2 was used for the study of the piers belonging to the Exposition viaduct, Cable stayed bridge, the SR6 and SR7 geotechnical profiles for Central viaduct and SR10 for South viaduct.

Fig. 17 shows the results for SR2 (Geodynamique et Structure, 1994b).

Corrections related with SPT tests due the depth effect and the equipment were performed following the recommendations of EC8 (1994).

The sieve curves of materials a₁ and a_{2b}, (Figs. 18 and 19) have given the following results (Table 6).

Table 6 Sieve characteristics of the materials

Structure	Material a ₁			Material a _{2b}		
	F _p (%)	D ₅₀ (mm)	NCG	F _p (%)	D ₅₀ (mm)	NCG
Main Bridge	17.8	0.13	4	-	-	-
Central Viaduct	15.9	0.14	16	10	0.4	18
South Viaduct	11.2	0.14	29	2.8	0.7	4

where the symbols have the following meaning:

- F_p- percentage of the material that passes through the sieve # 200
- D₅₀ - diameter corresponding to the percentage of 50%
- NCG - number of the sieve curves.

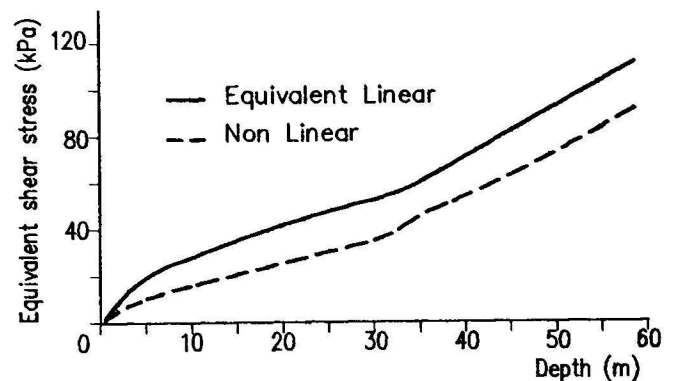
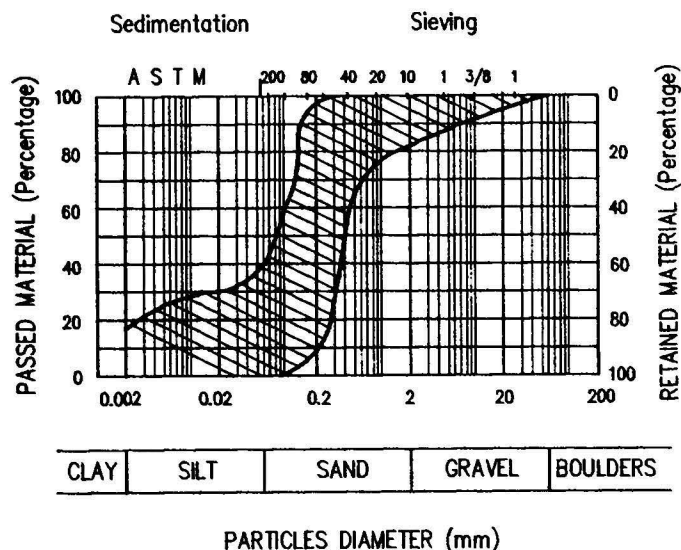
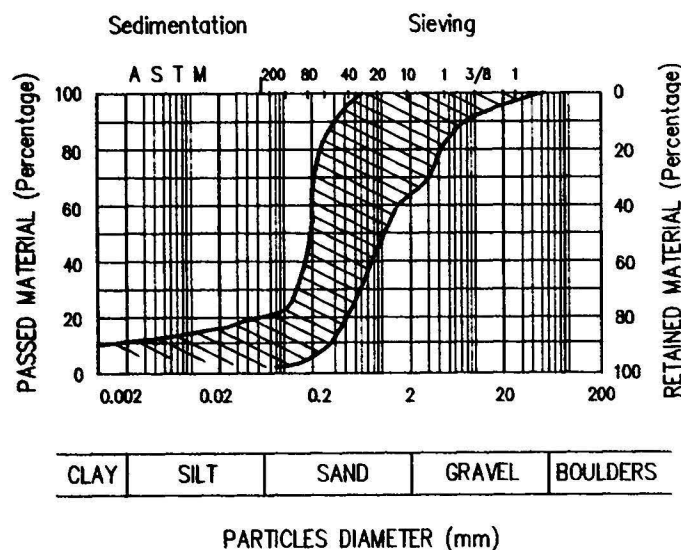


Fig. 17 Equivalent shear stresses computed from SHAKE and DYNAFLOW codes (adopted from Geodynamique et Structure, 1994b).

Fig. 18 Sieve curves for material a_1

Taking into account that we are dealing with underwater materials, the sieve curves exhibit percentages of fines lower than in reality, as a consequence of the washing effect during the sampling.

Fig. 19 Sieve curves for material a_{2b}

The liquefaction potential evaluation was given in tables and the columns have included the following data: (i) columns 1 to 4, reference to the pier, type of test (SPT or CPT), depth of the test and thickness of the layer; (ii) columns 5 and 6, values of N_m (SPT) and $(q_c)_m$ (CPT); (iii) columns 7 and 8, effective overburden pressure (σ'_o) and correction factor (C_N); (iv) columns 9 and 10, normalised values N_1 (60) (SPT) and $(q_c)_1$ (CPT); (v) column 11, τ_{equiv} (equivalent shear stress value); (vi) column 12 (τ/σ'_o ratio value), column 13 (τ/σ'_o ratio value with a safety factor of 1.1), column 14 (τ/σ'_o ratio value with the safety factor of 1.25); (vii) column 15, Ref. (reference of the analysed SPT or CPT value); (viii) column 16, liquefaction susceptibility analysis.

