Siting – Downtown Lisbon Metro blue line (*)

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(*) based on the invited Panel Lecture, MERCEA08

ABSTRACT

The estimation of liquefaction resistance is the first step in assessing the liquefaction hazard. The next important step is the assessment of possible consequences of liquefaction. The Siting case of downtown Lisbon metro blue line is presented to outstand some of the important differences, related with liquefaction assessment, between the recommendations proposed by the Eurocode 8 – Part 5 and those followed by the State of Practice (1996 NCEER and 1998 NCEER/ NSF Workshops, Youd et al., 2001), as well as to outstand that guidelines for assessing the residual strength and stiffness of potential liquefiable soils are still lacking in the Eurocode 8, or in any other international code. An effort should be made to fill in this gap.

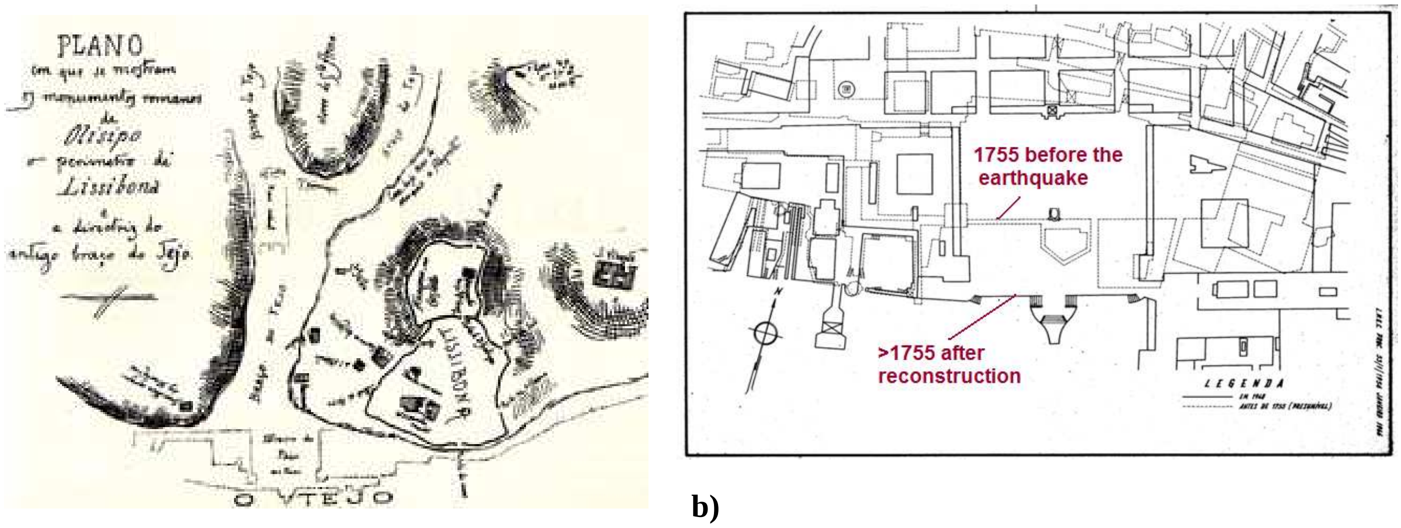
Keywords: Liquefaction assessment , State of Practice, Eurocode 8 – Parte 5, post-liquefaction residual shear strength and shear strain.

1. INTRODUCTION

LNEC researchers are often requested, by different government ministries and private consulting firms, to revise projects of major geotechnical works. For some extreme cases they have to carry out their own studies to provide reference guidelines for external project consulting firms. This was the case of the study for the reinforcement of the alluvium soils adjacent to the Metro tunnel and the West Tower building in Terreiro do Paço, Lisbon (Salgado, 2005, 2007 and 2008a). These studies outstand that the proceedings to assess the potential for liquefaction of alluvium soils, although, fairly well established, there are still some important differences, between the recommendations proposed by the Eurocode 8 – Part 5 and those recommended by the State of Practice (1996 NCEER and 1998 NCEER/ NSF Workshops, Youd et al., 2001). Another very important issue, outstanded by the Terreiro do Paço case is that there is a lack of guidelines, in engineering practice (Pike, R. 2001), or any international code, to study the possible consequences of liquefaction, including guidelines for estimations of residual shear strength and stiffness of potential liquefiable soils.

2. GEOLOGICAL AND HISTORIC SETTING

The "blue line" of Metropolitano de Lisboa (Lisbon Metro) has been expanded from its downtown Chiado's station towards the Tagus's river water front with two additional stations: Terreiro do Paço and St.ª Apolónia (Fernandes et al., 2007). Within the Terreiro do Paço area, in front of the Navy Tower, the metro tunnel cuts an existing fossil valley, of an old creek tributary to the Tagus River, Figure 1. This valley has been filled in, through the times, with alluvium deposits which vary alternatively from clayey and silty sands to sandy and silty clays, which overlay, in turn, formations of the Miocene. As the city of Lisbon grew to the river front these deposits were covered by heterogeneous fills. Before 1755 the water front was located as shown in Figure 1. In November 1755 the city of Lisbon was hit by a major earthquake, M ≈ 8, and downtown was completely destroyed. The reconstructed front line is shown in Figures 1 and 2, where is also shown the temporally embankment that was built, previously to the Metro tunnel construction, for consolidation, confining and uplift restrain purposes.



a) **b)**

FIGURE 1. Lisbon water front: a) before downtown construction, but showing the geometry of the water front after the 1755 earthquake; b) before and after the 1755 earthquake

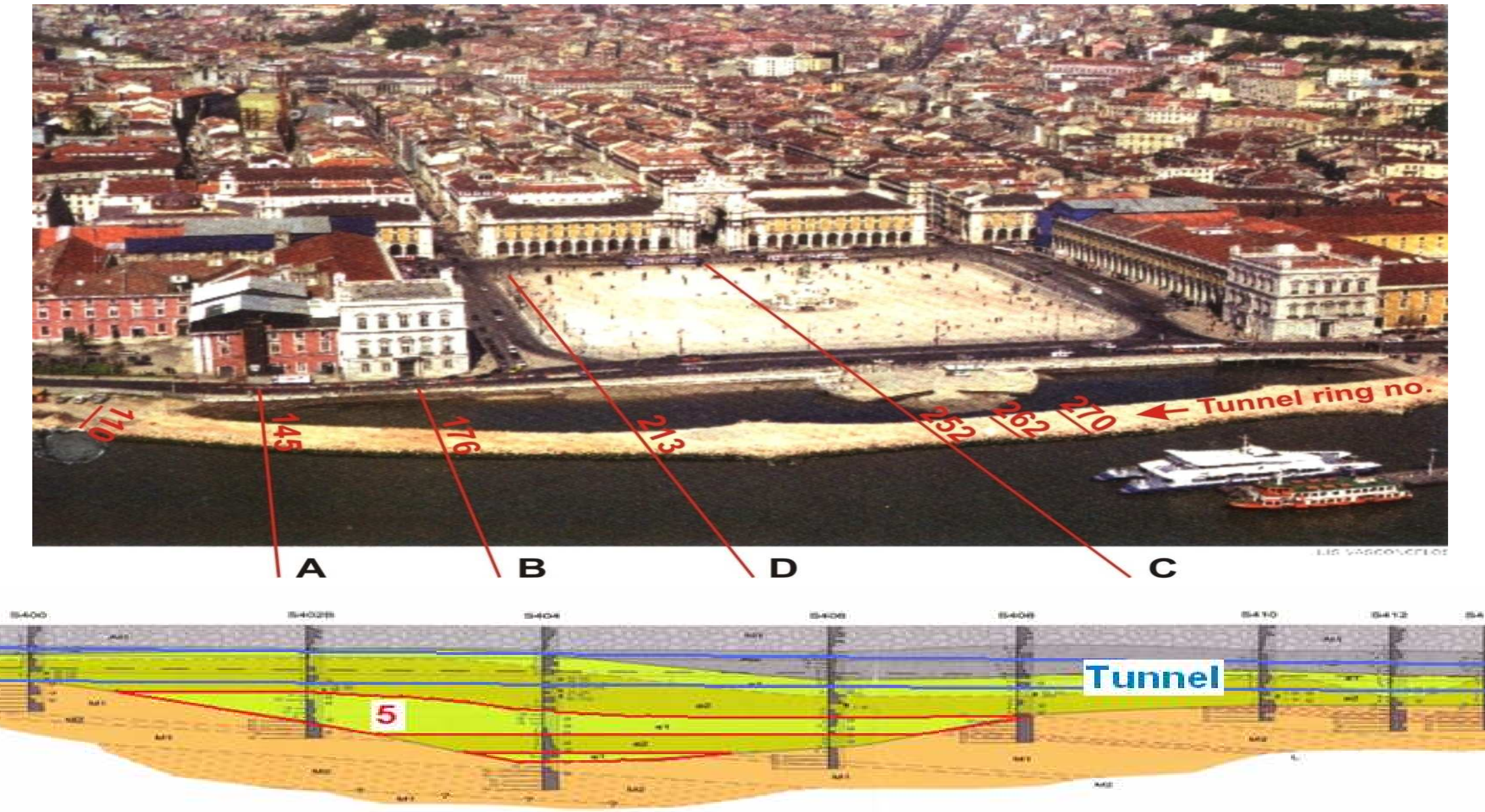


FIGURE 2. Air photograph of Terreiro do Paço (Vasconcelos, L. (Visão, Agosto 2000)). Photograph modified by Salgado (2008a) to show location of cross sections A (tunnel ring 145), B (176), D (213) and C (252). The metro tunnel (Blue line - troço 61) is located under the embankment. The tunnel diameter is about 10 m and the width of each tunnel ring is 1.2 meter. Liquefaction is predicted to occur in soil nº 5 between tunnel ring nº 110 and 270.

3. LIQUEFACTION ASSESSMENTS: COMPARISON BETWEEN THE RECOMENDATIONS BY THE EUROCODE 8 AND BY THE CURRENT STATE OF PRACTICE

Based on in situ (SPT, CPTU, Vs-Cross hole) and laboratory testing (sieve, sedimentation, Atteberg limits, resonant column, dynamic hollow cylinder, static and dynamic simple shear) and following State of o the Art and Practice procedures (Youd et al., 2001) the sandy alluvium deposits correspondent to soil nº 5, located between tunnel ring nº 110 (West side) and nº 270 (East side), are considered to be potential liquefiable (Salgado, 2005) if the site is shaken by an earthquake with high magnitude (M=8). The Factor of Safety against Liquefaction was computed to be between 0.5 and 0.7, i.e. significantly lower then 1.25, which is the limit recommended by the Eurocode 8-Part 5, EC8-P5.

The procedures recommended by the EC8-P5, regarding the liquefaction assessment were also followed. The results show, despite the differences between the EC8-P5 and those recommended by the State of Practice (Youd et al., 2001), that the same zones and about the same levels of Factor of Safety against Liquefaction are obtained. Nevertheless, is considered important to outstand these differences here for reference purposes.

3.1 Factor of safety against liquefaction

A soil is considered liquefiable when the cyclic resistance ratio, CRR, is less or equal to the cyclic stress ratio times a factor of safety against liquefaction, FSL:

$$CRR \leq FSL \,.\, CSR \tag{1}$$

where:

$$CSR = \tau_{av}/\sigma_{vo}' \tag{2}$$

τ_{av}= average cyclic shear stress mobilized by the seismic action
σ_{vo}' = effective vertical stress before the seismic action.

3.2 Estimations of CRR

The EC8-P5 recommends that:

$$CRR = CRR_{7.5} \,.\, MSF \tag{3}$$

Where $CRR_{7.5}$ is the cyclic resistance ratio correspondent to an earthquake with magnitude, M = 7.5 and MSF is the magnitude scaling factor.

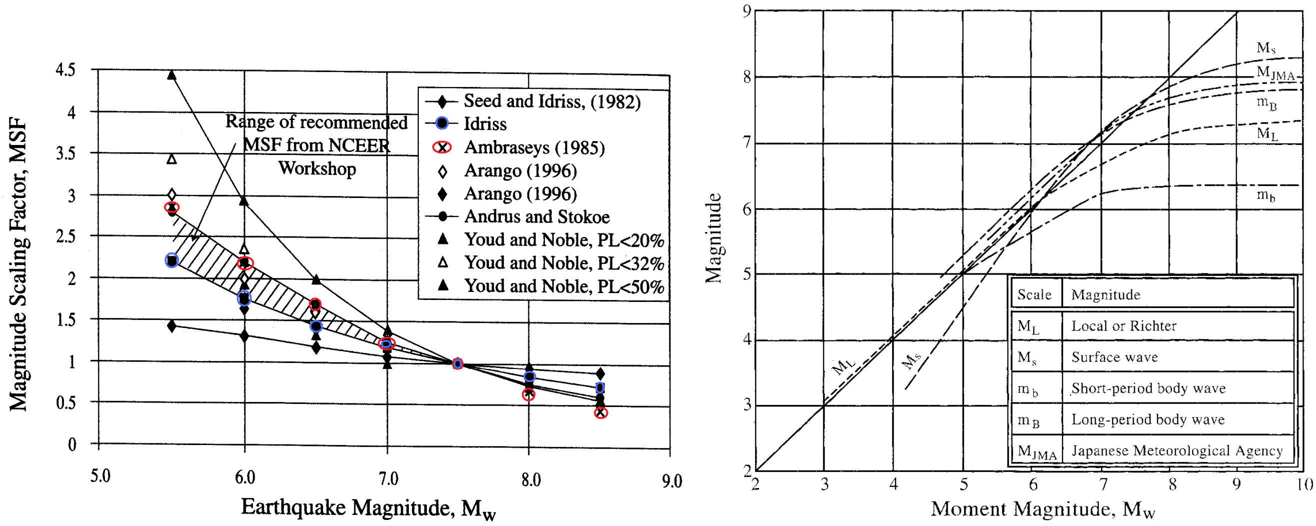
The State of Practice recommends that:

$$CRR = CRR_{7.5} \,.\, MSF \,.\, K\alpha \,.\, K\sigma \tag{4}$$

were $K\alpha$ is the static shear stress correction factor and $K\sigma$ is the overburden correction factor.

Both EC8 and the State of Practice recommend the use $CRR_{7.5}$ as the reference resistance ratio. Based on the work developed by Seed (1983) $CRR_{7.5}$ can be estimated from in situ SPT and CPT test data based on historic data correspondent to level ground conditions and confining effective vertical stress of about 1 atmosphere.

To take into account different magnitude values both proceedings recommend the use of the scaling factor MSF. However, one of the differences is the recommended MSF value to consider. The EC8-P5 recommends Ambrasey‘s (1988) scaling factor, while, Youd et al. recommends Idriss (1995) scaling factor. Several relationships between MSF and M_w are presented in Figure 3, where M_w is referred as the moment magnitude. Relationships between M_w and other magnitudes scales are presented in Figure 4.



a) **b)**

FIGURE 3. a)– Relationships between M_w and MSF (Youd et al., 2001); b) Relationships between Mw and other magnitude scales (Heaton et al., 1982 as referred by Youd et al., 2001)

For the case under study two different earthquake sources were considered, defined based on the EC8 (Serra, 2002), namely a nearby source characterized by a magnitude, M=5.9 and maximum base acceleration, a_{max} = 269 cm/s^2 and a distant source characterized by a magnitude, M=8.0 and maximum base acceleration, a_{max}= 160 cm/ s^2. The magnitude scaling factors, MSF, recommended to be used, by the EC8-P5 and the State of Practice, are as shown in Table 1.

TABLE 1. Recommended MSF values by the EC8-P5 and the State of Practice

Magnitude	MSF		(1)/(2)
	EC8 (1)	State of Practice (2)	
5.9	2.27	1.80	1.261
8.0	0.67	0.84	0.798

Seed (1983) to expand his empirical approach to sloping ground and higher levels of confining stresses developed the factors Kα and Kσ which are not considered in the EC8 proceedings but are referred in the State of Practice (Youd et al., 2001) and included in eq.(4).

Kα is a factor to take into account the effect of static bias mobilized by the sloping ground. Because there is, yet, no agreement to which value to use and, also, because the slopes at the site in Terreiro do Paço, are about 6%, then, a value of Kα = 1 was considered (Salgado, 2005) in the liquefactions assessment study.

Kσ is a factor to take into account the effect of higher confining stresses. Although the liquefaction resistance increases with increasing confining stress the ratio of this resistance with the confining stress is lower then unit for confining values higher then 1 atmosphere, as is shown in Figure 5. In this figure is also presented the State of the Art (Idriss and Boulanger, 2004) as well as the curves developed by Hynes and Olsen (1999) which were endorsed by the State of Practice (Youd et al., 2001).

The Terreiro do Paço study (Salgado, 2005) adopted Hynes and Olsen work, and Kσ was estimated by the following equation:

$$K\sigma = (\sigma'_{vo}/Pa)^{f-1} \tag{5}$$

using f = 0.75, which corresponds to the alluvium soil nº 5 (relative density, Dr≈50%), then, Kσ = $(\sigma_{vo}'/Pa)^{-0.25}$. Using the range of σ'_{vo} between 160 to 220 kPa, computed for Terreiro do Paço, then, Kσ varies between 0.89 and 0.82, with an average value of 0.855.

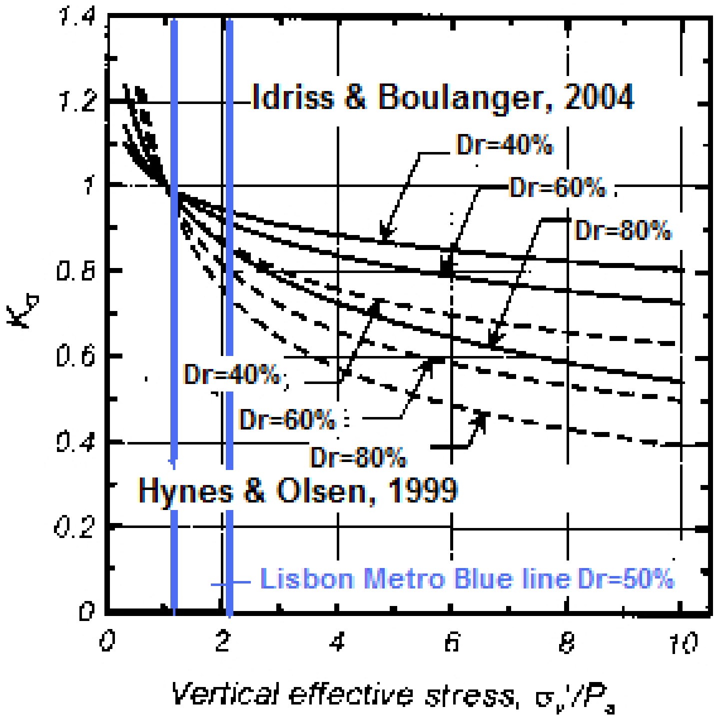


FIGURE 5. Kσ versus (σ'_{vo}/Pa), after Idriss and Boulanger (2004)

The CRR values, in terms of $CRR_{7.5}$, estimated by the two approaches, EC8 (from eq. (3)) and State of Practice (from eq. (4), with Kσ = 0.855), are presented in Table 2.

