MEASUREMENT OF DYNAMIC PROPERTIES AND EVALUATION OF LIQUEFACTION POTENTIAL OF A 500MW PROTOTYPE FAST BREEDER REACTOR SITE LOCATED IN SOUTH OF INDIA

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SUMMARY

A detailed geotechnical investigation has been carried out at a site located on the Bay of Bengal on the East Coast of Peninsular India at Kalpakkam for the construction of a 500 MW Prototype Fast Breeder Reactor (PFBR). Insitu penetration tests, Seismic wave velocity measurements, Block Vibration tests as well as cyclic plate load tests were performed to measure dynamic soil properties of different soil layers at various strain levels. Based on the field test data, variation of shear wave velocity with depth, a normalised shear modulus vs. shear strain plots are established for the selection of design dynamic soil parameters required for the analysis Nuclear Power Plant foundations / structures subjected to Earthquake, vibratory machine loads, etc.

A level ground liquefaction analysis was performed for the top saturated fine to medium coarse sand and silty sandy layers of varying density of 8 m thick using field performance data (SPT tests). The modified penetration resistance is obtained from the field N-value by applying correction for various equipment and procedural variations in the conduct of test. A design earthquake (S2 level) of magnitude of 6.0 and peak horizontal acceleration of 0.156g determined from the site-specific seismic hazard assessment was adopted. The analysis indicates that the factor of safety against liquefaction for sandy soil layers is much higher than 1.0 and for silty sandy layer is marginally above 1.0.

INTRODUCTION

A detailed geotechnical investigation with respect to the evaluation of static and dynamic parameters and liquefaction potential was carried out for a site located on the east coast of peninsular India at Kalpakkam which is situated about 68 km south of city Chennai in south of India. A 500 MW Prototype Fast Breeder Reactor is proposed to be constructed at the above site about 250 m from the seacoast.

The geotechnical survey to assess the static and dynamic parameters was based on the field tests and laboratory tests. The field tests performed at the site includes Geophysical tests using Crosshole technique, in-situ Block resonance tests, Cyclic plate load tests, 29 boring with SPT tests 1.0 to 1.5 m apart, CPT tests, etc.

In this paper general geological, seismic and geotechnical characteristics of the site are presented. The measurement of in-situ dynamic properties of soil at various strain levels as well as the evaluation of modulus reduction curve from the field dynamic test data is also presented. A liquefaction study performed for the subsurface granular soil layers for a site specific earthquake using field test data is also presented.

GENERAL GEOLOGICAL CHARACTERISTICS

The site falls at the depositional environment by the floods and coastal processes. The ground water table fluctuates with season and is about 1-2m below the ground level during monsoon.

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The alluvium covers the rock masses with few rock outcrops in the surrounding region. The geological setup of site consists of Charnockite suite of rocks and pyroxene granulates of Archaean age as a basement overlain by recent coastal alluvium with an unconformity. There are dolerite dyke intrusions in the post charnockitisation period in the Charnockite rock mass. The bedrock occurs at about 10 -15 m depth in this region. Rock mass rating is evaluated as 77 and Q- system as 17. This indicates the good quality of rock formations in and around the site.

Deep borehole investigations at the site shows that there is a North- South trenching major joint with pseudo tachylites along the fracture of joints. There are weak planes as the major fracture zones encountered at about 100, 200, 400 and 600m depth are traceable in other boreholes. The fractures seen in shallow boreholes upto 60 m depth at the site are of insignificant since the fractures are thin and sealed with pseudotachylites.

EARTHQUAKE CHARACTERISTICS

The largest earthquake in the 300-km radius around site had a magnitude of 5 (VI MM intensity). The largest earthquake to have occurred in Peninsular India was the Coimbatore earthquake of February 8, 1990, with a magnitude of 6.0. This epicentre lies about 400 km from the site.

An extract of important faults, their distance from site and maximum Magnitude are given in Table 1 (Ghosh 1992).

Table 1: Summary of Fault details

Fault	Distance, km	Magnitude
26	110	7
15a	40	6
15e	17	5
15c	20	6

The design basis peak ground acceleration evaluated for safe shutdown (S1 level) earthquake and operating basis (S2 level) earthquake are given in Table 2.

