Evaluation of Low Strain Dynamic Properties using Geophysical Method: A Case Study

The paper presents a technique for evaluating low strain dynamic properties of subsurface soil layers for the design of very sensitive equipment of Human Centrifuge foundation. Subsurface material properties are generally measured using conventional geotechnical investigation using drilled borehole, conducting standard penetration tests (SPT) and collecting undisturbed soil samples at different depth. In this paper, measurement of shear wave velocity using Multichannel Analysis of Surface Waves (MASW) is highlighted, which has been used to evaluate low strain dynamic properties of soil such as Young's modulus and shear modulus. Based on this study it is recommended to place the centrifuge foundation at a depth below 2.5 m from original ground level. Further for geotechnical design of the Human Centrifuge foundation, the liquefaction resistance and site response parameter of predominant frequency has been evaluated. From the site response study predominant frequency of site is given as 5 Hz. From these studies, it has been established that the site is safe against liquefaction and local site, if the foundation is placed below 2.5m from the ground level.

Keywords : *Design parameters, SPT, MASW, Frequency and Liquefaction*

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Introduction

Soil properties generally vary with space and time. These soil materials are subjected to a variety of natural processes over space and time and that has tendency to behave differently based on the loading type and pattern. Adequate knowledge of ground conditions is very important for analysis, design and construction of foundations. A detailed site investigation is necessary to characterise the soils at a proposed site for design and construction of safer foundations. Design of geotechnical engineering problems requires the dynamic properties of soils to study soil-structure interaction and settlement analysis. Proper investigation can help to avoid project delay, failures and cost overburden during projects. Several laboratory and field techniques are available to measure the dynamic properties. Most of them are employed in measurements at low-strain level and many are in the large strain levels. However, the choice of a particular technique depends on the specific problem to be solved. This paper presents the evaluation of dynamic properties using geophysical technique of MASW along with site characterisation for site response and liquefaction. The dynamic low strain shear modulus and young's modulus have been evaluated using the measured shear wave velocity from MASW and density from undisturbed soil samples. Liquefaction and site response analysis has been carried out for the proposed site.

Site Description

Site is located in south eastern part of Bangalore Mahanagar Palike (BMP) close to the Institute of Aerospace Medicine of Hindustan Aeronautical Limited having a dimension of 37mx52.7m. The topography of the site is a flat terrain with roads on the northern side and western side. On the south side existing airport staff quarters are present. The proposed facility along with locations of boreholes drilled and geophysical investigation is shown in Figure 1. Geology in this site 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.

Geotechnical Investigations

Five boreholes have been drilled for geotechnical characterisation i.e to identify ground water level, to collect undisturbed soil samples and to measure the SPT- N values. The ground water level has been measured on next day of borehole drilling completion to identify free water level in saturated soil. Undisturbed soil samples are collected at possible depth of 2.5 to 7.5m at an interval of 2m using 100mm diameter and 600mm length cylinder. The 'Standard Penetration Test' (SPT) is carried out in drilled boreholes, by driving a standard 'split spoon' sampler using repeated blows with a 63.5kg hammer falling through 750mm. The bore holes have been drilled using rotary hydraulic drilling of 150mm diameter up to the rock depth. The hammer is dropped on the rod head at the top of the borehole, and the rod head is connected to the split spoon by rods. The split spoon is lowered to the bottom of the hole, and is then driven for a depth of 450mm, and the blows are counted normally for each 150mm of penetration. The penetration resistance (N) is the number of blows required to drive the split spoon for the last 300mm of penetration. The penetration

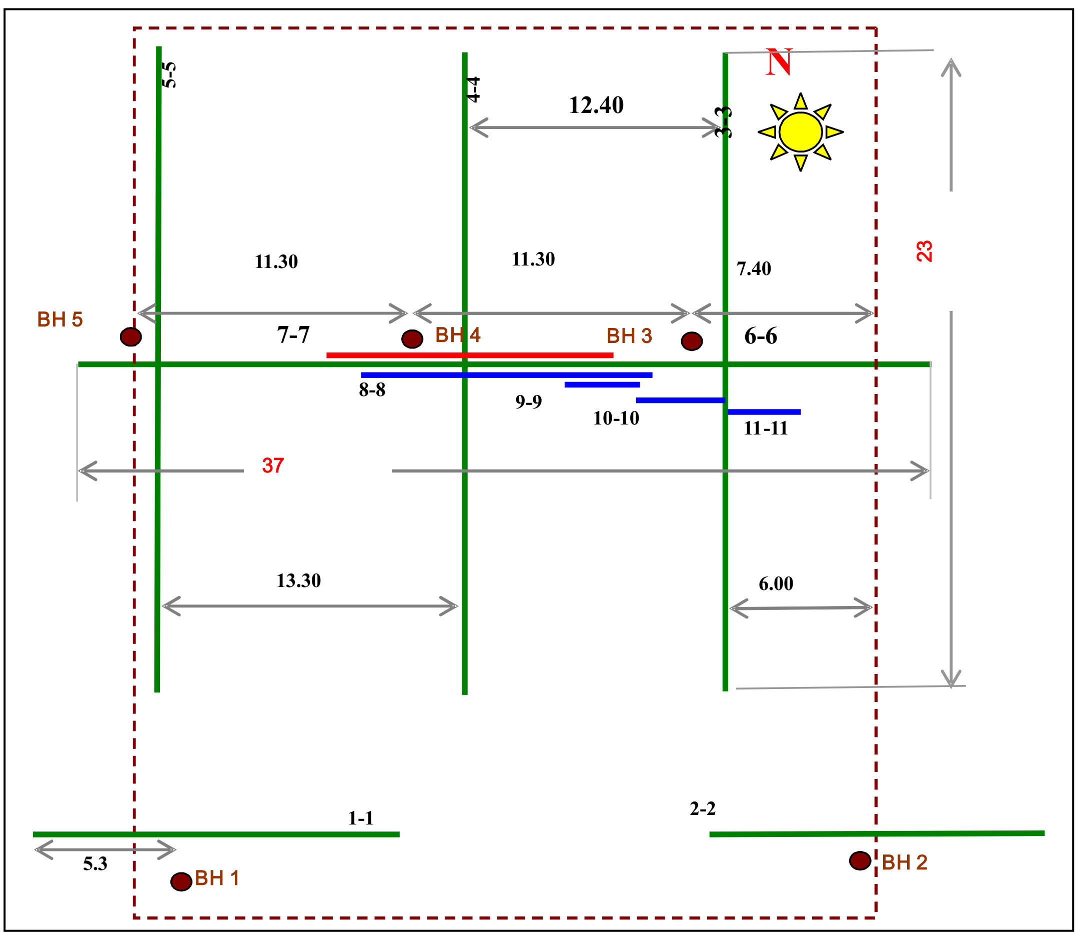

Figure 1 : Site map with marked testing locations

Table 1: comparison of thickness or depth of material layer using SPT and MASW

Location	Thickness/ Bottom of Layer (m)							
	Filled up soil		Soil (sandy silt and silty sand with clay) Overburden		Weathered Rock surface from Ground level		Hard Rock surface from ground level	
	SPT	MASW	SPT	MASW	SPT	MASW	SPT	MASW
BH1 (Line 1-1)	0-2.3	0-3	2.3-12	3-11.6	-12	-11.6	BT at 14.5	36.2
BH2 (Line 2-2)	0-2	0-2.4	2.0-3.5	2.1-4.2	-3.5	-4.2	BT at 7.5	16.8
BH3 (Line 3-3)	0-2.3	0-1.6	2.3-9.0	1.6-8.5	-9	-8.5	BT at 10.5	29.1
BH4 (Line 4-4)	0	0	0-12.0	0-12.5	-12	-12.5	BT at 13.5	NF up to 50
BH5 (Line 5-5)	0-2	0-2.5	2-16.5	2.5-17.0	-16.5	17	BT at 17.0	NF up to 39