TABLE 2. Recommended CRR values by the EC8 and the State of Practice

Magnitude	CRR		(1)/(2)
	EC8 (1)	State of Practice (2)	
5.9	2.27 $CRR_{7.5}$	1.54 $CRR_{7.5}$	1.474
8.0	0.67 $CRR_{7.5}$	0.72 $CRR_{7.5}$	0.931

It may be seen that when using the EC8 proceedings the liquefactions resistance correspondent to M=5.9 is 1.47 higher then the estimated by the State of Practice proceedings, however, when considering M=8 the EC8 resistance drops to 0.93 of the computed by the State of Practice.

3.3 – Estimation of CSR

According to the State of Practice (Youd et al., 2001) a simplified procedure to estimate CSR is the proposed by Seed, H.B. and Idriss, I.M. (1971), namely:

$$CSR = \tau_m/\sigma_{vo}' = 0,65\ (a_{max}/g).(\sigma_v/\sigma_{vo}').rd \qquad (6)$$

where τ_m is the characteristic value of the shear stress mobilized by the seismic action; σ_v is the total vertical stress; σ_{vo}' is the effective vertical stress; g is the acceleration of gravity; a_{max} is the maximum surface acceleration and rd is the reduction factor with depth. For routine practice and noncritical projects the following equations may be used to estimate average values of r_d (Liao and Whitman, 1986):

$$r_d = 1.0 - 0.00765\ z \quad (\text{for } z \leq 9.15 \text{ m}) \qquad (7a)$$

$$r_d = 1.174 - 0.0267\ z \text{ for } 9.15\text{m} < z \leq 23 \text{ m} \qquad (7b)$$

However, the Terreiro do Paço case is not a routine study. The area under study is vast (length of the tunnel is about 360 m and the thickness of the alluvium in the vicinity of the tunnel varies from 20 to 50 m), therefore it is difficult to quantify a single value for a_{max} due to the expected effects of local amplification or de-amplification of the local acceleration. Therefore, it was decided that the estimations of CSR would be carried out by one-dimensional dynamic analyses using the latest version of the program SHAKE. This program was originally developed by Schnabel et al. (1972) and later modified by Idriss and Sun (1991) and is referred here as SHAKE91.

The required input data for the analysis is listed below:

a) Maximum bedrock (base) acceleration, $(ab)_{max}$, expected in depth and a set of at least 3 accelerogrames defined according to the response spectra as recommended by part 1-1 of the European code 8 for a soil of class A and within the Portuguese seismic zone A;

b) Definition of the estratigraphy and type of soils above bedrock;

c) Characteristic values for unit weight, γ, and plasticity index, Ip, correspondent to each type of soil;

d) Distribution in depth of the maximum shear modulus, Gmax;

e) Estimations for the degradation of the shear modulus with the increase of the cyclic shear strain mobilized by the seismic action;

f) Estimations of the increase of coefficient of damping, β, with the increase of the cyclic shear strain mobilized by the seismic action.

These data were estimated as follows:

a) Maximum base (bedrock) acceleration, $(ab)_{max}$, and ten (10) artificial accelerograms were developed by LNEC (Serra, 2002). Two types of seismic actions were considered: i) seismic action 1, E1, with $(ab)_{max}$ =269cm/s^2, correspondent to a moderate earthquake located at short focal distance; ii) seismic action 2, E2, with $(ab)_{max}$ =160cm/s^2 correspondent to an earthquake with high magnitude and greater focal distance.

b) The geological stratigraphy and the type of soils were supplied by FERCONSULT (2002), based on the in situ results provided by the boreholes of the series 400 (Teixeira Duarte, 2001).

c) The unit weight,γ, and the plasticy index, IP, values presented in Table 3 were considered by LNEC (Salgado, 2004) for the analysis.

d) The characteristic values of Gmax were estimated using equation (8) where Vs is the shear wave velocity measured in situ, γ, the unit weight and g, the acceleration of gravity.

$$G_{max} = (Vs^2 . \gamma)/g \tag{8}$$

The in situ measurements of Vs were carried out by LNEC (2002). The measurements were carried out every 1.0m following the “cross-hole” methodology at four (4) locations near by the boreholes S400, S404, S406 and S414. The location of these holes is shown in Figure 6 and 7 (section B). The results obtained are in Figure 8. Estimations of Vs for the locations of the other boreholes (S401, S402B, S403, S405, S407, S408, S409A, S410, S411, S412, S413 e S415 were carried out by Salgado (2004, 2008) based on the measured Vs and correlations with other local geotechnical characteristics. This issue is addressed in section 3.3.1.

e) and f) Estimations of the degradation of Gmax and the increase of β with increasing cyclic shear strain were based on published values (Vucetic, M and Dobry, R, 1991) as shown in Fig. 9. These curves were validated by laboratory testing carried out at IST (Santos, J.A. and Lopes, I., 2001) on samples retrieved from the local alluvium deposits at the near by Metro Station, as presented in Figure 10.

It may be seen that the behavior of the soil samples, from the Terreiro do Paço alluvium tested (IP from 9% to 31%) match well the data reported by Vucetic and Dobry (1991).

TABLE 3. Soil types, unit weight and Plasticity Index

Soil type no. (see Figure 7)	Unit weight, γ (kN/m^3)	Ip(%) (average)
New fill	19	0
1	19	3,7
3	17	16,3
4	17	13,5
5	17	3,3
6	17	14,2
8	21	8,0

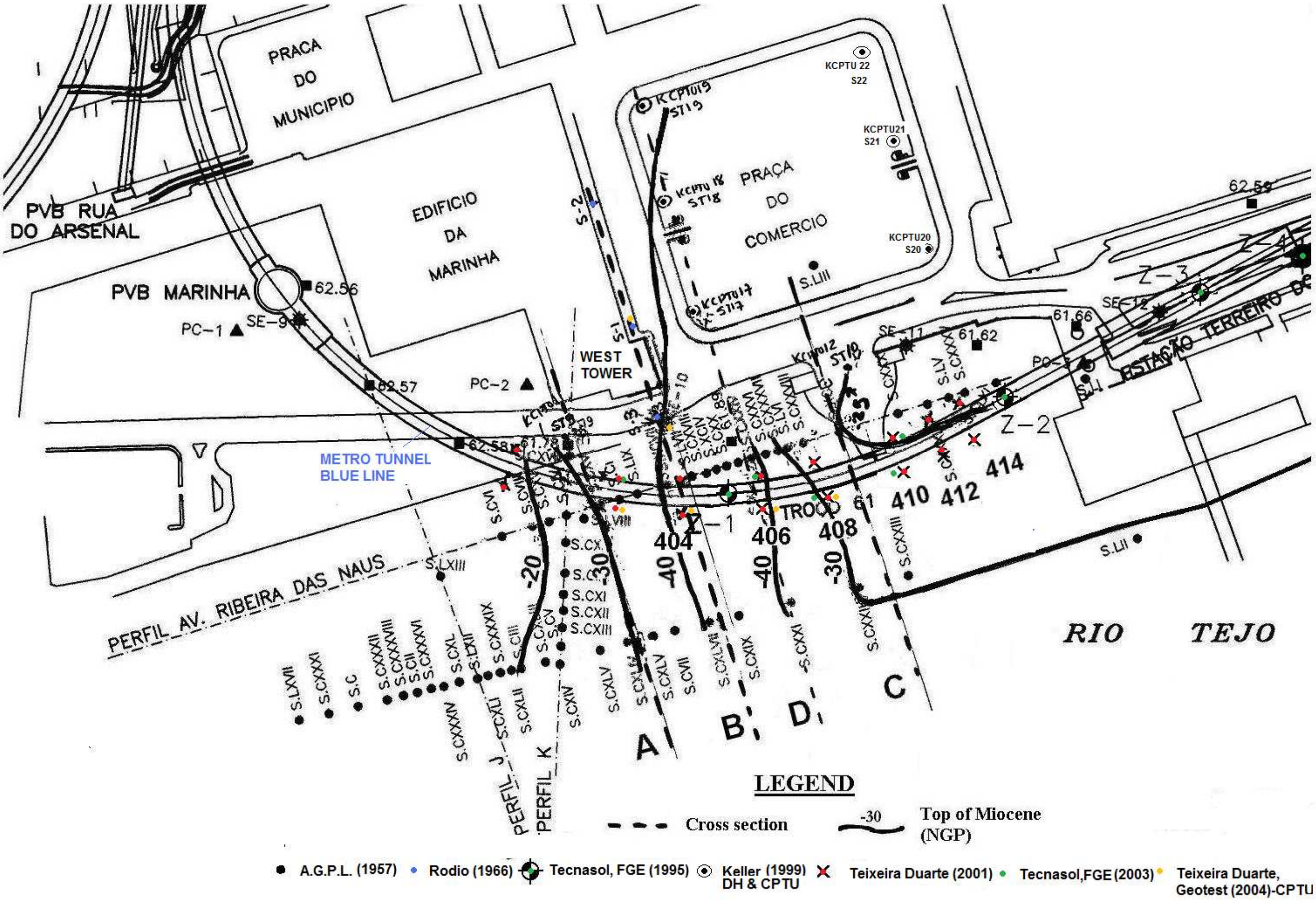


FIGURE 6. Site plant with location of in situ testing (Salgado, 2005, 2008)

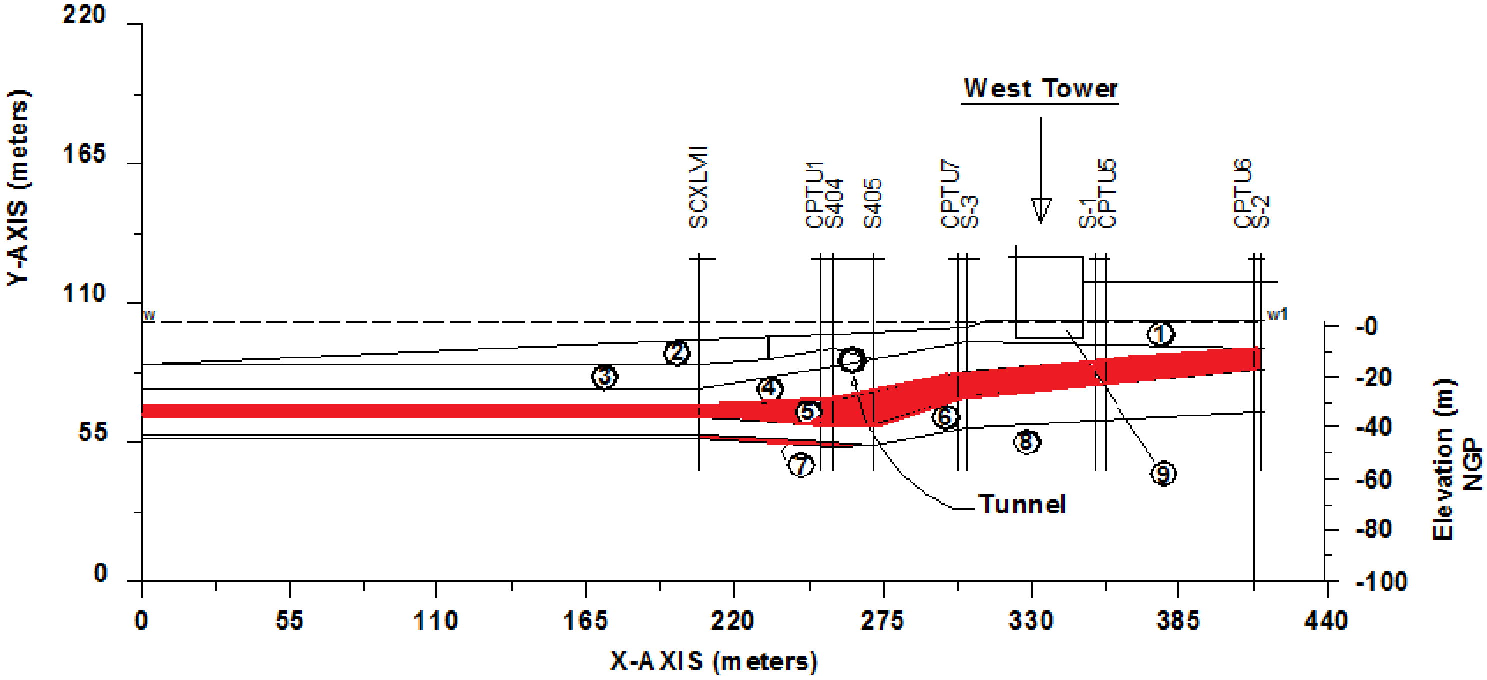


FIGURE 7. Cross section A : Heterogeneous fills (Soil 1); soft organic clay (soil 2); silty Clay to clayey Silt alluviums (soils 3 and 4); potentially liquefiable alluvium sand (soil 5); silty Clay alluvium (soil 6); coarse sand (soil 7); Miocene (soil 8), West Tower stony foundation (soil 9)

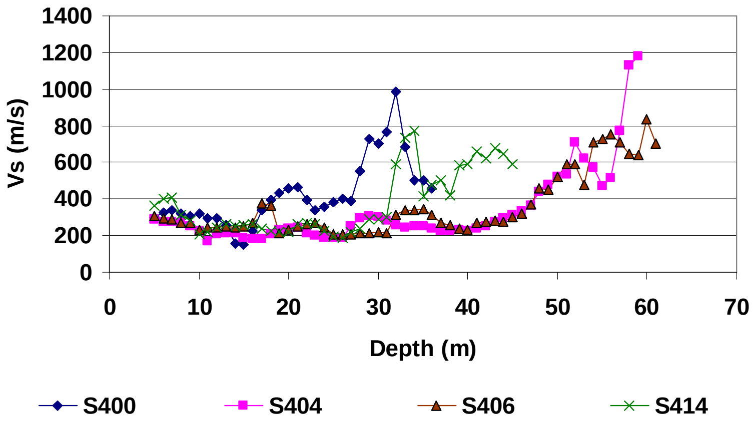


Figure 8. In situ measurements of Vs

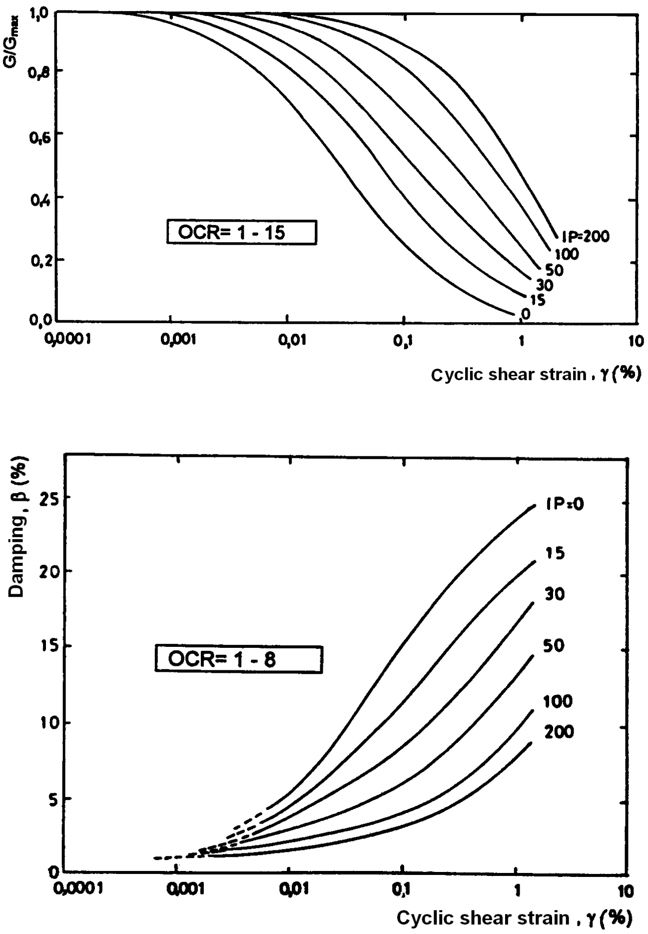


FIGURE 9. Reduction of Maximum shear moduli and increase of damping, β, with increasing cyclic shear strain as a function of plasticity index, Ip, (after Vucetic and Dobry, 1991)

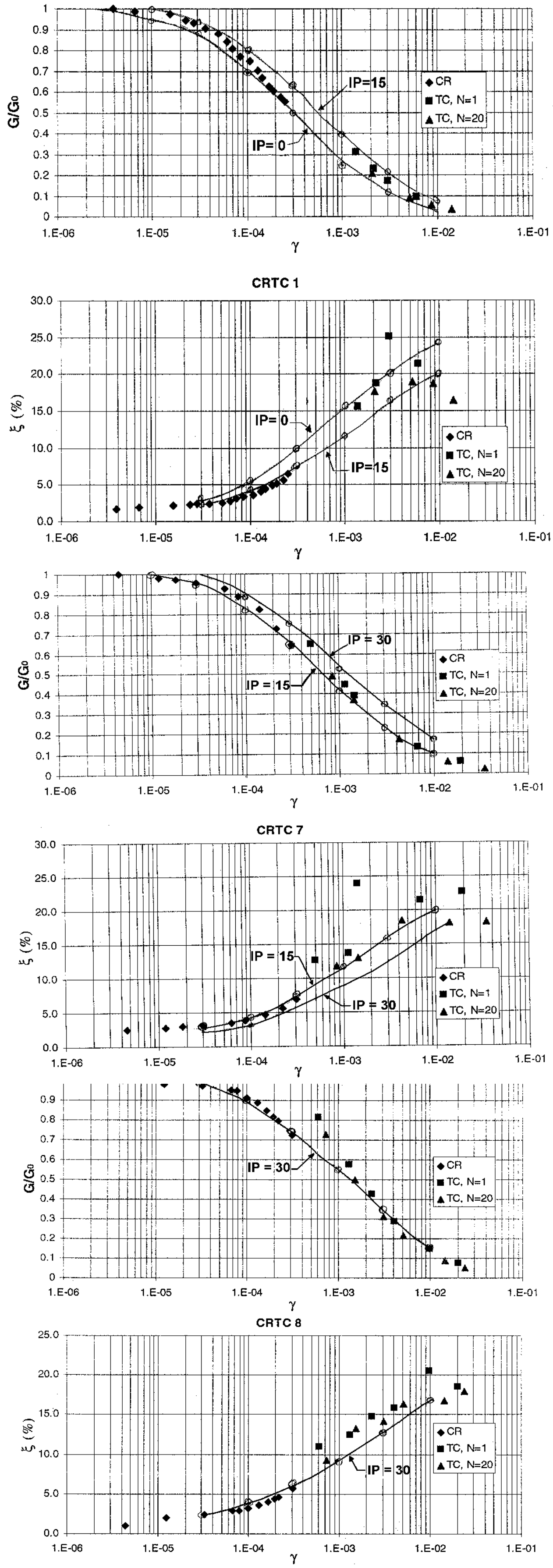


Figure 10. Comparison between laboratory test results (resonant column and cyclic torsional tests) carried out by Santos and Lopes (2001) on alluvium soil samples (IP from 9% to 31%) and Vucetic and Dobry reported data (1991)

3.3.1 – Estimation of Vs from CPT test data

Proceedings, that account for the soil type (cohesive or non-cohesive), for the estimation of Vs from the measurements of the point resistance, q_c, and lateral, fs, obtained during a CPT test were developed (Salgado 2005, 2008b) and used for the assessment of the liquefaction potential of the alluvium soils located in the vicinity of the underground tunnel of Metropolitano de Lisboa located in Terreiro do Paço.

