Nat Hazards (2010) 53:547–560
DOI 10.1007/s11069-009-9447-3

ORIGINAL PAPER

Probabilistic evaluation of seismic soil liquefaction potential based on SPT data

K. S. Vipin · T. G. Sitharam · P. Anbazhagan

Received: 16 May 2008 / Accepted: 1 August 2009 / Published online: 27 August 2009

Abstract The performance-based liquefaction potential analysis was carried out in the present study to estimate the liquefaction return period for Bangalore, India, through a probabilistic approach. In this approach, the entire range of peak ground acceleration (PGA) and earthquake magnitudes was used in the evaluation of liquefaction return period. The seismic hazard analysis for the study area was done using probabilistic approach to evaluate the peak horizontal acceleration at bed rock level. Based on the results of the multichannel analysis of surface wave, it was found that the study area belonged to site class D. The PGA values for the study area were evaluated for site class D by considering the local site effects. The soil resistance for the study area was characterized using the standard penetration test (SPT) values obtained from 450 boreholes. These SPT data along with the PGA values obtained from the probabilistic seismic hazard analysis were used to evaluate the liquefaction return period for the study area. The contour plot showing the spatial variation of factor of safety against liquefaction and the corrected SPT values required for preventing liquefaction for a return period of 475 years at depths of 3 and 6 m are presented in this paper. The entire process of liquefaction potential evaluation, starting from collection of earthquake data, identifying the seismic sources, evaluation of seismic hazard and the assessment of liquefaction return period were carried out, and the entire analysis was done based on the probabilistic approach.

Keywords Earthquakes · PSHA · SPT · Seismic soil liquefaction

1 Introduction

One of the most devastating geotechnical related effects of earthquakes is soil liquefaction. This causes failure of foundations, soil embankments and dams and these failures ultimately affect the social and financial status of the region. The devastating effects of liquefaction have been observed during the 1964 Niigata earthquake, 1985 Kobe

K. S. Vipin · T. G. Sitharam (✉) · P. Anbazhagan
Department of Civil Engineering, Indian Institute of Science, Bangalore 560012, India
e-mail: sitharam@civil.iisc.ernet.in

earthquake, 1989 Loma Prieta earthquake and 2001 Bhuj earthquake. These seismic soil liquefaction cases have stressed the need for assessment of liquefaction potential of seismically active sites. Effects of seismic soil liquefaction have been highlighted since 1964 Niigata earthquake in Japan, and attempts to understand soil liquefaction have been done since then. The most widely followed liquefaction evaluation method is the one suggested by Seed et al. (1985). In this method, the seismic loading is calculated using a single ground acceleration level ($a_{\max}$) and a single earthquake magnitude (M_w). The method suggested by Kramer and Mayfield (2007) considers the entire range of ground motion and magnitude levels for the evaluation of return period of seismic soil liquefaction.

The liquefaction susceptibility of a site can be assessed by laboratory tests or in situ tests. Due to the difficulties associated with obtaining good soil samples, in situ methods are widely used. The field tests which have gained common usage for evaluation of liquefaction susceptibility are the standard penetration test (SPT), cone penetration test (CPT), shear wave velocity test (V_s) and the Becker penetration test (BPT) (Youd et al. 2001). Among these tests, SPT being widely used, this paper explains the spatial evaluation of probabilistic liquefaction assessment of Bangalore, India, using SPT data and other soil properties obtained from 450 boreholes.

2 Study area

Bangalore, which is on south Karnataka Plateau (Mysore Plateau), is situated in south India at an average altitude of 910 m above mean sea level (MSL). The study area is the Bangalore metropolitan area, which covers about 220 km^2 and is shown in Fig. 1. Currently, Bangalore is in seismic zone II, according to the seismic zonation map prepared by Bureau of India Standards (IS 1893(Part I) 2002). But recent studies by Sitharam and Anbazhagan (2007) and Sitharam et al. (2006) suggest that the seismicity of Bangalore is higher, and it should be upgraded from seismic zone II to zone III. In addition, due to the rapid rise in population and industrialization, many lakes and other water bodies were filled up to provide additional residential and industrial areas. During an earthquake, these areas are vulnerable to liquefaction. These factors point towards the necessity of an assessment of liquefaction hazard for Bangalore.

3 Seismic hazard and site characterization of Bangalore

The seismic hazard for Bangalore at rock level was estimated using a deterministic method by Sitharam et al. (2006) and Sitharam and Anbazhagan (2007) and using a probabilistic seismic hazard analysis (PSHA) by Anbazhagan et al. (2009). For the evaluation of liquefaction hazard, the peak ground acceleration (PGA) at ground surface was calculated using the attenuation relations presented by RaghuKanth and Iyengar (2007) for the site class presented by Anbazhagan and Sitharam (2008a).

4 Evaluation of liquefaction potential

The methods suggested by Seed and Idriss (1971) and Cetin et al. (2002) for evaluating liquefaction potential do not consider the uncertainty in the earthquake loadings. Kramer and Mayfield (2007) incorporated the probabilistic method suggested by Cetin et al. (2002)

