Тем не менее, даже в самом европейском нормативном документе, а также и в национальных его приложениях требуется отказаться от утративших актуальность и недостоверных методов определения деформативных характеристик грунтов (модулей деформации, коэффициентов боковых давления и расширения грунтов) путем компрессионного сжатия, отражающего только спрессовывание образцов малой толщины и не соответствующего реальному поведению грунта.

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DESIGN OF GEOTECHNICAL STRUCTURES FOLLOWING EUROCODE 8

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1. INTRODUCTION

Following Part 5-EC8 “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 magnitude correction factors in EC8 follow the proposal of Ambraseys (1988) based in field tests that are different of the values proposed by Seed and Idriss (1982) and from the values proposed by NCEER (1997) based in laboratory tests. For liquefaction evaluation EC8 recommends the use of SPT tests and CPT tests.

In general for the Soil-Structure Interaction (SSI) the design engineers ignore the kinematic component, considering a fixed base analysis of the structure. 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.

Some future trends are pointed out. A summary of conclusions is presented.

2. POTENTIALLY LIQUEFIABLE SOILS

2.1 Introduction

Following 4.1.3. (2)-Part5-EC8 “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 1.

Table 1 – Magnitude scaling factors

Magnitude M	Seed & Idriss (1982)	Idriss NCEER (1997)	Ambraseys (1988)
5.5	1.43	2.20	2.86
6.0	1.32	1.76	2.20
6.5	1.19	1.44	1.69
7.0	1.08	1.19	1.30
7.5	1.00	1.00	1.00
8.0	0.94	0.84	0.67
8.5	0.89	0.72	0.44

A new proposal with a summary of different authors presented by Seed et al. (2001) is shown in Figure 1.

Empirical liquefaction charts are given with seismic shear wave velocities versus SPT values to assess liquefaction. A comparison between NCEER (1997) and EC8 proposal for pre-standard is shown in Figure 2. It is important to refer that the proposal for EC8 is based on the results of Roberston et al.(1992) and the proposal of NCEER(1997) incorporates very recent results.

However the EC8 standard version considers that these correlations are still under development and need the assistance of a specialist.

The importance of this topic has increased and the assessment of liquefaction resistance from shear wave crosshole tomography was proposed by Furuta and Yamamoto (2000).

A new proposal presented by Cetin et al. (2001) is shown in Figure 3 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 a better interpretation of SPT test; (iv) local effects; (v) cases histories related more than 200 earthquakes; (v) Baysiana theory.

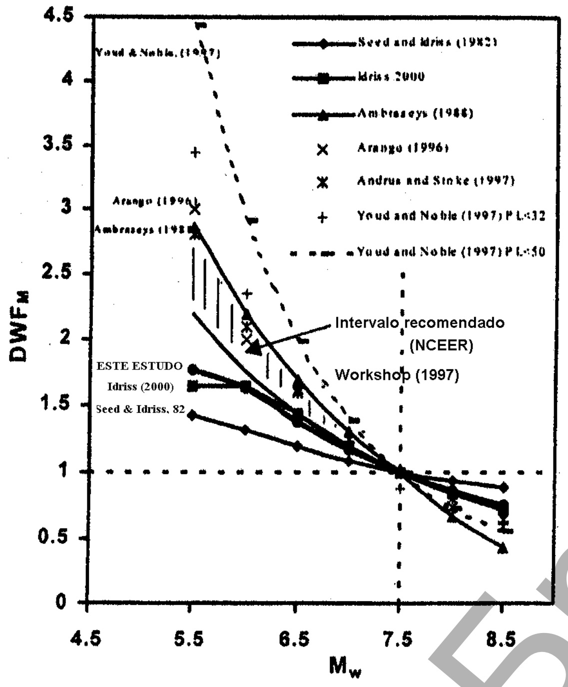

Figure 1 – Recommendations for correlations with magnitude (after Seed et. al., 2001)

Bray et al. (2004) have shown that the chinese criteria proposed by Seed and Idriss (1982) was not reliable for the analysis of silty sands liquefaction and have proposed the plasticity index.

Alba (2004) has proposed Bingham model, based in triaxial tests of large samples, to simulate residual strength of liquefied sands.

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 shesr tests; (iii) cyclic torsional shear tests. Due the difficulties to obtain undisturbed samples of high quality 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 laboratoty 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 of 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 2.

