SITE AMPLIFICATION AND LIQUEFACTION STUDIES FOR BANGALORE CITY

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ABSTRACT: The objective of this paper is to find the response of soil overburden for Bangalore City for the Maximum Credible Earthquake (MCE) having a magnitude of 5.1 with a peak ground acceleration of 0.153g (Sitharam and Anbazhagan, 2006) and to ascertain its amplification and liquefaction potential. In order to determine the ground response using a one-dimensional approach, several input parameters including soil profile, bedrock level and other geotechnical properties of the subsurface and the design earthquake are required. Using the collected Standard Penetration Test (SPT) data along with the available geotechnical information and synthetically generated ground motion, equivalent linear analysis was performed using the computer program SHAKE-2000. For the selected bore holes, the soil profile fundamental period, peak acceleration, and ground response spectrum at the surface are reported. Liquefaction study was done using simplified Seed and Idriss approach by considering the amplified PGA at the ground surface. The result of the cyclic triaxial test conducted on undisturbed samples from some of the sites confirmed the results of Seed and Idriss approach. This procedure is repeated for other boreholes and an attempt is being made to develop seismic microzonation map for Bangalore City.

1. INTRODUCTION

Southern India once considered as a stable continent has recently experienced many earthquakes indicating that it has become moderately active region. Study of seismic hazard and ground response is essential and has become mandatory for the design of important structures. Predicting site response and liquefaction potential are important step in estimating the effects of earthquakes since local ground conditions substantially affect the characteristics of incoming seismic waves during earthquakes. The authors made an attempt to study the 1D ground response using SHAKE2000 calculated the Liquefaction potential for amplified ground motion cross verified with laboratory testing. The site is located in front of international airport, Bangalore, India. For assessing seismicity of the site the deterministic seismic hazard analysis (DSHA) has been carried out by Sitharam and Anbazhagan (2006) and the synthetic ground motion generated has been used. The study of Sitharam and Anbazhagan (2006) shown that vulnerable source is Mandya-Channapatna-Bangalore lineament which has a maximum credible earthquake (MCE) with a moment magnitude of 5.1. The synthetic ground motion generated for the vulnerable source and dynamic soil properties from SPT bore logs are used for one dimensional ground analysis to study the site response of soil column in the site. The amplification of soil columns, peak horizontal acceleration variations and spectral acceleration both at rock level and ground surface have been studied. Liquefaction study was done using simplified Seed and Idriss approach while considering the amplified PGA at the ground surface.

2. SITE DESCRIPTION

The site is located in south west of Bangalore having a dimension of 122`x190`. It is located at Institute for Aero Space Medicine (IASM) Bangalore, India in front of the Bangalore International airport. The site rock formation is comprised of Gneissic complexes formed before 2700 to 2500 million years, formation identified as Sargur Group of rocks, which is followed by Peninsular Gneissic Complex.

2.1 Geotechnical Investigations

The SPT tests were carried out at five locations in such a way that they are distributed out the construction area and represent the site characteristics. SPT test results shows the general soil profile consists of a variable thickness of soil overburden, which can be classified as filled up soil extending to a depth of 2m to 2.3m in different locations. The field ‘N’ value for the filled up soil layer varies from 8 to 24 at different borehole locations. In the borehole BH-3 to BH-5 clayey sand is present below the filled up soil or at the top itself having a liquid limit of more than 35. Below this layer, a silty sand layer with clay or without clay is present to a depth of 9.0m to 16. The field ‘N’ value for this silty sand layer varies from 19 to 75.

The disintegrated weathered rock exists below the silty sand layer having a refusal strata with N>100. The thickness of the overburden varies from 3.5m to 16.5m from ground level at different borehole locations. Below the disintegrated weathered rock, weathered / hard rock exists (except in BH-5). The core-recovery of the weathered / hard rock samples (except in BH-5) is reported to be more than 75%. The rock formation is classified as granitic gneiss without faults and fissures. Water table in this area during the investigation is at about 1.5m below the ground level in all the boreholes. The weathered rock in BH-2 (south –east corner of the site) is at 3.5m depth when compared to BH-1 (weathered rock is met at 12m) dipping from east to west. However, in BH-3 and BH-4, the weathered rock is met at about 9m and 12m respectively. In BH-5, weathered rock is met at greater than 15m below the ground level. This indicates clearly that the rock is dipping from east to west direction and also in addition dipping from south to north. The field "N" values are corrected by considering different corrections factors are shown in Table1 for BH1. The N values measured in the field using standard penetration test procedure have been corrected for various corrections, such as:(a) Overburden Pressure (CN), (b) Hammer energy (CE), (c) Bore hole diameter (CB), (d) presence or absence of liner (CS), (e) Rod length (CR) and (f) fines content (Cfines) (Seed et al.; 1983, Skempton; 1986, Schmertmann; 1978, Sitharam et. al, 2005).Corrected "N" value i.e., (N60) is obtained using the following equation:

$$N_{60} = N \times (C_N \times C_E \times C_B \times C_S \times C_R \times C_{fines}) \qquad (1)$$

3. GROUND RESPONSE ANALYSIS

For the ground response of soil column for a given input ground motion data can be evaluated using well known 1D ground response analysis software SHAKE 2000. SHAKE2000 is windows based user friendly computer program and is widely used to evaluate site response, amplification and other dynamic parameters considering site-specific soil conditions. SHAKE2000 requires the geotechnical parameter like soil type, thickness of the layer, unit weight of the material, shear modulus value of the material, shear wave velocity of the material and earthquake acceleration file as input data. The common parameters of soil type, thickness of the layer, unit weight of the material have been obtained from geotechnical tests contacted but the shear modulus value of the material; shear wave velocity of the material is separately calculated using inbuilt equations in SHAKE2000 based on SPT "N" Value. SPT "N" values and laboratory results are used to evaluate max shear modulus value of the material. The synthesized ground motion generated by Sitharam and Anbazhagan (2006) has been used as earthquake acceleration file at weathered rock level ("N">100) in each analysis. Sitharam and Anbazhagan (2006) generated the synthetic ground motion using SMSIM- program for simulating ground motions, seismological model by Boore (1983, 2003). The strong motion data simulated for the Maximum Credible earthquake of Mw of 5.1 for a vulnerable source of Mandya-Channapatna-Bangalore lineament having a length of about 100km with hypocenter distance of 15.88km. The synthesized ground motions have PHA of 0.153g with predominant frequency of about 4 Hz at rock level. A transfer function technique is used for 1D ground response analysis. Here the time history of the bedrock (input) motion is in the frequency domain represented as a Fourier series using Fourier transform. Each term in the Fourier series is subsequently multiplied by the Transfer function. The surface (output) motion is then expressed in the time domain using the inverse Fourier transform. However the complex transfer function is only valid for linear behaviour of soils. Therefore this approach has to be modified to account for the non-linearity. The linear approach assumes that shear strength (G) and damping (ξ) are constant. However, the non-linear behavior of soils is well known and can be determined very well in a laboratory environment. Shear strength reduces with shear strain, while damping increases with shear strain. These relationships can be tested and plotted in curves, called shear modulus reduction curve and damping curve, respectively. The problem then reduces to determining the equivalent values consistent with the level of strain induced in each layer. This is achieved by using an iterative procedure on basis of these curves (Idriss and Sun, 1992, Slob et. al, 2002). Shear modulus and damping reduction curves are selected based on the soil properties available form geotechnical data, since the overburden soil for all the bore logs almost has similar properties, the curves proposed by (G/G_{max} - sand, average inbuilt in SHAKE2000) Seed & Idriss 1970 are considered for the silty sand, Similarly for rock material properties matches with the shear modulus and damping curves proposed (G/Gmax – rock inbuilt in SHAKE2000) by Schnabel 1973 have been used. The similar approach has been followed for all bore logs and ground response analyses have been carried out.

3.1 Response study using SPT "N" Value

Observed "N" values is corrected using the equation 1.to obtain N_{60}. The densities of the each bore log are evaluated form the undisturbed sample collected during the field testing. The inbuilt SHAKE2000 option1 to option 5 have been used to give the input parameters to the program, option 6 to option 11 have been used to get output data in a required format. From the "N60" and computed densities for each bore log the max shear modulus has been calculated by equation 2 which as given below:

$$G_{\max} = 325(N_{60})^{0.68} \text{(Imai and Tonouchi, 1982)} \qquad (2)$$

The shear wave velocity is back calculated from the well known equation $G = \rho V_s^2$. Input ground motion has been assigned at bore hole termination level of "N">100/ in weathered rock. The given soil parameters are processed for the assigned input motion to obtain the Peck Acceleration, Acceleration time history, Stress and strain time history, Response spectrum, Amplification spectrum and Fourier Amplitude spectrum. The site response study using geotechnical data shows that the PGA obtained is 0.82g for a given rock motion having PGA of 0.156g, indicating that the site is amplifying. Typical plots of peak acceleration with depth for BH5is shown in Figure 1. The predominant period of soil column of each bore log varies in between 0.04sec to 0.21sec due to variation in overburden thickness. Figure 2 shows a typical response spectrum for BH5. The shape of the response spectrum curve matches with the uniform hazard response spectra shape. Also the spectral acceleration obtained for the site, matches well with the shape of the spectral acceleration coefficient presented in IS1983, (2002). Amplification spectrum gives the amplification ratio which is obtained as the ratio between the Fourier spectrums of the rock to the soil. Amplification ratio has been used to identify the natural period of the soil column/site. Figure 3 shows the amplification ratio is calculated. Results indicate that the frequency amplification ratio for all borehole locations soil column lies in between 10 to 15.

4. LIQUEFACTION STUDIES

The laboratory test results on the undisturbed and representative soil samples clearly indicate that the "SC" material has Liquid limit (LL) close to 35%. As per the modified Chinese criterion (Seed and Idriss 1982), the SC material in the site is not prone to liquefaction. The silty sand layers has shown a very high field N values (N>19) indicating higher resistance to liquefaction. However, in some filled up earth field 'N' values are less than 12. Thus, in this work, liquefaction analyses have been carried out using the simplified Seed and Idriss (1971) approach based on SPT 'N' values and geotechnical test results on the soil samples. The ground level PHA as obtained for each bore log from the SHAKE 2000 amplification study is used here. The necessary magnitude-scaling factor has also been considered. The factor of safety against liquefaction is calculated and a typical one is shown in Table 1. Factor of safety obtained clearly show that the site is not prone to liquefaction. This can be attributed to the presence of % clay content in the soil and its plasticity characteristics. The silty sand layers are also not prone to liquefaction due to higher values of in-situ density. Liquefaction potential of these soils is verified by conducting a series of laboratory tests using constant strain cyclic triaxial tests. The test has been carried out as