SPT - Standard Penetration Test
MASW - Multichannel Analysis of Surface Wave
BH - Bore hole Number
BT - Bore hole Terminated at a depth
NF - Not found up to depth

resistance during the first 150 mm of penetration is ignored. The SPT testing was carried out in all five borehole locations and also with depth in such a way that they are distributed through out the construction area and represent the site characteristics (see Figure 1). Five bore holes of 150mm diameter up to the rock depth has been drilled using rotary hydraulic drilling. The undisturbed soil samples collected has been used to evaluate the index and engineering properties of soil using conventional geotechnical laboratory tests. Borelog obtained from drilled borehole shows that the site has the soil profile consisting of a variable thickness of soil overburden, which can be classified as filled up soil extending to a depth of 2m to 2.3m (See Table 1). The field “N” value for the filled up soil layer varies from 8 to 24. In the borehole BH-3 to BH-5 clayey sand is present below the filled up soil 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 16m. 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 soil overburden varies from 3.5m to 16.5m from ground level and below which the disintegrated weathered rock, weathered / hard rock exists. 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

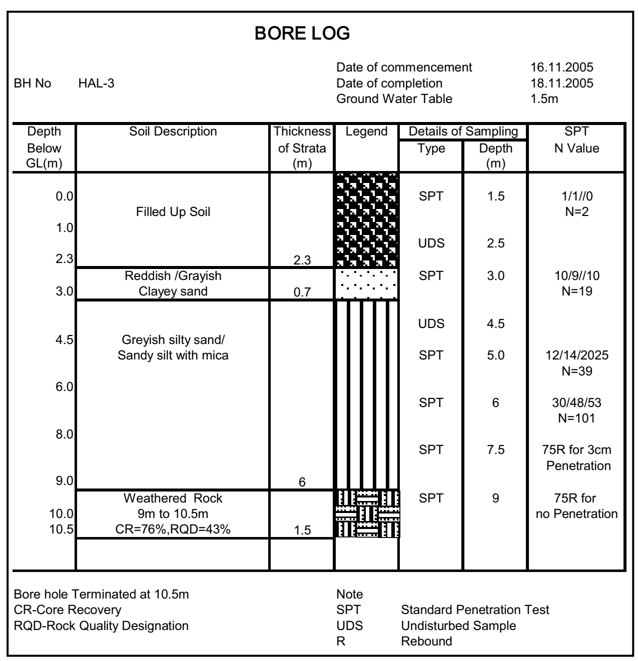

Figure 2 : Typical borelog for the site

1.5m below the ground level in all the boreholes, typical bore log is shown in Figure 2. The 'N' values measured in the field using SPT procedure have been corrected for various corrections recommenced for evaluating the seismic borehole characteristics of soil (Youd et al., 2001; Cetin et al., 2004; and Pearce and Baldwin, 2005). First, corrected 'N' value i.e., $(N_1)_{60}$ are obtained using the following equation :

$$(N_1)_{60} = N \times (C_N \times C_E \times C_B \times C_S \times C_R) \quad (1)$$

Then this corrected 'N' values $(N_1)_{60}$ is further corrected for fines content based on the revised boundary curves derived by Idriss and Boulanger (2004) for cohesionless soils as described below :

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \quad (2)$$

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

FC = percent fines content (percent dry weight finer than 0.074 mm).

A detailed equations and constant with example calculation has presented in Sitharam et al., (2007) and a typical "N" correction calculation table for a site is shown in Table 2.

Mapping of Subsurface using Geophysical method

Geophysical methods are nondestructive techniques that are used to provide information to characterisation site by mapping of subsurface. Recent development in geophysical methods helps to map the subsurface heterogeneity and undulations with their dynamic properties. A number of geophysical methods have been proposed for near-surface characterisation and measurement of shear wave velocity by using a great variety of testing configurations, processing techniques, and inversion algorithms. The most widely-used techniques are SASW (Spectral Analysis of Surface Waves) and MASW (Multichannel Analysis of Surface Waves). The spectral analysis of surface wave (SASW) method has been used for site investigation for several decades (e.g., Nazarian et

Table 2: Typical "N" correction table for borelog

Borehole 4			**T.S**	**E.S**	C_N	**Correction Factors For**				$\boldsymbol{\Delta}(N_1)_{60}$	**Water Table = 1.4 m/ 19-11-2005**		
Depth	Field	Density				Hammer Effect	Bore hole Dia	Rod Length	Sample Method		F.C	$(N_1)_{60}$	Corrected N Value
m	N Value	kN/m^3	kN/m^2	kN/m^2							%		$(N_1)_{60cs}$
1.50	19	20.00	30.00	30.00	1.47	0.7	1.05	0.75	1	15.36	48	5.613	21
3.50	28	20.00	70.00	50.38	1.29	0.7	1.05	0.8	1	21.26	43	5.597	27
4.50	26	20.00	90.00	60.57	1.22	0.7	1.05	0.85	1	19.79	60	5.602	25
6.00	41	20.00	120.00	75.86	1.12	0.7	1.05	0.85	1	28.77	48	5.613	34
7.50	55	20.00	150.00	91.14	1.04	0.7	1.05	0.95	1	40.02	37	5.541	46
9.00	100	20.00	180.00	106.43	0.97	0.7	1.05	0.95	1	67.84	28	5.270	73
10.50	100	20.00	210.00	121.71	0.91	0.7	1.05	1	1	66.90	28	5.270	72
12.50	100	20.00	250.00	142.09	0.84	0.7	1.05	1	1	61.70	28	5.270	67

T.S – Total Stress
E.S – Effective Stress
C_N – Correction for overburden correction
$(N_1)_{60}$ – Corrected 'N' Value before correction for fines content
F.C – Fines content
$\Delta(N_1)_{60}$ – Correction for Fines Content
$\Delta(N_1)_{60cs}$ – Corrected 'N' Value

al., 1983; Al-Hunaidi, 1992; Stokoe et al., 1994; Tokimatsu, 1995; Ganji et al., 1997) and uses the spectral analysis of a surface wave generated by an impulsive source and recorded by a pair of receivers. Evaluating and distinguishing signal from noise with only a pair of receivers by this method is difficult. Thus to improve inherent difficulties, a new technique incorporating multichannel analysis of surface waves using active sources, named as MASW, was developed (Park et al., 1999; Xia et al., 1999; Xu et al., 2006). The MASW has been found to be more efficient method for unraveling the shallow subsurface properties (Zhang et al., 2004). MASW is a geophysical method, which generates a shear-wave velocity (Vs) profile (i.e., Vs versus depth) by analyzing Raleigh-type surface waves on a multichannel record. MASW system consisting of 24 channels Geode seismograph with geophones has been used in this investigation. The seismic waves are created by impulsive source of 15 pound (sledge hammer) with 300mmx300mm size hammer plate with ten shots, these waves are captured by 24 vertical geophones of 4.5Hz capacity. The captured Rayleigh wave is further analyzed using SurfSeis software. SurfSeis is designed to generate Vs data (either in one –dimensional (1-D) or two –dimensional (2-D) format) using a simple three-step procedure: i) preparation of a Multichannel record (some times called a shot gather or a field file), ii) dispersion-curve analysis, and iii) inversion.

