

Assessment of Seismic Liquefaction through detailed Seismic Study of Soil- A case study

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Abstract - Soil liquefaction has caused several damages in the past to lifeline and structures. Liquefaction is a phenomenon where soils suffer loss in shear strength due to cyclic loading such as an earthquake. The mapping of liquefaction of soil can be done using seismic, geotechnical, and topographical parameters. Obtaining a suitable seismic parameter is a key challenge that can aid to qualitative mapping of Liquefaction potential of a soil stratum. This paper presents a comparative study conducted at a site in Gujarat to evaluate the liquefaction potential of soil using two different techniques, involving the usage of the obtained Peak Ground Acceleration (PGA). Bhuj (2001) earthquake was one of the most devastating earthquakes that occurred in Gujarat in recent years with a magnitude of 7.7. The site is 105 km from the epicenter of Bhuj (2001) earthquake for which attenuation relationship is incorporated, and the PGA is assessed using Linear Ground Response Analysis using the SPT information. Plasticity of soil plays a key role in determining the Liquefaction potential of the soil stratum. This paper devises a significant impact in the field of liquefaction assessment for soils with moderate to high plasticity and intends to add to the body of knowledge on liquefaction studies.

Keywords: Liquefaction, Plasticity, Clays, Sands

1. INTRODUCTION

The strength and serviceability of a structure determine its stability. Liquefaction can lead to ground subsidence, lateral spreads, disruption of vital services, and significant damage to infrastructure. Cyclic liquefaction is a phenomena whereby cyclic loading, like an earthquake, resulting in loss of its shear strength partially or completely.

2. LITERATURE REVIEW

The first conventional method to assess liquefaction using the Standard Penetration Test (SPT) was the streamlined process put out by Seed and Idriss (1982) [1]. Although liquefaction was long thought to be limited to sandy soil deposits, a few observations made after many earthquakes showed that it may also occur in fine-content soils with medium to low plasticity. The Haicheng and Tangshan earthquakes in 1975 and 1976 caused silty sand to liquefy into slightly sandy silt soils. Wang (1979)[2] was the first researcher to point out this phenomenon and proposed a criterion that clayey soils could only be liquefiable if all three of the following conditions were met- percent of particles less than 0.005 mm < 15%, liquid limit (LL) < 35% and ratio of water content and liquid limit (w_c/LL) > 0.9 (Wang, 1979 [2]; Seed and Idriss, 1982 [1]). This standard was termed as Chinese criteria due to its origin. Threshold-level plastic soils are generally immune to flow liquefaction failures and the ensuing deformation and strength loss. The study's findings led to the conclusion that the liquefaction response of soil can be defined by a specific range of the plasticity index and the water content to liquid limit ratio. The study categorized liquefaction response into susceptible, moderately susceptible, and non-susceptible classes. Bray and Sancio (2006) [3] reviewed previous works and defined a new set of PI and w_c/LL range criteria. Boulanger & Idriss (2006) [4] categorized soils as "sand-like" and "clay-like" considering their plasticity index values. The importance of PI as a parameter for analyzing the liquefaction potential of saturated clayey soils was verified by Gratchev et al. (2006) [5].

The correlation between Cyclic Resistance Ratio (CRR) and SPT-N value is presented Indian Standard Code 1893 (Part 1): 2016 [6]. It is one of the most simplified approaches to determine the liquefaction potential of silt and sand. SPT-N values is one of the most popular, oldest, and frequently used in-situ tests for the subsoil investigation because of its simplicity.

Factor of Safety (FOS) is calculated to determine the likelihood of liquefaction. FOS is calculated using SPT-N. SPT-N is the most common parameter for the prediction of liquefaction. FOS is the ratio of CRR to the Cyclic Stress Ratio (CSR) using equation (1). CRR is considered the fundamental parameter for the estimation of liquefaction, which is mainly observed

during medium and strong seismic excitation in sand-silt mixture and based on the soil properties. If $FOS < 1$, the soil is susceptible to liquefaction and if $FOS \geq 1$, soil is not prone to liquefaction. When the design ground motion is conservative, earthquake related permanent ground deformation is generally small, if $FOS \geq 1.2$ (IS 1893 Part-I, 1893)[6].

$$FOS = \frac{CRR}{CSR} \quad (1)$$

CSR is calculated using the Eqn. (2) proposed by Seed and Idriss (1971) [7].

$$CSR = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_v'}{\sigma_v} \times r_d \quad (2)$$

Where, a_{max} is PGA, g = acceleration due to gravity (9.81 m/s²), σ_v is total vertical stress, σ_v' is effective vertical stress and r_d is stress reduction factor with depth.

