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The Case of the New Tagus River Leziria Bridge

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Commemorate the Legacy of Ralph B. Peck
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Seventh International Conference on
Case Histories in Geotechnical Engineering
and Symposium in Honor of Clyde Baker

THE CASE OF THE NEW TAGUS RIVER LEZIRIA BRIDGE

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ABSTRACT

A brief description of the New Tagus River Leziria Bridge composed by 1695 m North Viaduct, by 970 m Main Bridge and by South Viaduct with a length of 9200 m is presented.
The observed thickness of the foundation alluvia material varies between 35m and 55m with a maximum value of 62m.
Hundred eighteen boreholes were performed with a depth between 21m and 71m and eight boreholes were performed from a maritime platform. Standard penetration tests (SPT) were carried out in all boreholes 1.5 m apart. In addition CPTu tests, seismic cone tests, crosshole and downhole tests were performed.
In three boreholes continuous undisturbed sampling with a triple sampler Geogor S was performed.
Related with static laboratory tests namely identification tests, triaxial tests, direct shear tests and oedometer tests were performed. In addition for the dynamic characterization reasonant columns tests and torsional cyclic tests were performed.
One of the most important considerations for the designers is the risk of earthquakes since Lisbon was wiped out by an 8.5 Ritcher magnitude earthquake in 1755. The seismic studies related to the design spectra were performed.
The liquefaction potential evaluation was performed only by field tests taking into account the disturbance that occurs during sampling of sandy materials. In this analysis attention was drawn for SPT and CPT tests as seismic tests have only been used when soil contains gravel particles. The shear stress values were computed from a total stresses model, that gave results on the conservative side using the code “SHAKE 2000”.
For the North and South Viaducts 1.5 m diameter piles were used and for the Main Bridge 2.2 m diameter piles were used.
For the construction of the piles metallic casings were driven by a vibrofonceur or a hydraulic hammer and the piles length varies between 20 m to 56 m.
Static pile load tests (both vertical and horizontal tests) were carried out on trial piles.
In addition pile dynamic tests were performed.
The construction aspects related with piles and bridge construction are addressed.
To assess the integrity of the piles reception tests by sonic diagraphies (crosshole tests) were performed.
Some problems that have occurred during piles construction in the Main Bridge, due to the gravel and cobbles dimensions, are described.
The bridge was monitored with the purposes of: (i) Validation of design criteria and calibration of mental model; (ii) Analysis of bridge behavior during his life; and (iii) Corrective measures for the rehabilitation of the structure.

INTRODUCTION

This paper is divided into four parts. In the first part a brief description of the New Tagus River Leziria 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.

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

The fourth part presents the construction issues, reception tests for piles, the characterization of gravel and cobbles materials, pile deteriorations and the objectives of monitoring during the construction phase and the long term.

Some final considerations are presented.

Part 1

“If wishes would prevail with me
my purpose should not fail with me”
Shakespeare, King Henry V.

BRIEF DESCRIPTION OF THE BRIDGE

The Project related with the Conception, Design, and Construction of Tejo Crossing in Carregado “(Sublanço A1/Benavente da A10 Auto-Estrada Bucelas/Carregado/IC3)” was awarded by BRISA to a Construction Consortium composed by the following companies: Moniz da Maia, Serra & Fortunato-Empreiteiros, S.A., Bento Pedroso Construções, S.A. Construtora do Tâmega, S.A., Lena Engineering and Construction, S.A., Novopca-Construction Associates, S.A and Zagope –Constructions and Engineering, S.A.

This Consortium has awarded the Conception and Design to a Group composed by the companies COBA, PC&A, CIVILSER and ARCADIS.

The crossing (Fig. 1) that integrates the North Viaduct, the Main Bridge and the South Viaduct is subsequently described (GRID, 2003).

The Basic Design of this 11.9 km long crossing of the Tagus river, located 25 km upstream of the Vasco da Gama Bridge was carried out in 2004. The schedule for the design and construction was 21 months.

The river, 1 km wide, runs in an alluvial plain corresponding to the Tagus valley, filled with soft sediments.
The 1695 m North Viaduct has 33 m spans. The deck is a concrete 2.0 m depth beam directed connected to 1.5 m diameter piers. There is a 62 m span to cross the railway (Fig.2).
The deck is 23 m above the water level (Design Group, 2004a).

The cross-section of the Main Bridge is composed by (Design Group, 2004b; 2005d, Portugal et al., 2005):
- a 0.30 m width reserve
- interior hard shoulder
-3 traffic lanes, each with 3.50 m with a total width of 10.50 m
- 2.525 m exterior hard-shoulder.
The platform includes a kerb on which rests a safety barrier, a maintenance footwalk and a edge beam with a total width of 1.15 m.
The total width of the platform is 29.95 m.

The deck is made of a pre- stressed cast in place concrete box-section 970 m long (Fig. 3). The individual spans are: 95 +6x130+95m. Piers P1 to P5 are monolitical with the deck and composed by two blades of reinforced concrete with 1.20m thick spaced 5.0m between axes. Piers P6 to P7 are similar with the blades spaced 7.40 m.
The thickness of alluvia materials is between 35 m and 55 m, with a maximum value of 62 m (Oliveira et al 2008).

The foundations are composed by 2.20 m diameter piles. The Piers P3 to P7 and the Piers P1 and P2 are supported by 8 piles and 10 piles, respectively. The piles were built by metallic casings 17 mm thick driven to the Miocene formations between 1m and 5.5 m depending of the gravel materials thickness.

The sacrificial thickness of the casings varies between 7.2 mm and 5 mm to face corrosion.
The pile caps with 11.0x22.0 m and 8 m thick to support piers P1C and P2C, were designed to resist ship impact. Pile cap with 11.0x16.0 m and 5.05 m thick supports piers P3C to P7C.

The South Viaduct integrates a set of 22 continuous viaducts with a total length of 9230 m with a concrete deck longitudinal prestressed with current spans of 36 m and 1.5 m of diameter piles.

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

Part 2

“Errors like straw, upon the surface
blow.
He who search for pearls must dive
below”.
John Dryden

MAIN GEOLOGICAL CONDITIONS

Regional geology

The new Tagus River crossing is located in the Cenozoic basin of the Tagus river and is composed by sedimentary materials of Miocene and Paleocene ages.
A simplified geological profile is presented in Fig. 4 (Design Group, 2004e).

Geomorphology

The morphology is flat located at levels of 4 to 5 m, and crossed by secondary water streams, protection dykes and water channels.

Geological structure

The tertiary formations, at regional scale, exhibit horizontal stratification with weak deformation.

Litostratigraphy

The site is composed by recent superficial deposits, namely Holocene alluvial and quaternary fluvial terraces above the bedrock composed by Miocene clay-grey materials.

The visual aspects of materials are shown in Fig. 5.

Hydrogeological conditions

The superficial layers with characteristics of free aquifer exhibit phreatic water level near the surface. The alluvial formations show characteristics for the occurrence of suspended, closed or half closed aquifers.

The Miocene formations exhibit favorable conditions for the occurrence of closed aquifers or semi closed aquifers with artesianism.

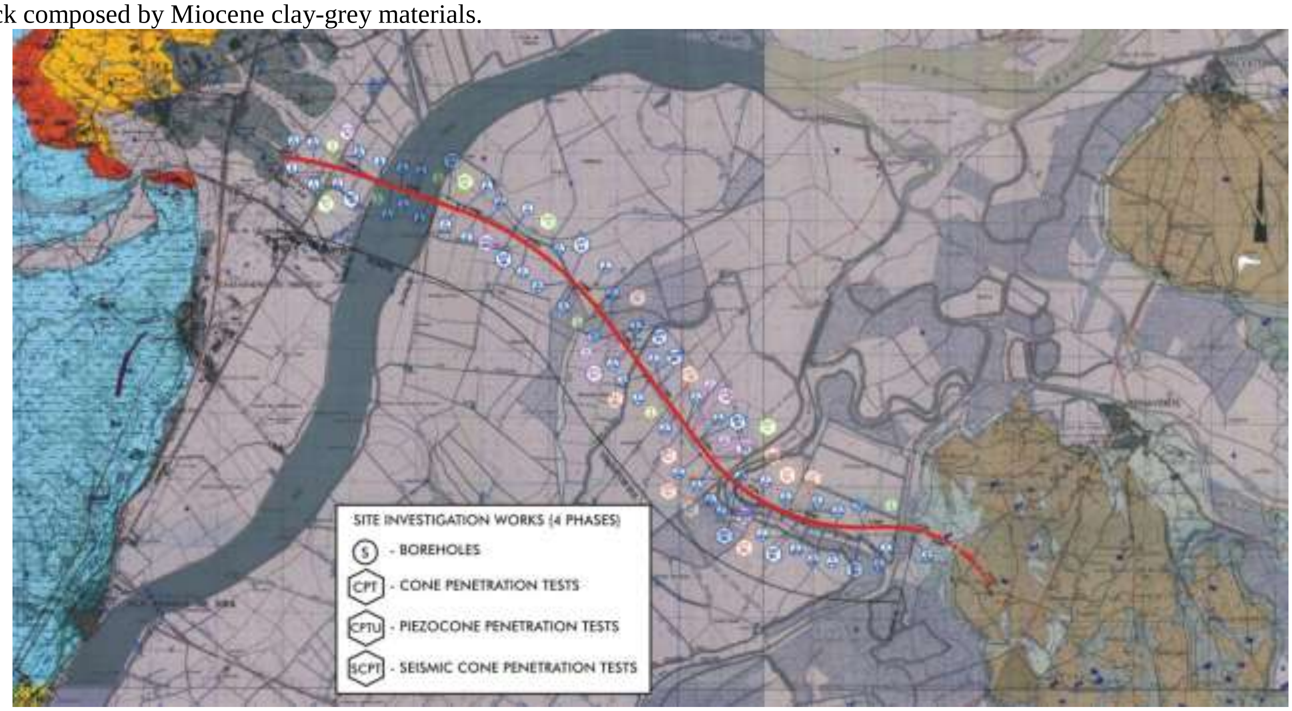

Fig 1. Leziria Tagus River Crossing site

Fig. 2. North Viaduct (courtesy of Charles Lavigne)

Fig. 3. Main Bridge (courtesy of Charles Lavigne)

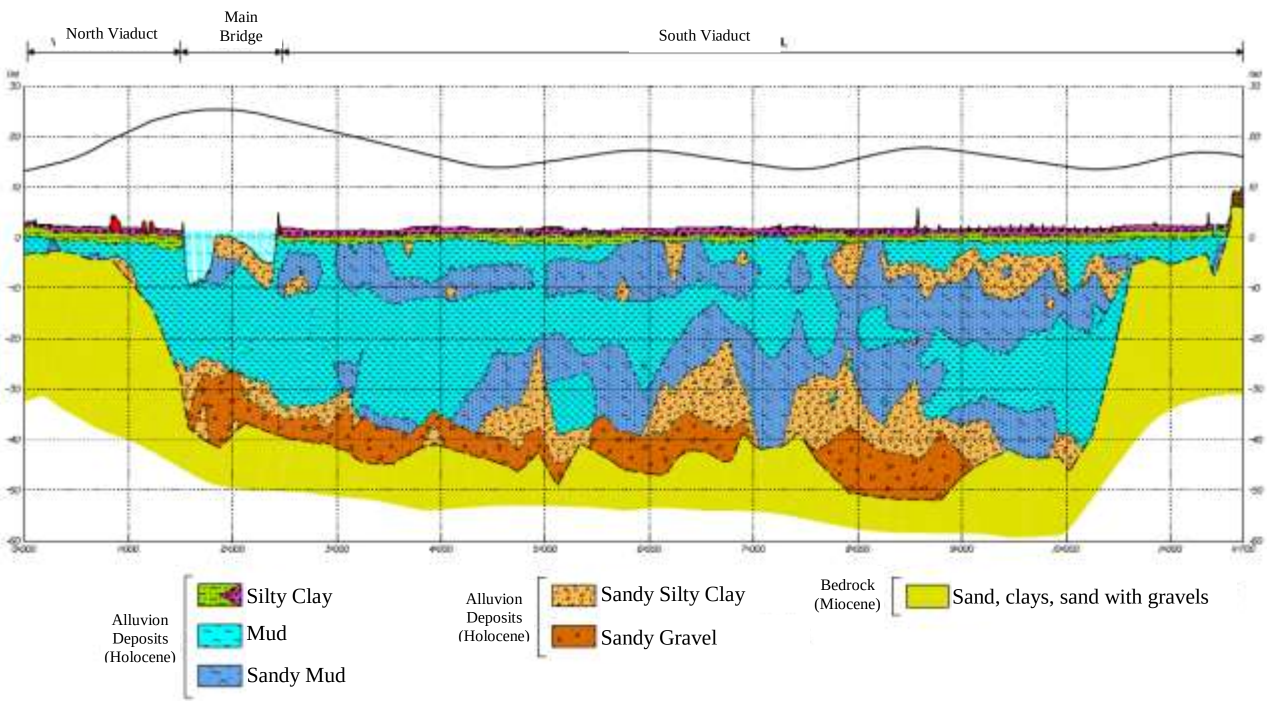

Fig. 4. Simplified geological profile

Fig 5. Visual aspect of the materials

FIELD INVESTIGATION

The field investigations have included 58 boreholes, namely 6 boreholes during the 1st stage of the Preliminary Studies, 49 boreholes in the 2nd stage and 3 boreholes during the complementary investigation program for the Basic Design. The boreholes were performed by Geocontrole (2004a).
In all boreholes the disturbed samples collected by Terzaghi sampler were classified, the water level was recorded and SPT tests, 1.5m apart, were performed.

