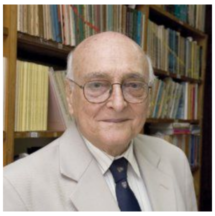

Prof. Victor de Mello acted as President of the International Society for Soil Mechanics and Foundations Engineering during the tenure 1981-1985 and will be remembered for his actions and passion to implement geotechnical activities worldwide.

Professor Victor de Mello is a man of prodigious energy and fine intellect. We are indebted for his outstanding contribution for the advancement of knowledge in soil mechanics and geotechnical engineering.

A genial thinker, Victor de Mello was one the bright talents that have enlighten 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 teach me to think, to investigate, to be in Geotechnique and which friendship is for me a great lesson.

Professor Victor de Mello is often invited to be Keynote Speaker at international conferences of geotechnical engineering and other events and we always listen to his lectures with great interest and pleasure, as they are challenge and open new avenues of research.

I would like to highlight from Prof. Victor de Mello outstanding curriculum:i) his solid scientific background and research contributions for 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.

We are indebted for his outstanding contribution for the advancement of knowledge in soil mechanics and geotechnical engineering and his legacy will maintain for many generations and will always be a source of great inspiration for all geotechnical engineers.

Victor de Mello has oriented his existence for a great and noble ideal and has always teached 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 in one soul, where the beauty and the truth give friendlyI believe that everybody fully agree with me in classifying his activity with Five Es - 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 simple contribution of the current and next generations to honor Victor de Mello memory.

Static and Seismic Pile Foundations Design by Load Tests and Experimental Models

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ABSTRACT: In this paper the Static and Seismic Pile Foundations Design by Load Tests and Experimental Models and soil-structure interaction are referred. The Eurocode 7 - Geotechnical Design and the Eurocode 8- Design of Structures for Earthquake Resistance are introduced. The ultimate limit states and the serviceability limit states are discussed. The potentially liquefiable soils and remedial measures are addressed. Two case histories related with pile design of New Tagus bridge and the pile design and liquefaction potential evaluation of Leziria bridge foundations are presented. Some conclusions are drawn.

1. INTRODUCTION

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

From 1975 to 1989 the Commission with the help of a Steering Committee with the Representatives of 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.

The Structural Eurocode programme comprises the following standards:

EN 1990 Eurocode – Basis of 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 alloy 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. Nevertheless, due to the preparation of these documents by several experts, some provisions of EC8 with the special requirements for seismic geotechnical design that deserve more consideration will be presented in order to clarify several questions that still remain without answer.

The actual 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 of factor was substituted by the partial safety factors applied to actions and to the strength of materials.

In this lecture a summary of the main topics covered by Eurocode 7 and the interplay with Eurocode 8 and also the identification of some topics that need further implementation is addressed.

In dealing with these topics we should never forget the memorable lines of Lao- Tsze, 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. The link between the design requirements in Part 1 and the results of laboratory tests and field investigations run according to standards, codes and other accepted documents is 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 dealt with 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).

Each geotechnical design situation 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 loads tests and experimental models and 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; (iii) or 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 this approach, 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 analysing 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 ground water 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.

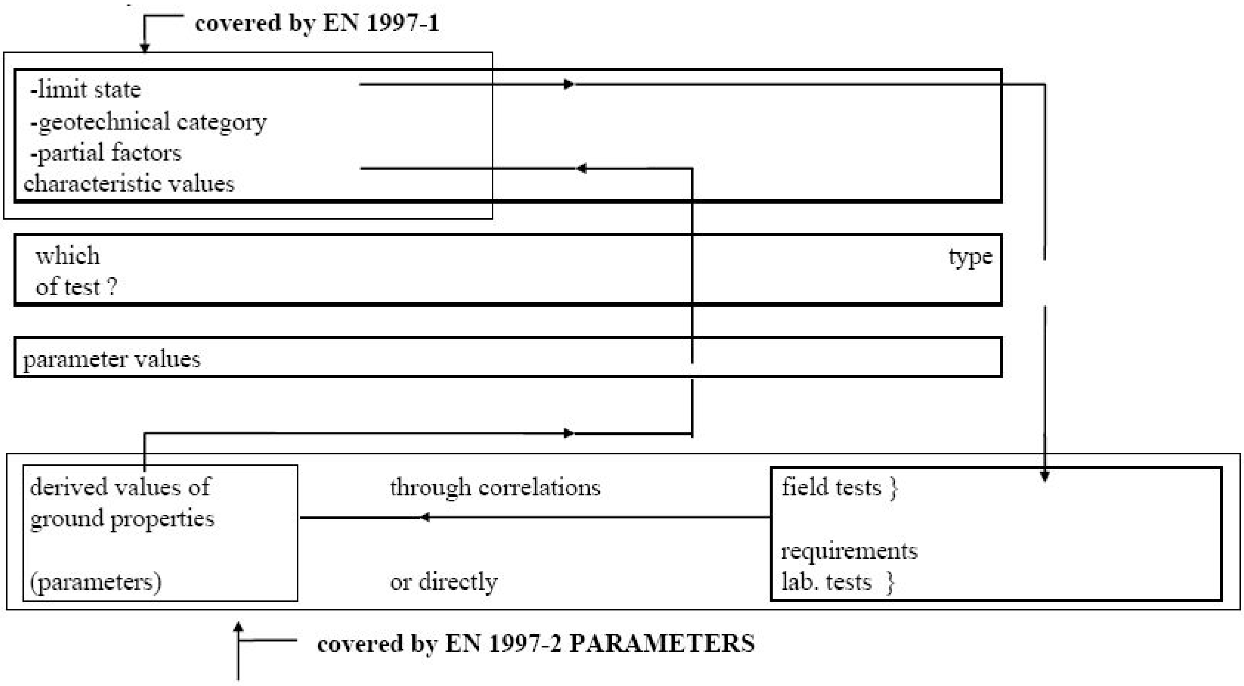

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Figure. 1. Flow chart that demonstrates the link between design and field and laboratory tests

3. EUROCODE 8 - DESIGN of STRUCTURES for EARTHQUAKE RESISTANCE

3.1. Introduction

The Eurocode 8 (EC8) "Design of Structures for Earthquake Resistant" deals with the design and construction of buildings and civil engineering works in seismic regions is divided in six Parts.

The Part 1 is divided in 10 sections:
Section 1 - contains general information;
Section 2 - contains the basis 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 steel-concrete composite 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 relevant provisions to bridges.
Part 3 presents provisions for the seismic strengthening and repair of existing buildings.
Part 4 gives specific provisions relevant to tanks, silos and pipelines.

Part 5 contains specific provisions relevant to foundations, retaining structures and geotechnical aspects.
Part 6 presents specific provisions relevant to towers, masts and chimneys.
In particular the Part 5 of EC8 establishes the requirements, criteria, and rules for siting and foundation soil and complements the rules of Eurocode 7, which do not cover the special requirements of seismic design.

The topics covered by Part 1- Section 1 namely: seismic action, ground conditions and soil investigations, importance categories, importance factors and geotechnical categories and also the topics treated in Part 5 slope stability, potentially liquefiable soils, earth retaining structures, foundation system, topographic aspects are discussed.

4. SEISMIC ACTION

The definition of the actions (with the exception of seismic actions) and their combinations is treated in Eurocode 1 "Action 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.

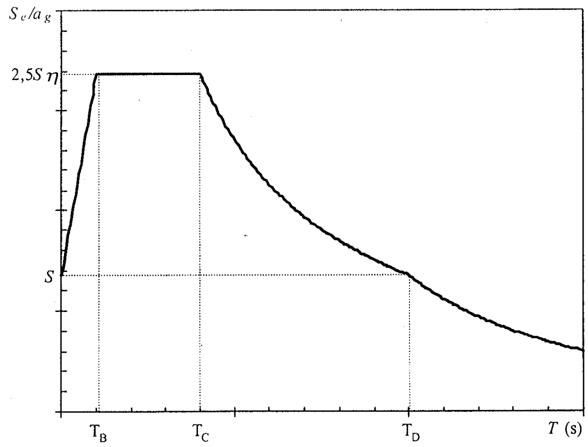

Figure 2. Elastic response spectrum (after EC8)

In EC 8, in general, the hazard is described in terms of a single parameter, i.e. the value a_g of the effective peak ground acceleration in rock or firm soil called "design ground acceleration"(Figure 2) expressed in terms of: a) the reference seismic action associated with a probability of exceeding (P_{NCR}) of 10 % in 50 years; or b) a reference return period (T_{NCR})= 475.
where:

Se (T) elastic response spectrum,

T vibration period of a linear single-degree-of-freedom system,

α_g 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 spectrum
S soil parameter with reference value 1.0 for subsoil class A,
η damping correction factor with reference value 1.0 for 5 % viscous damping

These recommended values may be changed by the National Annex of each country (e.g. in UBC (1997) the annual probability of exceedance is 2% in 50 years, or an annual probability of 1/2475).

It is recommended the use of two types of spectra: type 1 if the earthquake has a surface wave magnitude Ms 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.

5. GROUND CONDITIONS AND SOIL INVESTIGATIONS

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

Subsoil class A – rock or other geological formation, including at most 5 m of weaker material at the surface characterised by a shear wave velocity V_s of at least 800 m/s;

Subsoil class B – deposits of very dense sand, gravel or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanics properties with depth shear wave velocity between 360-800 m/s, N_{SPT}>50 blows and c_u>250 kPa.

Subsoil class C – deep deposits of dense or medium dense sand, gravel or stiff clays with thickness from several tens to many hundreds of meters characterised by a shear wave velocity from 160 m/s to 360 m/s, N_{SPT} from 15-50 blows and c_u from 70 to 250 kPa.

Subsoil class D – deposits to loose to medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft to firm cohesive soil characterised by a shear wave velocity less than 180 m/s, N_{SPT} less than 15 and c_u less than 70 kPa.

Subsoil class E – a soil profile consisting of a surface alluvium layer with Vs,30 values of type C or D and thickness varying between about 5m and 20m, underlain by stiffer material with Vs,30>800m/s.