Table 2 – Corrective values for magnitude

Magnitude M	Seed et Idriss (1982)	NCEER(1997)	Ambraseys (1988)
5,5	1,43	2,20	2,86
6,0	1,32	1,76	2,20
6,5	1,19	1,44	1,69
7,0	1,08	1,19	1,30
7,5	1,00	1,00	1,00
8,0	0,94	0,84	0,67
8,5	0,89	0,72	0,44

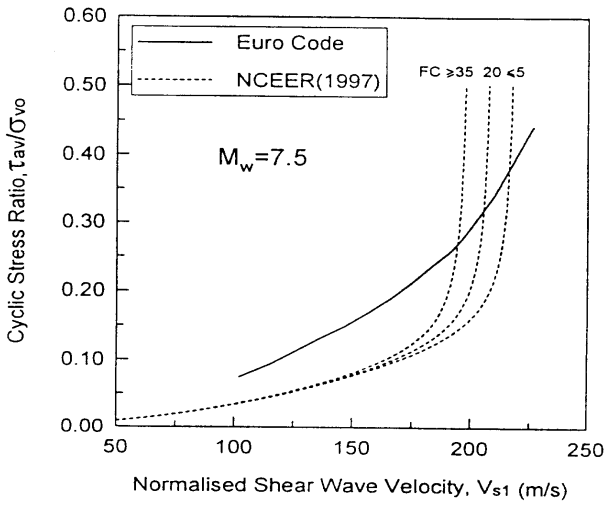

Figure 2 – Liquefaction potential assessment by NCEER (1997) and EC8 (pre-standard)

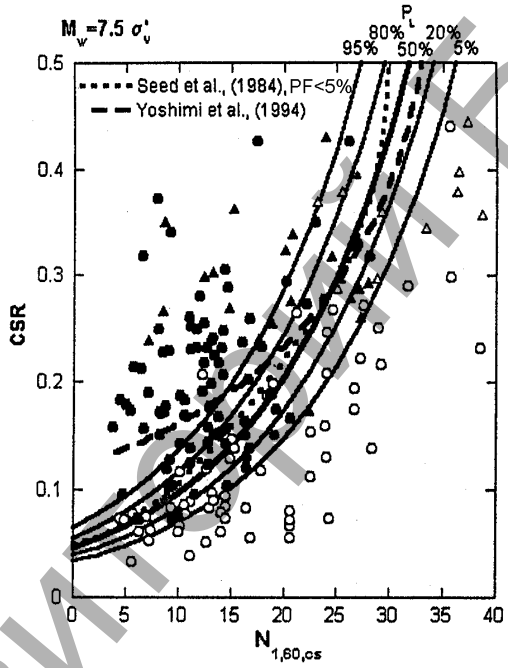

Figure 3 – Probabilistic approach for liquefaction analysis (after Cetin et al., 2001)

Bray et al. (2004) have shown that the chinese criteria proposed by Seed and Idriss (1982) was not reliable for the analysis of silty sands liquefaction and have proposed the plasticity index.

Due 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 (Andrus et al, 1999; Stokoe et al, 1999).

A probabilistic method considering the unknowns in liquefaction was proposed by Todorovsha and Trifunac (1999).

The liquefaction assessment of silty sands materials with different contents of silty and confining pressures were investigated by Amini and Qi (2000).

The post-liquefaction strength of silty materials is less than sandy materials, but superficial silty materials with moderate density are dilatant and with high 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 liquefy and soils with IP>20 are no liquefiable.

It is important to refer that Eurocode 8 (1998)-Part 5 considers no risk of liquefaction when the ground acceleration is less than 0.15 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 com $N_1(60)$>25.

The VELAC (Verification of Liquefaction Analyses by Centrifuge Tests) program has the purposes to calibrate the results of numerical models with the centrifuge tests involving 8 universities in USA and UK (Arulanandan e Scott (1993, 1994). Also CANLEX (Cooperation Canadian Liquefaction Experiments) program has benefited of the synergies of industry, consultants, universities and sampling of high quality in sandy materials (Roberston et al., 1995)

2.2 Post Liquefaction Strength

The topic related with the assessment of post liquefaction strength is not treated in EC8, but it seems that the following variables are important: fabric or type of compaction, direction of loading, void ratio and initial effective confining stress (Byrne and Beaty, 1999).

A relationship between SPT N value and residual strength was proposed by Seed and Harder (1990) from direct testing and field experience (Figure 4).

Ishihara et al.(1990) have proposed a relation of normalized residual strength and SPT tests, based on laboratory tests compared with data from back-analysis of actual failure cases (Figure 5). Also Ishihara et al. (1990) by assembling records of earthquake caused failures in embankments, tailings dams, and river dykes have proposed the relation of Figure 6, in terms of the normalized residual strength plotted versus CPT value.

Alba (2004) has proposed Bingham model, based in triaxial tests of large samples, to simulate residual strength of liquefied sands.