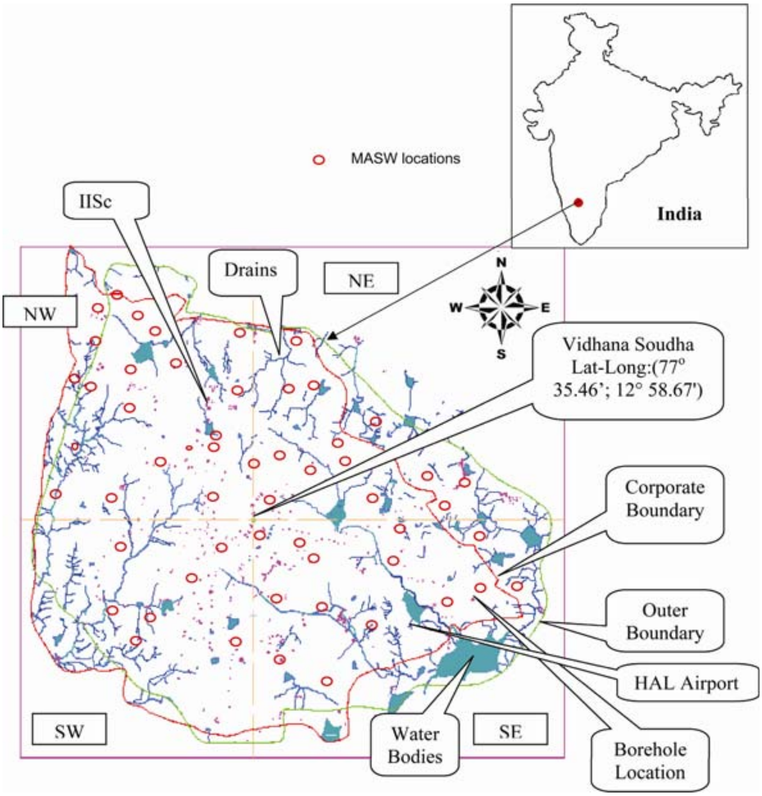

Fig. 1 Location of study area in India along with borehole locations and MASW locations

into a performance-based analysis to evaluate the return period of seismic soil liquefaction. In this approach, the contributions from all magnitudes and all acceleration levels are considered. Thus, the uncertainty in the earthquake loading for the initiation of liquefaction is explicitly included in the analysis. This is achieved by discretizing the seismic hazard "space" into a large number of acceleration and magnitude bins. Thus, instead of taking a single acceleration and earthquake magnitude, as in the conventional approach, it covers the entire acceleration and earthquake magnitude ranges. This method is formulated based on the probabilistic framework by Kramer and Mayfield (2007).

$$\lambda_{\mathrm{EDP}^*} = \sum_{i=1}^{N_{\mathrm{IM}}} P[\mathrm{EDP} > \mathrm{EDP}^* | \mathrm{IM} = \mathrm{im}_i] \Delta\lambda_{\mathrm{im}_i} \quad (1)$$

where EDP, engineering design parameter like factor of safety, etc.; EDP^*, a selected value of EDP; IM, intensity measure, which is used to characterize the earthquake loading like peak ground acceleration, etc.; im_i, the discretized value of IM; λ_{EDP^*}, mean annual rate of exceedance of EDP^*; $\Delta\lambda_{\mathrm{im}_i}$, incremental mean annual rate of exceedance of the discretized value of the intensity measure, IM. The following equation can be derived by considering the EDP as factor of safety and the intensity measure of ground motion as a combination of PGA and magnitude.

$$\Lambda_{\mathrm{FS}_L^*} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P\left[\mathrm{FS}_L < \mathrm{FS}_L^* \middle| a_i, m_j\right] \Delta\lambda_{a_i,m_j} \tag{2}$$

where $\Lambda_{\mathrm{FS}_L^*}$, annual rate at which factor of safety will be less than FS_L^*; FS_L, factor of safety against liquefaction; FS_L^*, targeted value of factor of safety against liquefaction; N_M, number of magnitude increments; N_a, number of peak acceleration increments; $\Delta\lambda_{a_i,m_j}$, incremental annual frequency of exceedance for acceleration a_i and magnitude m_j (this value is obtained from the deaggregated seismic hazard curve with respect to magnitude). The conditional probability in Eq. 2 is (Kramer and Mayfield 2007).

$$P\left[\mathrm{FS}_L < \mathrm{FS}_L^* \middle| a_i, m_j\right] = \Phi\left[-\frac{(N_1)_{60}(1+\theta_1\mathrm{FC}) - \theta_2 \ln\left(\mathrm{CSR}_{\mathrm{eq},i}\mathrm{FS}_L^*\right) - \theta_3 \ln(m_j) - \theta_4 \ln\left(\sigma'_{v0}/P_a\right) + \theta_5\mathrm{FC} + \theta_6}{\sigma_\varepsilon}\right] \tag{3}$$

where Φ, standard normal cumulative distribution; $(N_1)_{60}$, penetration resistance of the soil from the Standard Penetration Test (corrected for energy and overburden pressure); FC, fines content of the soil in percentage; σ'_{v0}, effective overburden pressure; P_a, atmospheric pressure in the same unit as σ'_{v0}; θ_1, θ_6 and σ_ε, model coefficients developed by regression.

$$\mathrm{CSR}_{\mathrm{eq},i} = 0.65 \frac{a_i \sigma_{v0}}{g\,\sigma'_{v0}} r_d \tag{4}$$

$\mathrm{CSR}_{\mathrm{eq},i}$, the CSR value calculated without using the MSF for an acceleration a_i, will be calculated for all the acceleration levels. The most widely used technique to calculate the stress reduction factor (r_d) is suggested by Seed and Idriss (1971). Further, Cetin and Seed (2004) evolved a method to evaluate the stress reduction factor as a function of depth, earthquake magnitude, ground acceleration and the average shear wave velocity of the top 12 m soil column. For a depth less than 20 m the value of r_d is given by:

$$r_d\left(d, M_w, a_{\max}, V_{s,12}^*\right) = \frac{\left[1 + \frac{-23.013 - 2.949 a_{\max} + 0.999 M_w + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341\left(-d + 0.0785 V_{s,12}^* + 7.586\right)}}\right]}{\left[1 + \frac{-23.013 - 2.949 a_{\max} + 0.999 M_w + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341\left(0.0785 V_{s,12}^* + 7.586\right)}}\right]} \pm \sigma_{\varepsilon_{r_d}} \tag{5}$$

where $a_{\max}$ and M_w are the maximum acceleration (in g) and corresponding earthquake moment magnitude values; $V_{s,12}^*$, average shear wave velocity in m/s for the top 12 m soil layer and $\sigma_{\varepsilon_{r_d}}$ is the standard deviation of model error.

