

# Soil liquefaction studies at Mumbai city

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**Abstract** Mumbai city is the economical capital of India and is situated about midway on the western coast of stable continental region of Peninsular India. Major part of the city being of reclaimed land, the soil type is of alluvium, sand, and recent conglomerate. There are some bigger water bodies within the city range. In this study, an attempt has been made to study the susceptibility of soil liquefaction using simplified empirical procedure based on number of blow counts ( $N$  values) of the soil layers from standard penetration test. The liquefaction susceptibility is quantified in terms of factor of safety along the borehole depths at available borehole locations using earthquake-induced cyclic stress on the soil and the cyclic resistance of the soil to withstand the load. The factor of safety against liquefaction is evaluated at different sites for two peak ground acceleration (PGA) levels pertaining to 10 and 2 % probability of exceedance in 50 years corresponding to uniform hazard response spectra for Mumbai city with 475- and 2,475-year return period, respectively. Contour maps are prepared that display the factor of safety at different depths for earthquake magnitude of  $M_w$  6.5. These contour maps show the liquefaction vulnerability at different sites in the city.

**Keywords** Liquefaction · Mumbai city · SPT  $N$  values · Cyclic loading · Cyclic resistance · Factor of safety

## 1 Introduction

Mumbai, a peninsular city situated about midway on the western coast of stable continental region of Peninsular India, is the economical capital of India and is the administrative

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capital of the state of Maharashtra. This is also a major industrial city in India and is 5th most densely populated cities in the world. The total population of Mumbai metropolitan region comprising of Mumbai city and the suburbs is 23.5 million as of April 1, 2011 (<http://www.citypopulation.de/>). The city, situated in the latitudes of 18°53'00''N to 19°19'00''N and longitudes of 72°45'00''E to 73°06'00''E, originally composed of a group of seven islands, namely Colaba on southernmost tip, Old Woman's island, Bombay, Mazagaon, Worli, Parel, and Mahim, and the islands were later joined to form the present city. Mahim Bay separates this group of islands from the Salsette Island to the north. The city is separated from the mainland by the estuary in the Vasai creek in the north, Ulhas River in the northeast, the Thane creek and the Harbour Bay in the east, and Arabian Sea to the south and the west. Major part of the city being of reclaimed land, the soil type is of alluvium, sand, and recent conglomerate. There are many hills and three hill ranges standing within the city limits. Sanjay Gandhi national park occupies most of northern part of suburban Mumbai city. There are three major lakes in the city, namely Powai lake, Vihar lake, and Tulsi lake.

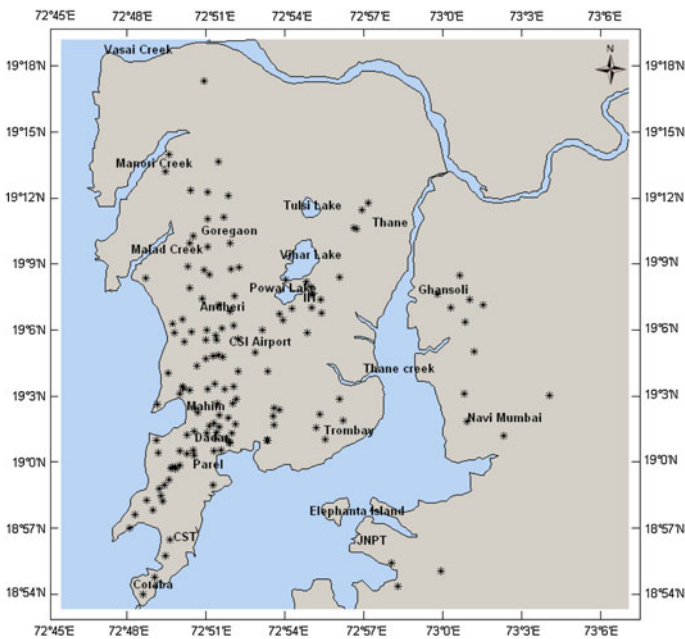
There are major fault lines around Mumbai that lie under the Thane creek, Panvel and Dharamtar creeks. Some minor fault lines also lie under the Powai lake, Vihar lake, and Tulsi lake, near the eastern suburbs, Ulhas river, Malad and Manori creeks. The Navi Mumbai zone to the east of the Thane creek is much more vulnerable. In the present scenario, an earthquake of magnitude 6 would cause severe damage in the city. According to Chandra (1977), Mumbai is in the seismically active Panvel zone. This zone strikes in north-northwest direction along the west coast. A large vertical fault with the same strike is exposed on the western slopes of the Western Ghats in the Belgaum-Vengurla region. Therefore, the volcanic rocks those appear very stable on the surface have a system of north-northwest running faults underneath in this zone. Raghukanth and Iyengar (2006) identified the presence of 23 major faults that are very likely to influence seismic hazard at Mumbai city. As per the IS 1893 (part 1): 2002, the city comes under seismic zone III. Raghukanth (2011) estimated that the seismic activity rate ( $\lambda$ ),  $b$  value, and maximum expected magnitude ( $M_{\max}$ ) for the 300-km control region surrounding Mumbai city are  $0.32 \pm 0.06$ ,  $0.82 \pm 0.07$  and 6.8, respectively. Mhaske and Choudhury (2010) studied soil liquefaction for Mumbai city and identified few liquefiable places. Martin and Szeliga (2010) estimated the probable return period for V, VI, and VII intensity for Mumbai city as 42, 78, and 145 years, respectively. Raghukanth and Iyengar (2006) estimated the seismic hazard for the city in the form of 5 % damped response spectra based on probabilistic seismic hazard analysis (PSHA) for 2 and 10 % probability of exceedance in 50 years corresponding to uniform hazard response spectra of nearly 2,475- and 475-year return period, respectively. In this study, an attempt has been made to study the soil liquefaction at Mumbai city in terms of factor of safety at different depths for peak ground acceleration (PGA) levels corresponding to 2 and 10 % probability of exceedance in 50 years.

