Indian Geotechnical Journal, 35 (2), 2005

Comparison of Free Field Ground Response in Layered Soil under Liquefiable Conditions

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Introduction

Earthquakes are one of the major natural hazards to human life and infrastructure. According to the recent reports there are on an average about 18 earthquakes of magnitude (M) 7.0 or larger worldwide each year. It is also estimated that average loss per year is about 1.0 billion USD (NSF 2003) due to earthquakes. The seismic wave fields that are generated by an earthquake are extremely complex and have large random and spatial variations, which cause an irregular pattern of damage in most earthquakes. For example during the 2001 Bhuj earthquake certain areas of Ahmedabad were more severely affected than others (Madabhushi et al., 2004). This is routinely attributed to the characteristics of the earthquake source, seismic wave propagation, type of structure, quality of construction, water table conditions and the site response. The modification of amplitude, frequency content and duration of ground motion as it propagates from bed rock through overlying soil to the ground surface is a typical manifestation of site response. Site response is due to the fact that that the characteristics of free field motion induced by a seismic event at a given site are functions of the property and geological features of the subsurface soil and rock. One dimensional ground response analysis is often used to obtain site-specific response spectra for seismic analysis of important structures. This type of analysis consists of the following steps:

1. Characterisation of the soil site: This is often based on the results of the laboratory and the field investigations for the site in question. An idealised soil profile is adopted based on the above results. Site

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characterization often includes the determination of dynamic soil properties for each layer at the site.

2. Selection of appropriate rock motions: Appropriate rock motions are developed or selected to represent the design rock motion at the site. In most cases the rock motion is assigned to a hypothetical rock outcrop at the site rather than at the base of the soil profile itself. This is essentially due to the fact that the recordings at rock outcrop are usually measured at the rock itself and unless the rock is rigid the motions at the base of the soil profile will be different from the rock outcrop motion.

3. Ground response analyses and development of ground surface response spectra: The rock time histories are used as input motions and one dimensional ground response analyses is conducted for the modelled ground profile to obtain the motions at the surface of the ground. Mathematically the problem is solution of one dimensional wave propagation in a continuous medium. Non-linear soil response is approximated either by equivalent linear methods or truly non-linear methods. The results can be used for calculating site natural time periods, assessing ground motion amplifications and providing structural engineers with various parameters for developing response spectra for design and safety evaluation of structures.

These steps are usually followed for the ground response analysis to account for site effects. Additionally soil structure interaction effects incorporate the fact that the dynamic response of a structure built on that same site depends in addition on the interrelationship of the structural characteristics and the properties of the local underlying soil deposits. Thus the dynamic behaviour of structures will change due to the site response as well changes due to soil structure interaction during shaking. There are two effects happening simultaneously in an interaction problem. The motion experienced at the base of the structure is greatly modified from the free field motion that will occur in absence of the structure. These, in many case includes, a rocking component in addition to a lateral or a translational component. This interaction is further complicated when the ground on which the structure is founded is made of loose saturated sandy/silty soil which are mostly liquefiable. These soils lose their shear strength and behave like a liquid for a short period of time during an earthquake. Following liquefaction the bearing capacity of the soil is sharply reduced and the building foundation may suffer excessive settlement and rotation. In many of past earthquakes ground failures involving soil liquefaction have resulted in the tilting and collapsing of buildings with the superstructure remaining intact as seen in Fig.1.

FIGURE 1 : Tilting of the Foundation with the Superstructure Remaining Intact at the Koaceli Earthquakes in 1999

Previous earthquakes like Bhuj (2001), Kobe (1995), Northridge (1994), and Loma Prieta (1989) have also depicted the role of local site conditions in modifying and changing the characteristics of strong motion data. Different amount of structural damage has been reported in the same general area depending upon the local site variations. Liquefaction adds further complexity to the problem due to the softening of the soil deposit. The onset of liquefaction alters the ground motion, and can lead to progressive attenuation of the high-frequency components in the ground motion transmitted to the ground surface. This phenomenon has been observed in the field (Zeghal and Elgamal, 1994) and corroborated by centrifuge tests for homogeneous loose soil. Tokimatsu et al. (1996) concluded that local site effects including those resulting from soil liquefaction was responsible for reducing the damage to superstructures particularly located near coastlines in the Kobe earthquake. In stratified soil the role of these attenuations is not very clear.

Different analytical methods have been proposed for the numerical analysis of such dynamic induced liquefaction of soil materials leading to these types of failures. A numerical technique requires verification and validation by comparison of its predictions with observed full scale in situ field performances for developing confidence in its usage for design purposes. The possibility of testing full-scale structures under earthquake loading is very rare. Hence dynamic centrifuge modelling is often used for checking the accuracy of these numerical models. This was the primary objective of the NSF sponsored VELACS (Arulanandan and Scott, 1993) project in the USA which brought numerical and physical modellers together. But most of these tests have been carried out on homogenous soil layers where the natural variability of the ground has not been taken into account. Very few calibration studies exist for inhomogeneous ground conditions as found routinely in the site.

In the present paper a comparison will be presented between different site response analysis methods for a benchmark problem. This problem consists of a layered soil profile as shown in Fig.2, where the free field motion will be predicted for the same bedrock motion using software's incorporating different soil models and methods of analysis. The free field motion at the surface was also measured during the centrifuge tests. Therefore it will be possible to directly compare the numerical prediction by various methods with the experimental data. These methods of prediction differ in the simplifying assumptions that are made, in the representation of the stress strain relation of soil and the methods used to integrate the equations of motion. The software's used are based on decoupled methods and fully coupled methods. In these comparisons the free field response for a layered soil profile will be predicted by using three independent softwares like EERA (Equivalent Linear Earthquake Site Response Analysis) developed by Bardet et al. (2000). CYCLIC 1D is an internet-based non-linear finite element program for execution of one-dimensional site amplification and liquefaction simulations (for level as well as gently inclined sites) developed by Elgamal and Yang (2001). 2D finite element code SWANDYNE (Chan, 1988) will also be used to model the benchmark problem of layered soil subjected to earthquake loading. In the next section the centrifuge test results will be discussed briefly.

