Journal of ENVIRONMENTAL Treatment Techniques

J. Environ. Treat. Tech.
ISSN: 2309-1185

Journal web link: http://www.jett.dormaj.com

SPT Based Soil Liquefaction Susceptibility Assessment: A Review

Sarah Tahsin Noor*, Sherajul Islam, Shaikh Mohammad Shamim Reza

University of Asia Pacific, Dhaka Bangladesh

Received: 23/01/2019 Accepted: 11/06/2019 Published: 30/09/2019

Abstract

Development projects in the low land areas are frequently carried out in Bangladesh after filled lands. Bangladesh lies in the seismic active zone. Therefore, during the earthquake, severe shaking or liquefaction of the ground may be experienced in these areas due to the presence of thick loose sand bed. When the loose sand is saturated and under moderate to high shear stresses, such as beneath a foundation or sloping ground, large shear deformations or even flow failure may take place due to the loss of shear strength accompanied by the softening. This paper presents the results of a study carried out to examine the variation of different variable parameters in the cyclic stress-based method while evaluating the liquefaction potential. The risk of liquefaction in Bangladesh and the issues that are needed to be addressed in evaluation in liquefaction evaluation are also discussed. The output of the study will enable the practicing engineer to assess liquefaction susceptibility of the construction site from the borehole data.

Keywords: liquefaction, SPT, cyclic stress ratio, cyclic resistance ratio

1 Introduction

Soil profiles, in seismic active zone, demands assessment of liquefaction susceptibility, in terms of liquefaction potential (alternately factor of safety against liquefaction), prior to the design of foundation for the proposed structure. This evaluation is important in choosing the type of foundation and in also its design with protection against liquefaction during any earthquake expected at that location during the life of the structure. During liquefaction, different foundation types suffers from different problems. If soil layer(s) at depth can liquefy during future seismic event, shallow foundation is not considered for supporting structures there, as it can sink into the liquefied soil and cause tilting the structure. In case of pile foundation, liquefaction causes two different problems. Most importantly, pile suffers from reduction in its capacity due to the development of negative skin friction on the pile surface, where positive skin friction was previously mobilized. Further, liquefied soil can pose lateral force on the pile [1]. Thus, the drag load due to negative skin friction needs to be considered during design of pile in liquefaction susceptible soil. Instead, by choosing raft foundation, sometimes liquefaction susceptible soil layers (if at shallow depths within top 15 m below the ground level) can be eliminated by planning an adequate number of basement floors. However, the basement walls needs to be designed with special consideration to the additional lateral pressures, thrusts and also to any changes in the effective lateral confinements, that may result from liquefaction in the lands adjacent to the site under consideration.

This paper presents the results of a study carried out to examine the variation of different variable parameters used in the cyclic stress based method, while evaluating the liquefaction potential. The risk of liquefaction in Bangladesh and the issues that are needed to be addressed in evaluation in liquefaction evaluation are also discussed. All equations are given in the literature section [2, 3, 4, 5, 6, 7].

Corresponding author: Sherajul Islam, University of Asia Pacific, Dhaka, Bangladesh. E-mail: sherajulbd@gmail.com.

2 Evaluation of Liquefaction Susceptibility

Liquefaction susceptibility of a soil profile can be evaluated by different methods based on the energy, the cyclic stress and the cyclic strain. The energy-based approach is theoretically based on the principle that the dissipated energy reflects both cyclic stress and strain amplitudes, while the theory of the cyclic strain based method is based on the fact that there might exist a threshold volumetric strain below, which densification does not occur. The time history of the cyclic shear strain is estimated from the ground response analysis.

In cyclic stress based method, both the earthquake induced loading (CSR) and the liquefaction resistance (CRR) of soil are expressed in terms of cyclic shear stress, and these two variables are compared in evaluating factor of safety (FS) against liquefaction or liquefaction potential. In general, soil liquefaction is expected to occur at the location, where the stress due to earthquake loading exceeds the resistance of the soil to liquefaction. The equation of determining FS, basically defined by the ratio of CRR to CSR, undergoes subsequent refinements over past 30 years [6]. equation 1 gives the present form of FS calculation against liquefaction:

$$FS = (CRR_{M=7.5}/CSR) \,.\, MSF.\, K_{\sigma}.\, K_{\alpha} \qquad (1)$$

Where, CSR= calculated cyclic stress ratio generated by the earthquake shaking; CRR = cyclic resistance ratio for magnitude 7.5 earthquakes; MSF = magnitude scaling factor; $K\sigma$ = correction factor for effective overburden pressure; and $K\alpha$ = correction factor sloping ground.