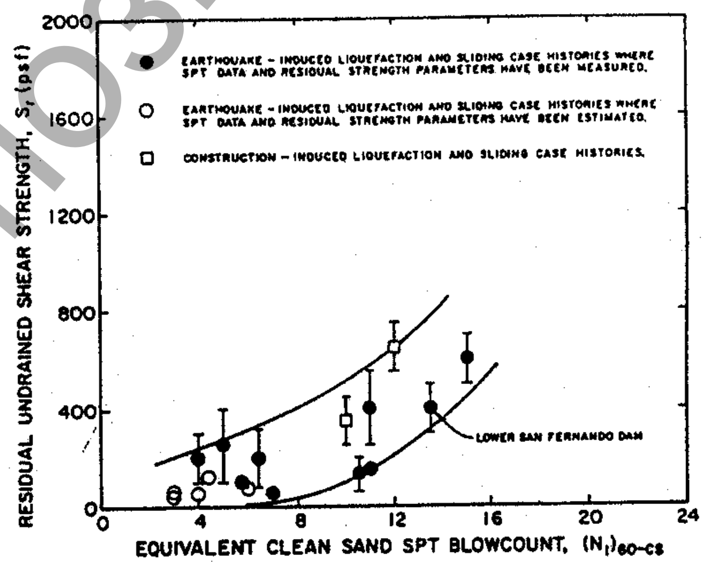

Figure 4 – Relationship between (N1) 60 and residual strength (after Seed and Harder, 1989)

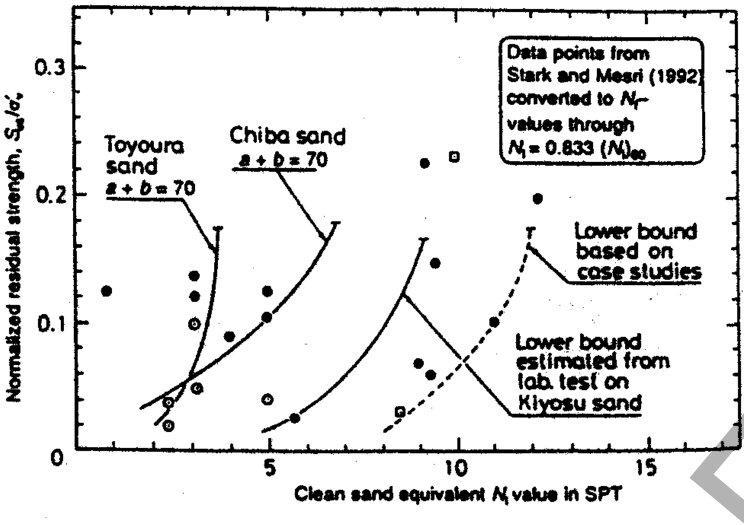

Figure 5 – Relation of normalized residual strength and SPT tests (after Ishihara et al., 1990)

2.3. 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 7).

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 8) was proposed by Ishihara (1993).

Shamoto et. al, 1998) have proposed Figure 9 for computation of shear deformations for sandy soils

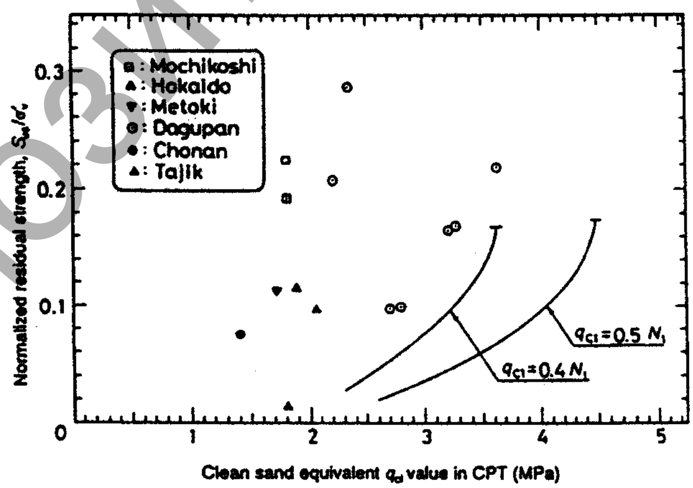

Figure 6 – Relation of normalized residual strength and CPT tests (after Ishihara et al., 1990)

2.4. 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 the large forces induced in the piles.

