Static and Seismic Pile Foundation Design by Load Tests and Calculation Models

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Abstract. In this paper geotechnical design is addressed and particularly Static and Seismic Pile Foundation Design by Load Tests and Calculation Models. Within this framework Eurocode 7 - Geotechnical Design and Eurocode 8 - Design of Structures for Earthquake Resistance are introduced. The ultimate limit states and the serviceability limit states are discussed. Potential liquefiable soils and remedial measures are addressed. Two case histories related with pile design, namely the New Tagus bridge foundation design and the pile design and liquefaction potential evaluation of Leziria bridge foundations based in Eurocodes 7 and 8, are presented. Some conclusions are drawn.

Keywords: bridges, case histories, eurocodes, liquefaction, pile foundations.

Foreword

Prof. Victor de Mello acted as President of the International Society for Soil Mechanics and Foundations Engineering during the tenure 1981-1985 and will be remembered for his actions and passion to implement geotechnical activities worldwide.

Professor Victor de Mello is a man of prodigious energy and fine intellect. A genial thinker, Victor de Mello was one the bright talents that have enlightened the Geotechnical Engineering road.

I had the opportunity to meet Prof. Victor de Mello in Mozambique in 1972, when he was acting as Consulting Expert for Massingir dam and I was initiating my first steps in geotechnical engineering. My debt of gratitude for him is so huge and I would like to recall this Master who taught me to think, to investigate, to be in Geotechnique and whose friendship was for me a great lesson.

Professor Victor de Mello was often invited to be Keynote Speaker at international conferences of geotechnical engineering and other events and we always listened to his lectures with great interest and pleasure, as they were challenging and opened new avenues of research.

I would like to highlight from Prof. Victor de Mello outstanding curriculum: i) his solid scientific background and research contributions to the advancement of knowledge of embankment dams and special foundations; ii) his significant contribution as author/co-author of papers for Journals, widely accepted throughout the world; (iii) his excellent lecturing and teaching ability to communicate, to support and to encourage students; (iv) his skill to establish synergies with Industry.

His legacy will last for many generations and will always be a source of great inspiration for all geotechnical engineers.

Victor de Mello has oriented his existence to a great and noble ideal and has always taught us that the correct method to learn science is to pursue the discovery of the scientific truth.

His legacies, where the Scientist, the Professor and the Engineer are integrated into one soul, were the beauty and the truth friendly given. I believe that everybody will fully agree with me in classifying his activity with five E’s - Exciting, Elegant, Efficient, Excellent and Extraordinary.

But it is not sufficient to remember the Master, it is important to follow his example, to give continuity with energy and perseverance to his heritage. This will be the best contribution of the current and next generations to honor Victor de Mello memory.

1. Introduction

The Commission of the European Communities (CEC) initiated in 1975 the establishment of a set of harmonized technical rules for the structural and geotechnical design of buildings and civil engineering works based on article 95 of the EC Treaty. In a first stage, they would serve as alternative to the national rules applied in the various Member States and in a final stage they will replace them.

From 1975 to 1989, the Commission, with the help of a Steering Committee with Representatives of the Member States, developed the Eurocodes programme.

The Commission, the Member states of the EU and EFTA decided in 1989, based on an agreement between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN.

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The Structural Eurocode programme comprises the following standards:

- EN 1990 Eurocode - Basis of structural design
- EN 1991 Eurocode 1 - Actions on structures
- EN 1992 Eurocode 2 - Design of concrete structures
- EN 1993 Eurocode 3 - Design of steel structures
- EN 1994 Eurocode 4 - Design of composite steel and concrete structures
- EN 1995 Eurocode 5 - Design of timber structures
- EN 1996 Eurocode 6 - Design of masonry structures
- EN 1997 Eurocode 7 - Geotechnical design
- EN 1998 Eurocode 8 - Design of structures for earthquake resistance
- EN 1999 Eurocode 9 - Design of aluminium structures.

The work performed by the Commission of the European Communities (CEC) in preparing the “Structural Eurocodes” in order to establish a set of harmonised technical rules is impressive. The current tendency is to prepare unified codes for different regions but keeping the freedom for each country to choose the safety level defined in each National Document of Application. The global safety factor was replaced by partial safety factors applied to actions and to the strength of materials.

In this lecture, a summary of the main topics covered by Eurocodes 7 and 8 for Static and Seismic Pile Foundation Design by Load Tests and Calculation Models is addressed for a better understanding of the New Tagus and Leziria bridges foundation design.

In dealing with these topics, we should never forget the memorable lines of Lao-Tze, Maxin 64 (550 B.C.):

“The journey of a thousand miles begins with one step.”

2. Eurocode 7 - Geotechnical Design

2.1. Introduction

The Eurocode 7 (EC7) “Geotechnical Design” gives a general basis for the geotechnical aspects of the design of buildings and civil engineering works. A link is established between the design requirements addressed in Part 1 and the results of laboratory tests and field investigations run according to standards, codes and other accepted documents covered by Part 2.

EN 1997 is concerned with the requirements for strength, stability, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

2.2. Eurocode 7 - Geotechnical design - Part 1

The following subjects are covered in EN 1997-1 - Geotechnical Design:
- Section 1: General
- Section 2: Basis of geotechnical design
- Section 3: Geotechnical data
- Section 4: Supervision of construction, monitoring and maintenance
- Section 5: Fill, dewatering, ground improvement and reinforcement
- Section 6: Spread foundations
- Section 7: Pile foundations
- Section 8: Anchorages
- Section 9: Retaining structures
- Section 10: Hydraulic failure
- Section 11: Overall stability
- Section 12: Embankments.

2.2.1. Design requirements

The following factors shall be considered when determining the geotechnical design requirements:
- site conditions with respect to overall stability and ground movements;
- nature and size of the structure and its elements, including any special requirements such as the design life;
- conditions with regard to its surroundings (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.);
- ground conditions;
- groundwater conditions;
- regional seismicity;
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

For each geotechnical design situation it shall be verified that no relevant limit state is exceeded.

Limit states can occur either in the ground or in the structure or by combined failure in the structure and the ground.

Limit states should be verified by one or a combination of the following methods: design by calculation, design by prescriptive measures, design by load tests and experimental models and an observational method.

To establish geotechnical design requirements, three Geotechnical Categories, 1, 2 and 3 are introduced:
- Geotechnical Category 1 includes small and relatively simple structures.
- Geotechnical Category 2 includes conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions.
- Geotechnical Category 3 includes: (i) very large or unusual structures; (ii) structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions; and (iii) structures in highly seismic areas.

2.2.2. Geotechnical design by calculation

Design by calculation involves:
- actions, which may be either imposed loads or imposed displacements, for example from ground movements;
- properties of soils, rocks and other materials;
• geometrical data;
• limiting values of deformations, crack widths, vibrations, etc.
• calculation models.

The calculation model may consist of: (i) an analytical model; (ii) a semi-empirical model; or (iii) a numerical model.

Where relevant, it shall be verified that the following limit states are not exceeded:
• loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance (EQU);
• internal failure or excessive deformation of the structure or structural elements, including footings, piles, basement walls, etc., in which the strength of structural materials is significant in providing resistance (STR);
• failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO);
• loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions (UPL);
• hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).