In addition 32 undisturbed samples were collected using Shelbi and Proctor-Moran samplers.

Thirty two cone penetration tests, namely 4 CPT tests during the 1st stage of Preliminary Studies, 20 CPT tests during the 2nd stage, 6 CPTu tests using electrical cone friction sleeve and porous ceramic filter stone located at the conical tip, and 2 seismic cones were performed (Geocontrole, 2004a).

Nineteen vane shear tests, namely 3 tests during the first stage of the Preliminary Studies, 16 tests during the second stage (Geocontrole, 2004a).
9 seismic crosshole tests were performed, namely 7 tests by GEOCISA (2003) and 2 tests by LNEC (2003) during the 2nd phase of Preliminary Study. In addition 7 downhole tests were performed.

During the Final Design the complementary geotechnical project has integrated (Geocontrole, 2004 b, 2004c):

i) 41 boreholes with SPT tests 1.5 m apart (Fig. 6);
ii) 10 vane shear tests;
iii) 25 undisturbed samples taken with Geabor S sampler (Fig. 7);
iv) 16 CPTU tests (Fig. 8 and Fig. 9)
v) 5 seismic crosshole tests.

A summary of field tests is presented in Table 1.
The crosshole tests have given the following results:
Shear wave velocities V_s from 53 to 350 m/s
Longitudinal wave velocities V_p from 665 to 1526 m/s.

The variation of V_s with depth is shown in Fig. 10.
SPT results were between 0 and 4 blows, with a large frequency of 0 values and the higher values related with silty materials.
Vane shear tests have given for undrained strength the following results:
peak values - 12.5 to 51 kPa
residual values - 4 to 26.3 kPa.
The variation of these values is shown in Fig. 11.
PCPT tests, with measurement of pore pressures, have given point resistances between 0.15 and 1.2 MPa, with an increase with depth. This trend is illustrated in Fig. 12.
Pore pressures values have allowed the identification of material, higher values were related with mud materials.

Fig. 6. Borehole equipment

Fig. 7. Geobor S sampler

Fig. 8 CPTu equipment

Fig 9. CPTu tip

LABORATORY TESTS

During the Basic Design 12 identification tests (sieve analyses and Atterberg limits) were performed by COBA.

During the 2nd stage of Preliminary Studies forty three 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.

A summary of laboratory tests is presented in Table 2 (Geocontrole, 2004c).

In three water samples PH tests, determinations of alkalis, sulphates content, magnesium content and ammonia content were performed.

Twenty two 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.

Six triaxial tests for the definition of the strength in terms of cohesion (c) and friction angle (ϕ) were done.

The curves (σ_1 - σ_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.

Nineteen 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.

Also twenty five particle density tests were performed.

Three cyclic torsional simple shear tests were done (IST, 2005).

The curves G (shear modulus) versus γ (shear strain), $\sqrt{G}$ versus γ, ξ (damping ratio) versus γ and γ versus $\tau/\sigma o$ were obtained.

A view of cyclic torsional simple shear apparatus is presented in Fig. 13.

The results of cyclic torsional tests are shown in Fig. 14 (IST, 2004b, 2005).

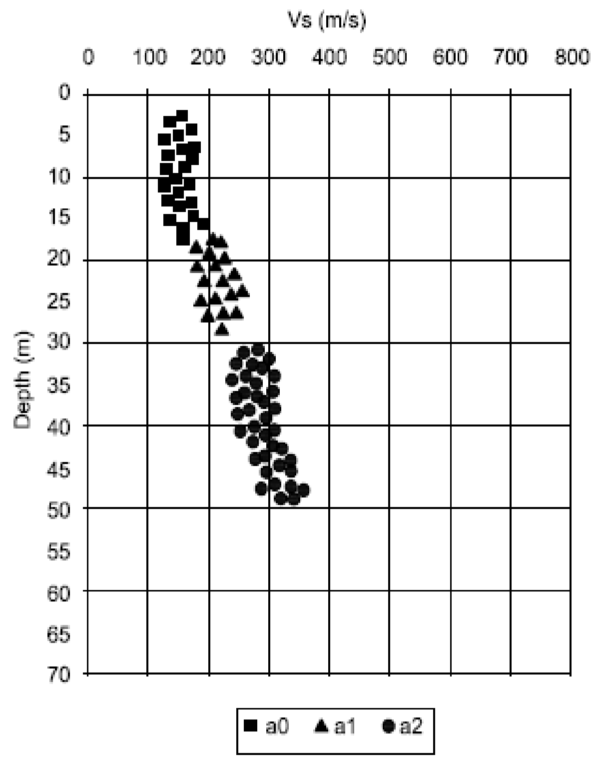

Fig. 10. Variation of V_s with depth

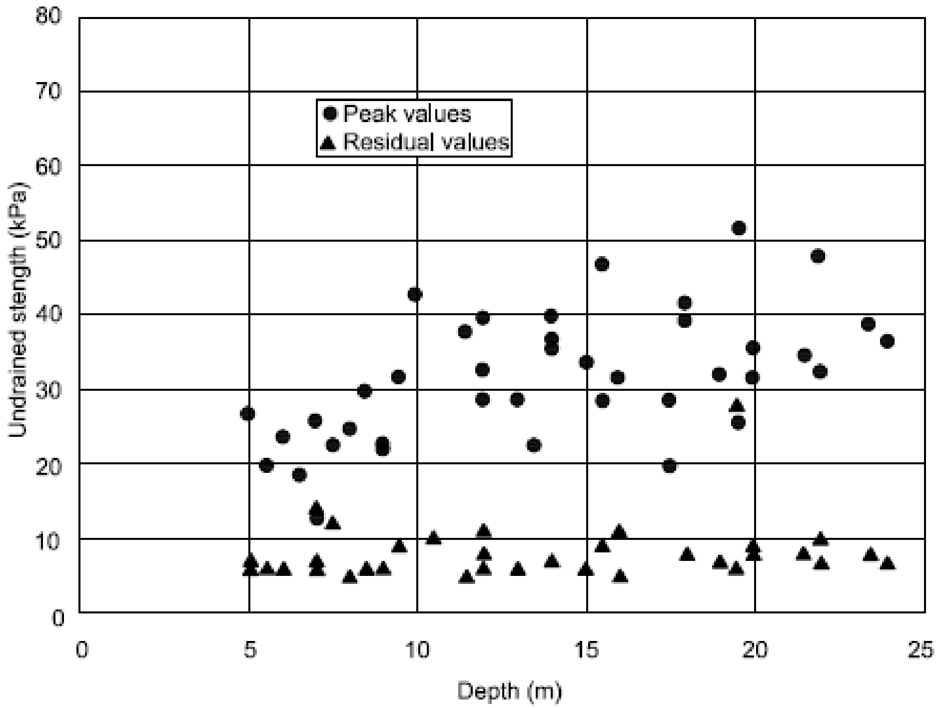

Fig 11. Variation of undrained strengths with depth

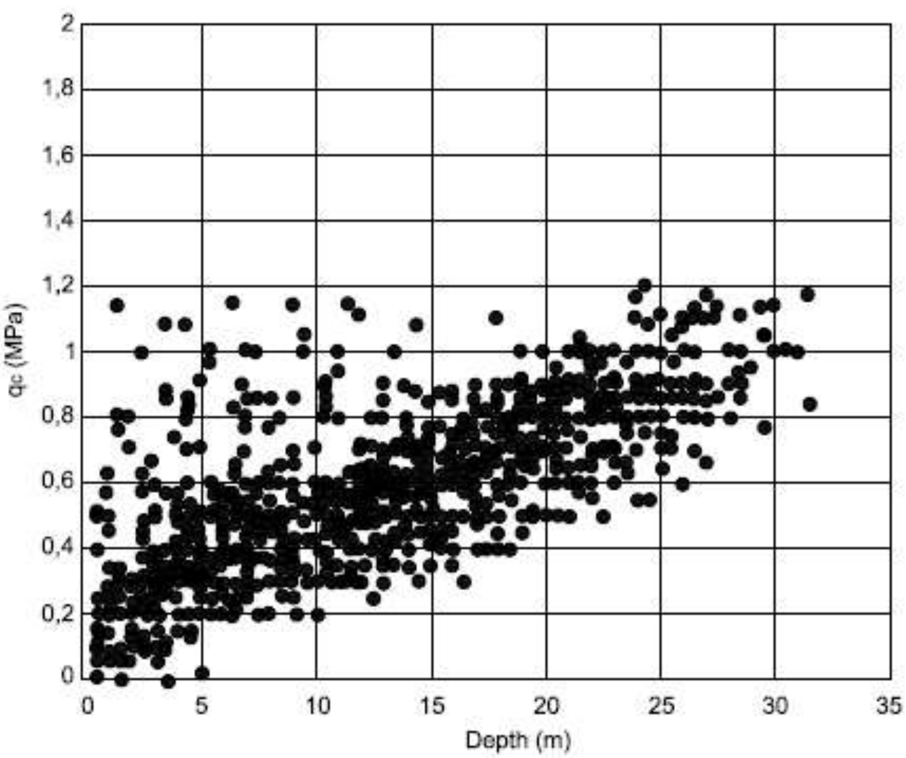

Fig. 12. Variation of q_c values with depth

Fig. 13. View of cyclic torsional shear apparatus (after IST, 2005)

Table 1 Distribution of field tests

TESTS	Basic Design	Final Design	TOTAL
BOREHOLES	58	60	118
BOREHOLES UNDISTURBED SAMPLING	0	3	3
VANE SHEAR TESTS	19	7	26
CROSSHOLE	9	6	15
CPTu/CPT	28	23	51
SEISMIC CONE	2	4	6

Table 2. Distribution of laboratory tests

TESTS	Basic Design	Final Design	TOTAL
IDENTIFICATION	55	180	235
SIEVE CURVES	55	180	235
OEDOMETRE	4	18	22
TRIAXIAL	0	6	6
DIRECT SHEAR	6	13	19
PERMEABILITY	6	18	24
CHEMICAL	3	9	12
RESONANT COLUMN	0	3	3
TORSIONAL SHEAR CYCLIC	0	3	3
PARTICLE DENSITY	3	22	25