Table 2: Design earthquake data

Peak Ground Acceleration, (g)			
	Horizontal		Vertical
	Comp. 1	Comp. 2	
S2	0.156	0.156	0.104
S1	0.078	0.078	0.052

SUBSTRATA CHARACTERISATION

To evaluate the static properties of the soil and rock materials field tests as well as laboratory tests were performed. The general stratification of the site (Figure 1) and its characteristics are as follows:

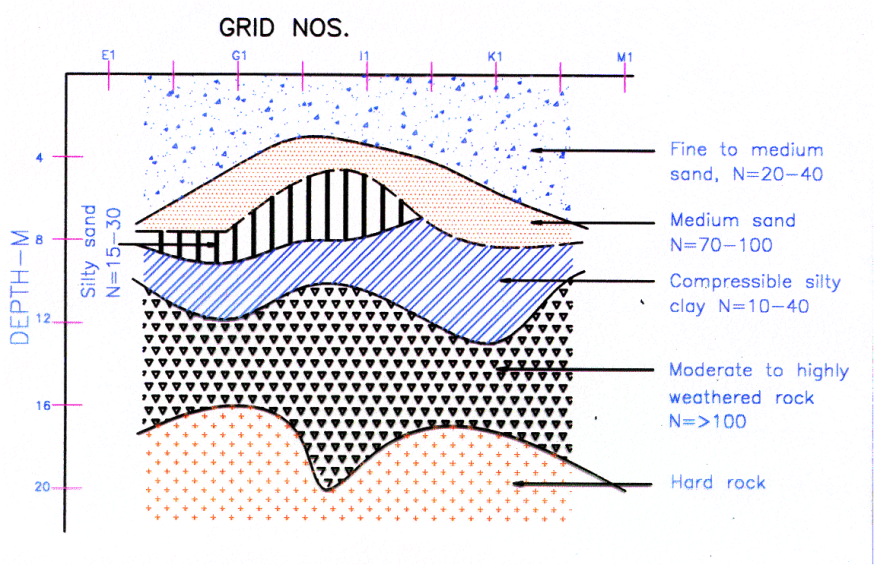


Figure 1: Typical cross section of the soil strata of PFBR site (North- South)

Stratum I

Top loose to dense medium sand. This stratum consists of 3 layers : fine to medium sand, medium sand and silty sand. The average thickness of this stratum is 7 m. The SCPT and DCPT test results indicate increase in strength with depth in this stratum upto 6m depth.

Stratum II

Clays of medium consistency and high compressibility (CH). The thickness of this layer varies from 0.5m to 5m. The N-value varies from 10 to 40. The physical properties of the clay layer can be summarised as follows: natural moisture content = 18 - 32 %, liquid limit = 42 to 160 %, plastic limit = 23 to 41 %, plasticity index = 14 to 120 %. The consolidation tests show higher values of compression index of 0.98 to 1.2. The value of cohesion obtained from shear test is in the range of 10 to 40 kPa.

Stratum III

Moderate to highly weathered rock. This stratum occurs at a depth of about 12.0 m. The N-value exceeded 100 and some cases rebound of SPT hammer was observed. The core recovery and the rock quality design (RQD) are nil in this layer.

Stratum IV

Medium to coarse grained hard rock comprising of Charnockite, granite and gneiss with garnet crystals. This stratum occurs at a depth of about 15 to 20 m. The core recovery and rock quality design is almost 100% at this layer. The average field permeability is 7 x 10^{-2} cm/s. The uniaxial compressive strength is in the range of 78 - 206 MPa and the triaxial compressive strength in the range of 396 - 587 MPa.

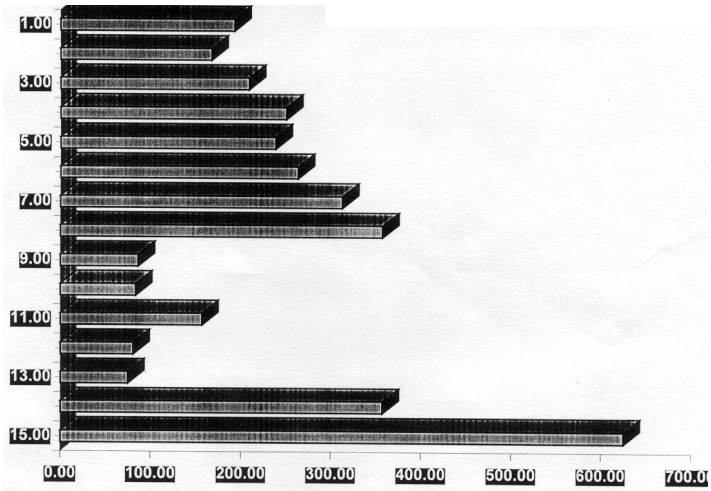
DYNAMIC SOIL PARAMETERS

The dynamic properties of soil are strain dependent and their best estimates and ranges of the variation can be obtained only by carrying out various types of field and laboratory tests. Therefore, low strain dynamic tests such as Seismic cross hole survey and high strain dynamic tests such as Block resonance tests, Cyclic plate load tests etc. are performed at the site.