Subsoil S_1 – deposits consisting - or containing a layer at least 10 m thick - of soft clays/silts with high plasticity index (PI>40) and high water content characterised by a shear wave velocity less than 100 m/s and c_u between 10-20 kPa.

Subsoil S_2 – deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A-E or S_1.

For the five ground types 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 Figures 3 and 4.

Table 1. Values of the parameters describing the Type 1 elastic response spectrum*

Ground type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Table 2. Values of the parameters describing the Type 2 elastic response spectrum*

Ground type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

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 S1 and S2.

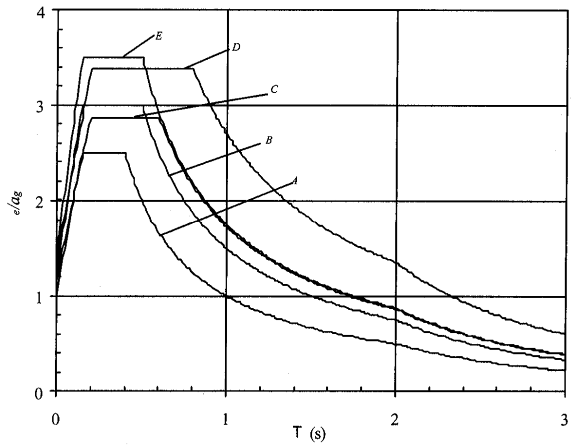

Figure 3. Recommended Type 1 elastic response spectrum (after EC8)

Table 3. Recommended values of the parameters for the five ground types A, B, C, D and E

Spectrum	α_{vg}/α_g	T_B(s)	T_C(s)	T_D(s)
Type 1	0.9	0.05	0.15	1.0
Type 2	0.45	0.05	0.15	1.0

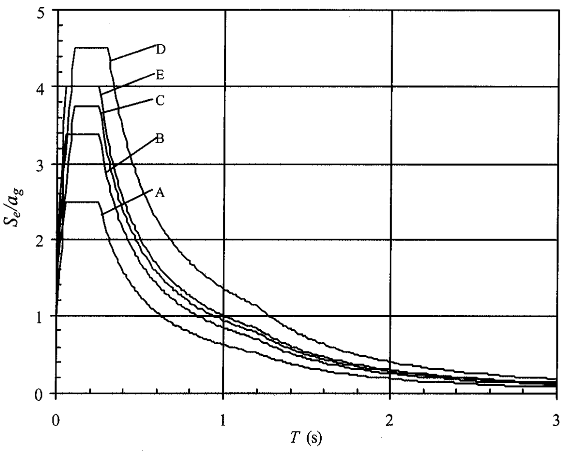

Figure 4. Recommended Type 2 elastic response spectrum (after EC8)

6. FOUNDATION DESIGN

6.1. Introduction

The foundation system with particularly emphasis to soil-structure interaction is analysed. The serviceability limit states are introduced. The liquefaction assessment of sandy, silty sandy materials is discussed. The remediation techniques are addressed. Two case histories related with the foundations design of New Tagus bridge and the liquefaction potential assessment of Leziria bridge foundation are presented.

For the pile foundation each geotechnical design situation 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 loads tests and experimental models and observational method (Eurocode 7, 1997)

For the pile design the following limit states shall be considered (Eurocode 7, 1997):

(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 ground and in the pile foundation; (vii) - combined failure in ground and in the structure; (viii) - excessive settlement; (ix) - excessive heave; (x) - excessive lateral movement of the ground; and (xi) - unacceptable vibrations.

In general 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.

There is strong evidence that 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 subsoil profile is of class D, S_1 or S_2, and contains consecutive layers with sharply differing stiffness; (ii) the zone is of moderate or high seismicity, α>0.10; (iii) the supported structure is of important category 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 the propagation of seismic waves (Figure 5). The decomposition of the problem in steps is shown in Figure 5 (Gazetas and Mylonakis, 1998). The complete solution is a 3D analysis very time consuming.

For the computation of internal forces along the pile, as well as the deflection and rotation at the pile head, both discrete (based in Winkler Spring model) or continuum models can be used.

The lateral resistance of soil layers susceptible to liquefaction shall be neglected.

In general the linear behaviour is assumed for the soil.

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.

6.2. SERVICEABILITY LIMIT STATES

The foundation movements shall not reach certain limit values to avoid the occurrence of an ultimate limit state or a serviceability limit state (Seco e Pinto & Sousa Coutinho, 1991).

Burland and Wroth (1974) proposed a consistent set of definitions based on the displacements, that are illustrated in Figure 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);

- deflection rate Δ/L, where L is the distance between the two reference points defining Δ;

- tilt (w) describes the rigid body rotation;

- relative rotation ($\Re$) – is the rotation of the line joining two points;

.

- average horizontal strain (e_n) is defined as the change of length δ_L over the length L.

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

The lateral resistance of soil layers susceptible to liquefaction shall be neglected.

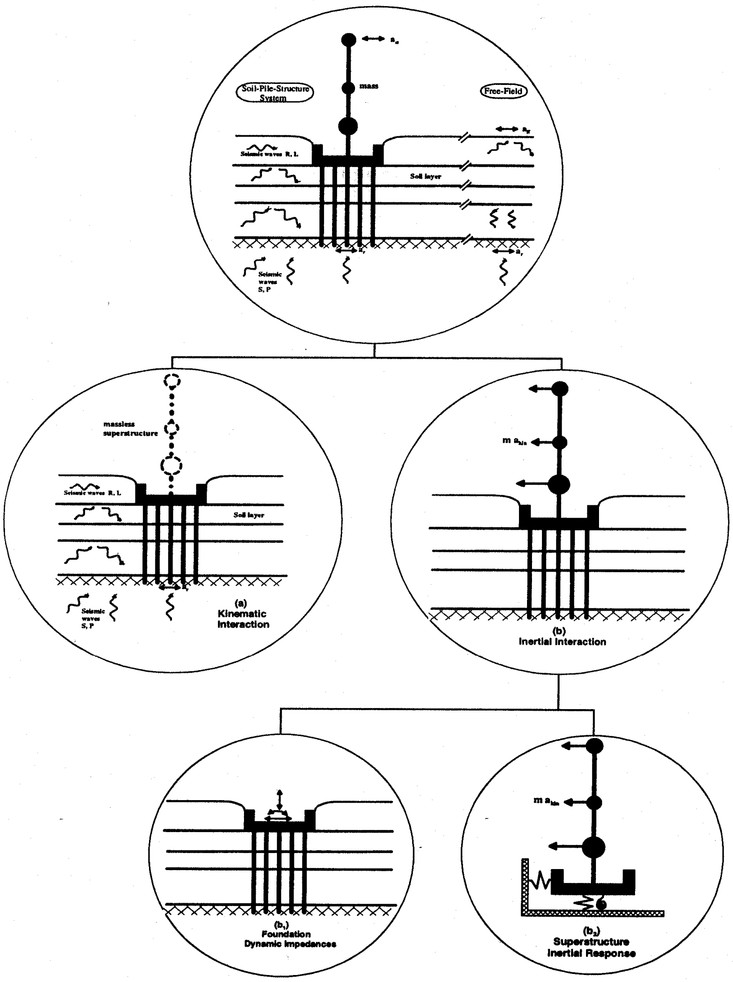

Figure 5. Soil-structure interaction problem (after Gazetas and Mylonakis, 1998)

(i) - the confidence with which the acceptable value of the movement can be specified; (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.

Table 1 presents a summary of allowable deformations proposed by different authors and EC7.

The allowable displacements for shallow foundations and raft foundations for sand and clay materials are summarised in Tables 4 to 6.

EC7 recommends that settlements for pile foundations for ultimate limit of states and serviceability limit of states shall include:

-the settlement of a single pile;

-the additional settlement due to group of action.

Bozozuk (1981), based on a study related with the allowable displacements in foundation bridges piles after the observation of 150 cases has proposed the limits for vertical settlement S_v and horizontal S_H defined in Table 7.

A recent study performed by Moulton (1986) based on 314 bridges located in United States and Canada has confirmed the proposal of Bozozuk (1981).

Burland et al (1977) have proposed 6 categories for damages in buildings related with Table 8 where categories 0, 1 and 2 are related with stetic damages, categories 3 and 4 are related with serviceability limit of states and category 5 with ultimate limit of states (stability).

Burland et al (1977) the concept of limit tension deformation e_{lim} to define the ultimate limit state.

Boscardin and Cording (1989) develop the concept of different levels of strain and have proposed, based in the analysis of 17 cases, the Table 9 to establish the relationship between category of damage and limiting tensile strain.

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

For preliminary evaluation of buildings a value of θ less than 1/500 or settlement less than 10 mm are considered of degree of severity negligible.

For buildings of damage category 3 or higher a detailed evaluation shall be performed.

A limit value for deformation is related with the occurrence of an ultimate limit state or of a serviceability limit state.

The differential settlements and relative rotations of foundations shall be established in order to avoid the occurrence of an ultimate limit state or a serviceability limit state (like cracking).

