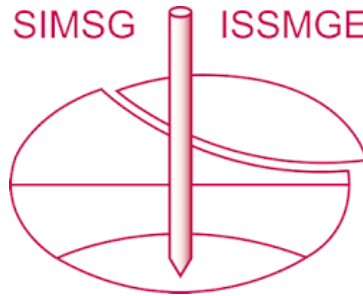


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Liquefaction – Causes and effects

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ABSTRACT: The liquefaction phenomenon can be induced by dynamic or static causes. Examples are presented and its main effects analyzed. A generalization of the simplified procedure developed by Seed and Idriss (1971) is considered to estimate the correspondent factor of safety against liquefaction. The historic cases presented herein consider different number of cycles to liquefaction, varying from ½ cycle to more than 400 cycles. For all cases the main liquefaction effects and the corresponding mitigation measures are presented.

1 INTRODUCTION

The liquefaction phenomenon develops mainly in loose to medium compact sandy soils due to the development of interstitial pressures induced by seismic actions. When the interstitial pressure equals the vertical effective stress these soils have, temporarily, drastic reductions of rigidity and shear resistance. The occurrence of this phenomenon can have devastating consequences and lead to the collapse of buildings, earth dams, supporting walls and other geotechnical structures. However, liquefaction can also be mobilized by the dynamic effect of waves on the sandy sea bottom, leading to the collapse of important structures such as oil or gas pipelines that are buried or on top of the sea floor. Another relevant dynamic action occurs in the North Sea, where for about 3/4 of the year the sea is covered by a layer of thick ice, which, when subjected to the action of wind can mobilize cyclic ruptures of the ice, against stationary oil exploration structures and trigger significant cyclic loads. On the other hand, the phenomenon of liquefaction can also be triggered by static causes, as the case that took place in 2000, in the foundation soil of the underground tunnel/station of the Metropolitano de Lisboa in Terreiro do Paço.

2 ANALYSIS PROCEDURES

A generalization of the simplified procedure developed by Seed and Idriss (1971) is considered to estimate the correspondent factor of safety against liquefaction for all the different cases.

The factor of safety against liquefaction, FSL , is estimated by the ratio between the cyclic resistance ratio, $(CRR)_N$ corresponding to a number of cycles, N , and the cyclic stress ratio, CSR .

$$FSL = (CRR)_N / CSR \quad (1)$$

The value of $(CRR)_N$ is estimated from Equation 2.

$$(CRR)_N = (CRR)_{15} NCSF \quad (2)$$

where $(CRR)_{15}$ is the cyclic stress resistance correspondent to an earthquake of magnitude 7.5 (characterized by a number of equivalent uniform cycles of 15) and $NCSF$ is a scaling factor correspondent to the number of cycles induced by the respective dynamic or static loading. An update of the correlation developed by Been (1988) between the number of cycles, N , and $NCSF$ is presented in Figure 1. In this figure the “Magnitude Scaling Factor”, MSF , data from Idriss

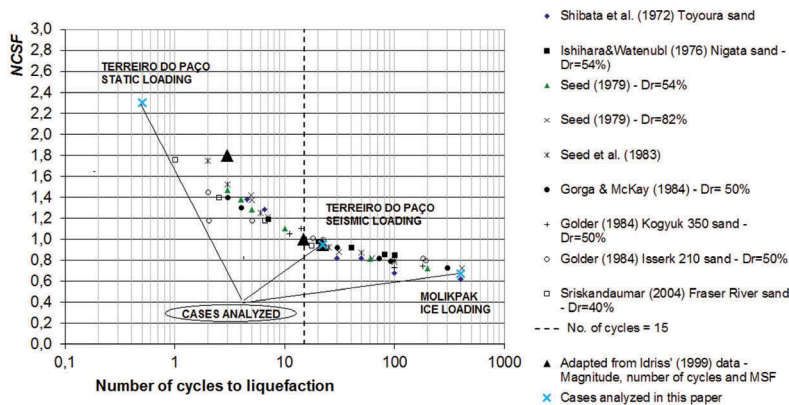


Figure 1. Correlation between number of cycles and $NCSF$

Table 1. Cases analyzed and corresponding values of $NCSF$

Case analyzed	Number of cycles	$NCSF$
Static liquefaction	0.5 (Salgado, 2009a, 2019)	2.30
Dynamic earthquake liquefaction	22 (Idriss, 1999)	0.95
Dynamic ice loading liquefaction	400 (Been, 1988)	0.67

(1999) corresponding to earthquakes of magnitudes 5 ($N=3$), 7.5 ($N=15$) and 8 ($N=22$) is included, as well as the $NCSF$ value correspondent to the Terreiro do Paço's $\frac{1}{2}$ cycle case.