## 2 Sources of data

Site conditions play a major role in the liquefaction of soil during seismic events. A vast stretch of land encompassing an area of 104 km<sup>2</sup> is being occupied by Sanjaya Gandhi national park to the north. Several water bodies are lying within the city range and there are tidal swamps to the east of the city along the Thane creek. The borehole locations obtained from several sources are observed to be densely clustered in inhabited areas. The SPT

*N* value at a particular site represents most of the important mechanical properties of the soil required for the liquefaction potential analysis. In an attempt to evaluate the liquefaction potential for the city, SPT soil data with borelog characteristics containing the details of various input parameters of the soil profiles with depth, namely groundwater depth, SPT *N* values, dry density, wet density, specific gravity and fines content (FC), etc., have been collected from 142 representative borehole locations in the city. Majority of the soil profile data used in this analysis are contributed by DBM Geotechnics and Constructions Pvt Ltd and IIT Bombay. The borehole locations considered for the liquefaction potential analysis in the present study are marked as stars (\*) in Fig. 1. The depth of boreholes are in the range of 7.5–30 m. The SPT *N* values are measured at every 1.5 m depth intervals until rock strata is encountered. The soil deposits are found to be comprising of sand, silt, clay, clayey sand, silty sand, clayey silt, silty clay, and silty gravel, etc. deposited in different layers. The SPT blow counts at some places are in the order of 2–10 indicating soft deposits of clay, whereas at most of the places it is up to 40 showing dense silty sand. Some of the sites also belong to rock site category.

The seismic site classification is carried out based upon SPT *N* values complying with the International Building Code (IBC 2003) and provisions in National Earthquake Hazards Reduction Program (NEHRP). Provisions in National Earthquake Hazards Reduction Program (BSSC 2009) suggest that the site conditions can be characterized into different categories according to the mean shear wave velocity in the upper 30 m of the soil layer ( $\bar{V}_s30$ ).  $\bar{V}_s30$  is defined as the ratio of 30 m depth to the time for vertically propagating shear waves to travel from 30 m depth to reach the ground surface.  $\bar{V}_s30$  is calculated as



**Fig. 1** Borehole locations in Mumbai city selected for liquefaction studies

$$\bar{V}_s30 = \sum_{i=1}^n d_i / \sum_{i=1}^n \frac{d_i}{V_{si}} \quad (1)$$

where  $d_i$  and  $V_{si}$  are the thickness and the shear wave velocity of  $i$ th soil layer for  $\Sigma d_i = 30$  m and  $n$  is number of layers.

If the shear wave velocities of the soil profiles are not available, it is also recommended that empirical relationships between shear wave velocity ( $V_s$ ) and SPT blow count ( $N$ ) may be used to obtain shear wave velocity of the soil profile. For this purpose, the correlations between  $V_s$  and SPT  $N$  proposed by Imai (1977), Ohta and Goto (1978), and Imai and Tonouchi (1982) as given in Table 1 are used for the determination of corresponding shear wave velocity. These relationships are reported to be valid for a wide range of soil type. The shear wave velocity for the soil layers at each borehole location in the city are taken as the average of these three values so obtained. The site classification system suggests that from a total of 142 sites, 9 sites correspond to E-type ( $\bar{V}_s30 < 600$  ft/s), 94 sites correspond to D-type ( $600 \text{ ft/s} \leq \bar{V}_s30 \leq 1,200$  ft/s), and 39 C-type sites ( $1,200 \text{ ft/s} \leq \bar{V}_s30 \leq 2,500$  ft/s). The majority of the sites in the study area based on the available soil profile data are observed to be of D and C-type site category.

### 3 Soil liquefaction during earthquake

Severe soil liquefactions in extensive areas have been widely observed during numerous devastating earthquakes causing liquefaction induced ground failures, namely ground settlements, flow slides, reduction in bearing capacity, horizontal displacements, lateral spreads, landslides, ground cracks, sand boils, dam instability, and highway embankment failures. It has been a major cause of damage to different soil structures, building foundations and lifeline structures and other associated failures. A number of studies on soil liquefaction and associated failures have been performed after 1964 Alaska and Niigata earthquakes. Under cyclic loading induced by seismic excitation, granular materials transform from a solid to a liquefied state due to increased pore water pressure, as the excess pore water pressure does not dissipate in insufficient time through natural drainage, and thus, shear strength and stiffness of a soil deposit are reduced. Liquefaction is most commonly observed in loose, saturated, clean to silty sands and also has been observed in non-plastic silts and low-plasticity silty clays.