Equation 5 is developed for a single earthquake magnitude and acceleration. Since the discretized magnitude (m_j) and acceleration (a_i) ranges are considered for calculation in Eqs. 3 and 4, the above equation for calculating r_d has been modified in this study to account for all the acceleration and magnitude values:

$$r_d\left(d, m_j, a_i, V_{s,12}^*\right) = \frac{\left[1 + \frac{-23.013 - 2.949 a_i + 0.999 m_j + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341\left(-d + 0.0785 V_{s,12}^* + 7.586\right)}}\right]}{\left[1 + \frac{-23.013 - 2.949 a_i + 0.999 m_j + 0.0525 V_{s,12}^*}{16.258 + 0.201 e^{0.341\left(0.0785 V_{s,12}^* + 7.586\right)}}\right]} \tag{6}$$

where a_i and m_j correspond to the discretized acceleration and magnitude values. Based on the shear wave velocity values available for the study area, the value of $V_{s,12}^*$ was calculated as 220 m/s using the following equation.

$$V^*_{s,12} = \frac{12}{\sum \frac{d_i}{V_{s_i}}} \tag{7}$$

where V_{s_i}, shear wave velocity at a depth d_i and $d_i \le 12$ m.

As an alternative to FS_L, liquefaction potential can be characterized by the SPT resistance required to prevent liquefaction; N_{req}, at a given location in the site and at a required depth. The probabilistic method can be applied to get the annual frequency of exceedance for N^*_{req}:

$$\lambda_{N^*_{req}} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P\left[N_{req} > N^*_{req} \middle| a_i, m_j\right] \Delta\lambda_{a_i,m_j} \tag{8}$$

where

$$P\left[N_{req} > N^*_{req} \middle| a_i, m_j\right] = \Phi\left[-\frac{N^*_{req} - \theta_2 \ln(\mathrm{CSR}_{eq,i}) - \theta_3 \ln(m_j) - \theta_4(\ln(\sigma'_{v0}/P_a) + \theta_6}{\sigma_\varepsilon}\right]. \tag{9}$$

The value of N^*_{req} is the corrected N value (for energy, overburden pressure and percentage of fines) required to prevent liquefaction with an annual frequency of exceedance of $\lambda_{N^*_{req}}$.

The N values obtained from the SPT data were corrected to get the standardized $(N_1)_{60}$ value as:

$$(N_1)_{60} = NC_N C_R C_S C_B C_E \tag{10}$$

where N, measured SPT value; C_N, correction for overburden pressure; C_R, correction for short rod length; C_S, correction for sampler; C_B, correction for borehole diameter; C_E, correction for hammer energy efficiency. The overburden pressure correction was done using the equation suggested by Kayen et al. (1992), which yields a maximum value of 1.7 for C_N.

$$C_N = 2.2\big/\left(1.2 + \sigma'_{v0}/P_a\right) \tag{11}$$

The other corrections for N values were applied based on NCEER (1997). The value of $(N_1)_{60}$ further corrected for fines content is (Cetin et al. 2004)

$$(N_1)_{60,C_S} = (N_1)_{60} \times C_{\mathrm{FINES}} \tag{12}$$

where $C_{\mathrm{FINES}} = (1 + \theta_1 \mathrm{FC}) + \theta_5(\mathrm{FC} / (N_1)_{60})$.

5 Probabilistic evaluation of liquefaction return period for Bangalore

The geotechnical data in the study area were collected from 450 boreholes. The depth of boreholes varied from 2 m to more than 30 m, and the average depth was about 20 m. The locations of these 450 boreholes are distributed over the entire study area as shown in Fig. 1. The borehole data at a particular location include the SPT values with depth, soil properties with depth such as density of soil, percentage fines, grain size distribution, Atterberg limits and the depth of water table. The rock depth in the study area varies from 1 to 33 m, and the details are shown in Fig. 2. The corrected N values were obtained using Eq. 10. A sample calculation for corrections applied to N values is shown in Table 1.