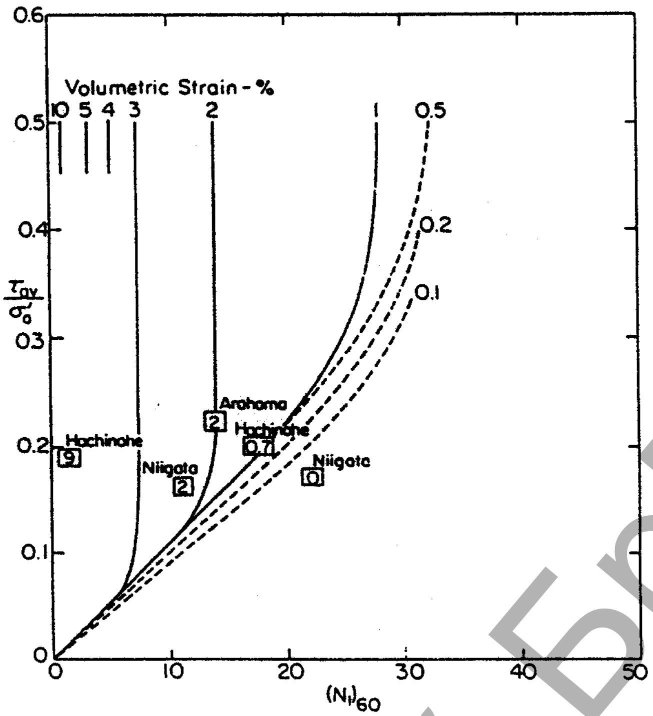

Figure 7 – Correlation between volumetric strain and SPT (after Tokimatsu and Seed, 1987)

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.

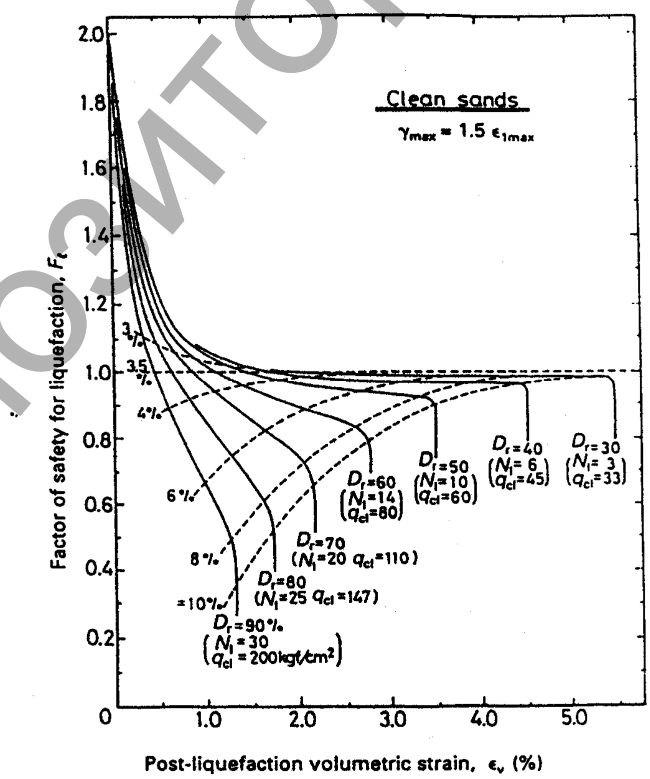

Figure 8 – Post cyclic liquefaction volumetric strain curves using CPT and SPT results (after Ishihara, 1993)

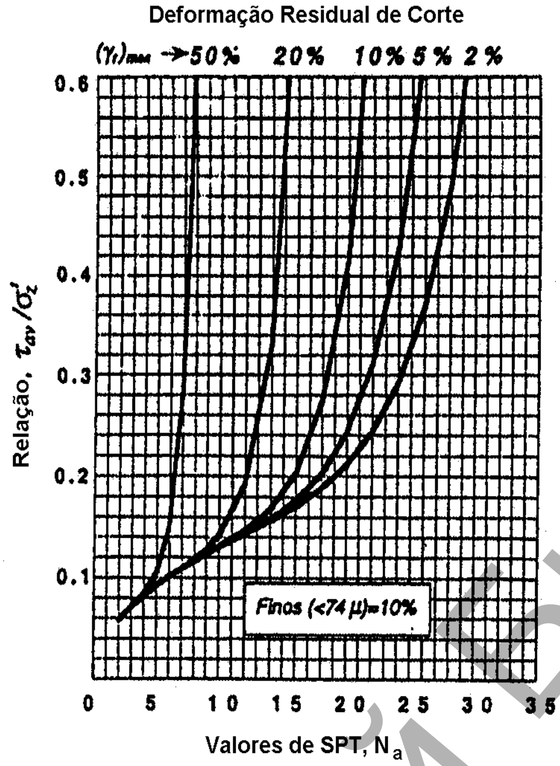

Figure 9 – Shear deformations for sandy soils (after Shamoto et. al, 1998)

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.

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.

In NEMISREF Project the following criteria for selection was used (Evers, 2005): (i) Potential efficiency; (ii) Technical feasibility; (iii) Impact on structure and environmental; (iv) Cost-effectiveness; (v) Innovation.

Two methods were selected: (i) Soil grouting using calcifying bacteria; (ii) confinement wall.

Related with calcifying bacteria the objective of soil consolidation is to create a cementation between the grains of soil skeleton increasing the cohesion.

With confinement wall even if partial liquefaction could occur the final deformations will be controlled.

