

# Soil Liquefaction Evaluation using Deterministic and Probabilistic Approaches: A case study

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**Abstract-**A study has been undertaken to provide a better understanding of the liquefaction potential evaluation. This paper discusses the evaluation of seismically induced Liquefaction based on semi-empirical field-based procedures using the Standard Penetration Test (SPT) profile of soil. This work has been carried out to initiate such studies in the region and to address the problem of seismically induced liquefaction based on deterministic as well as probabilistic approach. A case study for IIT Guwahati with 3 different borehole locations has been carried out for 3 different Earthquake Moment Magnitude of  $M=5, 6$  and  $7$  respectively. Liquefaction would be suspected to occur upto a depth of  $15-18$  m. The results obtained in probabilistic approach are in good agreement with deterministic approach. This Study provides a good understanding to account for the liquefaction assessment based on two approaches.

**Keywords:** Earthquake, Liquefaction assessment, SPT.

## I. INTRODUCTION

Guwahati is situated along the bank of river Brahmaputra in the North Eastern (NE) region of India. The Indian Standard code of practice (IS 1893) [8] identified north-east India, including Assam as a highly seismic zone by placing it in the highest seismic zonal level i.e. zone V. The northeastern part of India has an exceptionally complex tectonic and geologic setup. It has the Himalayan mountain belt in the north, Mishmi hills in the west, Naga Patkoi mountain range in the south, and the Brahmaputra plain at the middle, along with the Shillong Plateau, the Burmese arc, the Tripura folded

belt, and the Surma Valley, that makes this region among most seismically active regions of the world [14]. Since 1897, this region has experienced two great earthquakes of magnitude above 8 and about 14 large earthquakes of magnitudes varying between 8 and 7. The 1869 Cachar earthquake ( $M_w 7.6$ ) and 1987 great Assam earthquake ( $M_w 8.1$ ) caused severe damage in and around Guwahati city through liquefaction and settlement [13,14]. A massive stretch of natural water bodies have been occupied for construction of buildings, which are highly susceptible to local site effects such as soil amplification and liquefaction during earthquakes. Studies have been reported on seismicity of Guwahati city [15].

Liquefaction is the phenomena in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Loss of strength in saturated and cohesion-less soils occurs under undrained conditions, during dynamic loading, because of increased pore water pressures and hence significant reduction in effective stresses [22]. Soil liquefaction induced by an earthquake can be a major cause of damage to structures and facilities. Damage to structure in liquefied sediments can occur as a result of bearing capacity failure of a foundation, lateral spreading or slope failure and differential settlement. Evaluations of soil liquefaction potential by empirical methods and semi-empirical methods have become popular among practising engineers. These methods use deterministic relations implying the occurrence or triggering of liquefaction. A semi-empirical method originally developed by Seed and Idriss [16,17] was based on the evaluation of soil liquefaction resistance based on the results of the standard penetration test (SPT) values. Over the past decades, these methods have been modified successively [18,22] and have become more attractive

and a standard of practice for many engineers around the world. Recently Idriss and Boulanger [4] have proposed an updated and improved method for the evaluation of liquefaction potential of soil deposits, which has been used in the present analysis.

Although Deterministic analysis is well accepted method to evaluate the soil liquefaction, there are uncertainties left with traditional deterministic approach where factors of safety are often difficult to interpret. To overcome these uncertainties, probabilistic and statistical approaches have been employed in the literature[12,20,21]. There have been several notable studies done so far since last three decades. In a probabilistic analysis, the liquefaction potential is usually expressed as a function of liquefaction probability using various approaches. Christian and Swiger[1] analysed the available SPT data from field observation of soil performance and presented the probability of liquefaction. Toprak[20] suggested SPT based liquefaction triggering resistance curves corresponds to a specific percentage of probability of liquefaction. Halder and Tang [3] used the SPT results and carried out a second moment statistical analysis to examine the limit state and empirical procedure introduced by Seed and Idriss [16]. Iwasaki et al. [11]proposed a procedure to evaluate liquefaction potential index, which was based on the in situ test data over the entire borehole.

Till now, there have been limited studies done so far on probabilistic approach of liquefaction for Guwahati city. An attempt has been made to address the problem of assessment of seismically induced Liquefaction based on deterministic as well as probabilistic approach. Study area is IIT Guwahati campus where new construction activities are going on. In the initial stage of this study, the analysis has been carried out at three different locations inside the IIT Guwahati campus to get the proper understanding on seismically induced liquefaction. The analysis has been carried out for earthquake moment magnitudes of M=5, 6 and 7 respectively.