Taking into account the dilatant behaviour of the material

observed in the CPT tests and the values of the pore pressures developed in the cyclic simple shear tests and cyclic torsional shear tests, where the registered values of the pore pressures rarely reach the value of 80%, being frequently below 60%, a safety factor of 1.1 can be considered sufficient. Nevertheless, at the present case, a conservative analysis was performed, with a safety factor of 1.25 being adopted, as recommended in EC8, Part 5 (1994).

Table 7 presents an application for Cable stayed bridge. The liquefaction potential evaluation, by SPT and CPT tests, is shown in Figs. 20 and 21.

The liquefaction susceptible zones for the different structures are shown in Table 8.

Table 8 Summary table liquefaction susceptible zones

Structure	Material a_1			Material a_{2b}		
	Treated Values	Piers Zones Susceptible to Liquefaction	Percentage	Treated Values	Piers Zones Susceptible to Liquefaction	Percentage
Exposition Viaduct	7	0	0	6	1	17
Main Bridge	27	2	7	24	1	4
Central Viaduct	291	19	7	333	14	4
South Viaduct	90	20	22	43	0	0
TOTAL	415	41	10	406	16	4

SEISMIC PILE ANALYSIS

For the seismic analysis of piles with 2.2 m of diameter the program TRIMO was used to compute the frequencies and the modes of vibration (P&V, 1994 b).

The geotechnical characteristics of the materials were selected for a value of shear strain around 1%.

For each direction 88 frequencies were computed and the first 10 computed values are presented in Table 9.

Table 9 Natural computed frequencies

Mode	Longitudinal Direction	Transversal Direction
1	0.253	0.252
2	0.387	0.400
3	0.473	0.480
4	0.516	0.521
5	0.577	0.590
6	0.602	0.620
7	0.710	0.741
8	0.733	0.757
9	0.829	0.860
10	0.850	0.880

Table 7 Cable stayed bridge
Evaluation of liquefaction potential material a₁

(1) Pier	(2) No. of Borehole or CPT	(3) Depth (m)	(4) Thic kness (m)	(5) N _m	(6) (q _c) _m (MPa)	(7) σ' _o (kPa)	(8) C _N	(9) N ₁ (60)	(10) (q _c) ₁ (MPa)	(11) τ _{equiv.} (kPa)	(12) τ/σ' _o	(13) τ/σ' _o × 1,1	(14) τ/σ' _o × 1,25	(15) Ref.	(16) Remarks
PS	BD/PS	34,2-38,2	4,0	52	-	338	0,58	30	-	53	0,16	0,17	0,20	13	N.L
PS	CPTD/PS	33,5-38,0	4,5	-	8,5	324	0,44	-	3,7	48	0,15	0,16	0,19	14	N.L
PS	B/PS	34,5-35,3	0,8	14	-	311	0,61	9	-	45	0,14	0,16	0,18	15	L
"	"	35,3-36,7	1,4	45	-	324	0,59	27	-	48	0,15	0,16	0,19	16	N.L
"	"	36,7-38,8	2,1	20	-	338	0,58	12	-	53	0,16	0,17	0,20	17	L
PS	BU/PS	31,8-36,0	4,2	23	-	306	0,61	14	-	44	0,14	0,16	0,18	18	N.L
"	"	37,7-39,7	2,0	31	-	342	0,56	17	-	53	0,15	0,17	0,19	19	N.L
PS	CPTU/PS	31,8-36,0	4,2	-	7,5	306	0,46	-	3,45	44	0,14	0,16	0,18	20	N.L
PS/P4	B/PS-P4	33,5-38,5	5,0	48	-	315	0,59	28	-	47	0,15	0,16	0,19	21	N.L
"	"	38,5-41,5	3,0	26	-	356	0,55	14	-	58	0,16	0,18	0,20	22	N.L
P4	CPT/P4	33,5-45,0	11,5	-	8,5	347	0,44	-	3,74	54	0,16	0,17	0,19	23	N.L
P5	CPT/P5	34,0-44,0	10,0	-	9	351	0,44	-	3,96	55	0,16	0,17	0,20	24	N.L
P6	B/P6	33,0-38,0	5,0	51	-	324	0,59	30	-	48	0,15	0,16	0,19	25	N.L
"	"	38,0-42,0	4,0	58	-	360	0,55	32	-	59	0,16	0,18	0,20	26	N.L
"	"	42,0-46,0	4,0	43	-	396	0,53	23	-	66	0,17	0,18	0,21	27	N.L

N_m - SPT value
 (q_c)_m - CPT cone resistance value
 σ'_o - Effective overburden pressure
 C_N - Correction factor for overburden pressure

N₁ (60) - Normalized SPT value
 (q_c)₁ - Normalized CPT cone resistance value
 τ_{equiv.} - Equivalent cyclic shear stress
 L - Liquefaction
 N.L - No Liquefaction