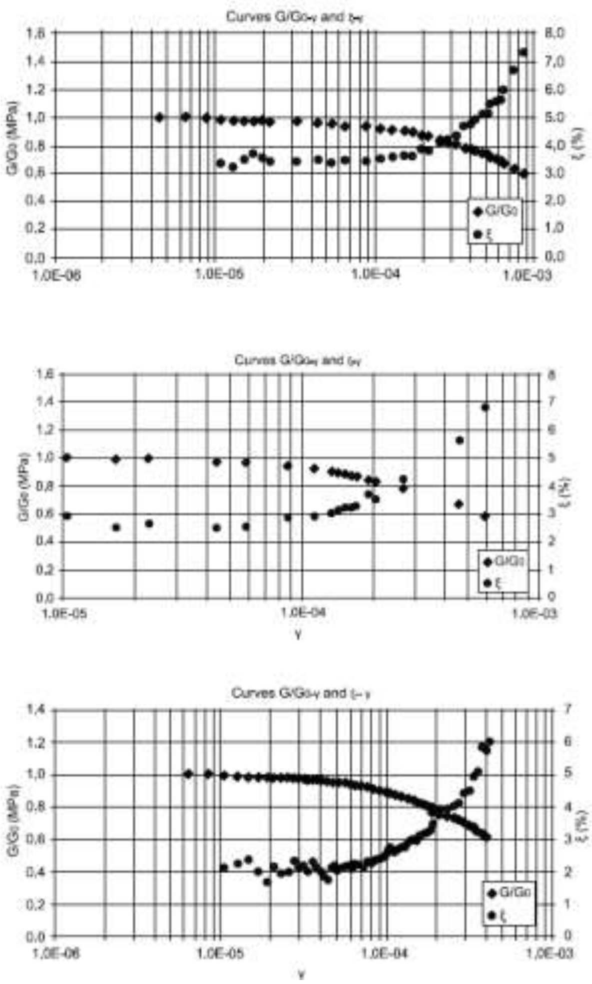

Fig. 14. Curves shear modulus and damping ratio versus shear strain (after IST, 2005)

GEOTECHNICAL CHARACTERISTICS

Based in the interpretation of site investigation programme and laboratory and in situ tests the following geotechnical units were identified (Design Group, 2004c; 2004d, Oliveira et al., 2008):

- Geotechnical unit a_{0a}
- Geotechnical unit a_0
- Geotechnical unit a1
- Geotechnical unit a2
- Geotechnical unit a3
- Geotechnical unit M

The description of each unit based in the geological and geotechnical characteristics will be presented.

Geotechnical Units

Unit a_{0a}

Composed by grey silty clay
Thickness from 2 to 3 m
Unified classification CH

AASHTO classification A-7-6
% passing sieve # 200 (ASTM) 95 to 99%
Liquid limit 64%
Plastic limit 38%
Natural water content 31.5 %
Density of particles = 1.86.

The crosshole tests have given the following results:
Shear wave velocities V_s from 130 to 160 m/s.
Longitudinal wave velocities V_p from 665 to 1526 m/s
Edin (MPa) values between 50 and 150.

Gdin (MPa) values between 20 and 100.

SPT results were between 2 and 6 blows.

PCPT tests, with measurement of pore pressures, have given point resistances between 1 and 2 MPa.

Vane shear tests have given for undrained strength the following results:
peak values - 22 to 26 kPa
residual values – 7 to 8 kPa.

Cohesion (total stress) c = 22 kPa
Friction angle $\phi = 30.^o$

Oedometre tests:
a_v (compressibility volumetric coefficient) = 0.172 to 0.6618.
Void ratio 1.234 to 2.025.

c_v (consolidation coefficient) = 2,1 to 28 x 10^{-8} m^2/s
k (permeability coefficient) = 0.34 to 1.8 x 10^{-10} m/s.

Taken into account the results of the tests and correlations from the literature the following mechanical characteristics were adopted (Design Group, 2005e, 2005f, 2005g):
Unit weight (kN/m3) - 18
Undrained cohesion (cu) (kPa) - 25 to 30
Ks values (kN/m^3)
Piles Φ = 1.5m from 4000 to 8000
Piles Φ = 2,0m from 3000 to 4000.

Unit a_0

Composed by mud material with intercalations of sandy material
Thickness = 20 m
Unified classification OH-OL
AASHTO classification A-7-6, A-7-5, A-4-(3), A-4-(6)

% passing sieve # 200 (ASTM) 94 to 100%
Liquid limit 29 % to 78 %

Plastic limit 27 % to 50 %

Natural water content 37.9 % to 87.2 %
Density of particles = 1.52 to 2.16
The crosshole tests have given the following results:

Shear wave velocities V_s from 120 to 170 m/s
Longitudinal wave velocities V_p from 665 to 1526 m/s
Edin (MPa) values between 50 and 150
Values of Gdin (MPa) between 20 and 100

SPT results were between 2 and 6 blows.

PCPT tests, with measurement of pore pressures, have given point resistances between 0 and 2 MPa.

Vane shear tests have given for undrained strength the following results:
peak values - 14 to 34 kPa
residual values – 6 to 13 kPa

Cohesion (in total stress) c = 3 to 22 kPa
Cohesion (in effective stresses kPa) c = 0 to 12
Friction angle (in total stresses) ϕ = 9 to 31 ^{o}
Friction angle (in effective stresses) ϕ = 13 to 20 ^{o}
a_v(compressibility volumetric coefficient)= 0.172 to 0.661
Void ratio 1.234 to 2.025
c_v (consolidation coefficient) = 2.3×10^{-8} to 3.8×10^{-7} m^2/s
k (permeability coefficient) = 1.6×10^{-10} to 1.4×10^{-9} m/s.

Taken into account the results of the tests and correlations from literature the following mechanical characteristics were adopted (Design Group, 2005e, 2005f, 2005g):
Unit weight (kN/m3) - 16
Undrained cohesion (cu) (kPa) - 25-30
Ks values (kN/m^3)
Piles Φ = 1,5m from 1000 to 2000
Piles Φ = 2,0m from 750 to 1500.

Unit a1

Composed by fine sandy materials with intercalations of silty clay material:
Thickness = 20 m
Unified classification SP, SP, SC
AASHTO classification A-3-6, A-6-2, A-6-6
% passing sieve # 200 (ASTM) 6 % to 42 %
Liquid limit NP to 40 %
Plastic limit NP to 18 %
Natural water content 22,5 % to 43,3 %
Density of particles = 1.52 to 2.16.
The crosshole tests have given the following results:
Shear wave velocities V_s from 130 to 240 m/s
Longitudinal wave velocities V_p from 665 to 1526 m/s
Edin (MPa) values between 100 and 300
Valores de Gdin (MPa) values between 30 and 100.

SPT results were between 2 and 20 blows.

PCPT tests, with measurement of pore pressures, have given point resistances between 2 and 8 MPa.

Cohesion (total stress) c = 0 kPa
Friction angle (in total stresses) ϕ = 42 ^{o}

Taken into account the results of the tests and correlations from literature the following mechanical characteristics were adopted (Design Group, 2005e, 2005f, 2005g):
Unit weight (kN/m3) - 18.5 to 19
Ks values (kN/m^3)
Piles Φ = 1.5m from 7000 to 30000
Piles Φ = 2.0m from 5000 to 22500.

Unit a2

Composed by fine sandy materials with intercalations of silty clay material with gravel material:
Thickness = 20 m
Unified classification SP, SM, SW
AASHO classification A1-b, A-3-(0), A-2-4.

% passing sieve # 200 (ASTM) 0 % to 23 %
Liquid limit NP
Plastic limit NP
Natural water content 22.3 %
Density of particles = 1.52 to 2.16
The crosshole tests have given the following results:
Shear wave velocities V_s from 140 to 300 m/s
Longitudinal wave velocities V_p from 665 to 1526 m/s
Edin (MPa) values between 100 and 500
Values of Gdin (MPa) values between 20 and 200.

SPT results were between 5 and 40 blows.

PCPT tests, with measurement of pore pressures, have given point resistances between 3 and 16 MPa.

Cohesion (total stress) c = 0 kPa
Friction angle(in total stresses) ϕ = 42 ^{o}

Taken into account the results of the tests and correlations from literature the following mechanical characteristics were adopted (Design Group, 2005e, 2005f, 2005g):
Unit weight (kN/m3) - 18.5 to 19.5
Ks values (kN/m^3)
Piles Φ = 1.5m from 8000 to 55000
Piles Φ = 2.0m from 6000 to 41000.

Unit a_3

Composed by medium sandy materials with intercalations of silty clay material with gravel material
The thickness of this layer is variable
Unified classification SP, SM,
AASHTO classification A-1-a.
% passing sieve # 200 (ASTM) 0 % to 6 %
Liquid limit NP
Plastic limit NP
Natural water content 22.3 %

Density of particles = 1.52 to 2.16
The crosshole tests have given the following results:
Shear wave velocities V_s from 320 to 400 m/s
Longitudinal wave velocities V_p from 665 to 1526 m/s
Edin (MPa) values between 500 and 1100
Values of Gdin (MPa) values between 200 and 400.

SPT results were between 40 and 60 blows.

CPT tests, with measurement of pore pressures, have given point resistances between 3 and 16 MPa.

Taken into account the results of the tests and correlations from literature the following mechanical characteristics were adopted (Design Group, 2005e, 2005f, 2005g):
Unit weight (kN/m3) - 20.5
Ks values (kN/m^3)
Piles Φ = 1.5m from 60000 to 90000
Piles Φ = 2.0m from 45000 to 68000.

Unit M

The bedrock Miocene is composed of 4 units namely: M_1, M_2, M_3 and M_4.
Unit M_1 is composed by clays and sandy silty materials
Unit M2 is composed by sands with intercalations of clay materials
Unit M3 is composed by sands with intercalations of gravel materials
Unit M4 is composed by sands with intercalations of gravel materials with sand silty
The crosshole tests have given the following results:
Shear wave velocities V_s from 400 to 500 m/s
Edin (MPa) values between 500 and 1700
Values of Gdin (MPa) values between 200 and 600
SPT results were higher than 60 blows.

Taken into account the results of the tests and correlations from literature the following mechanical characteristics were adopted (Design Group, 2004c):
Unit weight (kN/m3) - 215
Ks values (kN/m^3)
Piles Φ = 1.5m from 90000 to 120000
Piles Φ = 2.0m from 68000 to 90000.

A correlation between Vs and SPT values obtained by the tests with the proposal of some authors is shown in Fig. 15.

Part 3

"A first rate theory predicts,

a second rate theory forbids

and a third rate theory explain after the event".

A.I. Kitaigorowdswi, Russian Cientist, 1975.

DESIGN SURFACE SPECTRA

Introduction

To derive the design free field surface spectra a very comprehensive analysis was 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 five artificial time histories of acceleration were produced for seismic action type 1 and seismic action type 2 and for soil type A (IST, 2004a). For the computation of these accelerograms the validation criteria of EC8 (1998a) was considered (Fig. 16).
For the stochastic approach power spectral density functions based on RSA (1983) were used.