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

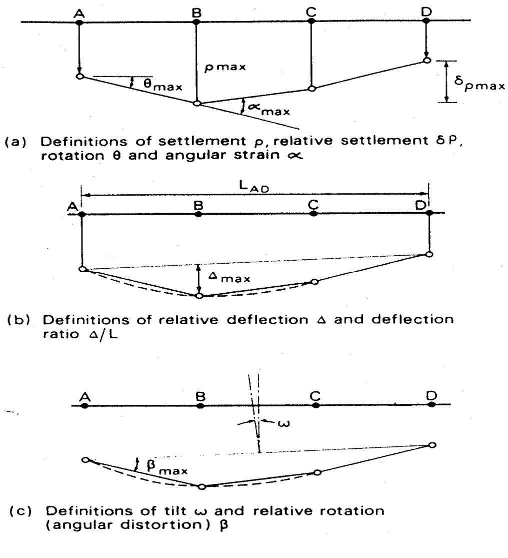

Figure 6. Definition of foundation movements (after Burland and Wroth, 1974)

Table 4. Allowable deformations

A – Concrete buildings and reinforced walls					
Allowab. values for rotations	Skempton and MacDonald (1956)	Meyerhof (1956)	Polshin et Tokar (1957)	Bjerrum (1963)	EC7 (1994)
Structure Damages & cracks on walls	1/150 1/300	1/250 1/500	1/200 1/500	1/150 1/500	1/150 1/300
B – Wall without reinforcement					
Deflect. ratio • /L	Meyerhof (1956)	Polshin e Tokar (1957)		Burland e Wroth (1975)	
Deform. ∪	1/2500	L/H < 3 1/3500 to 1/2500; L/H > 5 1/2000 to 1/1500		1/2500 L/H = 1 1/1250 L/H = 5	
Deform. ∩	--	--		1/5000 L/H = 1 1/2500 L/H = 5	

Table 5. Allowable settlements (in mm) for shallow foundations

Allowable values for isolated foundations	Burland et al. (1977)	Skempton e MacDonald (1956)	EC7 (1994)
Total settlements in sands. Differential settlements in sands	25 20	40 25	50 20
Total settlements in clays. Differential settlements in clays	45 25	65 40	High values

Table 6. Allowable values for raft foundations settlements (mm)

Allowable values for raft foundations settlements	Terzaghi and Peck (1948)	Skempton and MacDonald (1956)	Burland et al (1977)	EC7
Sandy soils	50	40 to 60	High values	50
Clay soils		65 to 100		

Table 7. Allowable values for bridge foundations

Damage classification	Limit values
Allowable or acceptable	$S_V < 50$ mm $S_H < 25$ mm
With acceptable damages	50 mm = S_V = 100 mm 25 mm = S_H = 50 mm
Non acceptable	$S_V < 100$ mm $S_H > 50$ mm

Table 8. Damages categories in buildings (after Burland et al, 1977)

Damage category	Degree of severity	Description of damage
0	Negligible	Hairline cracks 0,1 mm
1	Very light	Fine cracks ,easily treated
2	Light	Cracks easily filled
3	Moderate	Cracks required some opening
4	Severe	Extensive repair working involving breaking and replacement
5	Very Severe	Major repair involving partial or complete rebuilding

Table 9. Categories of damages in buildings (after Boscarding and Cording, 1989)

Category of d	Degree of severity	Limiting tensile
0	Negligible	0 - 0.05
1	Very slight	0.05 - 0.075
2	Slight	0.075 . 0.15
3	Moderate	0.15 - 0.3
4 to 5	Severe to very seve	>0.3

7. NEW TAGUS BRIDGE

7.1. Introduction

The 18 km Tagus bridge Lisbon, capital of Portugal, is composed by a number of structures. From the Sacavém interchange the estuary traffic will cross the north viaduct, the Expo viaduct, the cable stayed bridge, the central viaduct and, finally, the south viaduct (Figure 7).

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

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

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

Driven piles were installed by large barge mounted cranes were used to drive each pile as one piece. A handling capacity around 58 t was necessary by the cranes and the hammer to drive the piles into position.

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

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

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

7.2. Main geological conditions

Taking into account the geological data obtained from two site investigation programmes (TEJOPROJECTO (1993a), the ground is composed by the following two main units (Figure 7): a) Alluvial deposits (Al), aged Holocene and Pleistocene; b) The bedrock under alluvial deposits, formed by Plio-Pleistocene materials.

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

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

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

In the following paragraphs will be presented the general description of each type of the differentiated alluvial deposits (Oliveira et al, 1997):

a_0 This unit is formed by silty to very silty clay (mud), dark grey, with maximum thickness around 35 m.

a_1 Fine to medium sand with shells and shell fragments.

a_{2a} Silty clay to clayey silt.

a_{2b} Yellowish brown to grey medium to coarse (occasionally fine) sand, .

a_3 Fine to coarse gravel, rounded to angular, with sand, cobbles and occasionally some boulders,

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

7. 3. Pile load tests

7.3.1. Introduction

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 settlement and limit load;

ii) to check the performance of individual piles and to allow judgement of the overall pile foundation;

iii) to assess the suitability of the construction method.

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

The results of load tests should be used to calibrate the design parameters and so to optimize the suggested values for pile lengths, based only on the interpretation of site investigation and laboratory and in situ test results (Sêco e Pinto and Oliveira, 1998).

7.3.2. 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 had the following steps:

(i) installation by vibrodriving with a SOILMECH VTE 12000 of a permanent casing with an outside diameter: 1216 mm, a thickness of 8 mm and 16 mm at the shoe level and a length of 40 m;

(ii) excavation of the soil inside the casing with a bucket of 1180 mm diameter and a SOILMECH rotary machine RT - 3ST;

(iii) boring below the bottom of the casing for a length higher than 19 m with a bucket using a polymeric drilling fluid GEOMUD - 15 mixed with salty Tagus water with the following composition: 2 kg of polymer per 1000 l of water. The mixture had a Marsh viscosity 40” and a density 1.035.

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

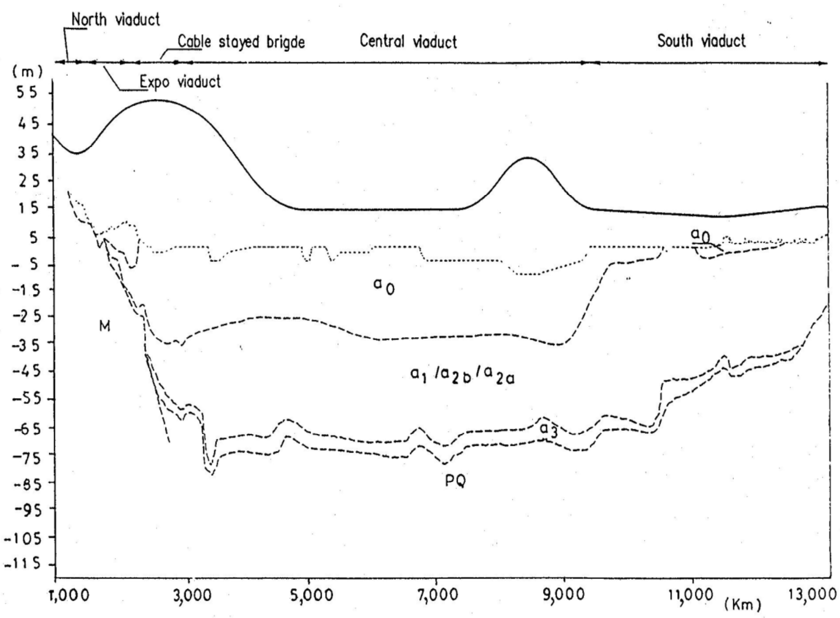

Figure 7. Simplified geotechnical profile

For the vertical load test the following equipments were installed: 8 electrical displacement transducers, 2 mechanical dial gauges, 2 strips of LCPC removable extensometers, with a resolution of 10^{-6}, 1 temperature sensor, 1 high precision pressure transducer, 1 hydraulically operated pump, 4 hydraulic jacks and 1 optical level .

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

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

The load - settlement curves for piles P8, P31 and P 79 are shown in Figure 9.

Failure loads were defined as settlement equal to 10% of the pile diameter, i.e. at 120 mm settlement. Table 10 gives the values of predicted failure loads based from CPT tests and the observed values.

The latter are lower than the predicted loads, with the exception of P79 (the length of this pile was increased 10 m) and the difference were attributed to the lower shaft friction values. The effect of grouting on the soil gave insufficient gain in bearing capacity, as can be assessed by P31i.

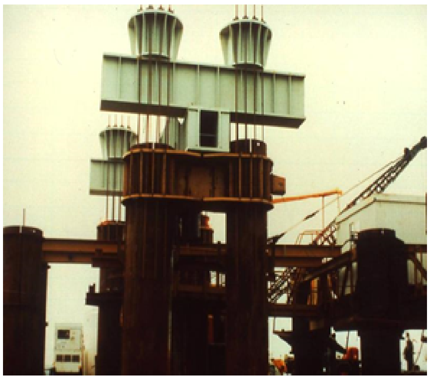

Figure 8. General view for vertical pile load tests

Table 10. Failure loads

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

m – measured p – predicted loads in MN

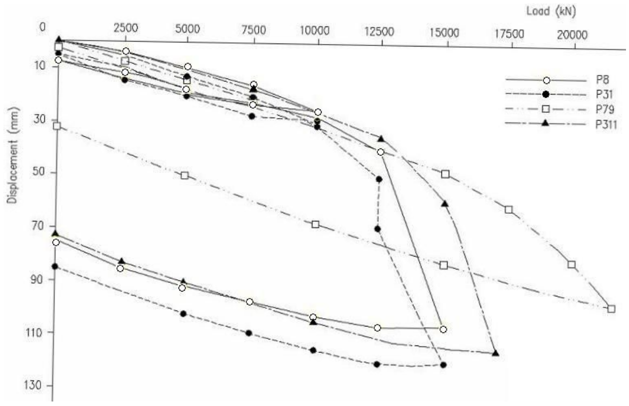

Figure 9. Load settlement curves for vertical test

Figure 10. General view for horizontal pile load tests

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These equipments were placed in several points in order to monitoring the horizontal and vertical displacements.(Figure 14).

