Seismic site characterization for nuclear structures and power plants

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Seismic site characterization is carried out for the construction of nuclear structures and power plants in earthquake-prone areas to establish the occurrence of severe seismic hazards such as tectonic rupture, surface faulting, large scale liquefaction, sliding and seismic settlement which may alter the overall stability of the site. Seismic characterization is required to finalize the design earthquake parameters including choosing input seismic data. As a part of the investigation, measurements of relevant dynamic parameters both in laboratory and *in situ* have been made for carrying out dynamic soil structure interaction analysis, for determination of dynamic deformation, seismic settlement and dynamic response spectrum of the site, and for calculating dynamic earth pressure acting on retaining structures. We discuss here the seismic investigation components and methods, measurement of *P*- and *S*-wave velocities in the field and estimation of important dynamic parameters such as maximum shear modulus, modulus reduction curve, damping ratio, seismic site classification, predominant site period, liquefaction analysis through case studies for nuclear structures at Kalpakkam and Kudankulam and power plant structures at New Delhi and Konaseema.

GEOTECHNICAL site investigations in seismically active regions should include gathering of information about the physical nature of the site and its environment that will allow an adequate evaluation of seismic hazard. The scope of the investigation will be a matter of professional judgement, depending on the seismicity of the area and nature of the site as well as of the proposed or existing construction. In addition to the effects of local soil conditions upon the severity of ground motion, the investigation should cover possible earthquake danger from geological or other consequential hazards such as fault displacement, subsidence, liquefaction, landslides, mudflows, etc. In this paper, various components and methods of seismic site characterization for nuclear structures and power plants are discussed through case studies: Prototype Fast Breeder Reactor (PFBR) site and Spent Fuel Storage Facility (SFSF) site, Kalpakkam (TN); Atomic power project site, Kudankulam (TN); Pragati power project site, New Delhi; and combined cyclic power plant, Konaseema (AP).

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Investigation components and methods

The major components of investigation specified here are particularly pertinent to the conceptual earthquake-resistant design of nuclear structures and power plants to be built in an earthquake zone.

Surface geo-seismic survey

Normally, supplementary survey is to be done on the basis of existing geological survey. Emphasis should be placed on the following[1]:

- Can any dislocation be traced to or even be visible in the quaternary deposit? If any suspicious surface rupture exists, sampling is necessary for dating and/or for microscopic studies to determine the nature of the rock structure.
- Can any seismically generated morphological features such as fractured valley, fault scarp, and fault spring be found? If any, what is its past behaviour in terms of seismic productivity?
- Can any scarp or colluvial deposit formed by past earthquakes be found? In particular, what is the maximum earthquake to be expected at the site? Can they be dated to obtain the age of the earthquake?
- Can any drunk tree or inclined old structure suspectedly caused by earthquake be found? (Drunk trees in forest caused by earthquake are normally characterized by a large area of trees inclining at a certain angle and pointing to a certain direction as a group).

Normally, geological investigation for nuclear structures is carried out not only at the earlier stage of investigation but also after the excavation of the site during construction. For example, geological investigation including geological mapping of founding strata at Kudangulam atomic power plant[2] is carried out at the foundation level of – 0.33 m from Mean Sea Level (MSL) for reactor building units. It is found that the site is covered by granite and gneisses under different weathering conditions. The structural features include fractures and joints that criss-cross the area; these are frequently filled by calcareous materials and intruded by granite–pegmatite veins. However, based on the borehole logs, it is observed that the calcareous zone is mostly narrow and occurs as a vertical band extending to a maximum depth of about 15 m below founding level. Later, consolidation cement grouting was carried up to a depth of 15 m and thus the joint and calcareous material in the discontinuities had been adequately cemented that improved the monolithic behaviour of founding rock mass.

Subsurface survey

Subsurface investigation is an important step in the analysis and design of proposed civil engineering buildings, where all relevant information about the sub-soil conditions need to be identified; in particular, type of soil/rock, its extent or thickness and its properties are generally obtained through sampling and testing or by *in situ* testing. The number, type, location and depth of investigations depend on the nature and size of the project, on the variation of subsurface conditions across the site and on the total cost of the project. However, for nuclear structures, power plants and other important structures, the extent of investigation should be governed by required technical information rather than the cost of the project. Under these circumstances the exploration should be conducted to meet the following requirements:

- For calculating the dynamic response, exploration should reach a depth of 50 m for tall structures or deep excavated foundations or to bedrock or a hard soil layer (where shear velocity $v_s > 500$ m/s) encountered within those depths. For example at Kalpakkam, 30 boreholes were drilled up to a depth of about 60 m for Prototype Fast Breeder Reactor (PFBR) site and 10 boreholes up to a maximum depth of 20 to 30 m were drilled for Spent Fuel Storage Facility (SFSF) site.
- For liquefaction assessment, exploration should go down to 30 m for power plants and nuclear structures. For example, several boreholes were drilled up to a depth of 30 to 40 m for Pragati power project site, New Delhi and sampling was carried out at closer spacing, mainly to access the liquefaction potential.
- For detecting suspicious faults or fracture zones, exploration should be extended to 50 m for tall or high-rise buildings or heavy structures; or to 30 m for common projects. For example, at Kalpakkam, two boreholes were drilled up to a depth of about 120 m for Prototype Fast Breeder Reactor (PFBR) site, and two boreholes up to a depth of 70 m were drilled for Spent Fuel Storage Facility (SFSF) site to find out the presence of faults or fractured zones below the proposed foundations of the nuclear structures.
- For detecting suspicious seismic settlements, exploration should be extended to a depth, $H = L/(2 \tan 30°)$, where L is the maximum horizontal length of the settled area.