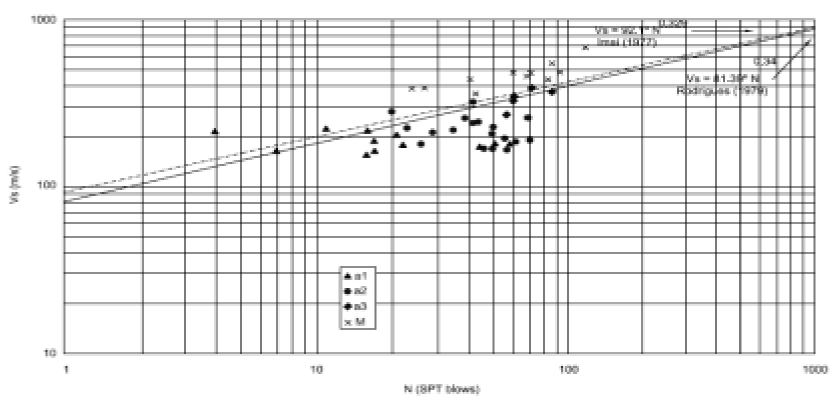

Fig. 15. A correlation between Vs and SPT values

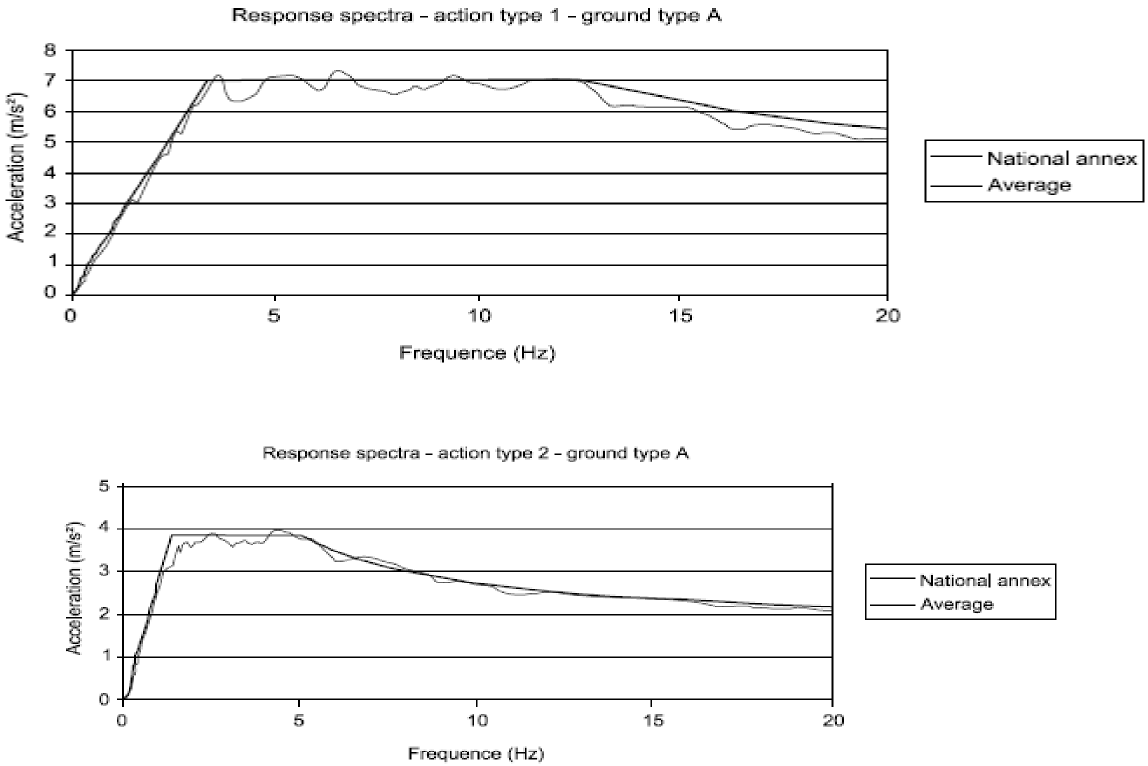

Fig. 16. Response spectra versus code spectra (after IST, 2004a)

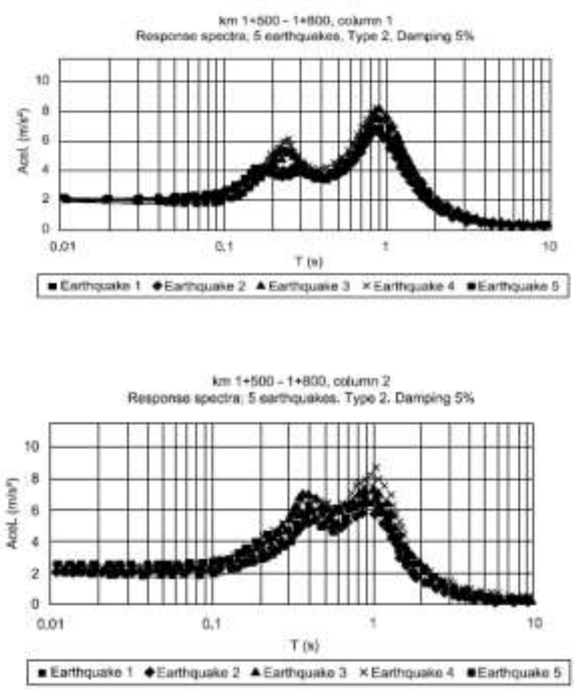

Fig.17. Response spectra acceleration km 1+500 – km 1+800 action type 1 and action type 2 (after IST, 2004a)

Due to the length of the bridge of 12 Km, 17 geotechnical profiles were analyzed to incorporate the variation of the geological and geotechnical characteristics.

Due to space limitations only the results obtained for the profile located between Km 1+500 and Km 1+800 where the main bridge is located are presented.

In Figs. 17 and 18 are presented the results of the response spectra (IST; 2004a), as well as the shear stress obtained by the code SHAKE 2000. The analyses were performed for seismic action type 1 and seismic action type 2 considering in the bedrock a ground type A.

LIQUEFACTION ASSESSMENT

Following 4.1.3. (2)-Part5-Eurocode 8(1998b) "An evaluation of the liquefaction susceptibility shall be made when the foundations soils include extended layers or thick lenses of loose sand, with or without silt/clay fines, beneath the water level, and when such level is close to the ground surface".

The seismic shear stress τ_e can be estimated from the simplified expression:

$$\tau_e = 0{,}65\ \alpha_{gr} \gamma_f\, S\, \sigma_{vo} \qquad (1)$$

where α_{gr} is the design ground acceleration ratio, γ_f is the importance factor, S is the soil parameter and σ_{vo} is the total

overburden pressure. This expression should not be applied for depths larger than 20 m. The shear level should be multiplied by a safety factor of [1.25].
The magnitude correction factors in EC8 follow the proposal of Ambraseys (1988) and are different from the NCEER (1997) factors. A comparison between the different proposals is shown in Table 3.

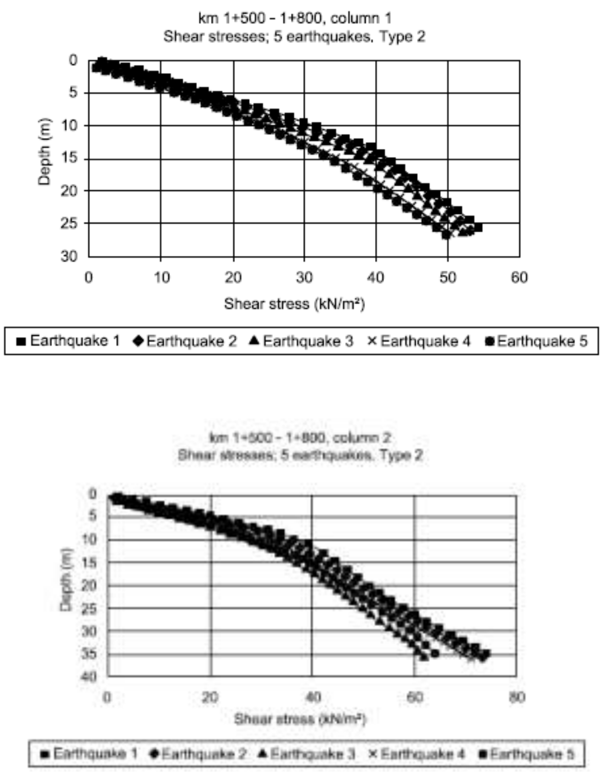

Fig. 18. Induced shear stress km 1+500 – km 1+800, action type 1 and action type 2 (after IST, 2004a)

Table 3. Magnitude scaling factors

Magnitude M	Seed & Idriss (1982)	NCEER (1997)	Ambraseys (1988)
5.5	1.43	2.20	2.86
6.0	1.32	1.76	2.20
6.5	1.19	1.44	1.69
7.0	1.08	1.19	1.30
7.5	1.00	1.00	1.00
.0	0.94	0.84	0.67
8.5	0.89	0.72	0.44

A new proposal with a summary of different authors presented by Seed et al. (2001) is shown in Figure 19.

A new proposal presented by Cetin et al. (2001) for liquefaction analysis is shown in Fig.20. It is considered advanced in relation with the previous ones, as integrates: (i) data of recent earthquakes; (ii) corrections due the existence of fines; (iii) experience related with a better interpretation of SPT test; (iv) local effects; (v) cases histories related more than 200 earthquakes; (vi) Baysiana theory.

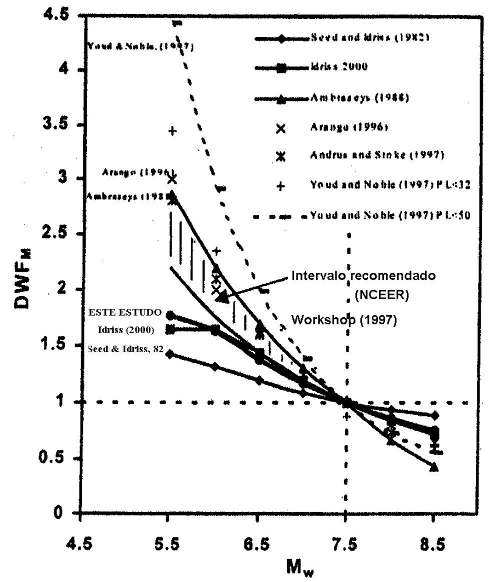

Fig. 19. Recommendations for correlations with magnitude (after Seed et. al., 2001)

For liquefaction evaluation of sandy materials two methods are used, namely, based in laboratory tests or field tests The following laboratory tests are used: (i) cyclic triaxial tests; (ii) cyclic simple shear tests; (iii) cyclic torsional shear tests. Due to the difficulties to obtain high quality undisturbed samples in general field tests are used: SPT tests, CPT tests, seismic cone tests, flat dilatometer tests and tests to assess electrical properties (Sêco e Pinto et. al, 1997).
For liquefaction assessment by shear wave velocities two methodologies are used: (i) methods combining the shear wave velocities by laboratory tests on undisturbed samples obtained by tube samplers or by frozen samples (Tokimatsu et al., 1991); (ii) methods measuring shear wave velocities and its correlation with liquefaction assessment by field observations (Stokoe et al., 1999).

EC8 uses corrective factors proposed by Ambraseys (1988), based in field tests that are different from the values proposed by Seed and Idriss (1982) and from the values proposed by NCEER (1997) based in laboratory tests. All the values are summarized in Table 3.

Due to the difficulties in performing CPT and SPT tests in soils with gravels some proposals to evaluate the susceptibility of liquefaction of these materials based in seismic tests with measurement of shear waves velocities Vs were proposed (Stokoe et al, 1999).

The post-liquefaction strength of silty materials is less than sandy materials, but superficial silty materials with moderate density are dilatant and with higher strength than clean sands (Youd and Gilstrap, 1999).

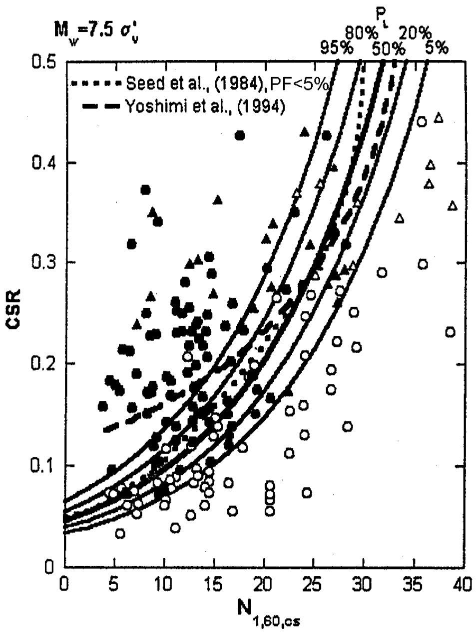

Fig. 20. Probabilistic approach for liquefaction analysis (after Cetin et al., 2001)

The authors have concluded that loose soils with IP<12 and wa/w_L> 0.85 are susceptible to liquefy and loose soils with 12< IP<20 and wa/w_L> 0.85 have higher strength to liquefaction and soils with IP>20 are not liquefiable.

It is important to refer that Eurocode 8 (1998b)-Part 5 considers no risk of liquefaction when the ground acceleration is less than 0.15g in addition with one of the following conditions: (i) sands with a clay content higher than 20 % and a plasticity index > 10; (ii) sands with silt content higher than 10% and $N_1(60)$>20; and (iii) clean sands with $N_1(60)$>25.

Post Liquefaction Strength

The topic related with the assessment of post liquefaction strength is not treated in EC8, but it seems that the following variables are important: fabric or type of compaction, direction of loading, void ratio and initial effective confining stress (Byrne and Beaty, 1999).
A relationship between SPT N value and residual strength was proposed by Seed and Harder (1990) from direct testing and field experience (Fig. 21).
Ishihara et al.(1990) have proposed a relation of normalized residual strength and SPT tests, based on laboratory tests compared with data from back-analysis of actual failure cases (Fig.22). Also Ishihara et al. (1990) by assembling records of earthquake caused failures in embankments, tailings dams, and river dykes have proposed the relation of Fig. 23, in terms of the normalized residual strength plotted versus CPT value.