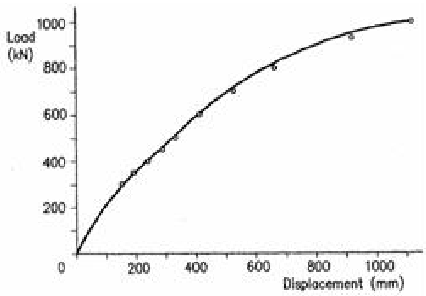


Figure 11. Measured load displacement curve for horizontal tests

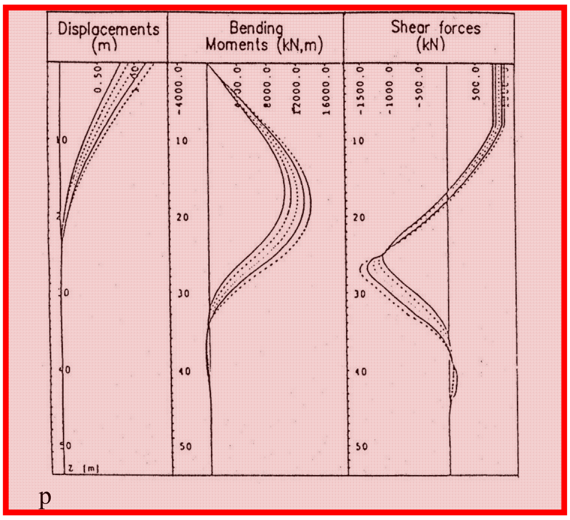


Figure 12. Computed values for pile displacements, bending moments and shear forces

Figure 13. General view of the shaker

Figure 14. General view of the velocity transducers

7.3.3. Horizontal pile load tests

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

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

For the horizontal load tests the following equipments were installed: (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.

A general view for horizontal pile load tests is shown in Figure 10.

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

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

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

7.3.4. Dynamic pile tests

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

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

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

In the dynamic test a shaker(Figure 13) built in LNEC was used to impose on the pile cap, harmonic horizontal loads, with different amplitudes and frequencies (LNEC, 1995 b).

The excitation frequencies were applied in steps of 0.1 Hz in the range from 0.5 to 20 Hz approximately. The dynamic response of the structure, for the various frequencies of excitation, was measured by means of velocity transducers and accelerometers.

These equipments were placed in several points in order to monitoring the horizontal and vertical displacements.(Figure 14)

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

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

The configuration of the two first modes of vibration and respective frequencies (observed and computed) is presented in Figure. 15. The first vibration mode corresponds to the bending of both piles following a direction perpendicular to the vertical plan that encloses both of them. The second mode corresponds to the bending of both piles in the vertical plan that contains them.

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

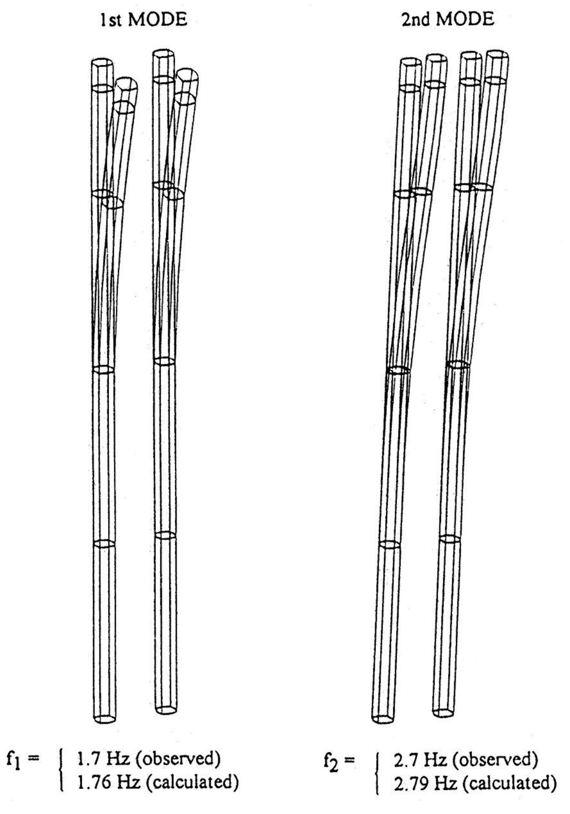


Figure 15. Configuration of the two first vibration modes. Observed and computed frequencies (adopted from Oliveira et. al., 1996).

Table 11. Modal damping values adopted in the mathematical model

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

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

Table 12. Frequencies of the first vibration modes

Vibration Modes	1	2	3	4
Observed Frequencies	1.7	2.7	-	-
Calculated Frequencies	1.76	2.29	8.78	11.70

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

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

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

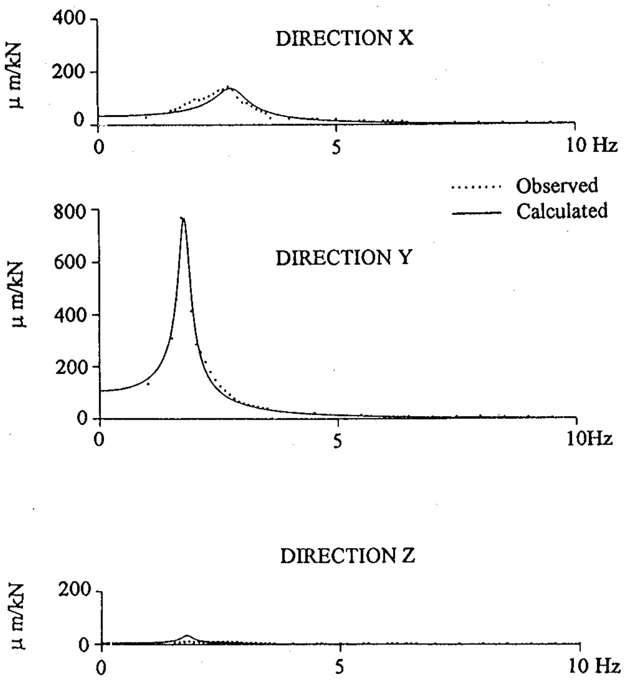


Figure. 16. Displacement transfer functions. Comparison between computed and observed values (adopted from Oliveira et al., 1996)

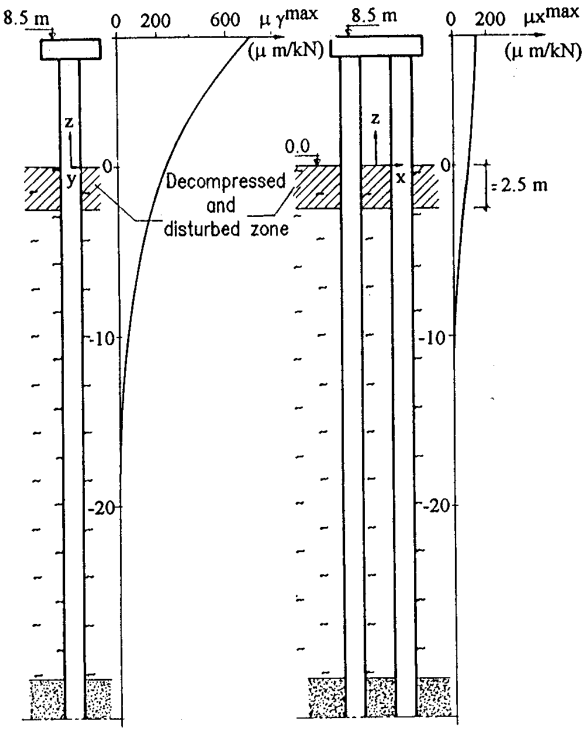


Figure 17. Variation with depth of maximum displacement of piles. X and Y directions

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

8. LEZIRIA BRIDGE

8.1. Brief Description

The Project related with the Conception, Design, and Construction of Tejo Crossing in Carregado was awarded by BRISA to a Construction Consortium

The crossing (Figure 18) that integrates the North Viaduct, the Main Bridge and the South Viaduct is subsequently described..

The Basic Design of this 11.9 km long crossing of the Tagus river, is located 25 km upstream of the Vasco da Gama Bridge. 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 2.0m depth beam directed connected to 1.5 m diameter piers. There is a 62 m span to cross the railway (Figure 19).
The deck is 23 m above the water level.

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 includes a kerb on which rests a safety barrier, a maintenance foot walk 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 (Figure 20). 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 of. In the event of serious seismicity activity the new Tagus bridge will be one of the main access for emergency vehicles crossing the estuary.

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

8.2. Main Geological Conditions

1. 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 Figure 21.

2. Geomorphology

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

3. Geological structure

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

4. 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 Figure 22.

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.

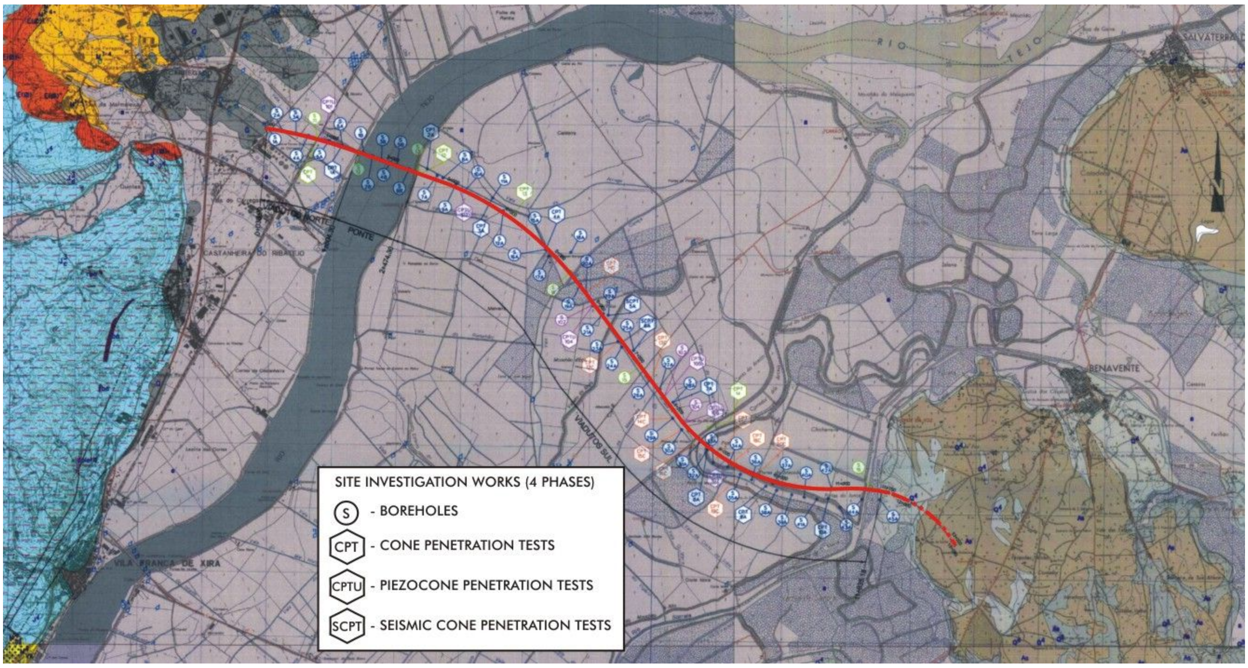


Figure 18. Leziria Tagus River Crossing site

Figure 19. North Viaduct (courtesy of Charles Lavigne)

