Updated issues regarding the evaluation of soil liquefaction potential under earthquake loading

Mise à jour des questions concernant l'évaluation du potentiel de liquéfaction des sols en cas de tremblement de terre

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ABSTRACT: There are several locations in Lebanon where soil investigations show the existence of loose sand layers at different depths. Several coastal areas with a groundwater level nearby the surface, present a substantial risk of liquefaction. The paper assesses the evaluation of the liquefaction potential based on in situ CPT and SPT tests. The seven studied methods are based on the cyclic stress approach. An extensive analysis is carried out to understand and compare the results obtained by the methods. Soil deposits in the northern suburb of Beirut are thoroughly analyzed. The achieved results help in the understanding of the geotechnical Lebanese context and the differences between the methods used to evaluate soil liquefaction potential. A contribution to the actual discussion taking place concerning the depth factor r_d is also given.

RÉSUMÉ: Il existe plusieurs régions au Liban où les reconnaissances géotechniques démontrent la présence de couches de sable lâche à différentes profondeurs. Des régions côtières avec une nappe d'eau proche de la surface présentent un risque substantiel de liquéfaction. Cet article analyse l'évaluation du potentiel de liquéfaction basé sur les essais in situ SPT et CPT. Les sept méthodes étudiées sont basées sur l'approche de contrainte cyclique. Une analyse exhaustive a été réalisée afin de comprendre et comparer les résultats obtenus par ces méthodes. Des dépôts de sol dans la banlieue nord de Beyrouth ont été analysés en détail. Les résultats obtenus aident à comprendre le contexte géotechnique libanais et les différences entre les méthodes utilisées pour évaluer le potentiel de liquéfaction des sols. Une contribution à la discussion actuelle concernant le facteur de profondeur r_d est aussi faite.

KEYWORDS: Liquefaction Potential, Earthquake, CPT, SPT.

1 INTRODUCTION

Historically, liquefaction induced damage was essentially first observed in 1964 during the Good Friday Alaska earthquake (March 27th, magnitude 9) and later the same year in Niigata, Japan (June 16th, magnitude 7.5). More recently during the March 11th 2011 great Japan earthquake (magnitude 9), beside the ferocious tsunami, liquefaction was thoroughly observed in many parts of Japan. On the other hand, spectacular liquefaction failures were observed on June 13th 2011 in the city of Christchurch during the New Zealand earthquake (magnitude 6.3). The above events prove that liquefaction has been and will continue to be a worldwide research field (Dobry and Abdoun, 2011; Taylor and Cubrinovski, 2011). In Lebanon, an earthquake prone country due to its presence on the Dead Sea fault, loose saturated sandy deposits exist especially in the recently gained fill areas on the Mediterranean Sea shore and next to river beds. Construction projects are scheduled for these areas in the framework of urban and touristic development of the country, which without any doubt makes the understanding of liquefaction phenomena of upmost importance. Many methods are actually used to assess soil liquefaction potential as already discussed in literature (Rahhal, 2008; Rahhal and Zakhem, 2011; Boulanger and Idriss, 2014).

This paper starts by presenting the seismic context in Lebanon and the specific geologic and geotechnical conditions at the site of interest in the current work. Afterwards, an overview of the seven methods actually used by engineers to evaluate soil liquefaction potential is given. These methods are applied to a Lebanese case. The analysis and discussion that follow, aim on clarifying the relation between the different methods especially regarding the role of the depth factor r_d presently under discussion (Boulanger, 2010; Idriss and Boulanger, 2010; Seed, 2010). The brought out conclusions should be an asset to the entire earthquake geotechnical engineering community.

