Liquefaction Hazard Mapping of Lucknow:
A Part of Indo-Gangetic Basin (IGB)

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ABSTRACT

Even though the size of the earthquake is moderate, presence of soft soil near the surface can cause devastating damage due to local site and induced effects like liquefaction. Evidence of liquefaction due to past Indian earthquakes was highlighted in many Paleo-seismic studies, particularly in the Himalayan region. The objective of this paper is subsurface characterization of part Indo-Gangetic Basin (IGB) and estimation of liquefaction hazards for the possible surface ground motions based on the region seismic study. Drilling of boreholes and measurement of standard penetration N values are carried out at selected locations for subsurface characterization. Possibility of liquefaction for soil deposits are assessed by comparing the grain size distribution curves obtained from laboratory tests with the range of grain size distribution curves of potentially liquefiable soils. The minimum factor of safety values has been identified for each location and presented in the form of maps showing FOS against liquefaction for average and maximum amplified peak ground acceleration (PGA) values. These maps have highlighted that the northern, western and central parts of Lucknow fall under very critical to critical for liquefaction while southern parts shows moderate to low critical area.

Keywords: Cyclic Resistance Ratio (CRR), Cyclic Stress Ratio (CSR), Factor of Safety (FOS), Ground motion, Indo-Gangetic Basin (IGB), Liquefaction, Liquefaction Potential Index (LPI), Soil, Subsurface

INTRODUCTION

Ground shaking induced during an earthquake causes large amount of vibrations for buildings and for the subsoil as well. Subsoil shaking results in differential settlement of the building foundations, sinking of pavements, railway lines etc. Failure of soil during an earthquake usually happens due to loss in the *in-situ* shear strength of soil. This phenomenon is termed as Liquefaction where the shear resistance of the soil is reduced significantly in comparison to shear stresses induced by an earthquake. This reduction in shear strength causes the soil to behave

almost like a liquid. Further, such subsoil cannot withstand any overcoming load on it resulting in tremendous settlements, failures of foundation of buildings and bridge abutments, slope failures etc. The terminology of liquefaction came into existence after the occurrence of the Good Friday earthquake of 1964 with Mw (moment magnitude) of 9.2 in Alaska followed by Nigata earthquake with Mw of 7.5 in Japan. These two earthquakes caused failure of slopes, sinking of bridge piers, tilting of houses, embankments, foundations, pavements and exposure of the buried structures. After 1964, the numerous examples are available where liquefaction had caused massive destruction. These may include the 1971 San Fernando earthquake (Mw-6.6), 1977 Argentina earthquake (Mw-7.4), 1989 Loma Prieta earthquake (Mw-6.9), 1995 Great Hanshin earthquake (Mw-6.8), 1999 Chi-Chi earthquake (Mw- 7.6), 2001 Bhuj earthquake (Mw-7.6), 2004 Niigata-ken Chuetsu earthquake (Mw-6.8) and 2011 Sendia earthquake (Mw-8.9), 2011 Sikkim earthquake (Mw-6.8) and many more. In India, damages due to liquefaction on a large scale were noticed during 26 January 2001 Bhuj earthquake (Mw-7.6). Historically ground failure due to liquefaction was not well reported in India. However, a few case studies on paleoliquefaction show evidence of liquefaction in India in historic times. Sand blows were evident during 1819 Bhuj earthquake and sand dykes at Beltaghat site during 1897 Shillong earthquake (Rajendran & Rajendran, 2001). Paleo-liquefaction studies in Assam also confirm liquefaction failures during Assam earthquake (Sukhija *et al*., 2011). The above case studies are the classical examples where the damages due to liquefaction were reported far away from the epicenter during an earthquake. Such examples clearly highlight the presence of the softer medium at the shallow depth which had caused the scenario more catastrophic even at distant regions. Considering the possible seismic vulnerability of Lucknow, the estimation of liquefaction potential for Lucknow is important for seismic microzonation which has been presented in this chapter. Boreholes are drilled at selected locations in

Lucknow and Standard Penetration Test N (N-SPT) values are measured. These have been used to evaluate the *in-situ* soil cyclic resistance ratio. Detailed seismic hazard analysis and site response study were carried out and the surface PGA are mapped for maximum amplification. The maximum amplified surfaces PGA has been used to estimate the cyclic stress ratio. These values have been used to estimate the factor of safety against liquefaction and also evaluation of the liquefaction potential index for Lucknow region has been attempted.

THE STUDY AREA AND NEED FOR THE STUDY

The Indo-Gangetic Basin (IGB) covers an area about 2, 50, 000 sq km extending between the latitude 24° N to 30° N and longitude 77° E to 88° E. Approximately 200 million live in the IGB which defines the area as one of the most densely populated regions of India. Ganga is the main river of the basin which flows from the Himalayas in the north to the Bay of Bengal in the northeast. The study region of Lucknow city has an area of about 370 km^2 and with latitude 26° 51.6' N and longitude 80°54.6' E is located in the central part of IGB. Figure 1 shows the study area of Lucknow with Himalayan belt and IGB. The elevation difference in the entire study area is about 29 m from its highest elevation of 129 m in the area of the Sarda canal and its lowest elevation of 100 m in the southeastern region of Dilkusha garden. The River Gomati flows from the middle of Lucknow in northwest-southeast (Husainabad-Dilkusha garden). The study area covers most part of River Gomati in the Lucknow city. Regional lateral slopes are toward the River Gomati from north-west towards south-east through the heart of the city. The rise in water level during the monsoon season has brought flooding in and around the River Gomati in the years 1923, 1960, 1971, 1985, and 1998. This had caused the level to rise up to a maximum of 111.5 m in 1960, 110.85 m in 1971 and 106.30 m in 1998. As per Geological Survey of India (DRM, 2001)

Figure 1. Study area of Lucknow with IGB in the map of India (modifed after www.freeworld-maps.net)
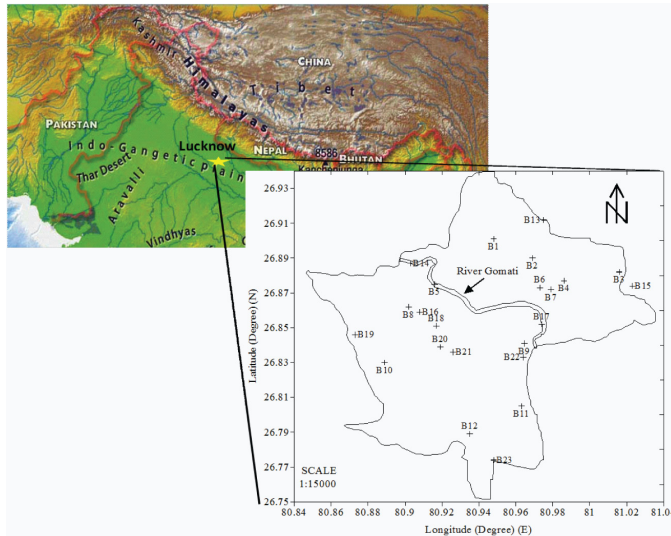
report, the entire area is composed of thick Quaternary sediments uncomfortably overlying the basement of Bundelkhand Granitoids and sedimentary rocks of the Vindhyas. As per the Central Ground Water Board (CGWB), the bedrock is not present till a depth of 298 m and 445 m in southern and western parts of the Lucknow urban center (DRM, 2001). Lucknow lies close to one of the most seismic regions of the world "The Himalayan region". Numerous damaging earthquakes have occurred and still are being observed at different segments of Himalayan belt. Events like 1855 Bihar-Nepal, 1897 Assam, 1905 Kangra, 1934 Bihar-Nepal, 1950 Shillong, 1991 Uttarkashi, 1999 Chamoli, 2005 Kashmir, 2007 Uttarkashi, and 2011 Sikkim are some of the earthquakes which have happened in the Himalayan region and were responsible for moderate to catastrophic damages in the surrounding regions. Important tectonic features present in the Himalayas are the Main Boundary Thrust (MBT), Main Central Thrust (MCT) and Indo-Tsangpo Suture Zone (ITS). Observed experiences from 2001 Bhuj, 2005 Kashmir,

2011 Sikkim earthquakes in the Indian subcontinent have clearly shown the radial extent of damages can extend even beyond even 350 km from the epicenter. Factors which control such damages are the event size, crustal rock properties and the local geology at the damaged site. The Lucknow urban center lies within 350 km radial distance from the central seismic gap. The central seismic gap is the portion of the Himalayan region which exists between 1905 Kangra and 1934 Bihar-Nepal earthquake. This seismic gap has been identified based on stress accumulation and non-occurrence of many great earthquakes in the last 78 years after 1934 Bihar-Nepal earthquake (Khattri, 1987; Rajendran & Rajendran, 2011) in this segment. A probability of 0.52 for the occurrence of a great earthquake in central seismic gap was reported by Khattri (1987). Thus, understanding the liquefaction hazard for Lucknow by considering different seismicity is important and helps to prepare the design of substructures and disaster planning of the region.