Where,

$$r_d = 1 - 0.00765z; 0 \leq z \leq 9.15 \text{ m} \quad (3)$$

$$r_d = 1.174 - 0.0267z; 9.15 \leq z \leq 23 \text{ m} \quad (4)$$

Beyond 23m from ground level, IS Code does not provide for the reduction factor.

CRR is ascertained by correcting $CRR_{7.5}$ for earthquake magnitude, high overburden stress level and high initial shear stress using the equation (5):

$$CRR = CRR_{7.5} (MSF) K_\sigma K_\alpha \quad (5)$$

Where, $CRR_{7.5}$ is Standard cyclic resistance ratio for a 7.5 magnitude earthquake correlated with the corrected SPT data,

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60CS}} + \frac{(N_1)_{60CS}}{135} + \frac{50}{(10 \times (N_1)_{60CS} + 45)^2} - \frac{1}{200} \quad (6)$$

Where, term $(N_1)_{60CS}$ depends on $(N_1)_{60}$ and given as:

$$(N_1)_{60CS} = \alpha + \beta (N_1)_{60} \quad (7)$$

Where,

$$\alpha = 0 \quad \beta = 1 \quad \text{for } FC \leq 5 \text{ percent}$$

$$\alpha = e^{[1.76 - (\frac{190}{FC^2})]} \quad \beta = 0.99 + \frac{FC^{1.5}}{1000} \quad \text{for } 5 \text{ percent} < FC < 35 \text{ percent}$$

$$\alpha = 5 \quad \beta = 1.2 \quad \text{for } FC \geq 35 \text{ percent}$$

$(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure of about 100 KPa and a hammer energy ratio or hammer efficiency of 60%.

$$(N_1)_{60} = C_N N_{60} \quad (8)$$

C_N is Correction factor for overburden.

$$C_N = \left(\frac{Pa}{\sigma_v'} \right)^{0.5} \leq 1.7 \quad (9)$$

$$N_{60} = N * C_{60} \quad (10)$$

N is the Uncorrected SPT blow count which is the quantification of resistance to penetration of a sampling spoon under dynamic or static loading. This is corrected to an overall 60% efficiency factor given below.

$$C_{60} = C_{HT} C_{HW} C_{SS} C_{RL} C_{BD} \quad (11)$$

C_{HT} - Correction factor for Non-standard hammer weight or height of fall,

C_{HW} - Correction factor for Non-standard hammer weight or height of fall,

C_{SS} - Correction factor for Non-standard sampler setup,

C_{RL} - Correction factor for short rod length,

C_{BD} - Correction factor for Nonstandard borehole diameter,

MSF - magnitude scaling factor given by the following equation:

$$MSF = \frac{10^{2.24}}{M_W^{2.56}} \quad (12)$$

M_W - Magnitude of earthquake.

K_σ = Correction for high overburden stresses when overburden pressure is high (depth>15m)

$$K_\sigma = (\sigma'_v/P_a)^{(f-1)} \quad (13)$$

Where,

σ'_v - effective overburden pressure,

P_a - atmospheric pressure,

f is an exponent, dependent on the relative density (D_r), for D_r is 40-60%, $f= 0.8\sim 0.7$ and for D_r is 60-80%, $f= 0.7\sim 0.6$

K_α = Correction for static shear stresses required for sloping ground and is not necessary in normal engineering practice and value assumed to be unity.