The soil liquefaction depends on the magnitude of earthquake, intensity and duration of ground motion, seismic attenuation characteristics, the distance from the source of the earthquake, site-specific conditions, type of soil, relative density, grain characteristics, grain size distribution, confining pressure, ground acceleration, age and thickness of the soil deposit, permeability characteristics of soil layer, depth of the ground water table, plasticity of fines, degree of saturation, rate of excess pore pressure generation, dissipation of pore pressure, reduction of effective stress, shear modulus degradation, and FC (Youd and Perkins 1978; Kramer 1996; Youd et al. 2001). Liquefaction of a soil deposit can also

**Table 1** Correlations between  $V_s$  and SPT  $N$  for all types of soil

Imai (1977)	$V_s = 91.0 N^{0.337}$
Ohta and Goto (1978)	$V_s = 85.35 N^{0.348}$
Imai and Tonouchi (1982)	$V_s = 97.0 N^{0.314}$

be preliminarily predicted by using historical, geological, hydrological, and compositional criteria (Kramer 1996). Accurate evaluation of liquefaction potential of soils is the first step toward the mitigation of the damages caused by liquefaction.

#### 4 Assessment of liquefaction potential

The simplified empirical procedure based on number of blow counts ( $N$  values) of the soil layers from standard penetration test (SPT) is widely used for evaluating liquefaction resistance of soils using seismic loading and soil resistance characterized in terms of cyclic shear stress amplitudes (Seed and Idriss 1971). Soil liquefaction potential is generally assessed in terms of factor of safety against liquefaction (FS), defined as the ratio of liquefaction resistance of the soil expressed in terms of cyclic resistance ratio (CRR) over earthquake-induced loading expressed in terms of cyclic stress ratio (CSR). CRR is the capacity of the soil to resist liquefaction, and CSR is the seismic demand on a soil layer. FS less than 1 at a particular depth indicate liquefaction susceptibility at that depth. This SPT-based empirical procedure has been revised and updated since 1971 (Seed et al. 1985; Youd and Idriss 1997; Cetin et al. 2004; Idriss and Boulanger 2006). Other in situ tests such as, cone penetration test (CPT), Becker penetration test (BPT) and shear wave velocity ( $V_s$ ) are also used to evaluate liquefaction potential of soils (Youd et al. 2001). As an alternative to the above procedures, the liquefaction potential is also assessed in terms of probability of liquefaction (Liao et al. 1988; Youd and Noble 1997; Juang et al. 2000a, b; Moss et al. 2006). In this study, seismic soil liquefaction potential at Mumbai city is investigated at 142 different sites in the city for earthquakes of  $M_w$  5.5 to  $M_w$  7.5 based on a simplified deterministic approach by Idriss and Boulanger (2006, 2010) using in situ SPT test data. Raghukanth and Iyengar (2006) derived uniform hazard response spectra for Mumbai city for A-, B-, C-, and D-type sites separately considering the influence of local site conditions. As most of the sites in the city considered in this study belong to D-type category, the PGA levels chosen from Raghukanth and Iyengar (2006) for the analyses are 0.18 and 0.30 g that correspond to 10 and 2 % probability of exceedance in 50 years, respectively, for different magnitudes of earthquake.

##### 4.1 Evaluation of cyclic stress ratio

Cyclic stress ratio characterizes the seismic loading. This can be estimated from peak ground surface acceleration that corresponds to ground motions for site-specific seismic hazard levels, and a stress reduction factor that represents the variation of shear stress amplitude with depth for the soil profiles of interest. The CSR induced by earthquake ground motions, at a given depth  $z$  below the ground surface, is usually expressed as an equivalent uniform value equal to 65 % of the maximum cyclic shear stress ratio (Seed and Idriss 1971). The CSR for a specific earthquake magnitude (moment magnitude,  $M_w$ ) and in situ  $\sigma'_v$  is computed as

$$CSR_{M_w, \sigma'_v} = 0.65 \left( \frac{\sigma_v a_{\max}}{\sigma'_v g} \right) r_d \quad (2)$$

in which  $a_{\max}$  is the peak horizontal acceleration at the ground surface,  $g$  is the acceleration due to gravity,  $\sigma_v$  is the total vertical stress, and  $\sigma'_v$  is the effective vertical stress at depth  $z$  below the ground surface. The parameter  $r_d$  is a stress reduction coefficient that accounts for the dynamic response of the soil column. The stress reduction coefficient ( $r_d$ ) is calculated using the following expressions (Idriss 1999).

$$r_d = \exp[\alpha(z) + \beta(z)M_w] \quad \text{for } z \leq 34 \text{ m} \tag{3}$$

$$r_d = 0.12 \exp(0.22M_w) \quad \text{for } > 34 \text{ m} \tag{4}$$

where

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \tag{5}$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \tag{6}$$

where  $z$  is the depth in meters, and  $M_w$  is moment magnitude. The arguments inside the sine terms are in radians. The depth of boreholes considered in the present analysis are less than 34 m. Thus, Eq. 3 is applied for the calculation of stress reduction coefficient.

The CSR induced during an earthquake of magnitude  $M_w$  is adjusted to the equivalent uniform shear stress induced by an earthquake, having a moment magnitude  $M_w$  7.5, is corrected through introducing the magnitude scaling factor (MSF). The updated empirical expression to evaluate the CSR is given by Idriss and Boulanger (2006).

$$CSR_{M_w=7.5} = 0.65 \left(\frac{\sigma_v a_{\max}}{\sigma'_v g}\right) \frac{r_d}{MSF} \tag{7}$$

MSF accounts for the effects of ground shaking duration on the liquefaction potential. Several different methods of calculation for MSF exist. Idriss (1999) proposed following equations to calculate MSF.

$$MSF = 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058 \leq 1.8 \quad \text{for } M_w < 7.5 \tag{8}$$

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad \text{for } M_w \geq 7.5 \tag{9}$$

These expressions for MSF provide a more conservative results and maintain similarity in values with other preferred approaches.