Soil distribution and layer depth

Standard borehole drilling and sampling procedures are satisfactory for determining layer thickness for most seismic response analysis purposes as well as for normal foundation design. In the upper 15 m of soil, sampling is usually carried out at about 0.75 or 1.5 m intervals; from 15–30 m depth, a 1.5 m interval may be desirable; while below 30 m depth, 1.5 or 3.0 m may be adequate, depending on the soil complexity. If the site is prone to liquefaction, thin layers of weak materials enclosed in more reliable material may need to be identified. The depth to which the deepest boreholes are taken will depend, as usual, on the nature of the soils and on the proposed construction. For instance, for the design of a nuclear power plant on deep alluvium, detailed knowledge of the soil is required to a depth of perhaps 200 m, while general knowledge of the nature of subsoil will be necessary down to bedrock. The typical cross section of geological profile obtained from borehole logs for PFBR site, Kalpakkam[3] is given in Figure 1. The site consists of loose to dense sand of 8.0 m thickness, which is followed by firm to stiff clay of about 0.5 to 5.0 m thickness. The weathered rock occurs till a depth of about 12.0 to 15.0 m and the thickness varies from 1.0 m to 3.0 m. Hard rock is encountered at a depth of about 15.0 m to 20.0 m. Hard rock consists of charnockite, granite and gneiss with garnet crystals. Numerous samples were collected from sand and clay layers at 1.0 m intervals to carry out various laboratory tests to determine index and engineering properties of sand and clay deposits. The Standard Penetration Test (SPT) was carried out at more than 200 depth-locations in the above strata.

Bedrock depth

For carrying out ground response analysis, knowledge of the depth to bedrock is essential. Beyond the ordinary borehole depth of 50–100 m, depth to bedrock may be determined from geophysical refraction surveys. For example, seismic refraction survey was carried out at the atomic power plant site, Kudankulam during initial stage of the investigation and confirmatory stage on the excavated pit[2]. The initial stage of the investigation has revealed that the layer 1 is a highly weathered rock with *s*-wave velocity of 1100 m/s and layer 2 is a good rock having *s*-wave velocity of 1900 m/s. In confirmatory stage, layer 1 is semi-weathered rock with *s*-wave velocity of 650 to 1600 m/s having thickness of about 10.0 m and layer 2 is bed rock with *s*-wave velocity of 2500 to 2600 m/s. The seismic refraction survey at initial and confirmatory stages indicates a few zones having inferior rock but both the surveys did not indicate any undesirable surface features like major shear zone of a geological fault which might have posed foundation problem. However, the presence of calcareous zones was not identified during the seismic refraction survey at the initial stage of investigation. Seismic cross-hole

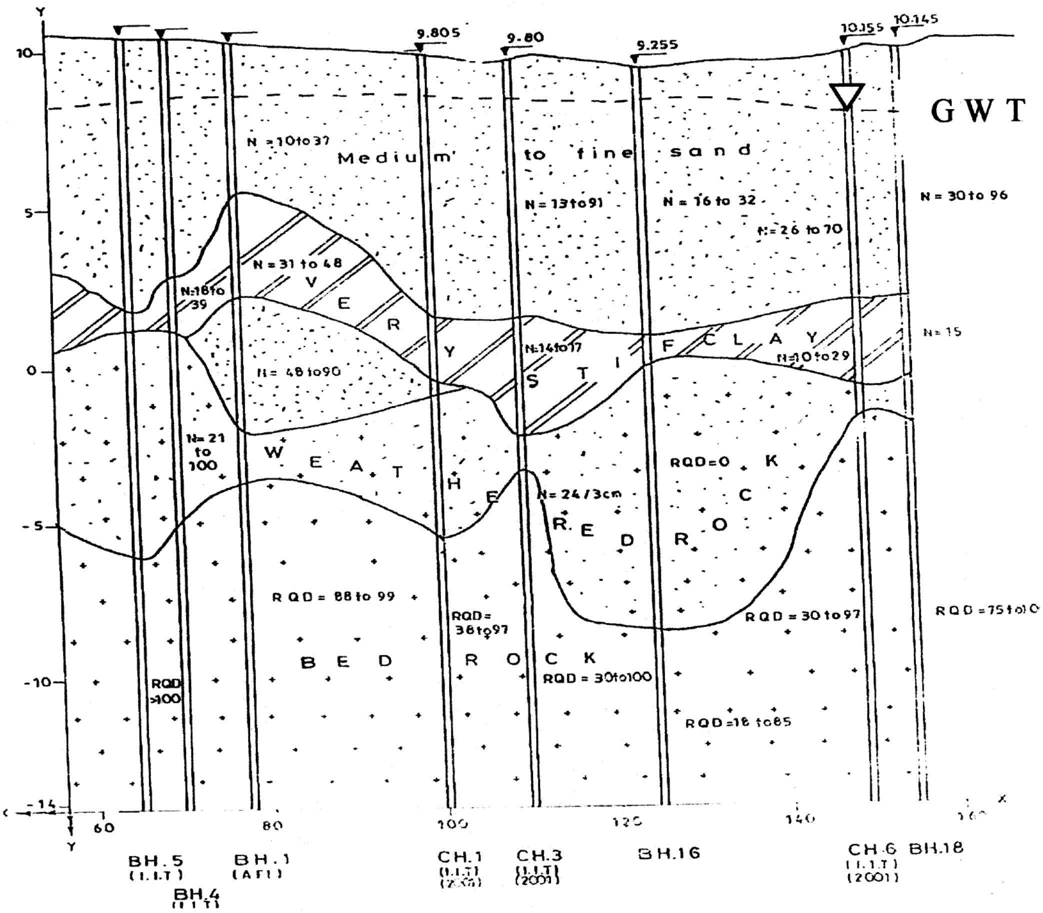

Figure 1. Typical geological profile at PFBR site, Kalpakkam.