The $NCSF$ values for the different cases analyzed in this paper are presented in Table 1. The correspondent values of $(CRR)_{15}$ are estimated from empirical correlations from N_{SPT} and (or) CPT *in situ* data, following Seed's procedures. The procedures to estimate CSR are presented with each one of the cases analyzed.

3 DYNAMIC ICE LOADING INDUCED LIQUEFACTION

The ice covers the Beaufort Sea for $\frac{3}{4}$ of the year with thickness up to 1.5 m. On April 1986 a strong storm hit the Molikpak, which is an exploratory station owned by GULF Canada resources Inc. This station is composed by a box type structure with its core (72x72 x20 m) filled with hydraulic placed sand. The box is placed on a sand embankment previously built on the sea floor. A good description of the Molikpak development is given by Jefferies and Wright (1988). The failure pressure of the ice, against a stationary structure like the Molikpak, is about 1 MPa. As the downstream ice breaks it is extruded from the failure zone and is replaced by another intact ice sheet coming from upstream, developing this way conditions for a dynamic continuous load event. On the 12th of April 1986 the East face of the Molikpak was hit, during several minutes, by an ice load storm with a static component of 279 MN and a cyclic load with amplitude of ± 118 MN with more than 400 cycles of 1Hz frequency. This ice load event caused part of the sand core of the Molikpak to liquefy and the platform to come close to liming stability. This paper presents a summary of the study of this event carried out by Salgado (1990).

The caisson's box core was filled in with Erksak 320/1 sand, characterized with $D_{50} = 320$ mm, $D_{10} = 200$ mm, $cu=1.6$ and 0.8% fines. It's *in situ* characterization was carried out with 20 CPTs and 3 SBP holes (20 tests). The caisson, the sand core and foundation were modelled with the 3D Finite Element (FE) mesh presented in Figures 2 and 3, and its behavior was analyzed with the hyperbolic model. Using the mean distribution of the cone resistance (Figure 4) and applying the

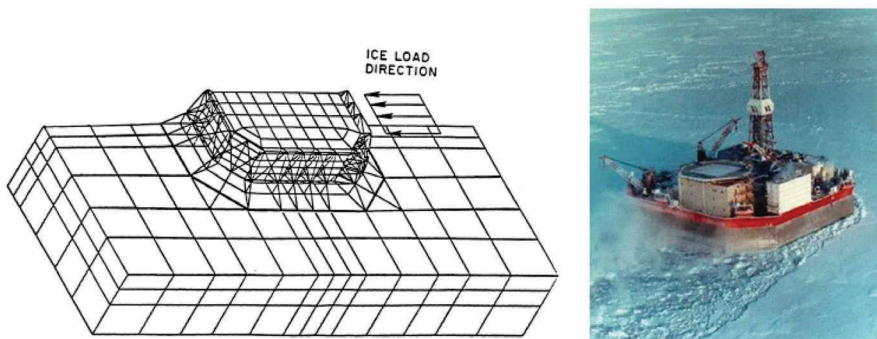


Figure 2. Photograph of Molikpak and Finite Element mesh considered in the 3D analysis.

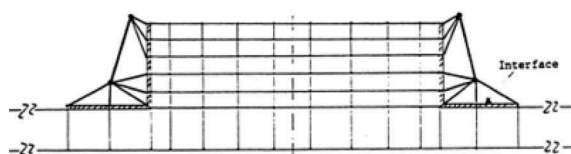


Figure 3. Partial cross section of the 3D FE mesh.