STANDARD PENETRATION TEST

The Standard Penetration Test (SPT) is one of the oldest, most popular, and commonly used *in-situ* test used for subsoil exploration in the Soil mechanics and foundation engineering because the equipment and test procedure are simple. The 'Standard Penetration Test' commonly known as the 'SPT', is carried out in a borehole, by driving a standard 'split spoon' sampler using repeated blows of a 63.5 kg hammer falling through 762 mm. The penetration resistance (N) is the number of blows required to drive the split spoon for the last 300 mm of penetration. The penetration resistance during the first 150 mm of penetration is ignored, because the soil is considered to have been disturbed in that depth. SPT bore log data includes the location of wells, SPT test results, ground water level, grain size, Liquid limit, Plastic limit and strength of the soil and rock. N-SPT values are useful for several applications in soil mechanics and earthquake geotechnical engineering in particular for seismic site classification, site response, and liquefaction studies towards seismic microzonation. In most of the cases, site specific response analysis, shear wave velocity, and shear modulus (G_{max}) of layers are estimated using relationships based on the N-SPT values (Anbazhagan *et al*., 2012 a and b).

In this work 12 boreholes up to 30 m were drilled at the selected location of Lucknow with measurement of N-SPT at a regular interval of 1.5 m in each borehole. These test locations were selected such way that the locations were uniformly distributed throughout the study area of Lucknow. In addition 11 boreholes data with N-SPT values were also collected from the existing database of the drilling agency. These 11 boreholes were drilled to a minimum depth of 30 m by the same geotechnical firm engaged to drill 12 boreholes in the study. These drilling locations have been selected such that it should not be close to locations where existing data is available. Figure 1 shows the location of all the boreholes. In total, 23 boreholes have covered the entire city area reasonably well (See Figure

1). These boreholes have been numbered from B1 to B23 and their distribution throughout the Lucknow urban centre can also be seen in Figure 1.

All the boreholes were drilled with a diameter of 150 mm as per IS: 1892 (1974) and N-SPT values are measured regularly at 1.5 m interval as per IS: 2131 (1981). Disturbed and undisturbed samples were collected at possible depths as per IS: 2132 (1986). Data on SPT N values, depth of sample collection and soil type identification, etc. were logged during field testing. Typical bore log in the eastern part of the study area is shown Figure 2. The physical properties were measured in the laboratory using disturbed soil samples as per IS: 1498 (1970) and used for soil classification in this paper. A typical report of soil properties measured from soil samples collected in a borehole is given in Table 1. It can be observed from the Table 1 that the subsoil information is available from surface level to 30 m depth. Surface layer consists of low compressibility clay with *in-situ* density of 1.77 g/cc and moisture content of 29%. Other laboratory test results in the surface layer shows a void ratio of 0.884, plasticity index (PI) of 12 and a specific gravity of 2.60. At deeper depth, a layer of medium compressibility clay underlain by silty sand is found up to a depth of 9.0 m. The *in-situ*-density shows a variation from 1.77 g/cc to 1.86 g/cc. With the increase in density, a decrease in the void ratio from 0.884 to 0.770 can also be observed from Table 1. This layer of sand is further underlain by alternate layers of clay and sand till 30 m. The clay layer is having a thickness of 7.45 m with PI ranging from 12 to 16. The *in-situ* density of this clay layer vary from 2.03 g/cc to 2.07 g/cc while the void ratio ranges from 0.55 to 0.77. The sandy clay which is underlain the clay layer is having a thickness of 9 m. The variation in the in-situ-density between 19.5 m and 28.5 m is from 1.84 g/cc to 1.87 g/cc. Again a layer of 1.5 m thick clay has been encountered at 28.5 m depth as given in Table 1. The void ratio for this layer as obtained from the laboratory testing is 0.809 and the *in-situ* density of 1.88 g/cc has been found. General soil encountered during

Figure 2. Typical borelog with SPT N values for Lucknow

BH No B-3
Ground Water Table 2.5m below Ground Level

Date of commencement 17.2.10
Date of completion 19.2.10

Depth Below GL(m)	Soil Description	Thickness of layer (m)	Legend	Soil Classification	Samples Type	Depth (m)	SPT N Values
0.0					DS		
1.0	SILTY-CLAY	3.0		CL			
2.0						2.3	N=7
3.0							
4.0						3.8	N=10
5.0						5.3	N=14
6.0							
7.0	SILTY-CLAY	9.0		CI		6.8	N=14
8.0						8.3	N=15
9.0							
10.0						9.8	N=16
11.0							
12.0						11.3	N=14
13.0	SILTY-CLAY	1.5		CL		12.8	N=18
14.0						14.3	N=21
15.0							
16.0	SILTY-CLAY	6.0		CI		15.8	N=20
17.0						17.3	N=19
18.0							
19.0						18.8	N=20
20.0	SILTY-CLAY	0.8		CL			
21.0						20.3	N=19
22.0	SILTY-SAND	4.5		SM		21.8	N=22
23.0						23.3	N=23
24.0							
25.0						24.8	N=23
26.0						26.3	N=24
27.0	SILTY-CLAY	5.2		SM-SP		27.8	N=22
28.0							
29.0						29.3	N=26
30.0							

Note
Bore hole Terminated at 30.0 m
DS-Disturbed Sample   R-Rebound

SPT-Standard Penetration Test
UDS- Undisturbed Sample

Table 1. Typical borelog information for Lucknow urban centre

Depth (m)	Sample Type	Atterberg Limit			Soil Classification	Bulk Density (g/cc)	Moisture Content (%)	Void Ratio (e)	Specific Gravity
		Liquid Limit (LL) (%)	Plastic Limit (PL) (%)	Plasticity Index (PI) (%)					
0	DS	33	21	12	CL	1.77	29	0.884	2.60
1.5	UDS	37	22	15	CI	1.77	28.60	0.884	2.60
3.0	UDS	NON-PLASTIC SOIL			SM	1.86	27.80	0.787	2.61
4.5	UDS	NON-PLASTIC SOIL			SM	1.83	26.60	0.787	2.61
6.0	UDS	NON-PLASTIC SOIL			SM	1.87	26.30	0.787	2.63
7.5	UDS	NON-PLASTIC SOIL			SM-SP	1.86	25.80	0.770	2.63
9.0	UDS	NON-PLASTIC SOIL			SM-SP	1.86	25.80	0.770	2.63
10.5	UDS	34	22	12	CL	2.06	21.50	0.770	2.64
12.0	UDS	37	22	15	CI	2.03	21.00	0.553	2.64
13.5	UDS	38	23	15	CI	2.07	20.90	0.553	2.68
15.0	UDS	38	23	15	CI	2.03	1.70	0.567	2.68
16.5	UDS	40	24	16	CI	2.03	22.10	0.567	2.68
18.0	UDS	36	22	14	CI	2.04	22.10	0.614	2.65
19.5	UDS	NON-PLASTIC SOIL			SM	1.86	27.00	0.614	2.65
21.0	UDS	NON-PLASTIC SOIL			SM	1.84	25.80	0.815	2.65
22.5	UDS	NON-PLASTIC SOIL			SM	1.84	25.80	0.815	2.66
24.0	UDS	NON-PLASTIC SOIL			SM-SP	1.85	25.80	0.815	2.66
25.5	UDS	NON-PLASTIC SOIL			SM	1.87	24.90	0.809	2.66
27.0	UDS	NON-PLASTIC SOIL			SM	1.88	26.10	0.809	2.66
28.50	UDS	NON-PLASTIC SOIL			SM	1.88	26.10	0.809	2.66
30.0	UDS	29	20	9	CL	1.88	26.10	0.809	2.66

boring is silty sand to clay of low compressibility. Study of bore logs shows that measured N-SPT values vary from 3 to 50 in the study up to a depth of 30 m. Whenever N-SPT value exceeds 50 for 300 mm penetration, it is treated as a refusal (R) and further N-SPT values were not measured for that depth as recommended in IS 2131 (1981). SPT test is crude and depends on many factors, the variations in procedure when performing the test and the equipments used in the test can alternate N-SPT values for the same site. In order to account the variation, correction factors are suggested. Various factors include the drilling methods, type of drill rods used, borehole sizes and stabilization methods adopted to keep the borehole stable, sampler used, adopted blow count rate, hammer configuration, hammer energy, fines content, and testing procedures (Schmertmann & Palacios, 1979; Kovacs *et al*., 1981; Farrar *et al*., 1998). The combined effect of all of these factors can be accounted by applying correction factors.