## II. METHODOLOGY

Seismically induced Liquefaction potential isevaluated using the simplified procedure based on the available SPT profile of soil (Idriss and Boulanger [4]). Based on the suggested method, the liquefaction potential can be estimated simply by using the fundamental properties of soils, i.e. SPT-N value, unit weight, mean particle diameter, and the maximum acceleration at ground surface.

### 2.1 Deterministic approach

Initial step in the evaluation of liquefaction assessment includes determination ofearthquake induced loading expressed in terms of cyclic stress ratio (CSR). This loading is compared with the liquefaction resistance of the soil, which is expressed in terms of cyclic resistance ratio (CRR). The Factor of safety (FOS) is evaluated using CSR and CRR as given below.Liquefaction is expected to occur when FOS is less than 1 i.e. when the loading exceeds the resistance.

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

#### 2.1.1. Cyclic stress Ratio (CSR)

In the deterministic approach, Cyclic Stress Ratio(CSR) induced by earthquake ground motions, at a given depth z below the ground surface is usually calculated using Seed and Idriss [16] relation

$$CSR = 0.65 \left( \frac{\sigma_{v0} a_{max}}{\sigma'_{v0}} \right) r_d \quad (2)$$

where  $a_{max}$  is the peak horizontal acceleration on the ground surface,  $\sigma_{v0}$  is the total vertical stress in soil,  $\sigma'_{v0}$  is the effective vertical stress at depth z and  $r_d$  is the stress reduction factor that measures the attenuation of peak shear stress with depth due to the non-elastic behaviour of soil. All the boreholes considered in the present work being less than 34 m in depth, the stress reduction coefficient ( $r_d$ ) is calculated [2, 5] using the following equations:

$$\ln(r_d) = \alpha(z) + \beta(z)M \quad (3)$$

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$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \quad (4)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.73} + 5.142\right) \quad (5)$$

It has been customary to adjust the values of CSR that pertain to the equivalent uniform shear stress induced by an earthquake having a moment magnitude M through the magnitude scaling factor (MSF), so that the adjusted values of CSR would pertain to the equivalent uniform shear stress induced by an earthquake having a moment magnitude M = 7.5, i.e.  $CSR_{M=7.5}$  given by the following equation.

$$CSR_{M=7.5} = \frac{CSR_M}{MSF} = 0.65 \left( \frac{\sigma_{v2}^{\Delta_{max}}}{\sigma_{v2}} \right) \frac{P_a}{MSF} \quad (6)$$

Idriss [5] proposed the MSF using the given equation as

$$MSF = 6.9 \exp\left(\frac{-M}{4}\right) - 0.058 \quad (7)$$

Since the liquefaction resistance increases with increasing confining stress, a correction factor  $K_\sigma$  is applied such that the values of CSR correspond to an equivalent overburden pressure  $\sigma_{v2}^i$  of 1 atmosphere.

$$CSR_{M=7.5, \alpha=1} = 0.65 \left( \frac{\sigma_{v2}^{\Delta_{max}}}{\sigma_{v2}} \right) \frac{P_a}{MSF K_\sigma} \quad (8)$$

where

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_{v2}^i}{P_a}\right) \leq 1 \quad (9)$$

$$C_\sigma = \frac{1}{18.9 - 2.22 \sqrt{(N_1)_{60}}} \leq 0.3 \quad (10)$$

$P_a$  is the atmospheric pressure = 101.3kpa.  $(N_1)_{60}$  is the corrected SPT-N value, limited to a maximum value of 37.  $N_{60}$  value, corrected for the field test procedures and to an average energy ratio of 60% of the SPT hammer,  $C_N$  factor to normalize N to a common reference effective over burden stress of 1 atmospheric pressure.

$$(N_1)_{60} = C_N(N)_{60} \quad (11)$$

$$C_N = \left( \frac{P_a}{\sigma_{v2}^i} \right)^\alpha \quad \text{where } \alpha = 0.784 - .0768 \sqrt{(N_1)_{60}} \quad (12)$$

## 2.1.2. Cyclic resistance ratio (CRR)

Idriss and Boulanger [4] recommended the following expression to be used for determining the CRR, for cohesion less soil with corrected SPT N value for clean sands ( $(N_1)_{60cs}$ ).