MSF and $K\sigma$ are to adjust CSR generated by any earthquake magnitude to a benchmark earthquake of moment magnitude (Mw) of 7.5 and to an equivalent $\sigma'v$ of 101kPa, respectively.

2.1 Cyclic Stress Ratio (CSR)

In literature, the intensity and duration of earthquake shaking, and the density and effective overburden pressure of the soil are considered the major influencing factors of liquefaction phenomena, as saturated and loose cohesionless soil liquefies due to earthquake tremor. CSR can be estimated in two ways: the simplified procedure as proposed by Seed and Idriss [8], and a detailed ground response analysis.

The simplified procedure (given by equation 2) is often used to calculate CSR generated by the earthquake shaking in practice [4, 9, 10]:

$$CSR = \frac{\tau_{avg}}{\sigma'_v} = 0.65 \times \frac{a_{avg}}{g} \times \frac{\sigma_v}{\sigma'_v} \times r_d \qquad (2)$$

where, τ_{av} = average equivalent uniform cyclic shear stress caused by the earthquake and is assumed to be 0.65 of the maximum induced stress; a_{max} = peak horizontal acceleration at ground surface generated by the earthquake; g = the acceleration of gravity; σ_v = total vertical overburden stresses; σ'_v = effective vertical overburden stresses; r_d = Stress reduction coefficient. The simplified procedure was verified with the case history data up to a depth of 15 m below the ground level.

2.2 Cyclic Resistance Ratio (CRR)

In literature, cyclic resistance ratio for moment magnitude of 7.5 (CRR_M=7.5) is formulated as a function of $(N_1)_{60}$ for clean sand and also for sand with different fines content (non-plastic). While deriving the relationship of CRR_M=7.5 with consideration for fines content, the SPT blow count of silty sands is converted to equivalent clean sand SPT blow count. Different relationships between (CRR_M=7.5) and $(N_1)_{60}$ are available in the literature [5, 7, 9]. The graphical representation of this relationship, given by [9], is widely used for calculating CRRM=7.5. Cyclic Resistance Ratio (CRR) can also be determined by equation 3, given by Rauch [7]:

$$CRR_{(M=7.5)} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10.(N_1)_{60}+45]^2} - \frac{1}{200} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Boulanger and Idriss [3] derived equation 4 for determining CRR value for cohesionless soil with any fines content:

$$CRR_{M=7.5} = \exp\left\{\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots.. (4)$$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

where, $\Delta(N_1)_{60}$ is the correction for fines content in percent (FC) in the soil and is expressed by equation 5:

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \qquad (5)$$

Standard penetration resistance $(N_1)_{60}$ value is used in this study after other corrections on the field measured value for overburden pressure, energy ratio, borehole diameter, rod length and the presence liner, according to equation 6:

$$(N_1)_{60} = N_m \,.\, C_N \,.\, C_E \,.\, C_B \,.\, C_R \,.\, C_S \qquad (6)$$

where, N_m= measured standard penetration resistance; C_N= factor to normalize Nm to common reference effective overburden pressure (approximately 100 kPa); C_E= correction for hammer energy ratio; C_B = correction factor for borehole diameter; C_R= correction for rod length and C_S= correction for samplers with or without liners.

3 Variables in Determining FS against Liquefaction

3.1 Peak ground acceleration (a_{max})

Ground movement is resulted from dispersion of earthquake energy in waves from its hypocenter. Peak ground acceleration (PGA) records the maximum rate of change of speed of these movements in absence of excess pore water pressure of liquefaction generated by the earthquake. PGA is generally considered the best determinate of damage in severe earthquakes. During earthquake, ground acceleration is measured in three directions: vertically for up-down shaking, and two perpendicular horizontal directions. In the cases where recorded motion were available, the larger of the two horizontal peak components of acceleration was considered as the a_{max} value of in the original derivation of CSR [6]. This provides a larger estimate of a_{max} but considered conservative and allowable. In the cases where recorded values are not available, peak accelerations are recommended for estimation from attenuation relationship. This method of determining the value of a_{max} is based on the geometric mean of the two orthogonal peak horizontal accelerations. As peak vertical accelerations are much smaller than a_{max}, this component of acceleration is ignored in the calculation of CSR. PGA of 0.5g is considered as very high level of ground shaking, as only well-designed buildings will survive after such an acceleration (even for a short period of time).