The 1-D MASW tests have been carried out at locations corresponding to 5 boreholes (BH-1 to BH-5). The spread length locations are shown in Figure 1 as survey line 1-1 to 5-5. The multichannel analysis of surface wave (MASW) spread length was selected in such a way that the mid point of the MASW spread length matches with the SPT borehole points. All tests have been carried out with geophone interval of 1m. Source has been kept on both side of the spread and distance to source from the first and last receiver have been varied from 5m, 10m and 15m to avoid the effects of near-field and far-field. These source distances were helped to record a good singles in very soft, soft and hard soils. These values are in conferment with the exploration services section at the Kansas geological Survey (KGS) as suggested offset distance by Xu et al., (2006). Typical recorded surface wave arrivals using source to first receiver distance as 5m with recording length of 1000ms is shown in Figure 3.

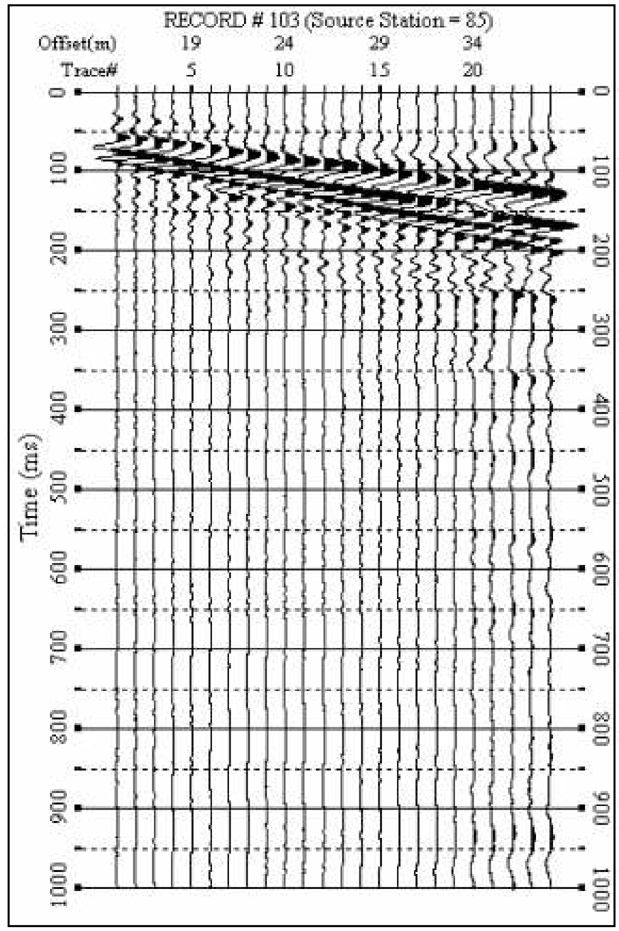

Figure 3: Typical seismic waves recorded in geode seismograph

Data Analysis and Results

The generation of a dispersion curve is a critical step in MASW method. A dispersion curve is generally displayed as a function of phase velocity versus frequency. Phase velocity can be calculated from the linear slope of each component on the swept-frequency record. The lowest analysable frequency in this dispersion curve is around 4 Hz and highest frequency of 75Hz has been considered. Typical