Figure 20. Main Bridge (courtesy of Charles Lavigne)

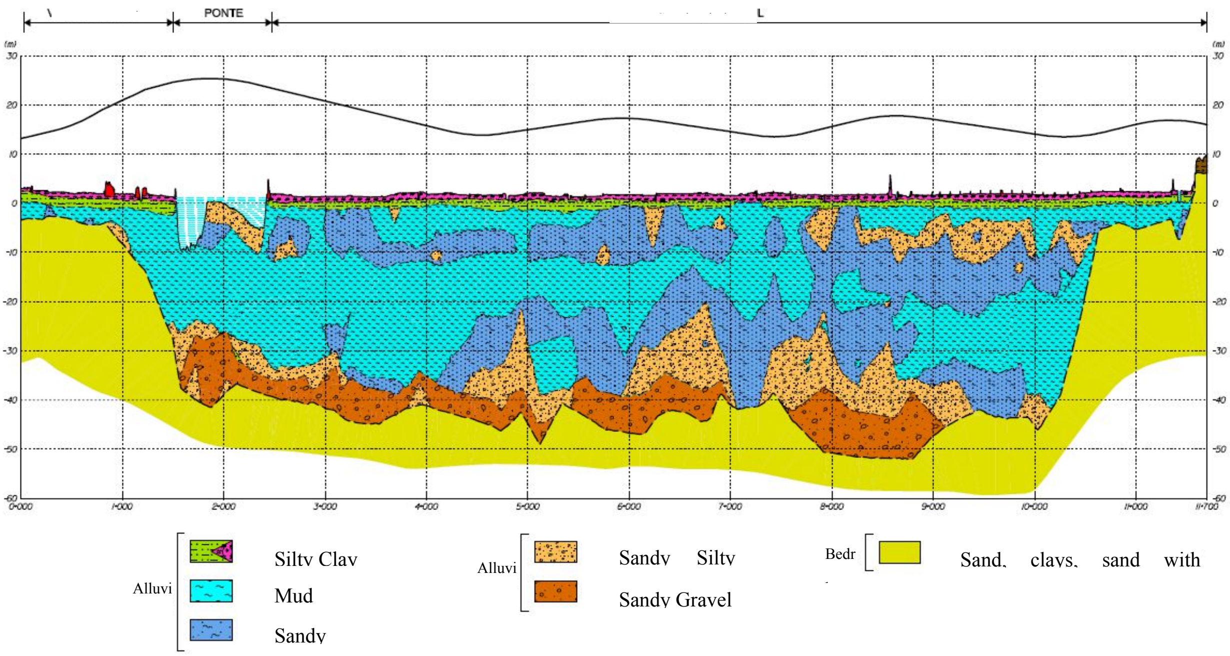


Figure 21. Simplified geological profile

Figure 22. Visual aspect of the materials

8.3. 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 by Geocontrole.
9 seismic crosshole tests were performed, namely 7 tests by GEOCISA and 2 tests by LNEC during the 2nd phase of 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 (Figure 23);
ii) 10 vane shear tests;
iii) 25 undisturbed samples taken with Geabor S sampler (Figure 24);
iv) 16 CPTU tests (Figure 25 and Figure 26)
v) 5 seismic crosshole tests.

A summary of field tests is presented in Table 13.

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 Figure 27.
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 Figure 28.
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 Figure 29.
Pore pressures values have allowed the identification of material, higher values were related with mud materials.

Figure 23. Borehole equipment

Figure 24. Geobor S sampler

Figure 25. CPTu equipment

Figure 26. CPTu tip

8.4. 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 14.

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.

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 Figure 30.

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

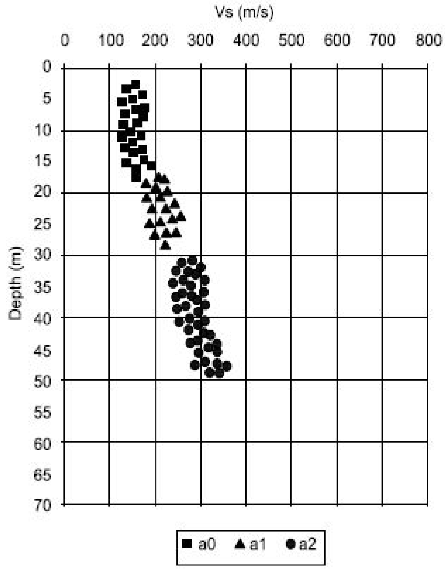


Figure 27. Variation of V_s with depth

Figure 28. Variation of undrained strengths with depth

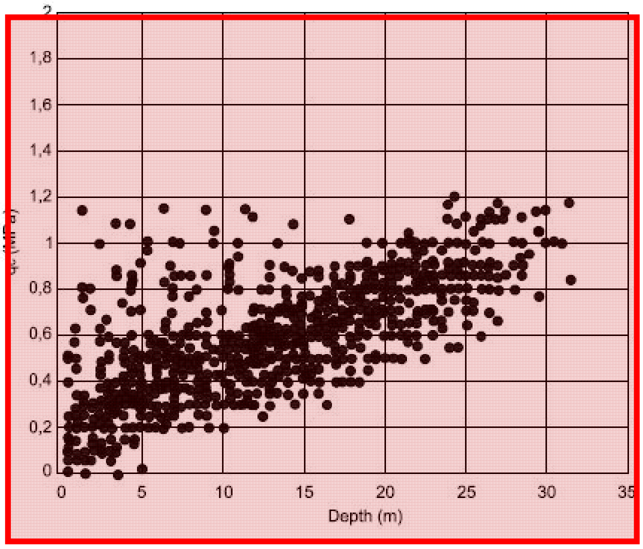
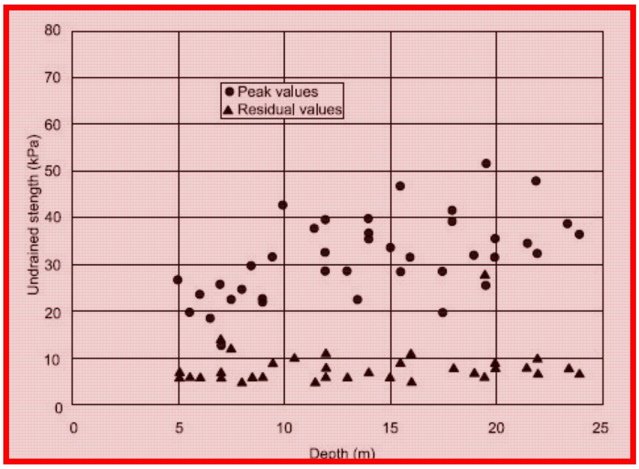


Figure 29. Variation of q_c values with depth



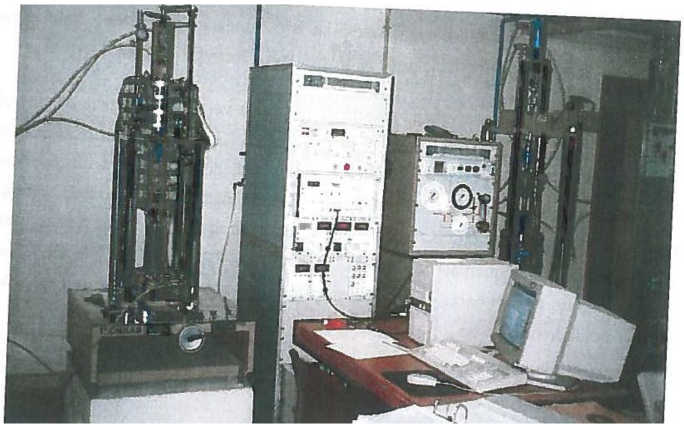


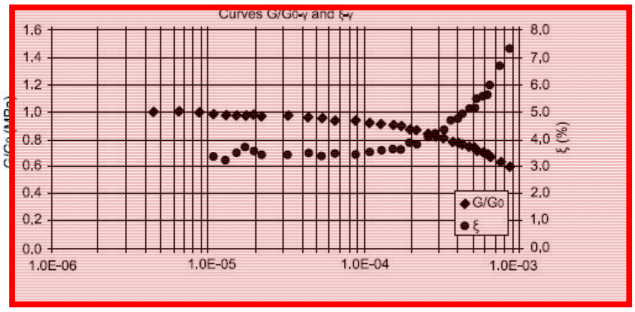
Figure 30. View of cyclic torsional shear apparatus (IST)

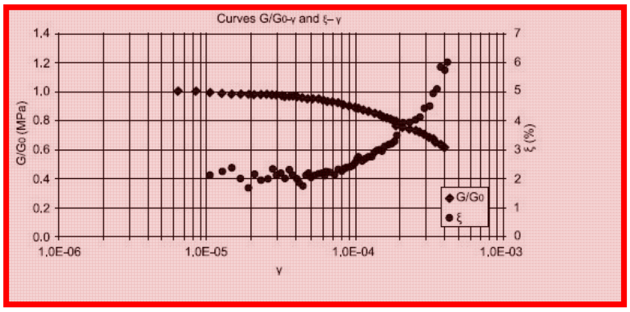
Table 13. Distribution of field tests

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 14. 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





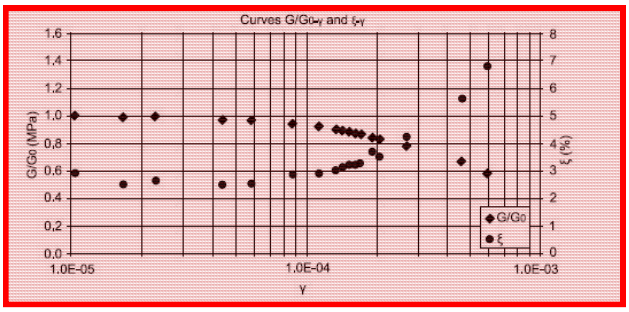


Figure 31. Curves shear modulus and damping ratio versus shear strain (after IST, 2005)

8.5. 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

A summary of each unit based in the geological and geotechnical characteristics is presented in Table 15.