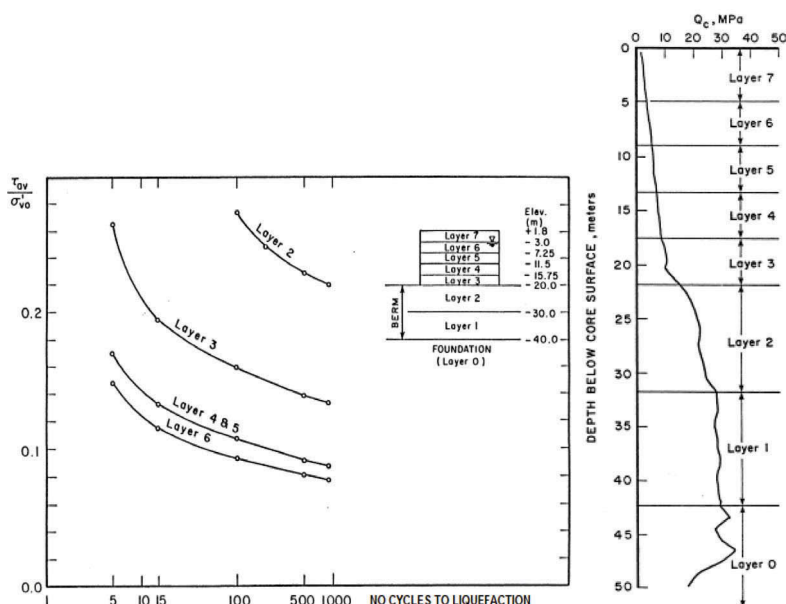


Figure 4. Distribution of CPT point resistance, Q_c , and liquefaction layers resistance

empirical chart developed by Seed and DeAlba (1986) together with the *NCSF* value of Table 1, than the liquefaction resistance curves, presented in Figure 4, correspondent to the different soil layers, were obtained.

The 3D FE mesh and soil model were first validated and checked against measured displacement data obtained from a previous ice load event (25th March 1986), with an estimated maximum load of 110 MN. A comparison between the measured 21 mm maximum displacement and

the calculations (considering two different ice load distributions) is presented in Figure 5. The calculation results were also checked against inclinometer readings in the sand core and foundation embankment. A reasonable good agreement is obtained (Salgado, 1990). Then, after this validation, an estimation of the 3D spatial distribution of the liquefied sand core was carried out, as shown in Figures 6 and 7, using the ice load time history characteristics (Jefferies and Wright, 1988) and the liquefaction soil layer resistance presented in Figure 4. The layer no. 2 (top layer of the foundation embankment), and below, was not predicted to liquefy.

Horizontal displacements as high as 220 mm is computed by the 3D anlysis. However, due to instrument failure, there are no field recorded data for comparison. Predictions of the residual settlement were also carried out using Tokimatsu and Seed’s chart (1987) and a very good agreement is obtained with the field mesurements as shown in Figure 7 (Salgado, 1990).

After the spring 1986, where the Molikpak caisson was almost decapitated by a dynamic ice loading event, GULF Canada resources Inc. decided that for future caisson construction in the Beaufort sea, the hydraulic placed sand core must be densified to a pre-design compaction degree, in order to resist future extreme ice load events, like the one occurred in 12th April 1986.

4 STATIC INDUCED LIQUEFACTION

The underground Lisbon blue line tunnel of Metropolitano de Lisboa was constructed with a large boring machine between Baixa-Chiado and St. Apolónia, passing, offshore, by Terreiro

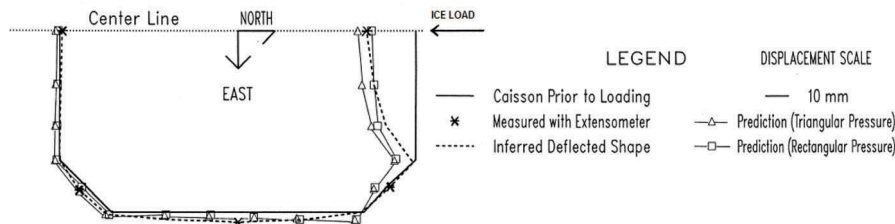


Figure 5. Caisson deformation due to the ice load event of 25th March 1986. 3D comparison between field measurements and calculations