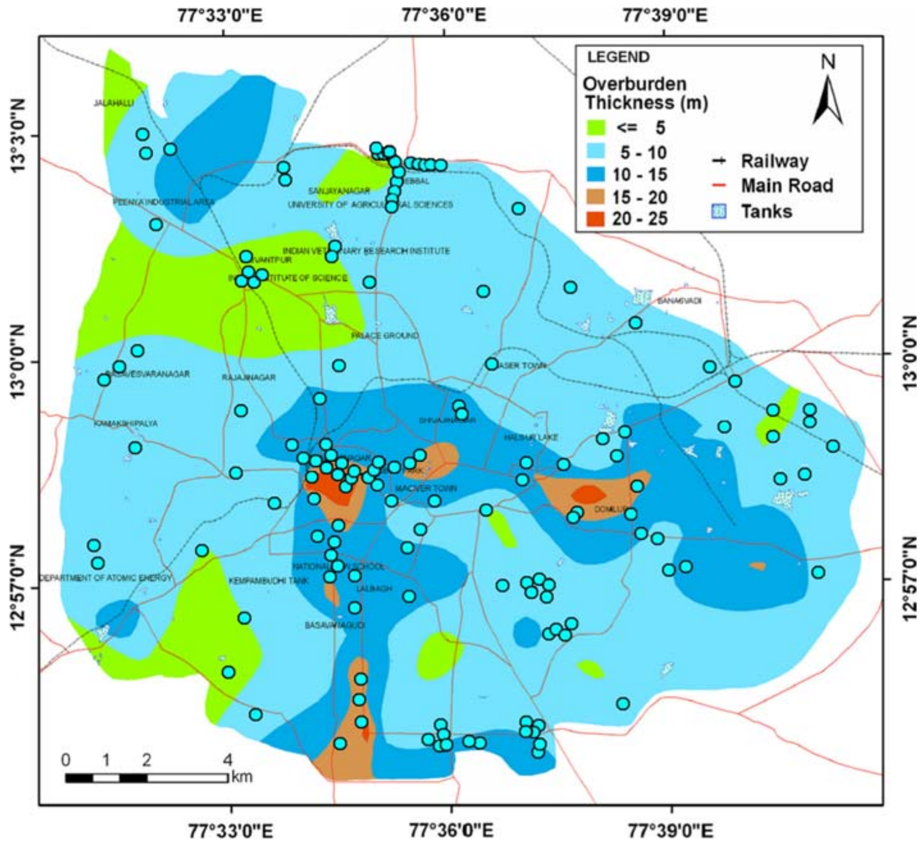

Fig. 2 Variation of rock depth in Bangalore

MASW surveys were carried out at 58 locations in Bangalore and based on equivalent shear wave velocity obtained, the study area was classified as "site class D" (Anbazhagan and Sitharam 2008b). The value of average shear wave velocity in Eq. 6 was evaluated based on the results obtained from the MASW survey. Earthquake data and the details of seismic sources were collected from an area within a radius of 300 km from the boundary of the study area. The seismicity parameters were evaluated using the Gutenberg–Richter method (Gutenberg and Richter 1944). The six seismic sources in the study area, which were identified by Anbazhagan et al. (2009), were used in the seismic hazard analysis. Probabilistic seismic hazard analysis (PSHA) was initially developed by Cornell (1968). Many authors have adopted this methodology for evaluating the seismic hazard, and recently this method has been adopted by Iyengar and Ghosh (2004), RaghuKanth and Iyengar (2006), Anbazhagan et al. (2009) and Vipin et al. (2009) for the probabilistic seismic hazard analysis of Delhi, Mumbai, Bangalore and south India, respectively. For evaluating the seismic hazard using probabilistic method, the magnitude recurrence rate (Cornell and Van Marke 1969), probability of hypocentral distance (Kiureghian and Ang 1977) and attenuation of seismic waves (Raghukanth and Iyengar 2007) were considered. For calculating the PGA at each point, all the seismic sources within a radius of 300 km were considered, and the magnitude and hypocentral distance ranges were deaggregated

Table 1 Various corrections for "*N*" value (depth of water table = 1.4 m; density = 20.0 kN/m^3; hammer effect, $C_E = 0.7$; sample method, $C_s = 1.0$; borehole diameter, $C_B = 1.05$)

Depth (m)	*N* value	TS (kN/m^2)	ES (kN/m^2)	C_N	Correction for rod length, C_R	FC (%)	$(N_1)_{60}$	$(N_1)_{60,C_s}$
1.50	19	30.00	30.00	1.47	0.75	48	15.36	21
3.50	28	70.00	50.38	1.29	0.8	43	21.26	27
4.50	26	90.00	60.57	1.22	0.85	60	19.79	25
6.00	41	120.00	75.86	1.12	0.85	48	28.77	34
7.50	55	150.00	91.14	1.04	0.95	37	40.02	46
9.00	100	180.00	106.43	0.97	0.95	28	67.84	73
10.50	100	210.00	121.71	0.91	1	28	66.90	72
12.50	100	250.00	142.09	0.84	1	28	61.70	67

TS total stress, *ES* effective stress, C_N correction for overburden pressure, $(N_1)_{60}$ '*N*' value corrected for overburden pressure, $(N_1)_{60,C_s}$ corrected '*N*' value, *FC* fines content

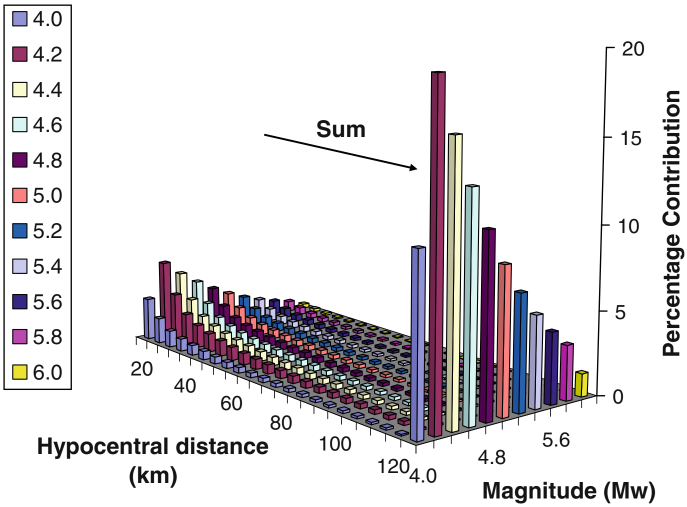

Fig. 3 Magnitude and hypocentral distance deaggregation at 13.00°N and 77.6°E for a return period of 475 years (PGA—0.19 g)

into smaller intervals. For doing the PSHA as well as the liquefaction analysis, we have developed our own computer program, and it was used in the analysis. The PGA values were calculated for site class D using PSHA method. The deaggregated ground acceleration values with respect to magnitude were calculated at all the borehole locations, and these values were used in evaluating the return period of liquefaction. Typical magnitude and hypocentral distance deaggregation at the location corresponding to 13.00°N and 77.6°E for a single source are shown in Fig. 3. The seismic hazard curve deaggregated with respect to magnitude is shown in Fig. 4.

In order to consider the worst scenario for the liquefaction analysis, the water table was assumed at the ground surface. The variation of factor of safety against liquefaction and the annual frequency of exceedance were evaluated using Eq. 2 for depths of 3 and 6 m from ground surface. The entire range of ground acceleration values was divided into very small

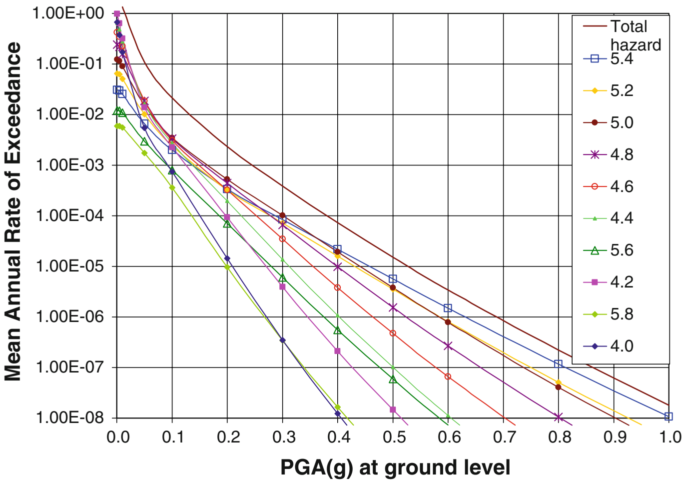

Fig. 4 Deaggregated hazard curve with respect to magnitude at 13.0°N and 77.6°E

intervals at lower acceleration ranges and as the acceleration value increases, the intervals were also increased. Such a division was adopted to account for the variation in annual frequency of exceedance more accurately, because at lower acceleration values the variation of annual frequency of exceedance will be more and at higher acceleration values it will be less.