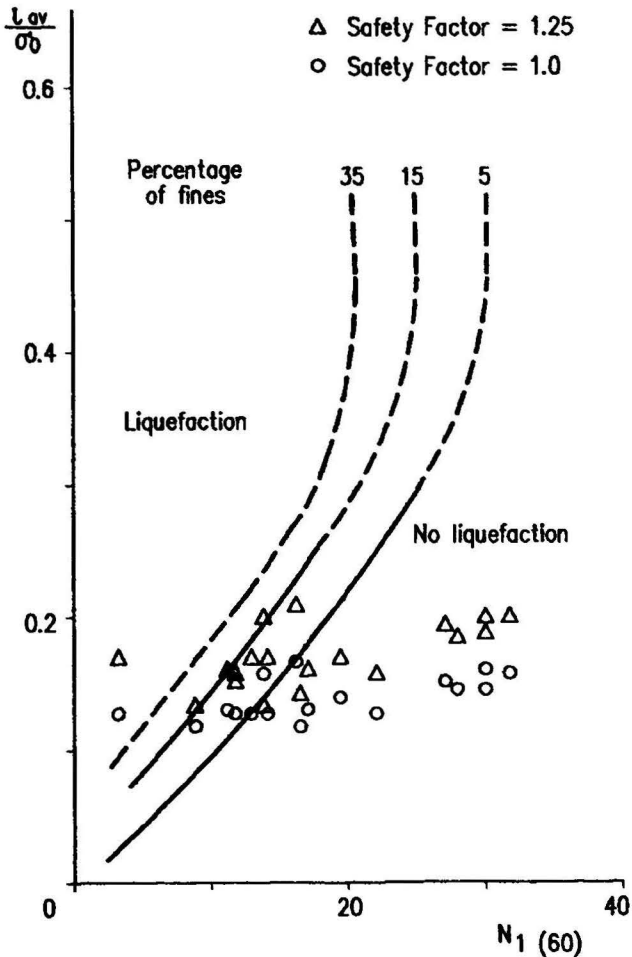


Fig. 20 Liquefaction potential evaluation by SPT tests

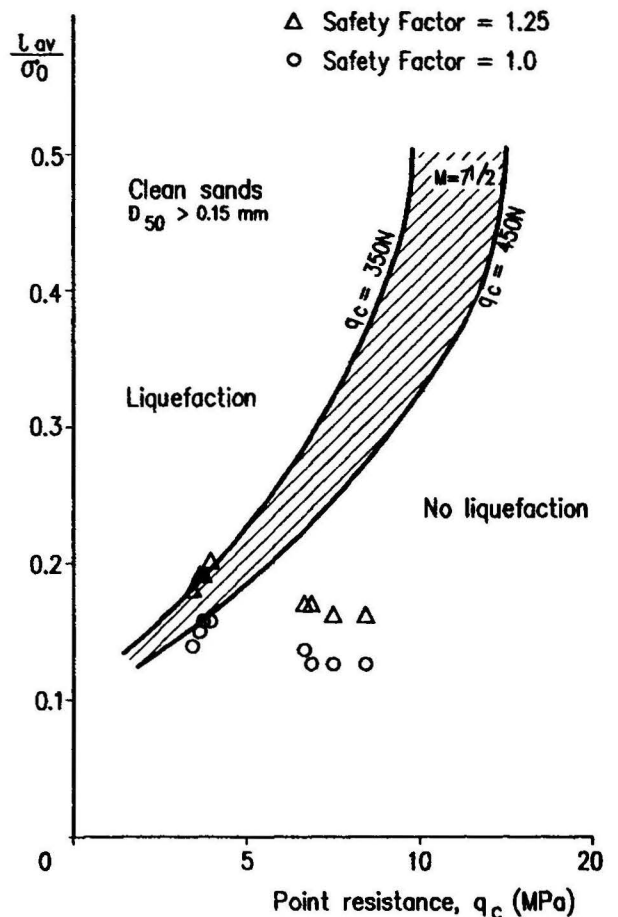


Fig. 21 Liquefaction potential evaluation from CPT tests

The displacement values, the axial forces, the shear forces and the bending moments for the piles were computed.

The computed values of displacements, axial forces and bending moments for longitudinal directions are shown in Figs 22 to 24.

PILE LOAD TESTS

Introduction

Pile load tests were performed with the following purposes (Tejoprojecto, 1993 b):

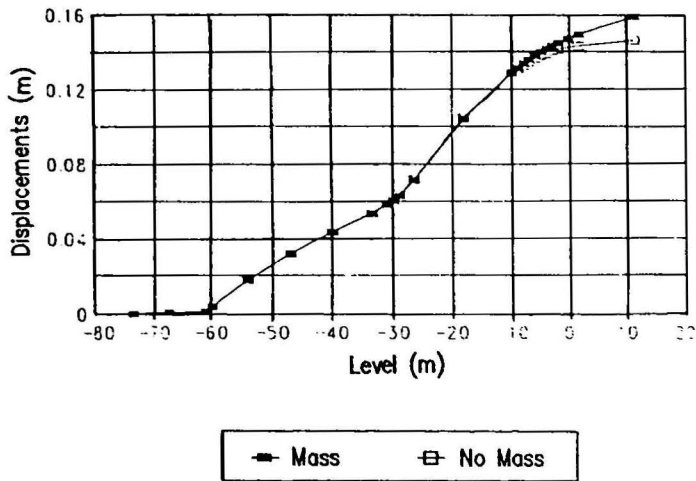


Fig. 22 Piles computed displacement values (adopted from P & V, 1994b)

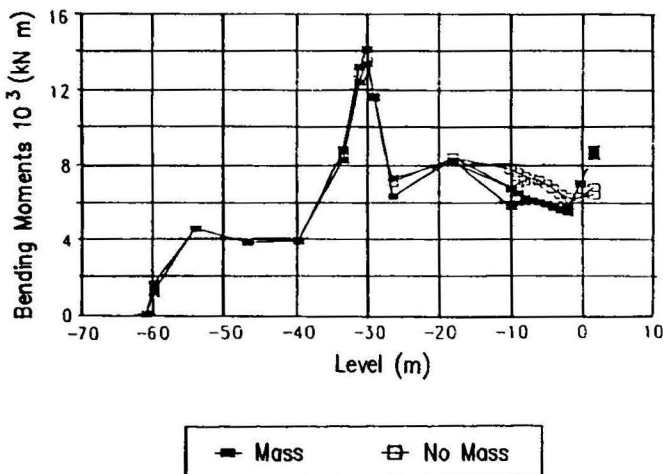


Fig. 23 Piles axial computed forces (adopted from P & V, 1994b)