The estimatiom of Vs is obtained through G_{max}. From eq. (8):

$$Vs = (G_{max}\,(g/\gamma))^{1/2} \tag{9}$$

Non-cohesive soils

Based on the work carried out by Hardin and Drnevich (1972) and Seed and Idriss (1970), Seed et al. (1986) show that:

$$G_{max} = 1000\,(K_2)_{max}\,(\sigma'_m)^{1/2} \quad \text{in psf} \tag{10}$$

where σ'_m is the mean efective stress and $(K_2)_{max}$ is a coeficient that reflects the relative density of the soil that can be estimated by the following equation:

$$(K_2)_{max} = 20(N_1)_{60}^{\ 1/3} \tag{11}$$

where $(N_1)_{60}$ is the normalized SPT N value corrected to a hammer energy of 60%. Equation (9) is expressed in psf units and can be converted to a general equation, as a function of the atmospheric pressure, pa:

$$G_{max} = 21{,}7\,(K_2)_{max}\ pa\,(\sigma'_m/pa)^{1/2} \tag{12}$$

then, substituting in (8) we can obtain Vs from $(N_1)_{60}$:

$$Vs = (21{,}7\,(20\,(N_1)_{60}^{\ 1/3})\ pa\,(\sigma'_m/pa)^{1/2}\ (g/\gamma))^{1/2} \tag{13}$$

To estimate Vs from CPT data is, then, necessary to estimate $(N_1)_{60}$ from the same CPT data. Several researchers have been developing such correlations as the presented in Figure 11, after Terzaghi et al. (1996). From the data reported from Seed and De Alba (1986), the following relationship ie developed:

$$q_c/N_{60} = 600\,(D_{50})^{0{,}228} \quad \text{in kPa} \tag{14}$$

where q_c is the cone point resistance and D_{50} is the mean grain size of the soil.

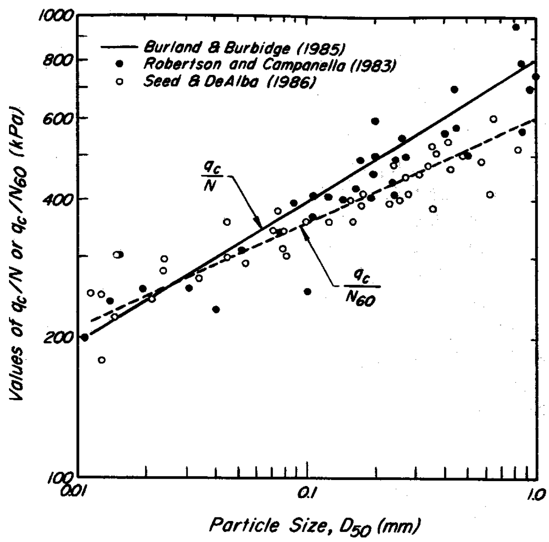


FIGURE 11. Correlations between D_{50} and the ratio between qc and N_{60} (Terzaghi et al. 1996)

Salgado (2005, 2008b), using test data from the alluvium samples of Terreiro do Paço, developed the following correlation between the fines content, FC and D_{50} (Figure 12) :

$$FC = 4{,}79\ D_{50}^{\ -0{,}856}$$

or

$$D_{50} = 10^{\ ((\log FC - \log 4{,}79)/-0{,}856)} \quad (15)$$

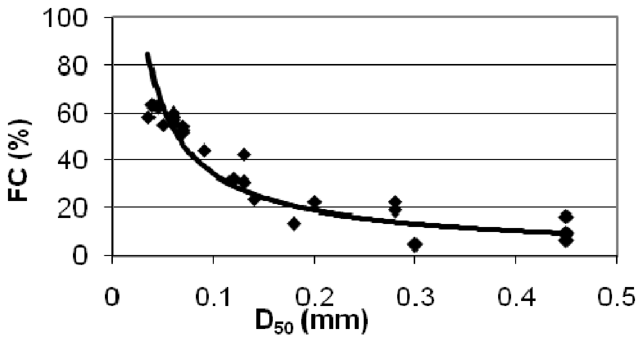


FIGURE 12. Correlation between D_{50} and FC (Salgado, 2005, 2008b)

Based on the work by Robertson (1990) the relationship between FC and Ic, the soil behavior type index, is given by:

$$FC = 0.0, \text{ when } Ic < 1.26 \quad (16a)$$

$$FC\ (\%) = 1.75\ Ic^{\ 3.25} - 3.7, \text{ when } 1{,}26 \leq Ic \leq 3{,}5; \quad (16b)$$

$$FC = 100.0, \text{ when } Ic > 3.5 \quad (16c)$$

The value of Ic (Robertson 1990) is given by:

$$Ic = ((3{,}47 - \log Q)^{2} + (\log F + 1{,}22)^{2})^{\frac{1}{2}} \quad (17)$$

where Q is the normalized point resistance of the cone:

$$Q = ((qc - \sigma_{vo}) / pa)\ (pa / \sigma'_{vo}) \quad (18)$$

and F the normalized friction:

$$F = (fs / (qc - \sigma_{vo}))\ x\ 100\ \% \quad (19)$$

where fs is the correspondent friction sleeve.

Therefore, using eq.s (14) to (19) it is possible to estimate N_{60} from the CPT test data (qc and fs). To obtain $(N_1)_{60}$ the following equation by (Liao e Whitman 1986):

$$(N_1)_{60} = N_{60}\ (pa/\sigma'_{v}) \quad (20)$$

The results estimated with the above procedures were compared (Figure 13 and Table 4) with the results obtained with the procedures proposed by Lunne et al. (1997):

$$qc_1/(N_1)_{60} = 0,85\ (1 - Ic/4,6) \tag{21}$$

where $qc_1 = qc\ (pa/\sigma'_v)$

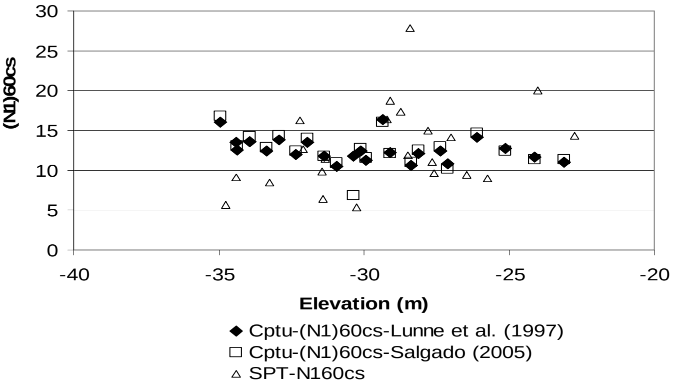


FIGURE 13. $(N_1)_{60cs}$ from SPT and CPT test data

TABLE 4. $(N_1)_{60cs}$ average values (Figure 13 data)

CPTU - Lunne et al. (1997)	CPTU - Salgado (2005, 2008b)	SPT
12.574	12.572	12.736

The test data considered was obtained in 3 CPTu tests (CPTu1R, CPTu2 e CPTu3, Teixeira Duarte 2004), and the SPT test data from 7 test holes (S402 to S407, Teixeira Duarte, 2001. The N_1 data was corrected to take into account the fines content following the procedures proposed by Idriss (Youd et al. 2001) and are referred by $(N_1)_{60cs}$.i.e., correspondent to a clean sand with fine content of 5% or less.

Cohesive soils

Weiler (1988) shows that the correlation between $G_{max,}$ and the undrained shear strength, c_u, can be expressed by the following equation:

$$G_{max} = k\ c_u \tag{22}$$

where k is a function of the plasticity index, IP, and the overconsolidation ratio, OCR, as presented in Table 5.

TABLE 5. Ratios for $(Gmax/c_u/)$, Weiler (1988)

IP(%)	OCR=1	OCR=2	OCR=3
15-20	1500	1250	1000
20-25	1100	650	800
35-45	600	520	450

Using the data correspondent to OCR=1, then Figure 14 was developed. Preliminary analysis carried out by Salgado (2005) considered k= $(Gmax/c_u)$ = 1500 correspondent to the silty clay alluvium (Ip=14%). Later the work was refined (Salgado, 2008b) using CPTu and Vs test data and it shows that, for IP values between 13.5 and 16.2, $k \approx 1750$. Therefore, knowing c_u is possible to estimate Vs using the following equation:

$$Vs = (G_{max}\ (g/\gamma))^{1/2} = (k\ c_u\ (g/\gamma))^{1/2} \tag{23}$$

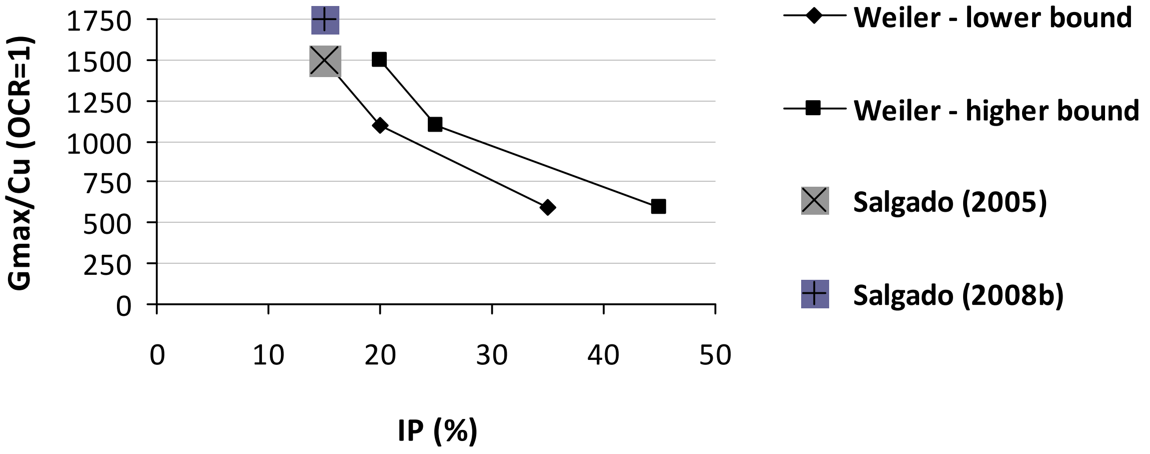


FIGURE 14. Correlation between IP and (G_{max} / c_u) for OCR=1

Then, the next step is to estimate Cu from CPT test data. The analysis of the undrained shear strength from CPT test data has been carried out using the following relationship:

$$N_{KT} = (q_t - \sigma_{vo}) / c_u \tag{24}$$

where q_t represents the total stress, given by:

$$qt = q_c + u_t\,(1\text{-}a) \tag{25}$$

where u_t is the total dynamic pore pressure (u_0 + Δ u) and "a" is net ratio area of the cone (Roberson and Camapanella, 1982). In the present study a = 0,76. Based on published work by Hamza et al. (2005), N_{KT} can vary between 10 and 30 with increasing Ip, as presented in Figure 15.

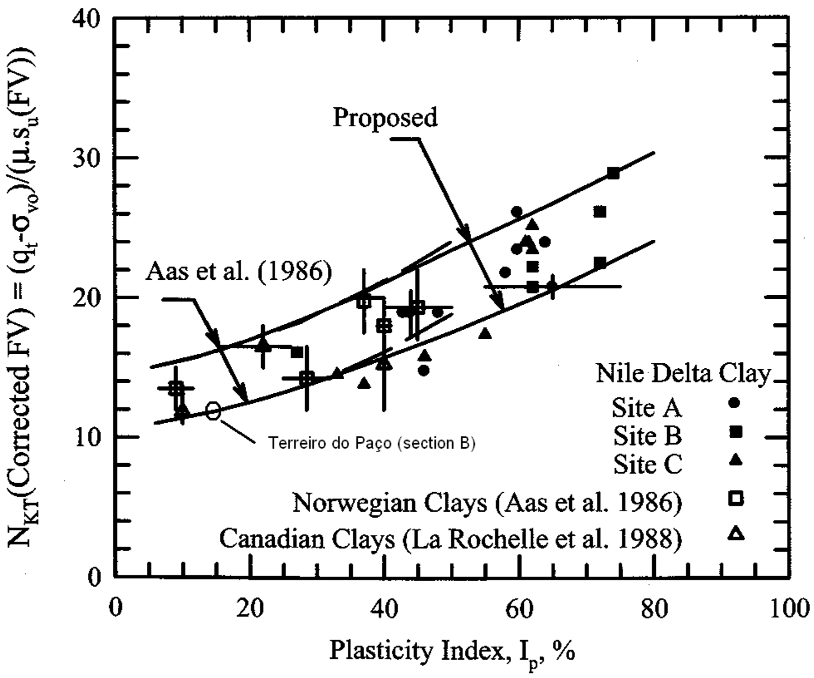


FIGURE 15. Correlations between Ip and N_{KT} considering field vane test data corrected with Bjerrum's factor (Hamza et al. 2005)

For the Terreiro do Paço study case using the field Vane and CPT test data an average value of N_{KT} = 12 was obtained (Salgado, 2005, 2008b), considering a value of μ = 1,04 from Figure 16 (Bjerrum, 1972).

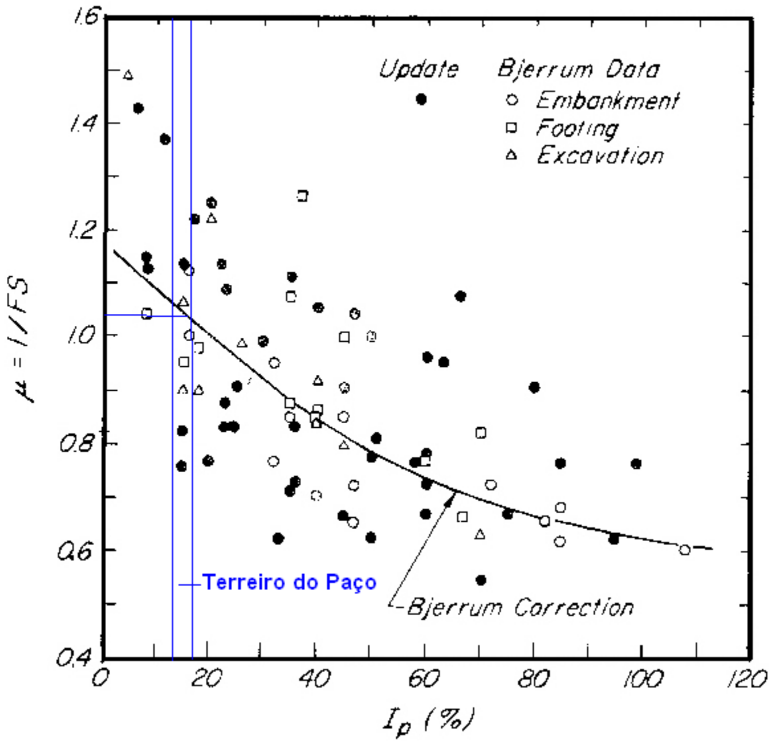


FIGURE 16. Bjerrum’s field vane correction factor

For calibration purposes in Figure 17 is presented values of cu obtained from field Vane tests, simple shear (SS) laboratory test results as well as values of cu estimated from CPTu test data, using N_{KT} = 12, and Vs field (CH) test data using k=1750. The data corresponds to in situ testing carried out along cross section B (Figure 18). A good correlation was obtained.

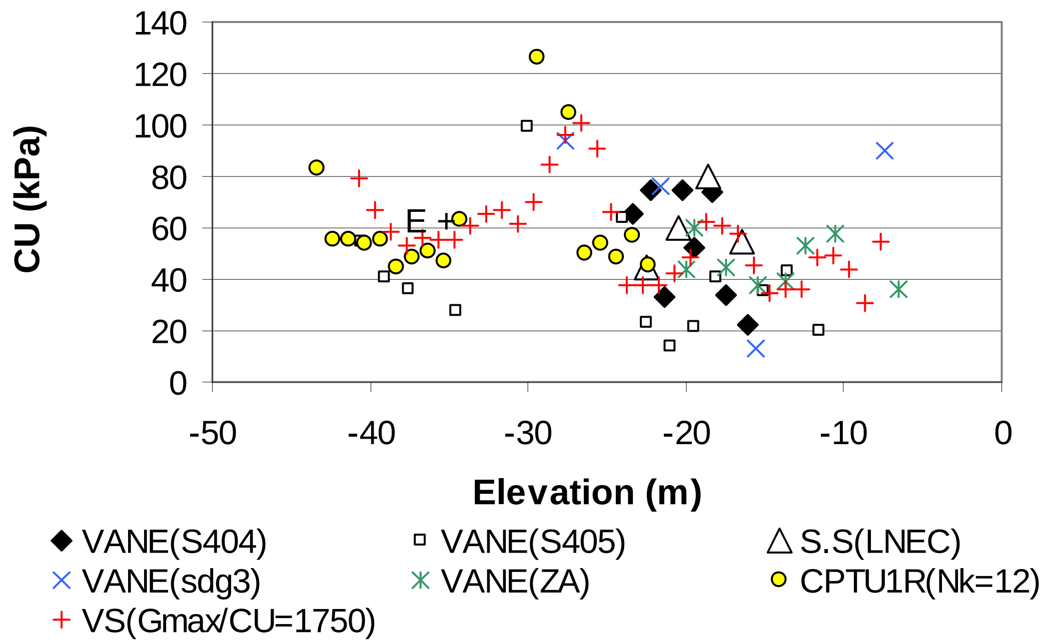


FIGURE 17. Undrained shear strength, cu, test data (Salgado, 2005, 2008b)

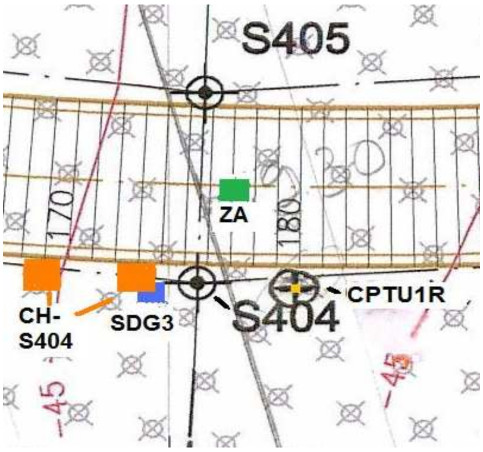


FIGURE 18. Site location of field tests (Salgado, 2005, 2008)

Then, to estimate the distribution of Vs from CPT test data we can use equation (13) for noncohesive soils and equation (23) for cohesive soils. To differentiate when the CPT cone is going through sandy soils or clayey soils, we can use the Soil Behavior Type Index, Ic, developed by Robertson (1990) as presented in Table 6 and Figure 19.