The improvement of soil properties, to prevent soil liquefaction, by soil cementation and solidification is performed by deep mix method (Port Harbour Research Institute, 1997), so within this framework the use of bacteria technique is innovative.

The structural strengthening is performed by pile foundation and sheet pile (INA, 2001) and so the confining wall can be considered innovative.

The proposed methods of remediation have an additional advantage minimizing the effects on existing structures during soil improvement.

Comments: From the analyses of this section it seems that the following items deserve more clarification (Sêco e Pinto, 1999b):

i) It is important to quantify the values of extended layers or thick lenses of loose sand;

ii) What is the meaning of "......when such level is close to the ground surface"? What depth? What is the maximum depth liquefaction can occur?

iii) No recommendation is presented to compute seismic shear stress τ_e for depths larger than 20 m;

iv) The use of Becker hammer and geophysical tests to assess the liquefaction of gravely materials should be stressed;

v) The recommended multiplied factor CM for earthquake magnitudes different from 7.5 deserves more explanation. It is important to refer that the well known correlation proposed by Seed et al (1983) for cyclic stress ratio versus N_1 (60) to assess liquefaction and adopted in Annex B of EC8 – Part 5 use different correction factor for earthquake magnitudes different from 7.5;

vi) No reference is given for the residual strength of soil.

3. EARTH RETAINING STRUCTURES

The methods of analyses of an earth-retaining structure shall incorporate: (i) the non-linear behaviour of the soil; (ii) the inertia effects associated with the masses of the soil; (iii) the hydrodynamic effects generated with by the presence; (iv) the compatibility between deformations of the soil, wall and the tiebacks.

For the pseudo–static analysis of rotating structures the seismic coefficients can be taken as:

$$k_h = \alpha_{gr}\gamma_f S/ g.r \tag{1}$$

$$k_V = \pm\, 0{,}5\, k_h \text{ when the ratio } \alpha_{vg}/\alpha_{gr} \text{ is greater than 0.6} \tag{2}$$

$$k_V = \pm\, 0{,}33\, k_h \text{ otherwise} \tag{3}$$

Where α_{gr} is the reference peak ground acceleration for class A ground, S is the soil parameter, γ_f is the importance factor of the structure and the factor r takes the values listed in Table 3.

For saturated cohesionless soils susceptible to develop high pore pressure the r factor should not be taken larger than 1.0, and the safety factor against liquefaction should not be less than 2.

The point of application of the force due to dynamic earth pressure shall be assumed to lie at midheight of the wall and for walls which are free to rotate about their toe it is appropriate to consider the dynamic force acting at the same point as the static force.

Table 3 – Factor affecting the horizontal seismic coefficient

Type of retaining structure	r
Free gravity walls that can accept a displacement $d_r \le 300\ \alpha$ S(mm)	2
As above with $d_r \le 200\ \alpha$ S(mm)	1.5
Flexural r.c. walls, anchored or braced walls, r.c. walls founded on vertical piles, restrained basement walls and bridge abutments.	1.0

For a soil permeability coefficient less than $5x10^{-4}$ m/s the pore pressure is not free to move and the soil will behave as an undrained situation, during the occurrence of seismic action.

The earth pressure coefficient can be computed from the Mononobe and Okabe formula.

The point of application of the force due to the hydrodynamic water pressure lies at a depth below the top of the saturated layer equal to 60% of the height of such layer.

The pressure distributions on the wall due to the static and the dynamic action shall be assumed to act with an inclination with respect to the normal to the wall not greater than (2/3) ϕ' for the active state and equal to zero for the passive state.

The stability of soil foundation shall be assessed for the following conditions: (i) overall stability; and (ii) local soil failure.

The anchoring system (tiebacks and anchors) provided behind walls and bulkheads shall have enough strength to assure equilibrium of the critical soil wedge under seismic conditions, as well as a sufficient capacity to adapt to the seismic deformations of the soil.

The EC8 does not refer to the behavior of reinforced walls. The behavior of these structures during recent earthquakes suggests that these types of structures are well suited for seismically active regions (Sitar et al., 1997).

The EC8 only refers the condition of walls to slide, but it is important to stress the rocking of large concrete gravity walls under earthquake loading (Sêco e Pinto, 1995).

For an embedded retaining structure characterized by a ductile behavior, it can be anticipated that the equivalent value of the acceleration to use in a pseudo-static calculation, as if it were constant in time, should be significant smaller than the expected peak acceleration (Anastassapoulos, 2004).

The authors have performed numerical analyses and the obtained results were in good agreement with the behavior observed during Northridge, Kobe, Parnitha and Kocaeli esrthquekes .