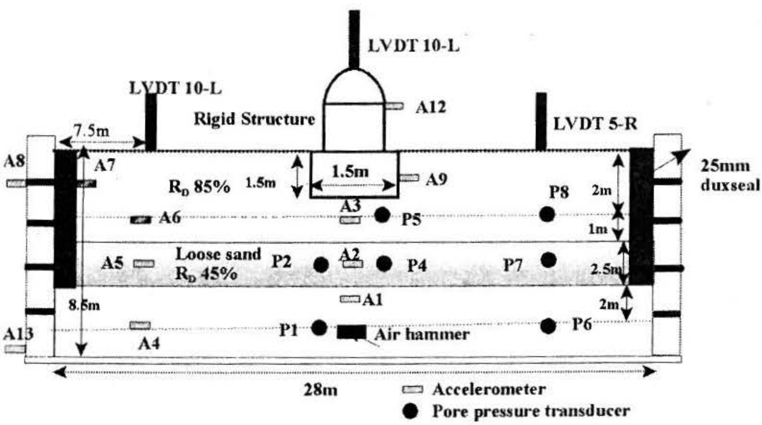

FIGURE 2 : Centrifuge Test Arrangement for Testing the Behaviour of the Structure in Layered Soil

Centrifuge Modelling

Centrifuge modelling is based on the principle of creating reduced scale models of geotechnical structures with stresses and strains at homologous points in the prototype and model structures being identical. As soil is a highly non-linear material, it is important for realistic behaviour to be observed that the stresses in the model are the same as those in the prototype. Together with a set of scaling laws (Schofield, 1980) that can be derived from this condition the model behaviour can be interpreted to give the behaviour of the full-scale or prototype structure. Madabhushi (2004) describes the application of centrifuge modelling to earthquake problems. Figure 2 shows the general arrangement for the centrifuge tests performed on the beam centrifuge at Cambridge University (U.K.) and reported in this paper. The dimensions shown in this figure are presented in prototype scale. The soil is finely graded laboratory Fraction E silica sand whose properties are reproduced on Table 1, enclosed in an ESB (Equivalent Shear Beam) container, which matches the stiffness of the end wall with the stiffness of soil column (Zeng and Schofield, 1996) during shaking. In these tests to avoid the reflection of the stress waves a highly plastic material duxseal was used at the sides of the container as seen in Fig.2. Duxseal is similar to plasticine and has a density of about 1800 kg/m^3. Madabhushi (1991) conducted 1g experiments to investigate the effectiveness of duxseal and showed that it was able to absorb about 65% of incident waves. The use of duxseal can simulate the free field radiation damping condition to a certain extent. Based on the radiation condition, at a sufficient distance from the source only outgoing waves are present, no incoming waves propagating from infinity towards the structure exist. Figure 3 compares the accelerations

TABLE 1 : Properties of Fraction E Silica Sand

Property	Value
D_{10} grain size	0.095 mm
D_{50} grain size	0.14 mm
D_{90} grain size	0.15 mm
Specific gravity G_s	2.65
Minimum void ratio	0.613
Maximum void ratio	1.014
Permeability (m/s)	9.8×10^{-5}
Critical angle of friction	32°
Shear wave velocity measured in dense soil	240 m/s
Shear wave velocity measured in loose soil	120 m/s

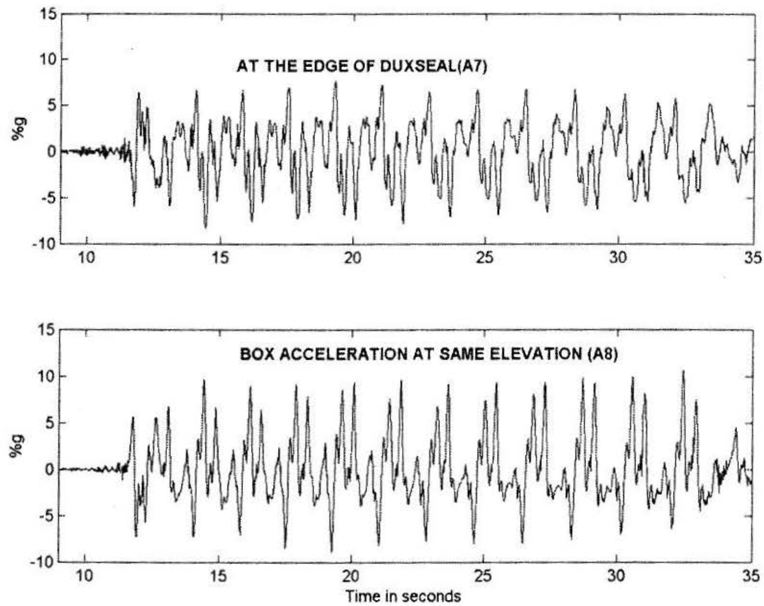

FIGURE 3 : Effectiveness of Duxseal as an Absorbing Material

measured at the same elevation by two accelerometers A7 and A8 (Fig.2) and it is seen that the duxseal layer has the ability to absorb some of the reflected waves from the boundary by comparing the magnitudes of measured accelerations.