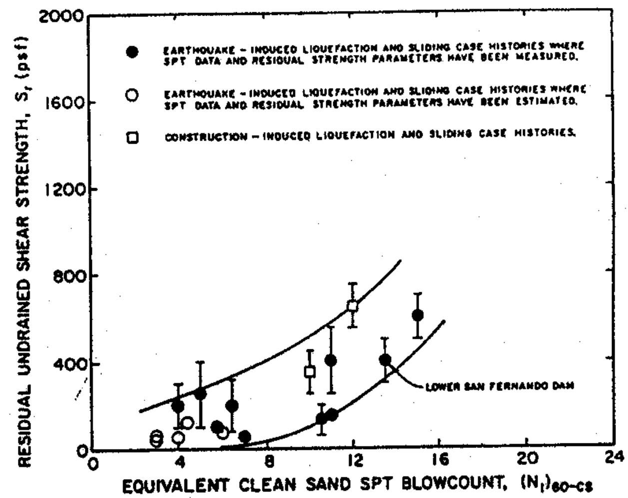

Fig. 21. Relationship between (N1) 60 and residual strength (after Seed and Harder, 1990)

Settlements Assessment

The susceptibility of foundations soils to densification and to excessive settlements is referred in EC8, but the assessment of expected liquefaction - induced deformation deserves more consideration.
By combination of cyclic shear stress ratio and normalized SPT N-values Tokimatsu and Seed (1987) have proposed relationships with shear strain (Figure 24).

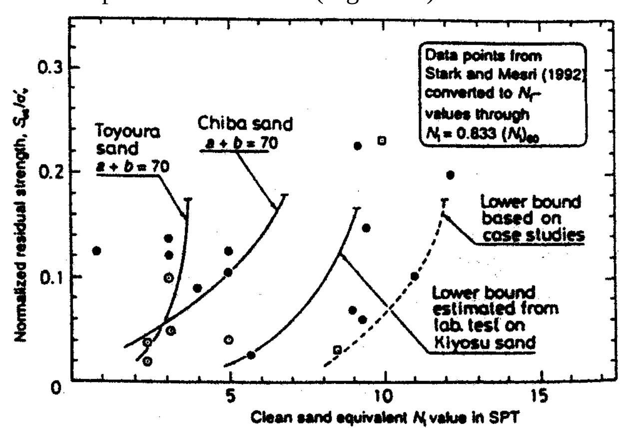

Fig. 22. Relation of normalized residual strength and SPT tests (after Ishihara et al., 1990)

To assess the settlement of the ground due to the liquefaction of sand deposits based on the knowledge of the safety factor against liquefaction and the relative density converted to the value of N1 a chart (Figure 25) was proposed by Ishihara (1993).

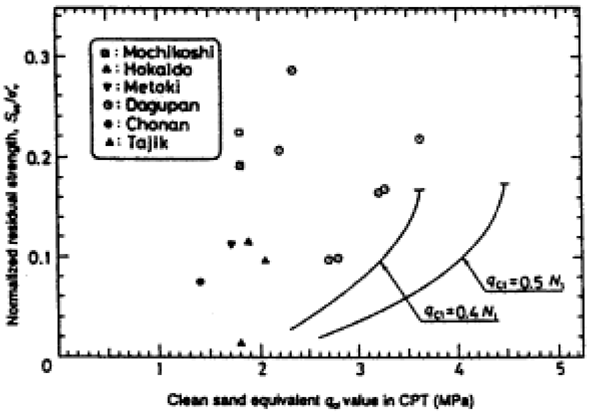

Fig. 23. Relation of normalized residual strength and CPT tests (after Ishihara et al., 1990)

<u>Remedial Measures</u>

Following EC8 ground improvement against liquefaction should compact the soil or use drainage to reduce the pore water pressure. The use of pile foundations should be considered with caution due to the large forces induced in the piles.

The remedial measures against liquefaction can be classified in two categories (TC4 ISSMGE, 2001; INA, 2001): (i) the prevention of liquefaction; and (ii) the reduction of damage to facilities due to liquefaction.

The measures to prevent of occurrence of liquefaction include the improvement of soil properties or improvement of conditions for stress, deformation and pore water pressure. In practice a combination of these two methods is adopted.

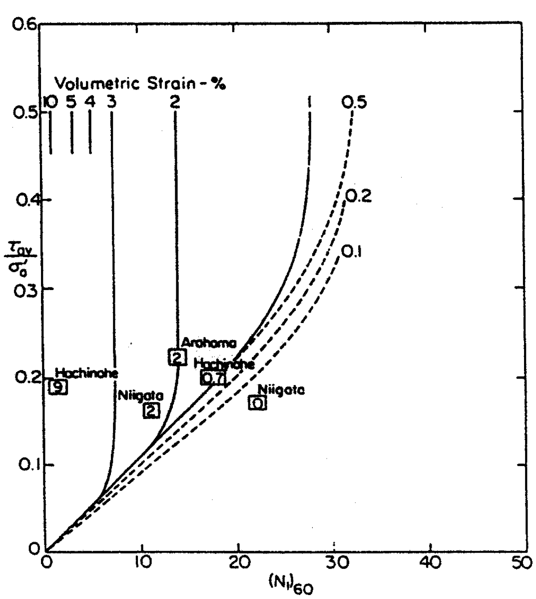

Fig. 24. Correlation between volumetric strain and SPT (after Tokimatsu and Seed, 1987)

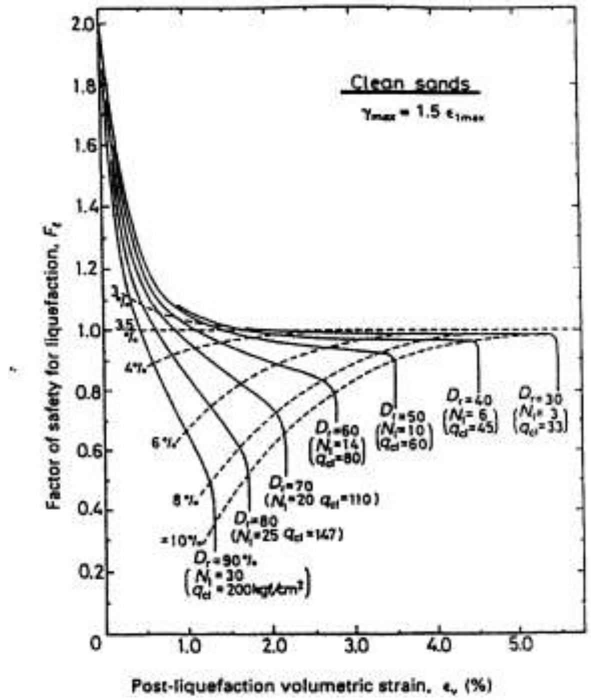

Fig. 25: Post cyclic liquefaction volumetric strain curves using CPT and SPT results (after Ishihara, 1993)

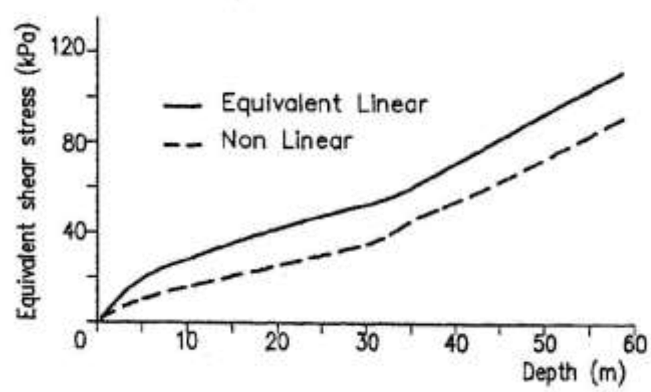

Fig. 26. Equivalent shear stresses computed from SHAKE and DYNAFLOW codes (after Seco e Pinto and Oliveira, 1998)

The measures to reduce liquefaction induced damage to facilities include (1) to maintain stability by reinforcing structure: reinforcement of pile foundation and reinforcement of soil deformation with sheet pile and underground wall; (2) to relieve external force by softening or modifying structure: adjusting of bulk unit weight, anchorage of buried structures, flattering embankments.

Liquefaction Evaluation

The liquefaction potential evaluation was performed only by field tests taking into account the disturbance that occurs during sampling of sandy materials (Jeremias et al, 2007).

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

The shear values were computed from a total stresses model, that gave results on the conservative side using the code "SHAKE 2000".
Just as an example Fig. 26 illustrates the differences between the total stress model and an analysis in effective stresses using the computer program DYNAFLOW for the Vasco da Gama bridge in Tagus river and with the same type of alluvia materials.

Corrections related with SPT test results due to the depth effect and the equipment were performed following the recommendations of EC8 (1998b).

The sieve curves of materials a_1 and a_2 are shown in Figs. 27 and 28.

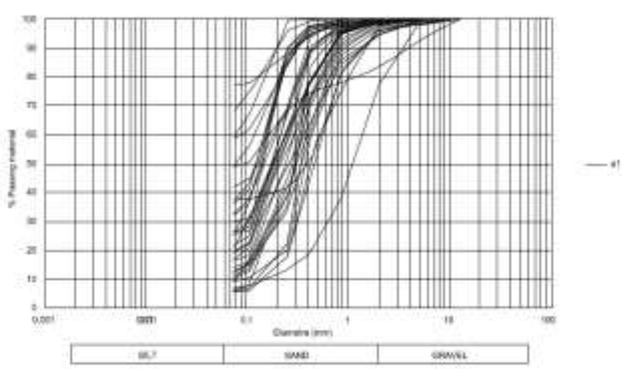

Fig. 27. Sieve curves for material a_1

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.

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) (for shallow soils due to disturbance

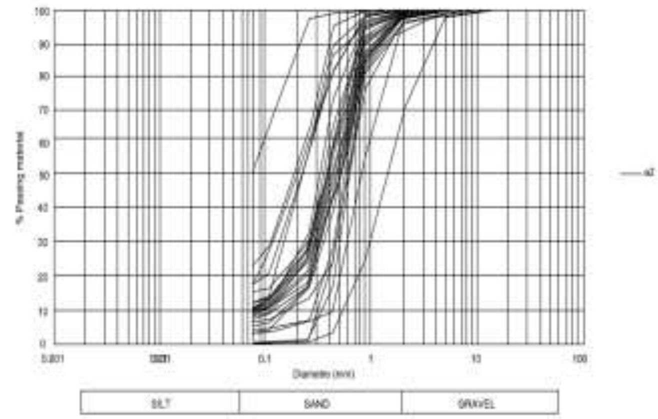

Fig. 28. Sieve curves for material a_2

effects reduced C_N values were considered) and $(q_c)_1$ (CPT); (v) column 11, τequiv. (equivalent shear stress value computed for action type 2 related with the highest magnitude 7.5); (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 behavior of the material observed in the CPT tests and the values of the pore pressures developed in the 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 8. 5 (1998b).

Table 4 presents an application of liquefaction evaluation for material a_1 and material a_2. The liquefaction potential evaluation, by SPT and CPT tests, is shown in Figs. 29 and 30.

Taking into account the Figs. 24 and 25 the estimated settlements of materials a_1 and a_2 are between 40 mm to 150mm.

PILE LOAD TESTS

Introduction

Following Eurocode 7(1997) pile design can be performed by (Design Group, 2005a, 2005b):
- prescriptives measures and comparable experience;
- design models;
- use of experimental models and load tests;
- observational method.