SUBSURFACE PROFILING

Boreholes are reasonably well distributed spatially throughout the study area. Limited subsurface lithological details are available for the region. In this study an attempt has been made to develop subsurface soil profiles using soil classification based on laboratory tests. The subsurface profiling has been attempted to understand the subsoil stratification in the study area of Lucknow. The lithology has been profiled through line BB' (Figure 3) and CC' (Figure 4) as marked in Figure 1 using the borehole data. Figure 3 indicates the presence of silty sand and poorly graded sand (SM-SP) in most part of the cross-section. Other portions of the cross-section are occupied by the presence of low to medium compressibility clay (CL-CI). River Gomati comes in between B16 and B14 (see Figure 3). Small traces of low compressibility silt (ML) can also be found under B20. Clay deposit has been encountered below the river course till a depth of 30 m as shown in Figure 3. Since, the actual elevation difference has not been recorded at the site; these cross-sections have been drawn considering the same elevation across Lucknow. Figure 4 shows the cross-section through CC' as per Figure 1. In this section layers of low compressibility clay (CL) have been found as surface layer on both the ends. Middle portions are covered by the presence by silty sand and poorly graded sand (SM-SP) which continues from surface till 30 m or more. Small traces of silts (ML) have been found under B7. River Gomati is occupied by sand sub-strata however the presence of clay can also be seen adjacent to the sandy layer. All the cross-sections have been drawn based on the borehole data. Both from Figure 3 and 4, it can be observed that a major portion of the subsurface has been covered by sand and clay mixed with silt. Cross-section based on deeper boreholes for the Lucknow region as given by Anbazhagan *et al.* (2012b) also shows the presence of clay and sand. Presence of Kankar layer has been encountered after 40 m depth in the deep soil profile. Stratification details obtained from all the three cross-sections have been found comparable with each other in terms of soil type and layer thickness in the respective location.

Figure 3. Lithology cross-section up to 30 m depth considering the drilled boreholes through line B-B' in Figure 1. (B.G.L. - Below Ground Level)
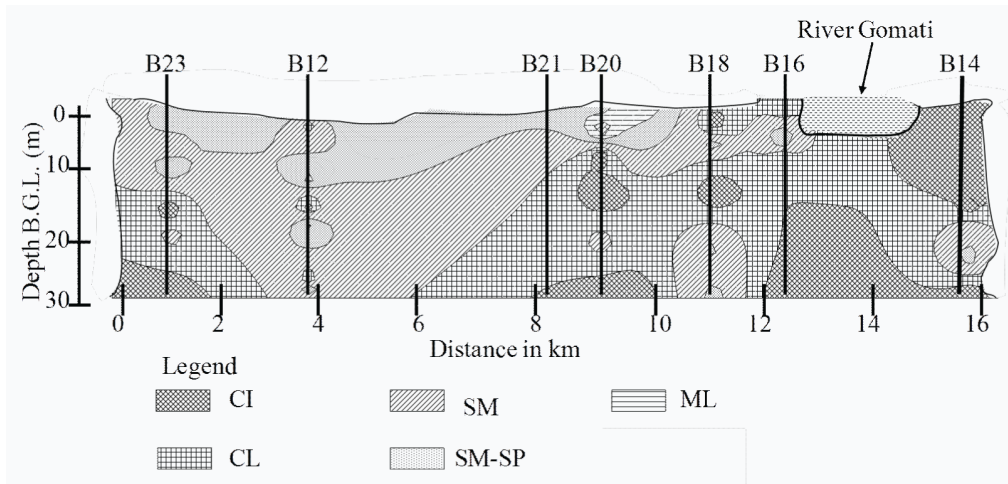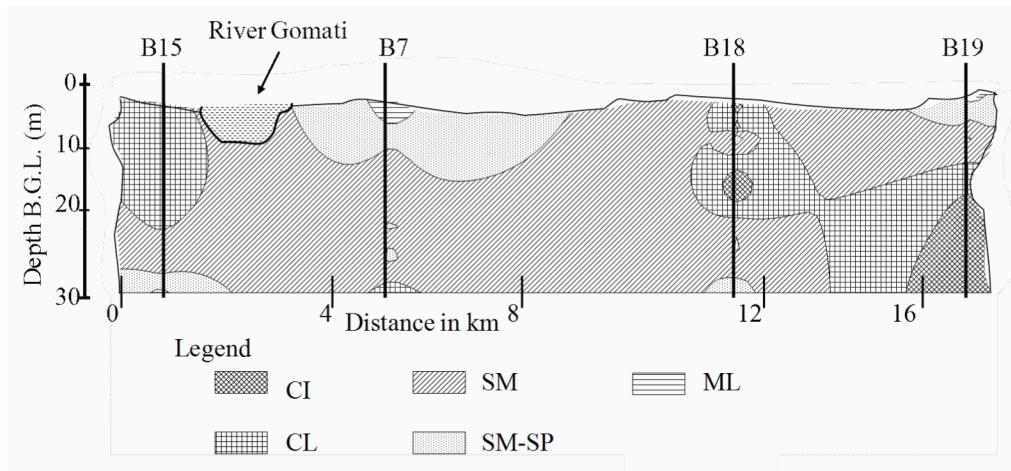
Figure 4. Lithology cross-section up to 30 m depth considering the drilled boreholes through line C-C' in Figure 1. (B.G.L. - Below Ground Level)

LIQUEFACTION POSSIBILITY

In total 23 borehole data till a depth of 30 m were available for liquefaction study. Analysis of these boreholes shows that the bulk density in the range of 1.77g/cc at the ground surface and 2.06g/cc at 30 m depth. Most of the area soft soil with an N-SPT<15 has been noticed up to a depth of 30 m except few locations where N-SPT values exceeding 30 corresponds to dense soil. Recorded N-SPT till 12 m depth suggested the presence of soft soil and thus there are more chances of possible Liquefaction at this location. However, detailed analyses are needed to confirm these observations. Similar conclusions can be made from all the 23 boreholes drilled in different parts of the city. Based on borehole data, mainly two types of soils have been encountered in Lucknow namely, sand (poorly graded to silty sand) and clay (low to medium compressibility) till a depth of 30 m. Also, the alignment of River Gomati shows the presence of sandy soil at shallow depths at many locations. The susceptibility of liquefaction in this work has been verified by comparing the typical grain size distribution curves of soil obtained from boreholes. The comparison of typical grain size distribution curves taken from different parts of the city with the range of gradation curves for potentially liquefiable soils as given by Tsuchida (1970) has been presented in Figure 5. It can be seen that all the particle size distribution curves for Lucknow soil fall within the upper and lower range. This observation confirms that the soils available at Lucknow are liquefiable. Estimation of Factor of safety against liquefaction using a simplified approach of Seed and Idriss (1971) has been carried out in the present work.

Historically the phenomenon of liquefaction was limited to sandy soils only. But there are reports where liquefaction has been reported even in cohesive soils as well (Wang, 1981; Wang, 1984). As per Wang, (1979) cohesive soils which satisfy the given below criteria are also susceptible to liquefaction:

- Clay fraction less than 20%;
- Liquid Limit (LL): 21-35%;
- Plasticity Index (PI): 4-14%;
- Moisture Content: 90% of LL;
- Liquidity Index: ≤ 0.75.

Soil resistance against liquefaction can be determined using laboratory as well as based on *in-situ* field tests. Laboratory tests include Cyclic Triaxial test, Resonant column

Figure 5. Comparison of grain size distribution curve of soils taken from different regions of Lucknow soil with the range of potentially liquefiable soils by Tsuchida (1970)
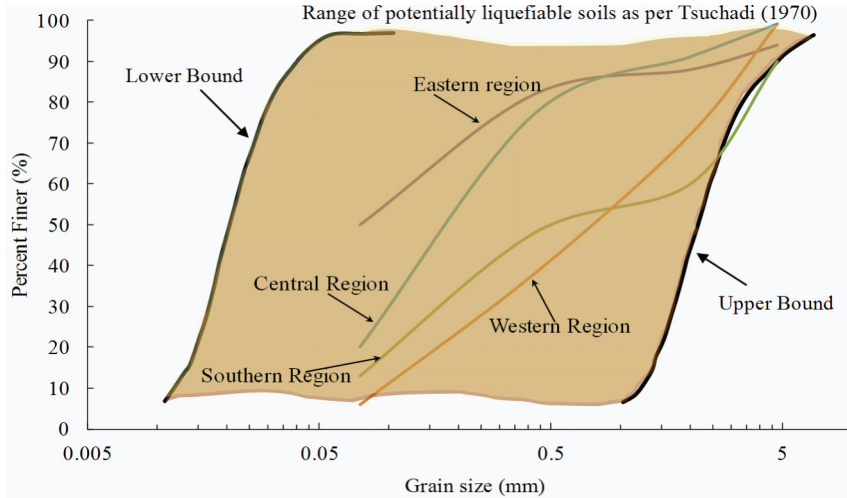
test, Simple shear test etc. *In-situ* tests include Standard Penetration Test (SPT), Cone Penetration Test (CPT), Dilatometer test, Shear Wave Velocity (SWV) measurement by Multichannel Analysis of surface waves (MASW) and Spectral Analysis of Surface Waves (SAWS) etc. SPT is a widely used test for estimating the liquefaction resistance of soil in terms of N-SPT values obtained from a standard test procedure. These field measured N-SPT values need to be corrected for various corrections discussed in the next section before using these for the estimation of liquefaction resistance of soil. Liquefaction susceptibility of *in-situ* soil is usually expressed in different forms such as Factor of Safety against Liquefaction (*FOS*) and Liquefaction Potential Index (*LPI*).