Idriss and Boulanger (2004) [8] provides one of the approaches to ascertain the liquefaction potential of silts and clays of plasticity index $\geq 7\%$. For liquefaction prediction of the soil deposits, the $CSR_{7.5}$ that indicates the loading imposed by earthquakes on the soil is provided in the following equation below:

$$(CSR)_{7.5} = 0.65 \left(\frac{\sigma_v a_{max}}{\sigma'_v} \right) \times \frac{r_d}{MSF} \times \frac{1}{K_\sigma} \quad (14)$$

In which, σ_v and σ'_v are respectively the total and effective overburden pressure at a depth z ; a_{max} is the peak horizontal ground acceleration in g 's; r_d is a stress reduction factor. To change the induced CSR to the reference earthquake magnitude of 7.5, the magnitude scaling factor, or MSF, is employed. K_σ represents the correction factor for effective overburden. Stress reduction coefficient (r_d) is calculated analytically and is a function of depth (z) and earthquake magnitude (M). Given below is the equation for r_d which is valid to a depth up to $z \leq 34$ m, where z is depth in meters and M is earthquake magnitude.

$$\ln(r_d) = \alpha(z) + \beta(z)M \quad (15)$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \quad (16)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \quad (17)$$

Whereas the following equation is valid for $z > 34$ m;

$$r_d = 0.12 \exp(0.22M) \quad (18)$$

Equations used to ascertain CRR , and the resultant FOS from the corrected blow count ($N_1)_{60}$ have been provided below:

$$CRR_{7.5} = 0.8 * \frac{S_u}{\sigma'_{vc}} * K_\alpha \quad (19)$$

Where, the term S_u is undrained Shear Strength. In this paper, Liquefaction analysis of soil is done using IS 1893:2016 [6] for Sand Like soil (Plasticity Index<7) and Idriss and Boulanger 2004 [8] for Clay like soil (Plasticity Index ≥ 7).

3. CASE STUDY

According to IS 1893:2016 [6], as given in Figure 1, the Kutch region of Gujarat is classified as a Zone V (high seismicity zone), making it vulnerable to earthquakes. This area has seen two significant earthquakes in recorded history: the 1819 Kutch earthquake and the 2001 Bhuj earthquake. With an epicentre at 23.4 N and 70.43 E with a focus of 18 km, the Bhuj earthquake had a magnitude of $M_w = 7.7$. The Bhuj earthquake had an impact on the Great Rann of Kutch, but there was only modest liquefaction at the location. A project located at the Great Rann of Kutch was selected for this investigation. The project involved the construction of Substation. Geotechnical Investigations conducted at the project site show subsurface profile as shown in Table 1 for Test location 1 and Table 2 for Test location 2

Table-1: Subsurface profile for Test location 1

| Depth (m) | Strata | SPT N_{avg} | Liquid Limit (LL) (%) | Plastic Limit (PL) (%) | Plasticity Index (PI) (%) |
|-------------|--------------------------------------------------------|---------------|-----------------------|------------------------|---------------------------|
| 0 - 2.5 | Yellowish Brown Clayey Silt of Intermediate Plasticity | 6 | 41.8 | 26.1 | 15.7 |
| 2.5 - 5.5 | Yellowish Brown Silty Clay of Intermediate Plasticity | 8 | 34.1 | 22.3 | 11.8 |
| 5.5 - 13.5 | Greyish Non-Plastic Sandy silt | 13 | 34.7 | NP | NP |
| 13.5 - 18.0 | Greyish Non-Plastic Sandy silt | 18 | 30.1 | NP | NP |

Table-2: Subsurface profile for Test location 2

| Depth (m) | Strata | SPT N_{avg} | Liquid Limit (LL) (%) | Plastic Limit (PL) (%) | Plasticity Index (PI) (%) |
|------------|-------------------------------------------------|---------------|-----------------------|------------------------|---------------------------|
| 0 - 2.5 | Brownish Clayey Silt of Intermediate Plasticity | 4 | 42.9 | 25.5 | 17.4 |
| 2.5 - 5.5 | Brownish Clayey Silt of Intermediate Plasticity | 3 | 39.7 | 25.6 | 14.1 |
| 5.5 - 11.5 | Greyish Non-Plastic Sandy silt | 8 | 35.1 | NP | NP |
| 11.5 - 18 | Greyish Clayey Silt of Intermediate Plasticity | 16 | 33.3 | 20.4 | 12.9 |

