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Liquefaction Potential and Liquefaction Hazard Analysis of Srinagar City, Kashmir Valley

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ABSTRACT

Liquefaction is one of the most important, interesting, complex and controversial topics in geotechnical earthquake engineering. Its effects became known only after it had caused significant damage already involving the Alaskan Earthquake of 1964 and the Niigata earthquake in Japan, both the earthquakes were highly devastating given the collapse they caused to buildings and bridges alike. The term "*liquefaction*" was originally coined by Mogami and Kubo (1953), and this has been used ever since in conjunction with a variety of phenomena that involve soil deformations caused by disturbance of saturated cohesionless soils under undrained conditions. When cohesionless soils are saturated, rapid loading occurs under undrained conditions so this causes excess pore pressures to increase and effective stresses to decrease. Large civil engineering projects require weeks or months to build which makes the rate of loading on the structures very low, thus giving ample amount of time for the drainage to take place from the soil, cohesionless soils in this case have an ample amount of time to draw water into or out of the voids as they expand or contract. Little or no excess pore water pressure develop in these situations because the potential rate of loading is lesser than the rate of drainage. However the rate of loading is sometimes so rapid that even cohesionless soils cannot drain quickly enough.

Introduction

The most noteworthy example is the loading due to an earthquake, which is much faster than the rate of drainage. This is especially problematic in loose saturated sands because they tend to compress when loaded which normally would force some water out of the voids. However since the loading is so rapid , the water cannot easily drain away and positive excess pore pressure develops instead. As these pressures build up, both the effective stress and the strength decrease. Sometimes the effective stress drops to zero, which means the soil loses all its shear strength and thus behaves as a dense liquid .Not all soils are susceptible to liquefaction, there are many works that have aimed to classify the soils based on their susceptibility to liquefaction, and among them the notable ones are : Youd's criteria of liquefaction, Boulanger and Idriss' criteria for liquefaction and there are Chinese criteria that are widely used. Because liquefaction has frequently been observed to occur at the same location when site conditions are unchanged (Youd, 1984), evidence of the historical occurrence of liquefaction, either observed or in the form of paleoliquefaction, can be taken as evidence of liquefaction susceptibility. Geologic conditions can also indicate susceptibility to liquefaction; soils deposited in fluvial deposits, and colluvial and aeolian deposits when saturated, are likely to be susceptible to liquefaction. Liquefaction is also observed in alluvial-fan, alluvial-plain, beach, terrace, playa and estuarine deposits, but not as consistently as in those listed previously.

Younger soil deposits are generally more susceptible to liquefaction than older deposits. The physical composition of a soil deposit will play a strong role in determining its liquefaction susceptibility (Kramer, 1996). Uniformly graded clean sands composed of rounded particles are inherently most susceptible to liquefaction. Well-graded soils and soils with angular particles are less susceptible. The presence of fines, particularly plastic fines (PI>10) tends to decrease liquefaction susceptibility.

The liquefaction susceptibility of a given soil is also influenced by its state, i.e., its in situ effective stress and density conditions. The tendency of a soil to contract or densify under cyclic loading conditions has long been known to be influenced by both density and effective stress. Loose soils are much more susceptible to liquefaction than dense soils and, for a given density, soils under high effective confining pressures are more susceptible to liquefaction than soils at a low effective confining pressure. High values of the state parameter (Been and Jeffries, 1985), defined as the difference between the void ratio and the steady state void ratio, indicate increasing contractiveness and hence, increasing susceptibility to liquefaction. The state parameter can be estimated from CPT resistance (Been et al., 1986, 1987).

The liquefaction potential of a soil deposit can be studied via the mechanical properties of the deposit too. It is done by defining two parameters which govern the behaviour of the deposit during dynamic earthquake loading.

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The first being the CRR called the cyclic resistance ratio and CSR called the cyclic stress ratio, the CRR tells the total stress ratio required to cause the liquefaction, and the CSR is the stress ratio that is acting on the soil during earthquake loading. Quite obvious is the fact that if the ratio CRR/CSR is less than unity it will mean that the soil can take less stress than what is acting on it and will liquefy.

Liquefaction Susceptibility of the Site Under Study

The first step is evaluating the susceptibility to liquefaction of a site under study, various important factors that were studied were.