- i) to determine the response of a representative pile and the surrounding ground to load, both in terms of settlements and limit load;

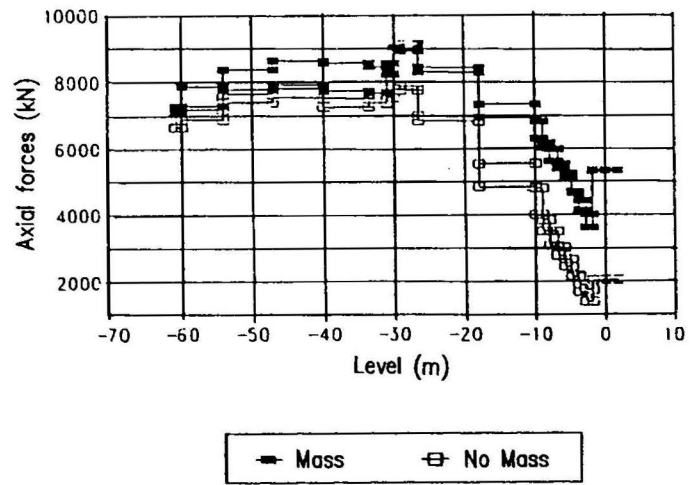


Fig. 24 Piles bending computed moments (adopted from P & V, 1994b)

- ii) to check the performance of individual piles and to allow judgement of the overall pile foundation;
- iii) to assess the suitability of the construction method.

Load tests were carried out on several test piles and the test locations were representative of the site of the pile foundation and one test pile was located where the most adverse ground conditions are believed to occur.

Load tests were carried out on trial piles which were built for test purposes before the final design.

The results of load tests should be used to calibrate the design parameters and so to optimise the suggested values for pile lengths, based only on the interpretation of site investigation and laboratory and in situ test results.

Vertical pile load tests

Vertical load tests were performed on 3 piles located at Main bridge (P8), Central viaduct (P31) and South viaduct (P79).

The construction of bored piles had the following steps:

- i) installation by vibrodriving with a SOILMECH VTE 12000 of a permanent casing with an outside diameter: 1216 mm, a thickness of 8 mm and 16 mm at the shoe level and a length of 40 m.
- ii) excavation of the soil inside the casing with a bucket of 1180 mm diameter and a SOILMECH rotary machine RT - 3ST.
- iii) boring below the bottom of the casing for a length higher than 19 m with a bucket using a polymeric drilling fluid GEOMUD - 15 mixed with salty Tagus water with the following composition: 2 kg of polymer per 1000 l of

water. The mixture had a Marsh viscosity 40'' and a density 1.035.

For the vertical load test the following equipments were installed: 8 electrical displacement transducers, 2 mechanical dial gauges, 2 strips of LCPC removable extensometers, with a resolution of 10^{-6} , 1 temperature sensor, 1 high precision pressure transducer, 1 hydraulically operated pump, 4 hydraulic jacks and 1 optical level (AGISCO, 1995). The loading program consisted in reaching 20000 KN with 8 load increments.

A general view for vertical pile load tests is presented in Fig. 25.

The load - settlement curves for piles P8, P31 and P 79 are shown in Fig. 26 (LCPC, 1995).

Failure loads were defined as settlement equal to 10% of the pile diameter, i.e. at 120 mm settlement. Table 10 gives the values of predicted failure loads based from CPT tests and the observed values. The latter are lower than the predicted loads, with the exception of P79 (the length of this pile was increased 10 m) and the difference were attributed to the lower shaft friction values. The effect of grouting on the soil gave insufficient gain in bearing capacity, as can be assessed by P31i.



