



ANALYSIS OF LIQUEFACTION POTENTIAL OF SOIL BY FIELD TEST DATA A COMPARISION STUDY ON LIQUEFACTION SUSCEPTIBILITY BASED ON DIFFERENT APPROACHES

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ABSTRACT

One of the main concerns for structures built on sandy soils is soil liquefaction. Liquification is the process by which tightly packed, damp granular soil near the ground's surface loses strength from severe ground trembling. Existing structures fall as a result of this phenomenon, taking lives and property with them. Thus, while constructing a foundation, it is essential to comprehend the zone of liquefaction at a particular area. In light of this, a sand-rich location in Kolkata's Sonarpur neighborhood has been chosen for the current study. The goal of the current study is to assess the soil's liquefaction potential using the SPT and CPT methods in Sonarpur, which is close to Kolkata. It has been determined how resistant soils are to liquefaction, measured in terms of the cyclic resistance ratio (CRR). The article highlights the site's distinctive liquefaction zone, which may be taken into consideration when constructing the geotechnical foundation there.

Keywords : Liquefaction Potential , SPT,CPT

1. INTRODUCTION

The process known as liquefaction occurs when saturated sand experiences cyclic or monotonic shear stresses. As a result, a significant portion of its shear resistance is lost, and the sand begins to flow more like a liquid. The process of a granular material changing from a solid to a liquefied state due to a decrease in effective stress and an increase in pore-water pressure is known as liquefaction. The loose to medium-density granular soils that tend to compress when sheared are especially susceptible to this phenomena. Pore-water pressure drainage in saturated soils can be hindered by silty or clayey seam inclusions, or it could not have enough time to happen because of sudden loading, such seismic pressures.

In this case, the pore-water pressure rises as a result of the inclination to compress. As a result, there is a drop in the frictional shear strength and a reduction in effective stress.

Since its initial coining by Mogami and Kubo (1953), the word "liquefaction" has been used to describe a wide range of processes involving soil deformations brought on by recurrent, transitory, or monotonous disturbance of saturated cohesion-less soils under undrained settings. All liquefaction processes are characterized by the production of excess pore pressure under undrained loading conditions. It is generally known that dry cohesion poor soils have a propensity to densify under both static and cyclic loads. However, fast loading occurs under undrained conditions when cohesion-less soils are saturated, therefore the trend toward densification results in

a rise in excess pore pressures and a decrease in effective stresses. Shaking causes liquefaction, a phenomenon that happens to unconsolidated silt that is soaked with water. When such material is present, the earth trembles, causing the grains to separate from one another and the substance to flow.

Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance, when subjected to monotonic, cyclic, or shock 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 (Sladen et al, 1985)".

The one of newest definition has presented with Idriss and Boulanger:

"Loose cohesionless soils tend to contract during cyclic loading, which can transfer normal stress from the soil skeleton to pore water, if the soil is saturated and largely unable to drain during shaking. The result is a reduction in effective confining stress within the soil and an associated loss of strength and stiffness that contributes to deformations of the soil deposit. This loss of strength and stiffness due to increasing pore pressures is called liquefaction (Idriss & Boulanger, 2008)."

Following the principle of effective stress states, effective stress equals to total stress minus the porewater pressure:

$$\sigma' = \sigma - u \quad (1)$$

σ' = effective stress

σ = total stress

u = pore water pressure

If the quantity of σ remains constant, as the pore water pressure u slowly increases, the effective stress σ' gradually decreased. If the pore water pressure builds up to the point at which it is equal to the total stress, the effective stress becomes zero. As the stiffness and strength of a soil depends on the magnitude of effective stress, the soil loses its strength completely when the effective stress becomes zero and the soil is in a liquefaction state (Seed & Lee, 1966).results.

2. OBJECTIVE & SCOPE

The Objectives of the paper are given below:

1. To evaluate liquefaction potential of soil (at Sonarpur near Kolkata) firstly by SPT method
2. To evaluate liquefaction potential of soil (at Sonarpur near Kolkata) firstly by CPT method
3. To find out the depth of liquefaction and minimum factor of safety against liquefaction of the site by two in situ methods (i.e. SPT and CPT)
4. To find out the Liquefaction proneness based on the present study.
5. To highlight the site-specific liquefaction zone which may be considered for geotechnical design of foundation at the site.