6 Results and discussions

Curves showing the variation of factor of safety against mean annual rate of exceedance at a depth of 3 m for three different locations in Bangalore are shown in Fig. 5. The main advantage of these curves is that the factor of safety against liquefaction for any given return period can be obtained directly. In a similar way, the curves between the corrected *N*

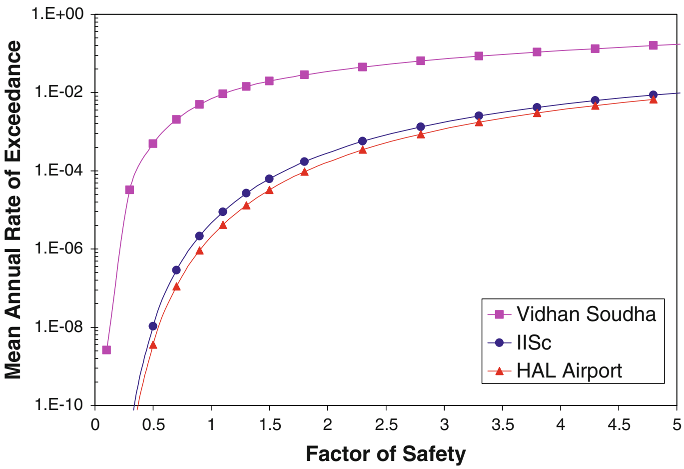

Fig. 5 Factor of safety versus mean annual rate of exceedance at 3 m depth for three locations in Bangalore

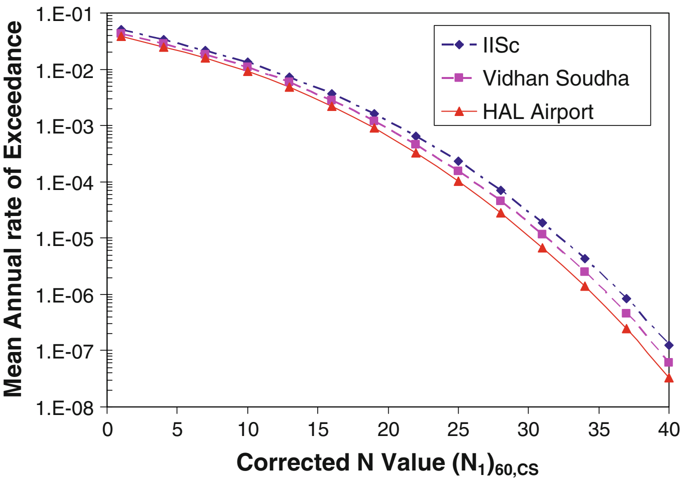

Fig. 6 $(N_1)_{60,C_S}$ versus mean annual rate of exceedance at 3 m depth for three locations in Bangalore

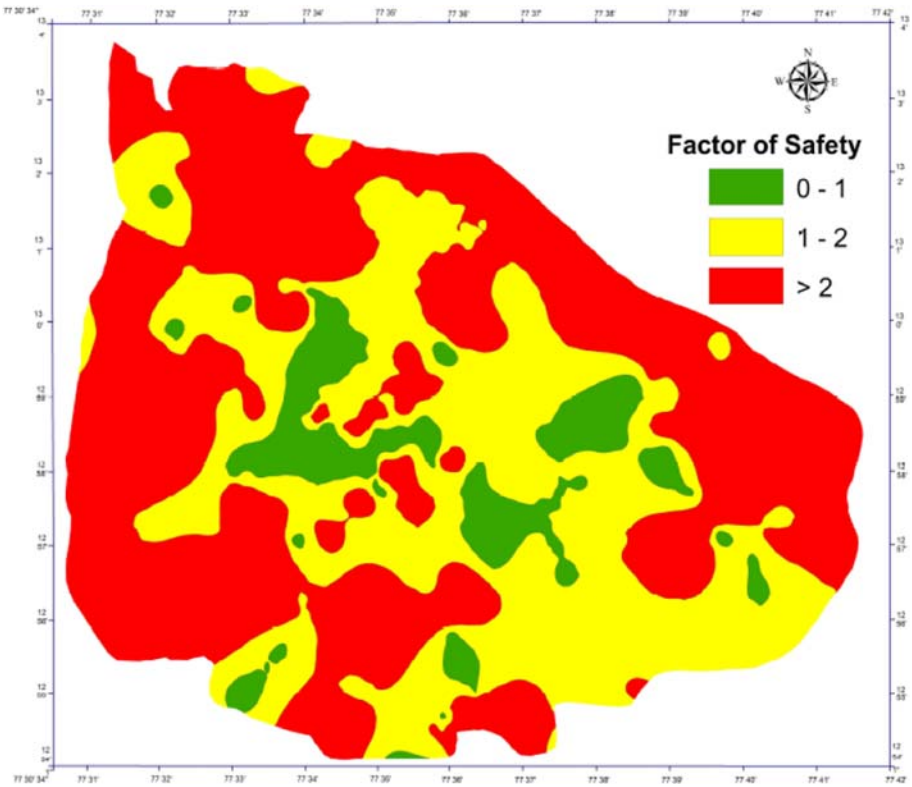

Fig. 7 Factor of safety against liquefaction for a return period of 475 years at 3 m depth