Comments: From the analyses of this section it seems that the following items deserve additional consideration:

(i) Design methods for the computation of permanent displacements that allow the couple computation of rotation and translation movements should be referred;

(ii) For retaining walls of medium heights (greater than 6 m) the computed displacements are larger than the values listed by EC 8 (Wu and Prakash, 2001);

(iii) The permanent displacements should be related with the height of the wall;

(iv) The good behaviour of geogrid – reinforced soil retaining walls in comparison with reinforced concrete cantilever retaining walls, during the occurrence of earthquakes, should be stressed.

The lessons related the behavior of retaining structures during the occurrence of earthquakes have appointed the importance of the following factors: (i) increasing of seismic pressures; (ii) variation of hydrodynamic pressures; (iii) decreasing of stable forces due the weigth of the structure; (iv) increasing of pore pressures and consequently reduction of effective pressures; and (v) liquefaction of backfill and foundations materials.

Due the restrictions of pseudo-static methods in predicting seismic displacements new methods for predicting displacements were proposed by Richard and Elms (1979) and Siddhartan et al. (1991) that allow the design of more economic structures

A comparison between the results of shaking table and numerical methods based in Zarrabi model was presented by Simonelli et. al. (2000).

The numerical methods and particularly the finite element method allow the analysis of variable geometries and complex constitutive laws incorporating soil hysteretic non linear behavior (Siddharthan e Norris, 1991) and elastoplastic non linear behavior with pore pressures generation (Allampalli e Elgamal, 1991).

4. SOIL-STRUCTURE INTERACTION

In general for the Soil-Structure Interaction (SSI) the design engineers ignore the kinematic component, considering a fixed base analysis of the structure, due 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 (Sêco e Pinto, 2003).

There is strong evidence that slender tall structures, structures founded in very soft soils and structures with deep foundations the SSI plays and 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, $\alpha>0.10$;(iii) the supported structure is of important category I or II.

The stability of footings for the ultimate state limit design criteria shall be analysed against failure by sliding and against bearing capacity failure.

For shallow foundations under seismic loads failure can not be defined for situations when safety factor becomes less than 1, but is related with permanent irrecoverable displacements.

The seismic codes recommend to check the following inequality:

$$S_d<R_d, \tag{4}$$

where S_d is the seismic design action and R_d the system design resistance.

In the inequality (4) partial safety factors shall be included following the recommendations of Eurocode 8.

Theoretical and experimental studies to provide bearing capacity solutions to include the effect of soil inertia forces led to the inequality (Pecker, 1997):

$$\phi(N,V,M,F)<0 \tag{5}$$

where $\phi = 0$ defines the equation of the bounding surface (Figure 10).

The combination of the loading lying the outside the surface corresponds to an unstable situation and the combination lying inside the bounding surface corresponds to a potentially stable situation.

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.

The complete solution is a 3D analysis very time demanding and it is not adequate for design purposes. The decomposition of the problem in steps is shown in Figure 11 and implies (Gazetas and Mylonakis, 1998): i) the kinematic interaction involving the response of the base acceleration of the system considering the mass of superstructure equal to zero; (ii) the inertial interaction that involves the computation of the dynamic impedances at the foundation level and the dynamic response of the superstructure.

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 (Finn and Fujita, 2004).

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

In general the linear behavior is assumed for the soil.

The nonlinear systems are more general and the term non linearities include the geometric and material nonlinearities (Pecker and Pender, 2000).

The engineering approach considers two sub-domains (Figure 12):

i) a far field domain where the non linearities are negligible;

ii) a near field domain in the neighbouring of the foundation where the effects of the geometrical and material linearities are concentrated.

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.

The use of inclined piles is not recommended to absorb the lateral loads of the soils. If inclined piles are used they must be designed to support axial as well bending loads.

Piles shall be designed to remain elastic, if this is not possible potential plastic hinging shall be considered for: (i) a region of depth 2 d (d-diameter of the pile) from the pile cap; (ii) a region of ± 2 d from any interface between two layers with different shear stiffness (ratio of shear moduli > 6).

Evidence has shown that soil confinement increases pile ductibility capacity and increases pile plastic hinge length. Piles have shown the capability to retain much of their axial and lateral capacity even after cracking and experienced ductibility levels up to 2.5 (Gerolymos and Gazetas, 2006).

The investigation methods for pile foundation damage are: direct visual inspection, the use of borehole camera inspection and pile integrity test. The ground deformation can be investigated by visual survey and GPS survey (Matsui et al. 1997).

The stability of footings for the ultimate state limit design criteria shall be analysed against failure by sliding and against bearing capacity failure.

For shallow foundations under seismic loads failure can not be defined for situations when safety factor becomes less than 1, but is related with permanent irrecoverable displacements

The combination of the loading lying the outside the surface corresponds to an unstable situation and the combination lying inside the bounding surface corresponds to a potentially stable situation.

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 (Mineiro, 2000).