$$CRR = \exp \left\{ \frac{(N_1)_{60cs}}{24.4} + \frac{(N_1)_{60cs}^2}{100} - \frac{(N_1)_{60cs}^3}{23.6} + \frac{(N_1)_{60cs}^4}{23.4} - 2.8 \right\} \quad (13)$$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \quad (14)$$

$\Delta(N_1)_{60}$  is the correction for fines content in percent (FC) present in the soil and is given as

$$\Delta(N_1)_{60} = \exp \left( 1.63 + \frac{9.7}{FC+1} - \frac{(15.7)}{FC+0.1} \right)^2 \quad (15)$$

## 2.2 Probabilistic approach

To know the risk of liquefaction, it is important to know not only if liquefaction is possible but also the probability of its occurrence during the lifetime of the facility. Sometimes, FOS has a weak physical interpretation and the selection of a convincing value for 'FOS' is somewhat difficult during the deterministic approach. Instead, in a probability analysis, the uncertainties in stress and strength characteristics are effectively incorporated into calculations. Idriss and Boulanger [4] proposed a single, composite relationship for the probability assessment of liquefaction which is a function of SPT N value, CSR, moment magnitude (M), Fine content (FC) and vertical effective stress as mentioned below

$$F_L(N_1, CSR, M_w, \sigma_v^i, FC)$$

$$= \Phi \left( \frac{N_{1,60} \cdot (1 + 0.004 \cdot FC) - 13.32 \cdot \ln(CSR) - 29.53 \cdot \ln(M_v) - 3.70 \cdot \ln\left(\frac{E_v}{P_a}\right) + 0.005 \cdot FC + 16.25}{2.70} \right) \quad (16)$$

where  $F_L$  = probability of liquefaction; CSR is not “adjusted” for magnitude or duration effects, FC = percent fines content in integer and  $\Phi$  = standard cumulative normal distribution.

PL (in %)	Liquefaction severity class
0-15	Very low
15-35	Low
35-65	Moderate
65-85	High
85-100	Very high

Table 1. Probabilistic liquefaction severity classification suggested by [19]

Probabilistic approach is used to assess the probability of initiation of liquefaction. Also, different probability values indicate different scale of severity of liquefaction initiation suggested by [19] is given in Table 1.

### III. STUDY AREA and TEST DATA

IIT Guwahati is situated along the bank of river Brahmaputra in the North Eastern (NE) region of India. The greater Guwahati region encompasses an area of about 600 km<sup>2</sup> (latitude 26.1838° N, longitude 91.7633° E). The overall topography of the greater Guwahati region consists of several continuous hill tracts varying in altitude from 200 to 400 m above mean sea level (MSL) and isolated hill locks up to 300 m above MSL [15]. The study area and borehole locations are shown in Figure 1. Three Boreholes at three different locations marked in circle are shown in figure 1. BH1 and BH3 were drilled by engineering firms during the construction of Student Activity Centre (SAC) while BH2 is located near Faculty quarters within the boundary

area of IIT Guwahati. Approximate distance between BH1 and BH3 according to engineering firm report is 40 meters.

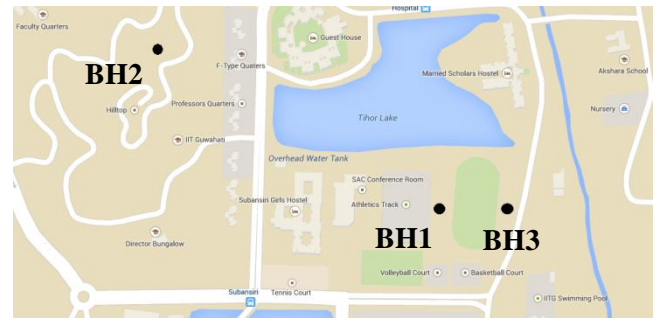


Figure 1. Study Area

#### 3.1. Standard Penetration Test (SPT) Data

The standard penetration test is a widely used in situ test to evaluate the geotechnical properties of soil. These soil investigations were carried out by engineering firms involved during the construction. The test was performed following Indian Standard code of practise [9]. A split spoon sampler with external and internal diameter of 50 and 35 mm, respectively, and 650 mm long is driven into the soil under the impact of a 63.5-kg hammer dropped from a height of 0.76 m. The sampler is driven to penetrate to a depth of 450 mm. The number of blows required by the sampler to penetrate the last 300 mm of depth is called the SPT-N value. For this study, boreholes BH1 and BH3 were drilled upto a depth of 25.5 m and BH2 was drilled upto a depth of 21 m, in order to get the knowledge of subsurface stratum and soil type.