The selection of characteristic values for geotechnical parameters shall be based on derived values resulting from laboratory and field tests, complemented by well-established experience.

The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

For limit state types STR and GEO in persistent and transient situations, three Design Approaches are outlined. They differ in the way they distribute partial factors between actions, the effects of actions, material properties and resistances. In part, this is due to differing approaches to the way in which allowance is made for uncertainties in modeling the effects of actions and resistances.

In Design Approach 1, partial factors are applied to actions, rather than to the effects of actions and ground parameters.

In Design Approach 2, partial factors are applied to actions or to the effects of actions and to ground resistances.

In Design Approach 3, partial factors are applied to actions or the effects of actions from the structure and to ground strength parameters.

It shall be verified that a limit state of rupture or excessive deformation will not occur.

It shall be verified serviceability limit states in the ground or in a structural section, element or connection.

2.2.3. Design by prescriptive measures

In design situations where calculation models are not available or not necessary, the exceedance of limit states may be avoided by the use of prescriptive measures. These involve conventional and generally conservative rules in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures.

2.2.4. Design by load tests and experimental models

When the results of load tests or tests on large or small scale models are used to justify a design, the following features shall be considered and allowed for:
• differences in the ground conditions between the test and the actual construction;
• time effects, especially if the duration of the test is much less than the duration of loading of the actual construction;
• scale effects, especially if small models are used. The effect of stress levels shall be considered, together with the effects of particle size.

Tests may be carried out on a sample of the actual construction or on full scale or smaller scale models.

2.2.5. Observational method

When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as “the observational method”, in which the design is reviewed during construction.

The following requirements shall be met before construction is started:
• the limits of behaviour which are acceptable shall be established;
• the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
• a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
• the response time of the instruments and the procedures for analyzing the results shall be sufficiently rapid in relation to the possible evolution of the system;
• a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.

2.3. Eurocode 7 - Part 2

EN 1997-2 is intended to be used in conjunction with EN 1997-1 and provides rules supplementary to EN 1997-1 related to the:
• planning and reporting of ground investigations;
• general requirements for a number of commonly used laboratory and field tests;
• interpretation and evaluation of test results;
• derivation of values of geotechnical parameters and coefficients.
The field investigation programme shall contain:

- a plan with the locations of the investigation points including the types of investigations;
- the depth of the investigations;
- the type of samples (category, etc.) to be taken including specifications on the number and depth at which they are to be taken;
- specifications on the groundwater measurement;
- the types of equipment to be used;
- the standards that are to be applied.

The laboratory test programme depends in part on whether comparable experience exists.

The extent and quality of comparable experience for the specific soil or rock should be established.

The results of field observations on neighbouring structures, when available, should also be used.

The tests shall be run on specimens representative of the relevant strata. Classification tests shall be used to check whether the samples and test specimens are representative.

This can be checked in an iterative way. In a first step, classification tests and strength index tests are performed on as many samples as possible to determine the variability of the index properties of a stratum. In a second step, the representativeness of strength and compressibility tests can be checked by comparing the results of the classification and strength index tests of the tested sample with entire results of the classification and strength index tests of the stratum.

Figure 1 shows the flow chart that demonstrates the link between design and field and laboratory tests. The design part is covered by EN 1997-1; the parameter values part is covered by EN 1997-2.

3. Eurocode 8 - Design of Structures for Earthquake Resistance

3.1. Introduction

Eurocode 8 (EC8), “Design of Structures for Earthquake Resistance”, deals with the design and construction of buildings and civil engineering works in seismic regions and it is divided in six Parts.

Part 1 is divided in 10 sections:
- Section 1 - contains general rules, seismic actions and rules for buildings;
- Section 2 - contains the basic performance requirements and compliance criteria applicable to buildings and civil engineering works in seismic regions;
- Section 3 - gives the rules for the representation of seismic actions and their combination with other actions;
- Section 4 - contains general design rules relevant specifically to buildings;
- Section 5 - presents specific rules for concrete buildings;
- Section 6 - gives specific rules for steel buildings;
- Section 7 - contains specific rules for composite steel-concrete buildings;
- Section 8 - presents specific rules for timber buildings;
- Section 9 - gives specific rules for masonry buildings;
- Section 10 - contains fundamental requirements and other relevant aspects for the design and safety related to base isolation.

Further Parts include the following:
- Part 2 contains provisions relevant to bridges.
- Part 3 presents provisions for the seismic strengthening and repair of existing buildings.

Figure 1 - Flow chart that demonstrates the link between design and field and laboratory tests.
Part 4 gives specific provisions relevant to tanks, silos and pipelines.

Part 5 contains specific provisions relevant to foundations, retaining structures and geotechnical aspects and complements the rules of Eurocode 7, which do not cover the special requirements of seismic design.

Part 6 presents specific provisions relevant to towers, masts and chimneys.

3.2. Seismic action

The definition of Actions (with the exception of seismic actions) and their combinations is treated in Eurocode 1 “Actions on Structures”.

In general, the national territories are divided by the National Authorities into seismic zones, depending on the local hazard.

The earthquake motion in EC 8 is represented by the elastic response spectrum defined by 3 components.

In EC 8, in general, the hazard is described in terms of a single parameter, i.e. the value $\alpha_s$ of the effective ground acceleration in rock or firm soil called “design ground acceleration” (Fig. 2) expressed in terms of: a) the reference seismic action associated with a probability of exceedance ($P_{ech}$) of 10% in 50 years; or b) a reference return period ($T_{ech}$) = 475, where: $S_e$ (T) elastic response spectra, $T$: vibration period of a linear single-degree-of-freedom system, $\alpha_s$: design ground acceleration, $T_B$, $T_C$: limits of the constant spectral acceleration branch, $T_D$: value defining the beginning of the constant displacement response range of the spectra, $S$: soil factor with reference value 1.0 for subsoil class A, $\eta$: damping correction factor with reference value 1.0 for 5% viscous damping.

It is recommended the use of two types of spectra: Type 1 if the earthquake has a surface wave magnitude, $M_s$, greater than 5.5, and Type 2 in other cases.

The seismic motion may also be represented by ground acceleration time-histories and related quantities (velocity and displacement).

Artificial accelerograms shall match the elastic response spectrum. The number of the accelerograms to be used shall give a stable statistical measure (mean and variance) and a minimum of 3 accelerograms should be used and also some others requirements should be satisfied.

For structures with special characteristics, spatial models of the seismic action shall be used based on the principles of the elastic response spectra.

3.3. Ground conditions and soil investigations

For the ground conditions five ground types A, B, C, D and E are considered:

Ground type A - rock or other geological formation, including at most 5 m of weaker material at the surface, characterized by a shear wave velocity $V_s$ of at least 800 m/s;

Ground type B - deposits of very dense sand, gravel or very stiff clay, at least several tens of m in thickness, characterized by a gradual increase of mechanical properties with depth shearing wave velocity between 360-800 m/s, $N_{sp}$ > 50 blows and $c_u$ > 250 kPa.

Ground type C - deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters, characterized by a shear wave velocity from 160 m/s to 360 m/s, $N_{sp}$ from 15 to 50 blows and $c_u$ from 70 to 250 kPa.

