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Assessment of Liquefaction potential of soil in Ahmedabad Region, Western India

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ABSTRACT

Estimation of liquefaction resistance, also called Cyclic Resistance Ratio (CRR) of soil, is an important aspect of geotechnical earthquake engineering; it is one of the most important secondary effects of earthquake which causes severe damages to engineering structures. The liquefaction potential is estimated in terms of factor of safety (FS). The present study involves evaluation of liquefaction potential of soil in the Ahmedabad city area, which dominantly consists of sandy to silty sediments and is witnessing significant constructional activities in recent times. Standard Penetration Test (SPT) data from 23 boreholes are collected for analysis. The SPT N-values were corrected for the estimation of CRR. Results of the study indicate that the soil in Ahmedabad city area is much compact, water table is much low and SPT- N values are high, which imply that liquefaction potential is much low. However, it is advisable to perform site specific detailed geotechnical investigation in case of high rise structure and or heavy engineering structures.

Key words: Geotechnical, SPT N-value, Cyclic Resistance Ratio (CRR), Cyclic Stress Ratio (CSR), liquefaction susceptibility, Factor of Safety (FS).

INTRODUCTION

Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance when subjected to cyclic loading induced by earthquakes and flows like a liquid (Sladen et al., 1985). The phenomenon of liquefaction of soil has been brought to the attention of researchers after the great Niigata (1964) and Alaska (1964) earthquakes. It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking. Liquefaction and related phenomenon have been responsible for tremendous amounts of damage due to earthquakes around the world (Yanagisawa, 1983; Borcherdt, 1991; Morales et al., 1995). Many structures, foundations and slopes did experience failures due to liquefaction during the earthquakes of Dhubri, Assam (1930), Bihar-Nepal (1934), Niigata (1964), San Fernando (1971), Tangshan (1979), Loma Prieta (1989), Kobe (1995), Koyna (1995), Turkey (1998), Chi-Chi, Taiwan (1999), Bhuj (2001), and the Great East Japan earthquake (2011).

India and its surrounding regions have witnessed large earthquakes in the recent past like Muzaffarabad (2005), Sumatra (2004), Bhuj (2001) and Nepal (2015). Such earthquakes have caused great destruction and damages to both low rise and high rise buildings and other engineering structures. The Kachchh rift basin in Gujarat region witnessed the 2001 large earthquake (Mw7.6) which caused widespread liquefaction, ground failures, huge damages and loss of lives (e.g. Rao and Mohanty, 2001, Rastogi, 2001, Dubey and Dar, 2015).

Thus, the progressive expansion of safe human settlements requires practical understanding of the impact of possible natural hazards like earthquakes, landslides and subsidence on built environment and necessitates systematic development with minimum risks. The investigation related to the estimation of liquefaction susceptibility of an area is considered to be one of the basic mandatory requirements of systematic planning of urbanization. The Ahmedabad city in Gujarat sate located in the western part of India is an industrial capital with many big petrochemical complexes and scientific organizations and institutes. The city is experiencing great thrust in constructional activities and expanding its settlement area due to increased population and industrialization. In view of the above, it becomes necessary to investigate liquefaction hazard, which is one of the major secondary effects of an earthquake. In the present study, site classification and evaluation of liquefaction potential in terms of factor of safety for different sites in the developed as well as in the developing parts of the Ahmedabad city area are carried out on the basis of SPT N-values (Idriss and Boulanger, 2004). These results are much useful to assess the risk associated with earthquake, which is crucial for ensuring safety of the engineering constructions.

Geology of Ahmedabad city

Ahmedabad is one of the largest cities in India with a population of around 6.2 million (Census 2011). It is located between latitudes N22° 50´ to N23° 10´ and