TABLE 15. Summary of units geotechnical characteristics

Material	WL	WP	Vs m/s	Vp m/s	Edin MPa	Gdin 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
a_3 sandy materail with silt,	NP	NP	320 - 400	665 - 1526	500-1100	200-400	40-60	
M bedrock Miocene			400 - 500		500-1700	200 - 600	>60	

A correlation between Vs and SPT values obtained by the tests with the proposal of some authors is shown in Figure 32.

"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.

8.6. 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, 2004). For the computation of these accelerograms the validation criteria of EC8 (1998a) was considered (Figure 33).
For the stochastic approach power spectral density functions based on RSA (1983) were used.

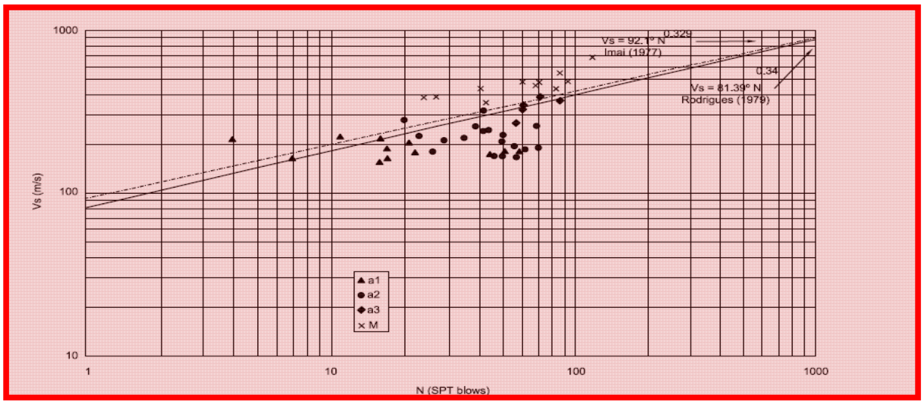


Figure 32. A correlation between Vs and SPT values

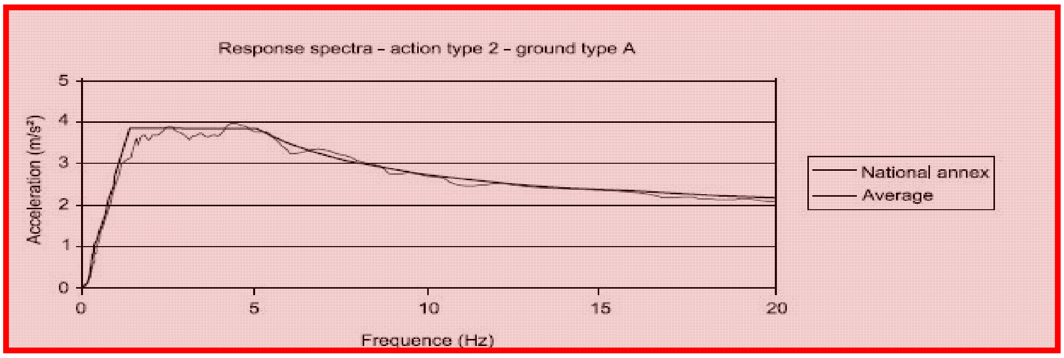


Figure 33. Response spectra versus code spectra (after IST, 2004a)

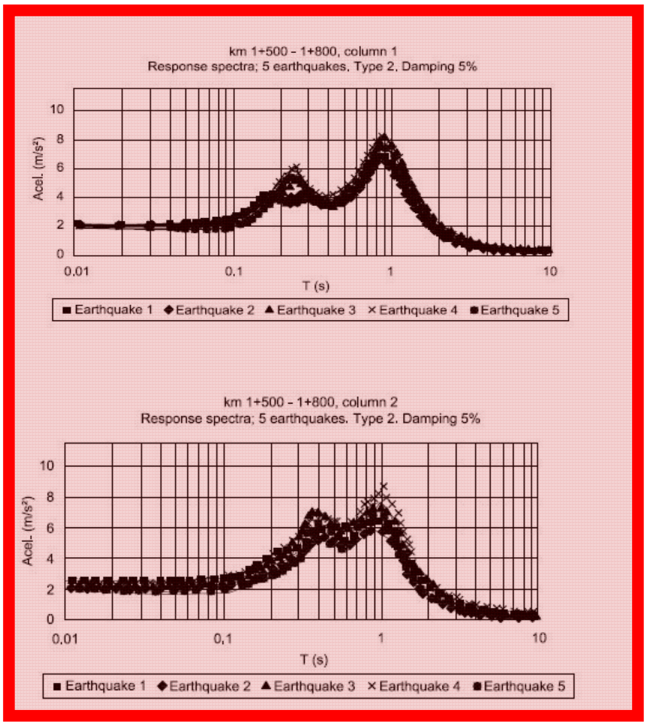


Figure 34. Response spectra acceleration km 1+500 – km 1+800 action type 1 and action type 2 (after IST, 2004a)

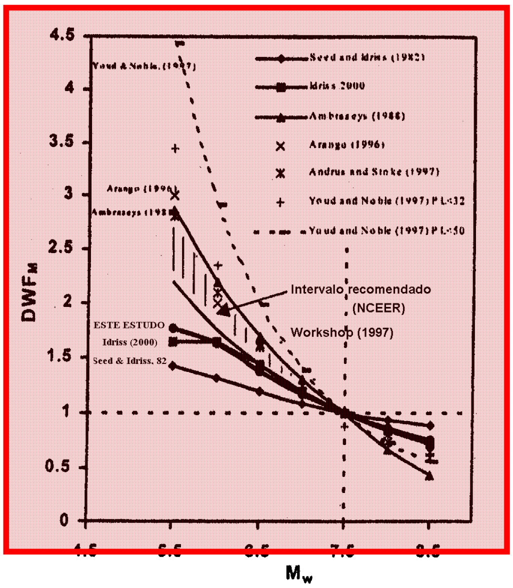


Figure 35. Magnitude scaling factors

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 Figures. 33 and 34 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.

8.7. 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 16.

In Figure 36 the computed induced shear stress are presented.

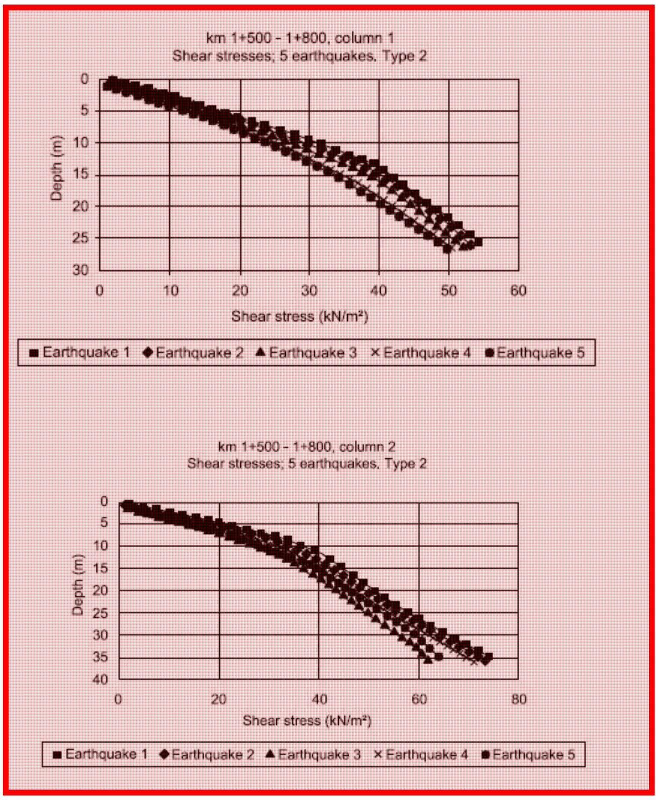


Figure 36. Induced shear stress km 1+500 – km 1+800, action type 1 and action type 2 (after IST, 2004a)

Table 16. 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 presented by Cetin et al. (2001)for liquefaction analysis is shown in Figure 37. 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; (v) Baysiana theory.

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 17.

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).

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.

8.7.1. 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 38).

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 39) was proposed by Ishihara (1993).

8.7.2. 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.

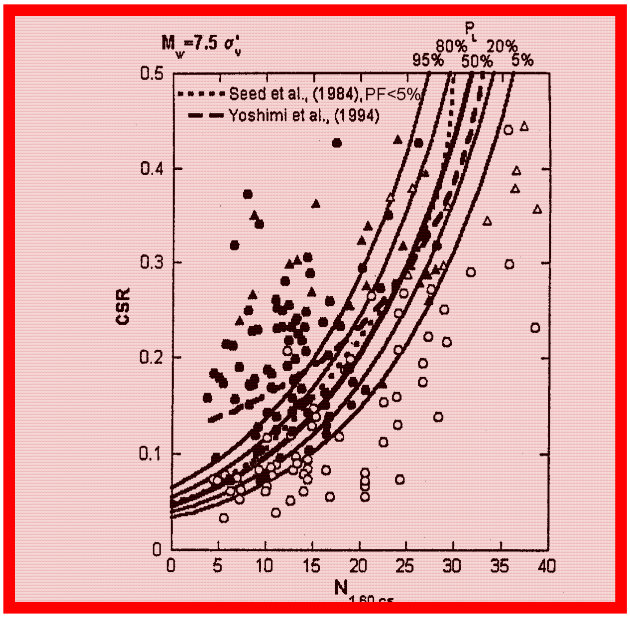


Figure 37. Probabilistic approach for liquefaction analysis (after Cetin et al., 2001)

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.

6. 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.