According to a prior SPT near the test site, the bedrock was roughly 30 meters deep. This assumption was maintained for the current study. For Test Location 1, the average SPT N values range from 0 to 10 between 1.5 m and 8 m and then increase to a range of 10-20 from 8 m to 16 m, provided in Figure-1. The soil at Test Location 1 consists of Sandy to Clayey Silt with a Plasticity Index greater than 7% up to 8 m, exhibiting clay-like behaviour during liquefaction. Below 8 m, the soil becomes less plastic (Plasticity Index < 7%) up to 18 m, exhibiting sand-like behaviour, which is more prone to liquefaction, as shown in Figure-2. Similarly, for Test Location 2, the average SPT N values are in the order of 0 to 10 from 1.5 m to 11.5 m and increase to 10-20 from 11.5 m to 18 m, given in Figure-1. The soil consists of Sandy to Clayey Silt with a Plasticity Index greater than 7% up to 5.5 m, exhibiting clay-like behaviour during liquefaction. Below 5.5 m, the plasticity decreases up to a depth of 11.5m (Plasticity Index < 7%), and the soil exhibits sand-like behaviour, which is also prone to liquefaction. Following 11.5m, clayey silt of intermediate plasticity having Plasticity index > 7% is encountered. Shear wave velocity (V_s) is also one of the important input parameters for Ground Response Analysis (GRA) studies which indicates the variations of soil stiffness along with depth.