2 EARTHQUAKE SETTINGS IN LEBANON

The Eastern Mediterranean is a seismic zone and Lebanon has witnessed major seismic events. Lebanon occupies 230 km from 600 km up the eastern coast of the Mediterranean on the border between the Arabian and African plates. This region is seismically active through the great fault of the Dead Sea. The fracture of the Dead Sea has a length of approximately 1100 km and a north-south direction. This major fault joins the Red Sea to the Taurus Mountains in southern Turkey: this slip fault is recognized as the fracture of the Levant. The fault Yammouneh located at the western edge of the Bekaa Valley is the main active slip fault. The Roum fault is independent of the fault that runs through the city of Beirut. In addition, a new seismotectonic study (Elias et al. 2007) revealed the existence of a thrust fault at sea between Sidon and Tripoli and a length of 150 km. This fault is very close to the north coast (8 km distance). The earthquake of magnitude 7.5 (AD 551) could be attributed to this fault. The return period of these large earthquakes is estimated to be between 1500 and 1750 years.

In the last fifty years, five major events were identified: first the double shock of March 1956 which hit Lebanon had its epicenter on the northern boundary of the fault of Rum and the magnitude was 5.8. June 3, 1983 an earthquake of magnitude 5.3 was felt. On March 9, 1992 an earthquake of magnitude 4 has produced a dozen small tremors in the north. And in March 21, 1997 an earthquake of magnitude 5.3 struck the country with its epicenter again, as in 1956, as the northern boundary of the fault Roum. This earthquake was felt in Beirut with intensity 6 on the Modified Mercalli scale. Finally, nine years ago, an earthquake of magnitude 5.1 struck southern Lebanon on February 15, 2008, and property damage were reported in the south of the country, especially near the historic city of Tyre. The above discussed earthquake settings prove how much it is important to take earthquake conditions into consideration when trying to understand soil response or behavior.

3 GEOLOGY AND GEOTECHNICAL CONDITIONS AT THE ANALYZED SITE

The analyzed soil deposits are located in the northern suburb of Beirut, namely in Antelias-Dbaye near the coastal highway (Figure 1). The area is underlain by alluvial sand of the Quaternary age followed by fine bedded Limestone and Marly Limestone. An interpretation of the boreholes made available, reveal that the ground formation beneath the site surface consists typically of a top layer of fill material (sand and gravel) having a thickness of around 5m followed by alluvial deposits consisting of interbedding sand, clay, silt and gravels. The alluvial soil is underlain by interbedding marl and limestone. The thickness of the alluvial soil varies between 21 and 55m, deeper values being closer to the sea. The water table is near the surface and CPT and SPT tests indicate the presence of weak layers with SPT values around 5 at depths between 10 and 16m as shown in the typical case of Figure 2.

4 SOIL LIQUEFACTION POTENTIAL EVALUAION METHODS CONSIDERED

Seed and Idriss (1982) discussed a simple approach to evaluate stresses induced by an earthquake. They estimated that the normalized induced cyclic shear stress (CSR) at a depth z was proportional to a_{max} being the maximum horizontal acceleration of the earthquake as a function of gravity g, σ_v being the vertical stress at a depth z, and r_d being a reduction factor taking into account the deformability of the soil column located above the considered point. More recently, Idriss and Boulanger (2004), proposed to relate r_d to the depth z expressed in meters, and to the moment magnitude of the earthquake; these relations are appropriate for a depth z< 34m. On the other hand, the cyclic resistance ratio (CRR) giving the resistance developed by soil against liquefaction is defined similarly to the CSR. Methods used to measure CRR are based either on laboratory tests or on situ tests. As far as in situ tests are concerned, SPT (standard penetration test), and CPT (Cone penetration test) are the most used to obtain CRR. The CRR may be obtained through two approaches: First by correlating the N value (number of blows in SPT) or q_c (point or bearing resistance measured at the tip of the cone in the CPT) with the history of stresses in soil to know whether it has liquefied or not, the CRR being the limit separating liquefaction from non-liquefaction. A traditional liquefaction potential evaluation for a site is generally presented in the form of factor of safety F_s defined by the ratio CRR / CSR. Theoretically, the occurrence of liquefaction is in the case where $F_s \leq 1$. This approach is known as the deterministic approach. However, due to uncertainties in the model or parameters used, a factor of safety $F_s > 1$ obtained in the deterministic approach does not always correspond to a non-liquefaction condition.