Susceptibility based on the location of water table

The location of water table plays an important role in as far as the susceptibility to liquefaction is concerned, the site was evaluated as per the criteria setup by different researchers from time to time and different building codes, among which the most widely used is the Chinese code according to which, If the estimated maximum past, current, and maximum future ground-water levels (i.e., the highest ground water level applicable for liquefaction analyses) are determined to be deeper than 50 feet below the existing ground surface or proposed finished grade (whichever is deeper), liquefaction assessments are not required. But as far as the site under study is concerned the water table was not deep enough, the depth of water table hardly exceeds 1m with depth in most of the cases ranging from 0.6m to 1m. Thus to a good extent justifying the liquefaction study and analysis.

Susceptibility based on SPT test results

For the estimation of susceptibility based on the **SPT N** the criteria is that the corrected **SPT** value **$(N_1)_{60}$** must be less than 30 in all samples with a sufficient number of tests. But as far as the results of the tests done are concerned the values were greater than 30 but not in all samples and not even in majority of samples, so the liquefaction study cannot be ruled out based on the SPT N value, thus further justifying the liquefaction study of the area. As per the Chinese criteria the liquefaction study becomes unavoidable if the following three conditions are met:

- Percent finer than 0.005 mm less than 15 percent.
- Liquid Limit less than 35
- Water Content greater than 0.9 x Liquid Limit.

The above three conditions if met make liquefaction study due, but if not then it's a matter of choice whether to carry out liquefaction study or not, because in no case can liquefaction be ruled out completely given the complicated nature of soil which cannot be a perfect cohesive or cohesionless media **Idriss and Boulanger** "***Soil Liquefaction during Earthquakes, EERI***" .

Procedure to Carry Out Liquefaction Triggering Analysis:

The procedure used to carry out the liquefaction analysis study was the simplified procedure given by the **Seed and Idriss 1971**. This procedure was given taking into account the fact that not all the borings go deep enough to reveal in detail the composition of the soil profile, thus making it a procedure independent of the composition of soil. The procedure involves the following important parameters to be used during the analysis,

Magnitude of the Earthquake and PGA

From IS 1983 it is clear that the zone factor **Z** for J&K is around 0.36, so here in the study a PGA of 0.4g was used, from the Magnitude Deaggregation results it was clear that the probability of the PGA being 0.4g is maximum when a Magnitude 5.7 or close to 6 hits the site. So the earthquake magnitude used in the study was 5.7 itself.

Standard Penetration Test values

The SPT results were obtained from the Geotechnical investigation report prepared by ERA, there were a total of 18 boreholes made and a **DPR** was prepared for all the boreholes. The depth of each borehole being about ***30m***. The data have been given at the end of the report in the appendix.

Corrections for SPT N values

The SPT N values were corrected for Dilatancy and for overburden as well

Evaluation of CSR (Cyclic Stress Ratio)

CSR of a soil deposit represents the induced stress in the deposit it numerically is a ratio of the total vertical stress to the effective vertical stress. Thus it is easy to infer that whether a soil liquefies or not depends on the CSR and the stress that the soil deposit can take without losing its strength completely. The total CSR that the soil can take while being stable is called the CRR of the soil.

The value of CSR was found using the relation given by **Seed and Idriss** given as under

$$CSR = 0.65 \frac{a}{g} \frac{\sigma_{vc}}{\sigma'_{vc}} r_d$$

σ'_{vc} *and* σ_{vc} are the effective vertical stress and the total vertical stresses respectively. r_d represents the stress reduction coefficient to take into account the flexibility of the soil deposit. This actually is a correction factor as the development of equation was based on rigid beam theory, wherein the soil column was assumed to act as a rigid body. 0.65 is an arbitrary stress reference level used in the equation and has been used ever since. It is found by using the expression given by Idriss et.al 1999.