Table 1. Order of accuracy of bed rock depth determination

Bedrock depth (m)	Approximate accuracy (m)
0–30	1.5
30–60	1.5–3.0
60–150	6–15
150–300	15–30
>300	60

tests and gamma logging have also been carried out at this site up to a depth of 15 m to determine seismic wave velocities and density of rock respectively. The order of accuracy of bedrock depth[4] determination to be used for realistic ground response analysis is given in Table 1.

In situ testing

Ideally, subsurface investigation must accompany the suitable *in situ* testing methods. However, for the seismic analysis of nuclear structures and power plants, some additional *in situ* tests are to be conducted in addition to the conventional *in-situ* tests such as SPT and SCPT, tests mainly to determine the dynamic soil properties. For dynamic response analysis, sampling and *in situ* testing must be carried out at least once in every homogeneous soil layer and up to the same depth as mentioned above. For more specific purpose, e.g. liquefaction potential assessment, undisturbed sampling, the SPT and the SCPT, etc. are normally undertaken simultaneously. Most of the *in situ* tests must aim to determine shear wave velocity (v_s) and compression wave velocity (v_p) for the calculation of the seismic response of ground and to evaluate the Poisson's ratio, modulus of elasticity and shear modulus of the soils to establish relationship among soil density, compressibility and rigidity and to be used as criteria for evaluating liquefaction potential of soils based on statistical relationships.

Measurement of shear wave velocity

Direct measurement of soil or rock stiffness in the field has the advantage of minimal material disturbance. In particular, there is a growing appreciation of measuring maximum shear modulus using seismic methods. There are several seismic wave propagation tests, namely, seismic cross-hole test, seismic down-hole test and spectral analysis of surface waves

(SASW) test. Among these tests, seismic cross-hole test is considered to be the most accurate for determination of wave velocities in the field.

The cross-hole technique measures shear wave velocities horizontally between adjacent boreholes and is well suited to response calculations of reasonably homogeneous or thick strata. The experimental setup used by IIT Madras for conducting the seismic cross-hole tests as per ASTM D-4428 (ref. 5) is shown in Figure 2. The tests are performed using three boreholes among which two receiver boreholes (R1, R2) are drilled in advance up to the required depth and one source borehole (S) drilled during the time of testing. Seismic cross-hole tests are carried out in PVC or aluminium cased boreholes of 150 mm diameter in soil overburden and weathered rock. An ordinary cement slurry grout or loose sand is used to fill up the gap between the casing and soil. The tests are carried out in 'NX' size boreholes in hard rock without casing. The center-to-center distance between the boreholes is usually 4.0 m. Blows on a standard penetration test hammer on a cone are used as a source for impulse in the source borehole. A borehole pick consists of three component acceleration transducers placed in the receiver borehole at the desired depth using the packer system to detect the arrival of waves traces. The waves signals sensed by transducers are magnified and recorded using a Data Acquisition System (DAS) consisting of HBM make multi-channel carrier frequency amplifier system and a digital storage oscilloscope. The seismic wave signals detected by DAS and digitized data are stored using a COMPAQ make laptop for further analysis. A typical wave trace recorded during the execution of cross-hole tests at Konaseema combined cyclic power plant, Ravulapalem, Andhra Pradesh[6] is shown in Figure 3. Measuring the travel time for arrival of shear wave and knowing the distance between the receiver boreholes, the shear wave velocities are computed.

Wave propagation characteristics obtained from the seismic cross-hole test carried out at nuclear structures site are used not only to carry out dynamic response analysis, but also to find out the presence of fracture zones and faults in rock strata. Seismic cross-hole tests performed at a typical rock site at nuclear island (PFBR), Kalpakkam are discussed herein[3]. The tests were carried out at an interval of 1.5 m up to a depth of 65 m and the *P*- and *S*-wave velocities are determined at different depths. A typical *P*- and *S*-wave velocity profile is shown in Figure 4. The Poisson's ratio and shear modulus of different layers estimated from velocities are presented in Table 2.

Seismic cross-hole tests were also carried out by IIT Madras at Pragati power project site, New Delhi[7] at 7 locations up to a depth of 30 m. The soil profile at the site consists of fly ash layer 15 m thick followed by sand and silty clay 9 m thick underlain by clayey silt layer. A typical *S*-wave velocity profile obtained from cross-hole tests is shown in Figure 5. The shear wave velocity ranges from 100 to 167 m/s in the top fly ash layer, 167 to 222 m/s in the middle sandy silty layer and 222 to 333 m/s in the clayey silt layer.