The Scope of the paper are given below:

To fulfill the objective following scope has been taken up:-

- a) Firstly, A suitable site has been identified near Sonarpur, Kolkata .
- b) Two boreholes have been dug at the site. The Standard penetration test (SPT) is carried out within the boreholes
- c) Two numbers of CPT tests i.e., CPT1 and CPT2 (aligned to the straight line with the SPT points), were carried out by giving 1000mm spacing between the respective SPT tests points.
- d) cyclic stress ratio (CSR) has been determined as per (Seed and Idriss 1971) or IS 1893 Part-1 2016,
- e) Based on the above collected Standard penetration test (SPT) data, cyclic resistance ratio (CRR) has been determined as per IS 1893 Part-1 2016.
- f) Further, CRR value computed from SPT as well as from CPT test is compared with respect to depth for two borehole locations (i.e SPT-1 and SPT-2) and Factor of safety (FOS) against liquefaction is calculated using the following equations

$$FOS = CRR/CSR$$

3. METHODOLOGY

3.1 Determination of CSR (Seed and Idriss 1971).

The "simplified procedure" calls for the computation of two variables: (1) the cycle stress ratio (CSR), which measures the seismic demand an earthquake places on a soil layer, and (2) the cyclic resistance ratio (CRR), which measures the soil's ability to withstand liquefaction. There may be liquefaction if CSR exceeds CRR. The cyclic stress ratio CSR is calculated by the following equation (Seed & Idriss 1971):

$$\text{CSR} = \tau_{av} / \sigma'_{vo} = 0.65 (a_{max}/g) (\sigma_{vo} / \sigma'_{vo}) r_d \quad (2)$$

Where τ_{av} = average cyclic shear stress, a_{max} = peak horizontal acceleration on the surface of soil caused by earthquake, g = gravitational acceleration, σ_v = vertical overburden stress, σ'_{vo} = effective vertical overburden stress,

r_d = Stress reduction factor.

3.2 Determination of CRR from SPT test data

(Seed and Idris, 1985) proposed CRR_{7.5} equation as mentioned below

$$\text{CRR}_{7.5} = \frac{1}{34 - (N1)_{60CS}} + \frac{(N1)_{60CS}}{135} - \frac{50}{[10 \times (N1)_{60CS} + 45]^2} - \frac{1}{200} \quad (3)$$

Where $(N1)_{60CS}$ is the clean-sand equivalence of the corrected SPT blow count as per (Youd et al., 2001). As per IS 1893 part (1) 2016,

$$(N1)_{60CS} = \alpha + \beta (N1)_{60} \quad (4)$$

$$(N1)_{60} = C_N N_{60} \quad (5)$$

$$\text{Where, } C_N = \sqrt{\frac{P_a}{\sigma'_{vo}}},$$

σ'_{vo} = effective vertical overburden stress and P_a = atmospheric pressure.

The subscript 7.5 in the CRR_{7.5} term indicates that this cyclic liquefaction resistance is evaluated at a magnitude of 7.5. Note that Eq. (1) is valid only for $(N1)_{60CS} < 30$, while the sandy soil is considered unliquefiable when $(N1)_{60CS}$ is greater than 30

(Idriss and Boulanger, 2006) noted that the trend of the CRR curve proposed by (Youd et al., 2001) would sharply increase as the $(N1)_{60CS}$ approaches 30, which may be irrational and would cause the unreasonable results when conducting the probabilistic analysis. They proposed a new model as follows (Idriss and Boulanger, 2006).

$$\text{CRR}_{7.5} = \frac{(N1)_{60CS}}{14.1} + \left(\frac{(N1)_{60CS}}{126} \right)^2 - \left(\frac{(N1)_{60CS}}{23.6} \right)^3 + \left(\frac{(N1)_{60CS}}{25.4} \right)^4 - 2.8 \quad (6)$$

3.3 Determination of CRR from CPT tests data.

The CPT calculations are based on empirical correlations as mentioned below as per ROCSCIENCE INC. (2021) CPT Manual. This manual helps to find some relevant properties to determine the liquefaction potential. The Cone Penetration Test allows for a continuous soil profile and can collect up to 5 independent readings in a single sounding. These readings, notably the cone tip resistance (q_c), sleeve friction (f_s), and penetration pore water pressure (u_2) are interpreted to give the soil parameters used to assess subsurface stratigraphy.