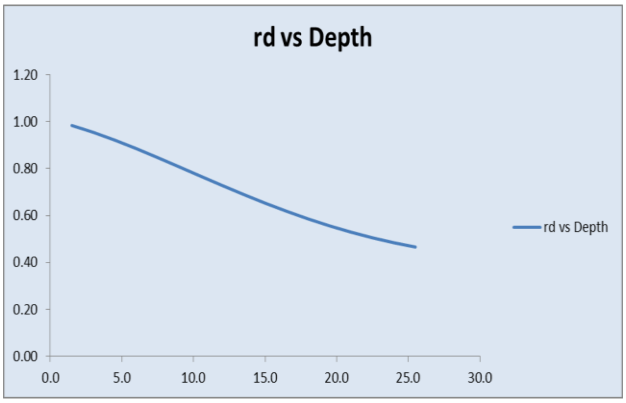
$$r_d = \exp(\alpha(z) + \beta(z)M)$$

where

$\alpha(z)$ =

$$\alpha(z) = -1.012 - 1.126 \sin(z/11.73 + 5.133)$$

$$\beta(z) = 0.106 + 0.118 \sin(z/11.28 + 5.142)$$



Variation of stress reduction coefficient with depth.

Evaluation of CRR (Cyclic Resistance Ratio)

The value of CRR was found using the SPT N values as under:

CRR = exp(N/14.1 + $(N/126)^2$ - $(N/23.6)^3$ + $(N/25.4)^4$ – 2.8)

The above expression was given by **Idriss and Boulanger** in 2004. While going for the liquefaction analysis as given by **Idriss in 1971** the liquefaction triggering is evaluated in terms of Factor of Safety which is a ratio between the CRR and CSR of the soil which if less than unity will trigger liquefaction. The method given by Idriss involved the analysis taking into consideration an earthquake of magnitude 7.5 and if the magnitude is any different than this value there are correction factors that need to be applied. In the present study the magnitude of earthquake that was used was a moment magnitude of 5.7 so there were correction factors that were to be applied. The correction factors applied are for magnitude and overburden.

Correction Factors Applied

K_σ is the correction for overburden and is to be applied
K_M is the magnitude correction factor.
The values of both the correction factors have been given by researchers in the literature and the values can be obtained from the following equations.

Overburden correction

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vc}}{P_a}\right)$$

The value of $C_\sigma = \dfrac{1}{18.9-2.55\sqrt{(N_1)_{60}}}$

The value of **$(N_1)_{60}$** to be used here is limited or restricted, its value to be used should not exceed 37 as per Idriss and Boulanger 2004. This was done in order to best fit the experimental findings.

Magnitude correction

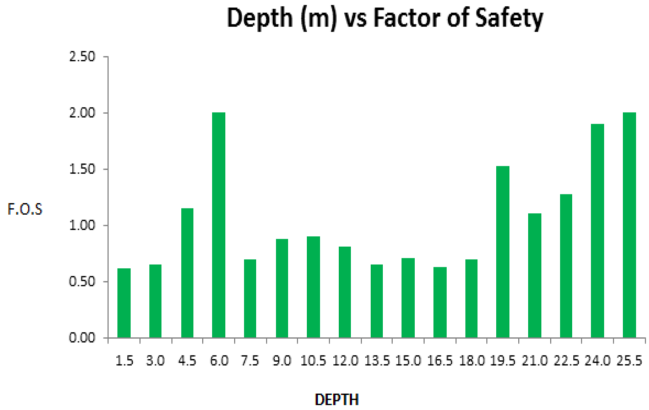
$$K_M = 6.9\exp(-M/4) - 0.058$$

Here M is the moment magnitude, this is also called the MSF or Magnitude Scaling Factor, its just a factor that is used to adjust the CSR and CRR to a common value of M.

After the evaluation of CSR and CRR the next step is to evaluate the factor of safety for the soil deposit by using the expression.

$$FS = CRR/CSR$$

The evaluation of the factors of safety at each interval depth has been done as shown in the table below in the bar chart. It can be seen that there are variations in the factor of safety at varying depths. As per the IS standards the value of the factor of safety good enough for important projects where the safety is of top priority is 1.3 and that for other projects is 1.1



It can quite clearly be seen that the depth does not have a clear cut relationship with the FOS, and that it varies randomly with depth which is obvious given the number of different parameters apart from depth that influence the FOS value.