Seismic site characterization

One of the major problems in geotechnical engineering is the risk of encountering unexpected geological conditions such as sudden variation in the soil profile, rock strata, failure planes and faults in the rock, etc. Failure to anticipate such conditions generally is due to an inadequate geological understanding of the site and may lead to issues concerning

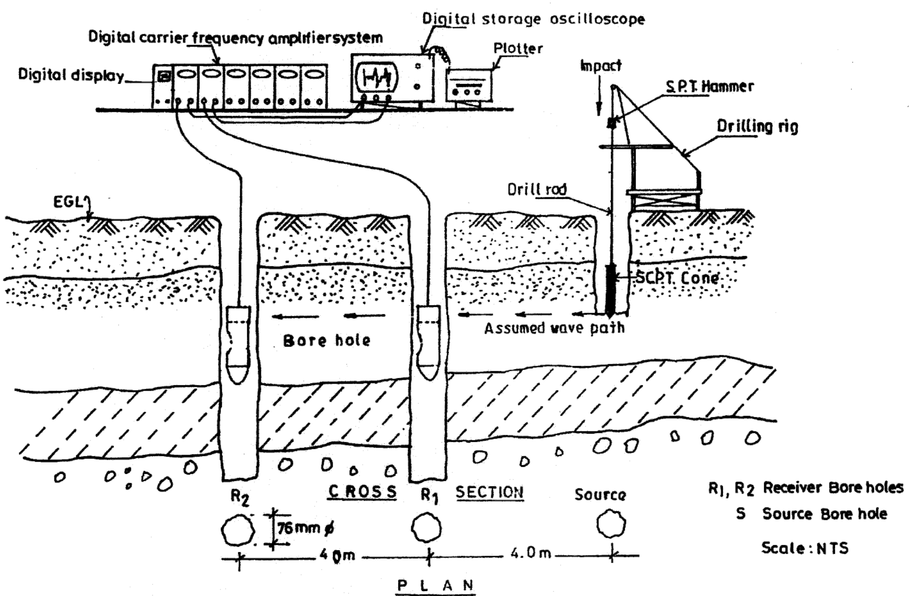

Figure 2. Typical experimental setup for seismic cross-hole test.

design and performance of critical facilities such as nuclear structures and power plants. Hence, it is very important that rigorous geological and geophysical analyses accompany the extensive geotechnical investigations to understand the behaviour of the site to seismic loads. The various methods of seismic site characterization and seismic analysis are discussed herein.

Site classification

The site classification and its level of response will depend heavily on the correspondence between the site periods and the periods of the incoming motion. The period and response to a given input motion depend on the thickness of layer and the shear wave velocity. The time-averaged shear wave velocity in a constant thickness layer comprising materials in the top 30 m of a site is taken as an index of potential site amplification. The US National Earthquake Hazards Reduction Program (NEHRP) for characterizing sites for code purposes has adopted this procedure. The classification of sites based on average shear wave velocity is given by Federal Emergency Management Agency, FEMA (1997) and Uniform Building Code, UBC (1997)[8,9]. The UBC site classification is given in Table 3.

The average shear wave velocity of the layers is determined by eq. (1)[9]:

$$\overline{v_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}, \tag{1}$$

where d_i = thickness of layer i in meters and v_{si} = shear wave velocity in layer i in m/s.

At PFBR site, Kalpakkam, the nuclear raft is to be located at a depth of about 15 m below the ground level. Hence, the average shear wave velocity of the site for 30 m depth below the founding level of raft is estimated as about 1500 m/s. Therefore this site is classified as a rock site (S_B) as per UBC (1997). The geological characterization of the hard rock has been assessed and the Rock Mass Rating (RMR) value of the hard rock is found to be 63. It is classified as Class II (Good rock) as per Bieniawski classification system[10].

The shear wave velocity for various layers below the founding level of reactor building foundation arrived from the confirmatory cross-hole test at Kudankulam power plant

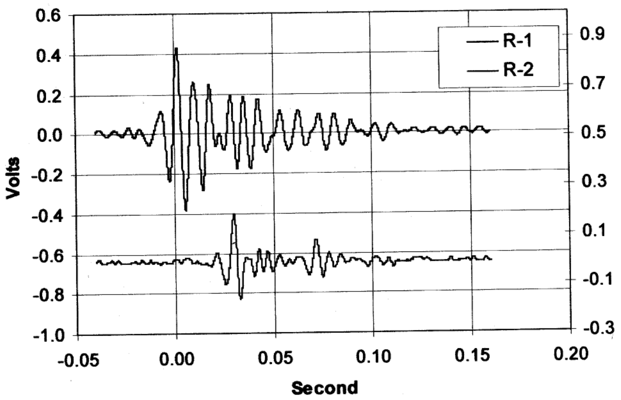

Figure 3. Typical wave traces obtained from seismic cross-hole test carried out at Konaseema Power Plant Site.