Different complex variable factors, including the length of fault, magnitude, the depth of the quake, the distance from the epicenter, the duration of the earthquake cycle and geology of the ground, are involved in the magnitude of ground acceleration (in terms of a_{max}) resulting from a given earthquake event. Thus, the value of a_{max}, during moderate to large earthquakes, can vary significantly within the sites those are a few kilometers apart, depending on the geologic features and ground type of the shaked zone. An earthquake of moderate magnitude can have the significant potential for generating a_{max} larger than that of larger magnitudes. Moreover, shallow focused earthquakes generate stronger acceleration than deep quakes. Furthermore, earthquakes of similar magnitude can generate different a_{max} due to variations in ground type.

3.2 Stress reduction coefficient (r_d)

Stress reduction coefficient (r_d) accounts for flexibility of the soil profile as a function of depth. At a depth, r_d varies within a range, depending on the variability at field sites. The range of r_d variation increases with depth, as noted from the r_d versus depth curves by Seed and Idriss [8]. These curves provide the maximum and minimum values of r_d considering different soil profiles.

In literature, several linear and polynomial equations [11, 12] were suggested for estimating the r_d values at different depths. For routine practice and noncritical projects, the equations by Liao and Whitman [12] are recommended for obtaining average values of r_d. These equations [Liao] yield almost the same average value for r_d. Minimum, maximum and average values of r_d are very close (such as 0.95, 0.98 and 0.97, respectively for a depth of 4 m) at shallow depths, while these values vary widely (such as 0.62, 0.92 and 0.75, respectively for a depth of 15 m).

3.3 Magnitude scaling factor (MSF)

MSF was first introduced by Seed and Idriss [13] to scale CRR value on the plot of $CRR_{(M=7.5)}$ versus $(N_1)_{60}$. In literature, several equations, as a function of earthquake moment magnitude (M_w), are available [6]. When M_w<7.5, MSF is greater than 1, and when M_w< 7.5, MSF is less than 1. For M_w of 5.5, MSF is found 1.43 and 2.2 from Seed and Idriss [13] and the revised equation by (reported in Youd et al. [6]). On the other hand, for a given M_w of 5.5, other equations give MSFs equal 3 or even greater than 4. It has been noted that Seed and Idriss [13] gives the lowest values of MSF for the M_w below 7.5, while the revised equation by Youd et al. [6] gives the smallest MSF for M_w greater than 7.5. Later, Boulanger and Idriss [2] formulated an equation of maximum MSF in terms of $(N_1)_{60}$ for including functional dependence on an index of the soil properties, in addition to the earthquake magnitude. According to their specifications, MSF should not exceed 2.2 in the calculation of FS against liquefaction.

4 Factor of Safety

4.1 Conclusions

The soil at depth of the measured SPT blow-count (employed in determining CRR) is predicted to liquefy when FS ≤ l, and is predicted as non-liquefiable when FS > l. The soil could be considered more resistant to liquefaction if calculated factor of safety is greater [14]. However, soil that has a factor of safety slightly greater than 1.0 may still liquefy during an earthquake, as FS against liquefaction depends on the magnitude of amax. For example, if a lower layer liquefies, then the upward flow of water could induce liquefaction of the layer that has a factor of safety slightly greater than 1. In this study, factor of safety is evaluated for three different magnitudes of a_{max} (0.1g, 0.2g and 0.3g), three different N-values (15, 25 and 30), and three fines content (less than 5%, 15% and 35%) up to a depth of 23 m below the ground level. The results are summarized in Table.