Seismic Cross Hole Survey (SCS)

The Seismic Cross Hole Survey (SCS) is the best geotechnical method for determining the variation of low-strain shear wave velocity with depth. The basis of this method is generating shear waves in a borehole (source borehole) and measuring their arrival time at the same elevation in the receiver boreholes. Standard penetration test hammer with split spoon sampler at the bottom is used as the seismic source as recommended by Stokoe & Woods (1972) and Gazetas (1992). The shear wave arrival due to the disturbance produced by seismic source is picked up by a borehole-pick. The borehole-pick essentially consists of acceleration pick ups, pneumatic packer assembly, pneumatic pump and controls. An acceleration pick up fixed at the SPT drilling rod is connected to the

carrier frequency amplifier and triggering circuit of oscilloscope. When the SPT hammer falls on to the drill rod it triggers the unit to active mode and the seismic waves are sensed by the borehole-pick in the receiver boreholes. Seismic traces obtained from cross hole test are critically analysed to determine the shear wave velocity Vs and dynamic shear modulus G. The typical Shear wave logging summary for the site with depth is given in Figure 2.



Depth, m

Shear Wave Velocity, m/s

Figure 2 : Shear wave logging summary

Block Resonance Tests

Forced vertical vibration tests were carried out on concrete blocks of 1m x 1m x 1.5m at a depth of 1.5m below ground level. The mechanical oscillator was mounted on the test block such that it generates purely vertical sinusoidal vibrations. The mechanical oscillator was connected through a flexible shaft with DC motor and speed control unit. Two acceleration pick-ups duly calibrated were mounted on the block such that they sense vertical motion of the block. Choosing a suitable value of angle of setting of eccentric masses, the oscillator was made to run at constant frequency. Out put signals from pick-ups were monitored and recorded using Carrier Frequency Amplifiers (CFA) and Digital Storage Oscilloscope. The frequency of the oscillator was then increased and the process was repeated. The tests were carried out for different settings of eccentric masses. Plots of amplitude versus frequency were made and the resonant frequency was established. A typical result of the block resonance test is given in Figure 3.

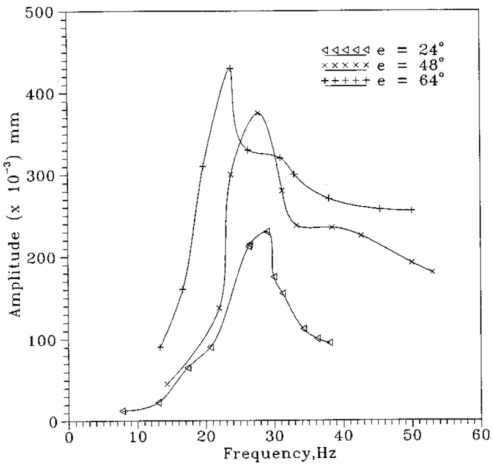


Figure 3 : Amplitude vs frequency response curve

Barkan's coefficient of elastic uniform compression, C_u is evaluated using the following expression.

$$C_u = \frac{4\pi^2 f_{nz}{}^2 M}{A}$$

where f_{nz} is the resonant frequency (Hz), M is the mass of the test block plus mechanical oscillator, A is the contact area of the test block with the soil. The test values of coefficient of uniform compression C_u varies from 52 MN/m^3 to 130 MN/m^3. The effective damping available in the soil is determined from the shape of response curves varies from 0.063 to 0.164. The value of dynamic shear modulus, G, the most important parameter in the response analyses of soil deposits during an earthquake is established from the values of C_u using the following relations,

$$C_u = \frac{1.13E}{(1-\nu^2)} \frac{1}{\sqrt{A}}$$

$$E = 2G\ (1+\nu)$$

where E is the Young's modulus, ν is the Poisson's ratio of the soil and A the contact area of test block.

The value of dynamic shear modulus from the test ranges from 16.6 MN/m^2 to 44.5 MN/m^2. The strain level associated with the block resonance test varies from 1 x 10^{-4} to 12.5 x 10^{-4}. The mean effective confining pressure is 2.21 x 10^{-2} MN/m^2.

