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CENTRIFUGE MODELLING OF DYNAMIC SOIL STRUCTURE INTERACTION IN LAYERED AND INHOMOGENEOUS LIQUEFIABLE SOIL

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ABSTRACT

In this paper the results of a series of dynamic centrifuge tests are reported. These tests were performed on different types of soil stratifications supporting a nuclear containment structure. Test results indicate that accelerations transmitted to the structure base are dependent on the stiffness degradation of the supporting soil. It is also conclusively shown that even partial liquefaction can be dangerous and the structure can tilt and rotate. Steady build up of excess pore pressure leads to softening of the soil, which decreases the shear modulus and shear strength and subsequently changes the dynamic responses. The characteristic frequency of the soil deposit gradually decreases to values that are closer to the natural frequency of the deposit. The presence of the structure reduces the translational component of the input base motion and induces rocking of the structure. Thus it can be concluded that rigid structures may not be as safe as believed.

INTRODUCTION

The destruction of critical facilities due to extreme natural events like earthquakes may cause catastrophic losses of life, property damage or disruption of society. Recent earthquake at Bhuj in Gujrat (India) on January 26th, 2001 was felt at three Nuclear Power Plants in India viz. Kakrapar in Gujrat, Narora in Uttar Pradesh and Rawatbhata near Kota in Rajasthan (Warudkar 2001). This earthquake had an epicentre at about 20 Km from Bhuj and occurred in a highly industrialised region of India. The reported magnitude was about 7.9 on the Richter scale. This region is known to be seismically active and is placed on zone V in the seismic zoning map of India. Unfortunately in this earthquake no near field measurement is available due to instrument malfunctioning and some far field measurements were made at locations close to Mumbai. The recorded values of acceleration at these locations (Figure 1) do not explain the high intensity of damage at some of the sites. This has focused the attention of researchers on the possible role of site effects and soil structure interaction effects in aggravating the damages. Most design codes ignore this effect for the vast majority of structures. In this earthquake old buildings in Ahmedabad remained firm; new, multi-storied buildings collapsed, due, it is said, to their being constructed on filled-up land, not on natural soil strata. Also, Ahmedabad was not considered a highly seismic zone when new buildings were constructed.

India's 14 power reactors have a total generating capacity of 2720 MW – with firm plans to expand to 8100 MW by year 2012 (Source NPCIL). THE 440-MW Kakrapar Atomic Power Station (KAPS) is located a few hundred km from Bhuj as seen in Figure 2. The two Indian nuclear power stations closest to the epicentre (located about 300 Km) of the recent severe earthquake went on operating normally at full power throughout. While nuclear plants were unscathed by the powerful shakes, many coal fired thermal power plants in the neighbourhood of Bhuj like Wankabori and Dhuran tripped, according to reports (NPCIL).

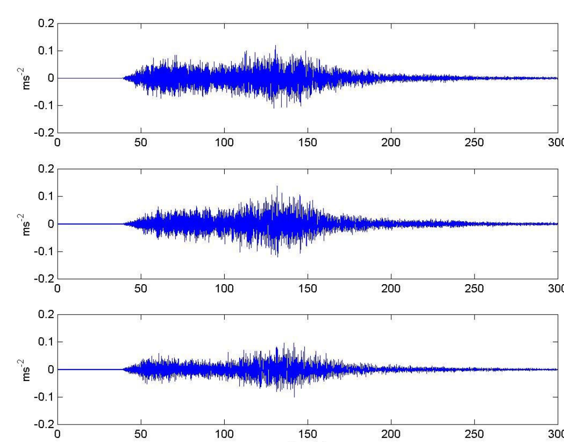

Fig. 1: Acceleration time histories recorded near Bombay (Source IIT Bombay website)

In India, the nuclear power plants are located in very mild to moderate seismic regions, whereas the regions with maximum hydroelectric power potential are the highly seismic regions. The fact that the Kakrapar nuclear power plant, 80 km outside Surat, continued to function even in the aftermath of the recent quake is considered evidence of their in-built quake-proof technology.

The earthquake resistance design incorporated in the nuclear plants, follows a two-layered approach. In stage one design, the nuclear plant is provided safety features that can withstand and operate during earthquakes, which have a return period of 100 years or moderate types. In the second stage, the power station is designed for a safe shut-down earthquake (SSE), which means in the event of a massive earthquake the plant automatically shuts down. The seismologists at the Bhabha Atomic Research Centre (BARC) have an elaborate design strategy for earthquake resistance that is simulated before the setting up of the power station itself. Once the power reactor is in place a network of vibration monitoring devices is put in place to detect and trip the plant automatically if necessary. Interestingly, the earthquake-monitoring network will activate the automatic shutdown mechanism and set off an alarm in case of the earthquake being powerful.