Comments: The following topics deserve more consideration:

i) The influence of pile cap;

ii) The moment rotation capacity of pile footing;

iii) The incorporation of the non linear behaviour of the materials in the methods of analysis;

iv) The instrumentation of the piles for design purposes;

v) Some guidelines about group effects, as there are significant different opinions on the influence of group effects related with the number of piles, spacing, direction of loads, soil types and construction methods of piles.

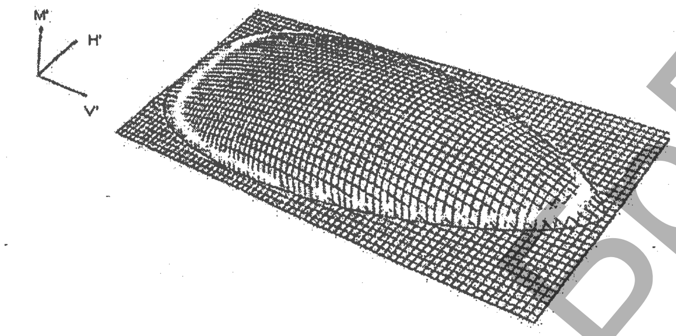

Figure 10 – Bounding surface for cohesive soils (after Pecker, 1997)

For the evaluation of mitigation methods a preliminary analysis of the following solutions was performed (Evers, 2005): (I) Stiffening solutions - hard layer, reinforced concrete walls, soil stiffening at foundation level and inclined piles; (ii) Soft material barriers - soft layer, expanded polystyrene (EPS) walls, air-water balloons and soft caisson; (iii) oscillators.

For the criteria of selection the following factors were used: Potential efficiency, technical feasibility, impact on structure and environment, cost-effectiveness and innovation.

From this analysis the following two mitigation methods: i) soil stiffening (inclined micro-piles) and ii) deformable soft barriers (soft caisson) were selected.

A 3D analysis considering non linear behavior for soil was done by Oliveira et al.(1996).

For the computation of pile inertia forces as well for the lateral displacement of pile and head rotation discrete models can be used (based on Winkler model) or continuous model.

The lateral strength of layers susceptible to liquefaction should be neglected.

5. FUTURE DEVELOPMENTS

The following topics deserve more consideration:

<u>Liquefaction</u>

i) The use of Becker hammer and geophysical tests to assess the liquefaction of gravely materials should be stressed;

ii) Determination of residual strength of soil;

iii) Evaluation of liquefaction consequences;

iv) Mitigation methods.

<u>Soil –structure interaction</u>

i) The influence of pile cap;

ii) The incorporation of the non linear behaviour of the materials in the methods of analysis;

iii) The instrumentation of the piles for design purposes;

iv) Analysis of piles group.