Fig. 25 General view for vertical pile load tests

Table 10 Failure loads

P8		P31		P79		P31i
m	p	m	p	m	p	m
15	20.3	15	21.4	>21.15	>22.7	>17.5
				24.5		

m - measured loads in MN
p - predicted loads in MN

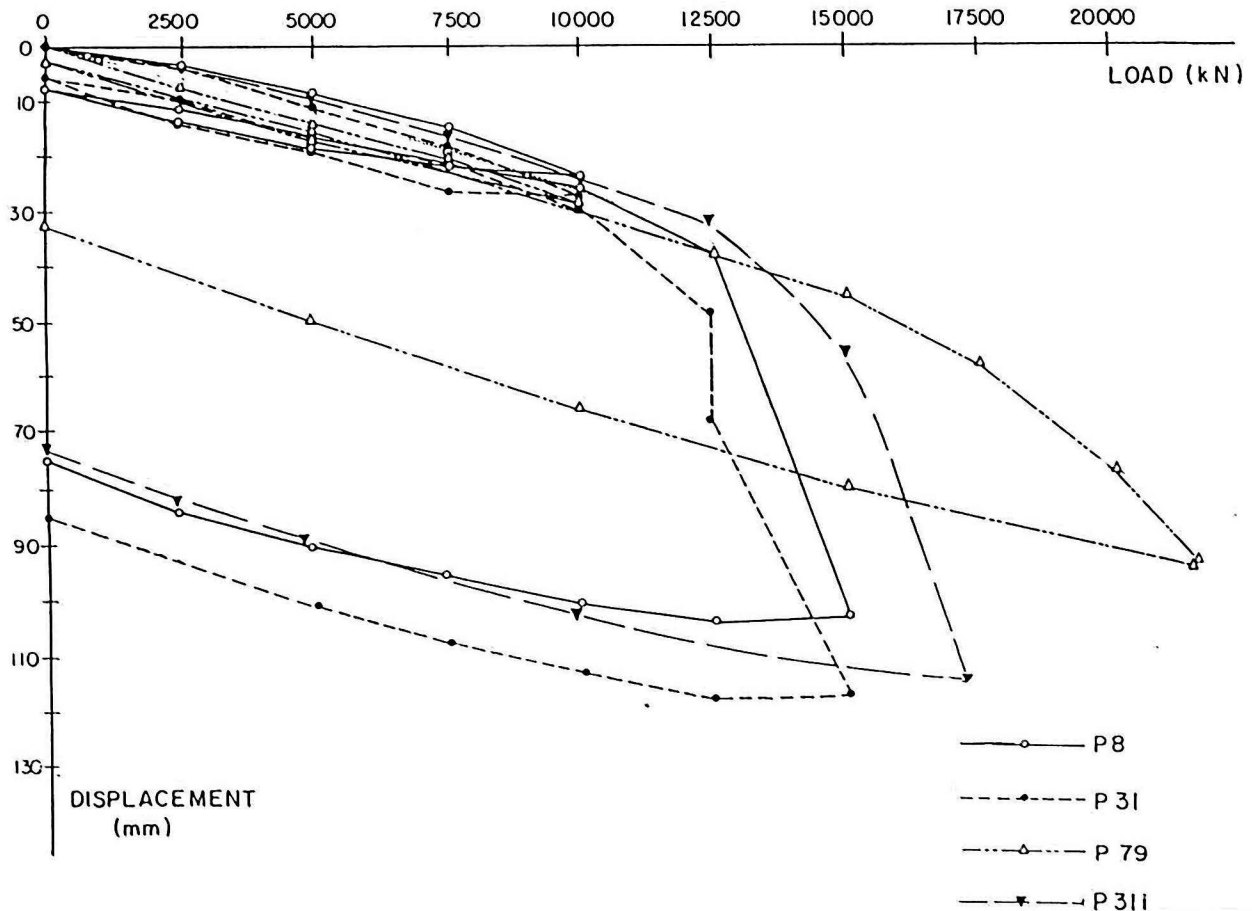


Fig. 26 Load settlement curves for vertical tests (adopted from Agisco, 1995)

Horizontal pile load tests

Horizontal load tests were performed on 2 piles located at main bridge (P8) and south pylon.

A general view for horizontal pile load tests is shown in Fig. 27.

The construction of the piles has followed the same procedure already described.

For the horizontal load tests the following equipments were installed:

- horizontal displacement
- load cell
- strain gauges to measure strains along the shaft
- inclinometer tubes to measure horizontal displacements
- temperature device.



Fig. 27 General view for horizontal pile load tests

The loading program consisted of: 10 load increments from 50 kN to 500 kN.

For the south pylon, after 10 hours, a second series of load increments were applied, from 500 kN to 1 000 kN, to evaluate the effect of ship impact.

The load displacement curve measured at 0.95 m below load level is shown in Fig. 28.

The computed values for pile displacements, bending moments and shear forces are shown in Fig. 29.

Dynamic pile tests

In order to have a better characterisation of the dynamic behaviour of the alluvial material for a bridge foundation a forced vibration test of a group of two piles was performed. A 3D finite element model was developed for the interpretation of the observed behaviour.

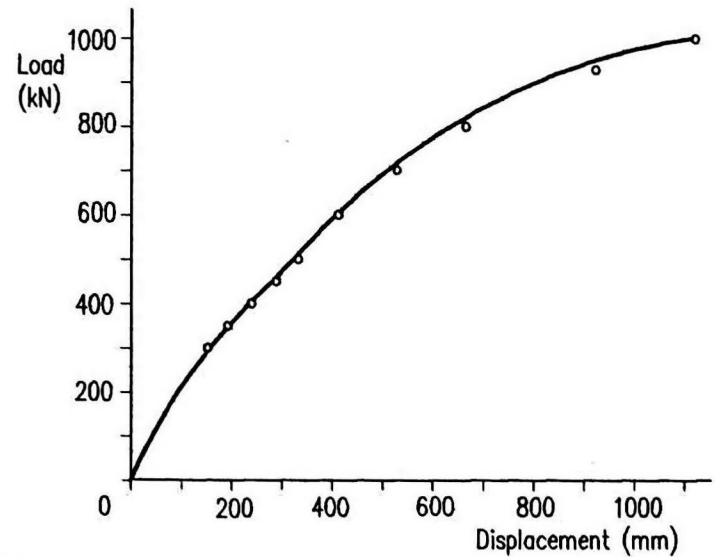


Fig. 28 Measured load displacement curve for horizontal tests (adopted from Terrasol, 1995)

