Evaluation of liquefaction potential based on SPT and CPT data in the Bandar-Abbas city

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Abstract

The geotechnical characteristics of the soil layers is one of the main factors influencing liquefaction potential of the ground. In common usage, *liquefaction* refers to the loss of strength in saturated, cohesionless soils due to the build-up of pore water pressures during dynamic loading. The following five screening criteria, are recommended for completing a liquefaction evaluation: Geologic age and origin, Fines content and plasticity index, Saturation, Depth below ground surface and Soil penetration resistance. The liquefaction resistance of soils can be evaluated using laboratory tests such as cyclic simple shear, cyclic triaxial, cyclic torsional shear, and field methods such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), and Shear Wave Velocity (Vs).
The present study is aimed at comparing the results of two field methods used to evaluate liquefaction resistance of soil, i.e. SPT and CPT.
Finally, it could be concluded that the liquefaction evaluation methods based on the SPT data show more conservative results compared with those based on the CPT data.

Key words: Liquefaction Potential, Standard Penetration Test (SPT), Pore Water Pressure, Dynamic Loading.

Introduction

Liquefaction is the phenomena when there is loss of strength in saturated and cohesion-less soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading.
A more precise definition as given by Sladen et al (1985) states that “Liquefaction is a phenomena wherein a mass of soil loses a large percentage of its shear resistance, when subjected to monotonic, cyclic, or shocking loading, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance”.
After initial liquefaction if large deformations are prevented because of increased undrained shear strength then it is termed,” limited liquefaction” (Finn 1990).
When dense saturated sands are subjected to static loading they have the tendency to progressively soften in undrained cyclic shear achieving limiting strains which is known as

cyclic mobility (Castro 1975; Castro and Poulos 1979). Cyclic mobility should not be confused with liquefaction. Both can be distinguished from the very fact that a liquefied soil displays no appreciable increase in shear resistance regardless of the magnitude of deformation (Seed 1979).
Soils undergoing cyclic mobility first soften subjected to cyclic loading but later when monotonically loaded without drainage stiffen because tendency to increase in volume reduce the pore pressures. During cyclic mobility, the driving static shear stress is less than the residual shear resistance and deformations get accumulated only during cyclic loading. However, in layman‟s language, a soil failure resulting from cyclic mobility is referred to as liquefaction.
Using the *SPT* data for evaluating liquefaction potential of the soil layers is nearly as long as the phenomenon was first recognized during 1964 Niigata earthquake.
Seed and Idriss (1971) developed the first experimental method based on the *SPT* data to evaluate the liquefaction potential of the ground during strong earthquakes.
In contrast to *SPT*, the *CPT* is also another in situ testing device and technique that can be used for the same purpose.
There are some initial requirements for each site to be under consideration in this study.
The results of the *SPT* and *CPT* studies must have been available and the points at which these tests are carried out cannot be far from each other.
Considering these fact, some different sites in the southern parts of Iran have been selected. These sites were located on the Hormozgan province near the coastal region of the Persian Gulf. The ground in these areas is usually consisted of deposits belonging to Testiary and Quaternary geological periods. The soil layers in these sites are between sandy silts to silty sands and can be classified as fine granular soils ($PI \leq 5\%$).
The water table in these sites are between 1.5-3.0 m depths and the densification of the top layers can be categorized between medium to loose.
The seismicity of the regions is relatively high compared with other areas of the country.

Effective Factors in Liquefaction

Liquefaction is most commonly observed in shallow, loose, saturated cohesionless soils subjected to strong ground motions in earthquakes. Unsaturated soils are not subject to liquefaction because volume compression does not generate excess pore water pressure.
Since liquefaction phenomena arises because of the tendency of soil grains to rearrange when sheared, any factor that prevents the movement of soil grains will increase the liquefaction resistance of a soil deposit.
Stress history is also crucial in determining the liquefaction resistance of a soil. Over consolidated soils (i.e. the soils that have been subjected to greater static pressures in the past) are more resistant to particle rearrangement and hence liquefaction as the soil grains tends to be in a more stable arrangement.
Liquefaction resistance of a soil deposit increases with depth as overburden pressure increases. That is why soil deposits deeper than about 15m are rarely found to have liquefied (Krinitzky et al.1993).
characteristics of the soil grains like distribution of shapes, sizes, shape, composition etc influence the susceptibility of a soil to liquefy (Seed 1979). While sands or silts are most commonly observed to liquefy, gravelly soils have also been known to have liquefied.