TABLE 6. Boundaries of soil behaviour type (Robertson, 1990)

Soil Behaviour Type Index, Ic	**zone**	**Soil Behaviour Type**
Ic<1,31	7	Areia com gravilha a areia densa
1,31<Ic<2,05	6	Areias: areia limpa a siltosa
2,05<Ic<2,60	5	Misturas arenosas: areias siltosas a siltes arenosos
2,60<Ic<2,95	4	Misturas siltosas: siltes argilosos a argilas siltosas
2,95<Ic<3,60	3	Argilas: argilas siltosas a argilas
Ic> 3,60	2	Solos orgânicos: turfas

Based on the Soil Type behavior index, Ic, sandy soils (non-cohesive) are characterized by Ic ≤ 2,60 (i.e. FC≤ 35%) and equation (12) will be used, and clayey soils are defined by Ic > 2,6 (i.e. FC> 35%) and equation (22) will be used.

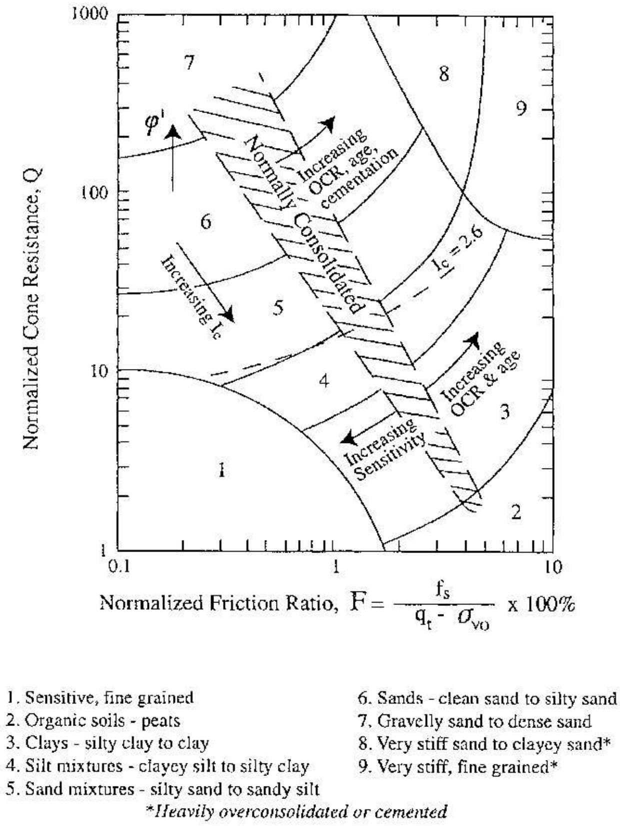


FIGURE 19. CPT-Based Soil Behavior-Type Chart proposed by Robertson (1990)

In situ Cross Hole (CH) test data correspondent to S404 and S406 (see Figures 6 and 18) are presented in Figures 20 and 21 respectively. In the figures is also presented the distribution of the fines content, FC, using red color for the clayey soils and blue color for sandy soils. It may be seen that a good agreement was obtained between the Vs data estimated from the CPT test data and the measured CH-Vs.

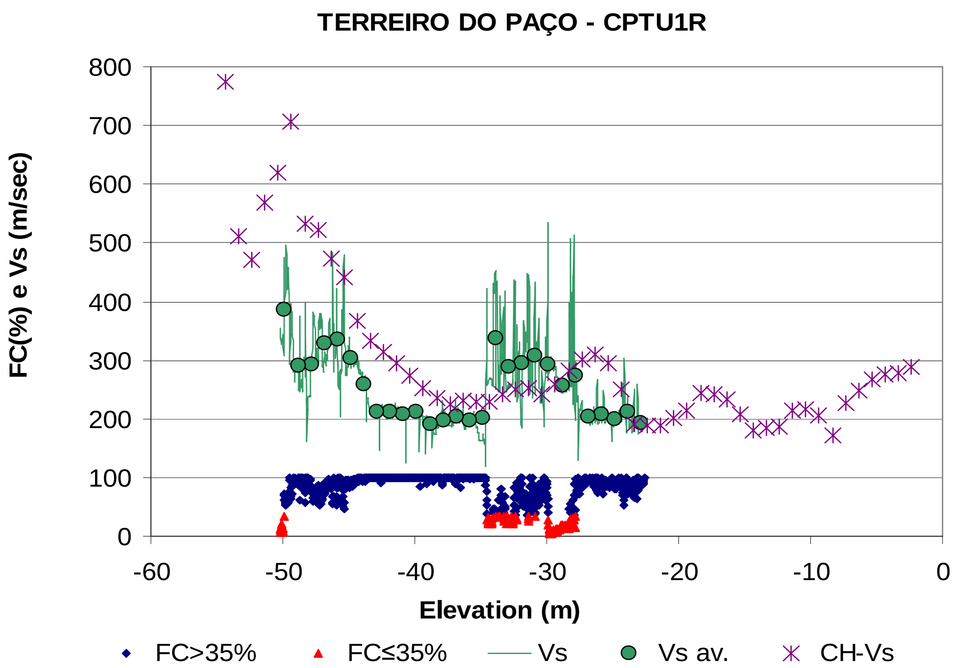


FIGURE 20. Comparison between CH Vs test data and predictions using CPT test data (S404 and CPTu1R)

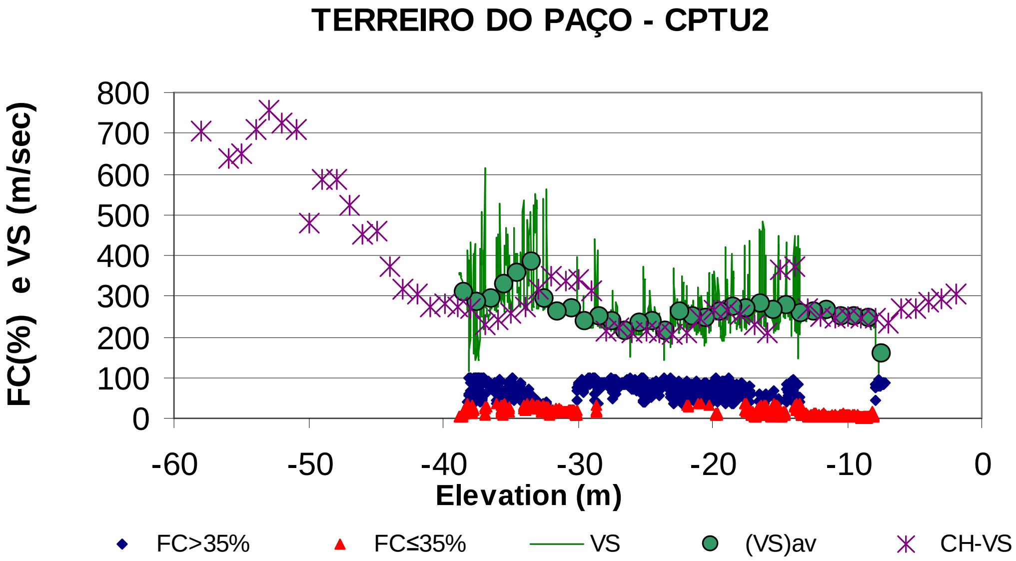


FIGURE 22. Comparison between CH Vs test data and predictions using CPT test data (S406 and CPTu2)

3.4 – Results

3.4.1 – Acceleration and CSR

To illustrate the results obtained from the dynamic Shake analysis in Figures 23 and 24 are presented respectively the distributions of accelerations and CSR versus elevation correspondent to the location of S404.

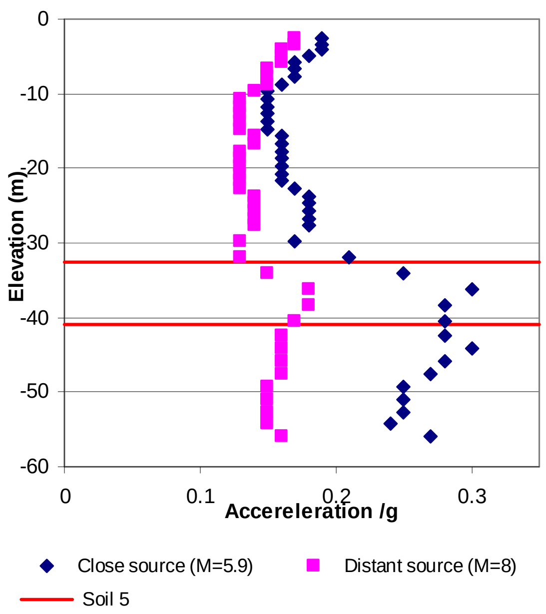


FIGURE 23. Distribution of acceleration (S404)

It may be seen that despite the difference between the bedrock acceleration correspondent to the close source, $(ab)_{max}$ = 269 cm/s^2 (M=5.9) and the acceleration correspondent to the distant source $(ab)_{max}$ = 160 cm/s^2 (M=8), the out come in terms of the CSR is reversed due to the site natural frequency, as shown in Figure 24, where the CSR mobilized within the liquefiable soil by the distant source earthquake is higher then the mobilized by the close source earthquake.

To see the influence of MSF (magnitude scaling factor) the values of CSR presented in Figures 24 were divided by the magnitude scaling factors, MSF. In Figure 25 was applied the MSF proposed by Idriss (Youd et al. 2001), where MSF = 1.8 for M=5.9 and MSF = 0.84 for M=8 (see Table 1). However, if the MSF proposed by the EC8 (after Ambraseys, 1985) is considered then Figure 26 is obtained instead.

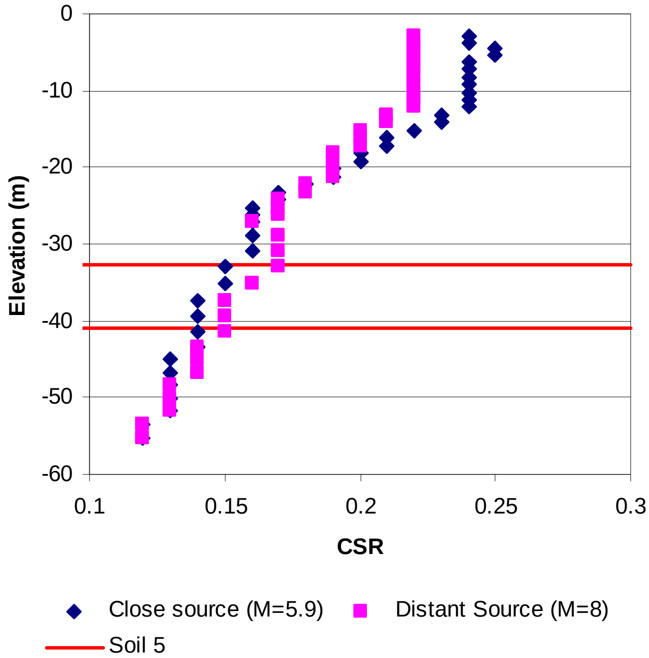


FIGURE 24. Distribution of CSR (S404)

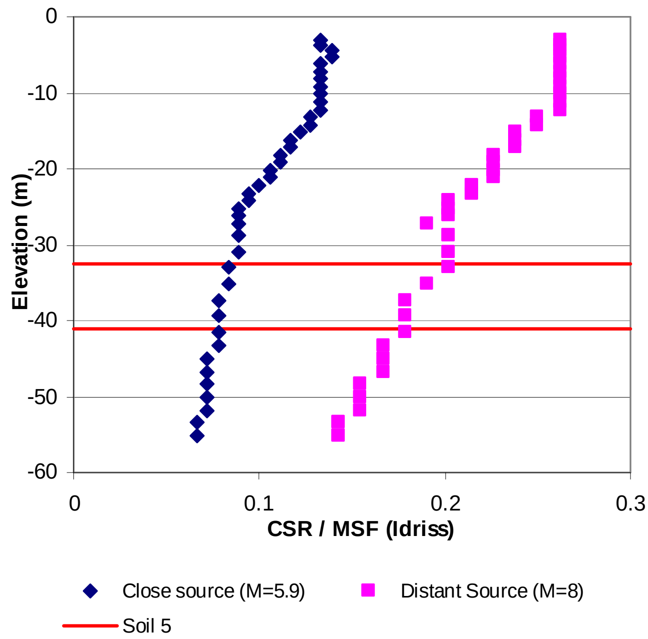


FIGURE 25. S404 - Distributions of CSR / MSF (from State of Practice, 2001 – after Idriss, 1996)

3.4.2 – Assessment of FSL

Comparing the cyclic resistance ratio, CRR, obtained following the procedures outlined by Youd et al. (2001) with the CSR values presented in Figure 24, then the distribution of the Factor of Safety against Liquefaction, FSL, was obtained for the location of all the selected SPT and CPT test holes (Salgado, 2005, 2008a). Plots of

CRR and CSR for the location of Section A (Figure 27), Section B (Figure 28) Section D (Figure 29) and Section C (Figure 31), where are presented the CRR data obtained using different sources (SPT, CPT and Vs).

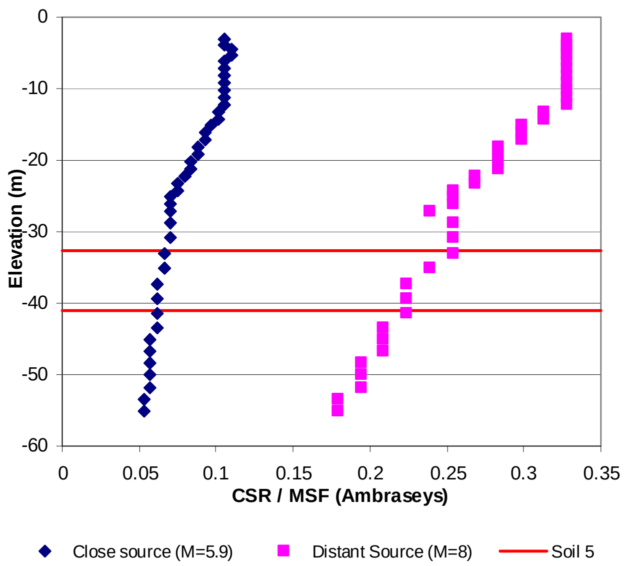


FIGURE 26. S404 - Distributions of CSR / MSF (from EC8, 2004 – after Ambraseys, 1988)

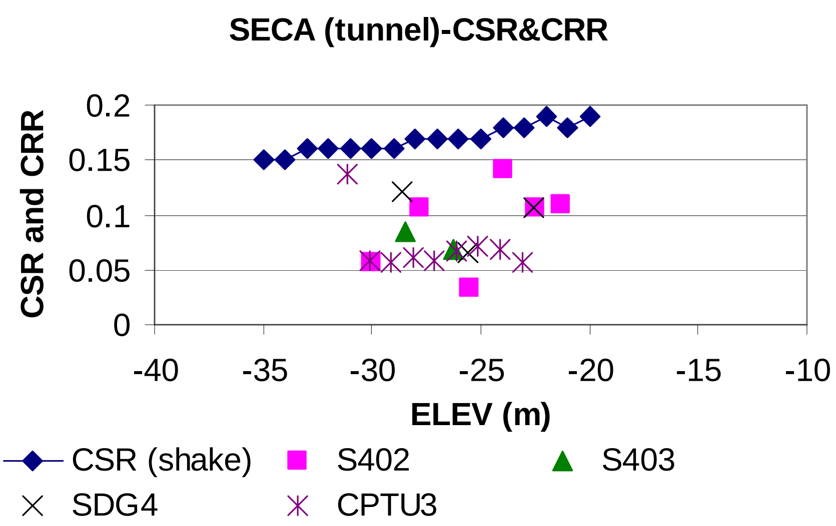


FIGURE 27. Section A: CRR and CSR

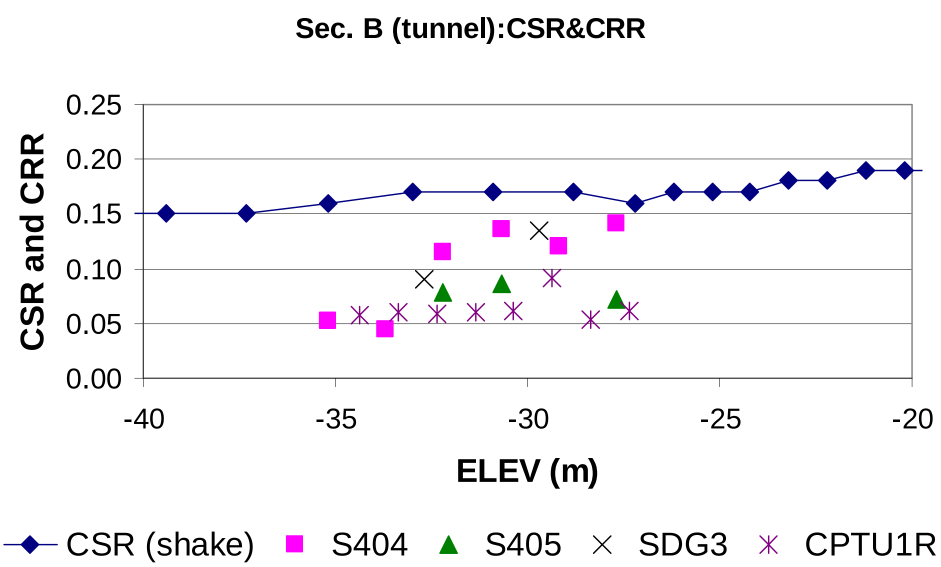


FIGURE 28. Section B: CRR and CSR

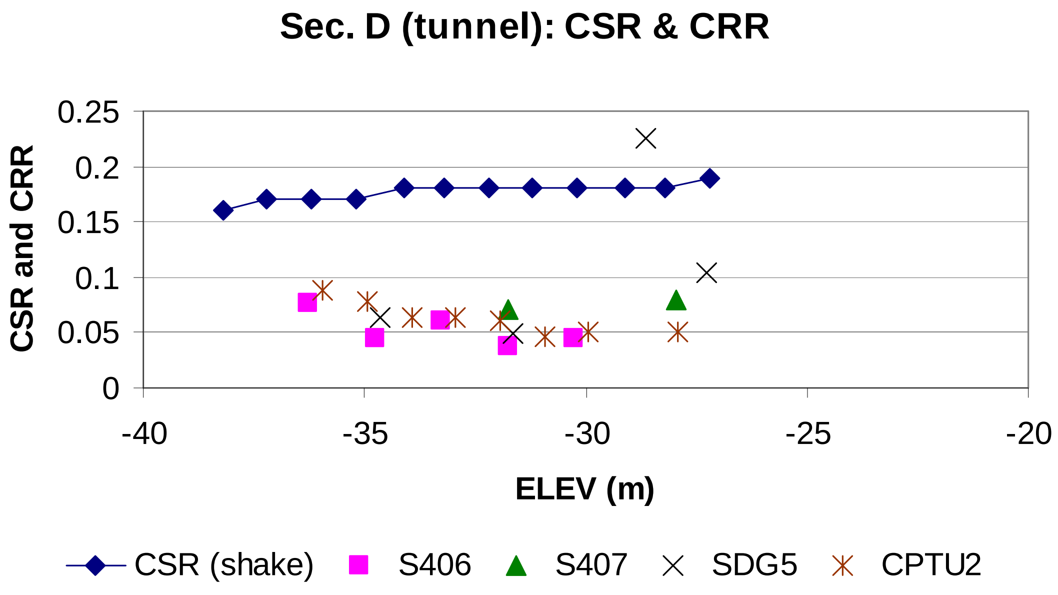


FIGURE 29. Section D: CRR and CSR

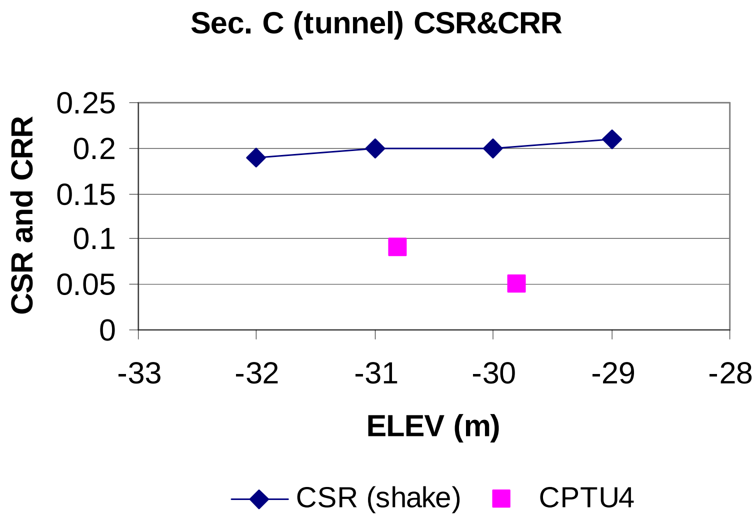


FIGURE 30. Section C: CRR and CSR

It may be seen that FSL, computed using the State of Practice procedures (Youd et al., 2001) varies between 0.5 and 0.7 and is quite lower then, 1.25, which is the limiting value recommended by the EC8. If FSL have been computed using the EC8-P5 procedures, then, soil no. 5 would, also, be considered liquefiable but with FSL varying between 0.47 and 0.65.