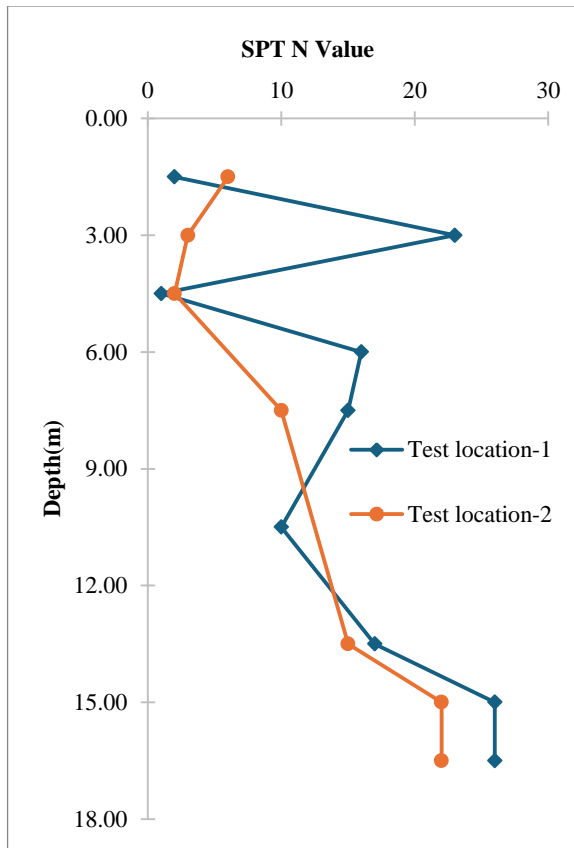


Figure-1: SPT N vs Depth

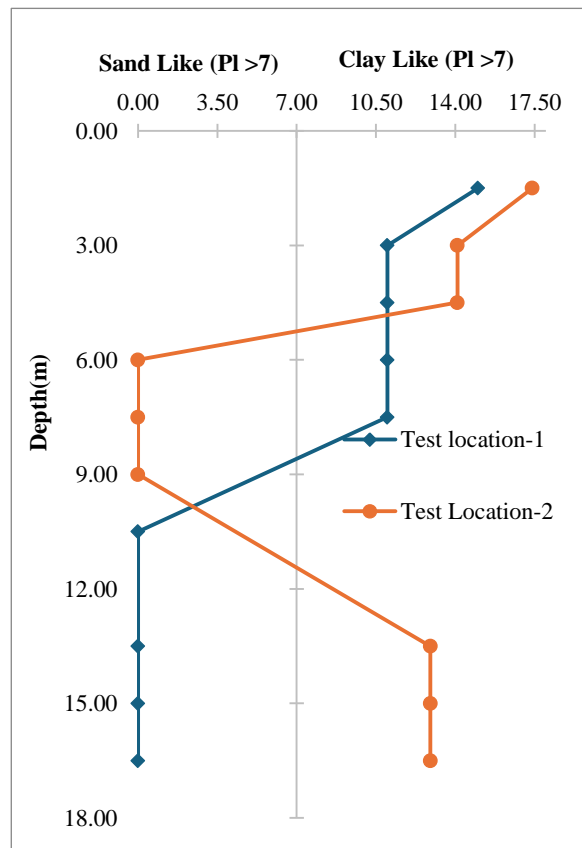


Figure-2: Plasticity Index vs Depth

For simplicity's sake, the epicentre of the Bhuj earthquake is considered a point source. This is approximately 105 km away from the test location. Tests were carried out at locations 1 and 2 down to a depth of around 30 meters. The energy emitted during an earthquake transmits in the form of shear waves, which, depending on the kind of soil strata they pass through, might amplify or de-amplify. When the shear waves reach the ground surface, the bedrock acceleration varies according to the different soil strata. A fundamental input for any liquefaction analysis is the PGA during an earthquake, which is derived from strong motion records that are accessible close to the epicentre. Attenuation relationships are employed to determine the reduced acceleration at the bedrock because the study location is 105 km from the epicentre. For this study, Iyengar and Raghukanth 2010 [9] attenuation relationship (see Eq. 20), is applied.

$$\ln Y = C_1 + C_2(M - 6) + C_3(M - 6)^2 - \ln R - C_4 R + \ln \epsilon \quad (20)$$

where M, Y, R refer to the magnitude, acceleration in g, and hypo-central distance in km, respectively. The values of the various constants C_1 , C_2 , C_3 and C_4 and error ϵ are obtained from Iyengar and Raghukanth 2010 [9] for western central regions of India.

In the present study, multiple correlations of shear wave velocity (V_s) based on SPT-N values were used to calculate the shear wave velocity (V_s) profiles, presented in Table 3. In this investigation, the average value of these correlations was utilized due to the uncertainty in the selection of shear wave velocity (V_s) profiles.

Table-3: Empirical correlations for the analysis of shear wave velocity obtained from the observed SPT-N Value

| S. No. | References | Correlations |
|--------|--------------------------------------|---------------------------|
| 1. | Ohba and Toriumi (1970) [10] | $V_s = 84 * N^{0.31}$ |
| 2. | Ohsaki and Iwasaki (1973) [11] | $V_s = 82 * N^{0.39}$ |
| 3. | Imai and Yoshimura (1972) [12] | $V_s = 91 * N^{0.337}$ |
| 4. | Ohta and Goto (1978) [13] | $V_s = 85.35 * N^{0.348}$ |
| 5. | Seed and Idriss (1981) [14] | $V_s = 61 * N^{0.5}$ |
| 6. | Imai (1982) [15] | $V_s = 97 * N^{0.31}$ |
| 7. | Athanasopoulos (1970) [16] | $V_s = 107.6 * N^{0.36}$ |
| 8. | Lee (1990) [17] | $V_s = 57.4 * N^{0.49}$ |
| 9. | Iyisan (1996) [18] | $V_s = 51.5 * N^{0.516}$ |
| 10. | Yokota et al. (1981) [19] | $V_s = 121 * N^{0.27}$ |
| 11. | Mhaske and Choudhury (2010) [20] | $V_s = 72 * N^{0.4}$ |
| 12. | Hanumantharao and Ramana (2008) [21] | $V_s = 82.6 * N^{0.43}$ |
| 13. | Kalteziotis et al. (1992) [22] | $V_s = 76.2 * N^{0.24}$ |
| 14. | Jafari et al. (2002) [23] | $V_s = 22 * N^{0.85}$ |
| 15. | Dikmen (2009) [24] | $V_s = 58 * N^{0.39}$ |
| 16. | Hasancebi and Ulusay (2007) [25] | $V_s = 90 * N^{0.309}$ |
| 17. | Maheswari et al. (2010) [26] | $V_s = 95.64 * N^{0.301}$ |

4. RESULTS AND DISCUSSION

The ground response analysis (GRA) record of acceleration vs. time is scaled using the value of bedrock acceleration found using Eq. (20), and a 1-D linear GRA of soil is performed using the software program DEEPSOIL v7.1 to determine ground acceleration. The acceleration at bedrock was obtained as 0.18g.