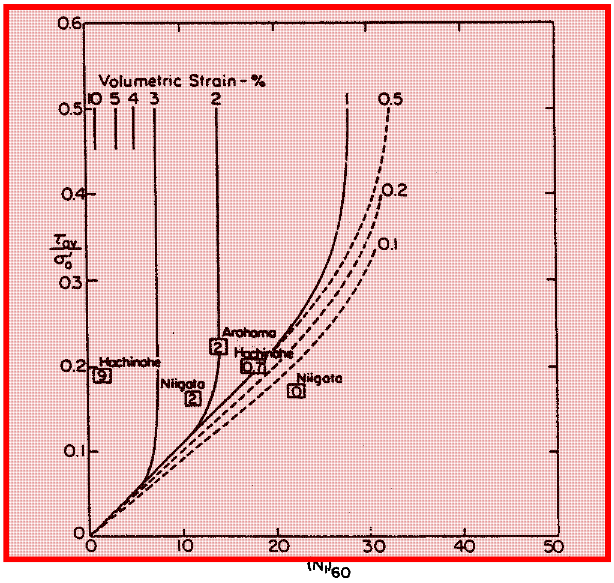


Figure 38. Correlation between volumetric strain and SPT (after Tokimatsu and Seed, 1987)

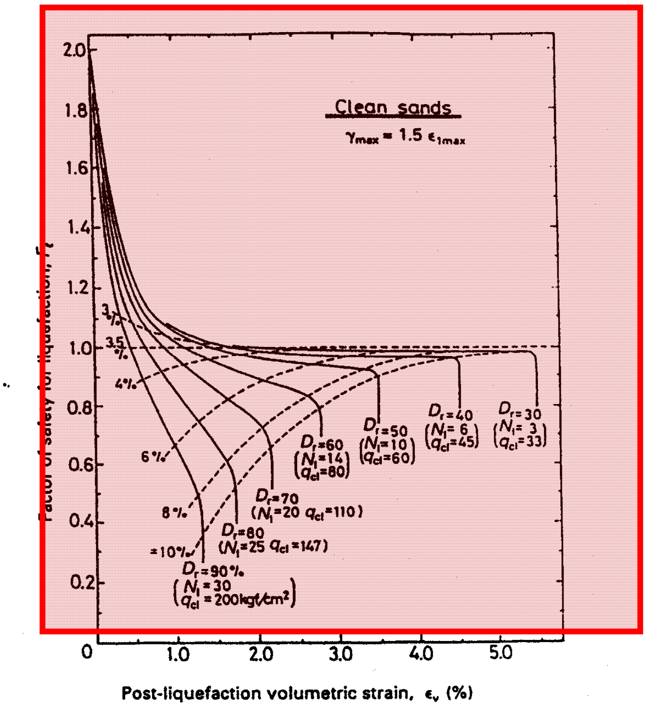


Figure 39. Post cyclic liquefaction volumetric strain curves using CPT and SPT results (after Ishihara, 1993)

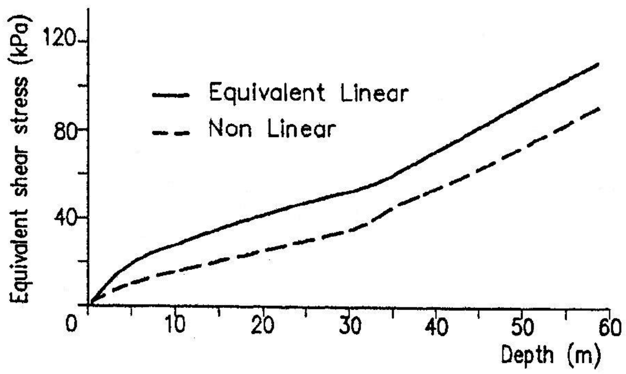


Figure 40. Equivalent shear stresses computed from SHAKE and DYNAFLOW codes (after Seco e Pinto and Oliveira, 1998)

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.

8.8. Liquefaction Evaluation

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 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 Figure. 40 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 Figures 41 and 42.

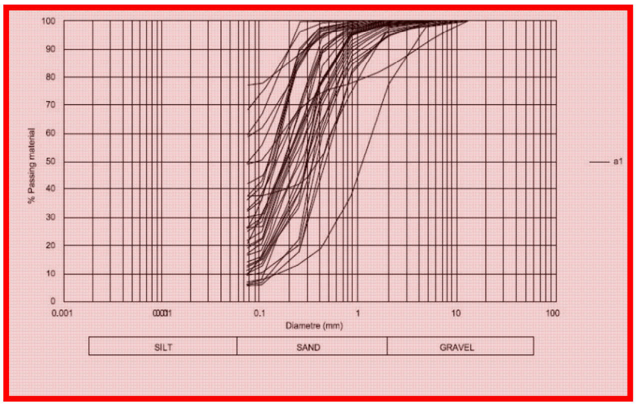


Figure 41. Sieve curves for material a_1

shallow soils due to disturbance

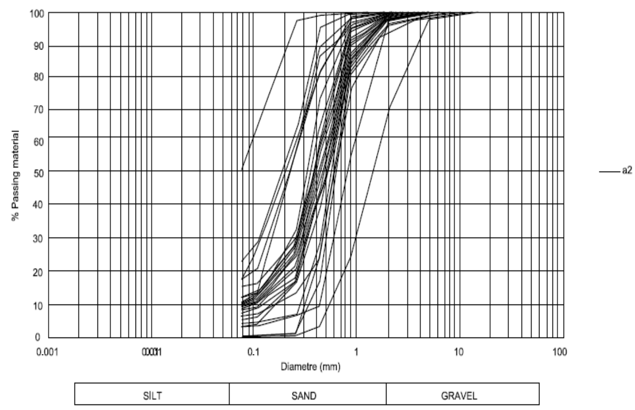


Figure 42. Sieve curves for material a_2

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 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 17 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 Figures. 43 and 44.

Taking into account the Figs. 38 and 39 the estimated settlements of materials a_1 and a_2 are between 40 mm to 150 mm.

8.9. Pile Load Tests

8.9.1. Introduction

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

Table 17. 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 Bore hole or CPT	Depth (m)	Thick-ness (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.	Re-marks
	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
N_1 (60) - Normalized SPT value
$(q_c)_m$ - CPT cone resistance value
$(q_c)_1$ - Normalized CPT cone resistance
σ'_0 - Effective overburden pressure
$\tau_{equiv.}$ - Equivalent cyclic shear stress
C_N - Correction factor for overburden pressure
L - Liquefaction
N.L - No Liquefaction

The piles of Leziria bridge 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;
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. Table 18 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.

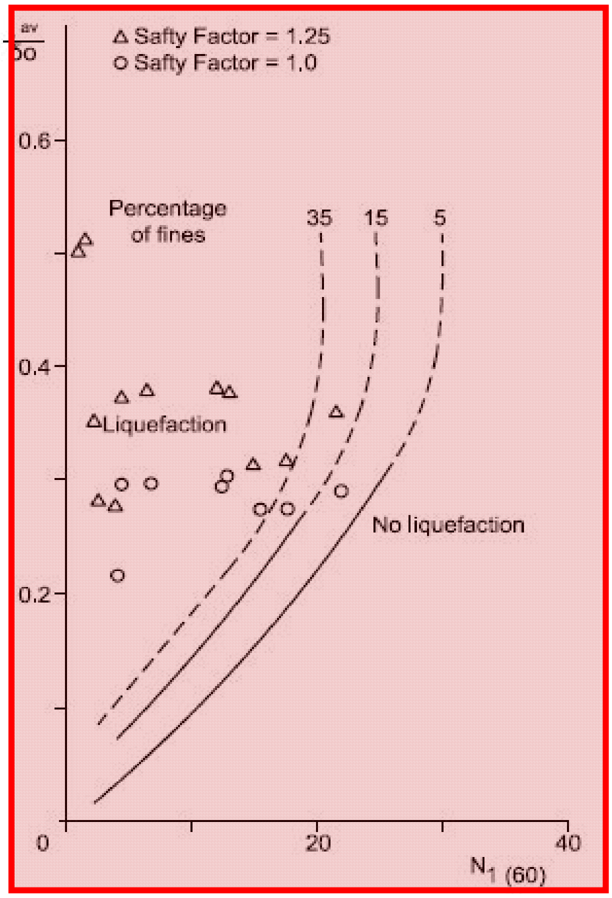


Figure 43. Liquefaction potential evaluation by SPT tests

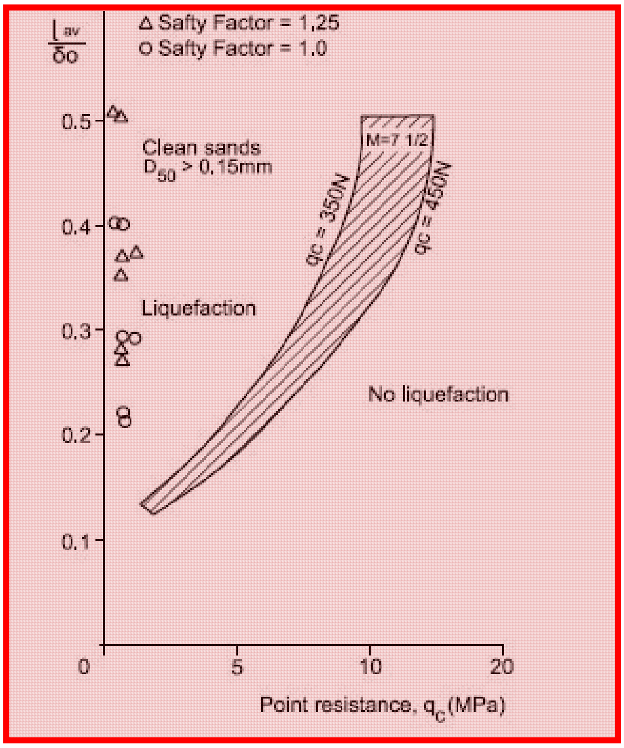


Figure 44. Liquefaction potential evaluation from CPT tests

Table 18. 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

8.9.2. Vertical pile load tests

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 (Figure 45) with removable extensometers (Figure 46), 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 Figure 47.
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

Figure 45. Displacement transducers

.