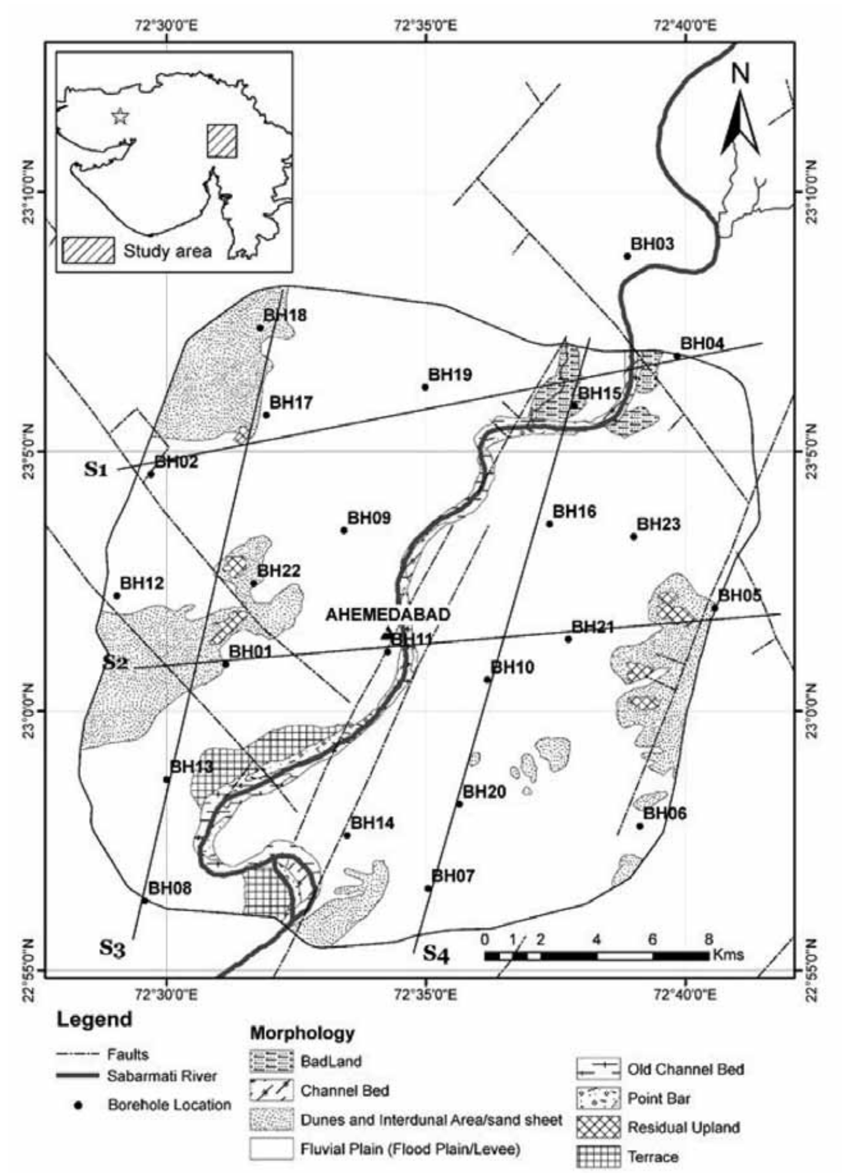

Figure 1. The map shows borehole locations, geomorphic features and selected profiles (S1-S4) in the study area. Inset: Map of Gujarat state, rectangle shows the study area, and the star indicates epicentre of the 2001 Bhuj earthquake Mw 7.7.

longitudes E 72° 26´ to E72° 43´ with an average elevation of 53m from the mean sea level (MSL). It is situated on the thick continental Quaternary sequences of Sabarmati basin, deposited by the erosion of hills of Aravalli mountains (Tandon et al., 1997). The Quaternary succession of the Sabarmati basin consists of conglomerate, sandy and silty soils and can be divided into four stratigraphic subdivisions viz. Waghpur Formation, Sabarmati Formation, Mahesana Formation and Akhaj Formation. The Waghpur Formation is characterized by well sorted fine buff sand, while the Sabarmati Formation is the youngest formation and consists of unconsolidated alluvium derived from Aravalli mountains (Sareen, 1992; Tandon et al., 1997). Geomorphologically, the study area can be divided into residential upland, low land, dune and inter-dunal regions with a few water reservoirs or ponds. The low land regions are further classified into flood plain, bad land, terrace, point bar, channel bar and recent channel. The Sabarmati river flows through the middle of the Ahmedabad city (Figure 1), which dries up in summer leaving only a small stream of water. Topographically, the study area is almost flat in nature except few small hills of Thaltej-Jodhpur Tekra. The average annual rainfall in the study area is 635 mm. According to Bureau of Indian Standards (BIS), the present study area falls under seismic zone III (IS-1893-Part I, 2002). The city has suffered severe damages at some places during the 2001 Bhuj earthquake, although the epicenter was ~250 km away from the main city.