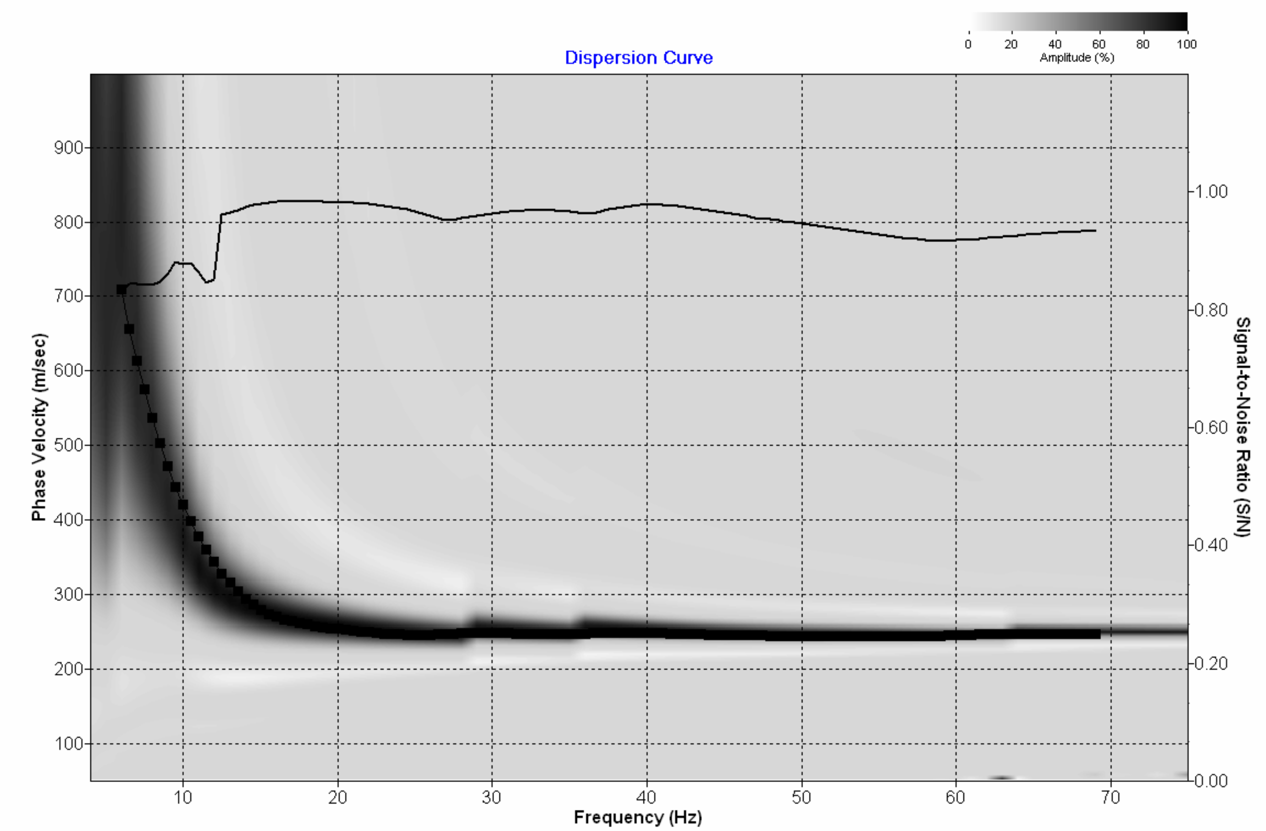

Figure 4: Typical dispersion curve obtained from MASW

dispersion curve is shown in Figure 4. Each dispersion curve obtained for corresponding locations has a very high signal to noise ratio of about 80 and above. A Vs profile has been calculated using an iterative inversion process that requires the dispersion curve developed earlier as input. A least-squares approach allows automation of the process (Xia et al., 1999) which is inbuilt in SurfSeis. Shear wave velocity has been updated after completion of each iteration with parameters such as Poisson's ratio, density, and thickness of the model remaining unchanged. An initial earth model is specified to begin the iterative inversion process. The earth model consists of velocity (P-wave and S-wave velocity), density, and thickness parameters. Typical 1-D Vs profile is shown in Figure 5. The sub-soil classification was made based on average shear wave velocity of 30-m depth (Vs 30) of sites using the National Earthquake Hazards Reduction Program (NEHRP) (BSSC, 2001) and International Building Code (IBC, 2006) classification. A layer with a shear wave velocity of more than 360 m/s is considered as a weathered rock. The weathered rock formation has been identified at 11.6m corresponding to lines 1-1, at 4.2m in line 2-2, at 8.5m in line 3-3, at 12.5m in line 4-4 and at 17.0m in line 5-5. The subsurface material having shear wave velocity of above 760 m/s is considered as hard/engineering bedrock. In this location, engineering rock identified at 36.2m in line 1-1, at 17.6m in line 2-2 and 29.1m in line 3-3. The hard rock is not encountered up to 50m in line 4-4 and even up to 40m in line 5-5. These observations using 1-D Vs profiles compare very well with the borehole data (see the Table 1). Table 1 shows the thickness of filled up soil, soil and weathered rock along with depth of hazard from geotechnical and geophysical methods.

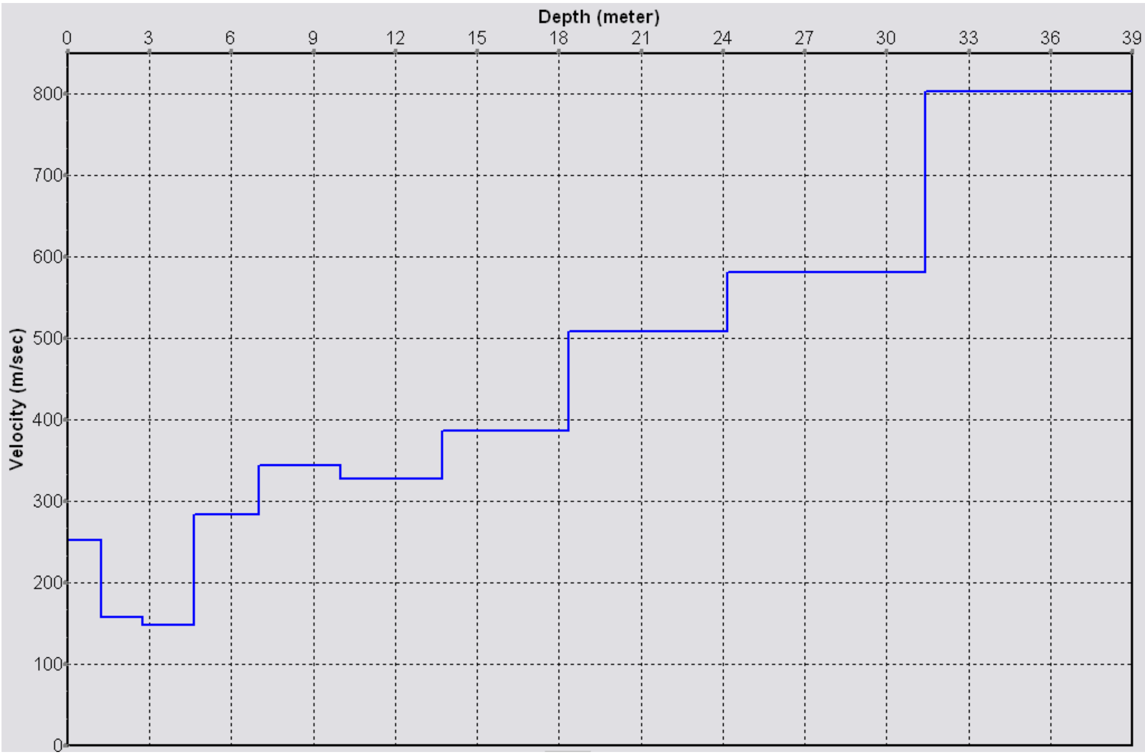

Figure 5: Typical one dimensional shear wave velocity profile

Soil and Rock Layer Mapping

2-D velocity profiling has been carried out to determine the soil and rock layer thickness spatially and to locate ground anomaly. To get the 2-D profile, a multiple number of shot gathers are acquired in a consecutive manner along the survey line by moving both source and receiver spread simultaneously by a fixed amount of distance after each shot. Each shot gather is then analyzed for 1-D Vs profile in a manner previously stated. In this way a multiple number of Vs profiles are generated. The Vs data are assigned into 2-D (x-z) grid. Various types of data processing techniques can be applied to this 2-D Vs data. A countering, a simple interpolation, data smoothing, or combination of these may be applied at this stage. When the Vs data are assigned to the grid, there is ambiguity in the horizontal coordinate (x) to be assigned because each Vs profile was obtained from a shot gather that spanned a distance too large to be considered as a the single point. It seems reasonable that the centre of the receiver spread be the most appropriate point because the analysed Vs profile represents an average property within the spread length (Park et. al, 2005). The 2D velocity profile has been used to find the layer thickness, subsurface anomaly and rock dipping directions. Typical 2D velocity profile for the line 7-7 is shown in Figure 6. Figure 6 shows that there is no considerable ground anomaly present in the line, however there is a slight reduction in velocity at mid point of the line. Reduction in velocity is due to presence of lose silty sand in borehole location (BH4) and this is confirmed during excavation of the site for constructions. The weathered rock velocity ranges are at shallow depth on left side of the line (Eastern side) and at deeper depth on right side (Western side) of line 7-7. The rock dipping observed in MASW matches well with the borehole observation.