Rounded soil particles of uniform size are mostly susceptible to liquefaction (Poulus et al.1985). Well graded soils, due to their stable inter-locking configuration, are less prone to liquefaction.
Clays with appreciable plasticity are resistant to relative movement of particles during shear cyclic shear loading and hence are usually not prone to pore water pressure generation and liquefaction.
Ishihara (1993) gave the theory that non-plastic soil fines with dry surface texture do not create adhesion and hence do not provide appreciable resistance to particle rearrangement and liquefaction. Koester (1994) stated that sandy soils with appreciable fines content may be inherently collapsible, perhaps because of greater compressibility of the fines between the sand grains.

The Liquefaction Evaluation Method Used in the Study

Although there are different methods for evaluating liquefaction potential of the sand layers using *SPT* and *CPT* data, in order to avoid scattering the results, one of them which proven to be the most appropriate one, and has been used in many cases by different researchers, has been selected and used as below:

Robertson and Wride Method

This method is in fact based on the method, originally suggested by Seed and Idriss (1971). In this method the values of tip resistance of the CPT and also the number of SPT blows, are corrected in terms of the fine content according to one of the two following ways:

$$(N_1)_{60Cs} = Ks\,(N_1)_{60} \qquad (1)$$

In which

$$K_s = 0.025\,\mathrm{FC} + 0.875 \quad \text{for} \quad \% \le FC \le 35\%,\ PI \le 5\%, \&$$
$$K_s = 1 \quad \text{for} \quad FC \le 5\%,\ PI \le 5\%$$

where *FC* is the fines content measured from laboratory gradation tests on retrieved soil samples and *PI* is Plasticity Index of the soil. $(N_1)_{60}$ is *SPT* blow counts corrected for overburden stress.
The tip resistance of the *CPT* can be corrected by these equations:

$$(q_{c1N})_{cs} = K_c q_{c1N} \qquad (2)$$

In which

$$\text{if} \quad I_c \le 1.64, \quad K_c = 1.0$$

$$\text{if} \quad I_c \le 1.64, \quad K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88$$

I_c is the soil behavior type index obtained by using an Iterative Method and q_{clN} is the cone penetration resistance corrected for overburden stress.
In the second way, which has been developed in 1997, the following equations can be used to correct the *SPT* numbers and also the *CPT* tip resistance, respectively.

Seed and Idriss Method

The following equations, developed by I.M. Idriss with assistance from H.B. Seed are recommended for correcting standard penetration resistance determined for silty sands to an equivalent clean sand penetration resistance:

$$(N_1)_{60Cs} = \alpha + \beta(N_1)_{60} \quad (3)$$

where α and β are coefficients determined from the following equations:

$$
\begin{array}{ll}
\alpha = 0 & \text{for } FC \le 5\%, \\
\alpha = Exp.[1.76 - (190/FC)^2] & \text{for } 5\% < FC < 35\%, \;\; \& \\
\alpha = 5.0 & \text{for } FC \ge 35\% \\
\beta = 1.0 & \text{for } FC \le 5\%, \\
\beta = [0.99 - (FC^{1.5}/1000)] & \text{for } 5\% < FC < 35\%, \; \& \\
\beta = 1.2 & \text{for } FC \ge 35\%
\end{array}
$$

And for *CPT:*

$$(q_{C1N})_{CS} = q_{C1N} + \Delta(q_{C1N}) \quad (4)$$

in which

$$\Delta(q_{C1N}) = K_{CPT}\,(q_{C1N})_{CS}$$
$$\Delta(q_{C1N}) = [K_{CPT}/(1-K_{CPT})](q_{C1N})$$

where

$$
\begin{array}{ll}
K_{CPT} = 0 & \text{for} \quad AFC \le 5\%, \\
K_{CPT} = 0.0267\,(AFC\ -5) & \text{for } \%5 < AFC < 35\%, \; \& \\
K_{CPT} = 0.8 & \text{for} \quad AFC \ge 35\%
\end{array}
$$

Where the *AFC* is Apparent Fine Content, to be determined as follows:

if $I_c < 1.26$ apparent fines content FC (%) = 0

if $1.26 \le I_c \le 3.5$

$$\text{apparent fines content FC (\%)} = 1.75 I_c^{3.25} - 3.7$$

if $I_c > 3.5$ apparent fines content FC(%) = 100

This method has been used in the present study.