Ground type D - deposits of loose to medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft to firm cohesive soil, characterized by a shear wave velocity less than 180 m/s, $N_{sp}$ less than 15 and $c_u$ less than 70 kPa.

Ground type E - a soil profile consisting of a surface alluvium layer with $V_{s30}$ values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_{s30}$ > 800 m/s.

For the five Ground type the recommended values for the parameters $S$, $T_B$, $T_C$, $T_D$, for Type 1 and Type 2 are given in Tables 1 and 2.

The recommended Type 1 and Type 2 elastic response spectra for ground types A to E are shown in Figs. 3 and 4.

The recommended values of the parameters for the five ground types A, B, C, D and E for the vertical spectra are shown in Table 3. These values are not applied for ground types $S_1$ and $S_2$. 

Figure 2 - Elastic response spectra (after EC8).
4. Foundation Design

4.1. Introduction

The foundation system is presented, with particular emphasis to soil-structure interaction. The serviceability limit states are introduced.

For pile design, the following limit states shall be considered (Eurocode 7, 1997a): (i) loss of overall stability; (ii) bearing resistance failure of the pile foundation; (iii) uplift or insufficient tensile resistance of the pile foundation; (iv) failure in the ground due to transverse loading of the pile foundation; (v) structural failure of the pile in compression, tension, bending, buckling or shear; (vi) combined failure in the ground and in the pile foundation; (vii) combined failure in the ground and in the structure; (viii) excessive settlement; (ix) excessive heave; (x) excessive lateral movement of the ground; and (xi) unacceptable vibrations.

For the Soil-Structure Interaction (SSI) the design engineers ignore the kinematic component, considering a fixed base analysis of the structure, due to the following reasons: (i) in some cases the kinematic interaction may be neglected; (ii) aseismic building codes, with a few exceptions e.g. Eurocode 8 do not refer it; (iii) kinematic interaction effects are more difficult to assess than inertial forces.

For slender tall structures, structures founded in very soft soils and structures with deep foundations, the SSI plays an important role.

The Eurocode 8 states:” Bending moments developing due to kinematic interaction shall be computed only when two or more of the following conditions occur simultaneously: (i) the ground profile is of class D, S1 or S2, and contains consecutive layers with sharply differing stiffness; (ii) the zone is of moderate or high seismicity, $c97 > 0.10$; (iii) the supported structure is of importance class I or II”.

Piles and piers shall be designed to resist the following action effects: (i) inertia forces from the superstructure; and (ii) kinematic forces resulting from the deformation of the surrounding soil due to the propagation of seismic waves (Fig. 5). The decomposition of the problem in steps is shown in Fig. 5 (Gazetas & Mylonakis, 1998).

The complete solution is a very time consuming 3D analysis.
To analyze the internal forces along the pile, as well as the deflection and rotation at the pile head, discrete models (based in Winkler spring model) or continuum models can be used.

For simplicity a linear soil behaviour is assumed.

The following effects shall be included: (i) flexural stiffness of the pile; (ii) soil reactions along the pile; (iii) pile-group effects; and (iv) the connection between pile and structure.

**Figure 5** - Soil-structure interaction problem (after Gazetas & Mylonakis, 1998).

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4.2. Serviceability limit states

To prevent the occurrence of an ultimate limit state or a serviceability limit state the foundation movements shall not reach certain limit values (Sêco e Pinto & Sousa Coutinho, 1991).

Burland & Wroth (1974) proposed a consistent set of definitions based on the displacements, that are illustrated in Fig. 6:

- rotation (θ) is the change in gradient of a line joining two reference points;
- the angular strain (α), defined in Fig. (6a), is positive for upward concavity (sagging) and negative for downward concavity (hogging);
- relative deflection (Δ) is the displacement of a point relative to the line connecting two reference points on either side (see Fig. 6(b));

\[
\theta_{\text{max}}, \alpha_{\text{max}}, \delta_{\text{max}}
\]

\[
\Delta L, \delta/L
\]

\[
\phi, \beta
\]

Figure 6 - Definition of foundation movements (after Burland & Wroth, 1974).

Table 4 presents a summary of allowable deformations proposed by different authors and EC7.

Table 4 - Allowable deformations.

|                | Skempton & MacDonald (1956) | Meyerhof (1956) | Polshin & Tokar (1957) | Bjerrum (1963) | EC7 (1994) |
|----------------|----------------------------|-----------------|------------------------|----------------|------------|
| **A - Concrete buildings and reinforced walls** |  |  |  |  |  |
| Allowable values for rotations Structural damage & cracks on walls | 1/150 | 1/250 | 1/200 | 1/150 | 1/150 |
|  | 1/300 | 1/500 | 1/500 | 1/500 | 1/300 |
| **B - Wall without reinforcement** |  |  |  |  |  |
| Deflection ratio Δ/L | Meyerhof (1956) | 1/2500 | Polshin & Tokar (1957) |  |  |
| Deform. ⊙ |  | L/H < 3 1/3500 to 1/2500; 1/2500 L/H = 1 |
| Deform. ⊙ |  | L/H > 5 1/2000 to 1/1500 1/1250 L/H = 5 |
|  |  | 1/5000 L/H = 1 |
|  |  | 1/2500 L/H = 5 |

Following EC7, the settlements for pile foundations for ultimate limit states and serviceability limit states shall include:

- the settlement of a single pile;
- the additional settlement due to group action.

The selection of design values for limiting movements shall take account of the following:

(i) the confidence to specify the acceptable value of the movement; (ii) the type of structure; (iii) the type of construction material; (iv) the type of foundation; (v) the type of ground; (vi) the mode of deformation; and (vii) the proposed use of the structure.

Bozozuk (1981), based on the observation of 150 cases related with the allowable displacement in bridge foundation piles, has proposed the limits for vertical and horizontal settlement, \(S_V\) and \(S_H\), defined in Table 5.

Moulton (1986) based on the analysis of 314 bridges located in the United States and Canada has confirmed the proposal of Bozozuk (1981).

Burland et al. (1977) have proposed 6 categories for damage in buildings shown in Table 6, where categories 0, 1 and 2 are related with esthetic damage, categories 3 and 4 are related with serviceability limit states and category 5 with ultimate limit states (stability).

Burland et al. (1977) have introduced the concept of limiting tensile strain, \(\varepsilon_{\text{lim}}\), to define the ultimate limit state.

Boscardin & Cording (1989) develop the concept of differing levels of tensile strain and based on the analysis of 17 cases have proposed Table 7 to establish the relationship...
between the category of damage and the limiting tensile strain.

Burland (1995) proposed three levels of risk for buildings: (i) preliminary evaluation; (ii) evaluation of second level; (iii) detailed evaluation.

The computation of the differential settlement shall take into consideration: (i) the variation of the ground properties; (ii) the distribution of loads; (iii) the construction methodology; (iv) the stiffness of the structure.

5. New Tagus Bridge

5.1. Introduction

The 18 km Tagus Bridge from Sacavém to Montijo (Lisbon, Portugal), integrates the North viaduct, the Expo viaduct, the cable stayed bridge, the central viaduct and the South viaduct (Fig. 7).

The bridge foundations are composed by bored and driven piles.