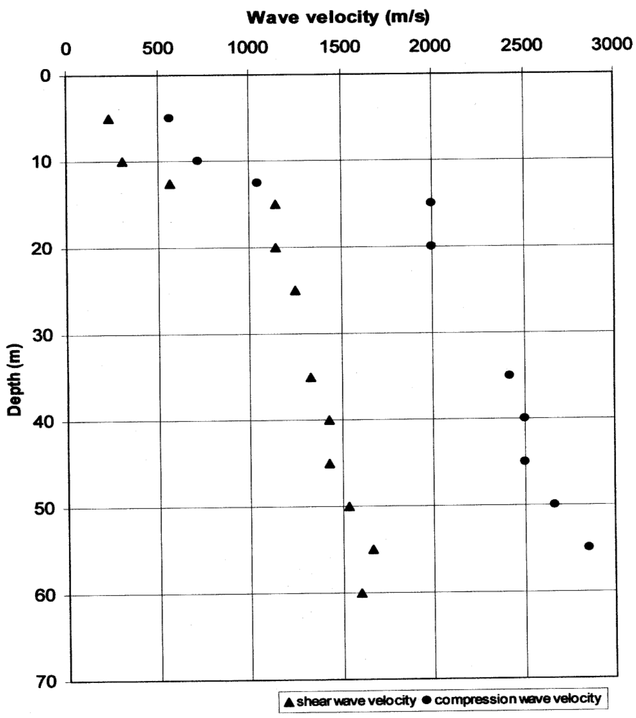

Figure 4. Typical *P*- and *S*-wave velocity profile at PFBR site, Kalpakkam.

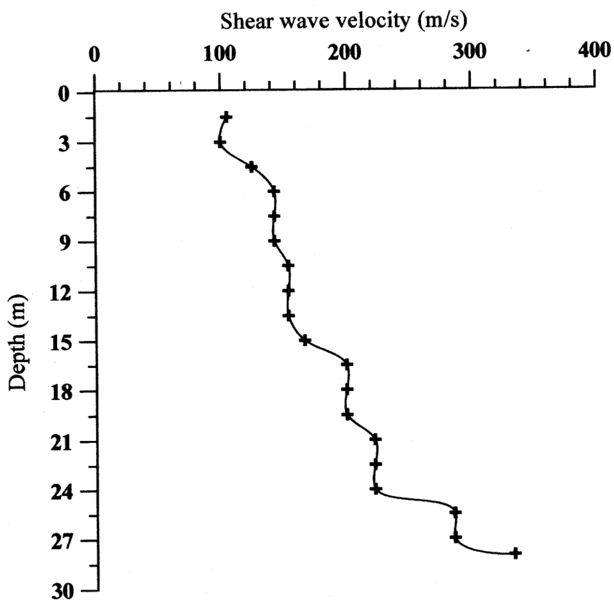

Figure 5. Typical *S*-wave velocity profile at Pragati power plant site, New Delhi.

Table 2. Dynamic properties of various layers at Nuclear Island, Kalpakkam

Type of stratum	S-wave velocity (m/s)	P-wave velocity (m/s)	Poisson's ratio	Shear modulus, G_{max} (MPa)
Sandy soil	167–285	470–570	0.33–0.43	53–154
Clay	180–380	420–1025	0.38–0.42	63–280
Weathered rock	571	1052	0.29	718
Hard rock	1142–1667	2000–2857	0.24–0.28	3725–7556

Table 3. UBC site classification

Soil profile type	Soil profile name/ generic description	Average shear wave velocity, v_s, for upper 100 feet of soil profile, ft/s (m/s)
S_A	Hard rock	> 5,000 (1500)
S_B	Rock	2,500 to 5,000 (760 to 1500)
S_C	Very dense soil and soft rock	1,200 to 2,500 (360 to 760)
S_D	Stiff soil	600 to 1,200 (180 to 360)
S_E	Stiff soil	< 600 (180)
S_F	Soils requiring site-specific evaluation	

Table 4. Shear wave velocity (v_s) of different rock layers at Reactor Building site, Kudankulam

		Average thickness (m)		Average v_s (m/s)	
Layer	Type of rock	RB-1	RB-2	RB-1	RB-2
I	Highly weathered rock (W-IV)	4.5	3.5	585 to 683	300 to 600
II	Moderately weathered rock (W-III)	10.0	10.0	803 to 1051	650 to 1500
III	Fresh rock (WII/WI)	Average level is at 15 m from GL		2500 to 2600	

site is given in Table 4. The average shear wave velocity at the location of reactor building RB-1 is about 842 m/s and at the location RB-2 is about 1350 m/s. Hence, as per UBC (1997), both locations RB-1 and RB-2 at this site are classified as rock sites (S_B).

Period of vibration of site

The natural frequencies or periods of vibration of any dynamical system comprise a fundamental indicator of the dynamic response characteristics of the system. In case of soil systems, the site period for shallow soil deposits are usually less than 0.6 s and for deep soil sites, greater than 0.6 s. However, considering a stratum of uniform thickness H, the period of vibration for any mode is given by eq. (2) (ref. 4):

$$T_n = \frac{4H}{(2n-1)\,v_s}, \qquad (2)$$

where n is an integer (1, 2, 3 . . .) and v_s is the mean shear wave velocity in the layer and a function of stiffness and density. The fundamental period of the site, corresponding to $n = 1$, is $4H/v_s$, which occurs when a shear wave of wave length passes through and is reflected in the stratum, while the larger integers correspond to the higher harmonics.

For PFBR site, Kalpakkam using the average shear wave velocity of top 30 m depth, the fundamental period of the site worked out using eq. (2) is 0.08 s. But, in the Kudankulam atomic power project site, the fundamental period is 0.071 s and 0.044 s for the locations RB-1 and RB-2 respectively. Though, both Kalpakkam and Kudankulam sites are classified as rock sites (S_B) as per UBC (1997), the fundamental period of the sites is quite different that will affect the seismic response of the site significantly.