CORRECTIONS TO THE FIELD RECORDED N-SPT VALUES

N-SPT values measured in the field have to be corrected by applying various types of corrections before these values should be used for site response and liquefaction studies. In order to minimize error associated with the testing procedure, corrected N-SPT values are used. The 'N' values measured in the field using Standard penetration test procedure need to be corrected for a) Overburden Pressure (C_N), (b) Hammer energy (C_E), (c) Borehole diameter (C_B), (d) presence or absence of liner (C_S), (e) Rod length (C_R) and (f) Fines content (C_{fines}) (Seed *et al.*, 1983, 1985; Youd *et al.*, 2001; Cetin *et al.*, 2004; Skempton, 1986; Pearce & Baldwin, 2005). A seismic borelog which provides the field recorded N-SPT, values of different types of correction factors and corrected N-SPT values is required (Anbazhagan, 2009). Various types of corrections which have to be applied to the field measured N-SPT has been discussed below.

Correction Due to Overburden Pressure (C_N)

In a semi-empirical approach Seed *et al.* (1975) suggested the normalization of penetration resistance in sand (σ'_{vo}) with one atmospheric pressure "P_a" to apply overburden correction "C_N". Kayen *et al.* (1992) gave the following form of equation for Seed and Idriss (1982) curve for overburden correction:

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a) \tag{1}$$

where, σ'_{vo} the vertical effective stress in kN/m², P_a is the atmospheric pressure of 100 kpa and C_N is the correction due to overburden. The maximum value of C_N from Equation 1 is restricted to 1.7. Thus, if the value of C_N from Equation 1 for any value of effective stress comes more than 1.7, it will be considered as 1.7. For deeper depths where effective stress exceeds 300 kPa, C_N need to be estimated by other means (Youd *et al.*, 2001).

Correction for Hammer Energy (C_E)

This correction accounts for the actual transfer of hammer energy to the split spoon sampler as compared to the theoretical hammer energy applied due to the fall of the hammer on the sampler rod. Energy ratio defined as the ratio of the transferred energy to the theoretical energy, depends upon the type of hammer, lifting and falling mechanism and alignment of the rod. It can be noted here that energy ratio varies from 45% to 90% depends on the type of hammer used in the different countries (Anbazhagan *et al.,* 2012a). Since, the testing procedure, drilling methods and instrumentation can vary in a wide range; the correction factors are given to cover this wide variation of hammers which are commonly used in the SPT test. Youd *et al.* (2001) has recommended correction factors based on the measured energy ratio for frequently used hammers in the field. However hammer energy need to be measured and used to estimate correction factors. Limited attempts have been made in India to measure SPT Ham-

mer energy (Anbazhagan *et al.*, 2012a). In case measurements are difficult to perform, a careful observation about the testing procedure and the setup used should be made in order to reach the value of C_E for a particular purpose (Youd *et al.*, 2001). Since there is no direct measurement of hammer energy in the field, it is assumed 60% energy ratio for donut type hammer used in the testing. For all the calculations, the hammer energy correction of 0.70 as per Robertson and Wride (1998) has been taken in the present work, which is similar to Anbazhagan (2009) study.

Correction for Borehole Diameter (C_B), Rod Length (C_R), Fine Content [$\Delta(N_1)_{60}$] and Liner Correction (C_S)

Similar to hammer energy correction (C_E) which is based on different types of hammer, other corrections are also important. These include correction for borehole diameter (C_B) which is a function of the diameter of borehole drilled to SPT test. More the borehole diameter, lesser will be the confinement and hence less will be the measured N-SPT value. Correction for rod length (C_R) is another correction which depends upon the length of the rod used for the testing. Table 2 shows correction factors for borehole diameter (C_B) as suggested by various researchers. In this work, all the boreholes have been drilled for 150 mm in diameter. Thus, for all the calculations, the value of C_B has been taken as 1.05 as per Table 2. Corrections for rod length (C_R) given by various researchers as presented in Table 3. Based on the depth of SPT test, appropriate value of correction for rod length has been taken as per Table 3. Similarly cor-

Table 2. Factor for borehole diameter (C_B)

Borehole Diameter (mm)	Correction Factor (C_B)		
	Skempton (1986)	Robertsen and Wride (1997)	Bowles (1996)
60-120	1	1	1
150	1.05	1.05	1.05
200	1.15	1.15	1.15

Table 3. Correction factor for rod length (C_R)

Rod Length (m)	Correction Factor (C_R)			
	Skempton (1986)	Robertsen and Wride (1997)	Bowles (1996)	Seed et al. (1984)
Over 10	1	1	1	1
6-10	0.95	0.95	0.95	1
4-6	0.85	0.85	0.85	1
3-4	0.75	0.75	0.75	1
0-3	0.75	-	0.75	0.75

rection for the presence of liner (C_S) has been presented in Table 4. Since, no liner has been used during the field tests, a value of 1.0 as per Bowles (1996) has been selected in this work for all the calculations.

After selection of suitable correction factors for N-SPT measurement in Lucknow, corrected N-SPT $[(N_1)_{60CS}]$ can be obtained using the given below formula:

$$(N_1)_{60} = N * C_B * C_N * C_R * C_S * C_E \qquad (2)$$

where, C_B, C_N, C_R, C_S and C_E are corrections due to borehole diameter, overburden, rod length, liner and hammer energy as discussed above.

These corrections are related to testing equipments used and separate correction need to be applied to soil fine content. Fine contents defined as the percent of dry weight finer than 0.074 mm. Presence of plastic fines can reduce

the skin friction between the soil and the sampler. Thus, the measured N-SPT value will be less than the actual N-SPT value.

Correction for the presence of fine content "$\Delta(N_1)_{60}$" has been proposed by Idriss and Boulanger (2004) for cohesionless soils based on the percent fine content as given below:

$$\Delta(N_1)_{60} = \exp\left[1.63 + \frac{9.7}{FC + 0.001} - \left(\frac{15.7}{FC + 0.001}\right)^2\right]$$
$$(3)$$

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60} \qquad (4)$$

where:

$$(N_1)_{60}$$

is corrected N-SPT without fine content correction:

Table 4. Correction factor for the presence of liner (C_S)

Presence of Liner	Correction Factor (C_S)		
	Skempton (1986)	Robertsen and Wride (1997)	Bowles (1996)
No liner	1.2	1.1-1.3	1
Liner: Dense Sand, clay	1	1	0.9
Liner: Loose Sand	1	1	0.8

$(N_1)_{60CS}$

is corrected N-SPT with fine content correction.

Each correction factor is assigned/calculated for each layer N-SPT measured values and corrected N values have been estimated. A typical calculation of various corrections based on the borehole report has been given in Table 5. The recorded N-SPT values vary from 3 to 36. Laboratory testing suggests the *in-situ* density variation from 17.36 KN/m³ at 1.85 m depth to 19.91 KN/m³ at 16.85 m depth below ground level. In between the ground surface and 16.85 m depth, two layers of denser material have been encountered with densities of 19.13 KN/m³ and 19.72 KN/m³ at 10.85 m and 13.85 m respectively as given in Table 5. Considering the worst condition, the water table has been taken at the ground surface for all the calculations. The estimation of effective stress shows a continuous increment from 13.97 KN/m² at 1.85 m depth to 181.72 KN/m² at 19.85 m depth. Correction factors discussed above are taken into account to determine corrected N-SPT without fine content correction $[(N_1)_{60}]$. The variation in fine content (F.C.) was obtained from the grain size analysis of soil samples in each layer and given in Table 5 column 7. With these values of F.C., correction due to fine content has been determined '$[\Delta(N_1)_{60}]$' have been estimated using Equation 3.