Corrected Cone Resistance, q_t

1. The corrected cone resistance, q_t , is calculated as:

$$q_t = q_c + u_2(1-a) \quad (7)$$

where

a = net area ratio.

In the absence of u_2 , $q_t = q_c$. (8)

2. Friction Ratio, R_f

The friction ratio is defined as the percentage of sleeve friction, f_s , to cone resistance, q_c , at the same depth.

$$R_f = (f_s/q_t) \cdot 100\% \quad (9)$$

3. Soil Unit Weight, γ

The following relationship from Robertson expresses the soil unit weight in terms of the friction ratio and cone resistance (Robertson, 2010).

$$\gamma/\gamma_w = 0.27(\log R_f) + 0.36[\log(q_t/P_a)] + 1.236 \quad (10)$$

where

R_f = friction ratio

γ_w = unit weight of water

P_a = atmospheric pressure

4. Total and Effective Overburden Stress, σ_{v0} and $\sigma_{v0'}$

The total and effective overburden stresses are calculated using the calculated soil unit weight for each depth.

$$\sigma_{v0} = \sum(z_i \cdot \gamma_i) \quad (11)$$

$$\sigma_{v0'} = \sigma_{v0} - u \quad (12)$$

where

γ_i = soil unit weight of the i th layer

z_i = depth of the i th layer from the ground surface

5. Normalized Cone Resistance, Q_t

$$Q_t = (q_t - \sigma_{v0}) / \sigma_{v0'} \quad (13)$$

6. Normalized Friction Ratio, F_r

$$F_r = [(f_s / (q_t - \sigma_{v0})) \cdot 100\%] \quad (14)$$

7. Soil Behaviour Type Index, I_c

The soil behavior type index can be thought of as a representative value that combines Q_t and F_r to produce concentric circles delineating Robertson's 1990 SBT chart zones. I_c expresses the radius of those concentric circles.

$$I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5} \quad (15)$$

The liquefaction potential can then be obtained from IS 1893 part (1) 2016 as follows:

The CPT procedure requires normalizations of measured cone tip resistance q_c using atmospheric pressure P_a and correction for overburden pressure C_Q as follows ,

$$q_{CIN} = C_Q (q_c / P_a) \quad (16)$$

where q_{CIN} is normalized dimensionless cone penetration resistance

$$C_Q = \sqrt{\frac{P_a}{\sigma_{v0}}} \quad (17)$$

The normalized penetration resistance q_{CIN} for silty sand is corrected to an equivalent clean sand value (q_{CIN})_{CS} by the following relation ,

$$(q_{CIN})_{CS} = k_C q_{CIN} \quad (18)$$

Where k_C = correction factor to account for grain characteristics

$$= 1.0 \text{ (for } I_C \leq 1.64) \quad (19)$$

$$-0.403I_C^4 + 5.581I_C^3 - 21.63I_C^2 + 33.75I_C - 17.88 \text{ (for } I_C > 1.64)$$

(20)

The CRR_{7.5} can be found using following equations,

$$\text{CRR}_{7.5} = 0.833((q_{\text{CIN}})_{\text{CS}}/1000) + 0.05, 0 < (q_{\text{CIN}})_{\text{CS}} < 50 \quad (21)$$

$$\text{CRR}_{7.5} = 93((q_{\text{CIN}})_{\text{CS}}/1000)^3 + 0.08, 50 \leq (q_{\text{CIN}})_{\text{CS}} < 160 \quad (22)$$

3.4 Computation of Factor of safety (FOS) against liquefaction

Factor of safety (FOS) against liquefaction is calculated using the following equation

$$\text{FOS} = \text{CRR}/\text{CSR} \quad (23)$$

Liquefaction is said to occur if FOS is less than 1.