Variations of SPT N value with depth in different borehole are shown in Figure 2. It is seen that the soil deposit comprises of sand, clay and silt. The specific gravity of soil was reported as 2.64. Whenever the N-SPT value exceeds 100 for 300 mm penetration, it is treated as refusal (R) and further N-SPT values were not measured for that depth as

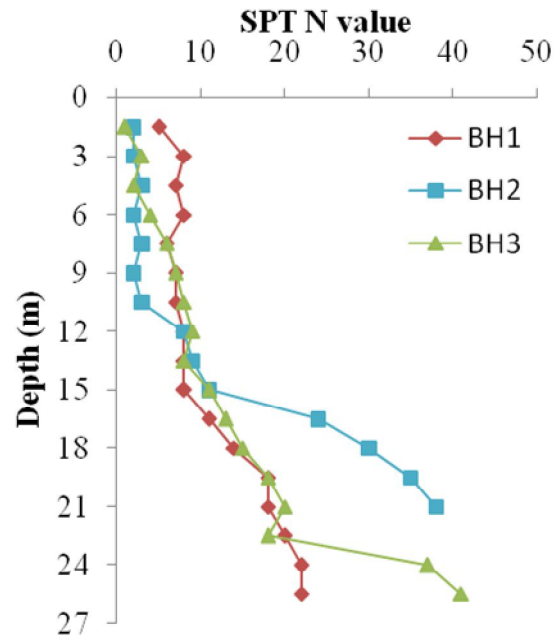


Figure 2. Variation of SPT N value with depth for location 1 with BH1, location 2 with BH 2 and location 3 with BH 3.

per IS 2131 (1981) [9]. All boreholes were drilled with diameter of 150 mm as per IS: 1892 (1974) [7] and N-SPT values were measured regularly at 1.5 m intervals. The SPT-N values of the soil strata of the investigating area varies from 1 to 38 indicating the degree of consistency is very soft to hard. Disturbed and undisturbed samples were collected at possible depths as per IS: 2132 (1986) [10]. The physical properties were determined in the laboratory using disturbed soil samples as per IS: 1498 (1970) [6].

#### IV. RESULTS

Liquefaction potential over the study area was evaluated using the simplified procedure based on the available SPT profile of soil (Idriss and Boulanger [4]). The analysis involved two approaches, Deterministic and Probabilistic respectively. The analysis has been done for three earthquake magnitudes (Moment magnitude  $M=5$ , 6 and 7). For all the three earthquakes magnitudes, surface Peak Ground Acceleration (PGA) is chosen as 0.18g in the study.

Figure 3 shows the results of liquefaction assessment at location 1 (BH1) for earthquake magnitude of  $M=5$ , 6 and 7 respectively. It can be observed that Factor of safety (FOS) is always greater than 1 for  $M=5$ . It implies that location 1 is not at all susceptible to liquefaction. Probabilistic model confirms this result with zero liquefaction probability (PL) within the depth range of 1.5-6m and 16.5-25.5m, although very low insignificant PL can be seen within the depth range of 7.5-16.5m indicating very low scale of severity of liquefaction initiation. For  $M=6$ , FOS is greater than 1 for depth ranges 0-6m and 15-25.5m and therefore, not vulnerable to liquefaction, where as, it is in close proximity to the value 1 between 7.5 to 15m. Also, probabilistic model gives high scale of severity of liquefaction initiation with 90% PL value within 7.5-15m depth. Hence, soil present in between 7.5 to 15m depth is likely to be susceptible by liquefaction. For  $M=7$ , it can be observed that FOS is less than 1 in between 4.5 to 18m depth. Hence, Liquefaction is expected to occur between 4.5 to 16m depth. Also, Probabilistic model give maximum PL value

of 100% within this depth indicating very high scale of severity of liquefaction initiation. Hence, the results obtained from deterministic as well as probabilistic approaches are in good agreement.