Cyclic Plate Load Test

Tests were carried out on 300mm square plate. The load on the test plate was applied through a reaction frame in increments of 5 ton/m^2 (50kPa). After recording the final settlement at each stage, the plate was unloaded completely and the subsequent elastic rebound of the plate was measured by means of dial gauges. From the test data, the elastic rebound of the plate corresponding to each intensity of loading was obtained. The value of Cu is calculated as the slope of the load intensity versus elastic rebound plot and it varies from 240 MN/m^3 to 280 MN/m^3. The dynamic shear modulus obtained from C_u ranges from 14 MN/m^2 to 40 MN/m^2.

Modulus reduction curve

The value of dynamic shear modulus and the corresponding strain level associated with the above dynamic tests are tabulated and the results are plotted as Figure 4. The variation of normalised shear modulus (G/G_{max}) with strain level are also established and plotted as Figure 5. This data can be used for carrying out ground response analysis of the site to predict ground surface motion for development of design response spectra and to evaluate dynamic stress and strains due to given earthquake. It can also be used for the selection of design parameters for analysis of Nuclear Power Plant foundations and structures subjected to machinery loads etc.

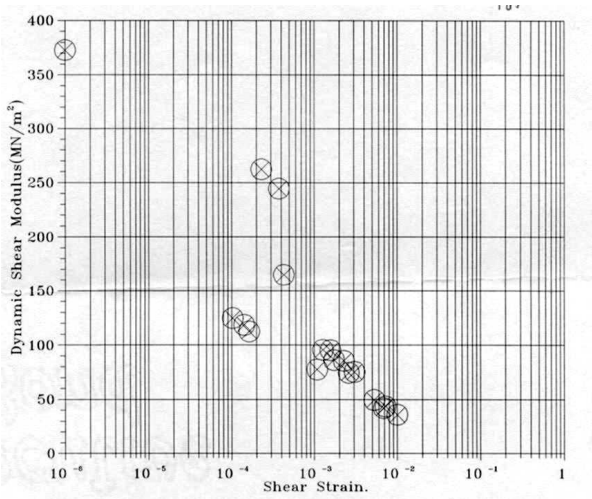


Figure 4 : Dynamic Shear Modulus vs Shear Strain Plot

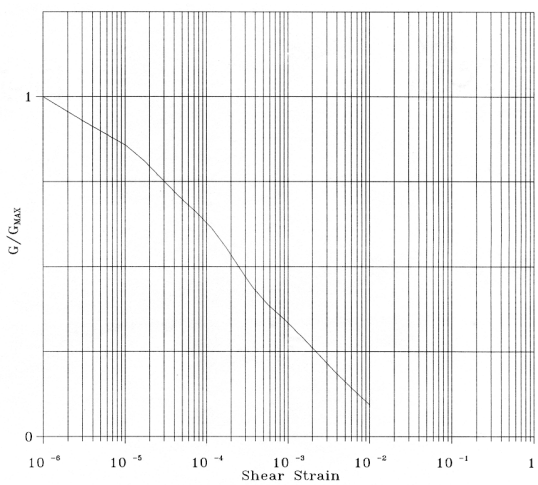


Figure 5 : Modulus reduction curve

LIQUEFACTION ASSESSMENT

A method proposed by Seed et al (1985) is adopted for the evaluation of liquefaction potential of this site. In this procedure cyclic stress causing liquefaction is evaluated using the correlation between normalised SPT N value and observed liquefaction due to past earthquakes in various parts of the world.

As per IAEA (1992) and AERB (1992) recommendations, liquefaction analysis was carried out for limiting earthquake S2 (SSE) level. The design earth quake of magnitude (M) 6.0 with the peak horizontal acceleration (PHA) of 0.156g are adopted for the analysis.

The layer with shear wave velocity above 500m/s is considered as equivalent to rock stratum as recommended by Wang and Law (1994) and hence the soil layers above stratum III are treated as overlying loose deposit. However, stratum II (Clay layer) has fines greater than 15% and liquid limit greater than 35%, it is not vulnerable to liquefaction as per Seed et al (1985). Therefore the level ground liquefaction analysis was carried out for the top sand and silty layers (Stratum I) upto 8m depth. The ground water table is assumed to be at the ground level. The modified penetration resistance, N_{60} obtained from the field SPT N value by applying corrections due to the effects of induced energy, length of drill rod, sampling barrel size and overburden pressure is adopted for the evaluation. The unit weight of granular soil is obtained from average field SPT N values.