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  $\sigma'_v$  of 1 atmosphere (Boulanger 2003; Idriss and Boulanger 2006).

$$CSR_{M_w=7.5, \sigma'_v=1} = 0.65 \left(\frac{\sigma_v a_{\max}}{\sigma'_v g}\right) \frac{r_d}{MSF K_\sigma} \tag{10}$$

where

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{p_a}\right) \leq 1.0 \tag{11}$$

$$C_\sigma = \frac{1}{18.9 - 2.5507\sqrt{(N_1)_{60}}} \leq 0.3 \tag{12}$$

where  $p_a$  is the atmospheric pressure (=100 kPa).  $(N_1)_{60}$  is the SPT  $N$  value corrected to an equivalent energy ratio of 60 % and effective overburden pressure of one atmosphere. The measured SPT  $N$  values,  $N_m$ , are corrected for overburden stress, equipment and energy factors by introducing a series of correction factors (Youd et al. 2001).

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{13}$$

where  $C_N$  is the correction factor for effective overburden stress expressed as follows (Boulanger and Idriss 2004),

$$C_N = \left( \frac{p_a}{\sigma'_v} \right)^{0.784 - 0.0768\sqrt{(N_1)_{60}}} \tag{14}$$

$C_N$  limited to a maximum value of 1.7.

$C_E$  is the correction factors for hammer energy ratio (=ER/60 %). ER is the measured energy ratio.  $C_B$ ,  $C_R$ , and  $C_S$  are the correction factors for borehole diameter, for rod length, and for sampler with or without liner, respectively. The suitable values of the correction factors are chosen from Youd et al. (2001) as per the requirement.

#### 4.2 Evaluation of cyclic resistance ratio

The CRR of a soil is a measure of capacity of soils to resist liquefaction and is generally characterized by penetration resistance. The CRR is correlated to corrected SPT  $N$  values. It is also influenced by the duration of shaking and effective overburden stress. The FC, in the soil, varies in a given layer that could significantly affect the penetration resistance, and hence, the correction for the FC is also accounted in the evaluation of corrected SPT values. The CRR at certain depth is determined for a reference  $M_w = 7.5$  and  $\sigma'_v = 1$  atm for the soil with FC as per the the following expression recommended by Idriss and Boulanger (2006).

$$CRR_{M_w=7.5, \sigma'_v=1} = \exp \left\{ \frac{(N_1)_{60cs}}{14.1} + \left( \frac{(N_1)_{60cs}}{126} \right)^2 - \left( \frac{(N_1)_{60cs}}{23.6} \right)^3 + \left( \frac{(N_1)_{60cs}}{25.4} \right)^4 - 2.8 \right\} \tag{15}$$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \tag{16}$$

$\Delta(N_1)_{60}$  is the equivalent clean sand adjustment that accounts for the effects of FC on the CRR and SPT  $N$  values and is expressed as

$$\Delta(N_1)_{60} = \exp \left( 1.63 + \frac{9.7}{FC + 0.1} - \left( \frac{15.7}{FC + 0.1} \right)^2 \right) \tag{17}$$

The evaluated CRR values are then adjusted to a specific earthquake magnitude,  $M_w$  and in situ  $\sigma'_v$  using the following expression.

$$CSR_{M_w, \sigma'_v} = CSR_{M_w=7.5, \sigma'_v=1} MSF K_\sigma \tag{18}$$

The expressions for MSF and  $K_\sigma$  are same as above in the Sect. 4.1.

### 4.3 Evaluation of factor of safety

The soil liquefaction potential is quantified in terms of a factor of safety after loading has been expressed in terms of CSR, and resistance has been expressed in terms of CRR. The liquefaction potential is evaluated at different depths within the soil profile since both CSR and CRR vary with depth. The factor of safety against liquefaction, FS, is defined by

$$FS = \frac{CRR(z)}{CSR(z)} \quad (19)$$

where  $CRR(z)$  and  $CSR(z)$  are the value of the CRR at depth  $z$  and CSR at same depth  $z$  respectively.