Dynamic soil properties

The behaviour of soils under cyclic or dynamic loading plays a crucial role in the determination of extent and distribution of ground displacements, which in turn are directly related to the potential for causing seismic damage. Of importance are stress–strain behaviour, i.e. moduli of soil and damping characteristics under dynamic loads. In general, different levels of induced strain require consideration of different types of material models for realistic analysis. This necessitates the determination of design values of modulus and damping characteristics as a function of strain to study and simulate the nonlinear behaviour of soil under dynamic or seismic loading. For seismic soil–structure interaction problems, the modulus of soil is often represented by shear modulus of soil.

Since the seismic geophysical tests induce shear strain lower than about 3×10^{-4}%, the measured shear wave velocities can be used to compute the maximum shear modulus, G_{max} as in eq. (3) (ref. 11):

$$G_{max} = \rho v_s^2 . \quad (3)$$

For many applications, it is essential to develop the variation of modulus reduction (G/G_{max}) with shear strain level, which is important for carrying out ground response analysis and for selection of design parameters for important foundations/structures subjected to dynamic loads. In order to plot a modulus reduction curve, the modulus values determined from various dynamic tests at different mean effective confining pressure can be converted to a standard mean effective confining pressure of 1 MN/m^2 using the eq. (4) (ref. 12):

$$\frac{G_1}{G} = \left(\frac{\sigma'_{01}}{\sigma'_0}\right)^{0.5}, \quad (4)$$

where G_1 and G are dynamic shear moduli for the standard mean effective confining pressure (σ'_{01}) of 1 MN/m^2 and for test mean effective confining pressure (σ'_0) respectively. Then the variation of dynamic shear modulus (G_1) versus magnitude of shear strain (γ_θ) is plotted.

Dobry and Vucetic[13] established modulus reduction curves for fine grained soils of different plasticity as shown in Figure 6 and concluded that the shape of reduction factor, i.e. degradation of the stiffness of soil is influenced more by the plasticity index (PI) than by the void ratio. These curves show that the linear cyclic threshold shear strain (γ_{tl}) is greater for highly plastic soils than for soils of low plasticity. This is an important characteristic that can strongly influence the manner in which a soil deposit will amplify or attenuate earthquake motions. When PI = 0, the modulus reduction curve proposed by Dobry and Vucetic[13] is similar to that proposed by Seed and Idriss[14] which is commonly used for sands.

The hysteric-damping ratio increases with increasing strain amplitude. At low strain levels, negligible dissipation of energy takes place and the damping is influenced by plasticity characteristics. Damping ratios of highly plastic soils are lower than those of low plasticity soils at same cyclic strain amplitude. The seismic geophysical tests do not provide the realistic determination of material damping, which could be determined accurately by laboratory tests. For the preliminary analysis, damping ratio vs. shear strain curves developed by Vucetic and Dobry[15] given in Figure 7 can be used.

Although the ground response analysis can be carried out using standard modulus reduction and damping curves, it is preferable to use site-specific modulus reduction and damping curves based on the *in situ* and laboratory tests. For example, at PFBR site, Kalpakkam, shear modulus of soil at different strain levels was determined through various field dynamic tests: low strain modulus ($\leq 10^{-4}$ % strain level) was obtained from seismic cross-hole test[3], modulus at intermediate strain level (10^{-4} to 10^{-3} %) from block vibration test and modulus at high strain levels (10^{-3} to 10^{-2} %) from cyclic plate load test. Modulus at different strain levels was normalized to the maximum shear modulus obtained from seismic cross-hole test and the variation of modulus ratio (G/G_{max}) with strain is shown in Figure 8. The variation of damping ratio with different strain level is also shown in Figure 8. However, these field tests are not adequate to establish more accurately the variation of modulus at all strain levels and hence laboratory tests such as resonant column and cyclic triaxial tests need to be carried out to determine modulus at medium and high strain levels.

Seismic soil–structure interaction

Seismic soil–structure interaction has the following beneficial effects: (a) the period and damping of the vibrating

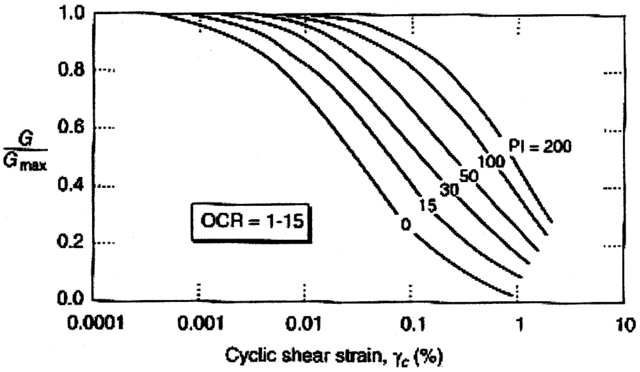

Figure 6. Standard modulus reduction curve[13].

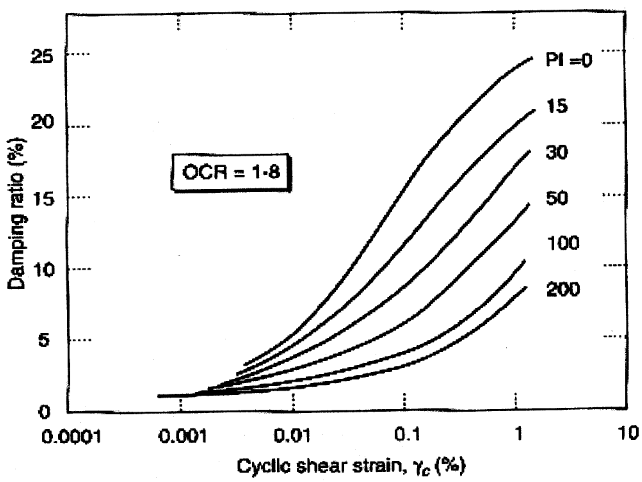

Figure 7. Standard damping curve[15].