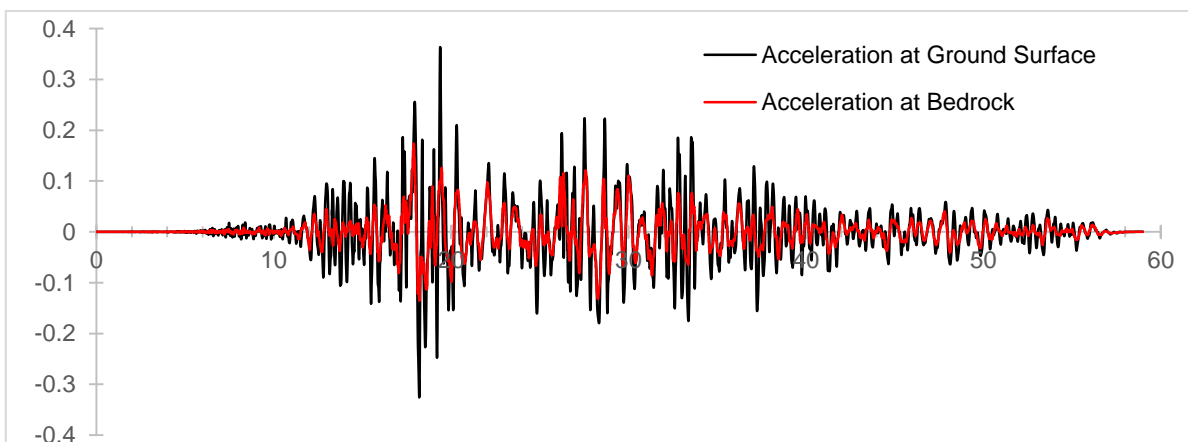


Figure-3: Acceleration time history at bedrock and at ground surface analyzed using DEEPSOIL.

The lowest soil layer in each test was assumed to extend to a depth of 29.5 m since the SPTs were not conducted up to the bedrock. The water table is at ground surface. The results from the GRA are depicted in Figure. 3, and it is inferred that PGA is 0.36g which conforms to the IS 1893:2016 [6] recommendation, i.e., 0.36g for Seismic Zone V. The peak value derived at ground surface was incorporated to calculate the CSR and Liquefaction analysis was done based on the obtained PGA.