The simplified approach to measure CRR is based on a reference earthquake magnitude of 7.5; the equivalent number of uniform cycles being proportional to earthquake magnitude, the minimum stress ratio (CSR minimum) required to cause liquefaction, that is equal to the CRR, decreases when magnitude M increases. A magnitude scaling factor (MSF) is used to correct the CSR value measured for an earthquake with a magnitude different from 7.5. This MSF is calculated based on correlations between the number of equivalent uniform cycles and the magnitude on one hand, and on the other hand based on relations obtained in the laboratory between the CSR required to cause liquefaction and the equivalent number of uniform cycles (Youd et al. 2001; Idriss and Boulanger 2004). The correlations giving CRR used to evaluate the liquefaction potential of a soil deposit under earthquake loading correspond to horizontal cases only (reference magnitude 7.5 and effective vertical stress of 100 kPa).

Figure 1. Aerial view of the studied coastal Area in the northern suburb of Beirut.

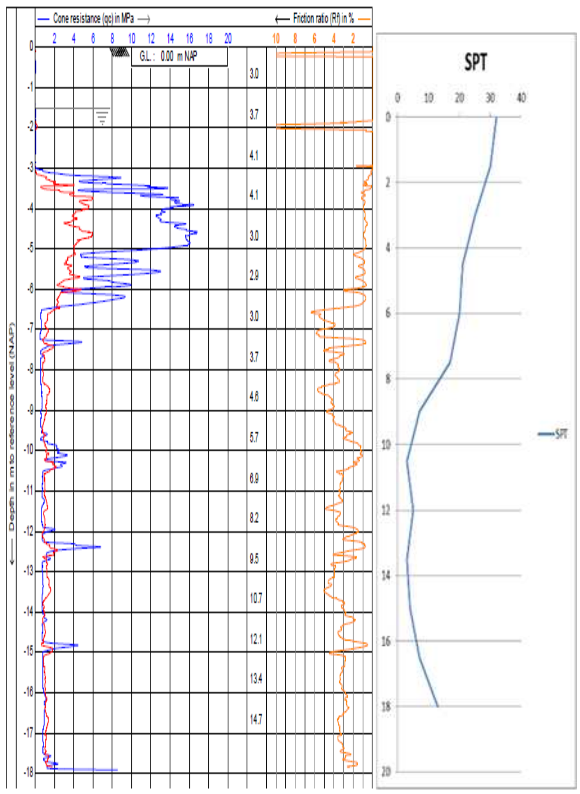

Figure 2. Representative CPT and SPT in situ soil data.

The terms K_α and K_σ are introduced respectively as correction coefficients taking into account the initial horizontal static shear and the effective vertical confining stress. Of course, CRR values increase as a function of fines content. In general, number of blows N_{60} and bearing resistance q_c are normalized for a confining stress σ'_{v0} of 100 kPa, in the aim of obtaining values depending on the relative density of sand, whatever the depth. Normalized values for N_{60} and q_c are designated respectively by $(N_1)_{60}$ and q_{c1}. The value of q_{c1N} is hence obtained. Robertson and Wride (1998) proposed to calculate the equivalent tip bearing resistance for a clean sand $(q_{c1N})_{cs}$. More recently, CRR curves as a function of $(q_{c1N})_{cs}$ have been reviewed by Idriss and Boulanger (2004).