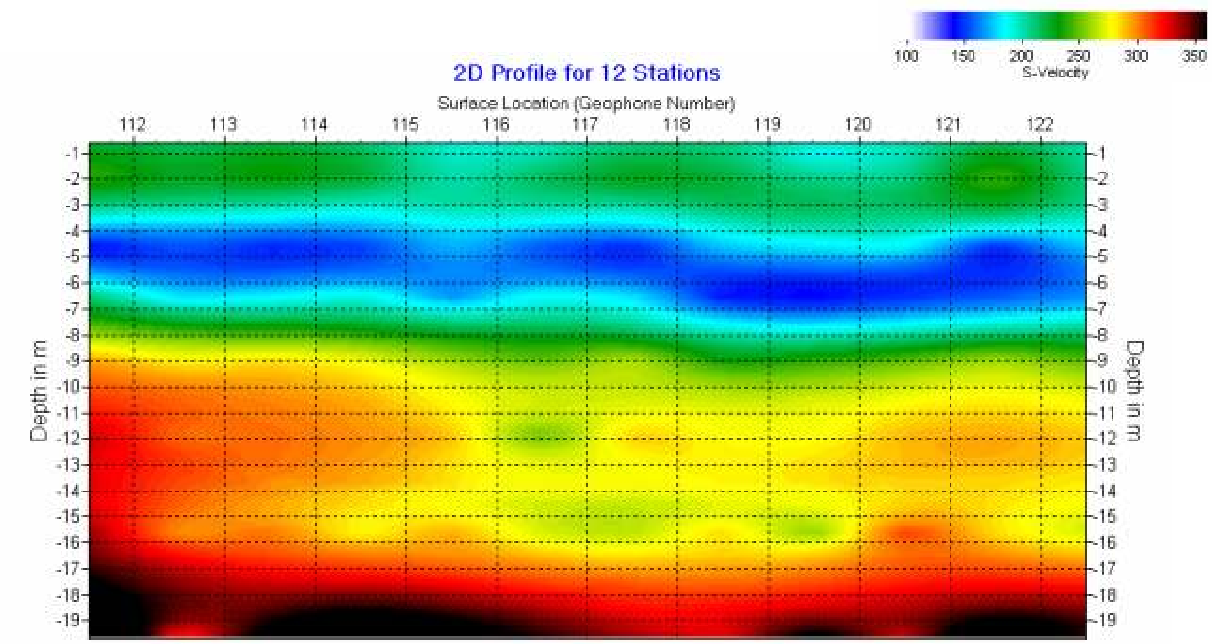

Figure 6: 2D Shear wave velocity profile obtained for the location7-7

Comparison

The spatial variability of the soil and rock layer, its thickness has been evaluated using borehole data and MASW data separately and compared. The three- dimensional (3-D) surface mapping technique has been used to map the soil and rock layers. Mapping has been carried out by using Surfer 8.0, it is a commercial contouring and 3-D surface-mapping program. Surfer converts data into contour maps and then surface plots. All aspects of 2-D or 3-D maps can be customized to produce unique presentations. The thickness identified from both experiments is converted as surfer grid file then the 3-D surface map is prepared. Using NEHRP and IBC velocity ranges for soil type, site soil layers are classification based on measured shear wave velocity. To find spatial variability of the thickness of filled up soil, soil overburden and rock surface using boreholes and MASW data, the values of thicknesses of layers are presented in Table 2. The values are matching very well with variation of ±0.5m, which are used to prepare to spatial variability map of site. Filled up soil thickness obtained using MASW data are matching well with the borehole information and slightly higher side, except location 3-3 where MASW value is lower than borehole value. Spatial variability of soil overburden thickness using borehole and MASW are shown in Figure 7 and 8 respectively. Overburden thickness obtained from both methods is matching well. Weathered rock identified using MASW data matches well with the borehole data (see Table 1). For identification of hard rock, the drilled borehole depth is not sufficient because boreholes are stopped after passing the few meters from weathered rock surface, in this investigation bore holes are terminated at 0.5m to 4m after meeting weathered rock surface (See Table 1). Hence, MASW 1-D as well as 2-D shear wave velocity profiles have been used to identify the depth of the hard rock. From Table 1, the hard rock is located within 16 to 36 m at south and eastern side of the site. But for the line number 4-4- and 5-5 up to 50m and 40m no hard rock was identified. From the above study, it is clear that soil and rock layers identified using MASW data matches well with the borehole information. For design purpose, the low strain dynamic properties (shear modulus and young's modulus) of subsurface materials are evaluated using measured soil in-situ density from