Once all the corrections have been estimated first the corrected N-SPT without the fine content correction has been evaluated using Equation 2. Further, the correction due to F.C. $(\Delta (N_1)_{60})$ has been evaluated using Equation 3 and fine content corrected N values are estimated using Equation 4. The corrected N-SPT with F.C. correction $[(N_1)_{60CS}]$ is the corrected N-SPT which will be used for further calculations. Table 5 column 14 lists all the corrected N-SPT values for the typical borehole. Corrected N values vary from 16 at 1.85 m depth to 22 at 16.85 m depth.

SEISMIC HAZARD ANALYSIS AND SURFACE LEVEL PEAK GROUND ACCELERATION (PGA)

Estimation of induced hazards like liquefaction for future earthquake is possible if ground motion hazard values are predicted precisely. In order to capture the worst scenario of surface PGA for cyclic stress ratio calculation of detailed seismic hazard analysis has been carried out considering deterministic and probabilistic approaches by Anbazhagan *et al.* (2012c). Further, Anbazhagan *et al.* (2012d) carried out detailed nonlinear site response analyses considering the measured shear wave velocity and multiple recorded regional ground motions based on bedrock hazard values. Brief summary of seismic hazard analysis and site response of the study area is presented here and more details can be found in the above work. A new seismotectonic map has been generated for Lucknow considering a radial distance of 350 km around the city centre, which also covers active Himalayan plate boundaries. Past earthquakes within the seismotectonic region have been collected from the United State Geological Survey (USGS), Northern California Earthquake Data Centre (NCEDC), Indian Meteorological Department (IMD), Seismic Atlas of India and its Environs (SEISAT) etc. A total of 1831 events with all the magnitude range were obtained. Collected events were homogenized, declustered and filtered for Mw ≥ 4 events. A total of 496 events were found within the seismotectonic region. Well delineated seismic sources for the study area have been compiled from SEISAT. Superimposing the earthquake catalogue on the source map, a seismotectonic map of Lucknow was generated. A total of 47 faults which have experienced an earthquake magnitude of 4 and above were found which were used for the seismic hazard analysis. Based on the distribution of earthquake events on the

Table 5. Typical calculation for N-SPT correction

Depth (m)	Density (KN/m³)	Effective Stress (KN/m²) (σ'_{vo})	Total Stress (KN/m²) (σ_{vo})	Correction due to Overburden (C_N)	Hammer Energy Correction (C_E)	Fine Content (%) (FC)	Correction due to FC [$\Delta(N_1)_{60}$]	Correction for Borehole Diameter of 150 mm (C_B)	Correction due to Rod Length (C_R)	Correction for the Presence of Liner (C_S)	Measured N-SPT (N)	Corrected N-SPT without FC $(N_1)_{60}$	Corrected N-SPT with FC Correction [$(N_1)_{60CS}$]
1.85	17.85	14.88	33.03	1.63	0.70	89	5.52	1.05	0.75	1.0	12	10.79	16
3.35	18.64	28.13	60.99	1.49	0.70	90	5.51	1.05	0.8	1.0	14	12.23	18
4.85	17.46	39.60	87.18	1.38	0.70	73	5.57	1.05	0.85	1.0	14	12.06	18
6.35	18.64	52.85	115.14	1.27	0.70	80	5.54	1.05	0.95	1.0	15	13.33	19
7.85	18.93	66.53	143.54	1.18	0.70	89	5.52	1.05	0.95	1.0	15	12.35	18
9.35	18.74	79.92	171.65	1.10	0.70	91	5.51	1.05	0.95	1.0	18	13.83	19
10.85	19.13	93.90	200.34	1.03	0.70	85	5.53	1.05	1.0	1.0	18	13.61	19
12.35	19.52	108.47	229.62	0.96	0.70	82	5.54	1.05	1.0	1.0	16	11.32	17
13.85	19.72	123.33	259.20	0.90	0.70	88	5.52	1.05	1.0	1.0	17	11.30	17
15.35	18.74	136.72	287.31	0.86	0.70	71	5.57	1.05	1.0	1.0	14	8.82	14
16.85	19.91	151.88	317.18	0.81	0.70	88	5.52	1.05	1.0	1.0	15	8.92	14
18.35	19.91	167.03	347.05	0.77	0.7	95	5.50	1.05	1.0	1.0	34	19.15	25
19.85	16.97	177.78	372.51	0.74	0.7	49	5.61	1.05	1.0	1.0	30	16.29	22

seismotectonic map, two regions have been identified. Region I which shows high density of seismic events in the area in and around of Main Boundary Thrust (MBT) and Region II which consists of area surrounding Lucknow with a sparse distribution of earthquake events. Data completeness analysis and estimation of seismic parameter "a" and "b" were carried out separately for both the regions. Anbazhagan *et al.* (2012c) study shows that earthquake data is complete for a period of 80 years for both the regions. Using the complete data set, the regional recurrence relations were developed and which gave "b" values of 0.86 for Region I and 0.9 for Region II respectively. Maximum possible earthquake magnitude in each source has been estimated using observed magnitude and doubly truncated Gutenberg-Richter relation. The study area of Lucknow is divided into 0.015° x 0.015° grid size and PGA at each grid was estimated by considering all sources and the three GMPEs. A Matlab code was developed for seismic hazard analysis and maximum PGA value at each grid point was determined and mapped. Deterministic seismic

hazard analysis (DSHA) shows that maximum expected PGA values at bedrock level varies from 0.05 g in the eastern part to 0.13 g in the northern region. Figure 6 shows bedrock level PGA values obtained from DSHA.

The entire PGA variation was divided into 4 classes and a total of 18 bedrock motions were selected as input ground motions for site response study. In the absence of recorded ground motion at the site, regional ground motions were selected by Anbazhagan *et al.* (2012d) in such way that the variation of frequency, duration and amplitude can be covered in the prediction of site response parameters. These motions were recorded during different earthquakes along the Himalayan belt. The deeper borehole reports at Lucknow showed absence of bedrock up to a depth of 150 m (Anbazhagan *et al.,* 2012a). Hence, input ground motions were given at depth where very dense soil layers having shear wave velocity 760±60 m/s were encountered rather than giving input motion at 30 m depth 30 m. Out of 47 SWV profiles obtained from MASW testing (Anbazhagan *et al.,* 2012a) only 29 profiles were found to have very dense

*Figure 6. Deterministic seismic hazard map of Lucknow urban centre*
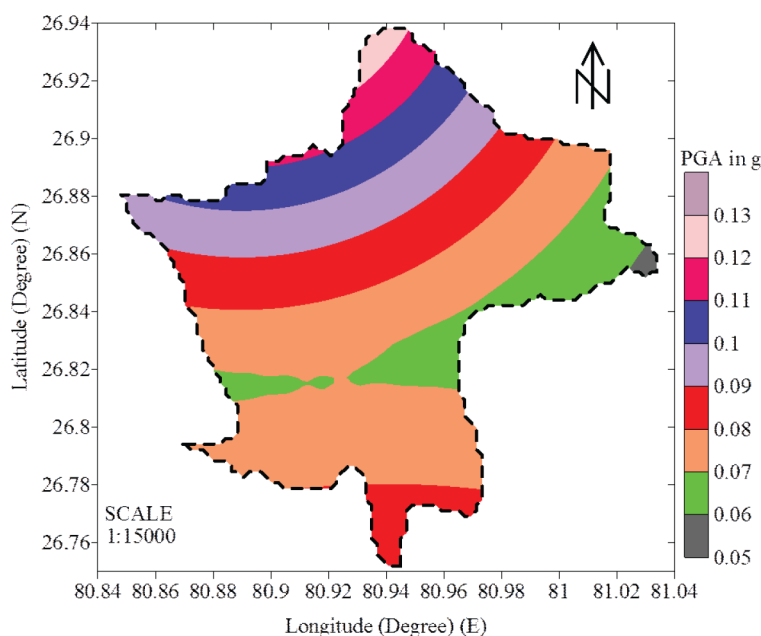
layer, which are used for site response analysis. Nonlinear site response model DEEPSOIL V 3.7 (Hashash *et al.*, 2009) was used for the study. Standard modulus reduction and damping curves were selected based for types of soil layers found in the borehole. The depth of water table and *in-situ* density for each soil column were taken from the nearest borehole. Each soil column was subjected to 18 ground motions and the response of the soil column in terms of peak spectral acceleration, amplification factor and predominant frequency was determined. Site response results from each borehole for 18 input motions were used to estimate the maximum and average amplification values at the respective burial location. Two maps showing the average and maximum amplification factors were prepared. These maps show that the amplification factor varying from 2 to 6 with lesser values in the central and southern parts of Lucknow while higher values were obtained from the northern and western parts (Anbazhagan *et al.*, 2012d). Surface PGA values were estimated considering PGA from the hazard analysis and based on average and maximum amplification factors. Surface PGA maps based on amplification factors show PGA variation from 0.12 g in the southern and eastern parts to 0.72 g in the northern and western parts of

Lucknow (Anbazhagan *et al.*, 2012d). Figure 7a and Figure 7b shows surface PGA obtained considering average and maximum amplification factors considering 29 shear wave velocity profiles. These surface PGA values have been used to estimate the cyclic stress ratio which is presented in the next section.