Figure 46. Recovery extensometers

Figure 47. General view for vertical pile load tests (after Ferreira et al, 2008)

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 Figure 48.

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

8.9.3. 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.
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 Figure 49.

The measured rotations values versus loads are shown in Fiure. 50.

Figure 51 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.

8.9.4. 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 (Figure 52).
- 4 transdutors
- topographic equipment

A dynamic test view is shown in Figure 53.
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 Figure 54.

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.

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)

8.10. 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 assure the stability during the driving of the casings. The system had the purpose to assure the verticality of the casings.

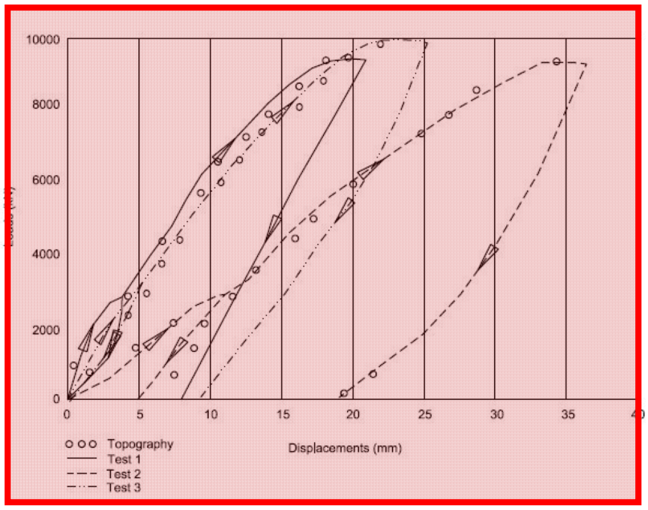


Figure 48. Load settlement curves for vertical tests (after ICIST-IST, 2005)

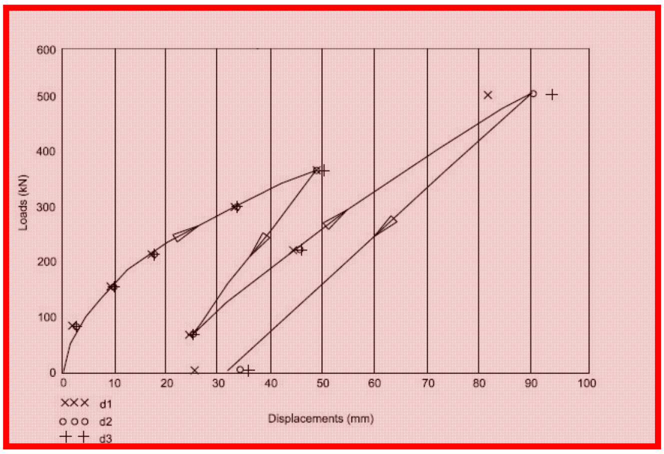


Figure 49. Measured load displacement curve for horizontal tests (after ICIST-IST, 2005)

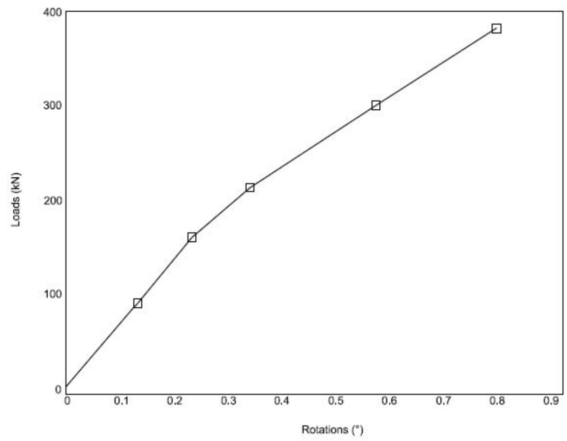


Figure 50. Measured load rotations curve for horizontal tests (after ICIST-IST, 2005)

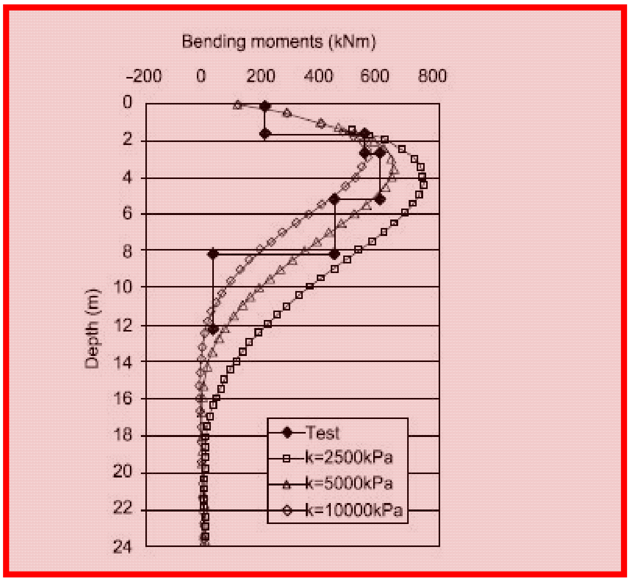


Fig. 51. Bending Moments (after ICIST-IST, 2005)

Figure 52. Transducers and accelerometers

Figure 53. Dynamic test (after Ferreira et al, 2008)

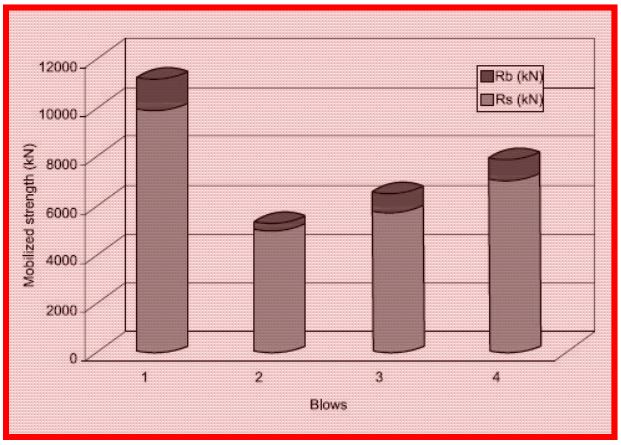


Figure 54. Mobilized resistances (after ICIST-IST, 2005)

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 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).

A view of North Viaduct construction is shown in Figure 55.

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

A view of South Viaduct construction is shown in Figure 57.

The placement of pile casing is shown in Figure 58 .

Figure 55. Construction of North Viaduct

Figure 56. Parallel Way

Figure 57. A view of South Viaduct construction

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

The placement of pile reinforcement and tremi pipes are shown in Figures 59 and 60.

In Figures 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 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.

Figure 59. Placement of pile reinforcement (after Ferreira et al, 2008)

Figure 60. Placement of tremie pipes (after Ferreira et al, 2008)

Figure 61. View of Caisson (courtesy of Perry da Câmara)

Figure 62. Pier under construction (courtesy of Perry da Câmara)

Figure 63. General view of the construction works (courtesy of Perry da Câmara)

8.11. 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).

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 :
diameter < 0.60 m = 2 tubes
0.60 m< diâmetro< 1.20 m = 3 tubes placed 120 º apart
diâmeter> 1.20 m = 4 tubes, as a minimum;
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.

Figure 64 shows a pile view with 4 tubes.
Taking into account that piles were 1.52m 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 .

Figure 64. 4 tubes for crosshole tests in a 1.5m diameter pile (after Ferreira et al, 2008)

8.12. 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:

(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 sensors in order to characterize the initial state of the bridge before the opening to traffic;
(ii) 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;
(iii) The annual inspections are related with the visual inspection of the foundations (measurements by sensors placed into the piles), supporting structures, bearings, expansion joints, superstructures and equipment;
(iv) After the opening to traffic, the first detailed inspection will be done after two years. During the operation of the bridge the frequency is five years.

9. CONCLUSIONS

The following conclusions can be outlined:

For Vasco de Gama bridge
(1) For the pile foundations each geotechnical design situation shall be verified that no relevant limit state is exceeded.
(2) Limit states should be verified by one or a combination of the following methods: design by calculation, design by prescriptive measures, design by loads tests and experimental models and observational method (Eurocode 7, 1997).
(3) In other to improve the pile behaviour field tests with instrumented piles are highly recommended for design purposes.
(4) The results of load tests performed in New Tagus bridge and Leziria bridge for design purposes have shown how they should be used to calibrate the design parameters, to check the performance of individual piles and to allow judgement of the overall pile foundation, and to assess the suitability of the construction method.

For Leziria bridge

5) 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.
6) The geotechnical characteristics were obtained after a balance between the results of the field and laboratory tests.
7) 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).
8) The Leziria bridge is located in zone A of Portugal the highest seismic zone.
9) 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.
10) 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.
11) 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.
12) Non destructive techniques of pile tests were performed to assess the quality of piles.
13) The objectives of monitoring during construction and long term were presented.

Lessons for Tomorrow

Today there is a need to work in large teams exploring the huge capacity of computers to analyze the behavior of bridges. Innovative methods and new solutions require high reliable information and teams integrating different experts, namely seismologists, geologist, geophysics, geotechnical engineers and structures engineers.

A joint effort between Owners, Decision-Makers, Researchers, Consultants, Professors, Contractors and General Public to face this challenge is needed.

It is important to understand the concepts of vulnerability and resilience. 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 received a great attention from Rousseau and Kant (1756). Resilience is a measure of the system`s capacity to absorb recover from a hazardous event. Includes the speed in which a system returns to its original state following a perturbation. The capacity and opportunity to recolate 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.

It is important that engineers educate themselves and the Public with scientific methods for evaluating risks incorporating the unpredictable human behavior and human errors in order to reduce disasters.

From the analysis of past bridges incidents and accidents occurred during the earthquakes it can be noticed 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 how 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. 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 followed Egyptians example and preserve the knowledge through the 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.

Also it is important to narrow the gap between the university education and the 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 that 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 these topics we should never forget the memorable lines of Hippocrates:

"The art is long
and life is short
experience is fallacious
and decision is difficult".

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