5 ANALYSIS AND DISCUSSION

In this paper, seven methods to evaluate soil liquefaction potential will be applied to a site in the northern suburb of Beirut, Lebanon. The first two methods have been adopted by the NCEER (Youd et al. 2001): these are the CPT method from Robertson and Wride (1998) and the SPT method from Youd et al. (1985, 1997). Another two methods are respectively an SPT method and a CPT method studied by Boulanger and Idriss (2004). The next two methods are based on the work of Idriss and Boulanger (2008) for SPT and CPT. And finally, the seventh method is proposed by Seed (2010) including his comments with respect to the depth factor r_d. Calculations following the seven methods have been performed in the absence of any initial static shear stress ($K\alpha$ =1). Only the first top 20 m of the soil profile were analyzed. Figures 3, 4, 5, 6, 7 and 8 present the results of the study. Two scenarios were considered: First a magnitude 7 earthquake with three peak horizontal accelerations (0.3g; 0.2g and 0.15g) is investigated. Afterwards, the analysis is repeated with a magnitude 6 earthquake keeping same accelerations. The choice of these values takes into account eventual local site effects like acceleration amplification. In these Figures, the value of Factor of safety FS=1 separates liquefiable depths from non-liquefiable ones. It is very interesting to observe how safety factor profiles move according to the following three criteria: The method used to evaluate liquefaction potential, the earthquake magnitude and the peak horizontal acceleration. The trend denoted in the graphics is very consistent.

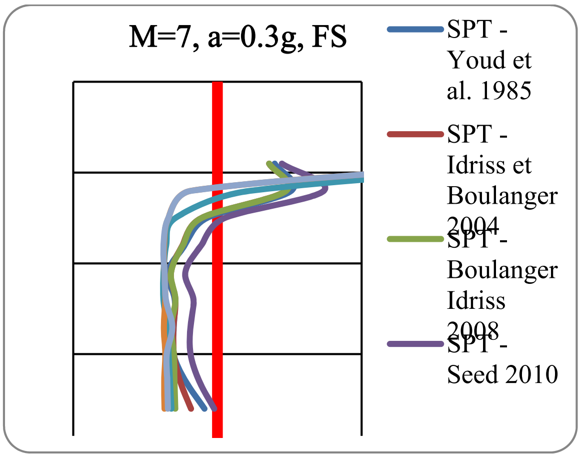

Figure 3. Factor of safety as a function of depth (m) for a_{max}=0.3g and M=7.

We may note that the methods of Boulanger and Idriss (2004 and 2008), the method of Youd et al. (1985) and the method of Robertson and Wride (1998) give higher depth factors rd, hence yielding lower security factors than the one calculated following the comments of Seed (2010). For a magnitude 7 earthquake, liquefaction is observed at all depths in the alluvial soils except in the case of a low acceleration of 0.15g where the factor of safety is still greater than 1 with the method of Seed (2010). The method of Seed (2010) is the least conservative because it always produces the highest security factors values. This is explained again by the fact that the r_d given by Seed (2010) is lower than in the other methods. For a magnitude 6 earthquake, the safety factors follow the same trend with higher values than in the preceding case (magnitude 7). The method of Seed (2010) giving still the highest safety factors, while the methods based on CPT (Boulanger and Idriss, 2004) yielding the lowest safety factors proving to be the most conservative among all. This result confirms previous findings by the authors (Rahhal and Zakhem, 2011).

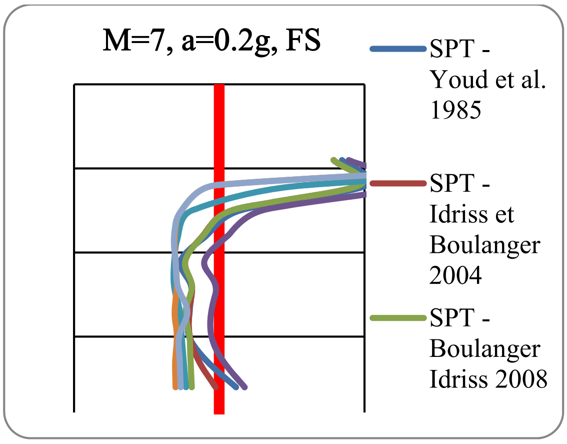

Figure 4. Factor of safety as a function of depth (m) for a_{max}=0.2g and M=7.