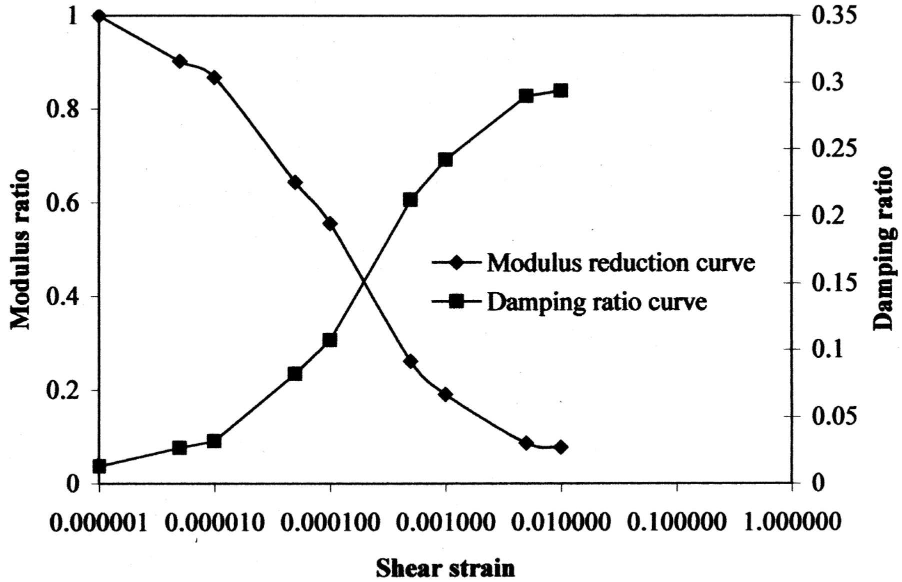

Figure 8. Modulus reduction and damping curves developed for PFBR site, Kalpakkam

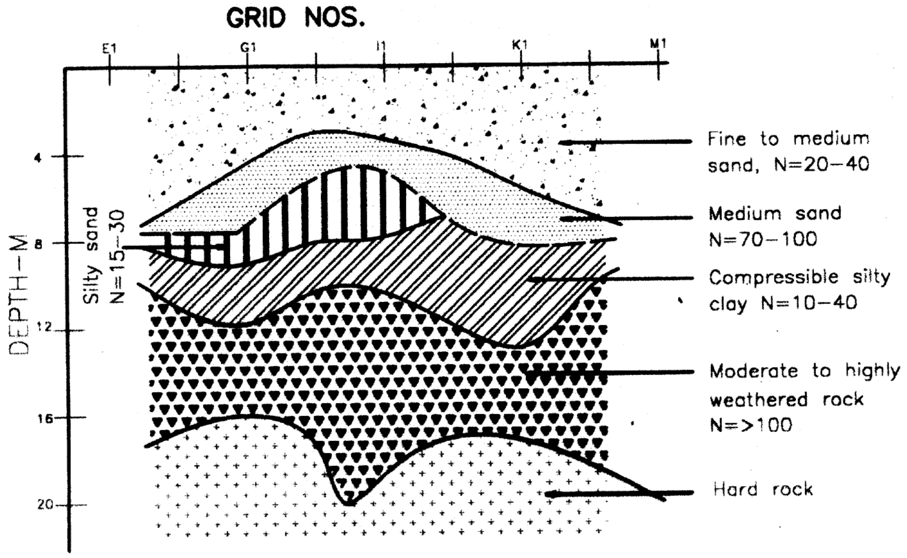

Figure 9. Typical soil profile at Kalpakkam site adopted for liquefaction analysis.

structure are increased; (b) energy can flow from the vibrating building back into the soil; (c) uplift of foundation slab reduces forces transmitted to building; (d) large foundation slabs can reduce the high frequency motions and hence reduce the input motions to the structure; but, (e) rocking increases the relative displacements and hence can increase the overturning moments (called the P–δ effect). Perhaps the leading question to be answered about soil–structure interaction is: 'For what soil conditions will the rigid base assumption lead to significant errors in the response calculations?' Veletsos and Meek[16] suggested that consideration of soil-structure interaction is only warranted for values of the ratio $v_s/f \times h$, less than 20 (ref. 16). Here, v_s is the shear wave velocity in the soil half-space, f is the fixed-base frequency of the SDOF structure and h is its height. Substituting $f \approx 30/\mathrm{h}$ for framed buildings, and $f \approx 45/\mathrm{h}$ for shear wall buildings in the above equation implies that soil–structure interaction effects may be important for framed buildings when $v_s \leq 600$ m/s and for shear wall buildings when $v_s \leq 900$ m/s[4]. However, while carrying out the dynamic analysis as per the recommendations of ASCE 4–98 (ref. 17), a parametric soil–structure interaction analysis is to be carried out by considering three design shear moduli (G_d) values, i.e. $G_d = G$, $G_d = 0.5G$ and $G_d = 2G$.