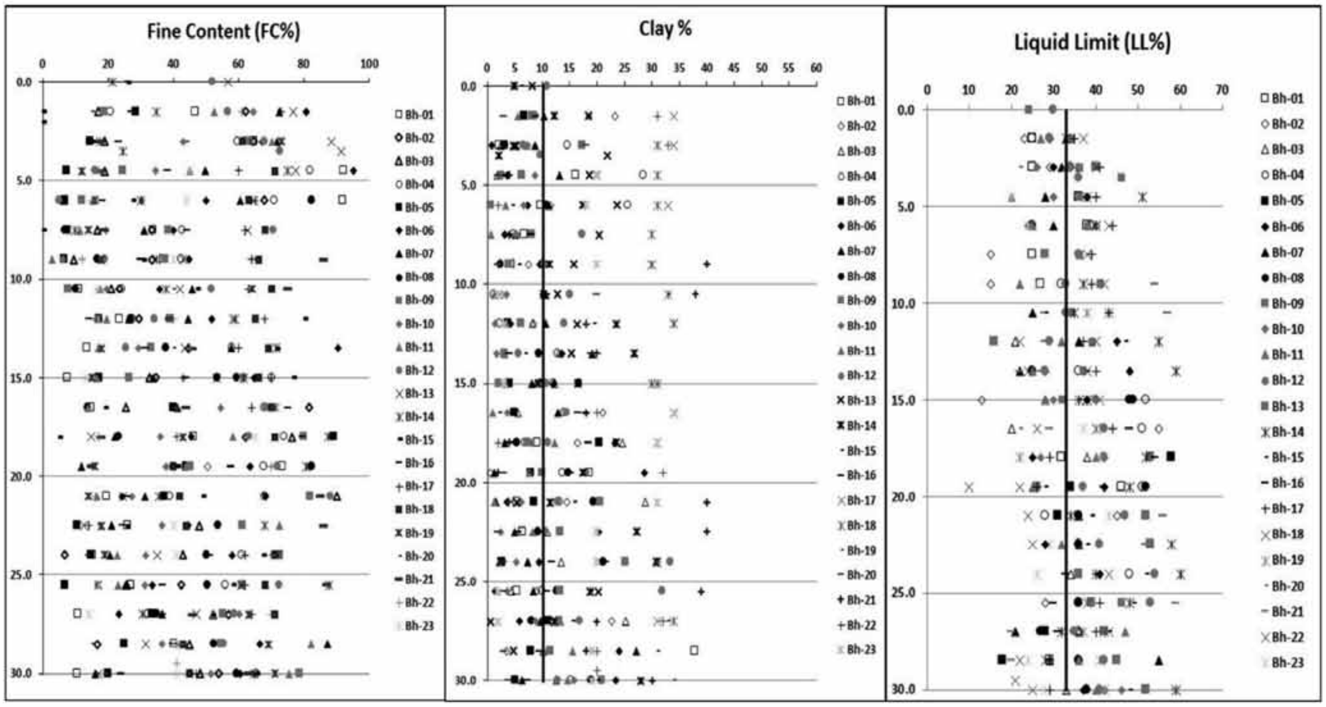

Figure 2. Fine content, clay % and liquid limit.

Geotechnical Investigations

In order to study the subsurface lithological characteristics of the region, a total of 23 boreholes (11 of 80m depth, six of 40m depth and five of 35m depth) were drilled at different locations (Figure 1). During drilling the soil samples, both disturbed and undisturbed, were collected for geotechnical investigation. The standard penetration tests were conducted, as per IS-code, at the depth interval of every 3m in the boreholes. The normalised SPT-N blow counts and lithology were recorded during sampling in the field. The depth of the water table measured during drilling was found to vary between 3m and 50m. Some variations in water table were, however, observed during different seasons, like that in monsoon and in summer. The SPT-N values measured are generally low in the shallow depths. The blow counts greater than 50 are generally encountered at depths of 6m to 9m which indicates that some shallow layers of the study area are prone to liquefaction. The measured SPT-N value N_{SPT}) however, depends on many factors such as hammer types, samplers used, drilling methods, types of rod used during drilling, borehole size, test procedure, etc. (Schmertmann and Palacios, 1979; Kavacs et al., 1981; Farrar et al., 1998; Sivrikaya and Togrol, 2006). Hence, the measured SPT blow count is first normalized for the overburden stress at the depth of the test and corrected to a standardized value of (N1)60. Using the recommended correction factors given by Robertson and Fear (1996), the corrected SPT blow count is calculated using the following equation (1):