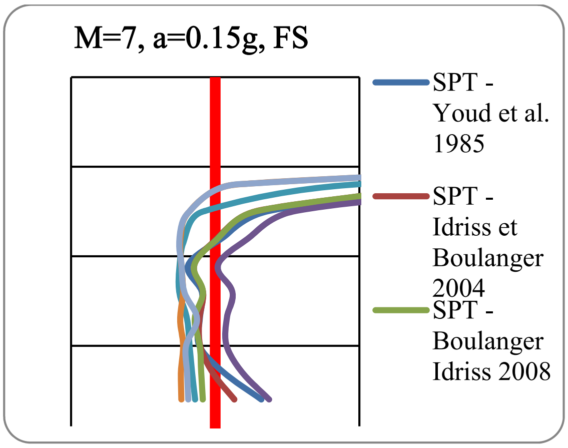

Figure 5. Factor of safety as a function of depth (m) for a_{max}=0.15g and M=7.

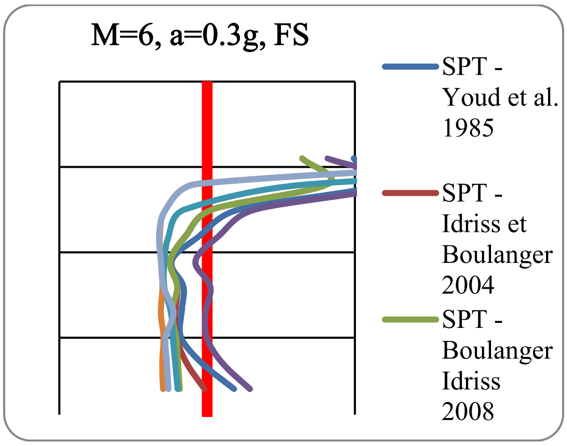

Figure 6. Factor of safety as a function of depth (m) for a_{max}=0.3g and M=6.

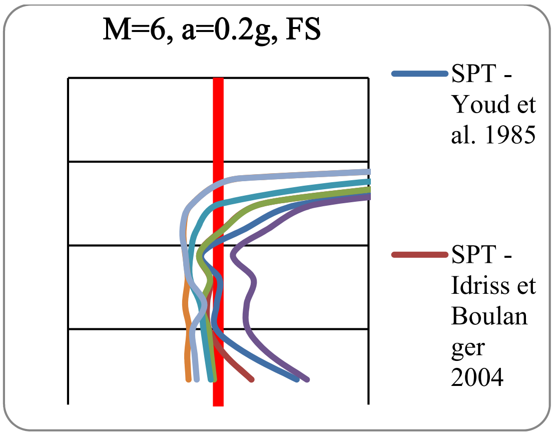

Figure 7. Factor of safety as a function of depth (m) for a_{max}=0.2g and M=6.

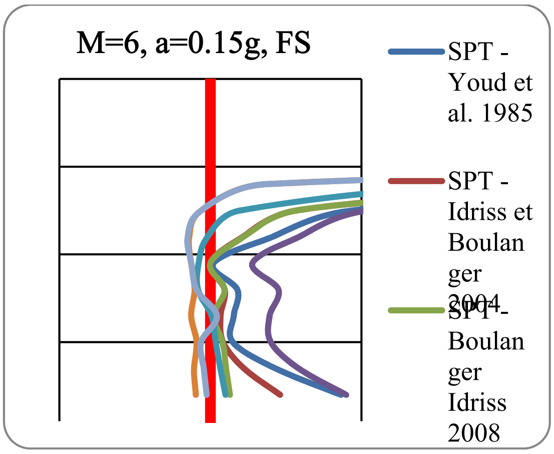

Figure 8. Factor of safety as a function of depth (m) for a_{max}=0.15g and M=6.

6 CONCLUSION

The present work allows us to highlight the importance of understanding liquefaction potential evaluation methods. Comparison between SPT and CPT tests based methods has been established. As far as factors of safety against liquefaction are concerned, the results are very close in general, and correspond to the geological and geotechnical soil profile. Analysis shows that the CPT based methods are the most conservative yielding the lowest safety factors. On the other hand, the effect of depth factor r_d on safety factor is also discussed. The achieved results help in the comprehension of the Lebanese geotechnical context and the differences between the methods used to evaluate soil liquefaction potential.

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