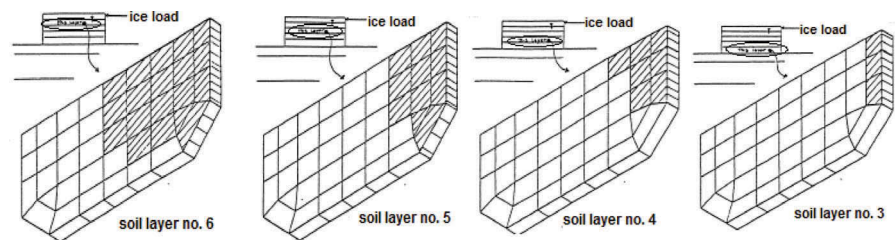


Figure 6. 3D distribution of liquefied zones in the submerged soil layers of the Molikpak sand core

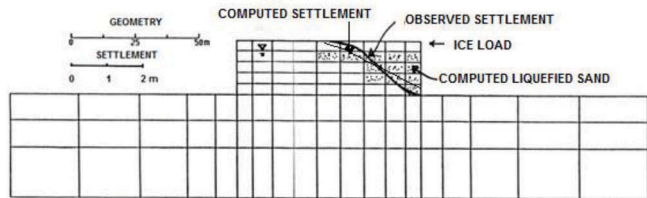


Figure 7. 2D distribution of liquefied soil layers of the Molikpak sand core and observed settlement

do Paço waterfront (Figure 8). The construction was considered to be extremely easy as compared with the major problems encountered under Carmo's Convent slope (Salgado, 2009a). Logistic reasons dictate that the tunnel was built first and the two underground stations built afterwards. Terreiro do Paço underground station is located within clayey soft alluviums deposits intercalated with silty sandy loose pockets. To overcome this difficult geotechnical conditions it was necessary, before starting the required excavation, to treat the submerged river alluviums at the location of the collar interception of the already built tunnel with the station's front and back, walls. The project considered a previous soil treatment, with jet grouting columns, carried out in two phases (Figure 8). The first phase treated the foundation tunnel soils from the inside of the tunnel, and the second phase from the top of an embankment that have been previously built to prevent an uplift scenario, during and after tunnel construction.

The drilling of the 152 mm diameter holes to perforate the bottom tunnel shaft started on the 7th of June 2000. The project considered about 250 holes. On the third drilling day, by 11h45, mud and water started to erupt, uncontrolled, from the 13th hole. After several attempts to plug it, mud and water erupted from adjacent holes and major settlements started to occur at surface. The stability of the nearby Av. Infante D. Henrique and the adjacent East Tower building of the Ministry of Finance were a major concern. To stabilize the inflow of soils and water, and minimize the surface settlements, it was decided to flood the 9.9 m tunnel with water from the adjacent Tagus River, between Poço da Marinha and Poço da Alfandega, totaling a length of 1,140 m (see Figure 8). The operation was a major success, and the rate of the surface settlement decrease to acceptable values (Figure 9). The tunnel was finally stabilized by the end of December 2000, by substituting, along a predefined tunnel length, the stabilizing water with a "light grout" material that was pre-designed and pre-tested *in situ*, with similar tunnel conditions, to insure enough placement fluidity and fast dry strength (Salgado, 2014). Afterwards, during the excavation, to build the Metro's station, it was possible to observe the damage caused to the tunnel's structure, as well as the complete "light grout" tunnel filling (Figure 10).

Detailed work carried out by Salgado (2009a, 2014, 2019), following the generalization of the simplified procedures developed by Seed and Idriss (1971), shows that this geotechnical incident can be treated as a static liquefaction phenomenon triggered by a $\frac{1}{2}$ cycle of unloading. A summary of this work is presented herein. Salgado considers that the tunnel shaft drilled holes caused an unloading increment of the deviator stress, Dq , in the foundation soil, equal to the decrease to zero from the value of the *in situ* effective vertical stress, s_v' , and that the correspondent shear stress increment, Dt , is given by Equation 3.