Table 4. Evaluation of liquefaction potential material a_1 and material a_2

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Pier	No of Borehole or CPT	Depth (m)	Thickness (m)	N_m	$(q_c)_m$ (MPa)	σ'_o (kPa)	C_N	N_1 (60)	$(q_c)_1$ (MPa)	$\tau_{equiv.}$ (kPa)	τ/σ'_o	τ/σ'_o x 1,1	τ/σ'_ox 1,25	Mat.	Remarks
	S1B	16.8-25.1	8.3	44	-	139.1	0.8	37	-	39	0.29		0.36	A2	N.L
	S2B-2	24.3-31.3	7.0	23	-	215.4	0.7	16	-	55	0.26		0.32	A2	.L
“	S3B-1	0.0-4.2	4.2	3	0.5	33.9	1.0	3	0.5	7.6	0.22		0.28	A2	.L
“	S3B-2	4.2-7.4	3.2	6	0.52	66.4	1.2	7	0.6	19.2	0.29		0.36	A1	L
“	S3B-3	7.4-9.6	2.2	12	0.65	89.4	1.1	13	0.71	26.3	0.29		0.37	A1	L
“	S3B-4	24.6-27.6	3.0	26	-	200.2	0.7	18	-	52.0	0.26		0.32	A2	L
“	S4B-1	0.0-3.6	3.6	4	0.5	31.2	1.0	4	0.5	6.6	0.21		0.26	A2	L
	S4B-2	3.6-6.2	2.6	3	0.52	58.5	1.0	3	0.52	16.5	0.28		0.35	A2	L
	S5B-1	0.0-4.5	4.5	3	0.5	20.3	1.0	3	0.5	8.3	0.41		0.51	A2	L
	S5B-2	26.0-28.8	2.8	31	-	191.1	0.7	22	-	55.1	0.29		0.36	A2	NL
	S6B-1	0-5.4	5.4	2	0.5	24.3	1.0	2	0.5	9.7	0.40		0.50	A2	L
	S6B-2	24.1-25.0	0.9	5	-	164.2	0.8	4	-	48.7	0.30		0.37	A2	L
	S6B-3	25.0-29.2	4.2	17	-	188.1	0.7	12	-	54.4	0.29		0.36	A2	L

N_m - SPT value
$(q_c)_m$ - CPT cone resistance value
σ'_0 - Effective overburden pressure
C_N - Correction factor for overburden pressure
N_1 (60) - Normalized SPT value
$(q_c)_1$ - Normalized CPT cone resistance
$\tau_{equiv.}$ - Equivalent cyclic shear stress
L - Liquefaction
N.L - No Liquefaction

The piles of Leziria bridge were designed by (Ferreira et al, 2008):
i) design models;
ii) pile load tests that have given information about the characteristics of gravel materials and techniques of driving the metallic casings;
iii) comparable experience.

Pile load tests were performed with the following purposes:
i) to determine the response of a representative pile and the surrounding ground to load, both in terms of settlements and limit load;
ii) to check the performance of individual piles and to allow judgment of the overall pile foundation;
iii) to assess the suitability of the construction method.
Load tests were carried out on trial piles which were built for test purposes before the final design.

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

The number of pile tests were selected taking into consideration the following aspects:

- the ground condition and the spatial variation;
- the geotechnical category of the structure;
- past experience related the use of same type of piles in same ground conditions;
- planning of the works.

The experimental piles for static and dynamic tests were located at Km 8+200 where the pile was embedded 1 diameter in the Miocene, at Km 7 + 900 where the pile was embedded 3 diameters in the gravel materials, and at Km 5 + 400 where the pile was embedded 3 diameters in the Miocene (Design Group, 2005a; 2005b, 2005c). Table 5 gives a summary o pile type and location.

In each place a 800 mm diameter pile was built for static test, two reaction piles with 1500 mm of diameter, 3.5 m apart from the pile test, and a fourth 800 mm diameter pile, 5.5 m apart from the first pile, for dynamic test.
To perform pile load tests 7 piles 1.5 m diameter and 7 piles 0.8 m diameter piles were built.

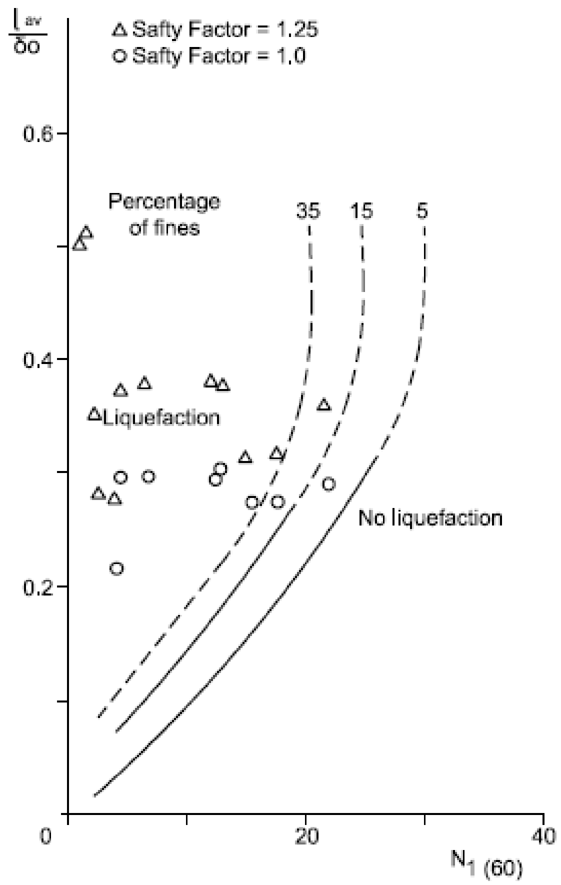

Fig. 29. Liquefaction potential evaluation by SPT tests

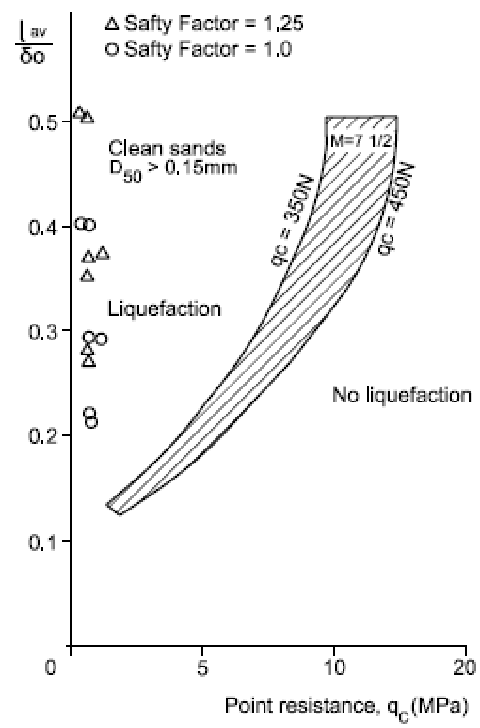

Fig. 30. Liquefaction potential evaluation from CPT tests

Table 5. Summary of pile type and location

Piles (Km)	Diameter (m)	Pile Embedding	Type LoadTest
5+400	0,8	3Ø (M)	• Vertical • Dynamic
7+900	0,8	3Ø (a3)	Vertical •Dynamic
8+200	0,8	1Ø (M)	Vertical •Dynamic
4+750	1,5	3Ø (M)	Horizontal •Dynamic

<u>Vertical pile load tests</u>

The methodology to perform static vertical pile load tests has followed "Axial Pile Loading Test, Suggested Method" recommended by ISSMGE and published in "ASTM D1143(1981).

The purpose was to incorporate the contribution of all the ground layers and their influence in the deformations until a depth of 5 diameters, unless the bedrock was situated at upper level.
Vertical load tests were performed on 3 piles.

For the vertical load test the following equipments were installed: 2 mechanical dial gauges, electrical displacement transducers (Fig. 31) with removable extensometers (Fig. 32) , with a resolution of 10^{-6}, and anchors, 1 temperature sensor, 1 tilmeter, 1 hydraulically operated pump, 2 hydraulic jacks and 1 optical level.

A general view for vertical pile load tests is presented in Fig. 33.

Fig. 31. Displacement transducers

For the vertical pile load tests a maximum load of 9100 kN was applied, i.e. 3.25 times the service load. The loads were applied in two cycles of load and unload, with a maximum load of service load for the first cycle and the loads were applied in 4 increments.

Fig. 32. Recovery extensometers

In the second cycle the loads were applied in 19 increments.
The number of load increments and the cycles of load and unload were defined with the purpose to reach some conclusions related to deformations, creep effects and ultimate load.

The load - settlement curves for 3 pile tests are shown in Fig. 34.

Failure loads were defined as settlement equal to 10% of the pile diameter, i.e. at 80 mm settlement.

Fig. 33. General view for vertical pile load tests (after Ferreira et al, 2008)

Horizontal pile load tests

The horizontal load tests were performed in two piles of 800 mm and 1500 mm of diameter located at km 5 +400. The maximum load was 600 kN to mobilize a displacement of 8cm and the loads were applied in steps of 75 kN (ICIST-IST, 2005).
For the horizontal load tests the following equipments were installed:
- clinometers
- vibrating wire transducers
- load cells
- retrieval extensometers
- inclinometer tubes to measure horizontal displacements
- temperature device.

The loading program consisted of: 10 load increments from 50 kN to 500 kN.
The load displacement curve measured is shown in Fig. 35.

The measured rotations values versus loads are shown in Fig. 36.

Fig. 37 shows a comparison between the bending moments values obtained by the tests and by the analyses for different values of k= 2500 kPa, 5000 kPa, and 10000 kPa.

Dynamic pile tests

Dynamic pile tests were performed in 9 piles with diameters of 800mm and 1500 mm.
. The piles were instrumented with:
- 4 pairs of acelerometers (Fig. 38).
- 4 transdutors
- topographic equipment

A dynamic test view is shown in Fig. 39.
During the tests the height of the hammer fall was increasing from 0.2 m to 3.0 m in steps of 0.2 m.
The point resistance (Rb) and the lateral resistance (Rs) for pile E 800-2 is shown in Fig. 40.

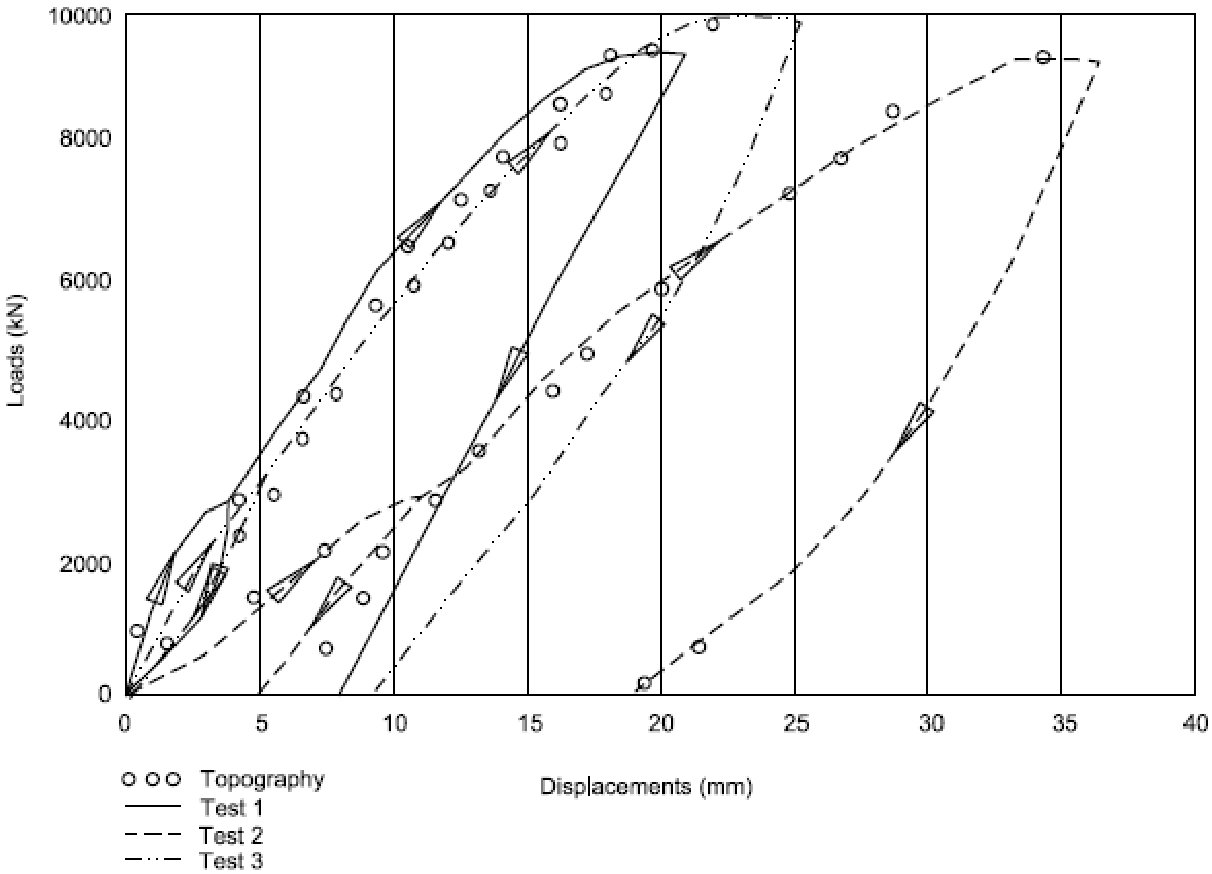

Fig. 34 Load settlement curves for vertical tests (after ICIST-IST, 2005)