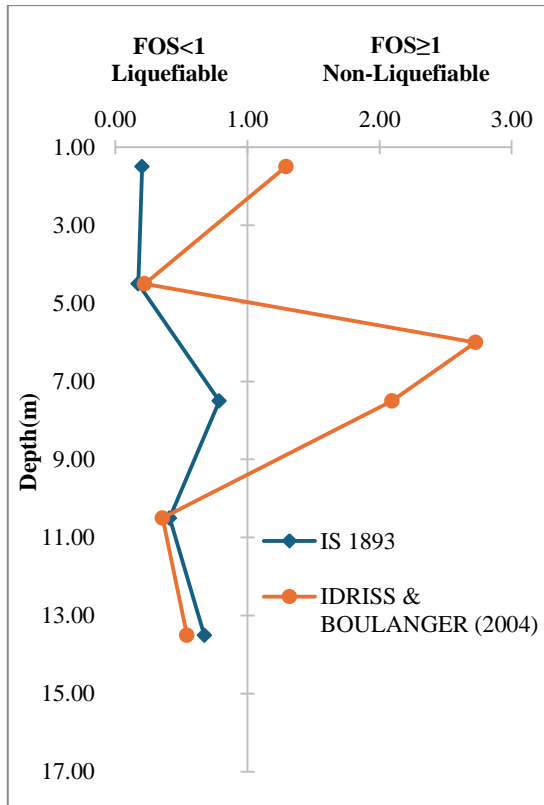


Figure-4: FOS vs Depth for Test Location1

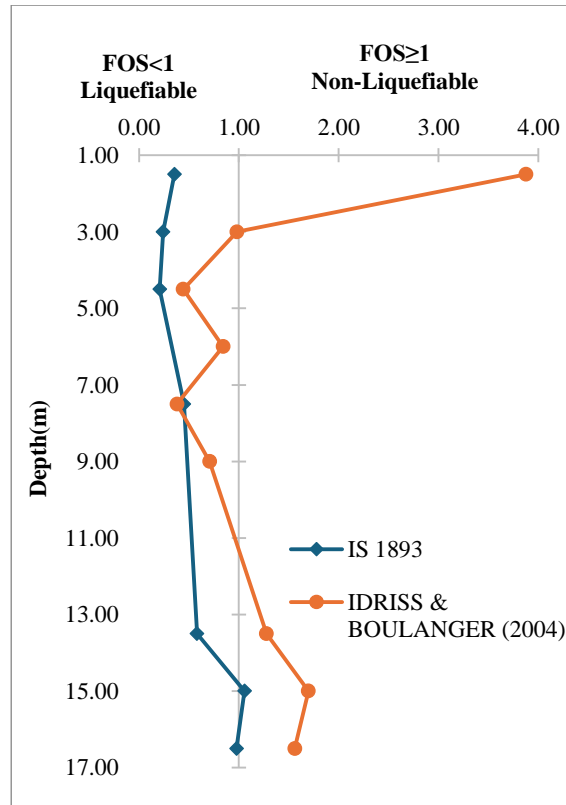


Figure-5: FOS vs Depth for Test Location2

Liquefaction does not occur predominantly for soils having Plasticity greater than 7. In this study liquefaction analysis was carried out from a depth of 1.5 m, which is the ground water level. As the liquefaction analysis method provided in IS 1893:2016 [6] is not applicable to fine grained plastic soils, such soil layers are evaluated using Idriss and Boulanger (2004) [8] for Soils having Plasticity Index greater than 7. This study aims for a comparative analysis of Factor of Safety against liquefaction calculated using both the methods for Plastic soils having Plasticity Index greater than 7. PGA of 0.36g evaluated from GRA was applied to ascertain the CSR. Figures 4 and 5 show the variation in FOS with depth for SPT N value and Undrained Shear Strength at the two test locations considered in this study. It can be observed that there is a difference in the FOS between 1.5 m to 9 m for test location 1, 1.5 m to 4.5 m and 9 m to 18 m for test location 2 by the two methods. This difference occurs as the strata in these depths are classified as clay like ($PI > 7$). As the Plasticity increases, the tendency of soil to liquefy decreases.

5. CONCLUDING REMARKS

The objective of this work was to compare liquefaction potential of plastic soils by different simplified approaches for moderately liquefied locations in Gujarat. Using the test results, factors of safety against liquefaction were established from the methods given in IS 1893:2016 [6] and Idriss and Boulanger (2004) [8] at all the test locations. The comparison of both the approaches was established graphically, showing the variation of the FOS against different levels of Plasticity Index. The major conclusions drawn from this study are summarized below:

- The liquefaction assessment at the location, based on damage indices calculated using both methods, revealed a significant discrepancy in the predicted liquefaction potential of plastic soils.

- The FOS against liquefaction for plastic soils was found to be higher using the Idriss and Boulanger (2004) [8] equations compared to the values obtained based on IS 1893:2016 [6] in both tests.
- The Idriss and Boulanger (2004) [8] method provides a more accurate assessment of liquefaction potential for plastic soils than IS 1893:2016 [6], which is primarily designed to evaluate liquefaction risk in sandy soils.
- Since SPT measures various soil stress components, conducting Consistency tests and Shear Strength tests at the location can offer a more comprehensive understanding of soil properties, enabling the selection of appropriate ground improvement techniques.