The central viaduct, 6.5 km long, is supported on 648 driven piles up to 60 m long. Related the central viaduct each pier is supported by eight piles with of 1.7 m diameter except in the vicinity of shipping channels.

Three channels cross the Tagus bridge: the main thoroughfare under the cable stayed bridge, and the piles supporting these piers are 2.2 m in diameter, to minimize possible ship impact; and two smaller channels under the central viaduct.

Driven piles were installed by large barge mounted cranes with a capacity around 58 ton.

For the foundations of the cable stayed bridge, with 0.83 km long, and the south viaduct, with 3.9 km long, bored piles were used. For the cable stayed 148 piles with 2.2 m diameter were used. For the south viaduct 60 piles with 2 m diameter and 280 piles with 1.8 m diameter were used.

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

One of the most important key issues for design is the risk of earthquakes, as Lisbon was wiped out by an earthquake in 1755 with 8.5 of Richter magnitude. During a serious seismic event the new Tagus bridge will be the main access for emergency vehicles crossing the estuary.

5.2. Main geological conditions

Based on the geological data obtained by two implemented site investigation programmes, the ground is composed by the following two main units (Fig. 7), namely: a) Alluvial deposits (Al), aged Holocene and Pleistocene; and b) the bedrock under alluvial deposits, composed by Plio-Pleistocene materials.

The maximum observed thickness of Al is around 78 m and in average, its thickness varies between 60 and 70 m.

Five sub-units were identified, named \(a_{\alpha}, a_{\beta}, a_{\gamma}, a_{\delta}\) and \(a_{\epsilon}\). The \(a_{\alpha}\) to \(a_{\epsilon}\) units show the common geological structure of alluvial deposits, with lenticular or interstratified layers, with exhibiting lateral variations within each sub-unit.
At the bottom of the alluvial deposits occurs a gravel layer (a3), integrating fine to coarse gravel, with sand, cobbles and occasionally boulders. The coarser elements (cobbles and occasionally boulders) occur scattered or concentrated in some zones, raising difficulties for the drilling equipment to cross the a3 layer.

The general description of each type of the differentiated alluvial deposits is the following (Oliveira, 1997):

- a0: This unit is composed by silty to very silty clay (mud), dark grey, with a maximum thickness around 35 m.
- a1: Fine to medium sand with shells and shell fragments.
- a2a: Silty clay to clayey silt.
- a2b: Yellowish brown to grey medium to coarse (occasionally fine) sand.
- a3: Fine to coarse gravel, rounded to angular, with sand, cobbles and occasionally some boulders.

The bedrock under the alluvial deposits is composed by Plio-Pleistocene materials.

5.3. Pile load tests

5.3.1. Introduction

Pile load tests were performed with the following purposes:

- i) to determine the response of a representative pile related to the settlement and limit load;
- ii) to verify the performance of individual piles and to extrapolate for the overall pile foundation behavior;
- iii) to optimize 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 to optimize pile lengths, based on the interpretation of site investigation and laboratory and situ tests (Sêco e Pinto & Oliveira, 1998).

5.3.2. 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 has respected the following procedure:

(i) installation by vibro-driving, with a SOILMECH VTE 12000, of a permanent casing, with an exterior diameter of 1216 mm and a thickness of 8 mm, and 16 mm at the shoe level with 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.

Figure 7 - Simplified geotechnical profile (after Oliveira, 1997).
Load tests were carried out on several test piles and the test locations were representative of the pile where the most adverse ground conditions are believed to occur.

The following equipment for the vertical load tests was 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.

The loading program consisted in reaching 20000 kN with 8 load increments.

A general view for vertical pile load tests is presented in Fig. 8.

The load - settlement curves for piles P8, P31 and P79 are shown in Fig. 9.

For the definition of the failure loads the criteria of settlement equal to 10 % of the pile diameter, i.e. at 120 mm settlement, was used. A comparison between the predicted failure loads, based from CPT tests, and the observed values is shown in Table 8.

| P8 | P31 | P79 | P31i |
|----|-----|-----|------|
| 15 | 20.3| 15  | 21.4 |
| m  | m   | m   | p    |

With the exception of P79 (the length of this pile was increased 10 m) the observed values are lower than the predicted loads and the difference were attributed to the lower shaft friction values. The results of P31i have shown an insufficient gain of the bearing capacity with soil grouting.

5.3.3. Horizontal pile load tests

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

The piles were constructed by the same procedure already described.

The installed equipment has integrated: (i) - horizontal displacement; (ii) - load cell; (iii) - strain along the shaft using strain gauges; (iv) - displacement along the vertical using inclinometer tubes; (v) - temperature.

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

Figure 10 shows a general view for horizontal pile load tests.

To evaluate the effect of ship impact, a second series of load increments were applied, form 500 kN to 1 000 kN, for the south pylon, after 10 h.

The load displacement curve measured at 0.95 m below load level is shown in Fig. 11.

The computed values for pile displacements, bending moments and shear forces are shown in Fig. 12.

5.3.4. Dynamic pile tests

For a better characterization of the dynamic behaviour of the alluvial material for the bridge foundation a forced vibration test of a group of two piles was performed. The piles with 1.20 m of diameter and 60 m long were connected by a cap with $5.5 \times 3.5 \times 1.2$ m.
A 3D finite element model was implemented for the interpretation of the observed behaviour. The soil-pile system was discretized with 3D finite elements of the second degree (cubic with 20 nodal points). The numerical results were compared with the observed values, in terms of displacement transfer functions.

To impose on the pile cap, harmonic horizontal loads, with different amplitudes and frequencies a shaker (Fig. 13), built in LNEC, was used.

The excitation frequencies were applied in steps of 0.1 Hz in the range from 0.5 to 20 Hz approximately.

Velocity transducers and accelerometers were installed to assess the dynamic response of the structure, for the various frequencies of excitation.

This equipment was placed in several points in order to monitoring the horizontal and vertical displacements (Fig. 14).

During the test time series of velocity were recorded on several points. The digital treatment of this time series was performed by a computer program developed at LNEC. Treated series are transported for frequency domain and the displacements were obtained by integration.

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

For the material behavior a simplified model was adopted considering for the piles a continuous, homogeneous and isotropic material with a linear and elastic behavior. The soil was considered a continuous material, with elastic behaviour, and composed of various homogeneous layers.

Figure 15 shows the configuration of the two first modes of vibration and respective frequencies (observed and computed). 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 based on the best adjustment to the transfer functions observed in the test. The adopted values are presented in Table 9.

The observed and calculated frequencies by the mathematical model are presented in Table 10. There is a good agreement for the two first vibration modes.

The displacement transfer functions of the force applied by the shaker are shown in Fig. 16.

The results observed in the test and those computed by the mathematical model in terms of displacement transfer functions of the force applied have shown that: (i) In order to improve the pile behaviour field tests instrumented piles are highly recommended for design purposes; (ii) The results of load tests performed in New Tagus bridge and Lezíria bridge for design purposes have shown how they should be used to calibrate the design parameters, to check the performance of individual piles, to allow judgement of the overall pile foundation, and to assess the suitability of the construction method.