4.0 – CYCLIC MOBILITY

4.1 – Liquefaction potential of silty soils

Another important issue was to confirm that the silty alluviums were not liquefiable, and, what it is the correspondent "Cyclic Mobility Potential". The major concern was the possible degradation of stiffness and shear strength of the alluvium soils located in the vicinity of the tunnel (soils 3 & 4, see Figure 7). Static, cyclic and post cyclic simple shear tests were carried out on undisturbed samples taken from the vicinity of the tunnel shaft. The results indicate (Serra, 2008) that there was no reduction in shear strength and in some cases was observed an increase. Also, no reduction of shear stiffness took place.

To complement the assessment the Modified Chinese Criteria (Seed and Idriss, 1982) was considered (Salgado, 2005, 2008a). Later an update was carried out following Boullanger and Idriss (2006) criteria.

Plots of the data used for the assessment is presented in Figures 31 to 33.

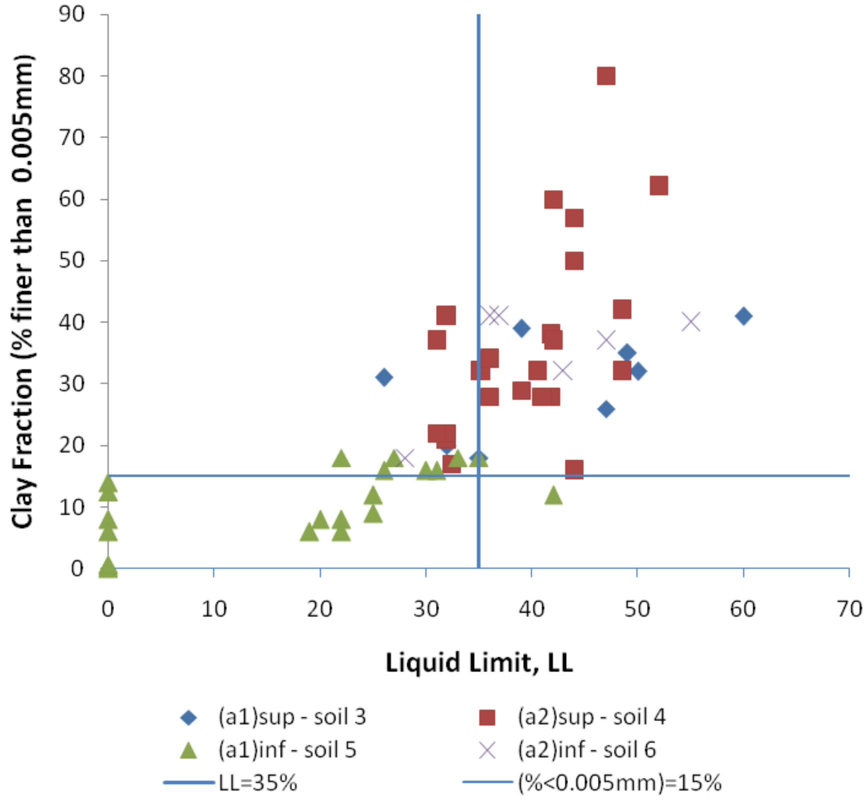


FIGURE 31. Chinese Criteria: LL<35% and W>0.9 LL and Clay fraction <15% (Seed and Idriss 1982)

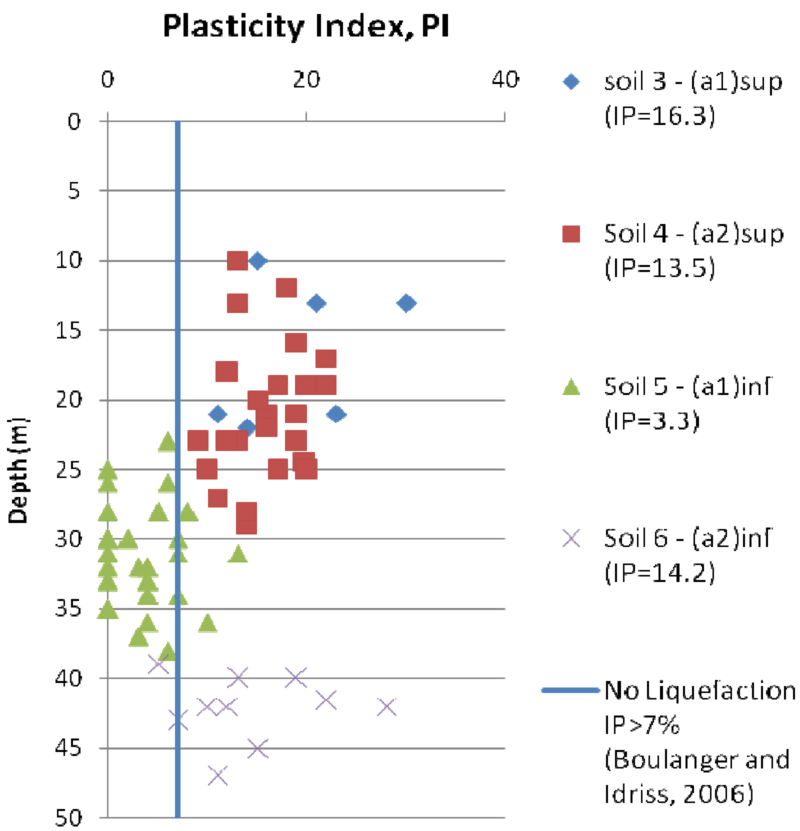


FIGURE 32. Boulanger and Idriss (2006) criteria Sections A, B, D and C

The data presented in Figure 32 was separated in Figure 33 (Salgado, 2008a).

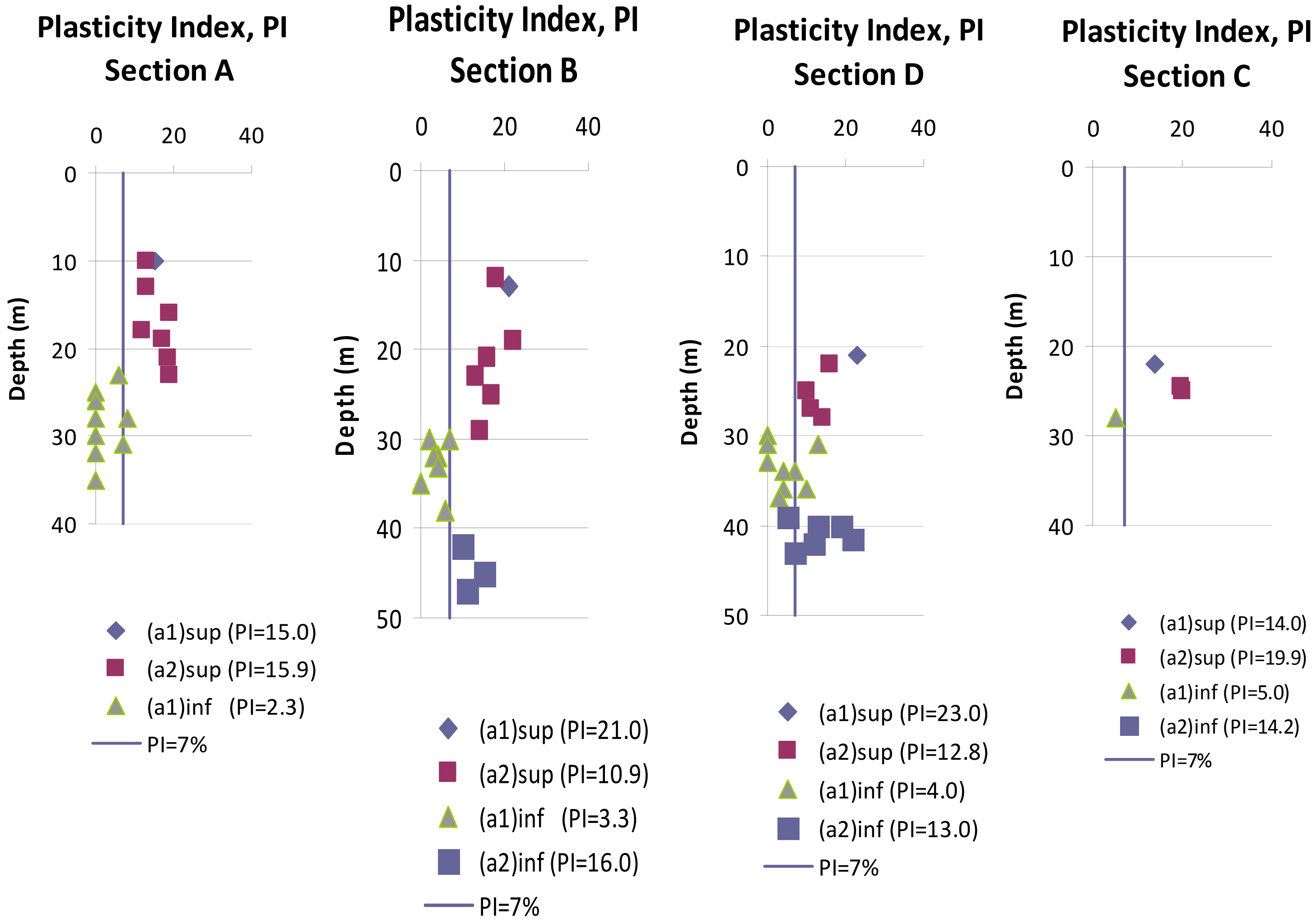


FIGURE 33. Boulanger and Idriss Criteria (2006) – Plot of data, separated, from sections A, B, D and C

4.2 Cyclic mobility of silty and clayey soils

The assessment of the cyclic mobility potential of the silty and clayey alluvium soils (no. 3, 4 and 6) was carried out (Salgado, 2005, 2008a) by “Block Dynamic” analysis assuming that the liquefiable sandy alluvium would be treated, therefore not interfering with the results. The analyses were carried out using a 1 degree of freedom dynamic model (Salgado, 1981) following Newmark´s approach (1965). The results obtained are presented in Figures 34 and 35 where are presented results published by the several researchers listed. The yield acceleration is designated by N. The analysis indicate that the displacements for sections A and B are sensitive to the adjacent building loads and that the differential displacements between sections B and D are sensitive to o the heterogeneity of the local undrained shear strength (Table 5). A summary of the results are presented in Table 6 considering a factor of safety, FS=1 and in Table 7

TABLE 7. Undrained shear strength

Section	Mean value of cu (kPa)
A soils3,4	46.5
B soils 3,4	47.4
B soil 6	43.9
D soils 3,4	73.8
D soil 6	89.3
C soil 3,4	60.0

TABLE 8. Cyclic mobility results (FS=1)

Section	Yield acc. (g)	Earthquake Source*	N/a	Max. Disp. (cm)
A	0.130	E1	0.48	3.0
B	0.063	E2	0.39	13.0
D	0.123	E1	0.45	<4.0
C	0.146	E1	0.54	1.5

E1 – close source (M=5.9 ; $(ab)_{max} = 269\ cm/s^2$)
E2 – distant source (M=8; $(ab)_{max} = 160\ cm/s^2$)

When considering a FS = 1.4 (the recommended value by the EC8)

TABLE 9. Cyclic mobility results (FS=1.4)

Section	Yield acc. (g)	Earthquake Source	N/a	Max. Disp. (cm)
A	0.053	E2	0.33	16.0
B	0.028	E2	0.18	67.0
D	0.123	E1	0.45	4.0
C	0.113	E1	0.54	2.7

The results computed with FS = 1.4 were later confirmed by FE pseudo-dynamic analysis and the soil treatment of sandy liquefiable soils was extended in depth to treat the clayey alluvium soil no. 6.

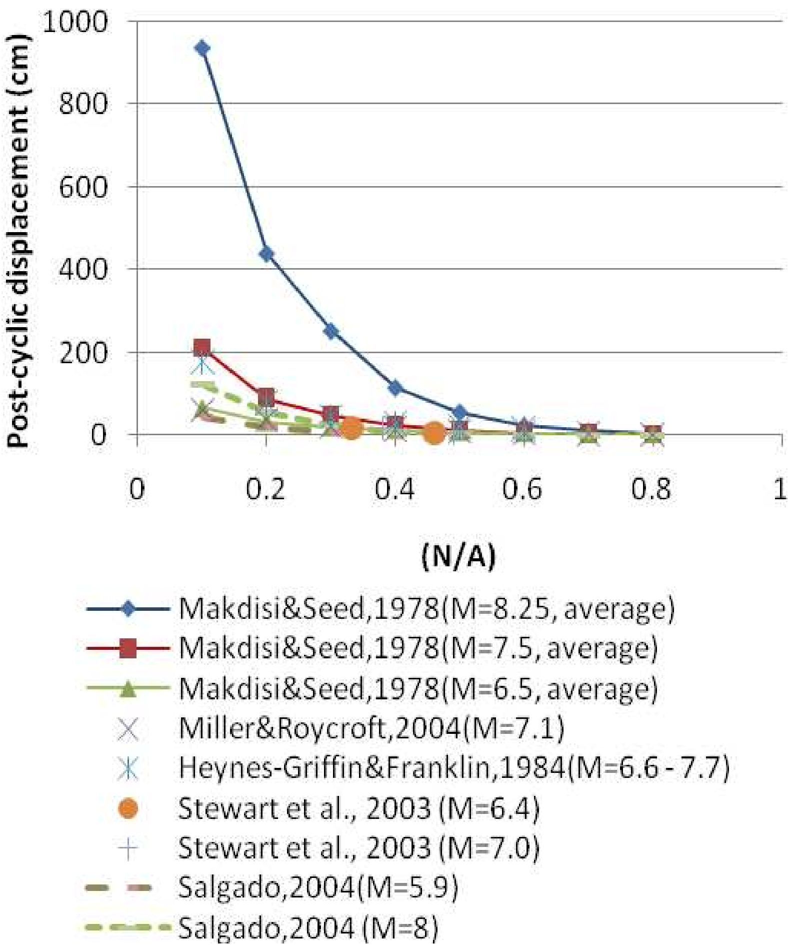


FIGURE 34. Post cyclic displacements versus acceleration ratio

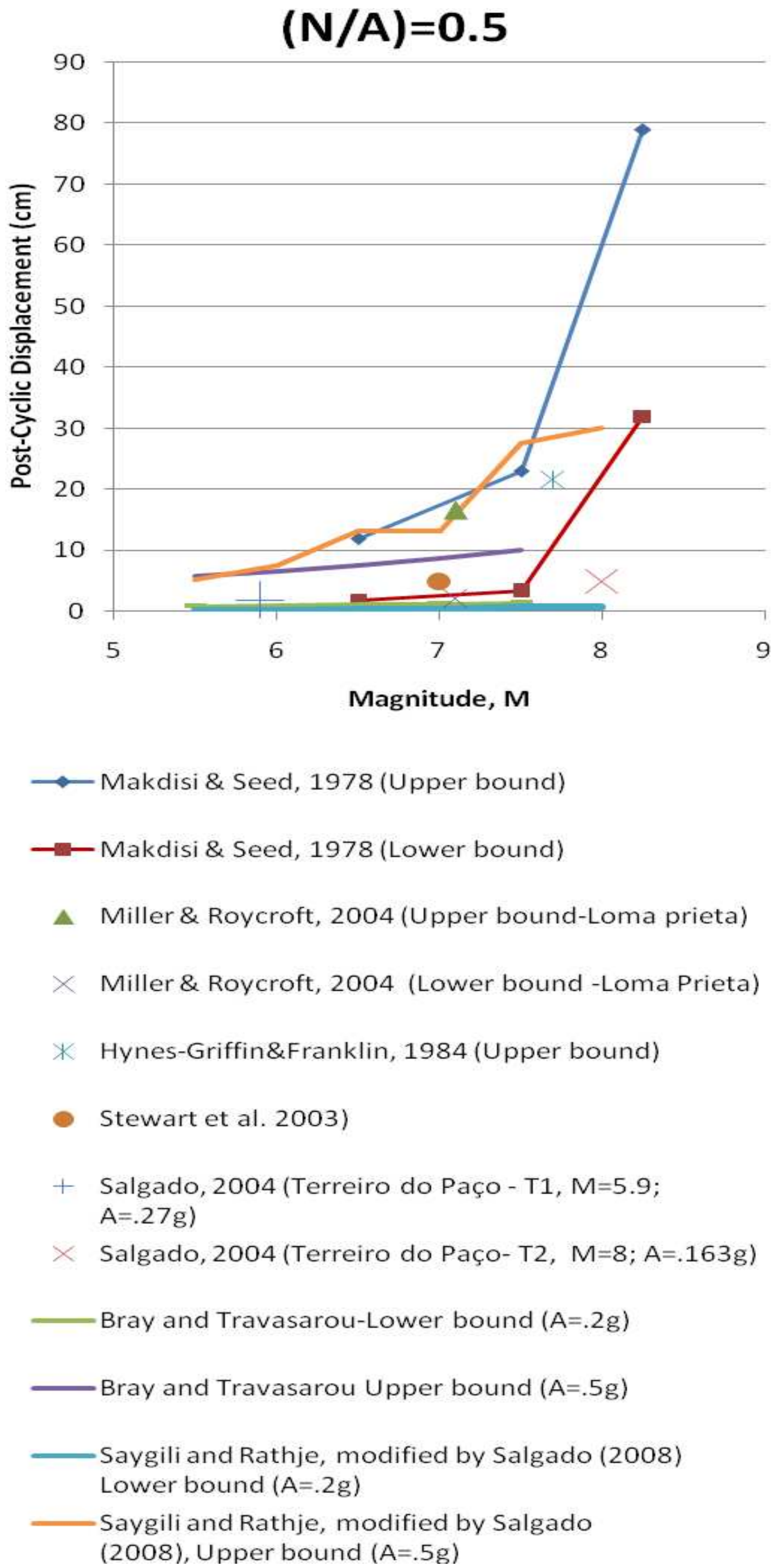


FIGURE 35. Post cyclic displacements versus magnitude, M for (N/A) = 0.5

5.0 – CONSEQUENCES OF LIQUEFACTION

Guidelines to help project engineers to deal with the consequences of the liquefaction problem are not covered in any international code. Based on the experience of LNEC researchers, while reviewing (national and international) geotechnical projects, it is possible to state that the need for such guidelines is a very important issue and an effort should be made to fill in this gap.