$$(N_1)_{60} = N_{SPT} \cdot (C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S) \ldots \quad (1)$$

where, C_N = overburden correction factor, C_E = hammer energy correction factor, C_B = borehole diameter correction factor, C_r = rod length correction factor, C_s = correction factor for presence or absence of liner in borehole. The corrected N-Value "(N1)60" is further corrected for fine content on the basis of revised boundary curves derived by Idriss and Boulanger (2004) for cohesion-less soils. The fine content corrected SPT-N values, $(N_1)_{60cs}$ were finally used for evaluation of soil liquefaction in the present investigation. The other geotechnical parameters required for the evaluation of soil liquefaction, such as grain sizes, specific gravity, consistency limits, moisture content etc., were determined as per IS code in the geotech laboratory. Fine content (silt and clay) percentage, and liquid limit against depth for all the boreholes were plotted, which suggest that the area has variable soil types with low overall percentage of clay, with an average value of 30% (Figure 2)

Vertical Correlation of Sub-surface lithology

Four cross sections, two east-west (S1, S2) and two north-south (S3, S4), were made for vertical correlation of the subsurface lithology of the study area (Figure 1). The section

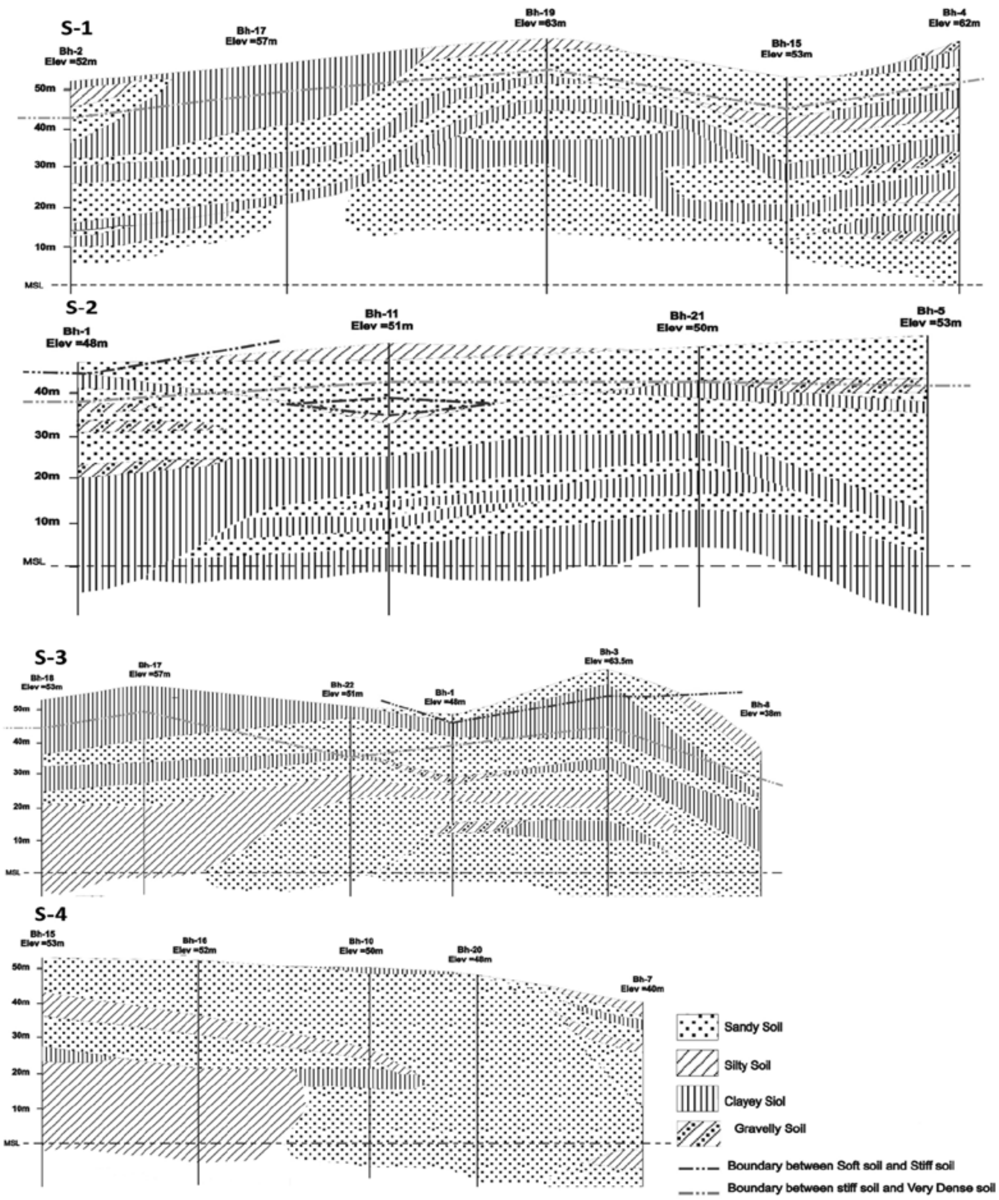

Figure 3. Vertical sections (S1-S4 shown in figure 1) of the sub-surface lithology.

S1 suggests that the subsurface lithology is characterized by fine grained clayey material at the topmost layer, which gets thicker towards the western side down to 15m. Silty soil is found in the middle part of the section with a thickness of 3m. This is followed by a layer of sandy soil with two thin layers of clayey soil. The SPT N-values were found to be greater than 50 blows at an average depth of 9m and at near surface in the eastern most part of the section.