The good agreement obtained shows that the mathematical model is well calibrated for simulation of the behaviour of the soil-pile system. The variation of pile maximum displacement with depth according to directions X and Y, as well as some displacement transfer functions computed at different depths, are shown in Fig. 17.

### 5.4. Reception tests for piles

Taking into account that the development and implementation of load tests is very costly and can only be performed in a small number of piles, non-destructive techniques, e.g. the use of core sampling and integrity tests to control the final quality of the piles, are getting more popular.

To assess the quality of piles sonic tests were performed. The involved costs are small, the execution is fast, it is possible to perform a great number of tests and the prin-
Principal structural singularities of the piles can be detected. With this technique, a blow on the pile head is performed with a hammer and the response is recorded by an accelerometer.

Also sonic diagraph tests were done and a continuous recording was performed along 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.

5.5. Monitoring during construction and long term

5.5.1. Introduction

The designer faces always the difficult task to define the loads and to characterize the materials for the project. In spite of the great progress performed in these domains it is necessary to compare the model with the prototype response in order to assess the structural behaviour and to define the corrective actions to be implemented in case of an anomalous behaviour.

The following advantages of instrumentation of bridges are pointed out:

i) Validation of design criteria and calibration of the model.
ii) Analysis of bridge behaviour during its life.
iii) Definition of corrective measures for the rehabilitation of the structure, if needed.
iv) Cumulative experience that will be important for the design of more economic and safer bridges.

5.5.2. Quantities to be measured

For the superstructure, the measurement of the following quantities was proposed:

- deck vertical displacement;
- pier cross-sections rotations;
- internal deck and piers deformation;
- internal deck deformation due to time-dependent effects;
- deck and stay temperatures;
- air temperature, relative humidity and wind speed;
- seismic and wind induced acceleration in the deck and piers;
- force in stays.

Related with the infrastructure the following measurements were proposed:

- pile head displacements using electronic theodolites and appropriate reflectors;
- installation of inclinometers to measure horizontal displacements along the pile shaft;
- strain distribution of the piles using extensometers;
- recording of the accelerations, velocities and displacements along the piles and in selected points of the ground (to assess amplification effects) by 3D accelerographs.

5.5.3. 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, the decision will be taken by a specialist.

5.5.4. Inspections

To complement the data given by the instrumentation of the bridge regular inspections would be performed. Four levels of inspection were proposed:

i) The reference situation corresponds to a detailed inspection of all parts of the structure (foundations, bearings and decks) and the measurement of all the instruments with the purpose to characterize the initial state of the bridge before opening to the traffic;

ii) The daily inspections aim at an efficient visual checking of the superstructure (drainage systems, road surface, expansion joints, handrail, gantries, safety barriers, lighting, etc.) to detect the need for 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 bridge operation, the frequency of the inspections is five years.

v) After a ship impact or earthquake with a magnitude superior to 4, a detailed inspection is recommended.
6. Leziria Bridge

6.1. Brief description

BRISA awarded to a Construction Consortium the bridge Project that integrates the Conception, Design, and Construction of Tejo Crossing in Carregado.

The crossing (Fig. 18) is composed by the North Viaduct, the Main Bridge and the South Viaduct.

This 11.9 km long crossing of the Tagus river, is located 25 km upstream of the Vasco da Gama Bridge.

The river, 1 km wide, runs in Tagus valley filled with soft sediments that exhibits a thickness between 35 m and 55 m, with a maximum value of 62 m (Oliveira et al., 2006).

The 1695 m North Viaduct has 33 m spans. The deck 23 m above the water level, 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. 19).

The cross-section of the Main Bridge is composed by (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 with a total width of 29.95 m includes a curb, on which rests a safety barrier, a maintenance foot walk and an edge beam with a total width of 1.15 m.

The deck is made of a pre-stressed cast in place concrete box-section 970 m long (Fig. 20). The individual spans are: 95 + 6/180 + 130 + 95 m. Piers P1 to P5 are monolithical with the deck and composed by two blades of reinforced concrete with 1.20 m thick spaced 5.0 m between axes. Piers P6 to P7 are similar with the blades spaced 7.40 m.

Figure 18 - Leziria Tagus River Crossing site.

Figure 19 - North Viaduct (courtesy of Charles Lavigne).
The bridge foundations are composed of 2.20 m diameter piles. The Piers P1 and P2, and the Piers P3 to P7 are supported by 10 piles and 8 piles, respectively. The piles were built by metallic casings 17 mm thick penetrating in the Miocene formations 1m to 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 effects.

The pile caps with 11.0 m and 8 m thick to support piers P1C and P2C, were designed to resist ship impact. For piers P3C to P7C pile cap with 11.0 x 16.0 m and 5.05 m thick were adopted.

The South Viaduct is composed by a set of 22 continuous viaducts, with a total length of 9230 m, with a concrete deck longitudinal pre-stressed with current spans of 36 m and 1.5 m of diameter piles.

One of the most important considerations for design is the risk of earthquakes since Lisbon was wiped out by an 8.5 Richter magnitude earthquake in 1755.

6.2. Main geological conditions

6.2.1. Regional geology

The new Tagus River crossing, located in the Cenozoic basin of the Tagus river, incorporates sedimentary materials of Miocene and Paleocene ages. Figure 21 illustrates a simplified geological profile.

6.2.2. Geomorphology

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

6.2.3. Geological structure

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

6.2.4. Lithostratigraphy

The site is composed of recent superficial deposits, namely Holocene alluvial and quaternary fluvial terraces above the bedrock that integrates Miocene clay-grey materials. The visual aspect of materials is shown in Fig. 22.
6.2.5. Hydrogeological conditions

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

6.3. Field investigation

The field investigations have included 58 boreholes, namely 6 boreholes during the Preliminary Studies, followed by 49 boreholes and 3 additional boreholes during the complementary investigation program for the Basic Design. The boreholes were performed by Geocontrole. In all boreholes, the disturbed samples were collected by a Terzaghi sampler, the water level was recorded and SPT tests were performed 1.5 m apart.

Thirty two undisturbed samples were collected using Shelbi and Proctor-Moran samplers.

Thirty-two cone penetration tests, namely 4 CPT tests during the Preliminary Studies, 20 CPT tests during the complementary investigation, 6 CPTu tests, and 2 seismic cones were performed by Geocontrole.

Nineteen vane shear tests, namely 3 tests during the Preliminary Studies and 16 tests during the complementary campaign by Geocontrole.

Nine seismic cross-hole tests were performed, namely 7 tests by GEOCISA and 2 tests by LNEC during the Preliminary Study. In addition, 7 downhole tests were performed.

During the Final Design the complementary geotechnical project has integrated:

i) 41 boreholes with SPT tests 1.5 m apart (Fig. 23);
ii) 10 vane shear tests;
iii) 25 undisturbed samples taken with Geobore S sampler (Fig. 24);
iv) 16 CPTU tests (Figs. 25 and 26);
v) 5 seismic cross-hole tests.

A summary of field tests is presented in Table 11. The cross-hole 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. 27.

SPT results were between 0 and 4 blows, with 0 values for soft soils 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. 28.
Table 11 - 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     |

Figure 25 - CPTu equipment (after Oliveira et al., 2006).

Figure 26 - CPTu tip (after Oliveira et al., 2006).