Figure 4 shows the results of liquefaction assessment at location 2 (BH2) for earthquake magnitude of M=5, 6 and 7 respectively. For M=5, FOS values are close to one upto 10.5m depth and it increases drastically beyond 10.5 m. Similarly, PL increases with depth showing moderate to high scale of severity with maximum value of 88% at 10m depth. Hence, location 2 is more vulnerable to liquefaction affects compared to location 1 for same earthquake magnitude M=5 but only upto a

depth of 10.5m beyond which FOS values goes on higher side as well as PL values drops down to zero. For M=6, FOS values are less than 1 upto 11m depth and increases drastically beyond 11m. Probabilistic model confirms this result with very high scale of severity i.e. 100% PL upto depth of 11m. For M=7, FOS values are less than 1 upto a depth of 12 m and increases drastically beyond 12m. Probabilistic model confirms this result with very high scale of severity i.e. 100% PL upto 12 m depth and it drops to zero beyond 12m. Therefore, location 2 is more vulnerable to liquefaction affects compared to location 1 for same earthquake magnitudes. Although, more devastated affects can be seen at M=7.

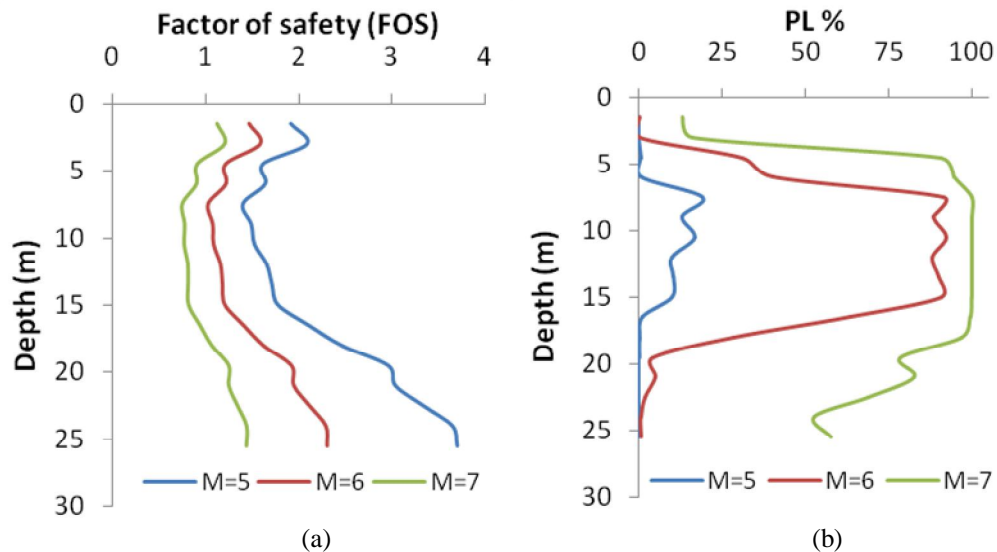


Figure 3. Liquefaction assessment at location 1 for M=5,6,7 using (a) Deterministic model, (b) Probabilistic model ; PL= Probability liquefaction in percentage.

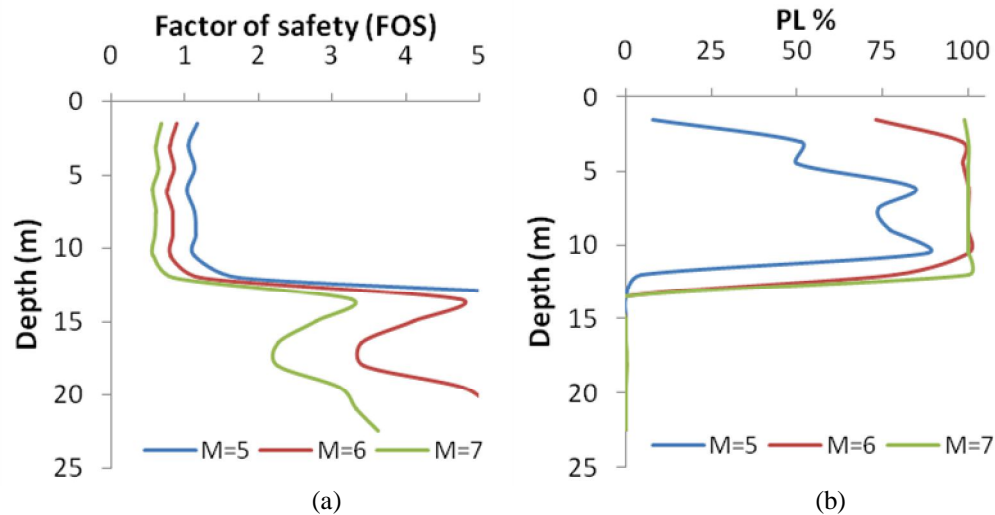


Figure 4. Liquefaction assessment at location 2 for M=5,6,7 using (a) Deterministic model, (b) Probabilistic model