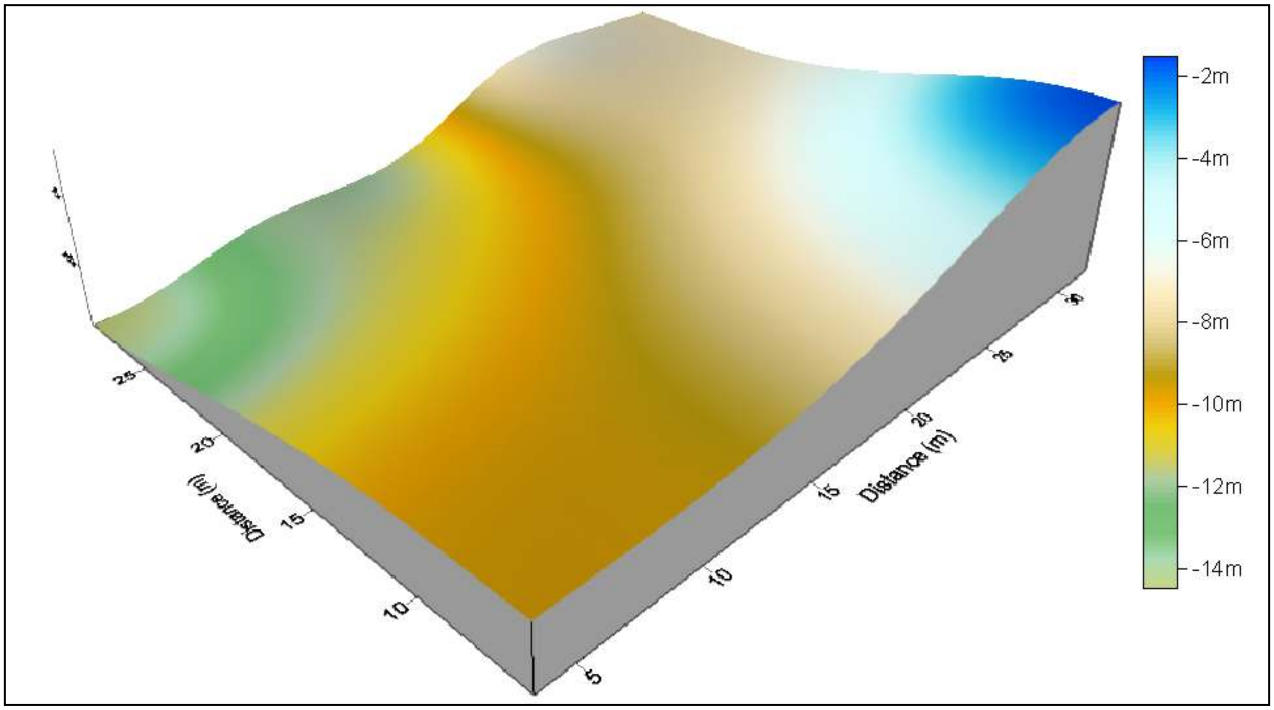

Figure 7: Soil overburden thickness using SPT data

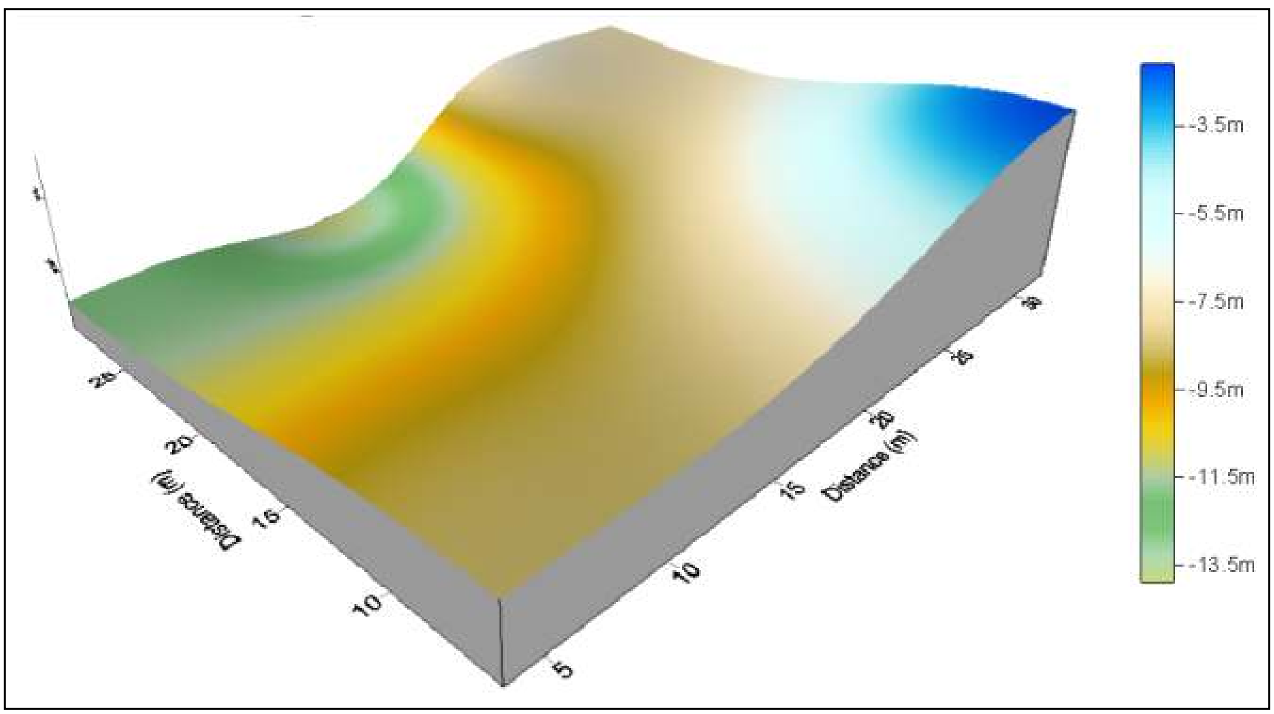

Figure 8: Soil overburden thickness using MASW data

Table 3 : Typical dynamic properties

Depth (m)	Shear wave velocity (m/s)	Measured Density (g/cc)	Shear Modulus (MN/m^2)	Poisson Ratio	Young's Modulus (MN/m^2)
0-3.2	250	1.94	121	0.30	315
3.2-8.0	150	1.94	44	0.30	113
8.0-28.5	280	2.00	157	0.30	408
>28.5	330	2.00	218	0.20	523

undisturbed soil samples collected in the boreholes and shear wave velocity from MASW. For the calculation of Young's modulus, the Poisson ratio has been assumed as 0.3 for soil and 0.2 for rocky layers. Soil and rock layers average dynamic properties have been estimated for location (1-1 to 5-5) corresponding to borehole locations of (BH1 to BH5). Typical results are presented in Table 3. Table 3 shows that the top layer of filled up soil and medium soil has higher value of velocity and modulus when compared to bottom layer of lose materials of silty sand with clay, followed by weathered rock and hard rock, which has larger velocity and modulus.

Ground Response Analysis

For the soil profiles evaluated using MASW and boreholes, the ground response of soil column for a given input ground motion data has been carried out using 1-D ground response analysis software SHAKE 2000. SHAKE2000 calculates the expected ground movement by combining wave propagation theory with material properties and seismic input motion. The geotechnical parameters like soil type, thickness of the layer, unit weight of the material, shear modulus values of the material and earthquake acceleration file is provided as input data. The common parameters of soil type, thickness of the layer, unit weight of the material have been obtained from the geotechnical tests and the shear modulus of the material is assessed using MASW survey. The synthetic ground motion generated by Sitharam and Anbazhagan (2007) has been used as earthquake acceleration file at weathered rock level in each analysis. The typical synthetic ground motion for site is shown in Figure 9. Shear modulus and damping reduction curves are selected based on the soil properties available form geotechnical data. Since the overburden soil for all the boreholes