The soil was saturated using silicone oil having a viscosity of 50 cS to correctly model the rate of generation of excess pore water pressure and rate of dissipation. The overburden consists of a rigid containment structure similar to the pre-stressed containments for nuclear power plants slightly embedded in the soil and applying a bearing pressure of 150 kPa at 50g. Test BG-04 consisted of a loose layer having a prototype thickness of 2.5 m deposited (R_D 45%) uniformly and sandwiched between dense layers having a R_D of 85%. The instrumentation for this test include miniature accelerometers to measure accelerations, pore pressure transducers to measure excess pore pressures and LVDT's to measure settlement. Previous research (Teymur and Madabhushi, 2003) has established that the region which is more than 150 mm (7.5 m on prototype scale for a 50g test) from the boundary of the ESB box can be considered to be free of any boundary effects. So most of the instrumentation was placed away from this zone of influence from the boundary as seen in Fig.2. The model was prepared by air pluviation of Fraction E silica sand whose properties are shown in Table 1. Different densities were achieved by varying the rate of pouring. The total depth of the prototype was 8.5 m. The sand was poured up to a depth of 1.5 m and then the air hammer (Ghosh and Madabhushi, 2002) was placed carefully in the model. The air hammer is a small actuator, which is used as a source to generate waves within the soil model. The propagation of shear waves through a model soil profile was measured in flight using an array of vertical accelerometers at different centrifugal accelerations in liquefiable soil. The values of the shear wave velocity measured were used in characterizing the soil layers for their evaluation of one-dimensional property and are shown in Table 1. These values were calculated at 50g at a depth of 6 m for the dense sand and 4.25 m for the loose sand. These experimentally measured values were used in modelling the ground response in the free field.

Test Result and Discussion

The tilt and rotation of the foundation after seismic shaking are considered as performance criteria for raft foundations. It was seen that the tilt and rotation of the foundation was reduced significantly as the founding soil was densified. The final settlement of the superstructure was 200 mm at prototype scale. A series of earthquakes were fired by using the SAM actuator (Madabhushi et al., 1998). These earthquakes are single frequency events with sinusoidal input motion. Figure 4 presents the accelerations recorded

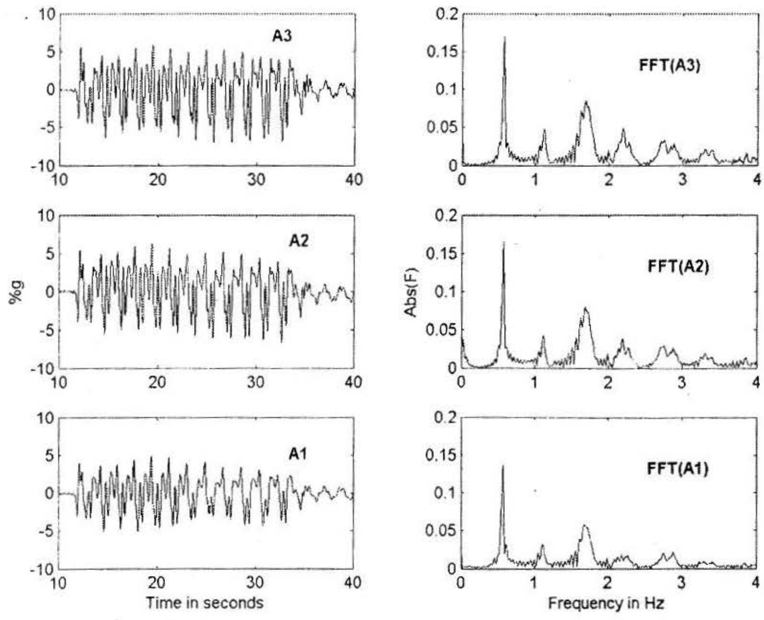

FIGURE 4 : Recorded Accelerations for Test BG-04 Underneath the Raft for Small Strength Earthquake

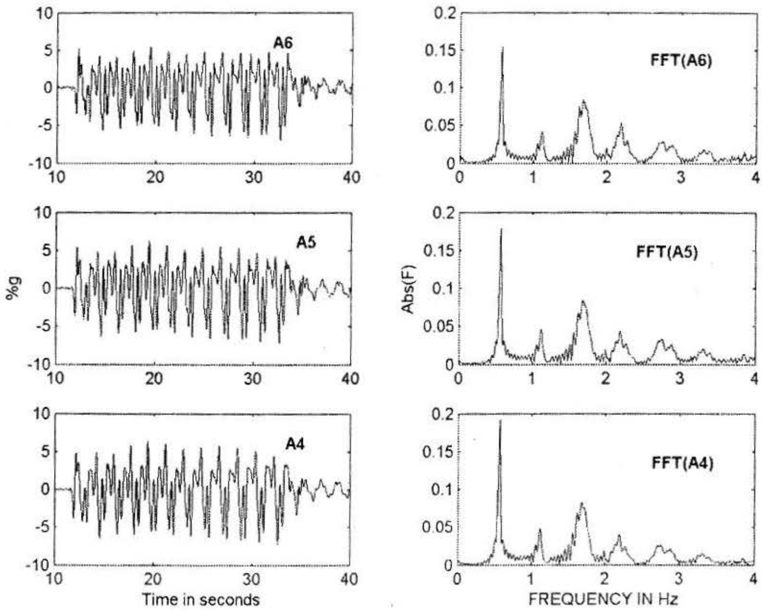

FIGURE 5 : Recorded Accelerations for BG-04 in the Free Field for Small Strength Earthquake

underneath the raft foundation by the accelerometers A1, A2 and A3, which are located at the depth of 6 m, 4.25 m and 2.0 m from the surface as seen in Fig.2. The FFT transformations of the input signal are also plotted in the same figure for test BG-04. The main driving frequency of the earthquake was 0.6 Hz, with a pga of about 0.1g at the bed rock, and had a duration of 25 seconds in prototype scale. It is seen that amplification occurs at small to medium peak accelerations throughout and higher harmonics are amplified by about 33%.

Figure 5 presents the traces of the accelerations measured in the free field for the same earthquake. It is seen that for shallow depth the motion at the free field is similar to the motion under the building (A6 and A3). This behaviour is dependent on the strength of the earthquake and the stiffness of the soil supporting the foundation. At the layering interfaces these motions are different indicating significant interaction due to the flexibility of the soil in the layered loose zone.