Figure 27 - Variation of $V_s$ with depth.

Figure 28 - Variation of undrained strength with depth.
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. 29.

Pore pressures values have allowed the identification of zones with higher values related with mud materials.

6.4. Laboratory tests

During the Basic Design, 12 identification tests (sieve analyses and Atterberg limits) were performed by COBA. During the Preliminary Studies, forty-three identification tests, consisting on sieve analyses as well as Atterberg limits, were performed. Determinations of natural water content, \( W_n \), were also done.

Table 12 summarizes the results of laboratory tests. In three water samples, \( \text{pH} \) tests, determination of alkalies, sulfate content, magnesium content and ammonia content were performed.

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

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

Twenty-two oedometer 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.

Twenty-four permeability tests were done.

Twelve chemical tests related with sulfate content, carbonate content and \( \text{pH} \) values were performed.

Also twenty-five particle density tests were performed.

Three cyclic torsional simple shear tests were done.

A view of cyclic torsional simple shear apparatus is presented in Fig. 30.

The curves \( G \) (shear modulus) vs. \( \gamma \) (shear strain), \( \sqrt{G} \) vs. \( \gamma \), \( \xi \) (damping ratio) vs. \( \gamma \) and \( \gamma \) vs. \( \tau/\sigma_3 \) were obtained.

The results of cyclic torsional tests are shown in Fig. 31.

6.5. Geotechnical characteristics

The interpretation of the site investigation programme, namely in situ and laboratory tests, has allowed the identification of the following geotechnical units (Oliveira et al., 2006): \( a_0, a_0, a_1, a_2, a_3 \). Table 13 summarizes the geological and geotechnical characteristics of each unit.

A correlation between \( V_s \) and SPT values is shown in Fig. 32, following the proposal of some authors.

6.6. Pile load tests

6.6.1. Introduction

Pile design can be performed by (Eurocode 7, 1997):

- prescriptive measures and comparable experience;
- design models;
-...
use of experimental models and load tests;
• observational method.

The piles of Lezíria bridge were designed by:
i) design models;
ii) pile load tests that have provided useful 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, related settlement and limit load;
ii) to verify the performance of individual piles as well the overall pile foundation;
iii) to assess the suitability of the construction method.

Load tests were carried out on trial piles that were built before the final design.

The results of load tests were used to calibrate the design parameters and so to optimize the preliminary values of pile lengths obtained by the interpretation of site investigation and laboratory and in situ test results.

The criteria to select pile tests has incorporated the following aspects:
• the geotechnical category of the structure;
• the ground condition and the spatial variation;
• past experience related the use of same type of piles in same ground conditions;
• planning of the works.

Due to spatial variation of foundations characteristics and to analyze the embedded effects, 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. Table 14 summarizes pile type and location.

For static tests in each selected place, one 800 mm diameter pile and two 1500 mm diameter reaction piles with were built, 3.5 m apart from the pile test, and, in addition, a fourth 800 mm diameter pile, 5.5 m apart from the first pile, for dynamic test.

To perform pile load tests, seven 1.5 m diameter piles and seven 0.8 m diameter piles were built.

6.6.2. Vertical pile load tests

The static vertical pile load tests have followed the “Axial Pile Loading Test, Suggested Method” protocol 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

![Figure 31 - Curves of shear modulus and damping ratio vs. shear strain (after IST, 2005).](image)

| Material               | $W_{i} (%)$ | $W_{r} (%)$ | $V_{i} (m/s)$ | $V_{r} (m/s)$ | $E_{s0} (MPa)$ | $G_{s0} (MPa)$ | SPT | CPT (MPa) |
|------------------------|-------------|-------------|---------------|---------------|----------------|----------------|-----|-----------|
| a0 - Fine to medium sand| 64          | 38          | 130-160       | 665-1526      | 50-150         | 20-100         | 2-6 | 1-2       |
| a1 - Sandy materials with silty clay | NP-40     | NP-18       | 130-240       | 665-1526      | 100-300        | 30-100         | 2-20| 2-8       |
| a2 - Fine sand with silt | NP         | NP          | 140-300       | 665-1526      | 100-500        | 20-200         | 5-40 | 3-16      |
| a3 - Sandy material with silt | NP         | NP          | 320-400       | 665-1526      | 500-1100       | 200-400        | 40-60|           |
| M - Miocene bedrock    | 400-500     |             | 500-1700      | 200-600       | > 60           |                |     |           |
until a depth of 5 diameters, unless the bedrock was situated at upper level.

Vertical load tests were performed on 3 piles. The following equipment was installed for the vertical pile load test: 2 mechanical dial gauges, electrical displacement transducers (Fig. 33) with removable extensometers (Fig. 34), with a resolution of $10^{-6}$ and anchors, 1 temperature sensor, 1 tilt meter, 1 hydraulically operated pump, 2 hydraulic jacks and 1 optical level.

A general view for vertical pile load tests is presented in Fig. 35.

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 loading and unloading, with a maximum service load for the first cycle and loads were applied in 4 increments.

In the second cycle, the loads were applied in 19 increments. The number of load increments and the cycles of loading and unloading were carefully selected with the purpose of collecting information related to deformation, creep effects and ultimate load. Figure 36 illustrates the load-settlement curves for 3 pile tests.

To define the failure load, a criterion of settlement equal to 10% of the pile diameter, i.e. 80 mm settlement, was adopted.

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**Figure 32** - A correlation between $V_s$ and SPT values.

**Table 14** - Summary of pile type and location.

| Piles (km) | Diameter (m) | Pile embedding | Load test type          |
|------------|--------------|----------------|-------------------------|
| 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)    |

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**Figure 33** - Displacement transducers.

**Figure 34** - Recovery extensometer.
6.6.3. Horizontal pile load tests

The horizontal load tests were performed in two piles of 800 mm and 1500 mm diameter located at km 5 + 400. The maximum load was 600 kN to mobilize a displacement of 8 cm and the loads were applied in steps of 75 kN.

For the horizontal load tests, the following equipment was 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 measured load-displacement curve is illustrated in Fig. 37. The measured loads vs. rotations values are shown in Fig. 38.

Figure 39 shows a comparison between the bending moments values computed from the tests results considering different k values (2500 kPa, 5000 kPa, and 10000 kPa).

6.6.4. Dynamic pile tests

Dynamic pile tests were performed in 9 piles with diameters of 800 mm and 1500 mm. The piles were instrumented with 4 pairs of accelerometers (Fig. 40), 4 transducers and topographic equipment. Figure 41 illustrates a dynamic test view. During the tests, the height of the hammer fall was increased from 0.2 m to 3.0 m in steps of 0.2 m. The mobilized point resistance \( R_p \) and the lateral resistance \( R_s \) values for pile E 800-2 are shown in Fig. 42.

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

6.7. Design surface spectra

6.7.1. Introduction

A very comprehensive analysis was performed to define the design free field surface spectra.
6.7.2. Seismic action

Following the Portuguese Code (RSA, 1983), the seismic action is defined by a stochastic Gaussian stationary vectorial process (two horizontal orthogonal components and one vertical component). For the Portuguese territory, it is important to consider two seismic tectonic sources, namely: (i) a near source that represents a moderate magnitude earthquake at a short epicentral distance with a duration of 10 s; (ii) a distant source that represents a higher magnitude earthquake at a larger epicentral distance with a duration of 30 s.