In S2 section, the topmost lithology is characterized by sandy soil, which continued down to the depth of 25m with some lenticular layey and gravelly deposits in the western and eastern corners of the section (Figure 3). The sandy layer overlies a clayey layer, which has two lenticular shape sandy deposits with thickness of 1.5m and 3m present at depths of 30m and 35m, respectively in the middle part of the section. The SPT N-values were found to be greater than 50 blows at an average depth of 10m, while at 15m depth near Sabarmati river, the SPT- blows suddenly decreases to 14, may be due to the presence of thin clayey layer in the section.

In S3 section, the clayey sediment occupies the top layer with a thickness of 20m in the northern part and gets thinner in the middle part of the section (Figure 3). This clayey layer is followed by thick sandy deposits, which continue down to 50m with ~ 6m thick clayey layer and gravelly sediment in the middle part. The SPT N-values were found to be more than 50 blows at an average depth of 9m in northern and southern parts, whereas it was found to be more than 50 blows at a depth of 15m in the middle part of the section.

The S4 section consists of sandy deposits down to 50m with small patches of clay (Figure 3). In the northern part, the sediments are silty. The SPT N-values were found to be greater than 50 blows at an average depth of 6m in northern and at 10m in the southern part.

Liquefaction Evaluation

It is well known that coarse grained sediments i.e. sandy soils are potentially vulnerable to liquefaction. However, liquefaction in fine grained soil especially in silts and silty-clay are major issues in liquefaction assessment. In 1999 the Kocaeli (Turkey) earthquake and the Chi-Chi (Taiwan) earthquake, widespread liquefaction induced damages occurred in cohesive soil sites, which include partial settlements or complete bearing failures of shallow founded structures (Seed et al., 2003).