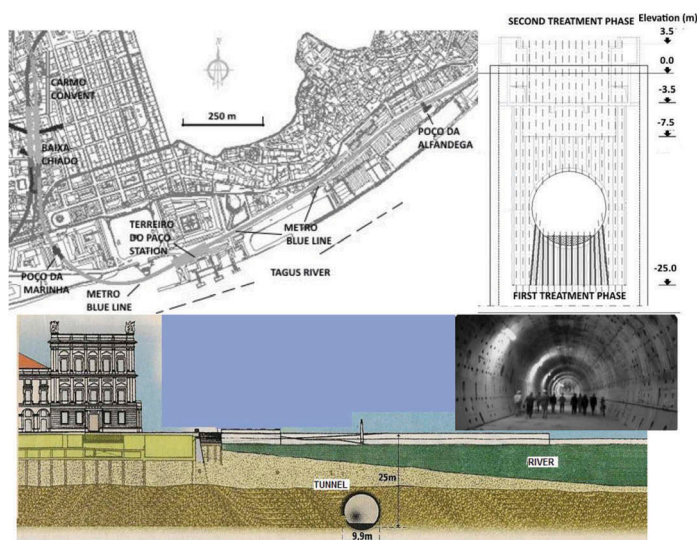


Figure 8. Location plant of Metro's Blue line. Sketch with the two treatment phases and cross section

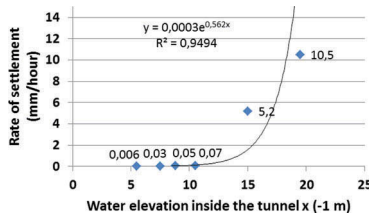


Figure 9. Water elevation versus rate of settlement (Salgado, 2009a, 2014)



Figure 10. Photographs of the damage tunnel (rings 348 to 350) and tunnel's cut showing "light grout" tunnel filling (Salgado, 2009a, 2014)

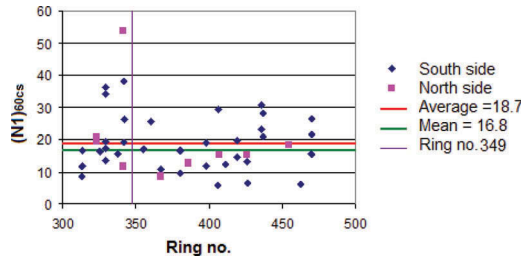


Figure 11. Distribution of $(N_1)_{60cs}$ along the tunnel

$$\Delta\tau = 0.5 \Delta q = 0.5 \Delta\sigma_V = 0.5\sigma_V \quad (3)$$

Therefore, the correspondent stress ratio, CSR , is given by:

$$CSR = \Delta\tau/\sigma_V = 0.5 \quad (4)$$

Considering $FSL = 1$ and applying equations 1 and 4, then, to avoid liquefaction $(CRR)_{0.5} \geq 0.5$. Since there is no published data for $NCSF$ correspondent to $\frac{1}{2}$ cycle, an estimation for the possible range of values is analyzed. The data from several SPT tests, carried out before and after the June's 2000 incident, is considered. The correspondent values of $(N1)_{60cs}$ are presented in Figure 11 together with the correspondent mean value (16.8) and the location of the tunnel ring 349 (see Figure 10). Using this mean value with Seed and Idriss's (1982) chart, that correlates $(N1)_{60cs}$ with $(CRR)_{7.5}$, a value of $(CRR)_{15} = 0.178$ is obtained. To achieve a value of $FSL = 1$ it implies that $NCSF$ should be equal to $(0.50/0.178) = 2.8$, i.e., $FSL = (CRR)_{0.5}/CSR = (0.178 \times 2.8)/0.5 = 1.0$. On the other hand, $NCSF$ should be ≥ 1.8 , because this is the highest published value of $NCSF$ correspondent to 1 cycle. Therefore, for the $\frac{1}{2}$ cycle case of Terreiro do Paço, $1.8 \leq NCSF \leq 2.8$, the correspondent range for FSL is $0.64 \leq FSL \leq 1.00$, with average values of $NCSF = 2.3$ and $FSL = 0.82$.