The average shear stress (τ_{avg}) developed in the soil deposit due to upward propagating shear waves during earthquake is related to ground acceleration by the following equation, Seed and Idriss (1971).

$$\tau_{avg} = 0.65 \frac{a_{max}}{g} \sigma_{vo} . r_d$$

where a max is the design peak horizontal acceleration in m/s^2 and σ_{vo} is the total overburden pressure in kN/m^2, g is the acceleration due to gravity in m/s^2 , rd is the correction factor for acceleration with depth.

The Cyclic Stress Ratio induced by Earthquake (CSRE) is calculated as

$$CSRE = \frac{\tau_{avg}}{\sigma_{vo}'} ,$$

where $\sigma vo'$ = effective overburden pressure in kN/m^2.

Cyclic stress ratio causing liquefaction for magnitude of 7.5 ($CSRL_{M=7.5}$) for various depths is obtained from the graphs showing relationship between Cyclic Stress Ratio causing liquefaction and modified penetration resistance ($CSRL_{M=7.5}$ vs. N_{60}) for sands with varying fines content in M=7.5 earthquake proposed by Seed et al (1985). Since the design earthquake magnitude at the investigated site is 6.0, a correction has to be applied for the above values determined at a earthquake magnitude of 7.5. Hence, the Cycile Stress Ratio to initiate liquefaction for design earthquake is obtained as follows:

$$CSRL_{M=6.0} = 1.32 \text{ x } CSRL_{M=7.5}$$

The detailed calculations can be found in Boominathan et al (1998).

The liquefaction potential of the granular soil layers are evaluated in terms of Factor safety against liquefaction FS_L, defined by Ishihara (1993) as

$$FSL = \frac{\text{Cyclic Shear Stress ratio to initiate Liquefaction (CSRL)}}{\text{Cyclic Shear Stress ratio Induced due to Earthquake(CSRE)}}$$

The variation of factor of safety against liquefaction, FSL with depth is given in Figure 6. It shows high factor of safety against liquefaction is high (above 1.5) for all sancy soil layers and marginally low (above 1.2) for sity sand layer of 1.0 m thick occurring at a depth of about 7.0 m. Hence it can be concluded that the saturated sandy and silty sancy layers at the site are not vulnerable to liquefaction under the design earthquake.

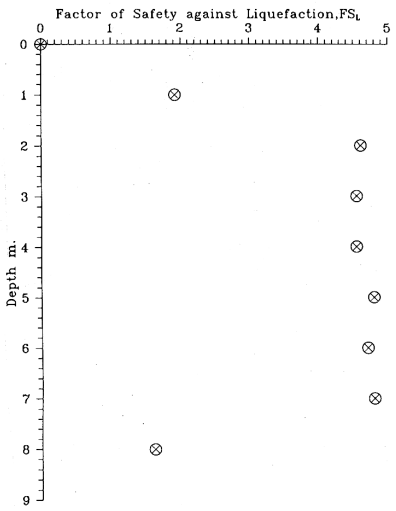


Figure 6 : Variation of Factor of Safety against liquefaction (FS_L) with depth

CONCLUSIONS

The range of variation of geotechnical properties for each layer required for analysis of Nuclear Power Plant foundations was estimated by various laboratory and field tests with reasonable accuracy.

Based on the geophysical test data and field vibration test data, variation of dynamic properties with strain level including modulus reduction curve for various layers are established. This can be used for carrying out ground response analysis of the site and for the analysis Nuclear Power Plant foundations / structures subjected to Earthquake, vibratory machine loads, etc.

Liquefaction analysis performed for the top 8m thick saturated sandy and silty sandy soil layers for design earthquake of M= 6.0 and PHA = 0.156g indicates that the factor of safety against liquefaction, FS_L for sandy soil layers is much higher than 1.0 and for silty sand layer is marginally above 1.0. Therefore the investigated site is not likely to liquefy at any depth even for the worst scenario of ground water table rising to the existing ground level.

ACKNOWLEDGEMENT

The authors gratefully recognise the financial support received from the Director, Indira Gandhi Centre for Atomic Research (IGCAR), Kalpakkam for carrying out the investigations. The author gratefully acknowledge the valuable comments and guidance provided by Professors N.R.Krishnaswamy and K.Rajagopal of Indian Institute of Technology Madras throughout the investigations.

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