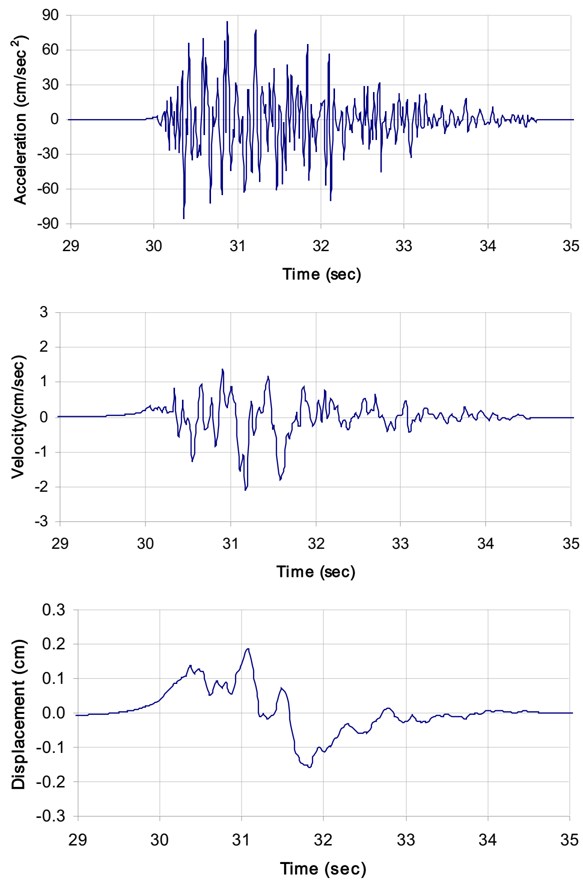

Figure 9: Typical input ground motion used for analysis

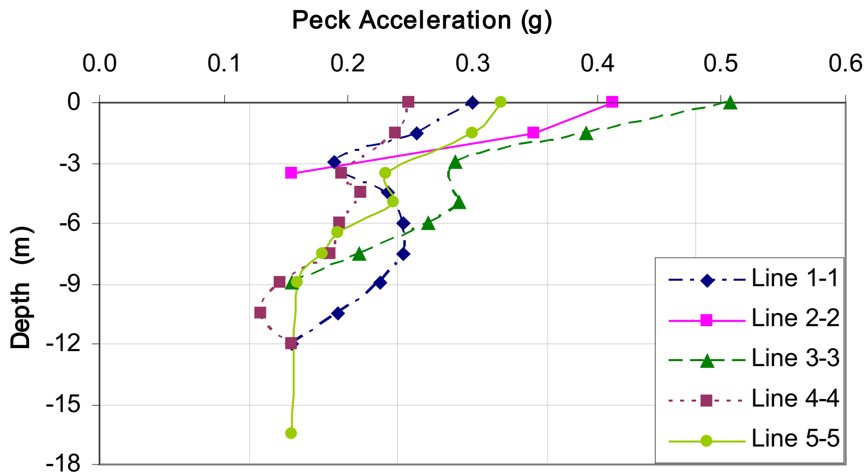

Figure 10 : Peak Acceleration with depth using shear wave velocity

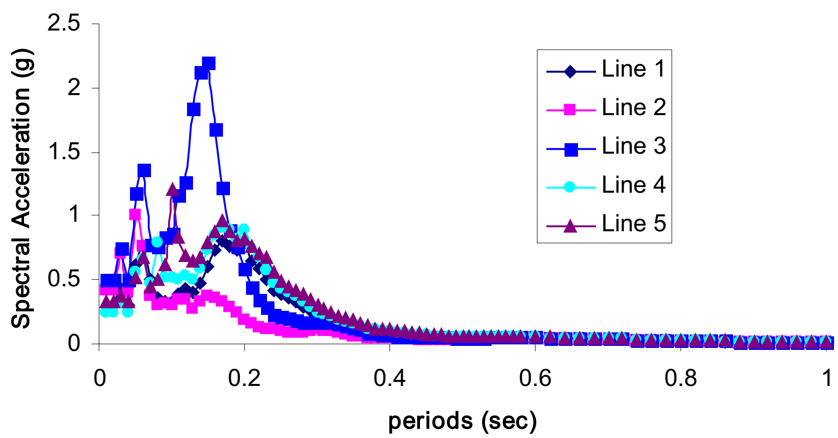

Figure 11: Response spectrum at surface using shear wave velocity

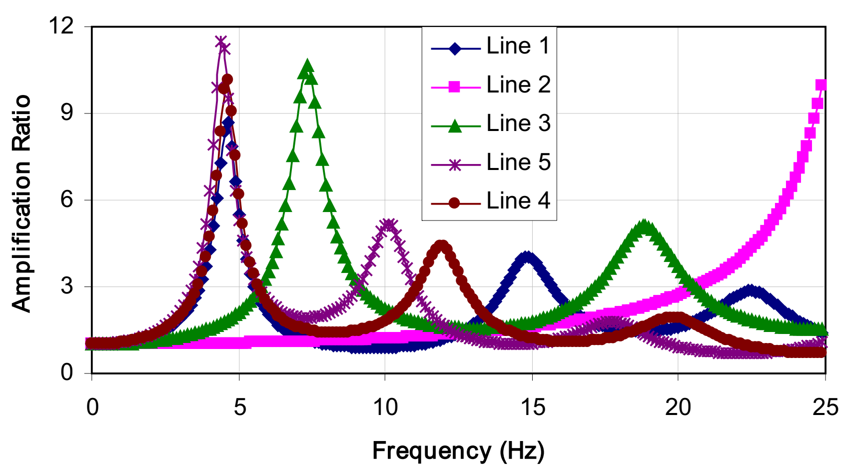

Figure 12: Amplification ratio using shear wave velocity

almost has similar properties, the curves proposed by Seed and Idriss (1970) are considered for the silty sand, similarly for rock the shear modulus and damping curves proposed by Schnabel (1973) have been used.

The peak acceleration at ground surface is obtained as 0.507g for given rock motion having peak acceleration of 0.156g. The peak acceleration of each layer for all boreholes is shown in Figure 10. Figure 10 shows similar amplification factors for all boreholes except borehole 2, which has a small overburden thickness and few numbers of layers. Figure 11 shows spectral acceleration soil of column for five borehole locations. The spectral acceleration obtained for the site, matches well with the shape of the uniform hazard spectrum. Figure 12 shows the amplification spectrum calculated using the MASW tests, the peaks of the amplification ratio are identified with in 8 Hz except in line 2. At line 2, the amplification ratio had not reached peak value until the frequency of 25 Hz, which may be due to very high shear wave velocity at the location. The response study shows that predominant frequency at center of the site is about 4.62 Hz. Ambient noise technique (Micro-Tremor study), by recording ambient micro seismic noise from the passive source, is used to obtain predominant frequencies of the site by using the Receiver Function/ Nakamura Technique (Nakamura, 1989). Micro seismic signals from passive source have been recorded from the site for a period of about 10 hours using seismograph. Spectra analyses software has been used to obtain H/V (Horizontal/ Vertical) spectral ratio of recorded seismic signals. The study shows that predominant natural frequency for the site is about 4.7 Hz, which is shown in Figure 13. The predominant frequency obtained from Microtremor method matches well with the site response study.

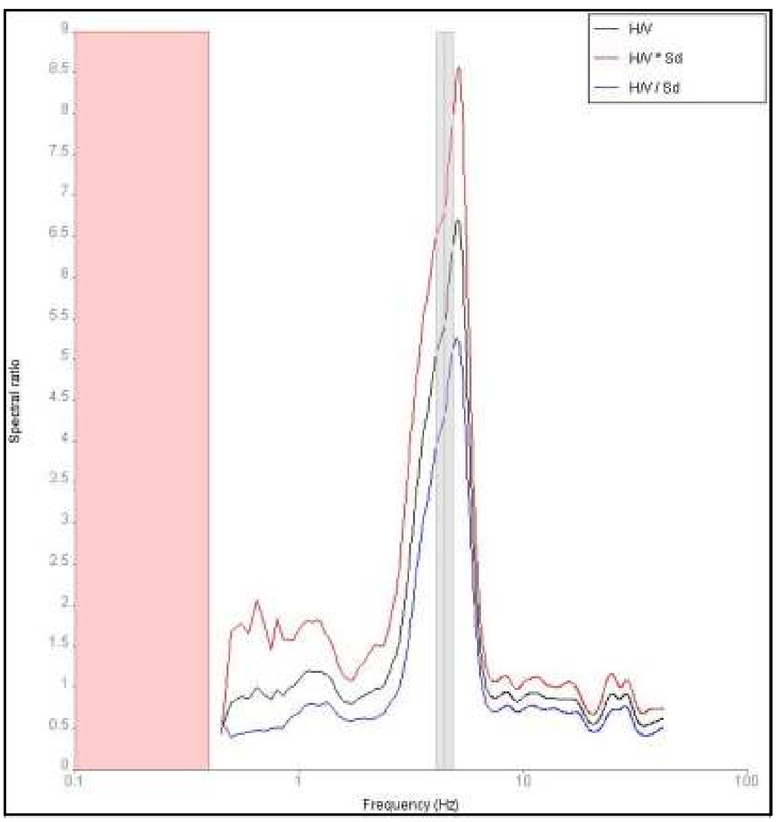

Figure 13 : Spectral ratio obtained using Micro tremor study.

Liquefaction Potential

Seismic and other dynamic loading can cause soil to liquefy, which induce the differential settlement. Hence it is mandatory to vary the liquefaction potential of site to design proper foundation. The borehole show that the site has silty sand layers with very high field 'N' values (N>19) indicating higher resistance to liquefaction. However, the top filled up soil has the field 'N' values less than 12 and presence of shallow water table, requires evaluating the liquefaction potential of the site considering SPT corrected 'N' values and amplified acceleration at ground surface. Liquefaction potential of the site for amplified surface peak acceleration has been estimated in terms of factor of safety. Factor of Safety against liquefaction of soil layer has been evaluated based on the simplified procedure by Seed and Idriss (1971) and subsequent revisions by Seed et al., (1983), Youd et al., (2001) and Cetin et al., (2004). In this study, the earthquake induced loading is expressed in terms of cyclic shear stress and this is compared with the liquefaction resistance of the soil. Liquefaction calculation or estimation requires two variables for evaluation of liquefaction resistance of soils. Two variables are defined based on cyclic stress approach as follows.

1. The seismic demand on a soil layer, represented by Cyclic Stress Ratio (CSR).This is cyclic stress ratio required to generate liquefaction
2. The capacity of soil to resist liquefaction represented by Cyclic Resistance Ratio (CRR).

The earthquake loading can be predicted using Seed and Idriss (1971) simplified approach. The earthquake is evaluated in terms of uniform cyclic shear stress amplitude as given below :

Cyclic stress ratio (CSR) =

$$0.65 = \left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right) r_d \tag{4}$$

In this equation $0.65\ \frac{a_{\max}}{g}$ a represents 65 % of the peak cyclic shear stress, $a_{\max}$ is peak acceleration at surface (obtained from site response study), g is the acceleration of gravity, σ_{vo} and σ_{vo}' are total and effective vertical stresses and r_d = stress reduction coefficient.