FACTOR OF SAFETY (FOS) AGAINST LIQUEFACTION

Factor of safety against liquefaction has been evaluated by comparing the external loading and the shear strength of the soil similar to any factor of safety estimation. Factor of Safety (*FOS*) against liquefaction is determined by comparing the shear strength of soil against the shear stresses develop during an earthquake. Since, the phenomena of liquefaction is a cyclic loading problem, all the external stresses and strengths used for assessing liquefaction susceptibilities are related to cyclic loading. Factor of Safety is estimated by comparing the cyclic shear stress due to earthquake loading with the cyclic resistance of the soil from the corrected N-SPT values. In the present work, Factor of Safety (FOS) against liquefaction has been estimated for the Lucknow Urban

Figure 7. (a) Surface PGA map based on average amplification for Lucknow & (b) Surface PGA map based on maximum amplification for Lucknow
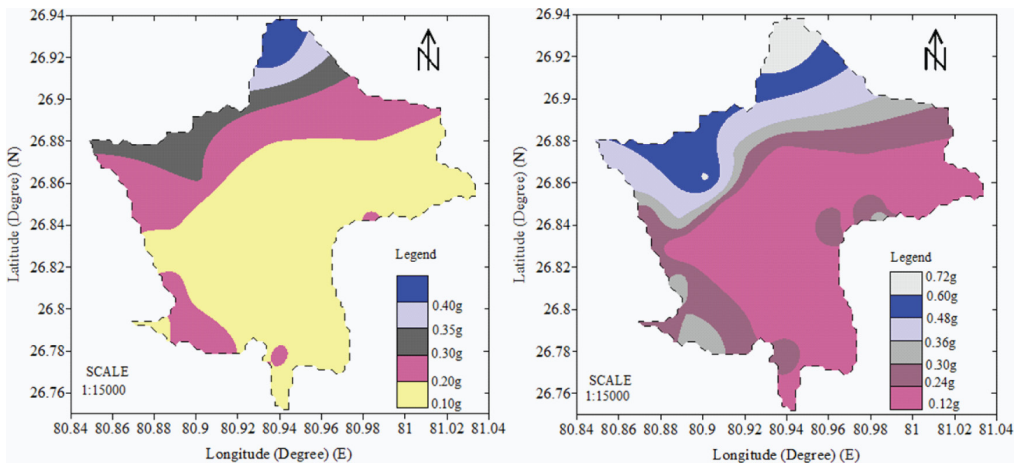
Centre following the simplified Seed and Idriss (1971) procedure. The updated procedure for liquefaction estimation and detailed review of different methods can be found in Idriss and Boulanger (2010) and has been used in this study. Simplified procedure by Seed and Idriss (1971) has been divided into three steps namely; 1) Estimation of Cyclic Stress Ratio (CSR), 2) Estimation of Cyclic Resistance Ratio of soil (CRR), 3) Determination of the FOS with magnitude scaling factor using CSR and CRR. Estimation of each of these three parameters has been presented here.

Cyclic stress Ratio (CSR) is a measurement of the stresses due to external loading of the earthquake. Following equation has been used to estimate the CSR values:

Cyclic Stress Ratio (CSR)

$$= 0.65 \left(\frac{a_{\max}}{g} \right) \left(\frac{\sigma_{vo}}{\sigma_{vo}'} \right) r_d \qquad (5)$$

where, a_{max} is the Peak Ground Acceleration (PGA) at the surface obtained from Site response study in terms of acceleration due to gravity, g is acceleration due to gravity, σ_{vo}' is effective vertical stress, σ_{vo} is the total vertical stress, r_d is the stress reduction coefficient. The value of r_d has been determined using the empirical formula by Liao and Whitman (1986) as given below:

$$r_d = 1.0 - 0.00765z \quad for \ z \le 9.15 \qquad (6)$$

$$r_d = 1.174 - 0.0267z \quad for \ 9.15 < z \le 23 \qquad (7)$$

where, z in Equations 6 and 7 is the depth of interest below ground level in meter. Once the value of r_d is known, using the effective stress and total stress as given in Table 5, the value of CSR as per Equation 5 has been determined using the surface PGA for that location. Corresponding to each borehole location as given in Figure 1, two separate values of a_{max} have been read from Figure 7a and Figure 7b. These a_{max} values are used to calculate two separate CSR values for each borehole location with depth using Equation 5. The initiation of Liquefaction is a function of the amplitude and duration of the earthquake which give rise to the generation of required pore pressure. The cyclic stress approach is based on the assumption that the gain in pore pressure is related to the increase in cyclic shear stress. In order to relate the gain in Cyclic shear stress with that in excess pore pressure, PGA value obtained from irregular earthquake loading has to be converted to an equivalent number of uniform stress cycle. Conversion factor of 0.65 in the above equation is used to estimate the equivalent number of uniform cycles which will produce the same increase in pore pressure as obtained from earthquake loading (Seed & Idriss, 1971). First six columns in Table 6 show an estimation of CSR values for the typical site.

Cyclic Resistance Ratio (CRR) can be estimated as per Seed et al. (1985), Youd et al. (2001), Cetin et al. (2004). Box 1 shows the relation proposed by Boulanger and Idriss (2006) to estimate CRR of soil using the corrected N-SPT value.

CRR is the Cyclic Resistance Ratio, $(N_1)_{60CS}$ is the corrected N-SPT after fine content correction as given in Table 5. The above equation yields the value of CRR for an earthquake magnitude of 7.5. Table 6 column 7 shows

Box 1.

$$CRR_{7.5} = \exp \left\{ \left[\left(\frac{(N_1)_{60CS}}{14.1} \right) + \left(\frac{(N_1)_{60CS}}{126} \right)^2 - \left(\frac{(N_1)_{60CS}}{23.6} \right)^3 + \left(\frac{(N_1)_{60CS}}{25.4} \right)^4 - 2.8 \right] \right\} \qquad (8)$$

Table 6. Typical calculation for the factor of safety against liquefaction for maximum amplification

Depth (m)	$(N_1)_{60CS}$	(σ'_{vo})	(σ_{vo})	(r_d)	CSR	CRR	MSF	FOS	FOS Against Liquefaction for the Borehole
1.85	16	14.88	33.03	0.98	0.43	0.17	1.44	0.59	
3.35	18	28.13	60.99	0.97	0.41	0.18	1.44	0.63	
4.85	18	39.60	87.18	0.96	0.41	0.18	1.44	0.63	
6.35	19	52.85	115.14	0.95	0.40	0.19	1.44	0.69	
7.85	18	66.53	143.54	0.94	0.40	0.18	1.44	0.67	
9.35	19	79.92	171.65	0.92	0.39	0.20	1.44	0.74	
10.85	19	93.90	200.34	0.88	0.37	0.20	1.44	0.77	0.59
12.35	17	108.47	229.62	0.80	0.35	0.17	1.44	0.71	
13.85	17	123.33	259.20	0.76	0.33	0.17	1.44	0.75	
15.35	14	136.72	287.31	0.74	0.31	0.15	1.44	0.70	
16.85	14	151.88	317.18	0.72	0.29	0.15	1.44	0.74	
18.35	25	167.03	347.05	0.68	0.28	0.28	1.44	1.47	
19.85	22	177.78	372.51	0.64	0.26	0.23	1.44	1.27	

CRR values for magnitude of 7.5. Thus, the calculated CRR values from Equation 8 have to be converted for CRR corresponding to the magnitude of interest. Seed and Idriss (1982) proposed a Magnitude Scaling Factor (MSF) for this conversion which was further revised by Ambraseys (1988), Arango (1996), Youd and Noble (1997). In the present work, MSF proposed by Youd *et al.* (2001) has been used as given below:

$$MSF = \left[\frac{10^{2.24}}{M_W^{2.56}} \right] \tag{9}$$

where, M_w represents the magnitude of earthquake obtained from site response study. In the present work, the value of Mw has been taken as 6.8. This value is the magnitude corresponding to ground motion which is responsible for maximum amplification. Anbazhagan *et al* (2012d) highlighted that an amplification of 5.54 was observed due to ground motion 6 in the site response analyses. This ground motion was corresponding to an earthquake of magni-

tude 6.8 (Mw). Hence, this value of Mw has been used in Equation 9 to calculate the value of magnitude scaling factor (MSF).