Comparison between Analysis Results

The comparison between the results of analysis has been made in terms of calculated safety factors, based on SPT data and CPT data belong to each site under consideration.
A linear regression has been used to correlate the analysis results and the correlation factors have been considered as the degree of relationship between these two methods. The safety factors against liquefaction using the Robertson and Wride method [7] for all sites have been calculated and shown in Figure (1).
As can be seen the results are very scattered. In ten points the absolute differences between their
safety factors are more than 1.0 (ABS> 1.0). If they are ignored.
the correlation factor will increase significantly, but this factor is still very small. The above points only cover 20% of all information poiI1ts, see Figure (1).
According to the general results of this study, as far as the fine non-cohesive soils are concerned, in spite of highly scattered results, an overall conclusion can be derived, in the way that the liquefaction potential evaluation of the ground by SPT data would be more conservative (Pessimistic) than that obtained by CPT data (Optimistic), see Figure (1).
As it was observed in this study, all sites selected were in the sandy silt to silty sand ranges, thus the results can be valid only for these fine granular soils. This classification can be also confirmed by CPT data belonging to the sites, see Figure (2).

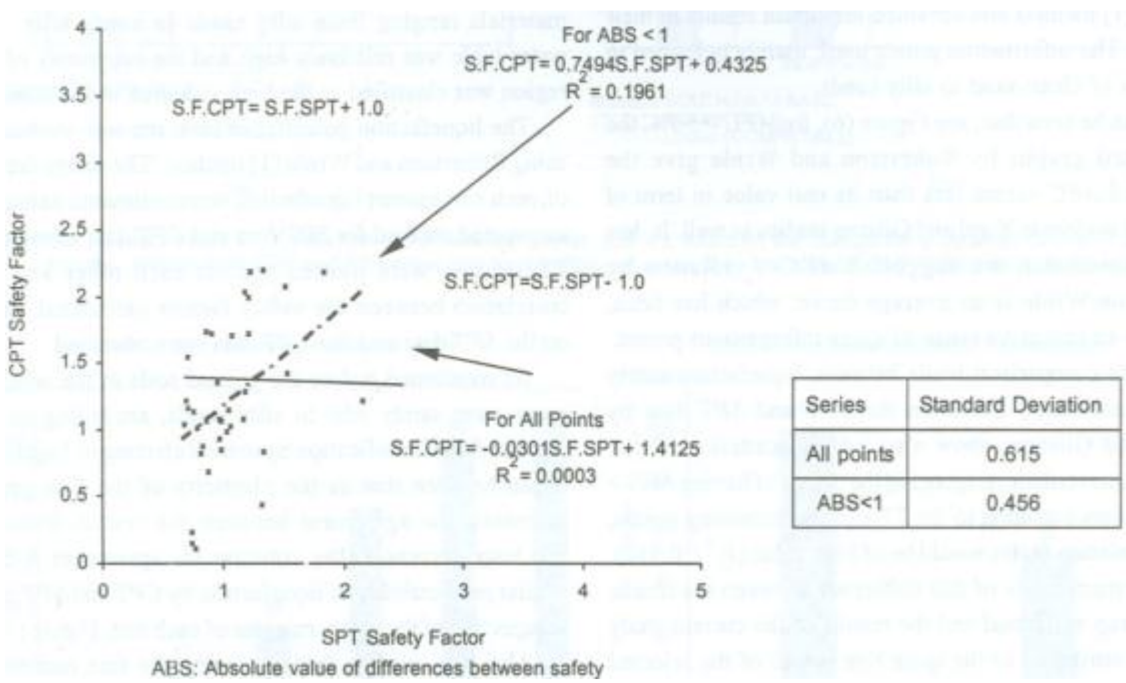

Figure 1. Comparison between safety factors against liquefaction using the method suggested by Robertson & Wride [8].