REFERENCES

- [1] Seed, Harry Bolton. "Ground motions and soil liquefaction during earthquakes." Earthquake engineering research insititue (1982).
- [2] Wang, Wenshao. Some findings in soil liquefaction. Earthquake Engineering Department, Water Conservancy and Hydroelectric Power Scientific Research Institute, 1979.
- [3] Bray, Jonathan D., and Rodolfo B. Sancio. "Assessment of the liquefaction susceptibility of fine-grained soils." *Journal of geotechnical and geoenvironmental engineering* 132.9 (2006): 1165-1177.
- [4] Boulanger, Ross W., and I. M. Idriss. "Liquefaction susceptibility criteria for silts and clays." *Journal of geotechnical and geoenvironmental engineering* 132.11 (2006): 1413-1426.
- [5] Gratchev, Ivan B., et al. "The liquefaction of clayey soils under cyclic loading." *Engineering geology* 86.1 (2006): 70-84.
- [6] 1893 (Part 1): 2016 "CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES".
- [7] Seed, H. Bolton, and Izzat M. Idriss. "Simplified procedure for evaluating soil liquefaction potential." *Journal of the Soil Mechanics and Foundations division* 97.9 (1971): 1249-1273.
- [8] Boulanger, Ross W., and Izzat M. Idriss. "Evaluating the potential for liquefaction or cyclic failure of silts and clays." (2004).
- [9] Raghukanth, S. T. G. "Estimation of seismicity parameters for India." *Seismological Research Letters* 81.2 (2010): 207-217.
- [10] Ohba, S., and I. Toriumi. "Dynamic response characteristics of Osaka Plain." *Proceedings of the annual meeting AIJ (in Japanese)*. Vol. 12. 1970.
- [11] Ohsaki, Yorihiro, and Ryoji Iwasaki. "On dynamic shear moduli and Poisson's ratios of soil deposits." *Soils and Foundations* 13.4 (1973): 61-73.
- [12] Imai, Yusuke, Hisato YOSHIMURA, and Hitoshi TAKEDA. "Water permeability and salt reabsorption in the duct system of the submaxillary gland of dogs." *The Japanese journal of physiology* 22.3 (1972): 271-280.
- [13] Ohta, Yutaka, and Noritoshi Goto. "Empirical shear wave velocity equations in terms of characteristic soil indexes." *Earthquake engineering & structural dynamics* 6.2 (1978): 167-187.
- [14] Seed, H. Bolton, and I. M. Idriss. "Evaluation of liquefaction potential sand deposits based on observation of performance in previous earthquakes." *ASCE national convention (MO)*. 1981.
- [15] Shinozuka, Masanobu, Chung-Bang Yun, and Hiroyuki Imai. "Identification of linear structural dynamic systems." *Journal of the Engineering Mechanics Division* 108.6 (1982): 1371-1390.
- [16] Athanasopoulos, G. A. "Utilization of sample disturbance for dating a marl deposit." *Geotechnical & Geological Engineering* 13 (1995): 93-104.
- [17] Lee, Shannon Hsien-Heng. "Regression models of shear wave velocities in Taipei basin." *Journal of the Chinese Institute of Engineers* 13.5 (1990): 519-532.
- [18] Iyisan, Recep. "Correlations between shear wave velocity and in-situ penetration test results." *Teknik Dergisi Insaat Muhendisleri Odasi* 7 (1996): 371-374.
- [19] Nakanishi, Ichiro, Kiyoshi Suyehiro, and Takashi Yokota. "Regional variations of amplitudes of ScSp phases observed in the Japanese Islands." *Geophysical Journal International* 67.3 (1981): 615-634.
- [20] Mhaske, Sumedh Yamaji, and Deepankar Choudhury. "GIS-based soil liquefaction susceptibility map of Mumbai city for earthquake events." *Journal of Applied Geophysics* 70.3 (2010): 216-225.
- [21] Hanumantharao, C., and G. V. Ramana. "Dynamic soil properties for microzonation of Delhi, India." *Journal of earth system science* 117 (2008): 719-730.
- [22] Kalteziotis, N., N. Sabatakakis, and J. Vassiliou. "Evaluation of dynamic characteristics of Greek soil formations." *Second hellenic conference on geotechnical engineering*. Vol. 2. 1992.
- [23] Jafari, Mohammad Kazem, Ali Shafiee, and Arash Razmkhah. "Dynamic properties of fine-grained soils in south of Tehran." *Journal of Seismology and Earthquake Engineering* 4.1 (2002): 25-35.

- [24] Dikmen, Ünal. "Statistical correlations of shear wave velocity and penetration resistance for soils." *Journal of Geophysics and Engineering* 6.1 (2009): 61-72.
- [25] Hasancebi, Nilsun, and Resat Ulusay. "Empirical correlations between shear wave velocity and penetration resistance for ground shaking assessments." *Bulletin of Engineering Geology and the Environment* 66 (2007): 203-213.
- [26] Uma Maheswari, R., A. Boominathan, and G. R. Dodagoudar. "Use of surface waves in statistical correlations of shear wave velocity and penetration resistance of Chennai soils." *Geotechnical and Geological Engineering* 28 (2010): 119-137.