Table: FS against liquefaction for different a_{max} and SPT N values

$(N_1)_{60cs}$	a_{max}	Factor of Safety		
		FC=35%	FC=15%	FC<5%
15	0.1g	2 – 2.5	1.75 – 3.2	1.3 – 1.8
	0.2g	1 – 1.25	0.9 – 1.6	0.66 – 0.89
	0.3g	<1	<1	<1
25	0.1g	≈4	3.1 – 4.5	2.5 – 2.9
	0.2g	1.6 - 2	1.6 - 2	1.2 – 1.5
	0.3g	1.3 – 1.6	1 – 1.4	<1
30	0.1g	-	4.4 – 5.9	3.1 – 4.2
	0.2g	-	2.2 – 2.9	1.6 - 2
	0.3g	-	1.5 – 1.9	1.1 – 1.4

It can be noted that FS is highly dependent on the magnitude of a_{max}. During an earthquake causing a_{max} of 0.1g, none of $(N_1)_{60cs}$ may not liquefy. The same profile will liquefy by an earthquake causing a_{max} of 0.3g.

5 Liquefaction Risk in Bangladesh

Two devastating earthquakes that took place in April and October of 2015, caused severe damage and great loss of lives in Nepal, and in Pakistan and Afganistan, respectively. The former earthquake event of M 7.9 jolted Bangladesh several times through northern India, and left a trail of damage in Bangladesh with several buildings developing cracks or tilts across the country, including capital Dhaka. According to the historical records, Bangladesh was affected by earthquakes since ancient times, as the country is surrounded by five active tectonic blocks. These recent earthquakes are due to strain energy accumulation that have been taking place over the years. Within the last 150 years, Bangladesh was jolted by some damaging tremors: the Mandalay earthquake of 1858 affecting Chittagong division; the Srimangal earthquake of 1918 affecting Sylhet; the Bihar-Nepal earthquake of 1934 felt from Dinajpur and Rangpur; the Assam earthquake in 1950 felt throughout Bangladesh. Instead, earthquake, considered as the most destructive type of natural disasters, are not receiving sufficient attention in Bangladesh. New developments that are carried out on uncontrolled filling up of wetlands, are highly vulnerable to earthquake triggering liquefaction. This paper has identified some issues that need to be considered by the concerned authority:

a) Bangladesh should have more seismic observatory stations where earthquake moment magnitude and ground accelerations would be recorded. Seismic records are important to obtain the a_{max} value while evaluation liquefaction potential. The value of a_{max} is found to vary widely from 0.11g to 0.51g due to an earthquake tremor of M7.3 – M7.6.
b) In the evaluation of FS against liquefaction, magnitude scaling factor (MSF) should be considered equal 1. MSF has not yet been studied for the soil types usually used for filling up the wetlands. MSF is known to be affected by several factors, including the earthquake source characteristics, distance from the site to the source, soil profile characteristics and depth of the soil profile.
c) While filling the low lands, percent fines content may be increased by adding lime or fly ash in order to reduce liquefaction susceptibility or factor of safety against liquefaction.

6 Conclusion

Liquefaction susceptibility may only be reduced by modifying the properties of the soil, as arrangements of recording the magnitudes of amax and Mw can be recorded but an earthquake event is absolutely inevitable. The seismic records will aid in evaluation of FS with regional data and allow to design a soil stabilization technique (by using additives) for minimizing the probability of liquefaction. The soil condition of (N1)60cs equal 15 is quite critical and unsafe during amax greater than 0.2g. On the other hand, the soil condition of (N1)60cs equal 25 may be improved from liquefaction point of view by increasing percent fines during filling. However, the condition of (N1)60cs equal 30 is found quite stable even under amax of 0.3g.

Acknowledgment

Authors are very grateful to all the instructors, employees and other stuffs of The University of Asia Pacific for their cordial help and support.

Ethical issue

Authors are aware of, and comply with, best practice in publication ethics specifically with regard to authorship (avoidance of guest authorship), dual submission, manipulation of figures, competing interests and compliance with policies on research ethics. Authors adhere to publication requirements that submitted work is original and has not been published elsewhere in any language.

Competing interests

The authors declare that there is no conflict of interest that would prejudice the impartiality of this scientific work.

Authors' contribution

All authors of this study have a complete contribution for data collection, data analyses and manuscript writing.

References

[1] Noor, S.T., Hanna, A.M. and Mashhour, I. (2013), Numerical modeling of piles in collapsible soil subjected to inundation, *International Journal of Geomechanics*, 13(5), 514-526.

[2] Boulanger, R. W., and Idriss, I. M. (2015), Magnitude scaling factors in liquefaction triggering procedures, *Soil Dynamics and Earthquake Engineering*, 10.1016/j.soildyn.2015.01.004.