Liquefaction susceptibility of the fine grained sediments is usually assessed following the procedure of Andrews and Martin, (2000). If the sediments are susceptible to liquefaction then factor of safety against liquefaction is estimated at a particular depth. Seed and Idriss (1971) proposed a simplified procedure, termed as Cyclic Stress Method. In this method, earthquake induced loading, characterized in terms of the Cyclic Stress Ratio (CSR), is compared with the liquefaction resistance represented in terms of the Cyclic Resistance Ratio (CRR). The CSR is calculated using Equation (2):

$$\left(CSR = 0.65\left(\frac{\alpha_{\max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right) r_d\right) \quad \ldots\ldots (2)$$

where, 0.65 (α_{max} /g) = 65% of the peak cyclic shear stress; α_{max} =peak horizontal acceleration at the ground surface generated by the earthquake; g = acceleration due to gravity; σ_{vo} =total overburden stress; $\sigma_{vo'}$ = effective overburden stress; and r_d = stress reduction coefficient. Liao and Whitman (1986) proposed an empirical formulae to estimate average values of r_d.

r_d =1.0 – 0.00765z (for z ≤ 9.15 m), r_d= 1.174 – 0.0267z (for 9.15 < z ≤ 23 m), where, z= depth below ground surface in meters.

The Ahmedabad city area is in zone III as per seismic zoning map of India (BIS 2000), where expected PGA is 0.16g. Trivedi (2011), however, estimated different PGA values for various sites of Ahmedabad city the maximum PGA being 0.19g. Taking note of these values, in the present study we have considered PGA 0.2g and 0.3g for the evaluation of liquefaction.

The CRR is a function of the soil properties and the magnitude of an earthquake. Higher magnitude earthquakes induce more cycles of shaking than lower magnitude earthquakes. Thus larger magnitude earthquakes will induce liquefaction at a lower CSR than a lower magnitude earthquake (Youd and Idriss, 2001). CRR can be estimated using standard penetration test (SPT) as proposed by Idriss and Boulanger (2004).

The value of CRR for a magnitude 7.5 earthquake and an effective vertical stress σ_{vo} =1 atmosphere can be calculated based on (N_1) 60cs using the following equation proposed by Idriss and Boulanger (2004):

$$CRR = \exp\left\{\frac{(N_1)60cs}{14.1} + \left(\frac{(N_1)60cs}{126}\right)2 - \left(\frac{(N_1)60cs}{23.6}\right)3 + \left(\frac{(N_1)60cs}{25.4}\right)4 - 2.8\right\} \quad \ldots\ldots (3)$$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta N1_{60}$$

$$\Delta(N_1)_{60} = \exp\left[1.63 + \frac{9.7}{FC+0.001} - \left(\frac{15.7}{FC+0.001}\right)(N_1)_{60} = N_c(C_N * C_E * C_B * C_R * C_S)\right.$$

where the various correction factors C_N , C_E ,C_B, C_S ,C_R are as explained at equation 1 earlier and FC is fine content. The results of the liquefaction assessment are presented in terms of factor of safety (FS) against liquefaction:

$$FS = (CRR7.5/\ CSR)\ MSF \quad \ldots\ldots\ldots\ldots. (4)$$

CRR curves represent the liquefaction susceptibility for a magnitude of 7.5. Therefore, the factor of safety is multiplied with a magnitude scaling factor (MSF). Various values of MSF have been proposed based on empirical data (Youd et al., 2001). In the present study, MSF value 1.32 for M 6.0 suggested by Seed and Idriss (1982) is used for the analysis.

The evaluation of liquefaction of Ahmedabad city zone was performed in 23 boreholes with 138 blows data for PGA 0.2g and 0.3g down to depth of 20m. The layers having corrected SPT N-values greater than 50 were considered as non- liquefiable. In the present study, three different classes of liquefiable status were provided based on the factor of safety. Factor of safety (FS) less than 1 at particular depth was classified as liquefiable (Seed and Idriss, 1971), FS between 1 and 1.2 as marginally liquefiable and FS greater than 1.2 as non- liquefiable (Ulusay and Kuru 2004; Seed and Idriss, 1982).