3.5 Identification of earthquake magnitude from peer ground motion database and earthquake zone.

As per Indian standard (IS 1893 Part-1 2016), peak ground acceleration is obtained for the study. According to the code Sonarpur is classified into the zone IV having PGA of 0.3g and seismic zone factor (Z) is considered as 0.24



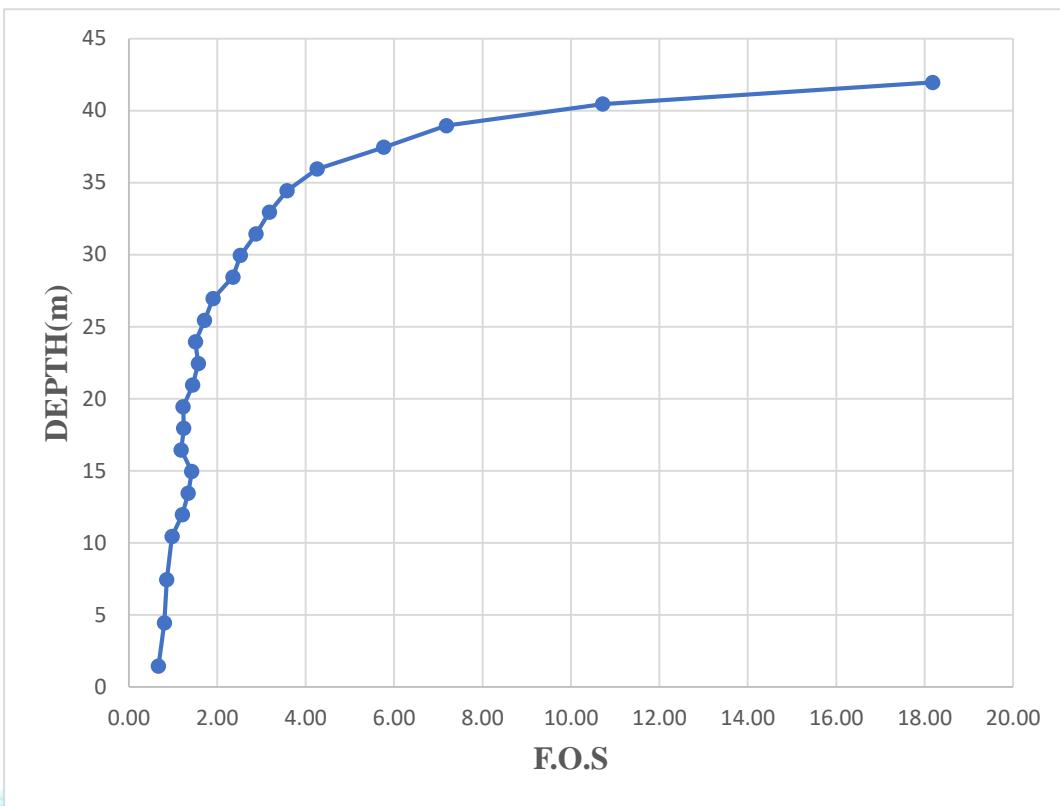
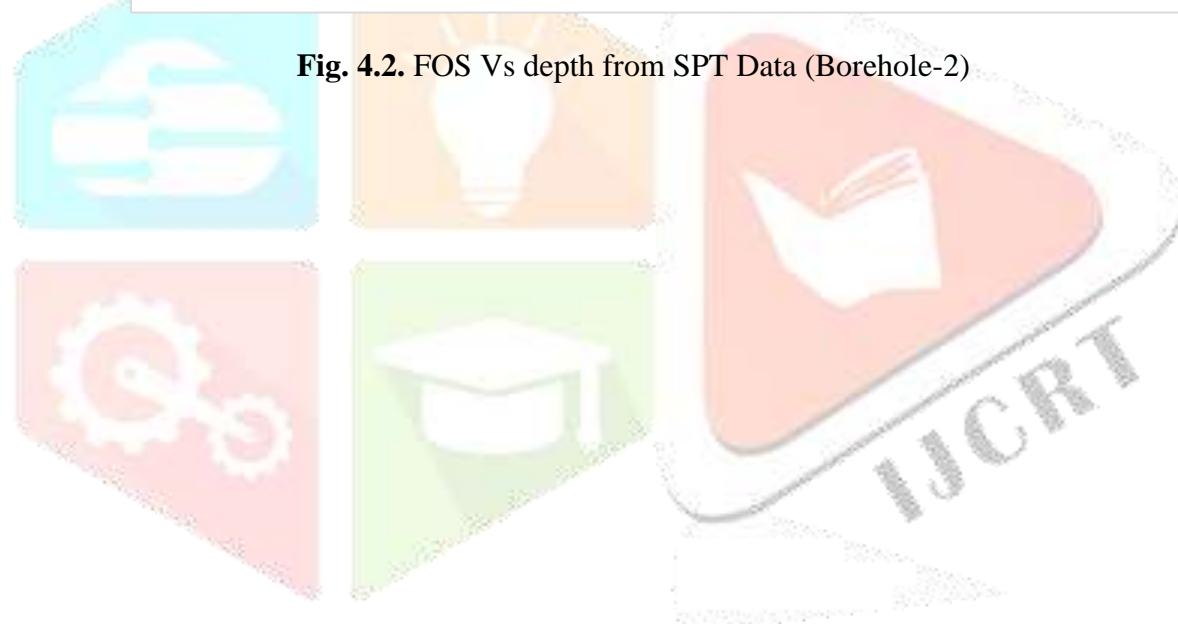