These CRR values with depth have been used to estimate the Factor of Safety against liquefaction by considering the following equation:

$$FOS = \left[\frac{CRR_{7.5}}{CSR} \right] MSF \tag{10}$$

Where, *FOS* is the Factor of Safety against liquefaction, $CRR_{7.5}$ is the Cyclic Resistance Ratio for a magnitude of 7.5 as obtained from Equation 8 and CSR is the Cyclic Stress Ratio as obtained from Equation 5 and MSF is the magnitude scaling factor as given in Equation 9.

MAPPING OF FACTOR OF SAFETY

The values of CSR and CRR with depth for each borehole have been used to estimate the FOS at different depths. These values are given

in Table 6 column 2 to 4 for each layer of N-SPT values. Based on the depth of each layer, the stress reduction coefficient 'r_d' value has been calculated as per Equation 6 and 7. The values of CSR have been calculated using a_{max} value obtained from surface PGA map based on maximum amplification. Table 6 column 6 shows the CSR variation from 0.43 at 1.85 m depth to 0.40 at 7.85 m depth for maximum PGA of 0.3 g. The estimation of CRR is based on corrected N-SPT values as per Equation 8 given in column 7 Table 6. It can be observed from the Table 6 that with the increase in $(N_1)_{60CS}$ values with depth, there is an increase in the value of CRR from 0.10 at 1.85 m to 0.68 at 19.85 m depth below ground level. The value of MSF has been calculated for Mw of 6.8 as highlighted earlier and given in column 8 Table 6. Once, the value of CSR, CRR and MSF are known, the FOS at various depths can be determined using on Equation 5. Table 6 presented a typical calculation of FOS against liquefaction based maximum amplification for the Lucknow urban center. Table 6 column 9 shows the FOS estimated at various depths. The value of FOS increases from 1.85 m depth till the end of borehole. The FOS against liquefaction for the borehole is taken as the minimum value of FOS among the various values estimated at different depths. Similar exercises have been repeated for all the 23 boreholes to estimate minimum FOS values considering the two PGA values given in Figure 7a and Figure 7b.

The minimum FOS values for all the 23 boreholes have been complied, a map showing the variation of FOS against liquefaction throughout the Lucknow has been generated. Since, two surface PGA maps were generated based on average amplification and maximum amplification, corresponding to each of these PGA values a FOS map has been generated. Figure 8 shows the map of FOS against liquefaction for Lucknow based on average amplification. It can be seen from Figure 8 that the northern area of the city is having the minimum FOS ranging from 0.5 to 0.7 while the central and the western parts are showing the moderate FOS ranging from 0.7 to 1.2. The southern and the

eastern part of Lucknow come under high FOS above 1.5. The alignment of river Gomati has also been shown in Figure 8. It can be observed that the majority of Gomati comes under FOS < 1.2. A small approach of river Gomati near to Lohia path adjacent to Gomati barrage is showing the FOS > 1.5. Further based on the maximum amplification, a map showing the distribution of FOS for Lucknow has been presented in Figure 9. Based on the Figure 9 most of the city area is showing the FOS < 1.2. Observing the alignment of river Gomati similar to Figure 8 indicates that the whole Gomati River is having FOS from 0.3 to 1.2. Also, similar to Figure 8, northern region of the city is showing FOS < 0.7 which indicates that the northern region of Lucknow is very critical to liquefaction. Other regions showing the similar FOS consist of eastern, western and the central part of the city. Figure 8 and 9 shows that the southwestern part and the southern part of the city have FOS > 1.5 which indicate a non-liquefiable area within Lucknow. However, the extreme southern region of Lucknow has FOS less than 1.2.

LIQUEFACTION POTENTIAL INDEX (LPI)

Factor of safety against Liquefaction is a very simple approach, which will only tell if the soil is liquefiable or not. Thus, FOS provides qualitative data only and minimum value are taken to represents site irrespective of the thickness of the respective layers. But this information will not be sufficient to develop liquefaction susceptibility maps and to comment about the liquefaction damage at a site. Iwasaki (1982) proposed Liquefaction Index (LPI) scale which tells the combined liquefaction potential of soil up to 20 m depth considering the subsoil information till 20 m. Iwasaki (1982) suggested that the damages will be more in case the liquefiable layer is thick and shallow with FOS < 1 (Anbazhagan, 2009). Further this classification was modified by Sonmez (2003) considering the threshold FOS as 1.2 rather than 1.0. LPI is

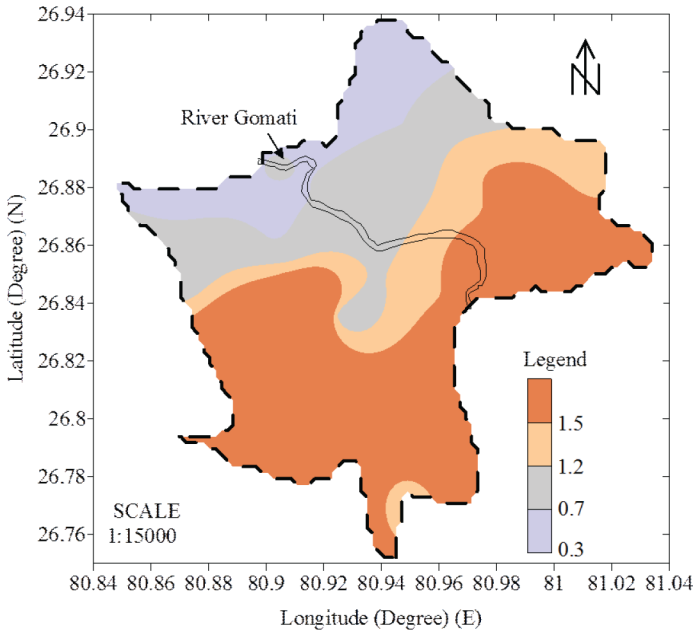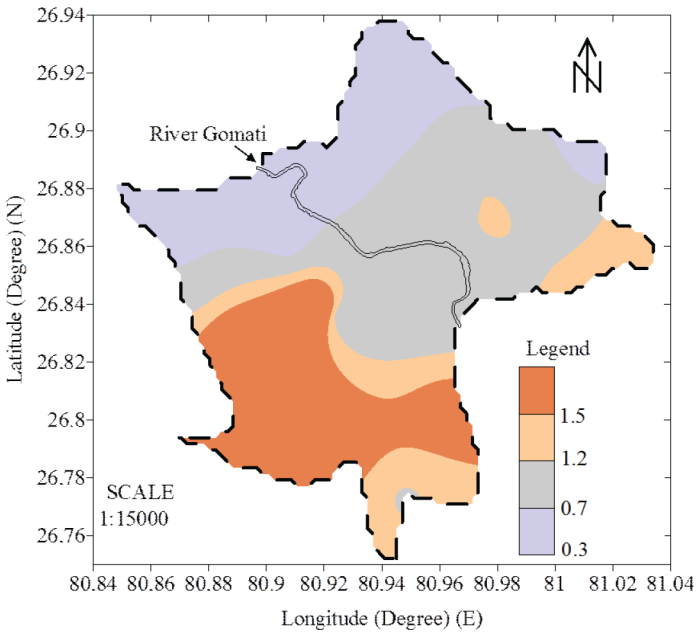*Figure 8. Map showing the FOS against liquefaction for Lucknow based on average amplification*

Figure 9. Map showing the FOS against liquefaction for Lucknow based on maximum amplification

a function of FOS against Liquefaction for each layer as estimated earlier and the depth of soil layer below the ground level. The value of LPI can be estimated using the following formulae (Sonmez, 2003; Sonmez & Gokceoglu, 2005):

$$LPI = \int_{0}^{20} F(z)\ W(z)\ dz \qquad (11)$$

$$F(z) = 0 \text{ for FOS} \geq 1.2 \qquad (12)$$

$$F(z) = 2x10^6\ \exp^{-18.427x\ FOS}\ \text{ for } 1.2 \succ \text{FOS} \succ 0.95 \qquad (13)$$

$$F(z) = 1 - FOS \text{ for FOS} \prec 0.95 \qquad (14)$$

$$W(z) = 10 - 0.5\ z \text{ for } z \leq 20m \qquad (15)$$

$$W(z) = 0\ for\ z \succ 20m \qquad (16)$$

Where, z in the above equations is the depth of the respective layer below ground level, FOS is the FOS as estimated at various depths as presented in the last section.