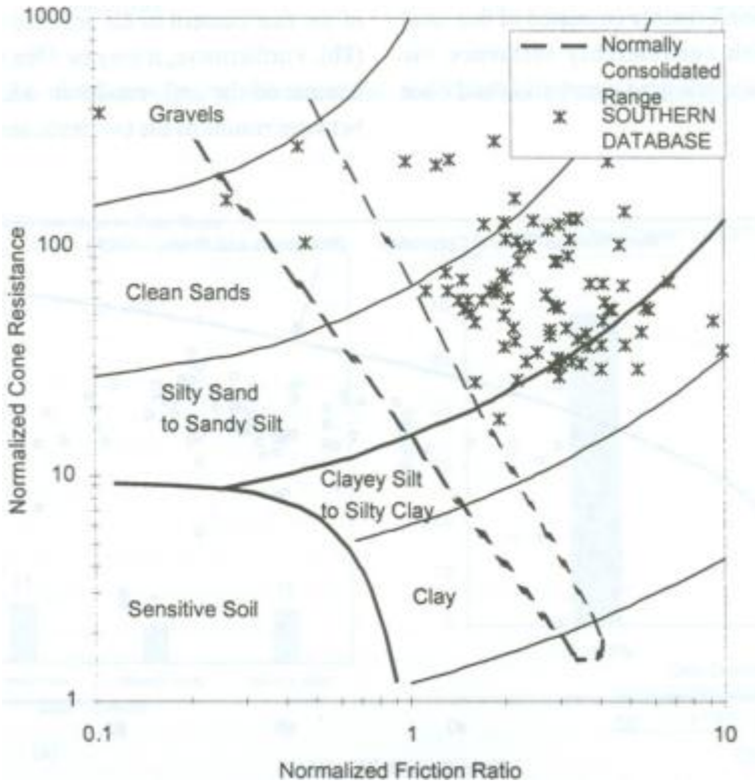

Figure 2. Soil Classification Based on CPT results.

Conclusion

Soil liquefaction is a major concern for structures constructed with or on sandy soils.
Due to the difficulty and the cost of obtaining high quality undisturbed samples, simplified methods based on in-situ tests such as the standard penetration test (SPT) and the cone penetration test (CPT) are preferred by geotechnical engineers for evaluation of liquefaction potential of soils.
Finally, it could be concluded that the liquefaction evaluation methods based on the SPT data show more conservative results compared with those based on the CPT data.

References

1. Castro, G. (1975) "Liquefaction and cyclic mobility of saturated sands". Journal of the Geotechnical Engineering Division, ASCE, 101 (GT6), 551-569.
2. Finn, W. L., Ledbetter, R. H., and Wu, G. *"Liquefaction in silty soils: design and analysis, Ground failures under seismic conditions"*, Geotechnical Special Publication No 44, ASCE, Reston, 51–79, 1994
3. Ishihara, K. (1993), "*Liquefaction and Flow Failure during earthquakes (Rankine Lecture)".*
Geotechnique, 43 (3): 351-415, 1993
4. Koester, J.P. (1994). "*The Influence Of Fine Type And Content On Cyclic Strength*" Ground Failures Under Seismic Conditions, Geotechnical Special Publication No. 44, ASCE, pp. 17-33
5. Krinitzky et al.(1993)
6. Poulos, S.J., Castro, G., and France, W. (1985). *"Liquefaction evaluation procedure"*, J. Geotechnical Engineering Div., ASCE, Vol. 111, No.6, pp. 772-792.
7. Robertson. P.K. and Wride, C.E. (1997*). " Cyclic Liquefaction and its Evaluation Based on SPT and, CPT "*, Final Contribution to the Proceedings of the 1996 NCEER Workshop on Evaluation of Liquefaction Resistance (T.L Youd, Chair).
8. Robertson, P.K. and Wride, C.E. (1998). *"Evaluating Cyclic Liquefaction Potential Using the Cone* Penetration Test", Canadian Geotechnical Journal, 35(3), 442-459.
9. Robertson, P.K.(1994*), "suggested terminology for liquefaction"*, An Internal CANLEX Report
10. Sladen, J. A., D"Hollander, R. D., and Krahn, J. (1985),"*The liquefaction of sands, a collapse surface approach"*, Can. Geotech. J., 22, 564– 578.
11. Seed, H.B. and Idriss, I.M. (1971). *"Simplified Procedure for Evaluating Soil Liquefaction Potential"*, J. of the Soil Mechanics and Foundations Division, ASCE, 97(SM9), 1249-1273.
12. Seed, H. B. (1979). *"Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquake"*, Journal of Geotechnical Engineering Division, ASCE, Vol 105, No. GT2, pp 201-225.
13. Selig, E.T., and Chang C.S.(1981), *"soil failure modes in undrained cyclic loading"* J. Geotech. Engg. Div.,ASCE, Vol.107, No.GT5, May, pp 539-551
14. Youd, T.L. and Idriss, I.M., eds (l997). *"Proceedings of the NCEER on Evaluation of Liquefaction Resistances of Soils Tech. Report NCEER-1997-0022"*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York.

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