Fig. 4.2. FOS Vs depth from SPT Data (Borehole-2)



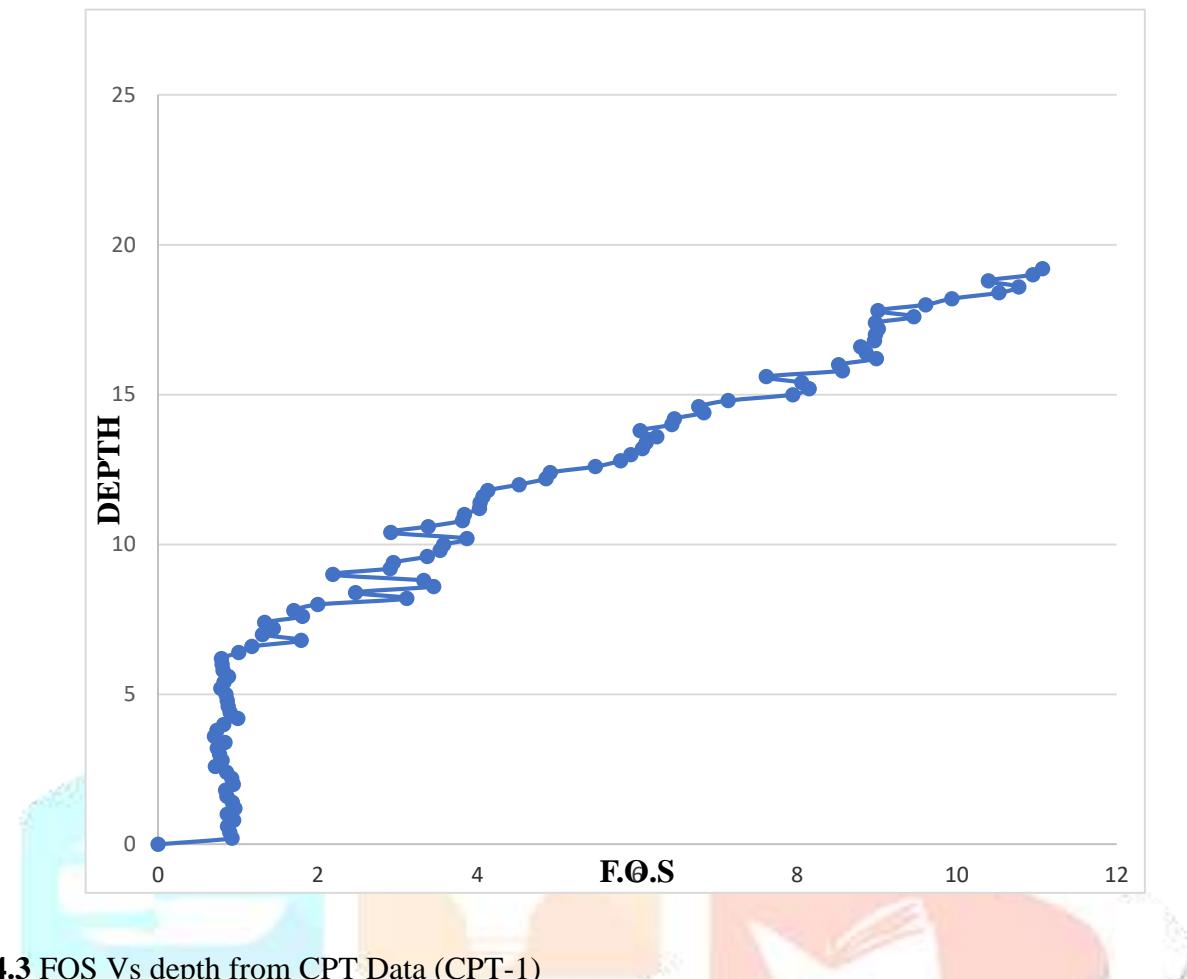


Fig. 4.3 FOS Vs depth from CPT Data (CPT-1)