5 MITIGATION OF SEISMIC INDUCED LIQUEFACTION AND CYCLIC MOBILITY IN TERREIRO DO PAÇO

The previous case brought the attention to further analyze the seismic stability of Terreiro do Paço tunnel. This subject is well described by Salgado (2007, 2009b). A brief summary is presented here. The location of the tunnel is shown in Figures 8 and 12. After an extensive *in situ* and laboratory work, detailed studies, following the State of the Arte procedures, show that for an earthquake with a magnitude $M = 8$ (no. of cycles = 22, see Table 1), soil no. 5 will liquefy, and soils no. 3, 4 and 6 are prone to cyclic mobility. The value of $(CRR)_{22}$ was estimated using $NCSF = 0.95$ and $(CRR)_{15}$ based on *SPT*, *CPT* and *VS in situ* data. *CSR* was estimated from SHAKE dynamic 1D analysis, using representative artificial acceleration series of a distant source earthquake. The study carried out by Salgado (2007, 2009b), using a pseudo-dynamic approach, show that if the local alluviums are not reinforced, then, large destabilizing displacements, up to 7.0 m, would develop at the site (Figure 13). Soil treatment was, therefore, recommended and carried out at the site in 2007. A grid of large diameter concrete piles and jet grouting was implemented at the site, based on a project developed by Mineiro et al., (2007).

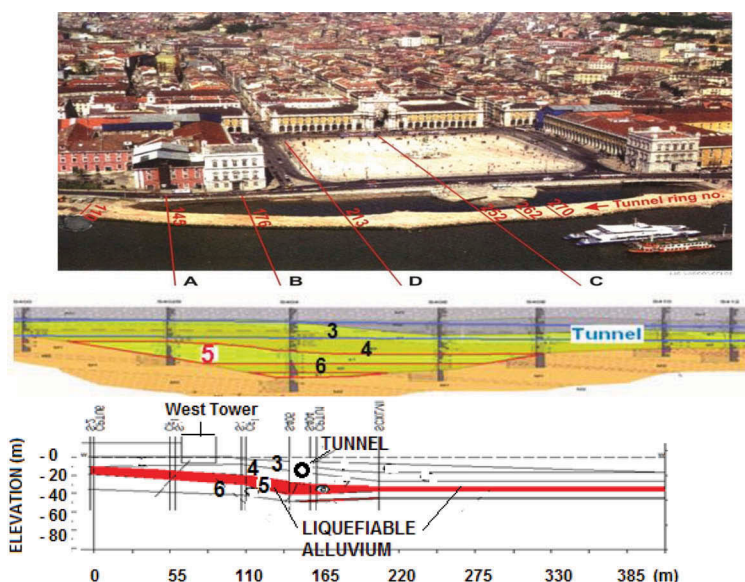


Figure 12. Terreiro do Paço's tunnel: aerial photograph; longitudinal and transversal cross section (B)

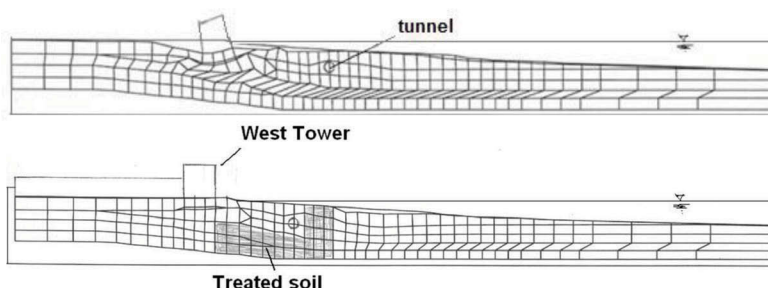


Figure 13. Results of analysis: No treatment (liquefaction effects); Full treatment

6 CONCLUSION

Different causes that can trigger the liquefaction phenomenon, from a range of $\frac{1}{2}$ static unloading cycle to 22 cycles of seismic loading and 400 cycles of ice loading, were successfully analyzed following a generalization of Seed and Idriss (1971) simplified procedure.

ACKNOWLEDGMENTS

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