As the strength of the earthquake is increased and a strong earthquake is fired (pga 0.15g) acceleration traces show that the entire loose sandwiched layer had nearly liquefied in test BG-04 and the transmission of the shear waves is significantly reduced as seen in Fig.6. In these locations initial

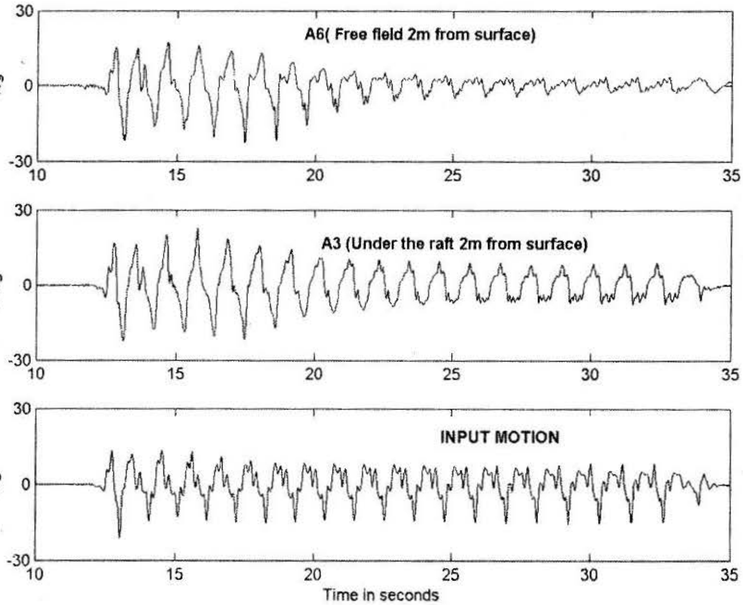

FIGURE 6 : Acceleration Transmission through Dense and Loose Soil in BG-04 with a Sandwiched Loose Layer being Liquefied after Strong Shaking

liquefaction is assumed to occur when the excess pore pressures reach the initial effective stress at that level. The excess pore pressure ratio reached for P7, which is located in the middle of the loose layer at the free field is very close to 1 indicating very low effective stresses by the end of the shaking period and conditions close to initial liquefaction as seen in Fig.7. Under the structure none of the pore pressure measurements reached their initial vertical effective stress. The presence of the structure created a sustained static shear stress in the soil and thus has a significant effect in the pore pressure build up. Accelerations in the loose sand layer show progressive and dramatic overall de-amplification of the earthquake motion. At the beginning of shaking cycle motion is amplified as it is transmitted through the dense soil in the first few cycles. The high frequency components are filtered out as the motion is propagated to the surface. It is seen in Fig.6 that the loose layer under the high overburden has softened considerably but was still transmitting accelerations whereas the free field had liquefied. The phenomena can be compared with the mass spring system transmitting motion from the base. In the initial stages the spring is stiff enough to transmit all the frequencies. The progressive degradation of the soil stiffness due to excess pore pressure generation and the cyclic shear strain amplitude can soften the spring. The system may undergo resonance at lower harmonics while the higher harmonics are being attenuated. At the final stage the spring can become so

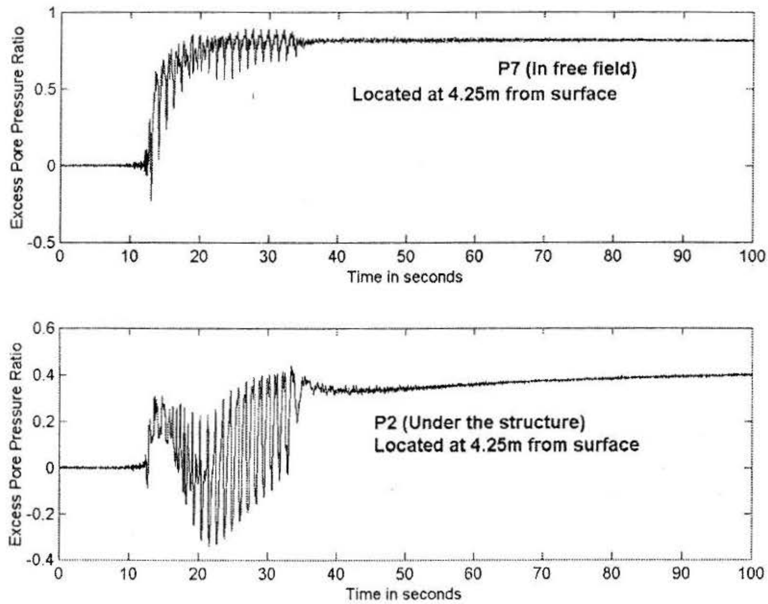

FIGURE 7 : Pore Pressures Measured throughout the Model for Medium Strength Earthquake

flexible that fundamental earthquake frequency band cannot be transmitted and this corresponds to the full liquefaction of the soil. This phenomenon is similar to the isolation mechanism observed in the Kobe earthquake. In the Kobe earthquake as reported by Tokimatsu et al. (1996) the peak values of accelerations measured in the heavily damaged areas were in the range of 0.7 to 0.8g and in the reclaimed areas the peak measured values of acceleration measured ranged from 0.3g to 0.6g. Thus ground motions were amplified by a factor of 1.5 to 2 times in the heavily damaged areas within deep sedimentary layers whereas in the reclaimed areas where there was widespread liquefaction the measured peak accelerations were the same as those in the rock. This has been widely reported due to the isolation effect of liquefied soil. The benefits of the isolation mechanism is however available only during strong shaking as seen in Fig.5 where for small strength earthquake considerable amount of shear waves is still being transmitted to the surface. Further the loose layer can also result in increased post liquefaction settlements.

Thus the centrifuge test results indicate that the loose layer sandwiched in between the dense layers is capable of changing the transmission characteristics of the input waves as they travel through the medium in the free field as well as underneath the structure. In the next section, the free field response for the experimental soil profile as discussed above is compared with the results obtained by modelling the benchmark problem in different site response programs.