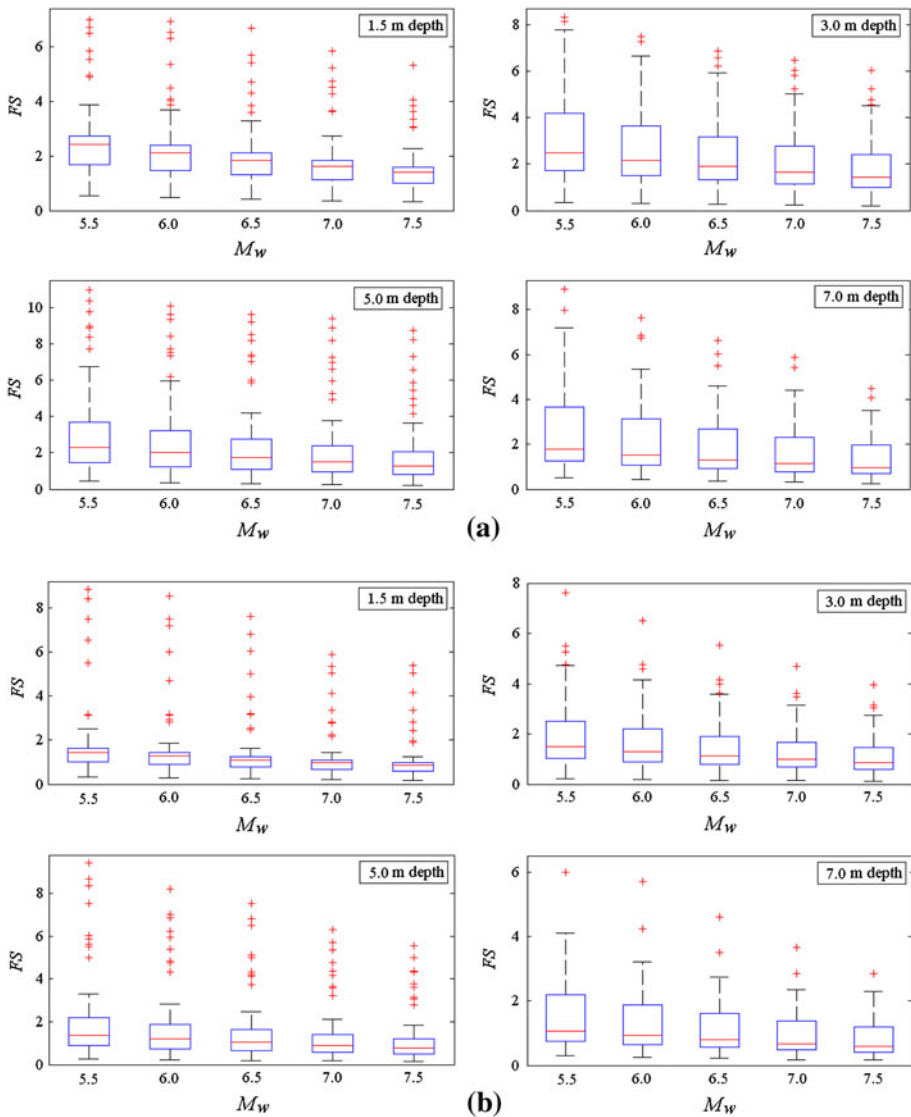
## 5 Results and discussions

Two important ground motion parameters, namely the earthquake magnitude and the PGA, used in this study are as per the recommendations of Raghukanth and Iyengar (2006) in order to evaluate the liquefaction potential using the SPT simplified procedure. Figure 2 shows box plots for the obtained factor of safety (FS) profiles for the two typical  $a_{max}$  values (0.18 and 0.30 g) to compare the factor of safety against liquefaction for magnitudes of  $M_w$  5.5,  $M_w$  6.0,  $M_w$  6.5,  $M_w$  7.0, and  $M_w$  7.5 for a return period of 475 years and return period of 2,475 years. These box plots display the medians (represented by the red line in the rectangular box), first quartile as the 25th percentile, third quartile as the 75th percentile, minimum and maximum value of the FS values. The outliers in the dataset of FS values are displayed separately as addition symbol (+) in red. The median value for certain depth at a site less than 1 indicates that soil liquefaction is likely to occur at that particular site. The median of the FS values for a particular depth is observed to be decreasing with increase in the magnitude. This indicates the vulnerability with increase in intensity of seismic events. Cumulative distribution functions (CDF) of factor of safety (FS) at several depths of 1.5, 3.0, 5.0, and 7.0 m has been computed from the FS profiles and are presented in Fig. 3. This figure shows the probabilities of liquefaction susceptibility at a depth for different magnitudes of earthquake. Contour maps for the city are prepared in the Figs. 4, 5, 6 and 7 to present the factor of safety against liquefaction at different depths for the earthquake magnitude  $M_w$  6.5 of PGA levels 0.18 and 0.30 g corresponding to 475-year return period and 2,475-year return period, respectively.

For the  $a_{max}$  value of 0.18 g, it is observed that the factor of safety (FS) at different depths at some places in the city, namely JNPT, Wadala, Marine line, Sewree, Bandra, Sion, are less than 1.0 for each seismic events considered in this study. The higher susceptibility of liquefaction at these places are due to the higher thickness of soft soil deposits and ground water table at shallow depths.

The factor of safety (FS) at different depths at many of the places in the city, namely JNPT, Wadala, Marine line, Sewree, Bandra, Sion, Andheri, Trombay, Goregaon, and Mahim, etc., are less than 1.0 for the  $a_{max}$  value of 0.30 g for different magnitudes of earthquakes. This increase in the liquefaction susceptibility may be attributed to the higher  $a_{max}$  value considered in the analysis for 2,475-year return period. The places to the east of Thane creek, such as Navi Mumbai, Mahape, are found to be safe against liquefaction. The factor of safety (FS) at many of the places are also found to be much greater than 1.0 for