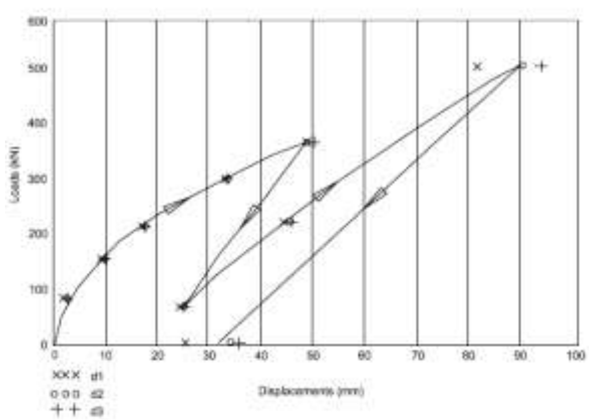

Fig. 35. Measured load displacement curve for horizontal tests (after ICIST-IST, 2005)

It is important to stress that the results of dynamic tests have confirmed the results of static tests pointing the higher contribution of the lateral resistance in comparison with the point resistance.

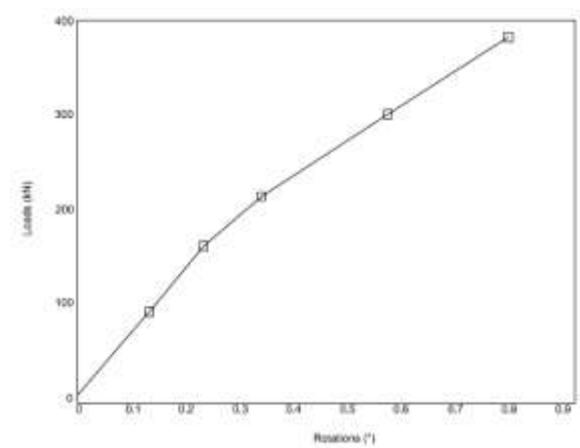

Fig. 36. Measured load rotations curve for horizontal tests (after ICIST-IST, 2005)

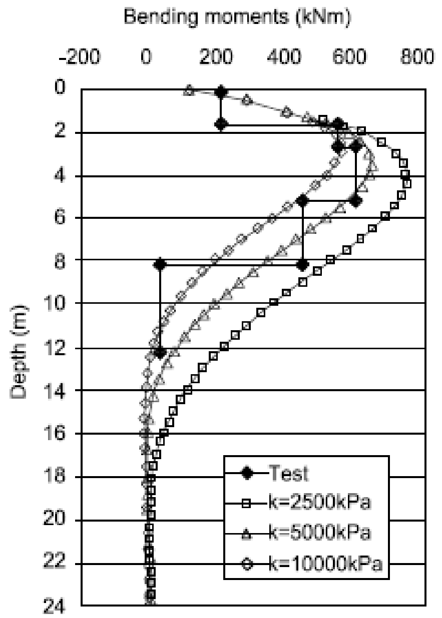

Fig. 37. Bending Moments (after ICIST-IST, 2005)

Fig. 38. Transducers and accelerometers

Fig. 39. Dynamic test (after Ferreira et al, 2008)

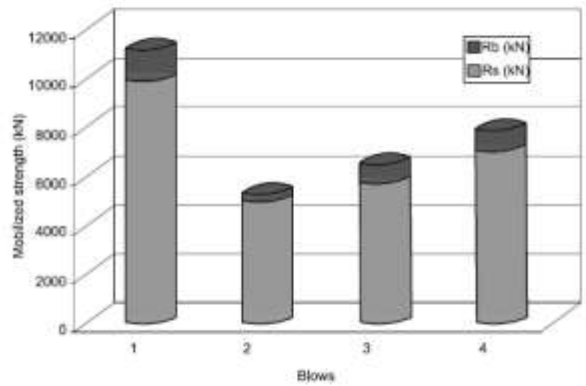

Fig. 40. Mobilized resistances (after ICIST-IST, 2005)

Part 4

“The important thing in science is not
so much to obtain new facts as to discover
new ways of thinking about them”.
(Sir W. Bragg, British Scientist, 1968)

CONSTRUCTION ASPECTS

The most important construction aspects are listed below:

i) After the temporary works through the execution of sheet piles the anchorage of the pontoon was done, in order to assure the stability during the driving of the casings. The system had the purpose to assure the verticality of the casings.
ii) Transportation of the metallic 2.2 m diameter and 17 mm thick casing. This casing was driven by a high capacity vibrator and a penetration of 1 to 2 m in geotechnical unit a_{oa} was assured.
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.

Subsequently a guidance system was used to drive the casing 1 diameter into gravel materials or into a compacted ground with a minimum value of SPT 10 blows.
i) Progress of the excavation with a 2.2 m diameter “hammergrab” of in order to reach the Miocene. For the wall stabilization polymers materials manufactured in a central located in the left bank were used. For the polymer control pH tests, density and viscosity tests, as well sand content tests were performed.
ii) After the excavation and the decantation of the polymer the reinforcement with the pipes for the cross-hole tests was

installed. To assure a minimum cover of 12 mm centralizers were placed.
i) Concreting of the piles with the use of “tremie” and pumping was done at a rate of 50 m3/hour.
The duration of these 5 phases was 2.5 days.

In the construction procedure proposed in the Basic Design (Design Group, 2004b) the pile caps for piers P1 and P2 were performed within cofferdams constructed by sheet piles driven into the mud materials trough equipments installed in barges. The voids under the casings were stabilized trough the use of polymers.

For caps P3 to P7 the constructive procedure consisted on the construction of prefabricated caissons in dry dock. The caissons were transported from onshore casted in situ and subsequently the metallic casings were driven trough the holes of the bottom slab and the openings under the casings being stabilized trough the use of polymers.

During the Final Design a solution of pre-fabricated caissons was developed with large caissons for piers P1C and P2C and small caissons for piers P3C to P7C (Design Group, 2005 h).

A view of North Viaduct construction is shown in Fig.41.
To avoid excavations of the protection dykes a parallel way(transient viaduct) was built (Fig.42).
A view of South Viaduct construction is shown in Fig. 43.
The placement of pile casing is shown in Fig. 44.

Fig. 41. Construction of North Viaduct

The pre-fabricated caissons were temporary supported by the casings of the definitive piles. With the support of hydraulic cylinders the temporary metallic structure was uplifted and subsequently the caisson was moved downward until the design level.
After the sealing of the joints between the piles and the bottom slab the water inside the caissons was removed by pumping.

Fig. 42. Parallel Way

Fig. 43. Construction of South Viaduct

Fig. 44. Placement of pile casing (after Ferreira et al, 2008)

The placement of pile reinforcement and tremi pipes are shown in Figs. 45 and 46.
In Figs. 47 to 49 a caisson view, a pier under construction and a general view of the construction works are presented.

Fig. 45. Placement of pile reinforcement (after Ferreira et al, 2008

Fig. 46. Placement of tremie pipes (after Ferreira et al, 2008)

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.

Anomalies that impair the integrity of a pile and that are expected to be identified by integrity tests include the presence of material of poorer quality than expected (locally and overall) and variations in the cross section of the shaft (e.g., crack, necking, and bulb) (Sêco e Pinto and Rodrigues, 1989).

Fig. 47. View of Caisson (courtesy of Perry da Câmara)

Fig, 48. Pier under construction (courtesy of Perry da Câmara)

Fig. 49. General view of the construction works (courtesy of Perry da Câmara)

Also sonic diagraphy 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.

The sound velocity in concrete is around 4000 m/s, but in the presence of anomalies, i.e. fissures, segregations or soil inclusions this value decreases.
The quality of the results depends of the following requirements:
i) Use of metallic tubes with diameter between 35 and 60 mm;
ii) The number of tubes depends of the pile diameter :
diâmeter < 0,60 m = 2 tubes
0,60 m< diâmetro< 1,20 m = 3 tubes placed 120 º apart
diâmetro> 1,20 m = 4 tubes, as a minumum;
iii) The connection between the tubes should be done by joints;
iv) A good contact between the tube and the concrete;
v) At the bottom of the tubes a sealing should be placed to avoid the uplift of the sediments or concrete;
vi) The tubes should be connected to the pile reinforcement along the total length;
vii) The top level of the tubes should be 0.5m above the pile head, as a minimum;
viii) The tubes should be placed vertical and parallel to the pile reinforcement;
ix) The pile test should be performed 3 days after the concreting, as a minimum.

Fig. 50 shows a pile view with 4 tubes.
Taking into account that piles were 1.52 m diameter 4 tubes 90º apart were placed.

In the experimental pile tests located at KM 5+ 400, KM 7+ 900, KM 8+ 200 a verification of integrity tests by cross hole tests were performed.
For piles 1.5 m diameter 4 tubes were placed. The records and tests interpretation were presented by GEOSOLVE (2005a, 2005b, 2005c).

Fig. 50. 4 tubes for crosshole tests in a 1.5m diameter pile (after Ferreira et al, 2008)

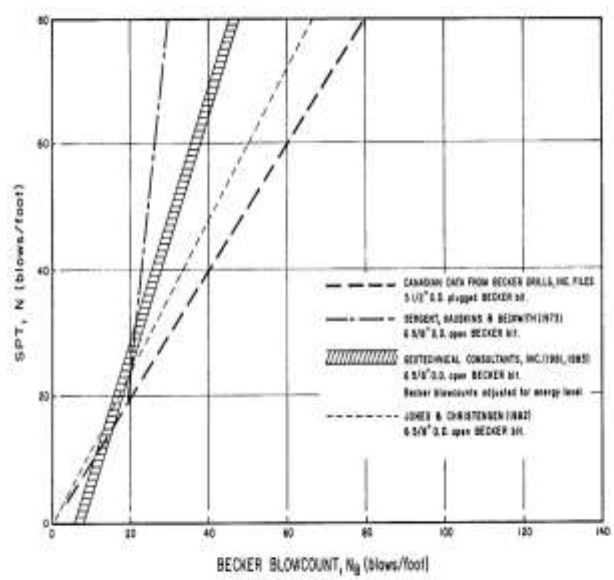

Fig. 51. Correlations between BPT and SPT values (after Harder and Seed, 1986)

SOME PROBLEMS DURING PILE CONSTRUCTION

Introduction

Some problems have occurred during piles construction in the Main Bridge due to the gravel and cobbles dimensions. Due to the difficulties to interpret SPT tests in sandy gravel materials Daniel et al (2004) have conducted a research trough: (i) a comparison between tests with SPT sampler (5.08 cm) and other samplers with higher dimensions, namely japonese with 7.3 cm, italian with 14 cm and american with 7.6 cm, with the purpose to define corrective factors incorporating : (i) the energy transmitted by driven equipment; (ii) using discrete models (DEM) for a better understanding of the crushing effects between the links of particles of gravel materials; (iii) using the theory of waves propagation associated with the records of axial forces, velocity of rods penetration for a better understanding of the particles displacements response and grain dimensions.
The outcome of this research is summarized in Fig. 51 and Fig. 52.

Gravel and cobbles dimensions

(i) Unit a_2 is composed by fine sandy materials with intercalations of silty clay material with gravel material;
(ii) Unit a_3 is composed by medium sandy materials with intercalations of silty clay material with gravel material with thickness varying from 3.3m to 14.10 m and SPT values between 32 to 52 blows and penetration from 11 to 29 cm for 60 blows; (iii) Unit M3 is composed by sands with intercalations of gravel materials; (iv) Unit M4 is composed by sands with intercalations of gravel materials with sand silty.