Different Approaches

There are basically three different approaches, which are usually used in the formulations of the constitutive relations for the analysis of dynamic behaviour of soil, and its consequences on the stability of the super imposed structures. These methods can be grouped as:

1. Decoupled methods (or a total stress approach)

2. Indirectly coupled methods

3. Fully coupled methods.

Decoupled Methods

These types of constitutive models model the non-linear stress strain behaviour of soils by using an equivalent linear elastic method of analysis. The basic assumption in these equivalent linear methods is that the non-linear response can be approximated by damped linear elastic model if the properties of the model are chosen appropriately. The stress strain properties of the soil are defined by strain dependent shear moduli and equivalent viscous damping factors. An equivalent modulus and damping ratio at any strain level are determined from the slope of the major axis of a hysterisis loop corresponding to that strain, and the area of the loop respectively. The initial value of moduli and damping are estimated based on small strain values. Decoupled (or total stress equivalent linear) methods of analysis have been used in programs like FLUSH (Lysmer et al., 1975), SHAKE (Schnabel et al., 1972), and QUAD-4 (Idriss et al., 1974)

Indirectly Coupled Methods

The major motivation for the development of more general constitutive relations has been the need to model the non-linear behaviour during dynamic loading in terms of effective stress and to provide reliable estimates in terms of excess pore water pressures. Indirectly coupled methods are similar to uncoupled methods. These models also assume that the soil behaves elastically in small stress increment, and changing the shear modulus and the bulk modulus of soil in each increment the non-linear behaviour can be handled effectively.

The improvement of these models over the uncoupled models come from the fact that the pore pressure generation model is introduced into the model. In 1975 pore water pressure generation model for cyclic loading was developed by Martin et al. (1975), applicable to level ground conditions. Such models relate the changes in pore presssure to changes in volumetric strain under drained conditions. The excess pore pressure generated during each cycle is used to calculate the current effective stress, which in turn is used to calculate the pressure dependent elastic moduli for the next iteration. DESRA and TARA are the most representative codes that have been developed using this class of indirectly coupled methods. They have two separate models incorporated into it, one for pore pressure generation and one for pore pressure dissipation.

Fully Coupled Method

In this method the differential equations governing the motion of the solid and the fluid phases are coupled with the mass balance equation resulting in fully coupled differential equation. These are then approximated by a weighted residual method. These equations are then solved by the finite element technique. In these types of constitutive models the pore pressure generation and the dissipation are fully coupled with the deformation of the soil skeleton according to Biots formulation (1956). This means that they are also controlled by constitutive relationships. DYSAC2 (Muraleetharan, 1988), DYNAFLOW (Prevost, 1981), and SWANDYNE (Chan, 1988) are the codes, which are capable of handling the coupled combinations. They are potentially the most accurate method of analysis. They do require a large number of model parameters for their formulation. One of these codes SWANDYNE is used in the analysis presented in this paper.

Modelling using Different Programs

In 1998 the computer program EERA was developed starting from the same basic concepts as SHAKE (Schnabel et al., 1972) by Bardet et al. (2000). It implements the well known concepts of equivalent linear earthquake site response analysis. In one-dimensional layered system the soil layers are assumed to be laterally homogeneous, of infinite horizontal extent and subjected to horizontal motion from bedrock. Thus the present soil profile (Fig.2) was defined based on the shear wave velocity measured during the centrifuge tests and shown in Table 1. EERA is fully integrated with a spreadsheet program and gives the users many new additional features like unlimited number of soil properties and soil layers.

The website (http://www.cyclic.ucsd.edu) allows remote users to operate the non linear finite element program (CYCLIC 1D) developed for numerical simulation of earthquake ground response and liquefaction effects. This is a two phase (solid and fluid) fully coupled two dimensional finite element program (Parra, 1996; Elgamal and Zeng, 1999). The dense and loose layers were modelled by the predefined materials present in the database of CYCLIC 1D without the raft foundation and the structure. Their properties were different in terms of shear wave velocities, friction angle and unit weight. The input parameters are show in Table 2. The base boundary condition was taken as total transmitting type, implying that the shear wave velocity at the base was similar to the shear wave velocity at the subsequent layers. The input motion recorded in the centrifuge test BG-04 as used as the input motion applied to the base of the 1D model used in CYCLIC 1D.

Analysis in SWANDYNE was performed in the following steps details of which can be seen at Ghosh (2003). A finite element mesh was initially created using an in house pre-processor written in MATLAB. The appropriate boundary conditions are then applied to the model. Static analysis is first performed to determine the initial stress state of the model. A no earthquake dynamic run is then performed to check if the initial stress state is in correct equilibrium condition. If it is not then a new static analysis is performed with modified parameters to obtain equilibrium. Following this a non-linear analysis is performed for the earthquake stage with the assumed cyclic loading similar to the earthquake applied in the centrifuge. In the present case the cyclic loading was the input acceleration applied during the testing. The analysis was performed using a Generalised Newmark scheme with non-linear iterations using initial linear elastic tangential global matrix. The constitutive model used is the Pastor Zienkiewicz Mark III model (Pastor et al., 1985, 1990). It is a generalised plasticity bounding surface model with a non-associative flow rule. It models the effects of dilation, permanent deformations and the hysteric properties of saturated sands under dynamic loads. The parameters needed to define the model completely can be obtained from routine triaxial tests as discussed by Jeyatharan (1991). The parameters used in the present analysis are shown in Table 2.