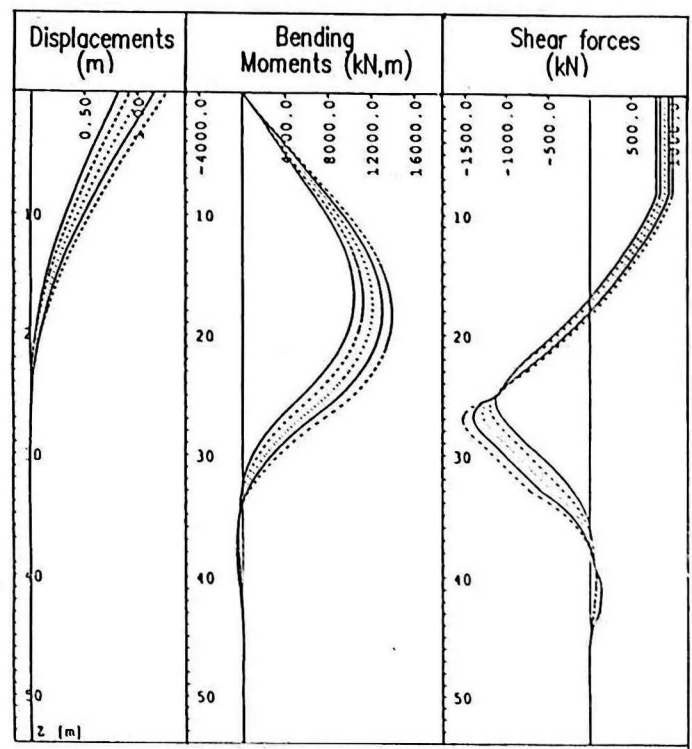


Fig. 29 Computed values for pile displacements, bending moments and shear forces (adopted from Terrasol, 1995)

The piles with 1.20 m of diameter and 60 m long were connected by a cap with 5.5 x 3.5 x 1.2 m.

The soil - pile system was discretized with 3 D finite elements of the second degree (cubic with 20 nodal points). The numerical results are compared with the observed values, in terms of displacement transfer functions.

In the dynamic test a shaker built in LNEC (Fig. 30) was used to impose on the pile cap, harmonic horizontal loads, with different amplitudes and frequencies (LNEC 1995 b).

The excitation frequencies were applied in steps of 0.1 Hz in the range from 0.5 to 20 Hz approximately. The dynamic response of the structure, for the various frequencies of excitation, was measured by means of velocity transducers and accelerometers. These equipments were placed in several points in order to monitoring the horizontal and vertical displacements (Fig.31).

Time series of velocity were recorded on several points, during the test. The digital treatment of this time series was performed by a computer program developed at LNEC (Portugal, 1990).

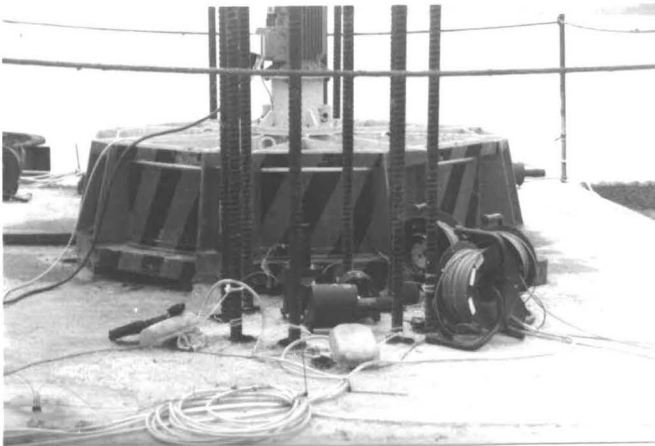


Fig. 30 General view of the shaker



Fig. 31 General view of the velocity transducers

Treated series are transported for frequency domain and the displacements were obtained by integration.

For the interpretation of the test results a 3 D model was used, to represent the soil, the two piles and the cap.

It was assumed that the piles were composed of a continuous, homogenous and isotropic material with a linear and elastic behaviour. The soil was considered as a continuous material, with elastic behaviour, and composed of various homogeneous layers.

The configuration of the two first modes of vibration and respective frequencies (observed and computed) is presented in Fig. 32. The first vibration mode corresponds to the bending of both piles following a direction perpendicular to the vertical plan that encloses both of them. The second mode corresponds to the bending of both piles in the vertical plan that contains them.

The modal damping values used in the mathematical model were the ones that were best adjusted to the transfer functions observed in the test. The adopted values are presented in Table 11.

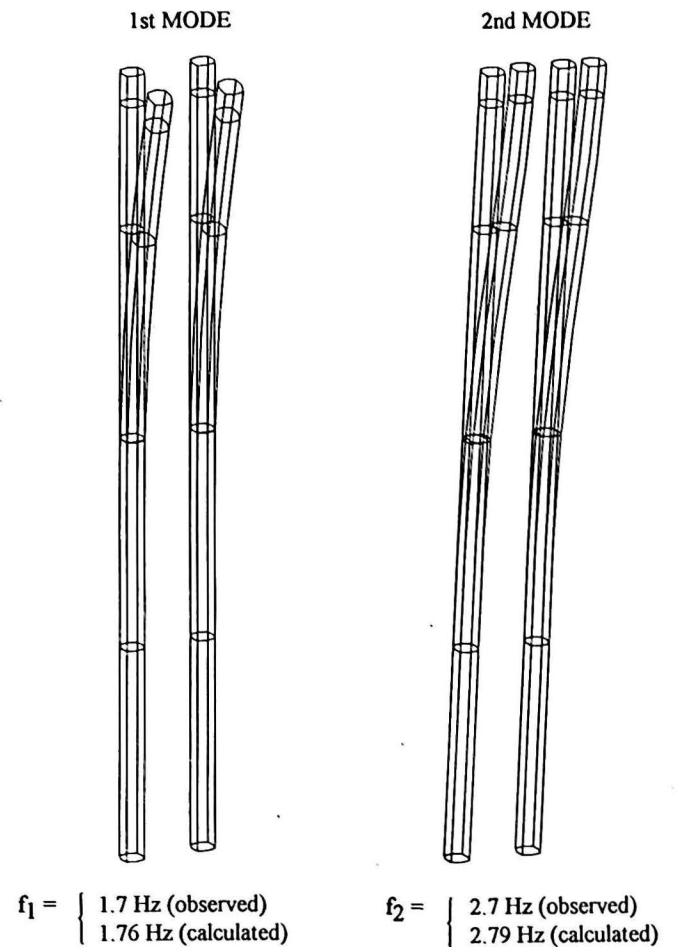


Fig. 32 Configuration of the two first vibration modes. Observed and computed frequencies (adopted from Oliveira et. al., 1996).