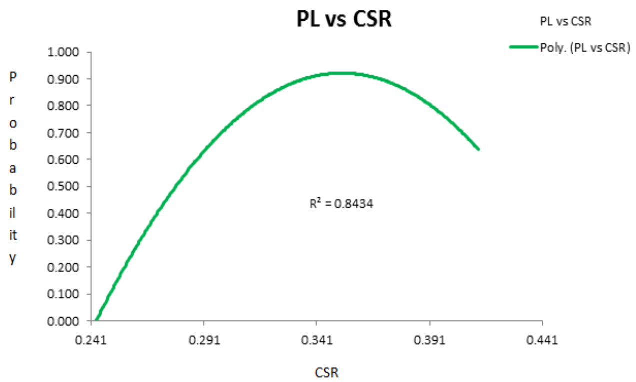
It was **Liao et.al** who gave an expression that evaluated the probability of liquefaction for a soil deposit based on the CSR value and the corrected N value for the site. The area under study was also checked for the probability of liquefaction. The expression used was as under

$$P_L = \frac{1}{1+\exp[-(\beta_0+\beta_1\ln(CSR)+\beta_2(N_1)_{60}}$$

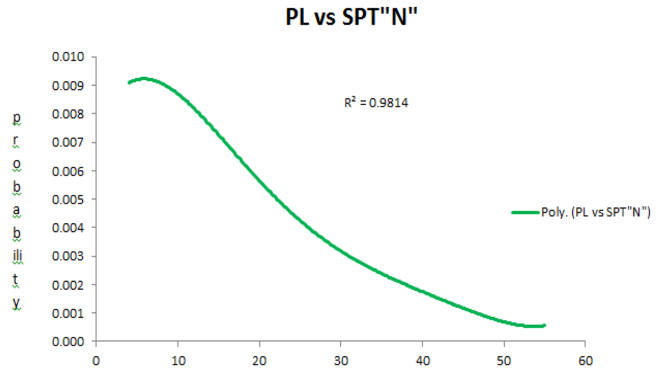
The values β_0, β_1, β_2 are found using the table derived by Krammer from Liao et.al.

Data	**Numberof cases**	**β_0**	**β_1**	β_2
All cases	278	10.2	4.196	-0.244
Clean sand cases only	182	16.4	6.46	-0.368
Silty snad cases ojnly	96	6.48	2.68	-0.182

The calculated probabilities of liquefaction can be arranged in the form of a graph plotted against CSR of the soil. The probabilities were calculated at each depth for the site in the study and the graph was plotted against the SPT N value for an earthquake magnitude 5.7 and PGA of 0.4g



CSR Vs Probability of Liquefaction



SPT N (Corrected)

Probability of liquefaction variation with SPT N

Results and Discussions

As far as the site under study is concerned the soil profile is not that highly prone to liquefaction, from the borehole results it can be seen that in most of the boreholes the factor of safety usually is closer to 1, and that deeper deposits don't always guarantee less liquefaction susceptibility as is evident from fig.2 which shows an erratic variation with depth. The variation with the CSR values is somewhat gradual with the probability of liquefaction increasing upto a value of around CSR=0.35 where the probability of liquefaction approaches unity. Thereafter the probability comes down with the increase in CSR value. The variation with SPT N however is more significant and reliable and conclusive as well. With the probability of liquefaction showing a gradual decrease with the increasing SPT N value.

References

Seed HB. and Idriss IM. "A simplified procedure for evaluating soil liquefaction potential." Journal of Soil Mechanics and Foundation Engineering, ASCE, 1971.

I.M. Idriss and R.W. Boulanger. "Soil liquefaction during Earthquakes" Earthquake Engineering Research Institute. 59-115.

Steven L Kramer. "Geotechnical earthquake engineering", Pearson Education.Liao SC and Whitman RV "Overburden correction factors for SPT in sand,"Journal of Geotechnical Engineering, ASCE, 1985.

Liao SC, Veneziano D, Whitman RV. "Regression models for evaluating liquefaction probability," Journal of Geotechnical Engineering, ASCE, 1988.

Youd TL, et al. "Liquefaction resistance of soils: Summary report from the 1996 NCEER/NSF workshops on evaluation of liquefaction resistance of soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 2001.

Seed HB, Tokimatsu K Harder LH and Chung R. "The influence of SPT procedures in soil liquefaction resistance evaluations," Journal of Geotechnical Engineering, ASCE, 1985.

Been and Jeffries, " A State Parameter for Sands" Journal of Geotechnique, 1985.