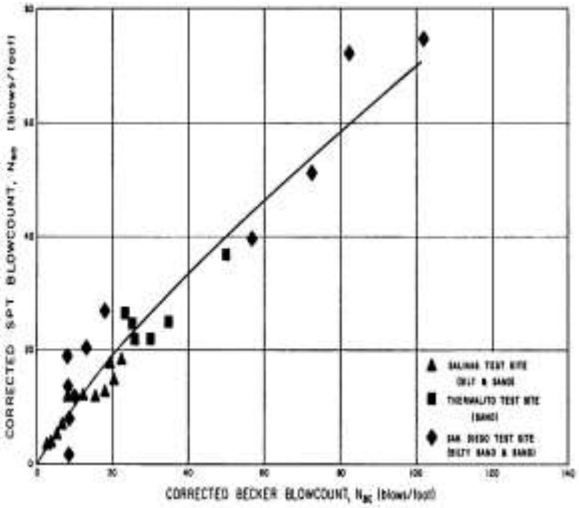

Fig. 52. Correlations between BPT and SPT modified values (after Harder and Seed, 1986)

The additional geotechnical campaign has shown that for material a_3 located between km 6+ 900-7+600 and km 8+900-10+400, the thickness was around 16m and in other sites values between (3-5m) and 8-12m were recorded.

Within this framework it is important to analyze the dimensions of gravels and cobbles based on the referred classifications. the dimensions of gravel materials are between 4.75 mm and 75 mm following the classifications of ASTM-D2487, ASTM –D6538 and USCS and from 2 mm to 60 mm following the classification of LNEC E 219-1968, MIT and BS 5930:1981.

ii) the cobbles have a size between 75 mm and 300 mm following the classifications proposed by USBR(1974), US Army Engineer (1960), USCS and ASTM D 653 and from 60 mm to 300 mm following the classifications of BS 5930:1981 and MIT.

In summary: Following the Geotechnical Report it was expected the occurrence of gravel materials with dimensions between of 2 mm and 75 mm and cobbles with dimensions between 60 mm e 300mm.
Unfortunately Balast Needam has not given the right attention to this issue during the metallic casings design.

Final comments

1) The recorded values of Vs for gravel materials between 320 and 400m/s are compatible with the existent knowledge and allow the definition of gravel compacity.

2) The recorded values of Vs and SPT values allow to classify the a_3 materials as ground category B-C, following EC8.

3) The description of borehole logs of S1B and S6B in the bridge zone has allowed to characterize the thickness and compacity of the gravel materials a_3.

4) Techniques to take undisturbed samples of a_3 materials by frozen techniques were disregarded due to the high costs.

5) The SPT values of a_3 materials between 45-60 blows are equivalent to BPT values between 60 to 80 more adequate to characterize sandy gravel materials following Harder and Seed(1986) proposal.

Records of driveability of metallic casings

The records of driveability of metallic casings have giving the following information: (i) level of penetration: (ii) number of blows to penetrate 25 cm; (iii) energy by blow; (iv) penetration by blow; (v) time for penetration; (vi) level of top and bottom of casing; (vii) method of excavation and type of material (Ballast Nedam, 2005a, 2005b, 2005c).

For Pylon P1C records of the following piles P1-1, P1-2, P1-3, P1-4, P1-5, P1-6, P1-7, P1-8, P1-9 e P1-10 were presented.

For Pylon P2C records of the following piles P2-1, P2-2, P2-3, P2-4, P2-5, P2-6, P2-7, P2-8, P2-9 e P2-10 were presented.

For Pylon P3C records of the following piles P3-5, P3-6, P3-7 e P3-8 were presented.

For Pylon P4C no records were presented.
For Pylon P5C records of the following piles P5-1, P5-2, P5-3, P5-4, P5-5, P5-6, P5-7 e P5-8 were presented.

For Pylon P6C records of the following piles P6-1, P6-2, P6-3, P6-4, P6-5, P6-6, P6-7 e P6-8 were presented.
For Pylon P7C records of the following piles P7-1, P7-2, P7-3, P7-4, P7-5, P7-6, P7-7 e P7-8 were presented.

- Unfortunately the records presented by Ballast Nedam have not given the following information: (i) height of hammer fall; (ii) frequency; (iii) control of pile verticality; (iv) comparison between the driven logs of the casings and the borehole logs.

Records of casing inspections

The records of casings inspections have given the following information (Balast Nedam, 2006a):
- Design Phase: (i) length of metallic casing; (ii) level of bottom pile; (iii) penetration in Miocene.
- Execution Phase: (i) level of casing head; (ii) level of casing bottom, (iii) drivelibility of casing; (iv) depth of excavation, (v) level of pile excavation; (vi) depth of Miocene; (vii) level of embedding in the Miocene; (viii) level of pile bottom; (ix) excavation to be performed in Miocene; (x) lack of excavation missing to fulfill design requirements; (xi) description of the

anomalies detected by the divers.

For pylon P1C records of piles P1-1, P1-2, P1-3, P1-4, P1-5, P1-6, P1-7, P1-8, P1-9 e P1-10, were presented.

From the visual inspections of the metallic casings that were performed by the divers the following occurrences were recorded: (i) casings without deteriorations; (ii) light ovalization at the bottom; (iii) conic deformations at the bottom without the possibility to perform additional excavation; iv) collapse of the casing.

Generally speaking the deteriorations have occurred in an extension of 2 to 4m, situated at depths between 35.3m and 42m, with the exception of casing P1-8 that have exhibited deteriorations in an extension of 16m, between depths of 22.5m and 38.4m.

The type of observed deteriorations is shown in Fig. 53.

The divers have considered that casings P1-1, P1-2, P1-7 e P1-9 have not exhibited deteriorations.

For pylon P2C records of piles P2-1, P2-2, P2-3, P2-4, P2-5, P2-6, P2-7, P2-8, P2-9 e P2-10 were presented.

From the visual inspections of the metallic casings that were performed by the divers the following occurrences were recorded: (i) uplift of the soil; (ii) light ovalization at the bottom; (iii) conic deformations at the bottom without the possibility to perform additional excavation; iv) collapse of the casing; (v) horizontal corrugation; vi) bended steel casing.

Generally speaking the deteriorations have occurred in an extension of 1 to 2m, situated at depths between 31.0m and 40m, with the exception of casing P2-4 that has exhibited deteriorations in an extension of 8-9m, between depths of 27.5m and 36.5m (TACE, 2005) .

For Pylon P3C records for piles P3-1, P3-2, P3-3, P3-4, P3-5, P3-6, P3-7 e P3-8 were presented.

(i) The length of excavation to respect the embedding in Miocene has varied between 0 and 0.5m.

MONITORING DURING CONSTRUCTION AND LONG TERM

Introduction

The designer has the difficult task to perform a correct definition of loads and an adequate characterization of the materials for the project. It is necessary to compare the mental model with the prototype response in order to assess the structural behavior, and to decide in face of an anomalous behavior.

Within this framework it is important to instrument the bridge with the following purposes:

i) Validation of design criteria and calibration of mental model.
ii) Analysis of bridge behavior during its life cycle.
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: 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:

pile head displacements using electronic teodolytes and appropriate reflectors;

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:

(ii) The reference situation corresponds to a detailed inspection of all parts of the structure (foundations, bearings and decks) and the measurement of all the sensors in order to characterize the initial state of the bridge before the opening to traffic;
(iii) The daily inspections aimed 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;
(iv) 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;
(v) 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.

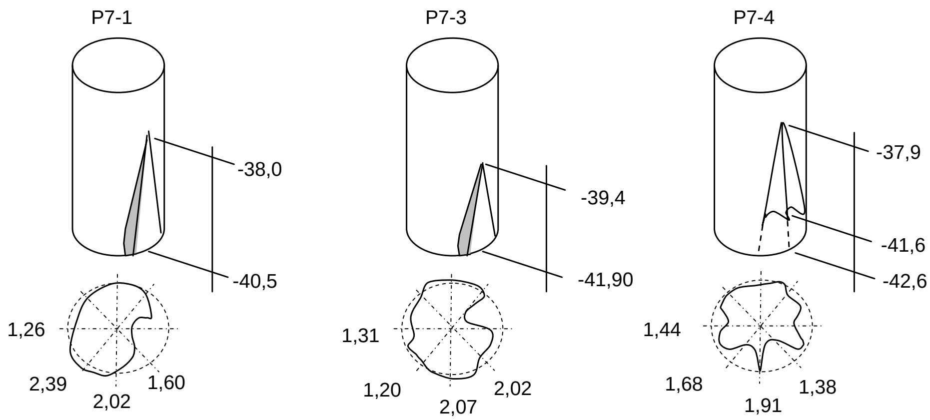

Fig.53 Type of observed deteriorations

CONCLUSIONS

The following conclusions can be outlined:

1) The different geotechnical campaigns implemented during the Preliminary Study (1st phase and 2nd phase) and during the Basic Design have allowed the definition of different geological and geotechnical profiles.
2) The geotechnical characteristics were obtained after a balance between the results of the field and laboratory tests.
3) The geotechnical study in the Basic Design fulfills the requirements of Eurocode 7, Specification 1536 Bored Piles prepared by CEN - Committee TC 288 and the Procedures and Specifications for Piles prepared by ICE (1978).
4) The new Tagus Crossing is located in zone A of Portugal the highest seismic zone.
5) From the Geotechnical Report of the Basic Design the occurrence of gravel materials between 2mm and 75 mm and cobbles materials between 60 mm and 300 mm is expected.
6) The recorded values of Vs for gravel materials between 320 and 400 m/s are compatible with the existent knowledge and allow the definition of gravel compacity.
7) The recorded values of Vs and SPT values allow to classify the a_3 materials as ground category B-C, following EC8.
8) The characterization of gravel materials due their size can not be defined adequately through SPT tests.
9) The description of borehole logs of S1B and S6B in the Main Bridge zone has allowed to characterize the thickness and compacity of the gravel materials a_3.
10) Techniques to take undisturbed samples of a_3 materials by frozen techniques were disregarded due to the high costs.
11) The SPT values of a_3 materials between 45-60 blows are equivalent to BPT values between 60 to 80 more adequate to characterize sandy gravel materials following Harder and Seed(1986) proposal.
12) Unit a_3 is composed by medium sandy materials with intercalations of silty clay material with gravel material with thickness varying from 3.3 m to 14.10 m and SPT values between 32 to 52 blows and penetration from 11 to 29 cm for 60 blows.
13) The additional geotechnical campaign has shown that for material a_3 located between km 6+ 900-7+600 and km 8+900-10+400, the thickness was around 16m and in other sites values between 3-5m and 8-12m were recorded.
14) The piles were designed by i) design models; ii) pile load tests that have given information about the characteristics of gravel materials and techniques of driving the metallic casings; and iii) comparable experience.
15) Static pile load tests both vertical and horizontal were carried out on trial piles to calibrate the design parameters and to optimize the pile lengths. Also dynamic pile tests were performed.
16) The liquefaction potential evaluation was performed only by CPT and SPT tests due to the disturbance that occurs during sampling of sandy materials. Both total and effective stress analyses were performed.
17) Non destructive techniques of pile tests were performed to assess the quality of piles.
18) Records of casing inspections have shown the occurrences of deteriorations in an extension of 2 to 4 m, situated at depths between 35.3 m and 42 m, with the exception of casing P1-8 that exhibited deteriorations in an extension of 16 m, between depths of 22.5 m and 38.4 m.
19) The objectives of monitoring during construction and long term were presented.

ACNOWLEGMENTS

Due to the complexity of Leziria Tagus River Bridge several construction companies and experts were involved, namely Construction Consortium and Design Group. The studies

carried out by them are greatly acknowledged.
As already mentioned the field investigations were carried by Geocontrole and LNEC, and the laboratory tests were performed by Geocontrole.
Special thanks are due to TACE and particularly to Mr. Secundino Vilar and also to BRISA for the permission to publish this paper.
It is important to refer the contributions by IST for the static pile tests and also for the dynamic pile tests.
The reception tests for piles were carried out by Geosolve.

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