Some project engineers still believe that if liquefaction develops at a site then a major flow slide and global disaster will take place. Such was the case of Terreiro do Paço (T.P.) square site.

To clarify this issue the writer followed an approach combining empirical experience and simplified modelling (Salgado, 2005, 2007, 2008a). Empirical experience based on sound historic data as the advantage of given enough confidence to our engineering decisions. Simplified modelling, when well supported by a sound data base is a powerful tool to analyse different situations for remediation purposes. Therefore, to assess the possible consequences of liquefaction at the T.P. site the following procedures were considered:

i) empirical and semi empirical models were used to make a 1st screening of the possible post-liquefactions displacements that might develop at the site;
ii) secondly, Limit Equilibrium, LE, together with Dynamic Block stability analysis were carried out, using published historic residual shear strength data, to assess the potential for local flow sliding;
iii) to compute displacements at key locations, such as the Metro tunnel and the West Tower building, Finite Element Pseudo Dynamic analysis were then carried out to analyse the post-liquefaction, local safety;
iv) the displacement results obtained by the three different methods were compared to confirm the potential seriousness of the problem;
v) a remediation solution was then design and tested, and refine it, with the Finite Element code used in the previous analysis (Salgado, 2005, 2007, 2008a).
vi)

5.1 Empirical and semi-empirical models

The empirical models developed by Hamada et al. (1986) and Youd et al. (2002) show that their predictions, based on regressions analysis, is within a factor of 0.25 to 2.00 of the recorded data. About the same trend was obtained by the semi-empirical models developed, Shamoto et al. (1998) and Zhang et al. (2004). The first three models were considered here with the data shown in Table 10.
The relationship between fines content, FC, and mean grain size, D50, of the potential liquefiable alluvium (deposit nº 5) encountered at the tunnel site (sections A, B and D) and Station site (data from 50 SPT soil samples) are presented in Figure 36. In this figure is also shown the boundaries recommended by Youd et al. (2002) for the use of their MLR model. Only 7 data points (with FC>53%), plot outside of the boundaries of the data analysed by Youd et al.

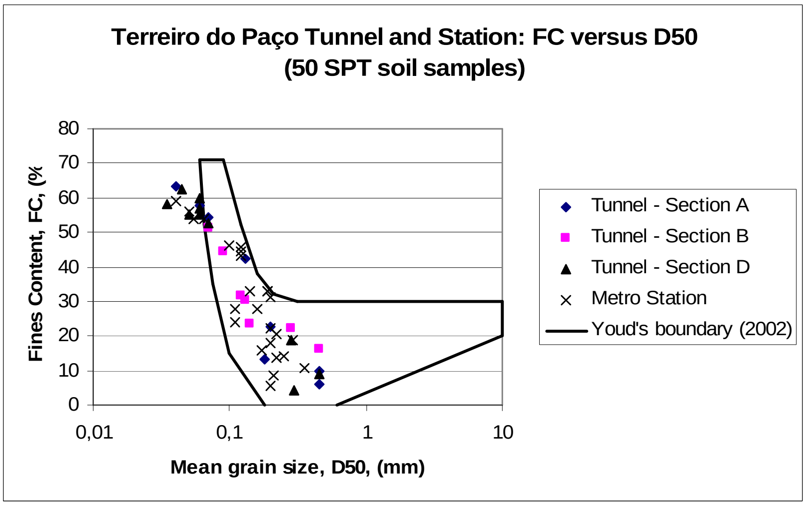


FIGURE 36. FC versus D50: Terreiro do Paço data

TABLE 10. Data used with the simplified models

Section	T (m)	FC (%)	D50 (mm)	Shamoto et al. (1998) $((\gamma_r)_{max})_{av}$(%)	$(N_1)_{60cs}$
A	10.0	22.0	.168	24.1	12.2
B	8.5	28.5	.125	22.8	12.6
D	6.0	23.0	.160	24.8	12.0
C	1.0	26.0	.139	21.3	13.1

where T is the thickness of the liquefiable deposit, $(N_1)_{60cs}$ the average equivalent clean sand normalized value of SPT and $((\gamma_r)_{max})_{av}$ is Shamoto's correspondent average maximum residual shear strain. The average slope, θ, is 6%, the maximum base acceleration, $(ab)_{max}$, is .163g, the earthquake magnitude, M, is equal to 8 and the equivalent source distance, R_{eq}, is estimated to be 55 Km (Youd et al, 2002). The following equations were considered to estimate the potential post-liquefaction horizontal displacements (D or Dh):

<u>Hamada et al</u>.,

$$D = 0.75 \times T^{1/2}\, \theta^{1/3} \tag{26}$$

<u>Youd et al</u>.,

$$\begin{aligned}\log Dh = &-16.213+1.532\, M-1.406 \log R^* \\ &-0.012\, R+0.338 \log S+0.540 \log T+3.413 \\ &\times \log (100-F)-0.795 \log (D50+0.1\text{ mm})\end{aligned} \tag{27}$$

where: $R^* = (10^{(0.89M-5.64)})+R$; $(R=R_{eq}.)$

<u>Shamoto et al</u>.,

$$Dh= Ch \times (Dh)max. = Ch \times \int (\gamma_r)_{max}\, dz \tag{28}$$

where: Ch=1 (water front) and(Dh)max is assumed = $((\gamma_r)_{max})_{av} \times T$

Estimations of the potential post-liquefaction displacements are presented in Table 11 and Figure 37.

TABLE 11. Post-liquefaction displacements (meter)

Section (Tunnel ring no.)	Hamada et al. (1987)	Youd et al. (2002)	Shamoto et al. (1998)
A (145)	4.30	2.42	2.41
B (176)	3.97	1.89	1.94
D (213)	3.34	1.80	1.49
C (252)	1.36	0.64	0.21

5.2 LE static and dynamic block analysis

These above predictions indicate that, based on historic data cases, significant potential total, and differential, displacements can develop at the T.P. site. However, because these predictions do not take into account the influence of the weight of the existing buildings, such as the West Tower (Figure 1), then the displacements can be significantly higher at section A and section B locations. This means that the estimations of the displacements must reflect the influence of the local Factor of Safety, FS.

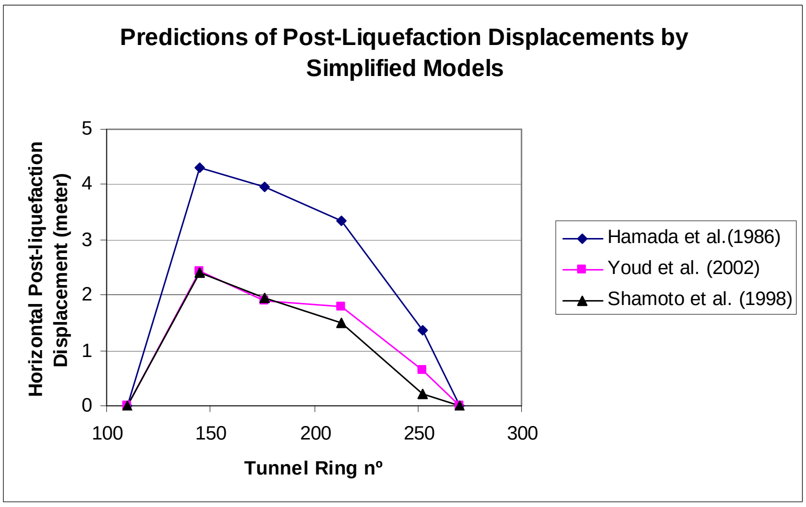


FIGURE 37. Predictions of Post-liquefaction displacements by simplified models

Dynamic Block analyses were carried out using a modified version of the computer code developed by Salgado (1981). The accelerograms were developed (Serra, J.P, 2002) based on the spectral acceleration response proposed by the Eurocode 8 and correspondent to a type 2 seismic action (great focal distance) with long duration (M=8) and a base acceleration, $(ab)_{max}$=160 m/s^2.
To account for the degradation of the shear modulus due to liquefaction a modified version of Newmark´s model (1965) was developed. This modified model (Salgado, 2005) considers an elastic-plastic stress-displacement relationship rather then the rigid-plastic approach considered in Newmark's model. Besides the acceleration data the other main input data parameters considered in the analysis were the yield acceleration ratio, a_y/g, computed from the LE analysis and the limiting displacement, D_L, estimated by $D_L = T \times \gamma_L$, where γ_L is the liming shear strain as defined by Seed et al. (1979).

5.2.1 – Residual and limiting shear strain

Residual shear strength and the correspondent liming shear strain are two key parameters to characterize the post-liquefaction response of sandy deposits. In the analysis carried out by Salgado (2005, 2007, 2008a) it is assumed that the stress-strain behaviour of the liquefied soils is characterized by an elastic plastic model where the yield strain corresponds to the limiting shear strain, that is estimated from the historic data set presented in Figure 38, and the yield shear strength corresponds to the post-liquefaction residual shear strength of the liquefied soils, as is discussed below.

At the time of the study (Salgado, 2005) estimations of residual shear strength could be assessed based upon the historic published data by Seed and Harder (1990) and by Olson and Stark (2002).

The major differences, between the two, besides the use of different sets of data, were that Seed and Harder consider the residual shear strength versus the "equivalent clean sand SPT corrected blowcount, $(N_1)_{60cs}$"(Figure 39) and Olson and Stark consider the residual strength ratio (residual shear strength / initial effective vertical stress) versus the "SPT corrected blowcount, $(N_1)_{60}$ " (Figure 40). Many people did not know how to use Seed and Harder´s data for the cases that considered effective vertical stresses higher then 1 atmosphere (100 kPa).

In 1999 Byrne and Beaty propose to normalize Seed and Harder´s data as shown in Figure 41.In this figure is also presented Idriss (1998) data that has also been normalized by Byrne and Beaty. These two sets of data are referred here as Modified Idriss (1998) and Modified Seed and Harder (1990).

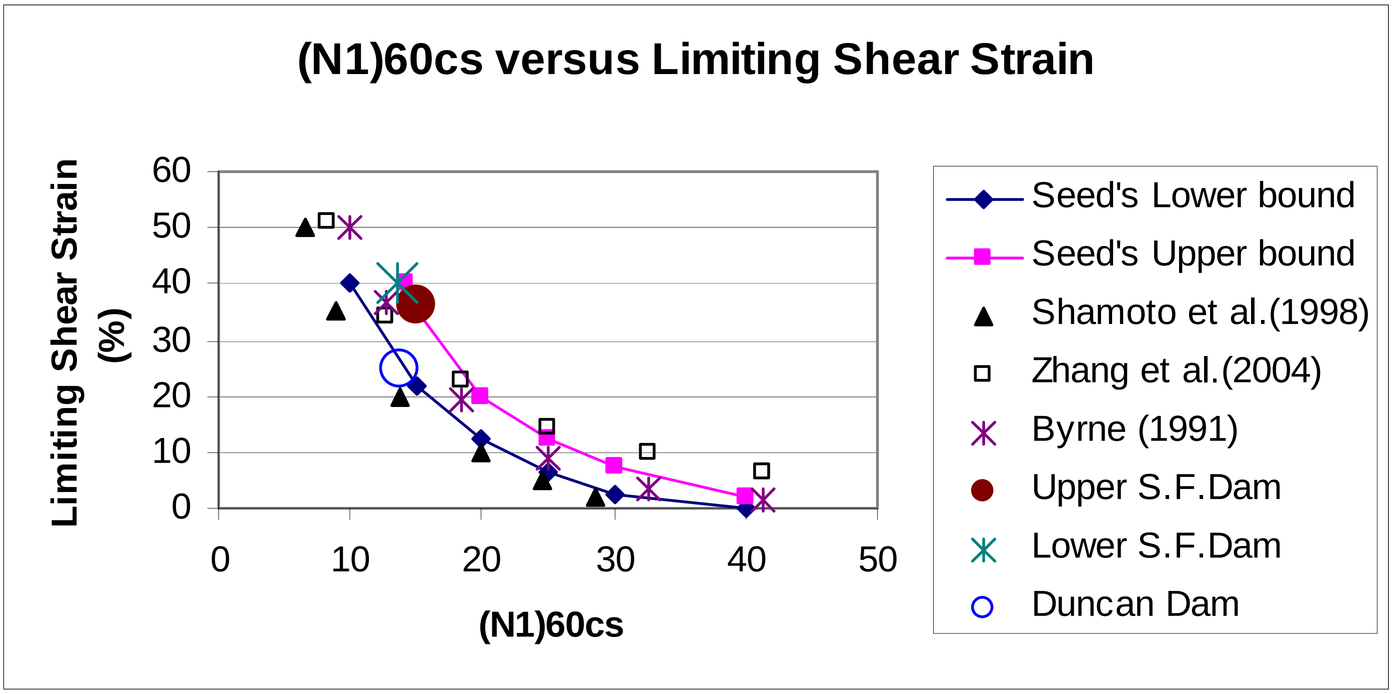


FIGURE 38. Limiting Shear strain data versus $(N_1)_{60cs}$ includes the lower and upper bounds from Seed (1979), and data correspondent to Duncan Dam (Pillai and Salgado, 1994); Upper S. Fernando Dam (Byrne et al. (1992) and Lower S. Fernando Dam (Salgado, 1992), Salgado(2005)

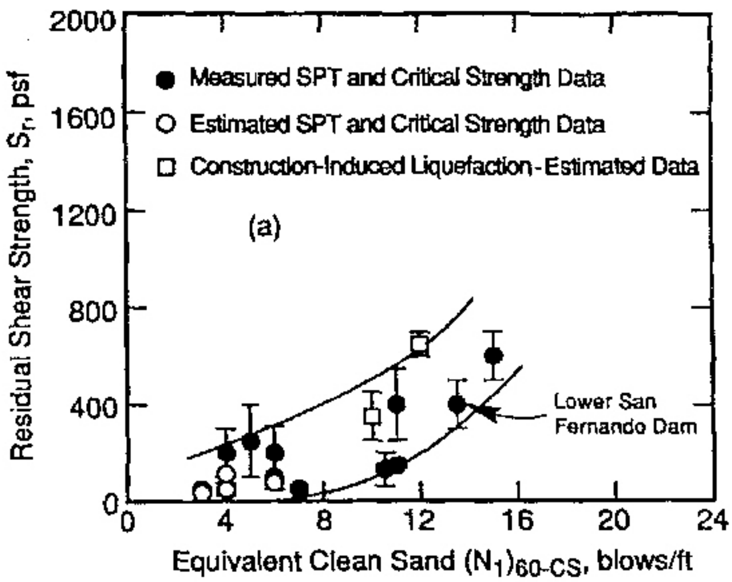


FIGURE 39. Residual Strength ratio versus $(N_1)_{60cs}$ (Seed and Harder, 1990)

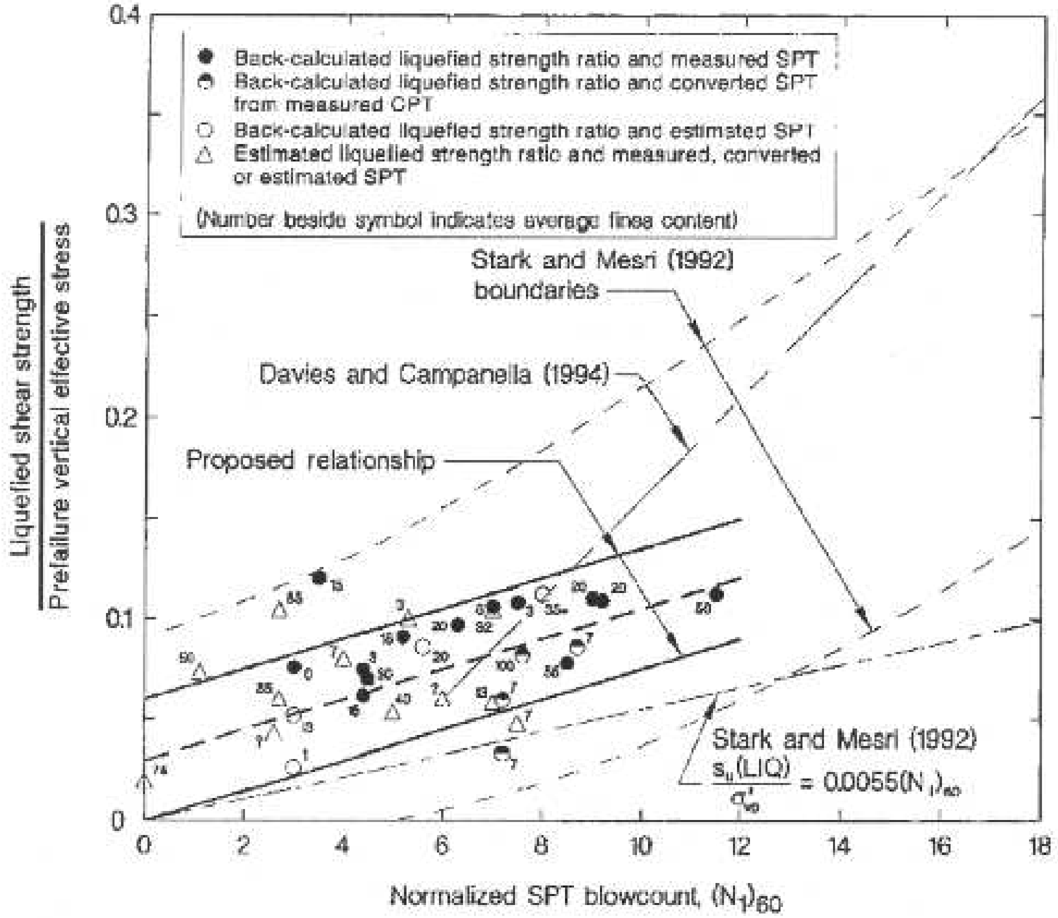


FIGURE 39. Residual Strength ratio versus $(N_1)_{60}$ (Olson and Stark, 2002)

Because the local confining effective vertical stresses within the liquefiable sandy alluvium range between 160 to 220 kPa the approach proposed by Byrne and Beaty (1999) was considered as well as the data published by Olson and Stark (2002), Salgado (2005).