Table 11 Modal damping values adopted in the mathematical model

Vibration Modes	1	2	3
Damping modal in % of the critical damping	7	13	20

The observed and calculated frequencies by the mathematical model are presented in Table 12. There is a good agreement for the two first vibration modes.

Table 12 Frequencies of the first vibration modes

Vibration Modes	1	2	3	4
Observed Frequencies	1.7	2.7	-	-
Calculated Frequencies	1.76	2.79	8.78	11.70

The results observed in the test and those computed by the mathematical model in terms of displacement transfer functions of the force applied by the shaker are shown in Fig. 33.

The good obtained agreement shows that the mathematical model is well calibrated for simulation of the behaviour of the soil - piles system.

The variation of maximum displacements of piles with depth according to directions X and Y, as well as some displacement transfer functions computed at different depths is shown in Fig. 34.

Part 4

The art is long
 And life is short
 Experience is fallacious
 And decision is difficult
 (Hippocrates)

RECEPTION TESTS FOR PILES

The development and implementation of non destructive techniques of pile tests have experienced a great increment as the use of core sampling and load tests to control the final quality of the piles are very costly and can only be performed in a small number of piles(Tejoprojecto, 1993 c).

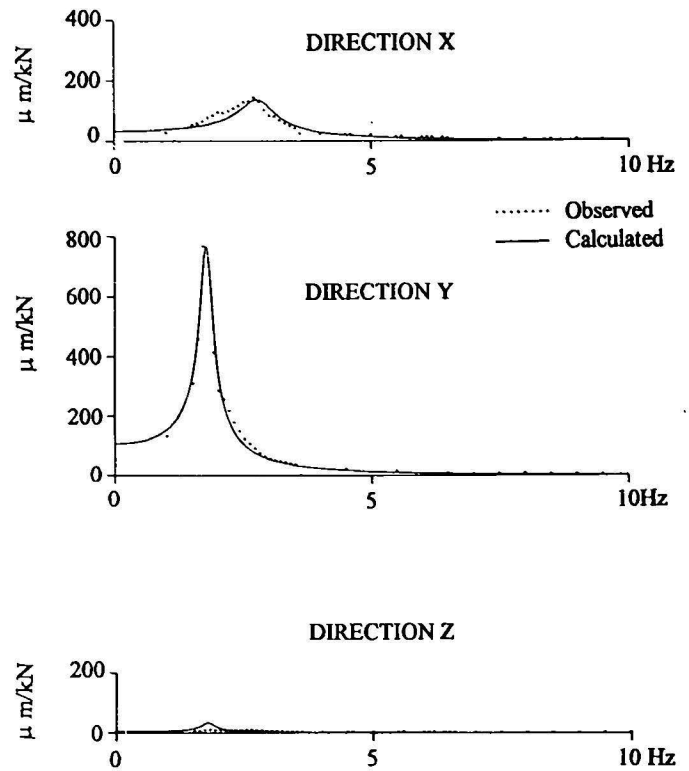


Fig. 33 Displacement transfer functions. Comparison between computed and observed values (adopted from Oliveira et al., 1996)

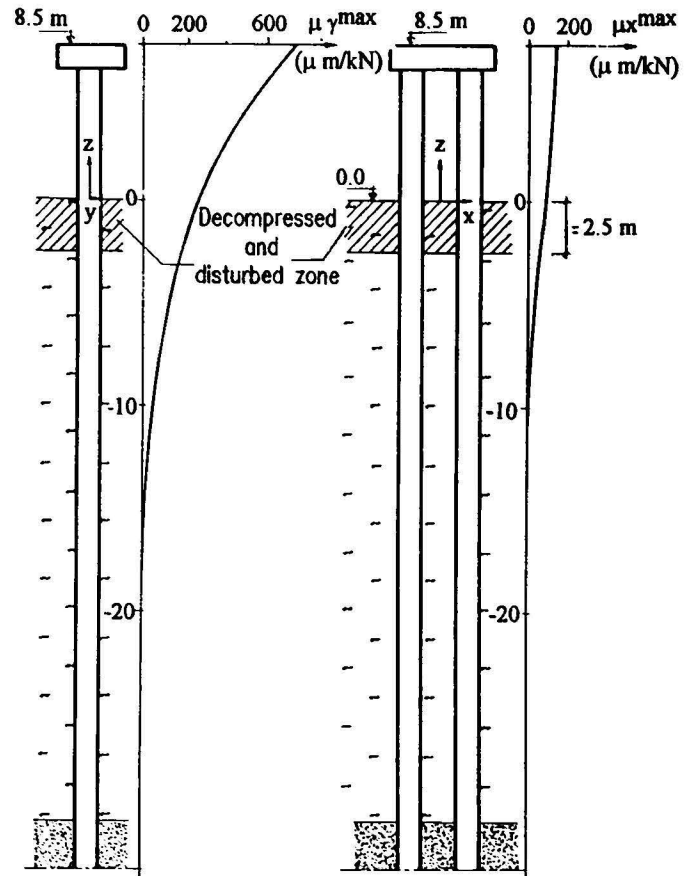


Fig. 34 Variation with depth of maximum displacement of piles. X and Y directions (adopted from Oliveira et al., 1996)

To assess the quality of piles the sonic tests were proposed. With this technique the principal structural singularities of the piles can be detected, the involved costs are small, the execution is fast and it is possible to perform a great number of tests. A blow on the pile head with a hammer and the record of the response by an accelerometer was performed. These tests were performed on piles with diameter of 1.0 m and length less than 25m (i.e. only for interchange viaducts).