The results obtained from liquefaction evaluation are assessed as (i) CSR vs. $(N_1)_{60cs}$ and (ii) factor of safety with depth (Figures 4 and 5). The CSR with $(N_1)_{60cs}$ defines the liquefiable condition as per Youd et al., (2001) guidelines. Samples which are liquefiable fall on the right side of the curve (FC=35%) and are composed of sandy and silty soil with intermediate to low plasticity. Marginally liquefiable samples fall between the curves of FC= 35% and FC≤ 5%. The samples falling on the left side of the curve (FC≤ 5%) are classified as non-liquefiable. In the present study out of 138 SPT- samples, for PGA value 0.2g, only two samples are found marginally liquefiable, and remaining non- liquefiable. About 97% of the investigated samples from different depths show non-liquefiable potential and only 3% are marginally liquefiable. For PGA value 0.3g,

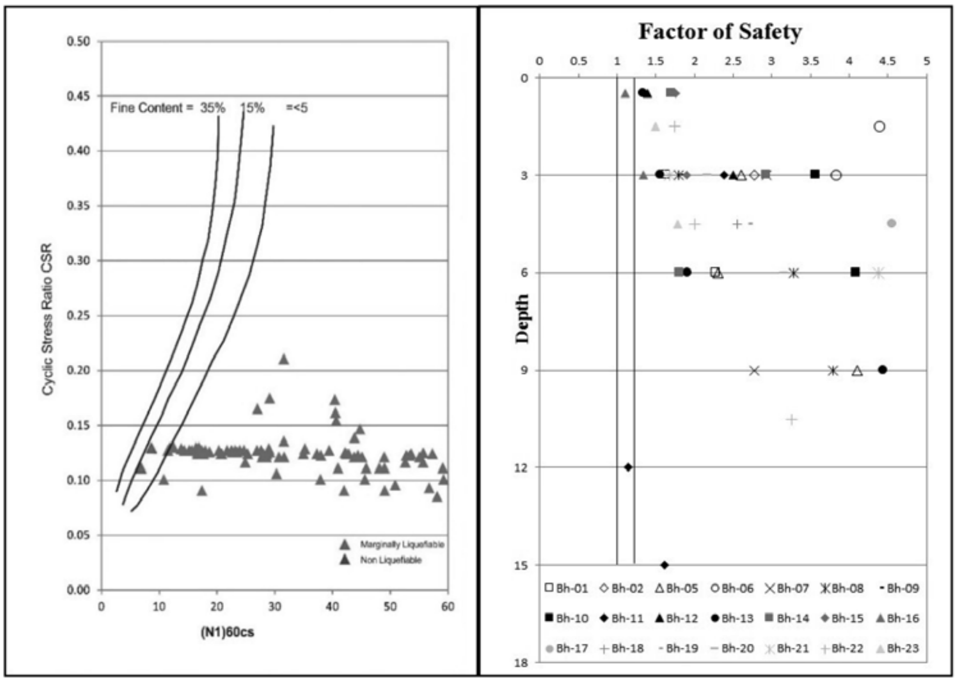

Figure 4. Plot showing CSR/$(N_1)_{60cs}$ and factor of safety against liquefaction for 0.2 g.