TABLE 2 : Input Parameters Used in Different Programmes

EERA	CYCLIC 1D	SWANDYNE	
Maximum shear modulus for dense layer 116 MPa	Dense soil friction angle 35°	Parametres for P-Z model	
	Possion's ratio 0.4	$M_g = 1.14$	$\beta_0 = 4.2$
Maximum shear modulus for loose layer 28 MPa	Loose soil friction angle 29°	$M_f = 0.65$	$\beta_1 = 0.2$
	Possion's ratio 0.4	$\alpha_g = 0.45$	$H_0 = 600$
Ratio of maximum and effective shear strain 0.5	Plastic non linear analysis	$\alpha_f = 0.45$	Huo = 4000
		Hevop = 770 kPa	$\gamma_u = 2.0$
		Hesop = 1155 kPa	$\gamma_{dm} = 0.0$

Free Field Responses

The behaviour regime under dynamic loads can be divided into two distinct regions: a near field region, which covers all the non-linearities, related to the interaction between soil, structure and foundation. This includes non linearities generated within the soil and associated with its stress-strain constitutive relation, as well as foundation uplift and sliding at soil-foundation interface. The energy dissipation mechanism in this zone is predominantly hysteretic. The second region includes the far field region where there is less non linearity most of which is associated to the seismic wave propagation in the soil. The energy dissipation mechanism in this zone is essentially due to radiation damping.

Figure 8 compares the site response accelerations obtained from EERA, CYCLIC 1D, SWANDYNE and the centrifuge tests at a depth of 7.5 m below the surface. At shallow depths the observed centrifuge accelerations match the predicted accelerations quite closely, though the peak acceleration observed during the centrifuge test is higher. This essentially implies that motion in dense sand is initially amplified and the codes are unable to match the predictions at the beginning of the shaking period. Some high frequency

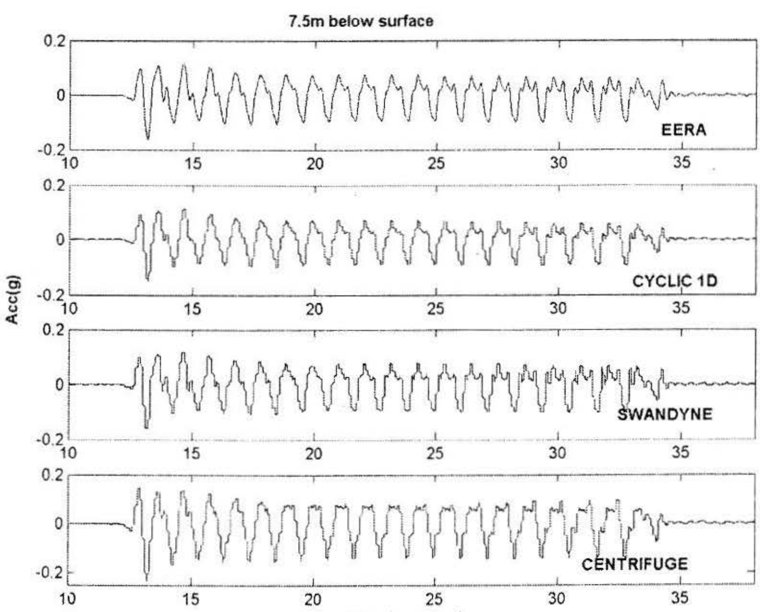

FIGURE 8 : Comparison of Responses in the Free Field for a Depth of 7.5 m Below Surface for Accelerometer A1

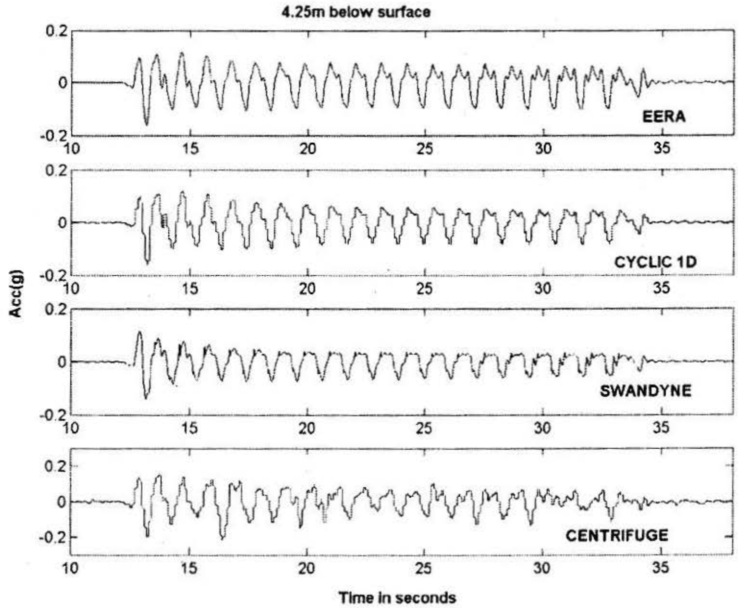

FIGURE 9 : Comparison of Responses in the Free Field for a Depth of 4.25 m Below Surface in the Loose Layer for Accelerometer A2

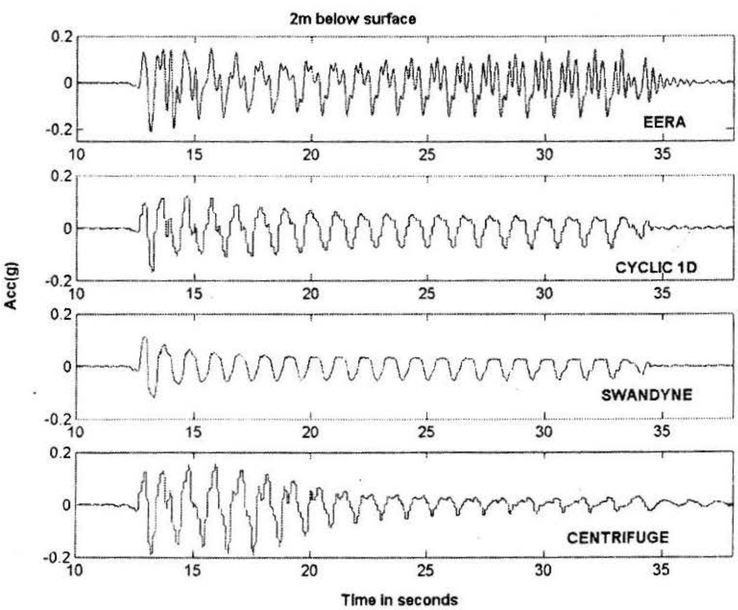

FIGURE 10 : Comparison of Responses in the Free Field for a Depth of 2 m Below Surface in the Dense Layer for Accelerometer A3

components are retained by the motion and this effect is captured well by all the codes.