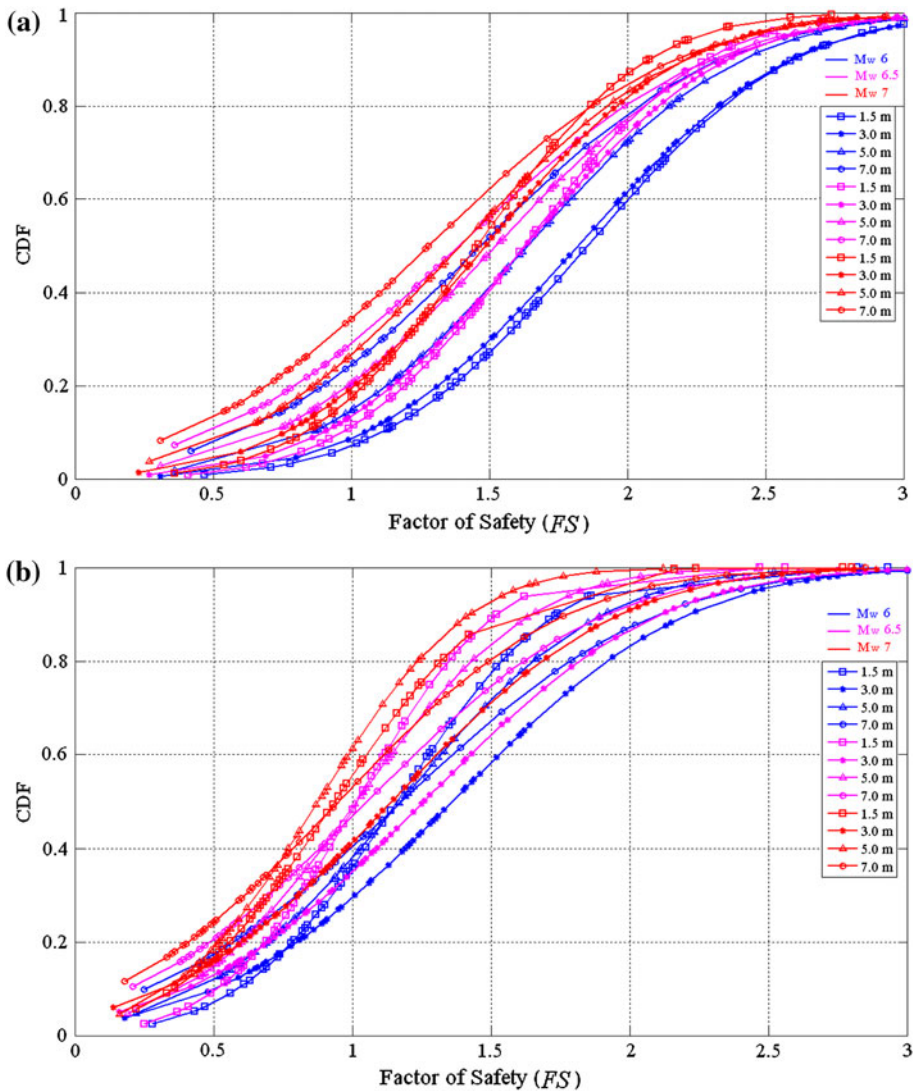


**Fig. 2** Factor of safety against liquefaction for different magnitudes of earthquake **a** for a return period of 475 years, and **b** for a return period of 2,475 years

both the peak ground acceleration ( $a_{max}$ ) scenarios due to higher SPT values at those corresponding rock sites.

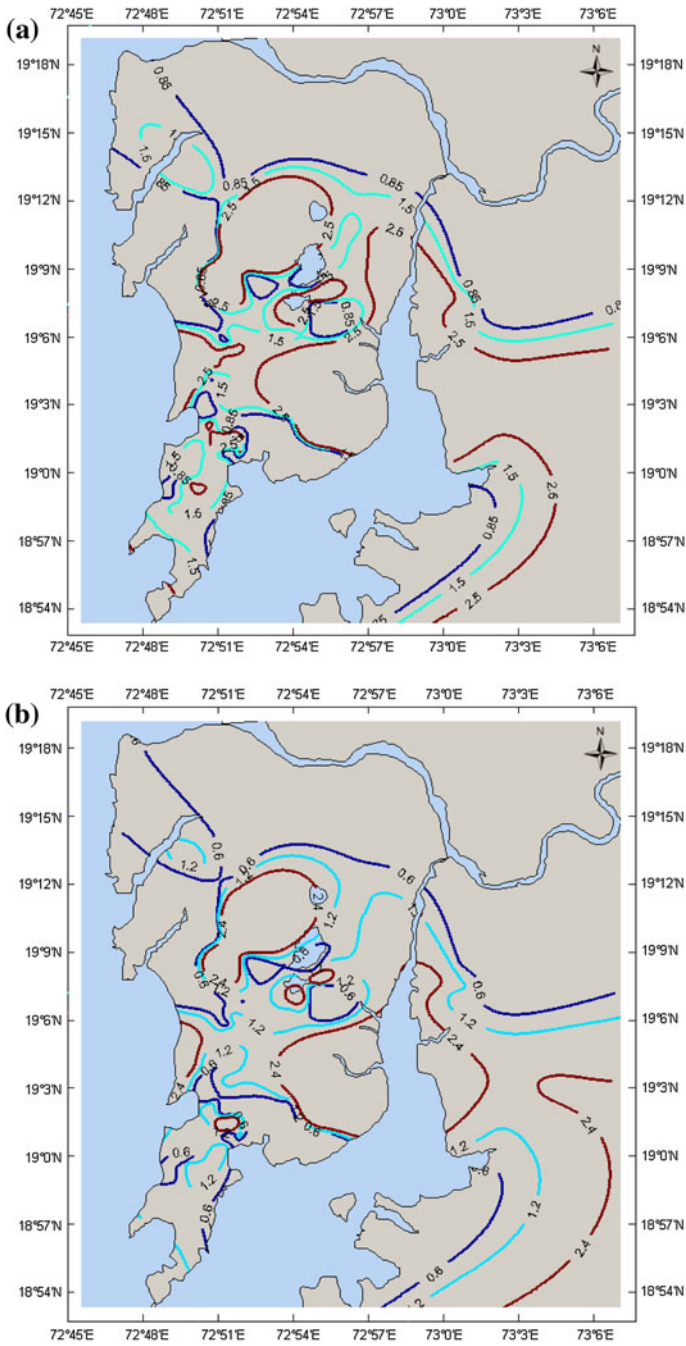
### 6 Conclusion

In this paper, an attempt has been made to study the soil liquefaction potential in terms of factor of safety at Mumbai city using SPT-based semi empirical procedure. The