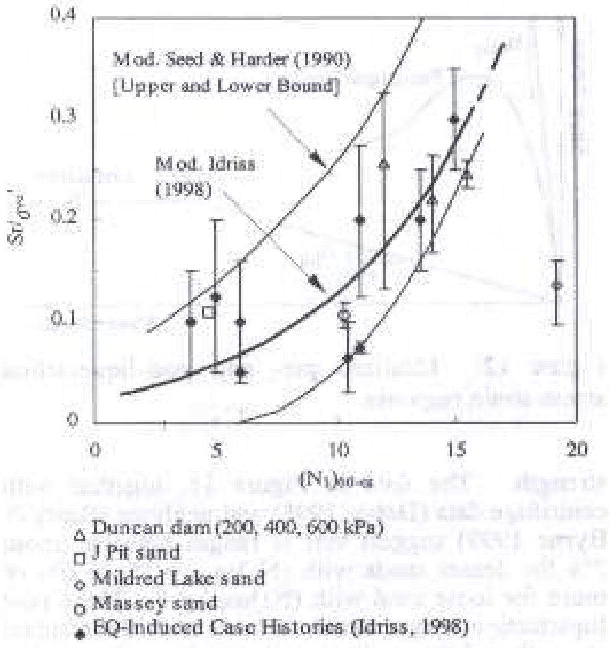


FIGURE 40. Residual Strength ratio versus $(N_1)_{60cs}$ (Byrne and Beaty, 1999)

Therefore to account for the effect of the confining effective stress, on the data shown in Figure 38, Salgado (2005) followed a similar approach as the proposed by Byrne and Beaty (1999):

$$(Sr/\sigma_{vo}') = (Sr/\sigma_{vo}')_1 \times K\sigma^* \tag{29}$$

where Kσ* is assumed to be equal do the factor Kσ defined by eq. (5), namely:

$$K\sigma^* \approx K\sigma = ((\sigma_{vo}'/Pa)^{f-1}\ ;\ \text{with } f=0.75 \tag{30}$$

and $(Sr/\sigma_{vo}')_1$ is the strength ratio correspondent to the confining stress of 1 atmosphere, Pa, and assumed by Salgado(2005) to be obtainable directly from Figure 5, using Hynes and Olsen (1999) data, i.e., Kσ* is assumed to be equal to the coefficient Kσ used to account for the reduction of the liquefaction resistance ratio with increasing confining stress (Youd et al. (2001) after Hynes and Olsen (1999)). The factor f = .75 corresponds to the relative density, Dr≈50%, of the alluvium soil nº 5.

Only recently Idriss and Boulanger (2007) proposed a correlation that considers the residual strength ratio (for confining effective vertical stresses up to 400 kPa) and takes into account Seed´s (1987), Seed and Harder (1990) and Olson and Stark (2002) sets of data, Figure 41.

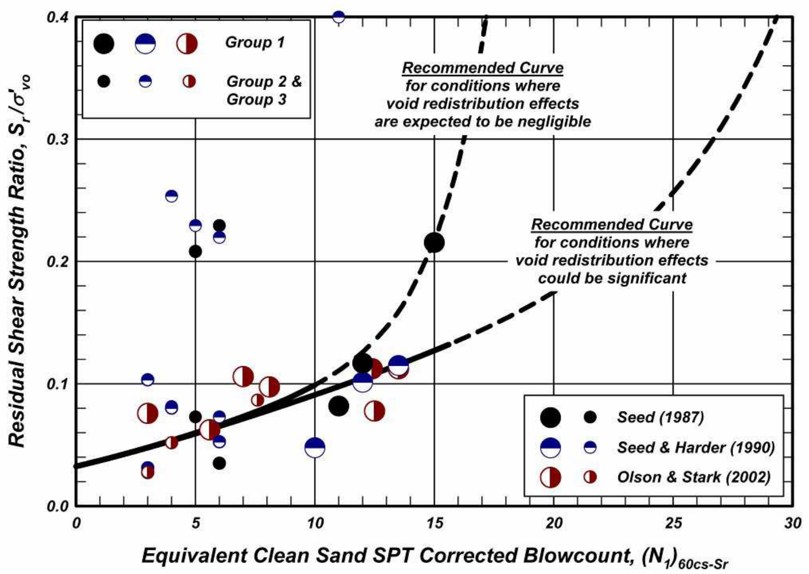


FIGURE 41. Residual shear strength ratio versus equivalent clean sand SPT blowcount $(N_1)_{60cs}$

Because of its importance the residual strength data correspondent to the Terreiro do Paço case is plotted here against the data published by Idriss and Boulanger (2007) as is shown in Figure 42.

It may be seen that a very good agreement is obtained between the data used by Salgado (2005) and the correlations developed later by Idriss and Boulanger (2007). In fact the soil data obtained from the sandy alluvium located beneath the Metro tunnel indicates that there is a high potential for void redistribution effects and therefore the shear strength obtained from Olson and Stark (2002) was considered for the design of the soil treatment project activated at the site in August 2007.

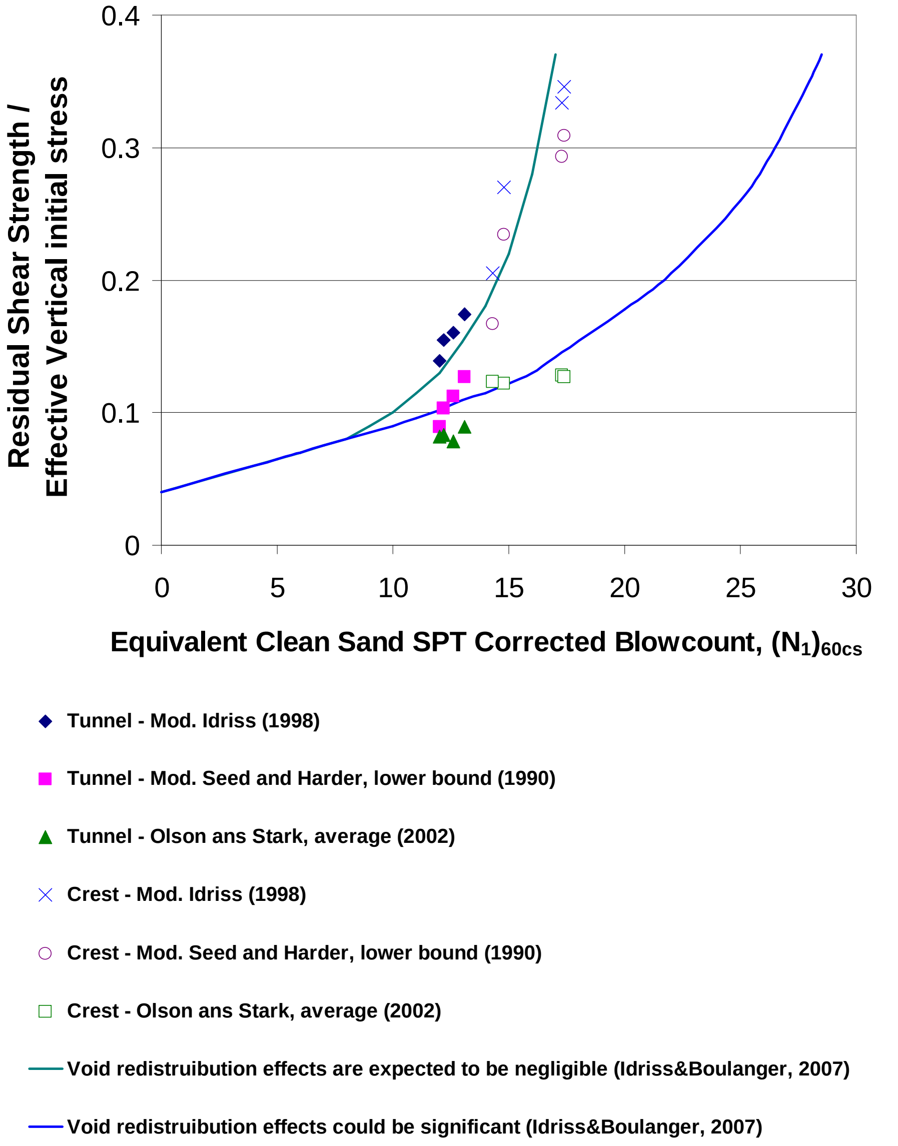


FIGURE 42. Residual shear strength ratio versus equivalent clean sand SPT blowcount $(N_1)_{60cs}$ (Terreiro do Paço data)

5.2.2 – LE and Block Analysis

Based on historic data presented in Figures 38, 40 and 41 the following different options, listed in Table 12, were studied by Salgado (2005). The soil types considered for the stability analysis are shown in Figure 43 (Cross section B).

The following data was considered in the analysis (Tables 13,14, 15 and 16). The LE results are presented in Table 17 and Figure 43 and the Block analysis results are in Table 18 and Figure 44.

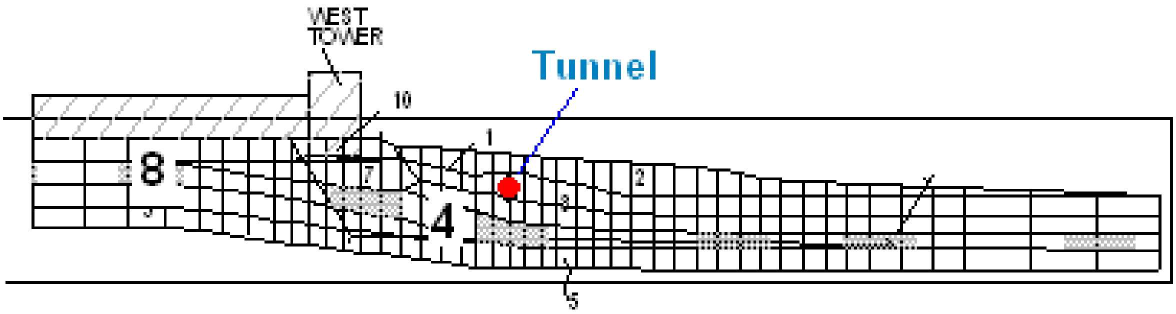


FIGURE 43. Cross section B. Location of Soil types and failure surface

TABLE 12. Options considered with the Block dynamic analysis and Finite Element analysis

Parameter	Mod. Newmark-1	Mod. Newmark-2	Mod. Newmark-3
Residual shear strength ratio (Sr/σ_{vo}')	Mod. * Idriss (1998) (Figure 40)	Mod.* Seed and Harder Lower bound (1990) (Figure 40)	Average of Olson and Stark (2002) (Figure 39)
Limiting shear strain (γ_L) (Figure 38)	Average of Seed et al. (1985)	Upper bound of Seed et al. (1985)	Upper bound of Seed et al. (1985)

* Byrne and Beaty (1999)

TABLE 13. $(N_1)_{60cs}$ and $(N_1)_{60}$ (Salgado, 2005)

Section	Tunnel		West Tower/Crest	
	$(N_1)_{60cs}$	$(N_1)_{60}$	$(N_1)_{60cs}$	$(N_1)_{60}$
A	12.2	7.1	17.3	13.1
B	12.6	6.4	14.8	12.3
D	12.0	6.9	17.4	13.0
C	13.1	7.8	14.3	12.4

Note: $(N_1)_{60cs}$ was computed from the average value of the data and $(N_1)_{60}$ computed from the mean value of the data.

TABLE 14. Residual shear strength ratio and limiting shear strain data. Tunnel location (Salgado, 2005)

Section	Mod. Newmark-1		Mod. New-mark-2		Mod. Newmark-3	
	S_r/σ_{vo}'	γ_L	S_r/σ_{vo}'	γ_L	S_r/σ_{vo}'	γ_L
A	.155	.39	.103	.50	.083	.50
B	.160	.37	.112	.48	.078	.48
D	.139	.40	.089	.52	.082	.52
C	.174	.34	.127	.46	.089	.46

TABLE 15. Residual shear strength ratio and limiting shear strain data. Crest location (Salgado, 2005)

Section	Mod. Newmark-1		Mod. Newmark-2		Mod. Newmark-3	
	S_r/σ_{vo}'	γ_L	S_r/σ_{vo}'	γ_L	S_r/σ_{vo}'	γ_L
A	.334	.22	.293	.29	.128	.29
B	.270	.26	.234	.34	.122	.34
D	.346	.21	.309	.28	.127	.30
C	.205	.31	.167	.40	.123	.40

Tables 14 and 15 present the strength parameters correspondent to the potential liquefiable alluvium (soil nº 5 in Figure 2) which was subdivided, in Figure 43, in to soil nº 4 (tunnel) and soil nº 8 (crest) respectively. Table 16 presents the strength parameters correspondent to the other soil types.

The dynamic model computes the displacements as well as the number of pulses, NP, using Newmark´s equation: $D = (V^2 / (2gN))\ NP$, where: $N= a_y/g$, D the displacement obtained with the rigid plastic model and the velocity, V is assumed = (ab)max. (m/sec). These values of NP were used with the Finite Element Pseudo-Dynamic analysis (Salgado, 2005).

TABLE 16. Strength parameters (Salgado, 2005)

Soil	Phi (º)	Cu (kPa)	Unit Weigh (kN/m3)
1	30.0	0.0	18.0
2	0.0	23.0	17.5
3	0.0	47.4	17.5
4	Table 14		17.5
5	0.0	43.9	17.5
7	0.0	59.2	17.5
8	Table 15		17.5
9	0.0	96.5	17.5
10	0.0	200.0	24.0

TABLE 17. Static Factor of Safety, FS (*), and yield acceleration ratio, a_y/g .No building loads, BL=0

Section	Mod. Newmark-1		Mod. Newmark-2		Mod. Newmark-3	
	FS	a_y/g	FS	a_y/g	FS	a_y/g
A	3.39	.073	2.62	.049	2.01	.032
B	3.81	.097	3.24	.062	2.62	.045
D	3.63	.073	2.79	.050	2.68	.046
C	3.85	.102	3.22	.079	2.69	.060

(*) LE analysis were carried out following Sarma’s method

TABLE18. NP and Disp.(meter) (BL=0)

Section /ring nº	Mod. Newmark-1		Mod. Newmark-2		Mod. Newmark-3	
	NP	Disp.	NP	Disp.	NP	Disp.
A/145	4.0	2.15	7.2	3.32	10.3	4.18
B/176	1.6	1.50	2.6	2.42	4.7	2.84
D/213	3.8	1.54	7.5	2.39	8.2	2.50
C/252	1.4	0.72	2.9	0.83	4.9	0.89

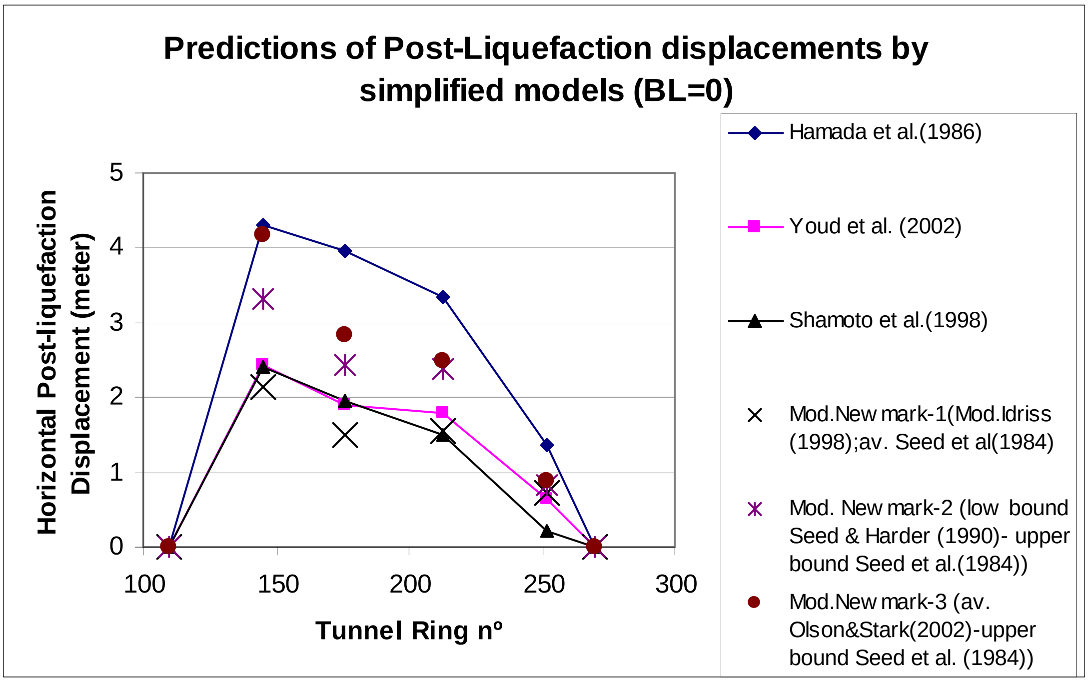


FIGURE 44. Dynamic Block analysis results (BL=0)

The above results correspond to the situation without building loads (BL=0). To study the effect of the weight of the buildings the analysis were repeated considering BL=127.5 kPa (Section A) and BL=200 kPa (Section B). The results obtained are presented in Tables 19 and 20 and Figure 45.

TABLE 19. Static Factor of Safety, FS, and yield acceleration ratio, a_y/g (BL>0)

Section	Mod. Newmark-1		Mod. Newmark-2		Mod. Newmark-3	
	FS	a_y/g	FS	a_y/g	FS	a_y/g
A	1.98	.053	1.55	.029	1.19	.011
B	1.60	.049	1.40	.032	1.22	.018

TABLE 20. NP and Disp.(meter) (BL>0)

Section	Mod. Newmark-1		Mod. Newmark-2		Mod. Newmark-3	
	NP	Disp.	NP	Disp.	NP	Disp.
A	6.5	2.56	11.0	4.39	13.3	7.01
B	7.6	2.28	11.3	3.52	13.2	4.56

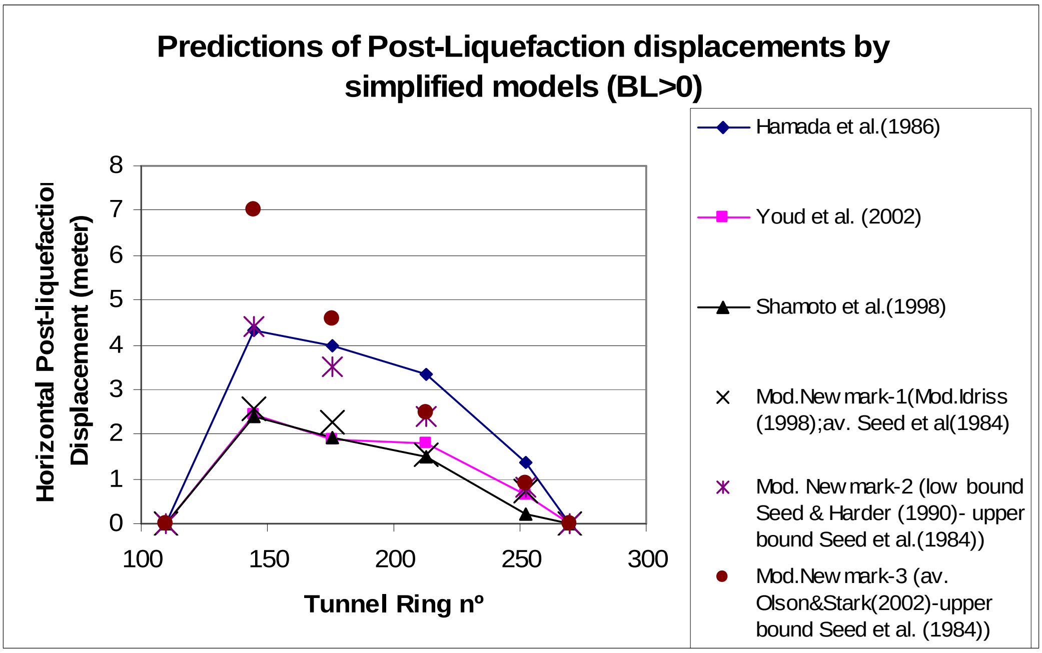


FIGURE 45. Dynamic block analysis results (BL>0)

It may be seen that a flow slide condition is not predicted from the analysis. However, the results indicate that potential high differential displacements might occur at the T.P. site. Therefore, was necessary to expand the above study with Finite Element (F.E.) analysis to assess the displacements mobilized at key locations such as the Metro tunnel and the adjacent West Tower building.