The liquefaction resistance can be calculated based on laboratories tests and in situ tests. Here our interest is to find liquefaction resistance using in situ test of SPT. Cyclic resistance ratio (CRR) is arrived for the earthquake magnitude of 7.5 using equation proposed by Idriss and Boulanger (2005) as given below :

$$CRR = \exp\left\{\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right\} \tag{5}$$

For the present study, the earthquake moment magnitude of 5.1 has been considered, hence it is necessary to apply the Magnitude Scaling Factor (MSF). The magnitude-scaling factor used in the present study for magnitude less than 7.5 is given below (Seed and Idriss, 1982) :

$$\mathrm{MSF} = \left[\frac{10^{2.24}}{M_W^{\;2.56}}\right] \tag{6}$$

Final factor of safety against liquefaction at each layer is evaluated by developing simple spreadsheets using Window macros. If cyclic stress ratio caused by the earthquake is greater than the cyclic resistance ratio of in situ soil, then liquefaction could occur during the earthquake. The factor of safety against liquefaction is defined as follows :

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) MSF \tag{7}$$

Typical liquefaction analysis spread sheet is shown in Table 4. From Table 4, the top layer up to 3m has the corrected 'N' value of less than 20, which results in the factor of safety of less than 1.5. Which look like the site is not safe against liquefaction, but if look at the properties of filled up soil, which has plastic index of more than 12, one can say that site is

Table 4 : Typical liquefaction calculation spread sheet

Magnitude, M_w = 5.1 | **Peak Acceleration = 0.507g**

Depth	Corrected 'N' value	σ_{vo}	σ_{vo}'	r_d	CSR	FC	Liquid Limit	Plastic limit	Plastic Index	CRR	MSF	FS	Liquefaction Possibility
(m)		kN/m^2	kN/^{m}2			%	%	%					
1.50	7	30.15	30.15	0.98	0.32	31	38	22	16	0.10	2.68	0.82	No
3.00	20	60.30	45.59	0.96	0.42	41	40	25	15	0.21	2.68	1.36	No
5.00	34	100.50	66.17	0.93	0.46	35	37	29	8	0.96	2.68	5.58	No
6.00	76	120.60	76.46	0.91	0.47	28	NP	NP	NP	NP	2.68	NL	No
7.50	77	150.75	91.89	0.89	0.48	22	NP	NP	NP	NP	2.68	NL	No
9.00	72	180.90	107.33	0.87	0.48	22	NP	NP	NP	NP	2.68	NL	No

σ_{vo} - Total Stress
CSR - Cyclic Stress Ratio
MSF - Magnitude scaling factor
NL - Non Liquefiable
σ_{vo}' - Effective stress
FC - Fine content
FS - Factor of safety
r_d - Stress Reduction factor
CRR - Cyclic Resistance ratio
NP - Not Possible to Evaluate

Table 5 : Recommended dynamic propertied for design

Depth (m)	Shear wave velocity (m/sec)	Density (g/cc)	Shear Modulus (MM/m^2)	Poisson Ratio	Young's Modulus (MN/m^2)
0 - 6	100	1.90	19	.30	49.4
6 - 15	220	2.00	97	.20	233
15- 25	300	2.00	180	.20	432
25 - 35	590	2.20	766	.20	1838
>35	1000	2.20	2200	.20	5280

safe against liquefaction. However, it is recommended that place foundation at depth of 2.5m below from original ground surface, so that the site is safe against liquefaction and its consequent settlements.

Cyclic Triaxial Experiments on Undisturbed Soil Samples

Undisturbed samples collected from boreholes 3, 4 and 5 were used to verify the liquefaction potential of the soil. This is done by conducting cyclic triaxial test in the laboratory on the undisturbed soil samples. The test has been carried out as per ASTM: D 3999 (1991) in strain controlled mode. Cyclic triaxial tests are carried out with double amplitude axial strains of 0.5%, 1% and 2% with a frequency of 1Hz. A typical cyclic triaxial test results are presented in Figures 14 and 15. Figure 14 shows the variation of deviatoric stress versus strain plot for more than 120 cycles of loading (axial strain = 0.25%; applied confining pressure 100 KPa, for the undisturbed sample corresponding to 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 15 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 stresses vs. strain plots have not become flat, indicating no liquefaction. The calculated factor of safety against liquefaction results, for this borehole is also very high indicating no liquefaction. These results match well with the factor of safety calculated based on the simplified method.

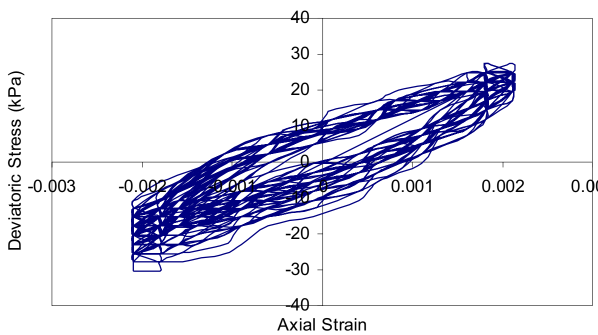

Figure 14: Typical hysteresis loops from a cyclic triaxial test

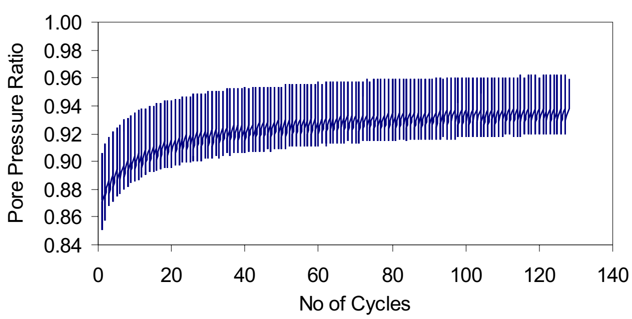

Figure 15: Typical pore pressure ratio plot with number of cycles

Summary

The subsurface investigation has been carried out using conventional standard penetration test at five locations and Multichannel Analysis of Surface Wave (MASW) survey at these locations. Five MASW 1-D survey have been carried out close to boreholes and 2-D survey carried out from borehole 3 to borehole 5. Subsurface material depth and type obtained from MASW matches well with the borehole data. Low strain dynamic properties of shear modulus and Young's modulus are evaluated using measured density and shear wave velocity of soil, which are recommended for the design. Finally the following recommendations are made for design of sensitive foundation.

i) For the purpose of convenience and design, one set of dynamic properties for the site at different depth are recommended which is given in Table 5.

ii) It is suggested to take the foundation below the filled up soil (minimum of 2.5m below ground level) to avoid loose earth below the proposed facility.

iii) The amplifications are more for the filled up soil and it considerably reduced for silty sand layers. If the foundations are placed below the filled up soil layers so the probability for amplification is very less.

iv) The predominant natural frequency for the overburden in the site is about 5.0 Hz, The results obtained from shake analyses using borelog details matches with the actual measurements from the ambient noise analyses.

v) Liquefaction analysis using Seed and Idriss simplified approach and cyclic triaxial testing results clearly indicate that the soils in the overburden layers are not prone to liquefaction. This is attributed to their high in-situ densities in 'SM' materials and presence of clay material in 'SC' material.

vi) From the MASW survey corresponding to borehole locations BH-3, BH-4 and BH-5 and also estimated 2-D profiles of shear wave velocity close to the centerline of the proposed human centrifuge facility, no anomalies have been identified.

From this study, it is very clear that the geophysical test of MASW can be used together with conventional technique to measure and map the subsurface features. In addition, this technique gives dynamic properties at very low strain level required for settlement and soil structure analysis.

Special Note

This paper presents the summary of the total work. More technical and theoretical background can be obtained by contacting the authors (anbazhagan @civil.iisc.ernet.in and sitharam@civil.iisc.ernet .in).

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