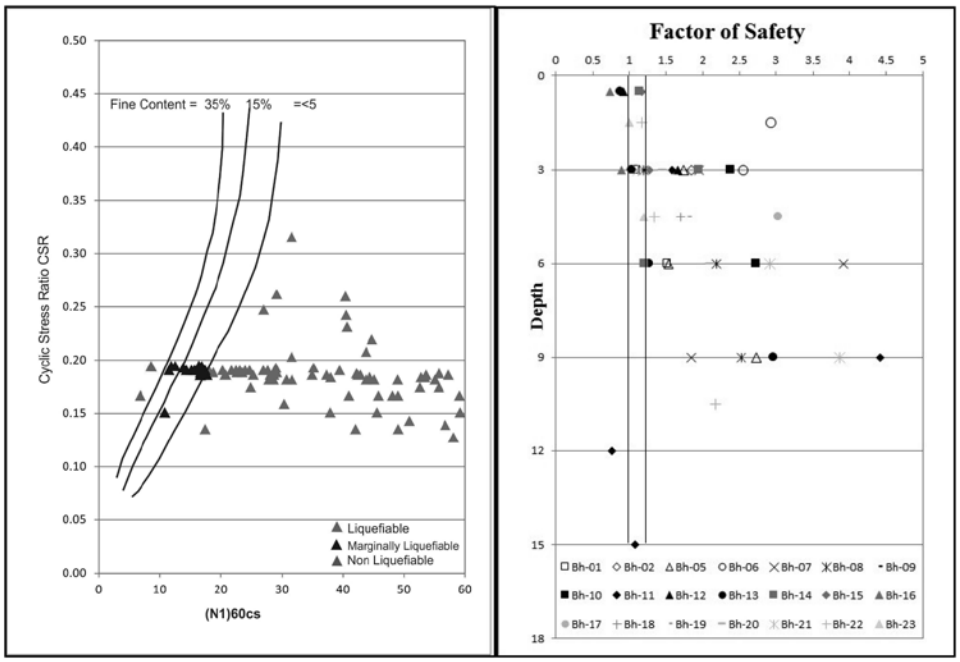

Figure 5. Plot showing CSR/$(N_1)_{60cs}$ and factor of safety against liquefaction for 0.3 g.

two samples are found liquefiable, nine are marginally liquefiable and remaining are non-liquefiable. About 90% of the investigated samples from different depths show non-liquefiable potential with only 8% as marginally liquefiable and only 2% liquefiable. The plot of factor of safety against liquefaction with depth shows that the area has high SPT N-values. Since, the N- values are high, the estimated FS for most of the samples were also relatively high. Liquefiable layers are found at depths of 12m and 15m with FS values of 0.185 and 0.174, respectively. However, marginally liquefiable layers are found down to the depth of 6m and one at 18m. Liquefaction below the depth of 10m is not expected since the sediments below this depth have higher relative densities.

Table1. Estimated factor of safety for PGA 0.2g and 0.3g at different boreholes.

Liquefaction Borehole No.	Depth of Liquefiable layers (m)	Factor of safety		Description	
		0.2g	0.3g	0.2g	0.3g
01	3.0	1.42	1.09	Non Liquefiable	Marginally Liquefiable
08	3.0	1.58	1.09	Non Liquefiable	Marginally Liquefiable
11	12.0	0.69	0.76	Liquefiable	Liquefiable
	15.0	0.93	1.04	Liquefiable	Marginally Liquefiable
13	3.0	1.41	1.04	Non Liquefiable	Marginally Liquefiable
14	6.0	1.12	1.20	Marginally Liquefiable	Marginally Liquefiable
16	3.0	1.19	0.89	Marginally Liquefiable	Liquefiable
21	3.0	1.48	1.11	Non Liquefiable	Marginally Liquefiable
22	1.5	1.75	1.16	Non Liquefiable	Marginally Liquefiable
23	1.5	1.5	1.01	Non Liquefiable	Marginally Liquefiable
	4.5	1.3	1.19	Non Liquefiable	Marginally Liquefiable

DISCUSSION AND CONCLUSIONS

The lithological variations delineated from 23 boreholes suggest that the Ahmedabad city area consists of different layers of gravel, sand and silty-clay. The sediment layers encountered in the boreholes have shown distinct variations in their thickness and shapes from location to location. The top six layers of five boreholes are found susceptible to liquefaction of which one borehole shows liquefiable potential and six of them show marginally liquefiable potential for PGA 0.3g. For PGA of 0.2g, no layer shows factor of safety less than 1.0 i.e. liquefiable, two layers show factor of safety between 1.0-1.2, i.e. marginally liquefiable (Table 1). Liquefaction in these layers may occur due to the presence of sandy-soil in the Clayey Sand (SC) layers. Besides, the water table is found at low levels except in boreholes BH08, BH07 and BH20 where it is at 3.5m, 5.2m and 6m, respectively. It is observed that the liquefiable layers are mostly confined to the central part of the city. Liquefaction is not observed below 10m due to the compaction and fine contents except in BH-11 at the depth of 12 m for PGA 0.2g and at depth 12m and 15m for PGA 0.3g at BH-11.

A total of eight layers at five different boreholes were found to be susceptible to liquefaction; out of which six comprises of top layers (down to 10 m depth). It can be concluded that the study area is marginally liquefiable at six layers in two boreholes in the western part, two boreholes in the eastern part and one borehole in the central part.

The different parameters analyzed in the present study indicate high compaction and presence of low water-table which give high SPT N-value and less liquefiable layers. Hence, it can be concluded that the study area is safe for construction activities. However, detailed site specific geotechnical investigations are required in case of high-rise constructions and heavy engineering structures.

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Compliance with Ethical Standards

The authors declare that they have no conflict of interest and adhere to copyright norms.

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