[3] Idriss, I. M., and Boulanger, R. W. (2006), Semi-empirical procedures for evaluating liquefaction potential during earthquakes, *Soil Dynamics and Earthquake Engineering*, 26, 115-130.

[4] Boulanger, R. W. and I. M. Idriss, (2004), State normalization of penetration resistances and the effect of overburden stress on liquefaction resistance, *Proc., 11th Int. Conf. on Soil Dyn. and Earthquake Engrg., and 3rd Int. Conf. on Earthquake Geot. Engrg,* D. Doolin et al., eds., Stallion Press, Vol.2, 484-491.

[5] Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K.., Harder, L. F., Kayen, R. E., & Moss, R. E. S. (2004), Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential, *J Geotechnical and Geoenvir Eng*, ASCE, 130(12), 1314–340.

[6] Youd, T. L., Idriss I. M., Andrus R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Liam Finn, W. D., Harder Jr, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson III W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe II, K. H. (2001), Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE,127 (10), 817-833.

[7] Rauch, A. F.(1997), EPOLLS: An Empirical Method for Predicting Surface Displacement Due to Liquefaction-Induced Lateral Spreading in Earthquakes,Ph.D.dissertation, Chapter 7,Faculty of the Virginia Polytechnic Institute and State University,VA. Page-120.

[8] Seed, H. B., and Idriss, I. M. (1971), Simplified procedure for evaluating soil liquefaction potential, *Journal of the Soil Mechanics and Foundations Division*, ASCE, 97(SM9), 12491273.

[9] Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. (1985), Influence of SPT procedures in soil liquefaction resistance evaluations, *Journal of Geotechnical Engineering*, ASCE, 111(12), 1425-1445.

[10] Seed, H. B., Idriss, I. M., and Arango, I. (1983), Evaluation of liquefaction potential using field performance data, *Journal of Geotecchnical Engineering Division,* ASCE, 109 (3), 458-482.

[11] Tokimatsu K, Yoshimi Y (1983), Empirical correlation of soil liquefaction based on SPT-N value and fines content. *Soils and Foundation*, JSSMFE, 23(4): 56-74.

[12] Liao, S. S. C., and Whitman, R. V. (1986), Catalogue of Liquefaction and Non-Liquefaction Occurrences during Earthquakes, *Research Report*, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA, USA.

[13] Seed, H. B., and Idriss, I. M. (1982), Ground motions and soil liquefaction during earthquakes, *Earthquake Engineering Research Institute*, Oakland, CA, 134 pages.

[14] Ishihara, K. (1993). Liquefaction and flow failure during earthquakes, *Geotechnique,* 43 (3), 351-415.

Author Profile

Dr. Sarah Tahsin Noor completed her PhD. in Civil Engineering from Concordia University, Montreal, Canada in 2011. She completed her Masters in Civil Engineering from Concordia University in 2005. She received the prestigious Alexander Graham Bell NSERC Canada Graduate Scholarship in her PhD program. She was also awarded with the Doctoral Research Scholarship from Quebec Government and four scholarships from Concordia University. Earlier, she completed her bachelor in Civil Engineering from BUET, Dhaka in 2002. Her present research topic includes pile foundation, unsaturated soil mechanics, problematic soils (including sensitive clay and collapsible soil), numerical modeling, etc. Sarah is a member of Golden Key International Honor Society.

Sherajul Islam received his Master of Engineering in Civil Engineering at University of Asia Pacific, Dhaka; in 2015 Also received his Bachelor of Science in Civil Engineering from Stamford University Bangladesh; Dhaka; in 2012. He has more then 6 years working experience on Water Supply, Sanitation, environmental impact assessment and Waste Management sectors. His research interests are in geotecnical engineering, environmetal polution control (air and noise), solid waste management, recycling of waste for sustainable development etc.

Shaikh Mohammad Shamim Reza received his Master of Engineering in Civil Engineering at University of Asia Pacific, Dhaka; in 2018 Also received his Bachelor of Science in Civil Engineering from Stamford University Bangladesh , Dhaka; in 2012. Currently he is doing Master of Business Administration at Army Isntitute of Business Administration, Savar under Bangladesh University of Professionals, Dhaka. He has more then 6 years working experience on Water Supply, Sanitation and Waste Management sectors. His research interests are in hazardous/medical waste management, municipal waste management, recycling of waste for sustainable development etc.