Figure 9 compares the responses in the middle of the sandwiched loose layer obtained from using different codes. The predictions are different in all the codes. EERA predicts the maximum peak acceleration close to the centrifuge measurement and also predicts increase in the acceleration after the shaking has continued for 10 s. There are some high frequency components in the motion, which persist throughout the shaking period. SWANDYNE and CYCLIC 1D predict a gradual attenuation in the acceleration. This is possibly because both these codes are based on the coupled formulations and they can model the excess pore pressure generation due to seismic shaking. Thus the softening of the loose soil due to generation of higher shear strains and generation of excess pore pressure attenuates the motion in the later part of shaking.

In Fig.10 the free field accelerations are compared at very shallow depth 2 m from the surface. EERA predicts an amplification of primary harmonic and all other harmonics as well. CYCLIC 1D shows a gradual attenuation of the input motion as the shaking progresses. This behaviour is also seen in SWANDYNE but the centrifuge test result at this depth is dramatic. The strong shaking induced a rise of excess pore pressure, which made the entire loose sandwiched layer liquefiable. This reduced the transmission of shear waves as seen in the centrifuge test results. This effect is captured very well by the coupled codes for homogeneous loose soil as seen in the VELACS project but when layering is induced the match is not satisfactory at shallow depths. From a geotechnical practice point of view this is quite significant as often the shallow motions are used in the design of the superstructure.

The motions recorded at the free field are completely modified in presence of the structure due to the interaction effects. Figure 11 compares the acceleration measured at a depth of 4.25 m from the surface underneath the raft foundation by accelerometer (A2 in Fig.2) with the predicted free field response from CYCLIC 1D at the same location. CYCLIC 1D underestimates the peak values of accelerations in presence of the high overburden stress.

The input motion generated by the SAM actuator is basically a single frequency motion and real earthquakes are multi frequency events. Thus the recorded bed rock motion in the Takatori station in the Hyogo Ken Nanbu (Kobe) earthquake in 1995 was used as an input motion for comparing the responses predicted by different codes. The input motion is shown in Fig.12. The peak acceleration is 0.6311g occurring at 6 seconds after the shaking started. The entire duration of shaking was 20 seconds and FFT

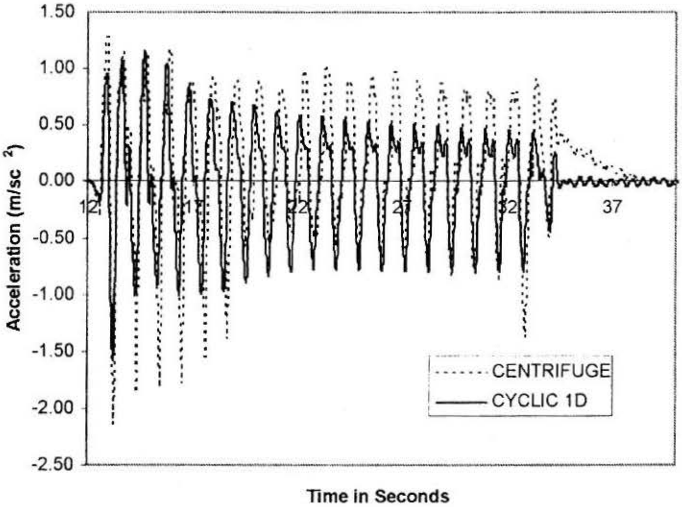

FIGURE 11 : Comparison of Accelerations Underneath the Raft Foundation Obtained from CYCLIC 1D (without Structure) – and Centrifuge Test

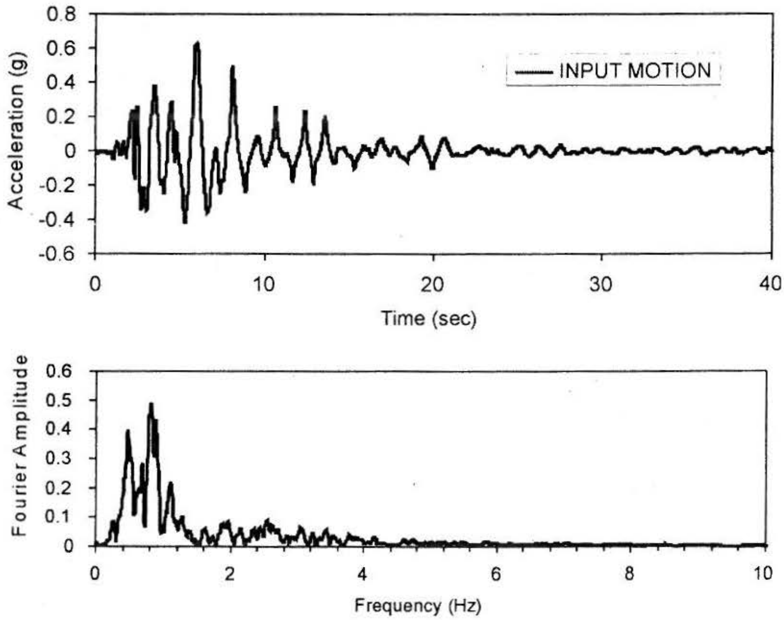

FIGURE 12 : Input Motion for the Motion Recorded at Takatori Station in the Hyogo Ken Nanbu (Kobe) Earthquake (b) FFT of the Input Motion