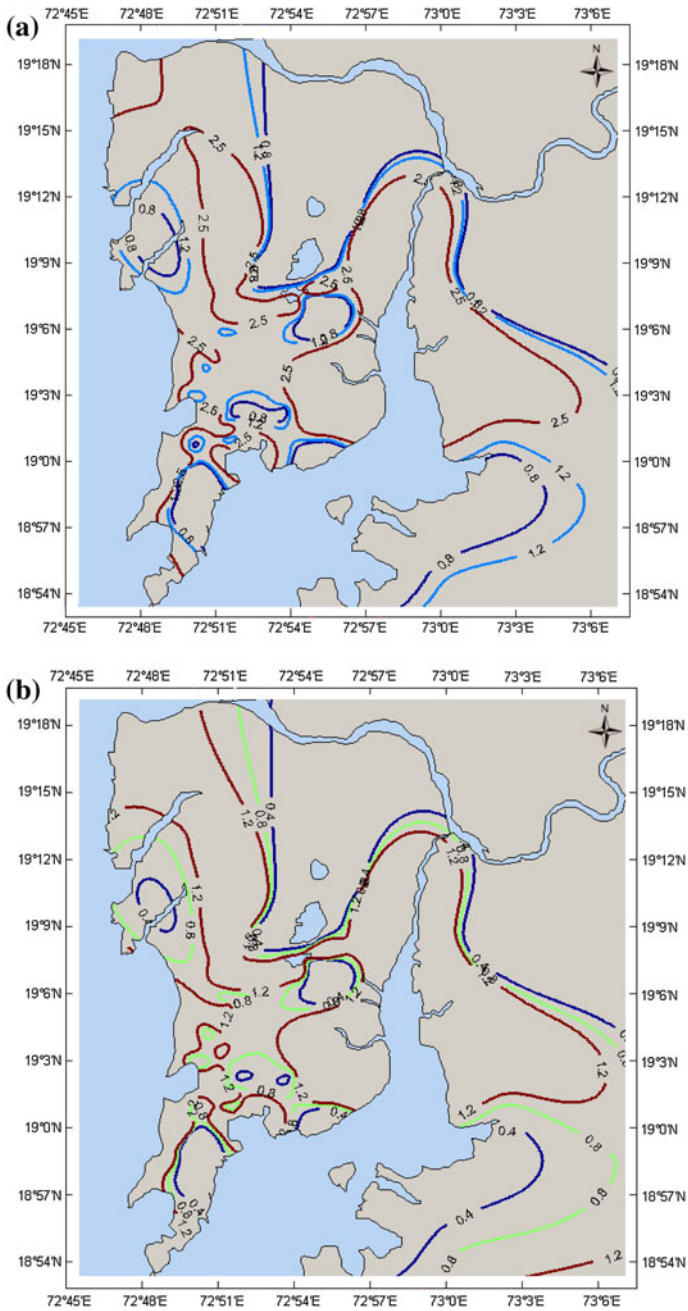


**Fig. 3** Cumulative distribution function for factor of safety against liquefaction at different depths for different magnitudes of earthquake **a** for a return period of 475 years, and **b** for a return period of 2,475 years

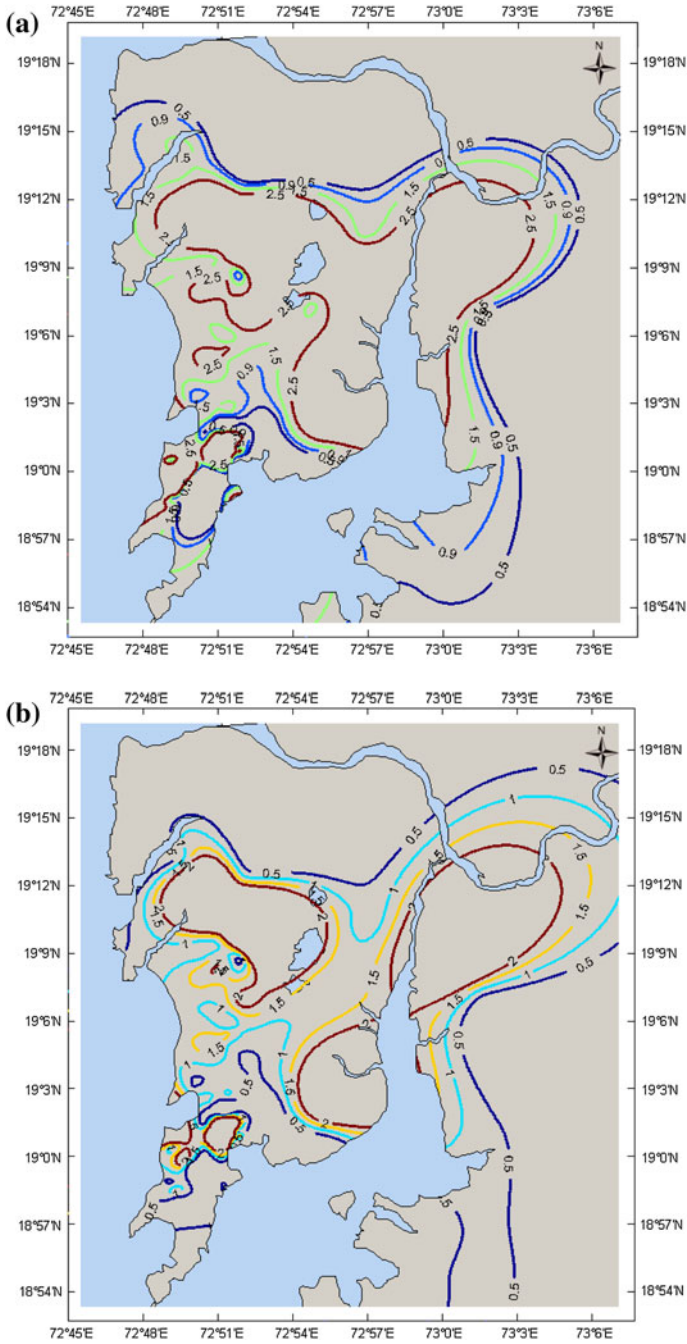
liquefaction potential has been presented in the form of contour maps. It can be observed from the obtained results and the contour maps that high degree of liquefaction failures are likely to occur at many sites in the city during severe seismic event. The areas developed on reclaimed land along coastline having large thickness of soft soil and shallow ground water levels are observed to be more susceptible to liquefaction. These contour maps will help the designers and city planners to check the vulnerability of the area against liquefaction.