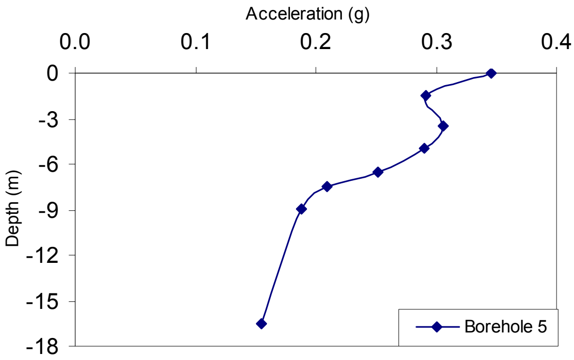

Fig. 1 Typical Acceleration vs. Depth

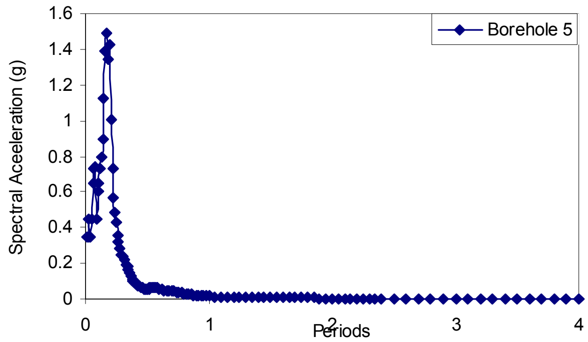

Fig. 2 Typical Spectral Acceleration

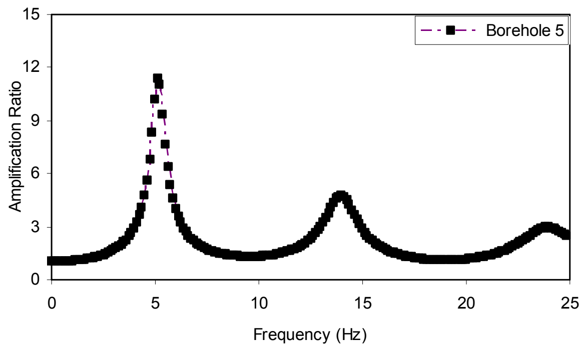

Fig. 3 Typical amplification Ratio

per ASTM: D 3999 (1991). Cyclic triaxial tests are carried out for double amplitude axial strains of 0.5%, 1% and 2%. All the tests are carried out with a frequency of 1Hz and tests are continued for more than 100 cycles on the undisturbed soil samples from bore holes. The results of the cyclic laboratory studies show that the soil samples are not liquefiable. Typical cyclic triaxial test results are presented in Figures 4 and 5. Figure 4 shows the variation of deviatoric stress versus strain plot for more than 120 cycles of loading (Applied single amplitude axial strain = 0.25%; applied confining pressure 100 Kpa, for the undisturbed sample corresponding to borehole 3 at depth

3m below GL, in-situ density of the soil sample 2.0 gm/cc with in-situ moisture content 15%, at 3.0m depth). Figure 5 shows the pore pressure ratio versus number of cycles. From these plots it is clear that even after 120 cycles, the average pore pressure ratio is about 0.94 and deviatoric stress vs. strain plots have not become flat, indicating no liquefaction. The resistance to liquefaction is very high.

5. CONCLUSIONS

The amplification using SHAKE 2000 and liquefaction potential using Seed and Idriss simplified approach a site has been studied using SPT data. High Liquefaction factor of safety obtained using simplified Seed and Idriss approach was confirmed through laboratory cyclic triaxial tests.

ACKNOWLEDGEMENTS

Authors thank Seismology Division, Department of Science and Technology, Government of India for funding the project titled “Microzonation and geotechnical site characterization of greater Bangalore region (Ref no. DST/23(315)/SU/2002 Dated October 2003)”.

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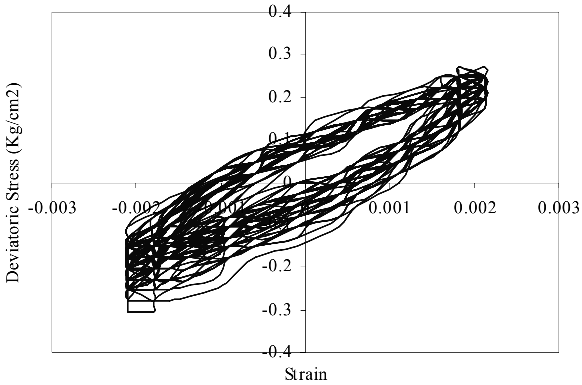

Fig. 4 Typical hysteresis loop from Cyclic Triaxial tests

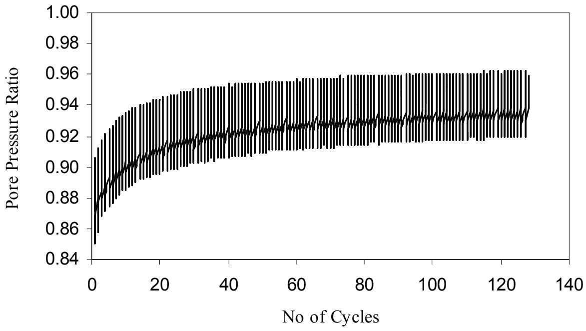

Fig. 5 Typical Pore Pressure Ratio from Cyclic Triaxial test

Table 1. Factor of Safety against Liquefaction

Magnitude, M_w = 5.1 Peak Acceleration,g = 0.6

Depth	Corrected N value	Total stress	Effective stress	r_d	CSR	FC	Liquid Limit	CRR	MSF	FS
(m)		kN/sq.m	kN/sq.m			%	%			
1.50	17	29.55	29.55	0.98	0.38	32	24	0.27	2.68	1.90
3.00	63	59.10	44.39	0.96	0.50	30	28	0.55	2.68	2.98
4.50	79	88.65	73.94	0.93	0.44	20	26	0.55	2.68	3.38
6.00	94	118.20	103.49	0.91	0.41	15	0	0.55	2.68	3.64
7.50	100	147.75	133.04	0.89	0.38	33	0	0.55	2.68	3.84
9.00	90	177.30	162.59	0.87	0.37	35	0	0.55	2.68	4.01