Five artificial time histories of acceleration were produced for seismic action type 1 and seismic action type 2 and for soil type A for the deterministic approach (IST, 2004). For the computation of these accelerograms, the validation criteria of EC8 (1998a) were considered (Fig. 43).

Based on RSA (1983), power spectral density functions were used for the stochastic approach.

In order to incorporate the spatial variation, taking into consideration the 12 km length of the bridge, 17 geotechnical profiles were selected to incorporate the variation of the geological and geotechnical characteristics.

Only the results obtained for the profile located between km 1 + 500 and km 1 + 800, where the main bridge is located, are presented, due to space limitations.

Figures 44 and 45 illustrate the results of the response spectra (IST, 2004), as well as the shear stress computed by the SHAKE 2000 code. The analyses were performed for seismic action type 1 and seismic action type 2, considering for the bedrock type A ground.
6.8. Liquefaction assessment

Following Eurocode 8 - Part 5 - 4.1.3. (2) (1998b), “An evaluation of the liquefaction susceptibility shall be made when the foundation 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 $\tau_s$ can be estimated from the simplified expression:

$$\tau_s = 0.65 \alpha_p \gamma_i S \sigma_{vo}$$  \hspace{1cm} (1)
NCEER (1997) factors. A comparison between the two proposals is shown in Table 15 and Fig. 46.

Cetin et al. (2001) have presented a new proposal for liquefaction analysis (see Fig. 47). It is considered advanced in relation to the previous ones, as it integrates: (i) data of recent earthquakes; (ii) corrections due to the existence of fines; (iii) experience that incorporates a better interpretation of SPT test; (iv) local effects; (v) cases histories that incorporate lessons of more than 200 earthquakes; (v) Bayesian theory.

For liquefaction evaluation of sandy materials, two methods are used, namely, based in laboratory tests or field tests. In general, the following laboratory tests are used: (i) cyclic triaxial tests; (ii) cyclic simple shear tests; and (iii) cyclic torsional shear tests. Due to the difficulties to obtain high quality undisturbed samples, in general field tests are used: SPT, CPT, seismic cone tests, flat dilatometer tests and tests to assess electrical properties (Sêco e Pinto, 1997).

For liquefaction assessment by shear wave velocity, 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).

To overcome the difficulties of CPT and SPT tests in soils with gravel, some proposals to evaluate the susceptibility of liquefaction of these materials based in seismic tests with measurement of shear waves velocities $V_s$ were made (Stokoe et al., 1999).

Following Youd & Gilstrap (1999), the post-liquefaction strength of silty materials is less than that of sandy materials, but superficial silty materials with moderate density are dilatant and with higher strength than clean sands. The authors have concluded that loose soils with $IP < 12$ and $w/w_i > 0.85$ are susceptible to liquefy and loose soils with $12 < IP < 20$ and $w/w_i > 0.85$ 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 0.15 g in addition with one of the following conditions: (i) sands with a clay content higher than

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**Table 15 - 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             |
| 8.0         | 0.94                 | 0.84         | 0.67             |
| 8.5         | 0.89                 | 0.72         | 0.44             |

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**Figure 47 - Probabilistic approach for liquefaction analysis (after Cetin et al., 2001).**
20% and a plasticity index $> 10$; (ii) sands with silt content higher than 10% and $N_1(60) > 20$; and (iii) clean sands with $N_1(60) > 25$.

6.8.1. Settlement assessment

The susceptibility of foundation soils to densification and to exhibit excessive settlement is referred in EC8, but the assessment of expected liquefaction induced deformation is not adequately treated.

By combination of cyclic shear stress ratio and normalized SPT N-values, Tokimatsu & Seed (1987) have proposed relationships with shear strain (Fig. 48).

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, Ishihara (1993) has proposed a chart (Fig. 49).

6.8.2. Remedial measures

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) prevention of liquefaction; and (ii) reduction of damage to facilities due to liquefaction.

The measures to prevent occurrence of liquefaction include the improvement of soil properties or improvement of conditions for stress, deformation and pore water pres-

6.9. Liquefaction evaluation

Due to the disturbance that occurs during sampling of sandy materials, the liquefaction potential evaluation was performed only by field tests.

Attention was drawn to SPT and CPT tests as the seismic tests have only been used when the soil contains gravel particles.

To compute the shear values a total stress model, the code “SHAKE 2000”, was used and the obtained results are on the conservative side.

Just as an example, Fig. 50 compares the results obtained by the total stress model and the effective stress analysis using the computer program DYNFLOW for the Vasco da Gama bridge in the Tagus river and with the same type of alluvial materials.

Corrections of SPT test results due to the depth effect and the equipment were performed, following the recommendations of EC8 (1998b).
The sieve curves of materials a₁ and a₂ are shown in Figs. 51 and 52.

Taking into account that we are dealing with underwater materials, the sieve curves exhibit percentages of fines lower than the reality, as a consequence of the washing effect during 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 Nₘₜ (SPT) and (q_c)ₘₜ (CPT); (iii) columns 7 and 8, effective overburden pressure (σ_v'₀) and correction factor (C_N); (iv) columns 9 and 10, normalized values N₁ (60) (SPT) (for effects reduced C_N values were considered) and (q_c₁) (CPT); (v) column 11, τₑₗₑₗₑₗₑ (equivalent shear stress value computed for action type 2 related with the highest magnitude 7.5); (vi) column 12 (τ/σ_v'₀ ratio value); (vii) column 13 (τ/σ_v'₀ ratio value with a safety factor of 1.1), column 14 (τ/σ_v'₀ ratio value with a safety factor of 1.25); (viii) column 15, Ref. (reference of the analyzed SPT or CPT value); (ix) 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 pressure rarely reach 80 %, being frequently below 60 %, a safety factor of 1.1 can be considered sufficient. Nevertheless, in the present case, a conservative analysis was performed, with a safety factor of 1.25 being adopted, as recommended in EC8 - Part 5 (1998b).

Table 16 presents an application of liquefaction evaluation for materials a₁ and a₂. The liquefaction potential evaluation, by SPT and CPT tests, is shown in Figs. 53 and 54.

Taking into account Figs. 48 and 49, the estimated settlement of materials a₁ and a₂ are between 40 mm to 150 mm.

6.10. 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 ensure stability during the driving of the casings. The system had the purpose of ensuring 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 $a_0$ was ensured.

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 ton was necessary for 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 SPT value of 10 blows.

iii) Progress of the excavation with a 2.2 m diameter “hammergrab” in order to reach the Miocene. For wall stabilization were used polymer materials manufactured in a central located in the left bank. For polymer control, pH tests, density and viscosity tests, as well sand content tests, were performed.

iv) After the excavation and the decantation of the polymer, the reinforcement with the pipes for the cross-hole tests was installed. To ensure a minimum cover of 12 mm centralizers were placed.

v) Concreting of the piles with the use of “tremie” and pumping was done at a rate of 50 m$^3$/h.

The duration of these 5 phases was 2.5 days.