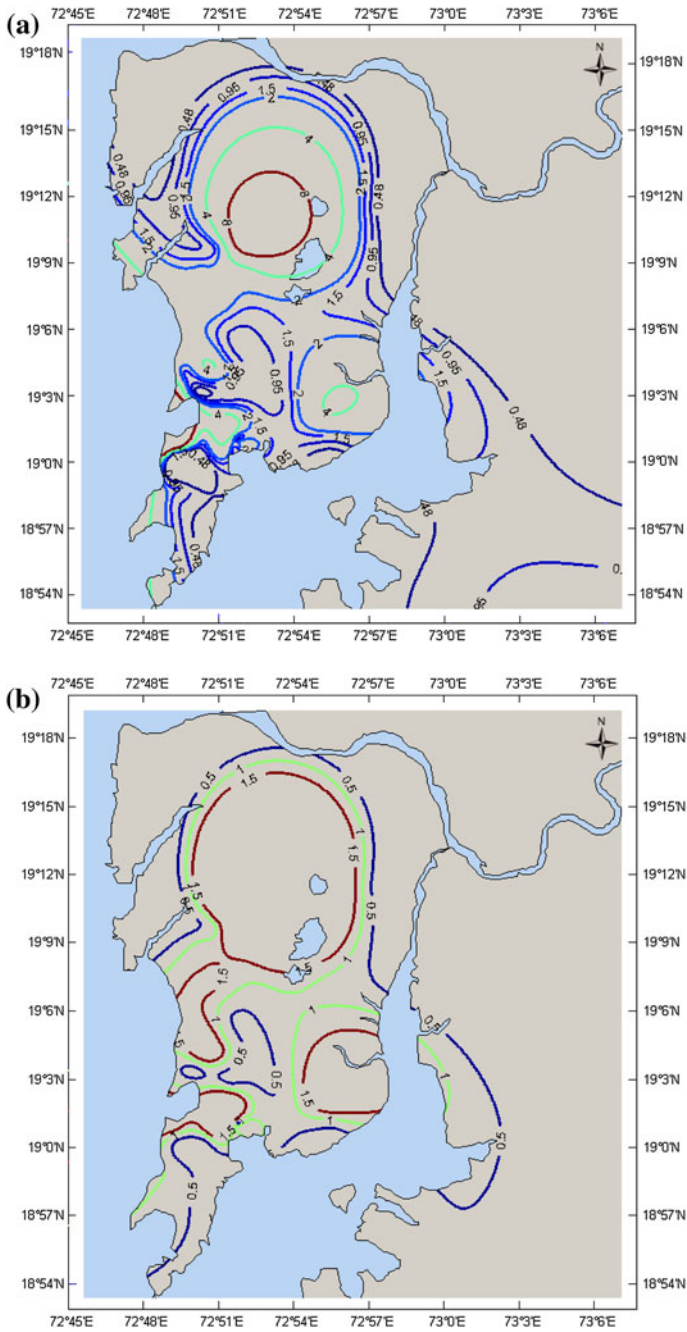
**Fig. 4** Contour maps of factor of safety against liquefaction at 1.5 m depth for earthquake magnitude of  $M_w$  6.5 **a** for 475-year return period, and **b** for 2,475-year return period



**Fig. 5** Contour maps of factor of safety against liquefaction at 3.0 m depth for earthquake magnitude of  $M_w$  6.5 **a** for 475-year return period, and **b** for 2,475-year return period



**Fig. 6** Contour maps of factor of safety against liquefaction at 5.0 m depth for earthquake magnitude of  $M_w$  6.5 **a** for 475-year return period, and **b** for 2,475-year return period



**Fig. 7** Contour maps of factor of safety against liquefaction at 7.0 m depth for earthquake magnitude of  $M_w$  6.5 **a** for 475-year return period, and **b** for 2,475-year return period

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