FOS for each borehole at various depths have been taken from Table 6 and used to estimate LPI values considering Equations 11 to 16 for each borehole. Typical calculation of LPI is given in Table 7 for maximum amplification. It can be observed from Table 7 that the values of FOS are varying from 0.59 to 1.47 and obtained value of LPI is 24.02 for this location. Similar steps have been followed for all the 23 boreholes and the LPI values have been estimated. Similar to FOS maps, two maps showing the LPI distribution for Lucknow based on average amplification and maximum amplification have been generated and are shown in Figures 10 and 11 respectively. As discussed earlier, borehole data are available for 23 locations, thus LPI values have been calculated for these 23 locations. The intermediate values have been interpolated using Kriging interpolation method. Figure 10 shows the distribution of LPI based on average amplification. It can be observed from the Figure 10 that the northern part of the city shows LPI in the range of 5-15 and thus the area can be classified as a high liquefaction potential area as per Sonmez (2003). Major portions of central and western

Table 7. Typical calculation for the estimation of Liquefaction Potential Index (LPI)

Depth (m)	FOS Based on Maximum Amplification	F(z)	W(z)	F(z) x W(z)	LPI for the Borehole
1.85	0.59	0.65	9.075	5.94	
3.35	0.63	0.58	8.325	4.83	
4.85	0.63	0.49	7.575	3.67	
6.35	0.69	0.28	6.825	1.90	
7.85	0.67	0.44	6.075	2.69	
9.35	0.74	0.33	5.325	1.76	
10.85	0.77	0.45	4.575	2.04	24.02
12.35	0.71	0.23	3.825	0.88	
13.85	0.75	0.16	3.075	0.49	
15.35	0.70	0	2.325	0	
16.85	0.74	0	1.575	0	
18.35	1.47	0	0.825	0	
19.85	1.27	0	0.075	0	

parts of Lucknow show LPI from 0 to 5. Based on these values, this area can be classified as a low to moderate liquefaction potential. Since, LPI values less than 0 cannot be drawn, the classes <0 and 0-2 have been approximated with <0.01 and 0.01-2 respectively. The eastern and southern regions of Lucknow as obtained from Figure 10 shows LPI value 0-0.01 which indicate that the regions are non-liquefiable. Observing the LPI values along the alignment of river Gomati in Figure 10 shows that a major portion of Gomati falls in LPI from 2 to 15. This range of LPI indicates that considerable approaches of Gomati in the northern and central parts of Lucknow are under high to very high liquefaction potential. Similar to FOS map as shown in Figure 8, certain length of Gomati in the eastern part of the Lucknow show LPI of 0-2 which indicates low liquefaction potential to non-liquefiable zones in these locations.

Figure 11 shows the LPI variation for Lucknow urban centre based on maximum amplification. It indicates the presence of very high liquefaction potential in the northwestern

parts of Lucknow with LPI ranging from 5-15 and higher values. Entire western parts of Lucknow urban centre show LPI ranging from 2 to 15, which indicates that this region is under moderate to high liquefaction potential as per Sonmez (2003). Similar observations can be made from the eastern part of Lucknow. The central part shows moderate liquefaction potential. The southern part of Lucknow comes under LPI of 0 to 2 which highlights that the southern part of Lucknow is non-liquefiable or has low liquefaction potential. Observing the alignment of river Gomati for liquefaction potential based on maximum amplification suggests that the entire alignment has LPI ranging between 2-15 and higher which indicates that the areas which lie adjacent to the alignment of river Gomati fall under moderate to very high liquefaction potential. A small portion of the River Gomati in the central part of Lucknow indicates LPI < 2 or low liquefaction potential. Borehole report at this location shows the presence of stiff soil in the top 12 m with measured N-SPT value of 30 in the top 5 m.

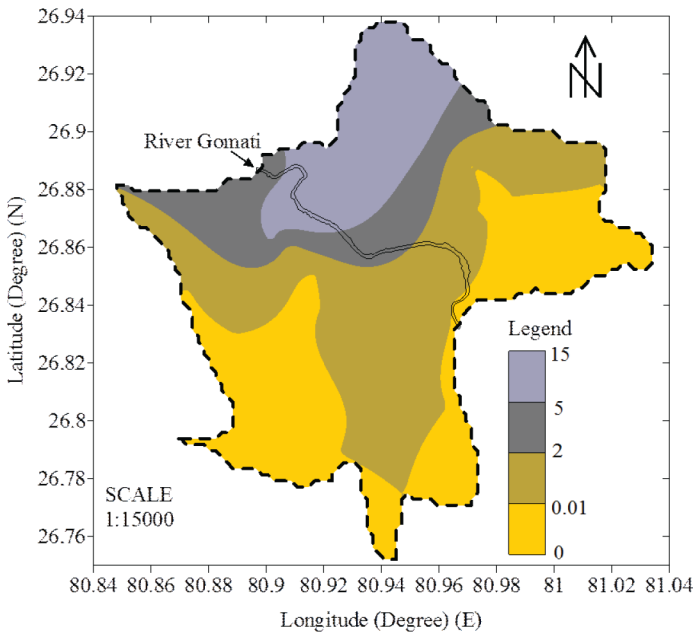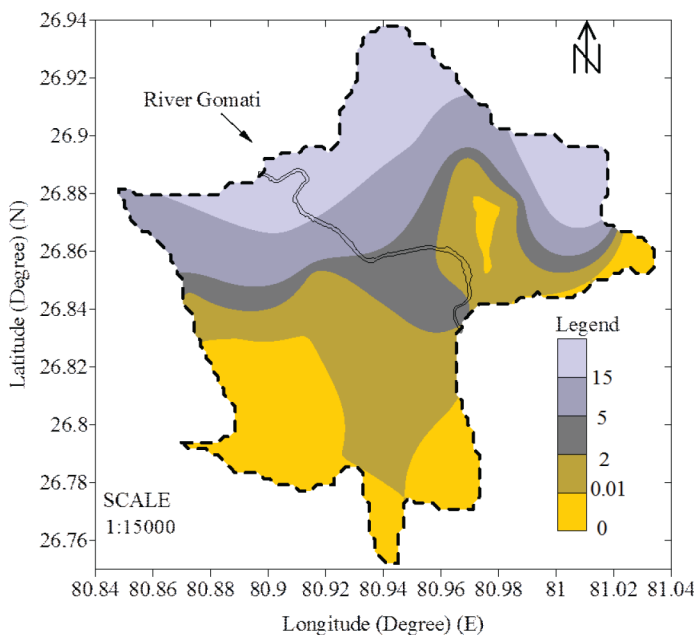*Figure 10. Variation of LPI for Lucknow based on average amplification*

Figure 11. Variation of LPI for Lucknow based on average amplification

RESULTS AND DISCUSSION

In the present work, the liquefaction potential of Lucknow region has been estimated. In total 23 boreholes which were distributed throughout the city have been used for the analysis. Field recorded N-SPT values have been corrected for various corrections applicable to liquefaction analysis. Corrected N-SPT for all the 23 locations have been estimated with depth. Based on the corrected N-SPT, the FOS against liquefaction (FOS) based on simplified Seed and Idriss (1970) approach has been evaluated. Two separate maps showing the FOS based on average amplification and maximum amplification from each borehole have been presented. Observations made from these Figures show that the FOS for the northern area of Lucknow falls in the range of 0.3 to 0.7 which suggest that the northern regions are most susceptible to liquefaction occurrence. All the regions which fall in the eastern part of Lucknow urban centre show low to non-liquefiable regions based on FOS values. A major portion of the River Gomati area comes under very critical to critical.

Small stretch of Gomati in the eastern Lucknow shows low to non-liquefiable zone. The reason may be the presence of barrage in that region. Owing to this construction, the subsoil has got densified in those locations. Borehole reports in this region show N-SPT vary from 9 to 20 indicating the presence of soft to stiff soil in the region. Southern part of Lucknow shows that the area is under low to non-liquefiable regions.

Further, in order to map the liquefaction potential, estimation of LPI as per Sonmez (2003) and Sonmez and Gokceoglu (2005) have been attempted. Similar to FOS, two maps showing the variation of LPI in the Lucknow urban centre have been prepared. Observations from these Figures suggest that the northern regions have high to very high liquefaction potential with LPI > 5. Western part of the Lucknow urban center falls under low to moderate liquefiable potential zones. Similar observations have been made in the eastern part of Lucknow. Some portions in the eastern part show low to non-liquefiable zones. Comparing the LPI values along the alignment of river Gomati highlights that the major portion of river Gomati is under

moderate to very high liquefaction potential. The above studies have highlighted a high potential of Lucknow soil against liquefaction. Overall it can be concluded here that Lucknow which lies beyond 350 km from active Himalayan belt can undergo large scale liquefaction even due to moderate size future earthquakes.

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