6 F.E. PSEUDO-DYNAMIC ANALYSIS

6.1 Summary of the approach

The analyses were carried out, by Salgado (2005), following the pseudo-dynamic approach developed by Byrne et al. (1992) and later considered in the Duncan Dam study (Pillai and Salgado, 1994). The approach incorporates inertia forces through the use of a seismic coefficient. The magnitude of the seismic coefficient is such that the computed displacements satisfy the total energy balance of the system (eq. 30). The displacement of a single degree of freedom system can be computed directly by solving eq. (30) as described by Byrne (1991). For a multi-degree-of-freedom system, a pseudo-dynamic method incorporating post liquefaction stress-strain curves can be used with a static finite element code (Byrne et al (1992). The appropriate seismic coefficient is found by an iterative converging procedure.

$$\text{Wext - Wint} = -\ 1/2\ M\ V^2 \qquad (31)$$

Applying this concept to Newmark's model for a single pulse, the displacement, D, is given by:

$$D = V^2 / (2gN) \qquad (32)$$

where N is the yield acceleration ratio of the sliding block. For a number of pulses, NP > 1, then the displacement is given by:

$$D = (V^2 / (2gN))\ NP \qquad (33)$$

where V is the velocity at the time liquefaction is triggered and assumed to be equal to 0.163 m/sec (the value of $(ab)_{max}$ (in m/sec). The values of NP used with the FE analysis are in Tables 18 and 20.

6.2 Cases studied

The three different options presented in Table 12 were considered by Salgado (2005) to characterize the possible post-liquefaction response of the liquefiable sandy alluvium soil (soils nº 4 and 8).
A first a set of analysis was carried out with no building loads (BL=0). The results are presented in Figure 46, together with the results estimated by the empirical models.

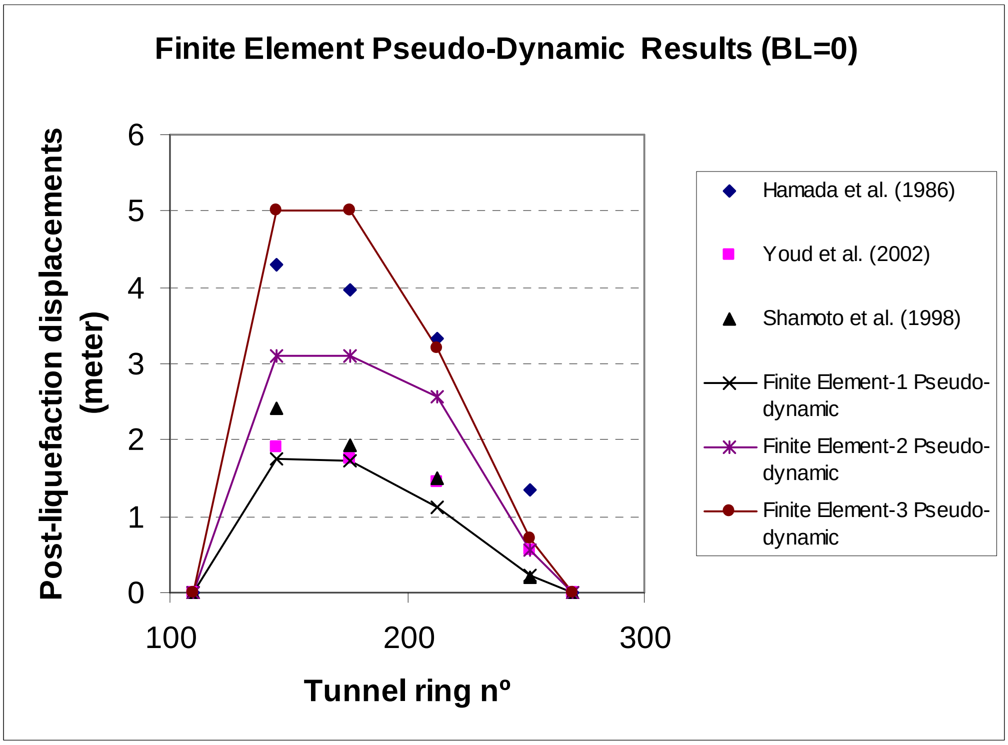


FIGURE 46. F.E. Pseudo-dynamic results (BL=0)

To study the effect of weight of the existing buildings (see figure 1) the analysis for section A and B were repeated. The results are presented in Figure 12 together with the results obtained with the dynamic Block analysis. A good agreement was obtained.

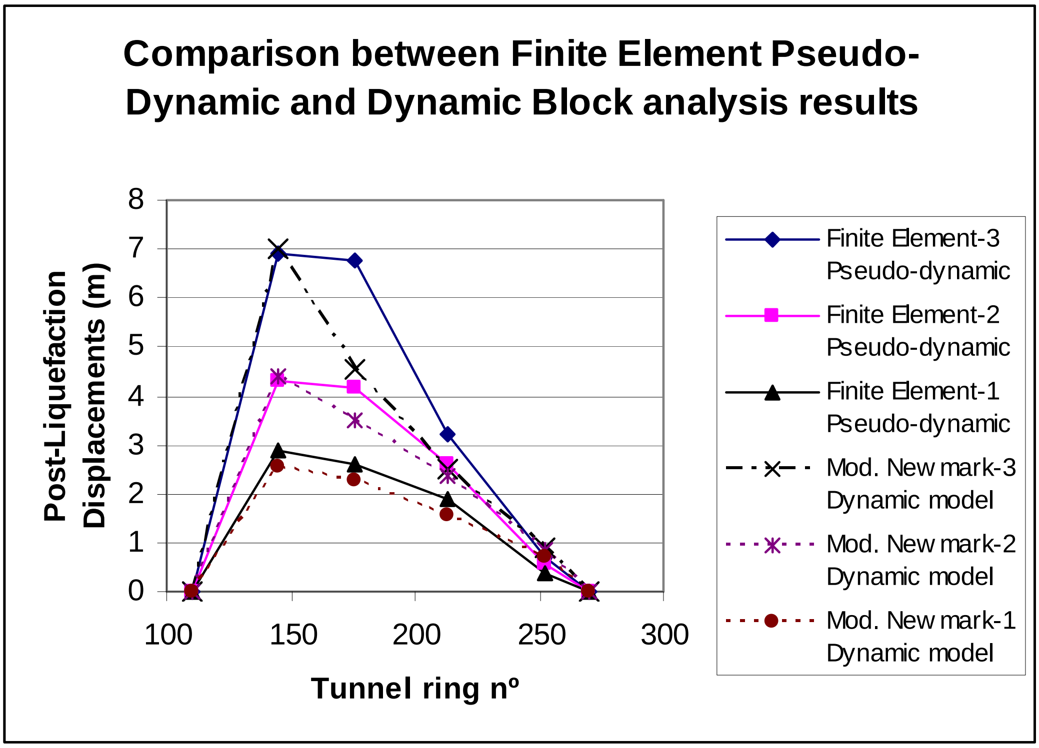


FIGURE 47. Study of the influence of building loads on the tunnel location.

The correspondent global F.E. displacements, by option 3, are presented in Figures 13. It may be seen that very large horizontal and vertical displacements are also computed for the location of the West Tower building.

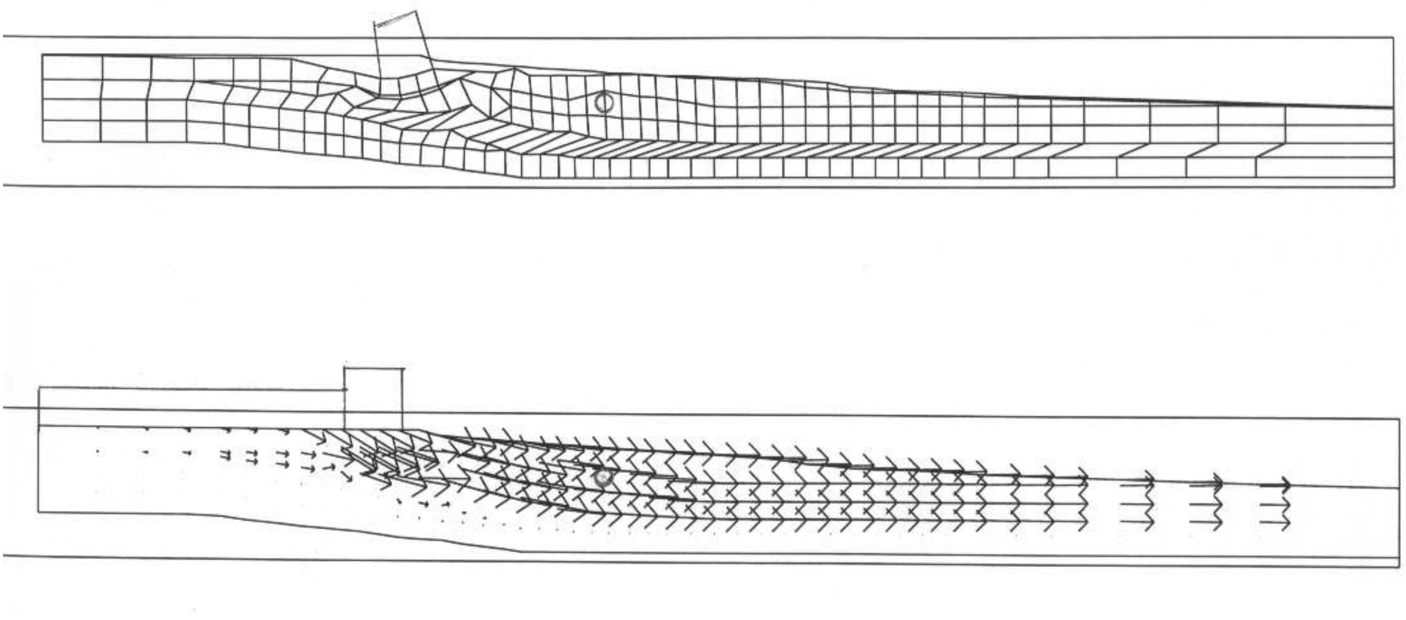


FIGURE 48. Diagram of displacements. Deformed mesh and vectorial plot. Section B - Option 3 (scale exaggeration = 3)

6.3 Soil treatment

Several solutions were studied to assess the most efficient and economic one. The analysis were carried out considering the alluvium soil treated, with columns of jet-grouting, to be equivalent to a soil characterized with an equivalent shear resistance and stiffness and function of the percentage of the ground treatment (GT%).The stability analysis (LE and FE) considered the Partial Factors of Safety recommended by the Eurocode 8, namely $\gamma_{cu} = 1.4$ and $\gamma_{\Phi'} = 1.25$. The results obtained in terms of GT% when considering the three (3) different stress-strain options (see Table 12) were not very different as presented in Table 19, however Option 3 (Olson and Stark) is requires a higher value of GT. Normalizing by the values correspondent to Option 3 it is obtained the data presented in Table 20. Considering that the costs to implement Option 3 are 5 million euros then the correspondent costs of the other options are listed in Table 21.

TABLE 19. Required ground treatment data (Salgado, 2005).

Eurocode 8 Partial Factors of Safety	Ground Treatment (%) – Section B (assuming shear resistance of jet grout = 3.65 MPa)		
	Option 1 – Mod. Idriss (1998)	Option 2 – Mod. Seed and Harder, low bound (1990)	Option 3 – Olson and Stark (2002)
ON (>1*)	GT=7.00%	7.35%	7.67%
OFF (=1**)	GT=5.60%	5.96%	6.32%

*$\gamma_{cu} = 1.4$ and $\gamma_{\Phi'} = 1.25$;
** $\gamma_{cu} = 1.4$ only for the treated soil

TABLE 21. Required ground treatment data normalized by 7.67%

Eurocode 8 Partial Factors of Safety	Ground Treatment (%) – Section B (assuming shear resistance of jet grout = 3.65 MPa)		
	Option 1 – Mod. Idriss (1998)	Option 2 – Mod. Seed and Harder, low bound (1990)	Option 3 – Olson and Stark (2002)
ON (>1*)	0.91	0.96	1.00
OFF (=1**)	0.73	0.78	0.82

*, ** see Table 20

TABLE 22. Required investment (Euros)

Eurocode 8 Partial Factors of Safety	Ground Treatment (%) – Section B (assuming shear resistance of jet grout = 3.65 MPa)		
	Option 1 – Mod. Idriss (1998)	Option 2 – Mod. Seed and Harder, low bound (1990)	Option 3 – Olson and Stark (2002)
ON (>1*)	4.55 M	4.80 M	5.00 M
OFF (=1**)	3.65M	3.90M	4.10 M

*, ** see Table 20

Based on the above results the Portuguese Government, trough the Administration of Metropolitano de Lisboa, decided to carry out a 5 million Euro ground treatment operation, which was carried out in August 2007 to improve the local ground conditions. The final project design was carried out by Mineiro et al. (2006), based on this study, with LNEC's continuous technical support.
The final, agreed, solution, which was implemented trough the execution of 2.0 m diameter jet-grouting columns and 1.2 and 1.4 m diameter cast in place reinforced piles, is similar to one of the solutions studied by Salgado (2005) as shown in Figure 49.

6.0 - CONCLUSIONS

This study outstands that the procedures for liquefactions assessment recommended by the EC8-P5 are slightly different from the recommended by the State of Practice (Youd et al. 2001), namely on the recommended magnitude scaling factors, MSF, where for a magnitude, M=8, the State of Practice recommends a MSF= 0.84, while the EC8-P5 recommends a MSF=0.67. On the other hand, the State of Practice recommends the use of the overburden correction factor, $K\sigma$, but the EC8-P5 omits its use. For the Terreiro do Paço case (where σ'_{vo} ranges between 160 to 220 kPa) $K\sigma \approx 0.86$. Therefore, combining the influence of the MSF and $K\sigma$, when following the State of Practice procedures it becomes that MSF x $K\sigma$ = 0.84 x 0.86 = 0.72 which is very close to the MSF=0.67 recommended by the EC8-P5. This means that for the Terreiro do Paço case it did not make any difference in the outcome of the study. Both procedures predict very low Factors of safety against Liquefaction, FSL varying between 0.5 to 0.7.

This study also outstands that, at present, there are no guidelines in the Eurocode 8 to assess the possible consequences of the liquefaction phenomenon.

A summary of the procedures followed by LNEC (Salgado, 2005) to deal with this issue are presented. The studies carried out show that simplified empirical and semi-empirical modeling, based on historic lateral post-liquefaction displacement data, were fundamental to give enough engineering confidence to the FE Pseudo-Dynamic analysis procedures followed in the present study, as well as, to the political and engineering decision to develop and activate the correspondent remediation soil treatment project. The study also show that the post-liquefaction residual shear strength considered in the analysis by Salgado (2005) is in very good agreement with the data published by Idriss and Boulander (2007).

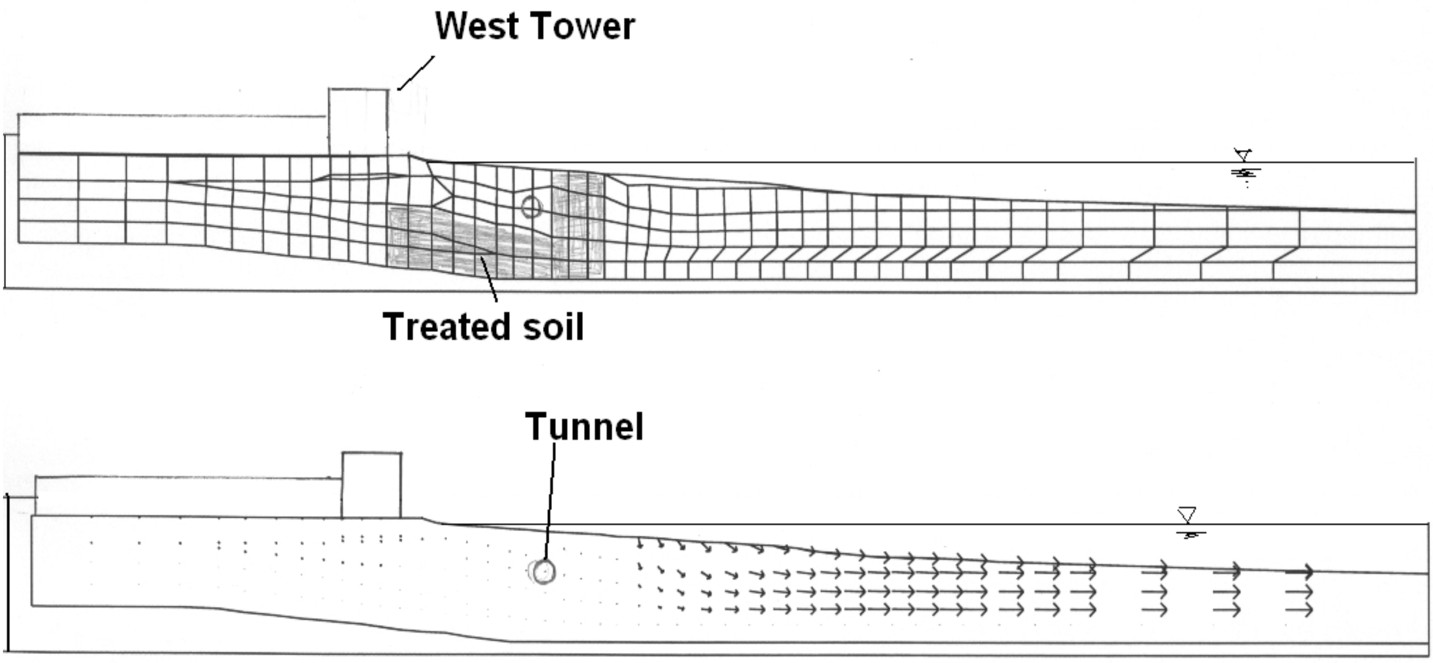


FIGURE 49. Location of the treated zone (top); vectorial displacement plot. Section B - Option 3 (scale exaggeration = 5)

ACKNOWLEDGEMENT

The writer expresses his deep appreciation to the Administration of Metropolitano de Lisboa for the investment made in all the in situ and laboratory testing, as well as, the approval for all the analytical work carried out, including the divulgation of the correspondent results.

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