values required to prevent liquefaction and annual frequency of exceedance are presented in Fig. 6. The $(N_1)_{60,C_S}$ required to prevent liquefaction for any specified return period can be obtained from these curves. If the corrected N value (obtained from the site investigation) at the site is less than the value obtained from this curve, then the site is vulnerable to liquefaction for that return period. Similar analysis has been done for the entire study area at depths of 3 and 6 m. The factors of safety against liquefaction and $(N_1)_{60,C_S}$ required to prevent liquefaction for a return period of 475 years at depths of 3 and 6 m are shown in Figs. 7, 8, 9 and 10. The factor of safety range (Figs. 7, 9) of 0–1 indicates that these locations are highly vulnerable to liquefaction, the range of 1–2 is moderately vulnerable, and the factor higher than 2 indicates that these locations are safe against liquefaction. For most of the places, the factor of safety against liquefaction at 6 m depth is higher than that at 3 m depth. This may be attributed to residual nature of the soils and their consolidated state. However, for some locations near 77.58°E and 13.03°N FS_L at 3 m depth, the factor of safety range is greater than 3, but at 6 m depth for some parts of this region, it is in the range of 1–2, and for some other parts it is even less than 1. The $(N_1)_{60,C_S}$ required to prevent liquefaction (Figs. 8, 10) increases slightly with depth, i.e., from 3 to 6 m, due to the increase in overburden pressure and decrease in r_d with depth.

The results obtained from this study were compared with the factor of safety values obtained using deterministic analysis done by Sitharam et al. (2007) (Fig. 11). The FS_L values obtained in the deterministic analysis were generally higher than the values obtained

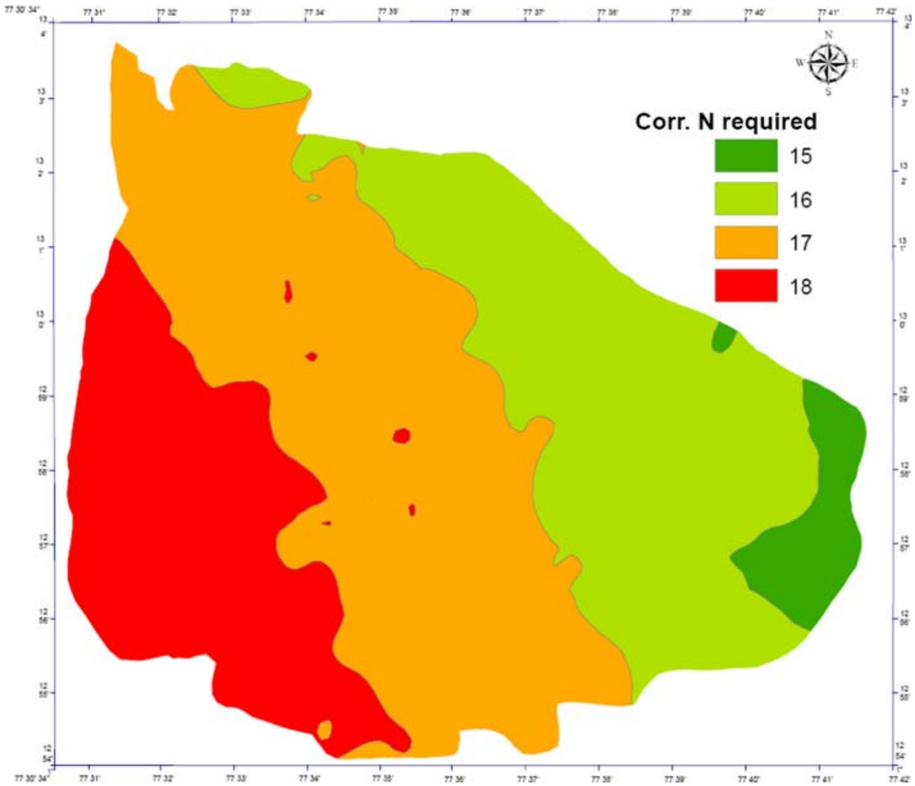

Fig. 8 $(N_1)_{60,C_S}$ required to prevent liquefaction for a return period of 475 years at 3 m depth

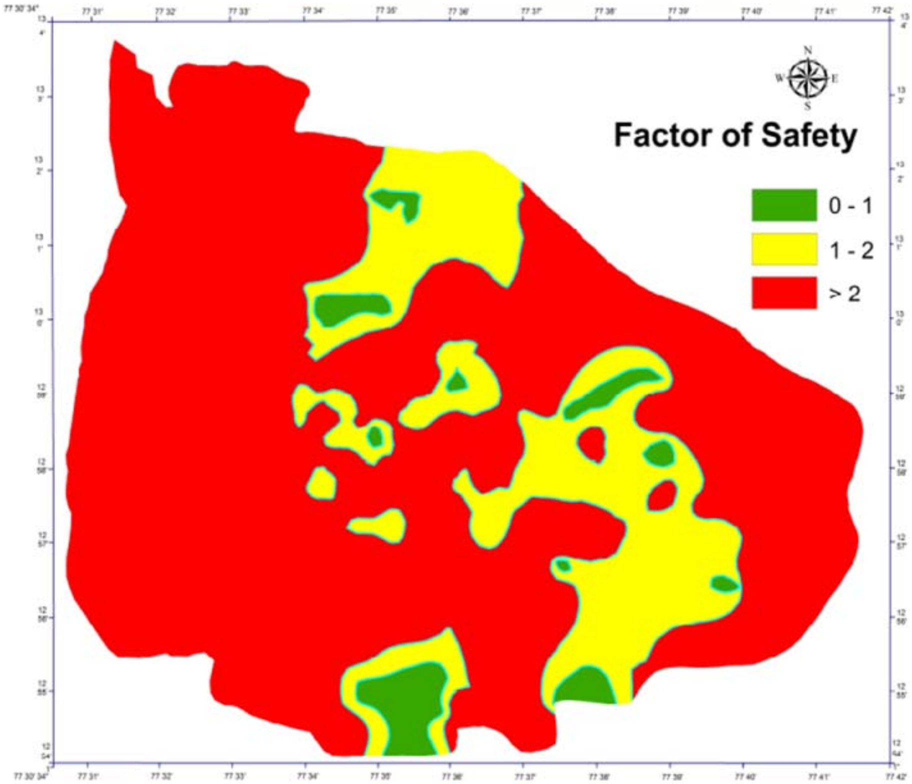

Fig. 9 Factor of safety against liquefaction for a return period of 475 years at 6 m depth

in the present study. However, both methods indicate that some areas at the eastern and central parts of study are vulnerable to liquefaction. The comparison of factor of safety values obtained from both the methods for a few boreholes are given in Table 2.