In the construction procedure proposed in the Basic Design, the pile caps for piers P1 and P2 were performed within cofferdams constructed with sheet piles driven into the mud materials by equipment installed in barges. The voids under the casings were stabilized through the use of polymers.

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

A view of the North Viaduct construction is shown in Fig. 55.

To avoid excavations of the protection dykes, a parallel way (transient viaduct) was built (Fig. 56).

A view of the South Viaduct construction is shown in Fig. 57.

The placement of pile casing is shown in Fig. 58.

The placement of pile reinforcement and tremie pipes are shown in Figs. 59 and 60.

In Figs. 61 to 63 a caisson view, a pier under construction and a general view of the construction works are presented.

The pre-fabricated caissons were temporarily 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.
Table 16 - Evaluation of liquefaction potential (material a1 and material a2).

| Pier or CPT | Depth (m) | Thickness (m) | N60 | (q)c (MPa) | σ0' (kPa) | Cn | N60 (60) | (q)c1 (MPa) | τs(eqv) (kPa) | τ/σ0' x 1.1 | τ/σ0' x 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       | 7          | 6              | 19.2        | 0.29        | 0.36  | A1      | L      |
| S3B-2       | 4.2-7.4   | 3.2          | 6   | 0.52       | 66.4      | 1.2 | 7       | 6          | 0.6           | 19.2        | 0.29        | 0.36  | A2      | L      |
| S3B-3       | 7.4-9.6   | 2.2          | 12  | 0.65       | 89.4      | 1.1 | 13      | 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        | 0.32  | A2      | L      |
| S4B-2       | 3.6-6.2   | 2.6          | 3   | 0.52       | 58.5      | 1.0 | 3       | 0.5        | 16.5          | 0.28        | 0.35        | 0.41  | 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        | 0.51  | A2      | L      |
| S5B-2       | 26.0-28.8 | 2.8          | 31  | -          | 191.1     | 0.7 | 22      | -          | 55.1          | 0.29        | 0.36        | 0.51  | A2      | N.L    |
| S6B-1       | 0.5-4.2   | 5.4          | 2   | 0.5        | 24.3      | 1.0 | 2       | 0.5        | 9.7           | 0.40        | 0.50        | 0.50  | A2      | L      |
| S6B-2       | 24.1-25.0 | 0.9          | 5   | -          | 164.2     | 0.8 | 4       | -          | 48.7          | 0.30        | 0.37        | 0.41  | A2      | L      |
| S6B-3       | 25.0-29.2 | 4.2          | 17  | -          | 188.1     | 0.7 | 12      | -          | 54.4          | 0.29        | 0.36        | 0.36  | A2      | L      |

N60: SPT value, N60(60): Normalized SPT value, (qc): CPT cone resistance value, (qc)1: Normalized CPT cone resistance, σ0': Effective overburden pressure, τs(eqv): Equivalent cyclic shear stress, Cn: Correction factor for overburden pressure, L: Liquefaction, N.L: No Liquefaction.
Figure 55 - Construction of North Viaduct.

Figure 56 - Parallel Way.

Figure 57 - A view of South Viaduct construction.

Figure 58 - Placement of pile casing (courtesy Ferreira et al., 2008).

Figure 59 - Placement of pile reinforcement (courtesy Ferreira et al., 2008).
7. Conclusions

The following conclusions can be outlined:

For the Vasco de Gama bridge
1) For the pile foundations, each geotechnical design situation shall be verified that no relevant limit state is exceeded.

2) For the verification of limit states, one or a combination of the following methods can be used: design by prescriptive measures, design by calculation, design by loads tests and experimental models and observational method.

3) For design purposes and for a better knowledge of pile behaviour, it is recommended to perform field tests with instrumented piles.

4) Field load tests performed in the New Tagus bridge and Lezíria bridge for design purposes have shown their importance to calibrate the design parameters, to check the performance of individual piles, to assess the overall pile foundation behavior and to analyze the suitability of the construction method.

5) For pile quality control, the use of non-destructive of pile test techniques is recommended.

6) Monitoring during construction and in the long term is important to assess bridge behavior.
For the Lezíria bridge
7) The geotechnical campaigns implemented during the Preliminary Study and Basic Design have allowed the definition of the geological and geotechnical model.
8) The geotechnical characteristics were obtained by a combination of field and laboratory test results.
9) The geotechnical study in the Basic Design was performed respecting 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).
10) As the Lezíria bridge is located in zone A of Portugal seismic map, the seismic studies are important.
11) For pile design were used: i) design models; ii) pile load tests that have contributed for the characterization of gravel materials and techniques for driving the metallic casings; and iii) comparable experience.
12) To calibrate the design parameters and to optimize the pile length, static pile load tests (both vertical and horizontal) were carried out on trial piles. In addition, dynamic pile tests were performed for seismic design.
13) As sampling of sandy materials is always difficult, the liquefaction potential evaluation was based in in situ tests, namely CPT and SPT. The computation of shear stress in total and effective stress analyses was performed.

8. Lessons for tomorrow

Due to the complexity of bridge analysis, there is a need to work with multidisciplinary teams exploring the huge capacity of computers. Innovative methods and new solutions require highly reliable information and multidisciplinary teams integrating seismologists, geologists, geophysicists, geotechnical engineers and structural engineers.

The success of this challenge requires the joint effort of Owners, Decision Makers, Researchers, Consultants, Professors, Contractors and General Public.

The understanding of the vulnerability and resilience concepts is crucial. Vulnerability is associated with two dimensions, one is the degree of loss or the potential loss and the second integrates the range of opportunities that people face in recovery. This concept has received a great attention from Rousseau and Kant (1756). Resilience is a measure of the system’s capacity to absorb recovery from a hazardous event. It includes the speed with which a system returns to its original state after a perturbation. The capacity and opportunity to relocate or to change are also key dimensions of disaster resilience. The purpose of assessing resilience is to understand how a disaster can disturb a social system and the factors that can disturb the recovery and to improve it.

The education of engineers and Public with scientific methods for evaluating risks incorporating the unpredictable human behavior and human errors is crucial for disaster reduction.

The analysis of past bridge incidents and accidents occurred during earthquakes have shown that all the lessons have not deserved total consideration, in order to avoid repeating the same mistakes. We need to enhance a global conscience and to develop a sustainable strategy of global compensation to better serve our Society. The recognition of a better planning, early warning, that we should take for extreme events which will hit our civilization in the future, is important. Plato (428-348 BC) in the Timaeus stressed that destructive events that happened in the past can happen again, sometimes with large time intervals between and for prevention and protection we should follow the Egyptians example and preserve knowledge through writing.

We should never forget the 7 Pillars of Engineering Wisdom: Practice, Precedents, Principles, Prudence, Perspicacity, Professionalism and Prediction. Following Thomas Mann we should enjoy the activities during the day, but only by performing those will allow us to sleep at the night.

Efforts should be done to narrow the gap between university education and professional practice, but we should not forget that Theory without Practice is a Waste, but Practice without Theory is a Trap. Kant has stated that Nothing better than a good theory, but, following Seneca, Long is the way through the courses, but short through the example. I will add through a careful analysis of Case Histories.

In dealing with this subject we should always have in mind:

All for Love
“Errors, like straws, upon the surface flow;
He who would search for pearls must dive below”.
(John Dryden)

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