Also sonic digraphy tests were performed and a continuous record through the length of the pile of the velocity of sonic waves between the source and the geophones introduced in two pipes attached to the pile reinforcement was done.

Taking into account the pile diameter (>1.2 m) four pipes with 60 mm of diameter were used (LNEC, 1996; Simecsol, 1996).

MONITORING DURING CONSTRUCTION AND LONG TERM

Introduction

The designer has the difficult task to perform a correct definition of loads and an adequate characterisation of the materials for the project. In spite of the great progress performed in these domains it is necessary to compare the mental model with the prototype response in order to assess the structural behaviour, and to decide in face of an anomalous behaviour.

With this background the following advantages of instrumentation of bridges are pointed out (Tejoprojecto, 1993d):

- i) Validation of design criteria and calibration of mental model.
- ii) Analysis of bridge behaviour during his life.
- iii) Corrective measures for the rehabilitation of the structure.
- iv) Cumulative experience that will be useful for the construction of more economic and safer bridges.

Quantities to be measured

For the superstructure the measurement of the following quantities were proposed: (Tejoprojecto, 1993 d).

- a) deck vertical displacements; b) piers cross-sections rotations; c) internal deck and piers deformations; d) internal deck deformations due to time-dependent effects; e) deck and stays temperatures; f) air temperature, relative humidity and

wind speed; g) seismic and wind induced accelerations in the deck and piers; h) forces in stays.

Related with the infrastructure the following measurements were programmed:

- a) pile head displacements using electronic teodolytes and appropriate reflectors; b) horizontal displacements along the piles shaft by inclinometers; c) strain distribution of the piles using extensometers; d) distribution of the accelerations, velocities and displacements inside the piles and selected points of the ground (an assessment of amplification effect can be done) recorded by 3D accelerographs.

After a ship impact or earthquake with a magnitude superior to 4, a detailed inspection is recommended.

Warning levels

Four warning levels were defined:

- (i) warning level 1 - no interruption of traffic; (ii) warning level 2 - limitation of traffic; (iii) warning level 3 - interruption of traffic; (iv) warning level 4 - decision concerning the traffic.

For warning levels 1 to 3 the maintenance team can deal with the problem alone. For warning level 4 a specialist is necessary to take the decision.

Inspections

To complement the data given by the sensors placed in different sections of the bridge regular inspections should be performed.

Four levels of inspection were proposed:

- (i) The reference situation corresponds to a detail inspection of all parts of the structure (foundations, bearings and decks) and the measurement of all the sensors in order to characterise the initial state of the bridge before the opening to traffic;
- (ii) The daily inspections aims an efficient visual checking of the superstructure (drainage systems, road surface, expansion joints, handrail, gantries, safety barriers, lighting etc.) to detect the need of small repairs;
- (iii) The annual inspections are related with the visual inspection of the foundations (measurements by sensors placed into the piles), supporting structures, bearings, expansion joints, superstructures and equipment.
- (iv) After the opening to traffic, the first detailed inspection will be done after two years. During the operation of the bridge the frequency is five years.

CONCLUSIONS

Extensive field investigation and laboratory tests campaigns were performed to characterize the alluvial deposits in order to study the foundations of the new Tagus bridge.

The geotechnical characteristics were obtained after a balance between the results of the field and laboratory tests.

Due the earthquake activity of the zone a very comprehensive seismic analysis was performed. Both deterministic and stochastic approaches were used. For the soil material upper and lower bound values were considered and linear and non linear analyses were performed.

The liquefaction potential evaluation was performed only by CPT and SPT tests due the disturbance that occurs during sampling of sandy materials. Both total and effective analyses were performed.

Static pile load tests both vertical and horizontal were carried out on trial piles to calibrate the design parameters and to optimise the pile lengths. Also dynamic pile tests were performed.

Non destructive techniques of pile tests were performed to assess the quality of piles.

The objectives of monitoring during construction and long term were presented.

ACNOWLEGMENTS

Due to the complexity of the New Tagus Bridge several consultant companies and experts were involved. The studies carried out by them are greatly acknowledged.

As already mentioned the field investigation was carried by Seacore with the collaboration of Teixeira Duarte, Fugro and LNEC, and the laboratory tests were performed by LNEC, CEBTP, FUGRO and Mecasol.

Special thanks are due to NOVAPONTE and particularly to Mr. Wastiaux and also to GATTEL and LUSOPONTE for the permission to publish this lecture.

The authors should like to thank the valuable cooperation of Mr. Virgilio Rebelo and Mr. Vicente Robrigues of COBA, leader of Tejoprojecto Consortium, for the geological and geotechnical studies.

It is important to refer the contributions of P & V and Geodynamique et Structures for the seismic studies. Also the works performed by AGISCO and TERRASOL for the static pile tests and LNEC for the dynamic pile tests should be referred.

The reception tests for piles were carried out by Simecsol and LNEC.

The installation of monitoring equipment was done by Simecsol and COBA.

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