The average shear wave velocity of PFBR site, Kalpakkam and Kudankulam Atomic Power Project sites is greater than 600 m/s. According to Veletsos and Meek[16], the soil–structure interaction is not very important for both Kalpakkam and Kudankulam power plant sites. However, seismic soil–structure interaction analysis had been carried out at both the sites as per the ASCE 4-98 recommendations.

Liquefaction analysis

Liquefaction is the one major phenomenon that occurs during earthquake, which could cause severe damages to various lifeline facilities and civil structures. In view of this, the evaluation of liquefaction potential becomes much more essential. In engineering practice, the liquefaction analysis is carried out by SPT-method or shear wave velocity method by using liquefaction assessment charts, where the correlation between the liquefaction resistance and *in situ* penetration resistance (SPT and SCPT) and shear wave velocities are readily available. The effective average cyclic shear stress induced during an earthquake can be obtained by simplified approach or by carrying out ground response analysis by equivalent linear approach using SHAKE program[18]. The factor of safety against liquefaction is worked out to find the liquefaction susceptibility.

The shear wave velocity method has been used for the liquefaction analysis of PFBR site, Kalpakkam[19]. The soil profile used for the liquefaction analysis is shown in Figure 9, which consists of sandy and silty sand layer up to 8.0 m, which is followed by silty clay and weathered rock stratum. The bedrock occurs at a depth of about 15.0 m. The water table is close to the ground level. The shear wave velocity increases for the top sands and silty sand layers from 208 to 313 m/s. The Cyclic Stress Ratio (CSR) induced by earthquake has been computed by carrying out one-dimensional ground response analysis using SHAKE-91. The modulus reduction and damping curves (Figure 8) developed for the site are used. The acceleration time history of the site specific Earthquake of Magnitude 6.0 (Figure 10) is used as input ground motion[20].

Cyclic Resistance Ratio (CRR) is determined at various depths using the correlation between CRR and the shear wave velocity[21]. The variation of cyclic shear stress induced by earthquake and average cyclic shear resistance is given in Figure 11. It can be easily observed from Figure 11 that the cyclic shear resistance at all depth is much greater than cyclic shear stress induced by earthquake and hence the layers will not liquefy for the given site-specific earthquake motion. The factor of safety against liquefaction, i.e. the ratio of CRR to CSR as defined by Ishihara[22] is above 1.6 at all the depths and hence the PFBR site will not liquefy for the design ground motion considered.

Conclusions

Confirmatory geological and geotechnical investigations carried out after excavation of strata to the founding level at various sites for nuclear facilities show the presence of weaker zones which have not identified in the original investigation. Therefore geological and geotechnical investigations shall be well planned and executed by reputed agencies at the beginning stage of the investigation.

Application of consolidation cement grouting at the founding rock strata with joint and calcareous material in the discontinuities improves the monolithic behaviour of founding rock mass due to cementation.

It is preferable to carry out seismic cross-hole tests for accurate measurement of *P* and *S* wave velocities of various soil and rock strata that can be used for ground response analysis, seismic soil–structure interaction analysis and liquefaction analysis for nuclear structures and power plants successfully. Seismic cross-hole test data also enables us to identify the presence of faults, fracture zones and weaker zones. However, this technique cannot be used for estimating the material damping of different strata.

Site-specific modulus reduction and damping curves must be used to carry out ground response analysis and seismic soil–structure interaction analysis. Commonly used field tests are not adequate to establish variation of modulus

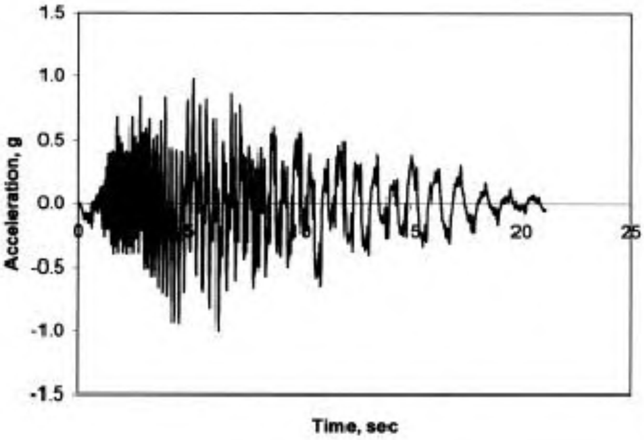

Figure 10. Input design ground motion.

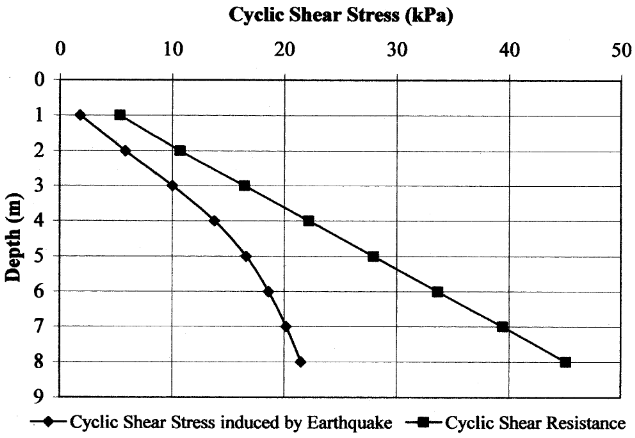

Figure 11. Variation of cyclic shear stress induced by earthquake and cyclic shear resistance with depth.

at all strain levels and hence laboratory tests such as resonant column test, cyclic triaxial test, etc. are also required for the determination of modulus at medium strain levels.

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