7 Summary and conclusions

Conventional methods for the evaluation of seismic soil liquefaction potential utilize only a single ground acceleration value and a single earthquake magnitude. In the probabilistic method of evaluating the liquefaction return period, the uncertainties in these two earthquake loading parameters are explicitly taken into account. This paper presents an integrated probabilistic approach for evaluating seismic hazard and the liquefaction return period. The liquefaction potential of Bangalore urban centre was evaluated using this method after extensive field study to collect the geotechnical data. From this study, the summary of observations and conclusions is as follows:

1. Uncertainty in earthquake loading was well accounted in this method by considering all the combinations of accelerations and magnitudes for the evaluation of liquefaction potential.
2. The results from this study were compared with the deterministic liquefaction potential evaluation done by Sitharam et al. (2007) and on a broad scale the spatial variation of the factor of safety against liquefaction potential matches well.

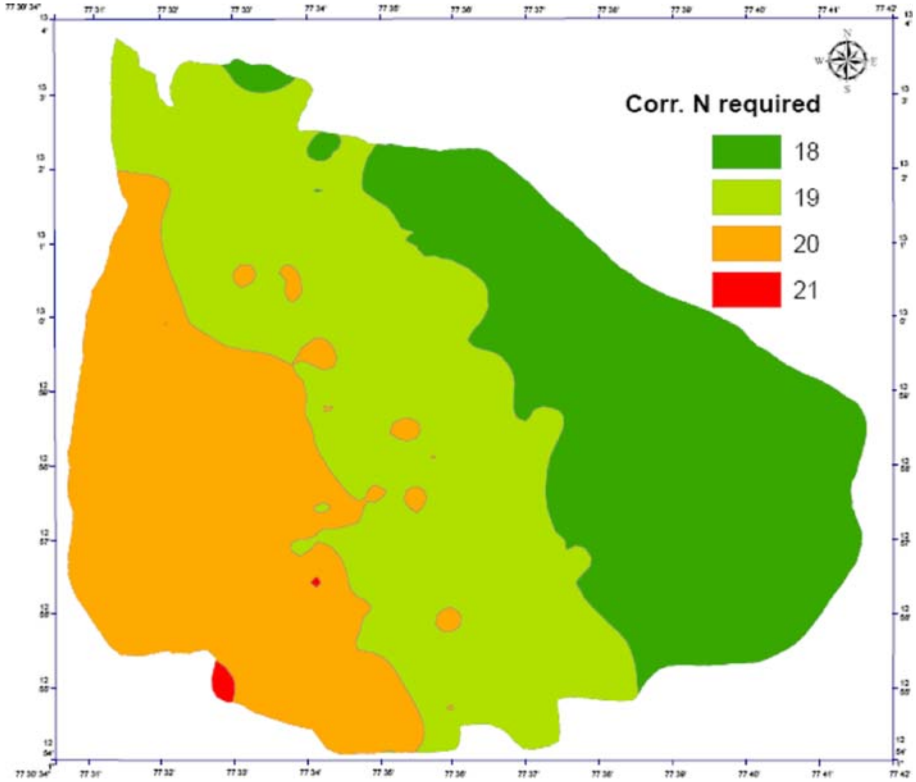

Fig. 10 $(N_1)_{60,C_S}$ required to prevent liquefaction for a return period of 475 years at 6 m depth

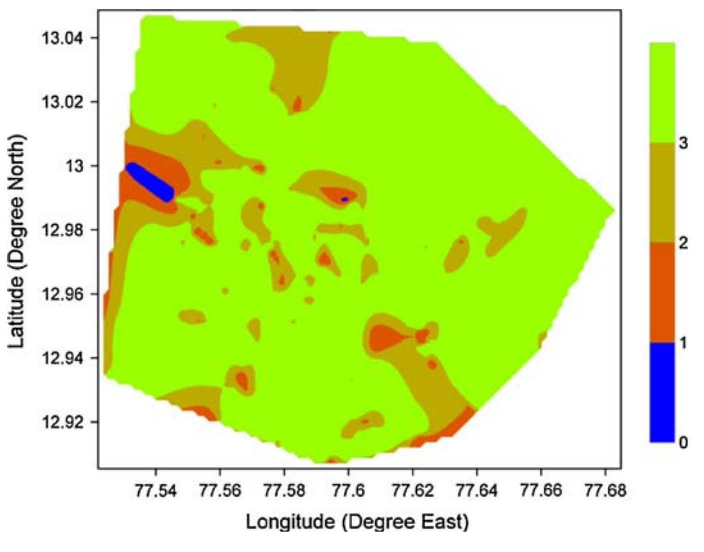

Fig. 11 Factor of safety against liquefaction using deterministic approach (Sitharam et al. 2007)

Table 2 Comparison of FS_L values obtained from this study with the deterministic analysis by Sitharam et al. (2007)

Bh. number	Longitude	Latitude	Factor of safety against liquefaction	
			Deterministic approach	Present study (depth 3 m; return period—475 years)
402-3	77.609	12.945	1.2	0.8
BHL 1017-6	77.585	13.017	1.58	1.39
BHL 104-6	77.548	13.023	3.5	2.39
BHL 187-2	77.624	12.915	2.15	1.9
BHL 217-1	77.601	12.932	2.7	0.72
BHL 389-1	77.609	12.944	0.82	1.15
BHL 610-2	77.587	12.959	1	0.59
BHL 809-4	77.605	12.987	3.9	1.91
BHL 874-2	77.600	12.921	1.89	1.12
BHL 965-5	77.600	12.914	1.03	0.74
BHNSL-8	77.577	12.95	2.5	2.93
NS-7	77.583	12.953	3.14	1.42