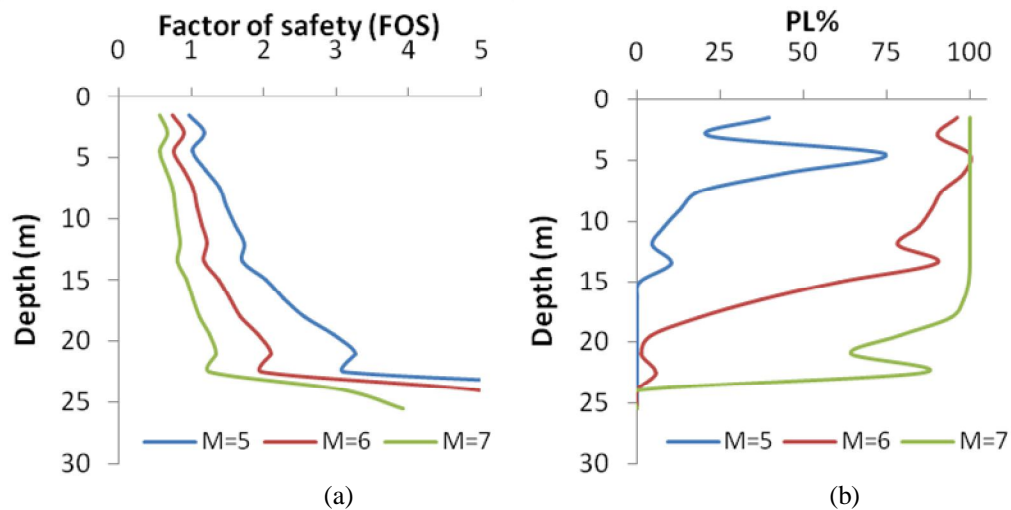


Figure 5. Liquefaction assessment at location 3 for M=5,6,7 using (a) Deterministic model, (b) Probabilistic model

Figure 5 shows the results of liquefaction assessment at location 3 for earthquake magnitude of 5, 6 and 7 respectively. For M=5, FOS values are close to one upto depth of 4.5m and beyond 4.5 m, it increases with depth. PL value indicates moderate scale of severity of liquefaction initiation. Hence, it is more vulnerable to liquefaction affects compared to location 1 but only upto

a depth of 4.5m beyond which FOS values goes on higher side as well as PL values drops down to zero. Similarly for M=6, FOS values are less than 1 upto a depth of 9m and hence highly susceptible to liquefaction. Probabilistic model confirms this result, having PL values of high severity i.e. 80-90% upto 9m depth. It can be concluded that location 2 is more susceptible to

liquefaction affects compared to location 1 and 3 for earthquake magnitude moment  $M=5$  and  $M=6$ . For  $M=7$ . FOS values are less than 1 upto a depth of 16.5m and hence vulnerable to liquefaction. Probabilistic model confirms this result, having maximum PL value of 100% upto 16.5m depth, indication very high scale of severity of liquefaction initiation. Therefore, it can be concluded that location 3 is more vulnerable to liquefaction affects compared to location 1 and 2 for  $M=7$ .

## V. CONCLUSION

This paper discusses the evaluation of seismically induced Liquefaction based on semi-empirical field-based procedure for 3 different earthquake moment magnitudes ( $M=5, 6$  and  $7$ ). The results obtained in probabilistic approach are in good agreement with deterministic approach. IIT Guwahati lies in most seismic severe hazard zone and is highly susceptible to local site effects like liquefaction due to earthquakes in future. Location 2 is highly susceptible for liquefaction effects compared to location 1 and location 3 at lower earthquake moment magnitude of  $M=5$  and  $6$ . At higher earthquake magnitude of  $M=7$ , location 3 is more vulnerable to liquefaction affects compared to location 1 and 2. Damage would likely to occur within the layers up to depths of 15-18 m.

The present analysis highlights the advantage of probabilistic analysis for liquefaction evaluation and also the need to consider various soil profiles in same area.

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