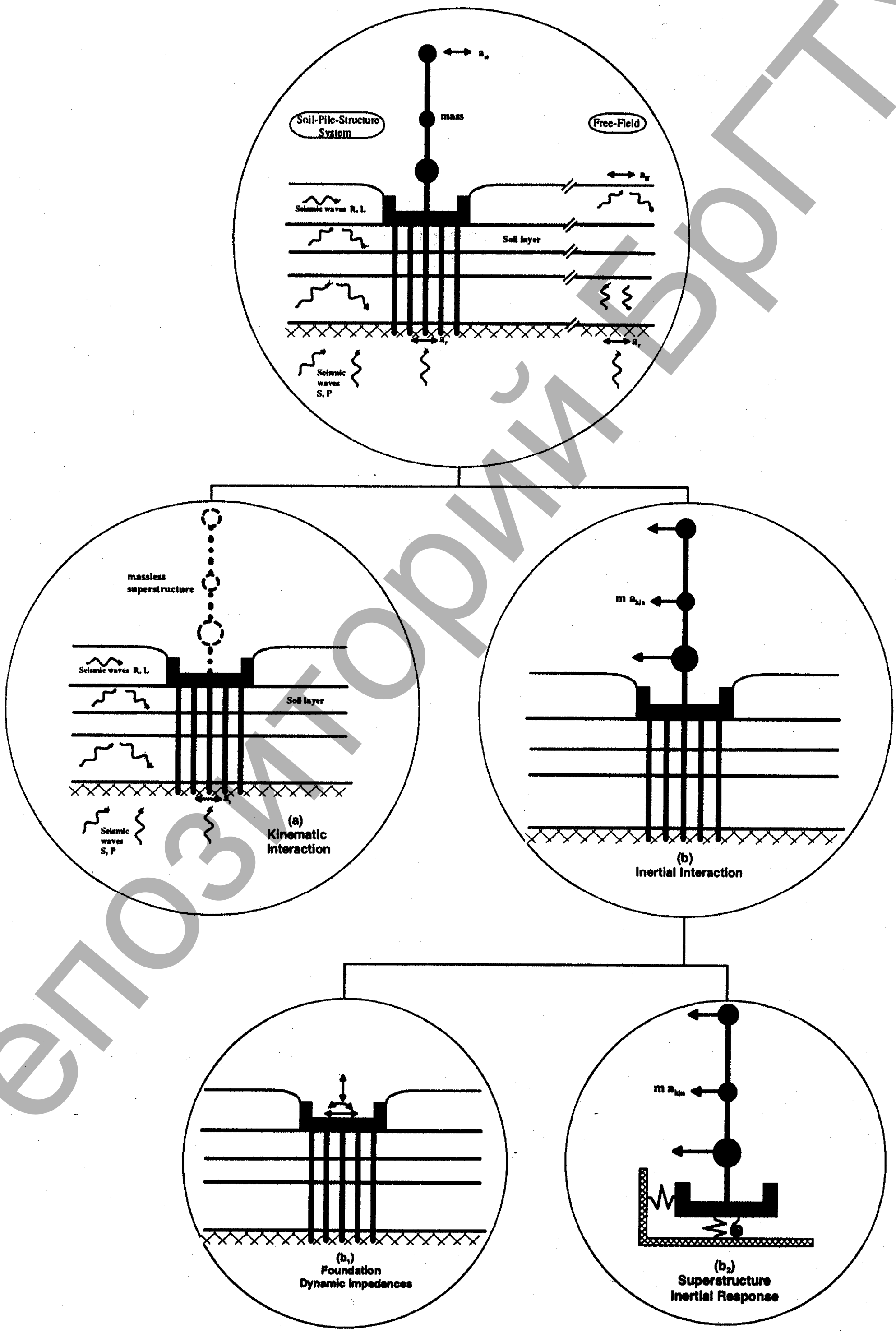

Figure 11 – Decomposition of the problem

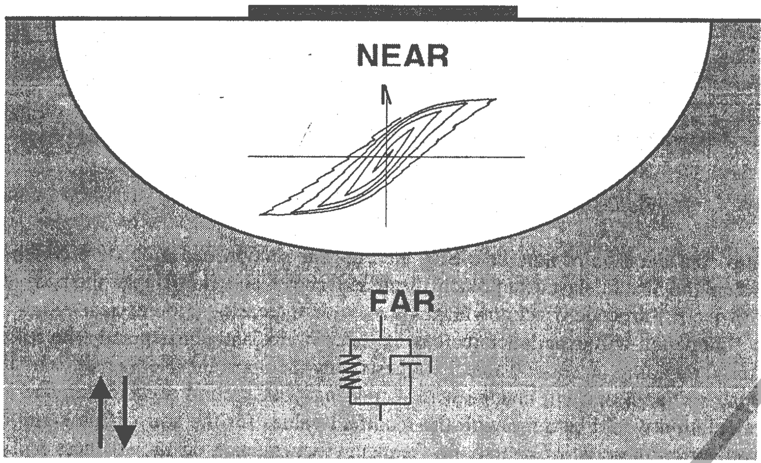

Figure 12 – Conceptual subdomains for dynamic soil structure analyses (after Pecker and Pender, 2000)

5. CONCLUSIONS

Earthquakes are very complex and dangerous natural phenomena, which occurs primary in known seismic zones, although severe earthquakes have also occurred outside these zones in areas considered being geologically stable. As a result, regulatory agencies became more stringent in their requirements for demonstration of adequate seismic stability and design engineers responded by developing new and more convincing design approaches than had previously used. Thus the past years have seen a major change in interest and attitude towards this aspect of design..

The lessons learned from recent earthquakes such as: Northridge (1994), Kobe (1995), Umbria-Marche (1997), Kocaeli (1999), Athens (1999), Chi-Chi (1999) and Bhuj (2001) have provided important observational data related with the seismic behavior of geotechnical 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. However we feel that some topics deserve more consideration.

The need of cost effective methods to upgrade buildings by developing new specific foundations techniques is a major problem. So the objective of reducing the earthquake motion transferred to the structure through the foundation by developing innovative constructive techniques for soil improvement and soil reinforcement is getting increase attention.

One very important question to be discussed is: (i) how detailed a seismic code must be; (ii) what is the time consuming to establish a set of harmonised technical rules for the design and construction works? (iii) How to improve the relations between the users: relevant authorities, clients and designers? and (iv) how to implement in practice that codes may not cover in detail every possible design situation and it may require specialised engineering judgement and experience? It is hoped that the contributions to be presented by CEN members, in the next years, will help to clarify several questions that still remain without answer.

From the analysis of past 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.

It is important to stress that a better understanding of geotechnical structures during the occurrence of earthquakes can only be achieved by a continuous and permanent effort.

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

There`s a fount about to stress
There`s a light about to beam,
There`s a flower about to blow,
There`s a warmth about to glow;
There`s a midnight darkness changing
Into grey,
Men of thought and men of action,
Clear the way”

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