3. The corrected N value required to prevent liquefaction varies from 15 to 18 at depth of 3 m and 18 to 21 at a depth of 6 m for a return period of 475 years. About 35 and 60% of the study area have a factor of safety against liquefaction greater than 2 at depths of 3 and 6 m, respectively.
4. There are no standard guidelines as to what should be the minimum liquefaction return period to be specified for different types of sites or buildings. Hence, more research work has to be done in this regard to come up with standardized return periods for different types of sites with respect to their importance.
5. The maps presented in this study can be used by town planners for delineation of areas vulnerable to liquefaction. Those locations, where the factor of safety against liquefaction is greater than 2, can be considered safe against liquefaction for the given return period and do not require any detailed liquefaction analysis. However, for locations with a factor of safety less than 2, it is recommended to do site investigation studies. If the corrected SPT values obtained from the site investigation are greater than the SPT values required to prevent liquefaction (Figs. 8, 10), the site can be considered safe against liquefaction. If the corrected SPT values from the site investigation are less than the SPT values required to prevent liquefaction, the site should be considered susceptible to liquefaction, and a study of the possible effects of liquefaction should be initiated.

Acknowledgments Authors thank ISRO-IISc Space Technology Cell, Indian Institute of Science, Bangalore, India for funding the project titled "Assessment of Seismicity of Peninsular India—Using Remote Sensing and GIS" (Ref. No. ISTC/CCE/TGS/195 Dated 7 March 2007).

References

Anbazhagan P, Sitharam TG (2008a) Seismic microzonation of Bangalore. J Earth Syst Sci 117(S2):833–852

Anbazhagan P, Sitharam TG (2008b) Mapping of average shear wave velocity for Bangalore region: a case study. J Environ Eng Geophys 13(2):69–84

Anbazhagan P, Vinod JS, Sitharam TG (2009) Probabilistic seismic hazard analysis for Bangalore. Nat Hazards 8:145–166
Cetin KO, Seed RB (2004) Non linear shear mass participation factor (r_d) for cyclic shear stress ratio evaluation. Soil Dyn Earthq Eng 24(2004):103–113
Cetin KO, Der Kiureghian A, Seed RB (2002) Probabilistic models for the initiation of soil liquefaction. Struct Saf 24:67–82
Cetin KO, Seed RB, Kiureghian DA, Tokimastu K, Harder LF, Kayen RE, Moss RES (2004) Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential. J Geotech Geoenviron Eng 130(12):1314–1340
Cornell CA (1968) Engineering seismic risk analysis. Bull Seismol Soc Am 58:1583–1606
Cornell CA, Van Marke EH (1969) The major influences on seismic risk. Proceedings of the third world conference on earthquake engineering, Santiago, Chile, vol A-1, pp 69–93
Gutenberg B, Richter CF (1944) Frequency of earthquakes in California. Bull Seismol Soc Am 34:185–188
IS 1893(Part I) (2002) Indian standard criteria for earthquake resistant design of structures, part 1—general provisions and buildings. Bureau of Indian Standards, New Delhi
Iyengar RN, Ghosh S (2004) Microzonation of earthquake hazard in greater Delhi area. Curr Sci 87:1193–1202
Kayen RE, Mitchell JK, Seed RB, Lodge A, Nishio S, Coutinho R (1992) Evaluation of SPT, CPT, and shear wave-based methods for liquefaction potential assessment using Loma Prieta data. Proceedings of the 4th Japan–US workshop on earthquake-resistant design of lifeline facilities and countermeasures for soil liquefaction, vol 1, pp 177–204
Kiureghian DA, Ang AHS (1977) A fault-rupture model for seismic risk analysis. Bull Seismol Soc Am 67(4):1173–1194
Kramer S, Mayfield RT (2007) Return period of soil liquefaction. J Geotech Geoenviron Eng 133(7):802–813
National Center for Earthquake Engineering Research (1997) Proceedings of the NCEER workshop on evaluation of liquefaction resistance of soils. In: Youd TL, Idriss IM (eds) Technical Report No. NCEER, 97-022, NCEER, Buffalo, NY
RaghuKanth STG, Iyengar RN (2006) Seismic hazard estimation for Mumbai city. Curr Sci 91(11):1486–1494
RaghuKanth STG, Iyengar RN (2007) Estimation of seismic spectral acceleration in peninsular India. J Earth Syst Sci 116(3):199–214
Seed HB, Idriss IM (1971) Simplified procedure for evaluating soil liquefaction potential. J Soil Mech Found Eng 97:1249–1273
Seed HB, Tokimatsu K, Harder LF, Chung RM (1985) Influence of SPT procedures in soil liquefaction resistance evaluations. J Geotech Eng 111(12):1425–1445
Sitharam TG, Anbazhagan P (2007) Seismic hazard analysis for the Bangalore region. Nat Hazards 40:261–278
Sitharam TG, Anbazhagan P, Ganesha Raj K (2006) Use of remote sensing and seismotectonic parameters for seismic hazard analysis of Bangalore. Nat Hazards Earth Syst Sci 6:927–939
Sitharam TG, Anbazhagan P, Mahesh GU (2007) 3-D subsurface modeling and preliminary liquefaction hazard mapping of Bangalore city using SPT data and GIS. Indian Geotech J 37(3):210–226
Vipin KS, Anbazhagan P, Sitharam TG (2009) Estimation of peak ground acceleration and spectral acceleration for South India with local site effects: probabilistic approach. Nat Hazards Earth Syst Sci 9:865–878
Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, Christian JT, Dobry R, Finn WDL, Harder LF Jr, Hynes ME, Ishihara K, Koester JP, Liao SSC, Marcuson III, Martin GR, Mitchell JK, Moriwaki Y, Power MS, Robertson PK, Seed RB, Stokoe KH (2001) Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. J Geotech Geoenviron Eng 127:817–833