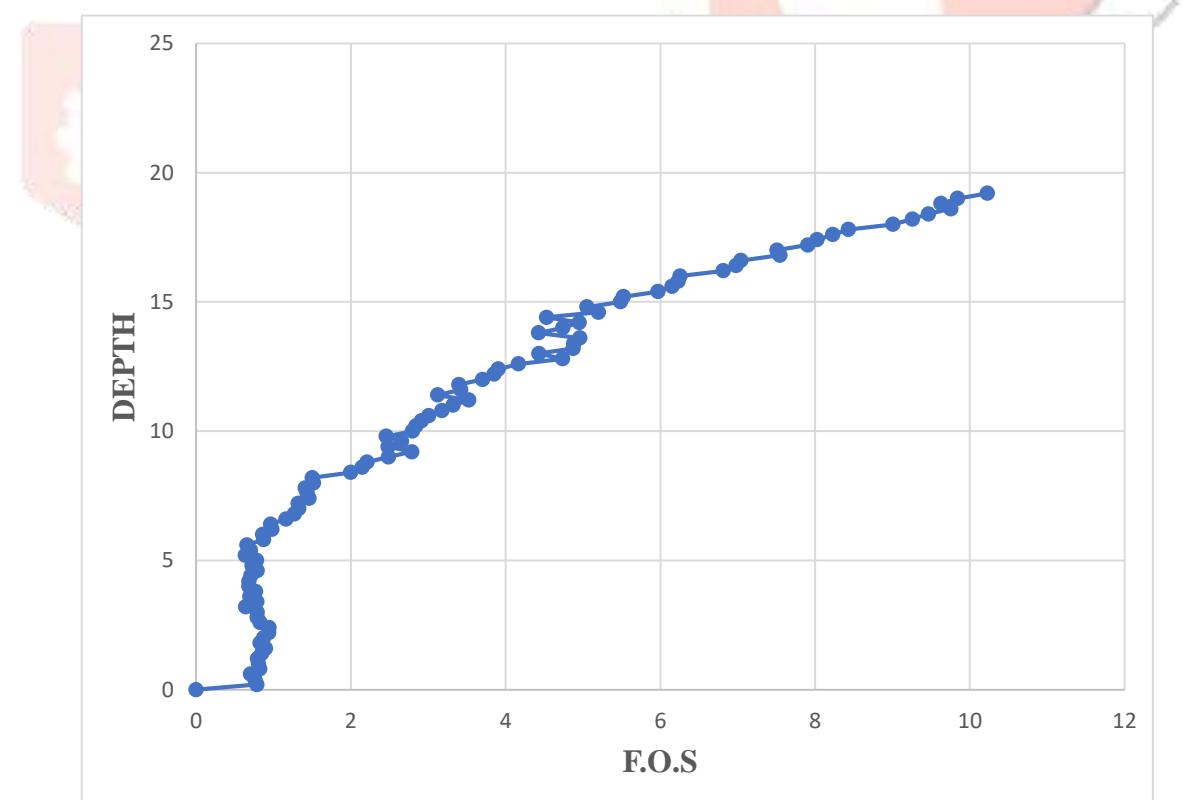


Fig. 4.4. FOS Vs depth from CPT Data (CPT-2)

5. CONCLUSIONS

The following conclusions may be drawn from the present study:

1. SPT results show liquefaction zone up to depth 9.95m below EGL at BH-1 and up to 10.50m below EGL at BH-2. It is further observed from the Tables that the minimum factor of safety of 0.67 occurring at 1.45m below GL in BH-1 and the same is 0.67 at 1.45m below GL in BH-2.
2. CPT results show liquefaction zone up to 6.20m at CPT-1 with a minimum factor of safety of 0.70 at a depth of 3.60m. Further it is seen that in case of CPT-2 the depth of liquefaction is up to 6.40m with a minimum factor of safety of 0.63 at a depth of 3.20m.

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