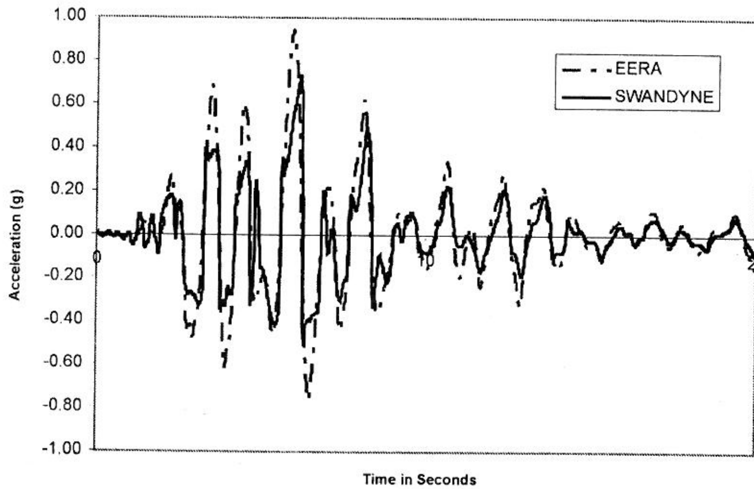

FIGURE 13 : Surface Acceleration Predicted by EERA and SWANDYNE for an Assumed Soil Profile Similar to Test BG-04 for an Input Motion Recorded at Takatori Station

transformations (Fig.12) of the input motion indicate that significant energy is concentrated in the frequency band below 2 Hz

The input motion was used to evaluate the site response for the ground profile similar to that tested in the centrifuge test. Figure 13 presents the surface acceleration predicted by using EERA and SWANDYNE. The surface accelerations predicted by EERA are much higher compared to those predicted by SWANDYNE. EERA overestimates the shear stress under large earthquakes and thus peak accelerations are also overestimated. The shear stress are overestimated as EERA assumes an equivalent linear stress stain curve where the effective strains are smaller than the maximum strain and thus maximum stress is over estimated. In SWANDYNE where the 'u-p' formulation is used such overestimation is not possible. The high frequency components are filtered at the surface better by SWANDYNE.

Figure 14 presents the acceleration response spectrum obtained at the surface predicted by three different types of site response software's. There is no centrifuge data comparison at this level as there was no measurement of surface accelerations in the experiment. It is visible that for the ground profile considered peak response is predicted for time period 1.2 s corresponding to a frequency of 0.83 Hz. SWANDYNE predicts that significant spectral amplification will be seen for high frequency structures placed in such profiles and CYCLIC 1D predicts a lower response for these high frequency structures. From the above observations it can be seen that

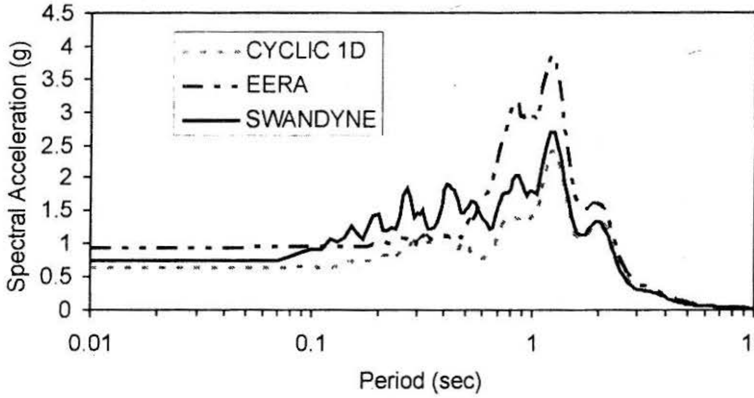

FIGURE 14 : Comparison of Response Spectrum Predicted by EERA, CYCLIC 1D and SWANDYNE

the simple CYCLIC 1D code is able to capture the seismic response of structure placed in similar layered soil but underpredicts the peak ground acceleration. The more elaborate FE code SWANDYNE is able to capture higher amplification expected at the soil surface. This suggests that the use of 2D codes such as SWANDYNE maybe necessary while designing important structures founded on stratified deposits.

Conclusions

It is important in geotechnical design to establish the ground response at a specific site during an earthquake event. There are various methods with varying degree of complexity available to establish ground response. In this paper the complex problem of numerical simulation of dynamic soil structure interaction in layered soil was discussed. It was noted that most of the validation exercises done to date have been on homogeneous loose soil where the natural variability of the ground surface has not been taken into account. Thus centrifuge tests were planned where the ground surface was layered. Earthquakes of different intensity were applied at the base of the model and the response was monitored. The reduction in acceleration amplitudes due to the isolation effects of the trapped liquefiable layer was clearly seen and it was concluded that the behaviour in layered soil is completely different from the behaviour in homogeneous soil. Density, rigidity, thickness and other physical properties (like void ratio) of the soil strata as well as the intensity of the seismic motion are the prime factors affecting the characteristics of seismic waves.

The free field response obtained from the centrifuge test was compared with the predicted free field response obtained from routine site response analysis softwares. It was seen that the predictions at deeper depth are better and at shallow depths the match was not as satisfactory. The maximum transferred acceleration was over predicted by EERA. CYCLIC 1D and SWANDYNE did not model the attenuation at shallow depth appropriately. The actual recorded earthquake input motion was also used as the input motion and the response spectrum plotted for the accelerations at the surface. Site response analysis for the layered soil also showed that there is visible advantage of using a 2D program for designing structures in stratified soil deposits if the foundation is located at some depth below the ground surface.

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Notations

M_g	=	Slope of the critical state line for the determination of loading vector.
M_f	=	Slope of the critical state line for the determination of the plastic strain vector.
μ_g	=	Relationship of dilatancy and stress ratio for the loading vector.
μ_f	=	Relationship between the dilatancy and the stress ratio for the plastic strain vector.
Hevop	=	Bulk modulus of the material.
Hesop	=	Three times the shear modulus of the material.
b_0	=	Shear hardening parametreparameter.
b_1	=	This is usually taken as 0.2.
H_0	=	Constant for loading plastic modulus.
H_{uo}	=	Constant for the unloading plastic modulus.
g_u	=	Plastic deformation during unloading
g_{dm}	=	Plastic deformation during unloading.