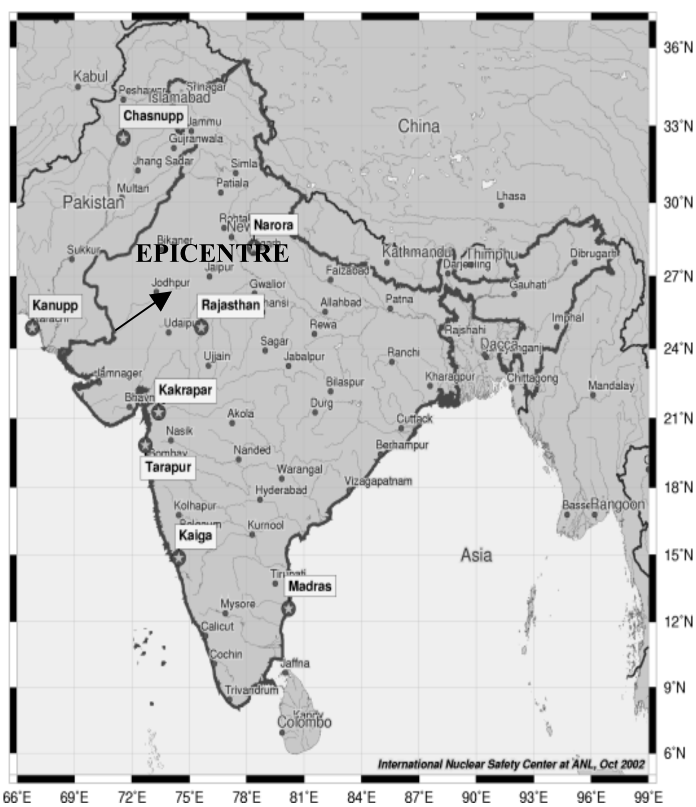

Figure 2: Map of Location of Indian nuclear power plants and the epicenter of the Bhuj earthquake. Source: Department of Atomic Energy (India).

An acceleration time history consists of energy concentration at different frequencies. In most earthquakes the response spectrum is relatively rich in the frequency content between 1-15Hz and is very less beyond 33Hz. The peak acceleration of the time history plot corresponding to acceleration value of the response spectrum plot at 33Hz is called Zero Period Acceleration (ZPA). Structures, systems and equipments of a Nuclear Power Plant are designed for two levels of earthquake. The safe shut down earthquake (SSE) is the high intensity low probability earthquake, which dictates that safety systems maintain their structural integrity during such events. The Operating Basis Earthquake (OBE) is the high probability low intensity earthquake expected in the site once in 100 years. Usually the intensity of OBE is taken as half of SSE intensity. Figure 3 presents the design SSE values for the power plants in India reported by Bhardwaj (2001). Thus the highest values for which the plants are designed are 0.3g.

Thus what the Bhuj earthquake has taught us is that earthquakes are highly likely in those areas where there are nuclear power plants. Indeed at present 20% of the power plants are operating in highly seismic regions like Kobe and California. The present paper investigates the wave propagation characteristics and soil structure interaction effects for power plants founded on layered soils. Dynamic centrifuge modelling is used as a tool to investigate the seismic behaviour of a typical power plant like Kakrapara. A containment structure is founded on soil with different properties and subjected to different magnitudes of earthquakes. It is shown that the base motion is significantly modified due to the presence of the structure and significant rocking is induced in the structure due to kinematic interaction effects.

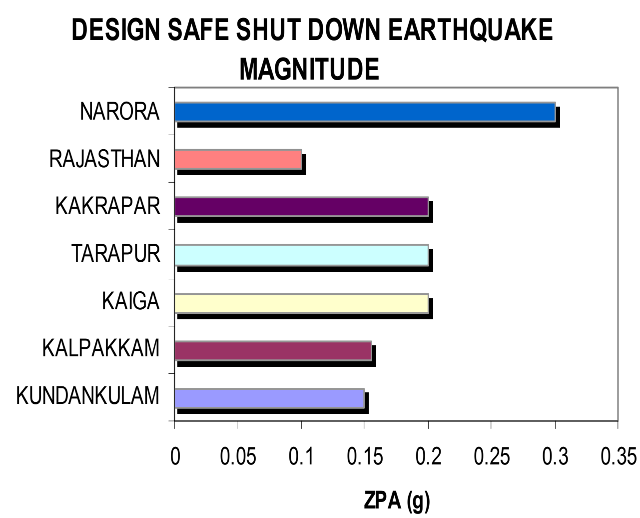

Figure 3: Magnitude of Design earthquake for different nuclear plants in India.

CENTRIFUGE MODELLING

The need for dynamic earthquake modelling is closely associated with the nature of infrequent earthquakes in the field. Very little quantitative field data exists which quantify and qualify the nature of soil structure interaction effects during strong shaking.

Thus dynamic centrifuge modelling provides an excellent opportunity to observe the seismic behaviour in a scaled model. All the tests reported here were performed at 50g centrifugal acceleration field in the 10m Beam Centrifuge at Cambridge University. Table 1 presents the test configurations reported in this paper. Instrumentation consisted of miniature accelerometers, pore pressure transducers, LVDT's and pressure cells. A typical test layout is presented in Figure 4.

The superstructure model represented the containment structure of a nuclear reactor. This building is often the last barrier in the "Defense in Depth" policy of a containment design. This essentially implies that there will be several layers of protection in case of accidents. The structural performance of such a building is often vital in any soil structure interaction studies. Most of these buildings are bottom heavy having a low center of gravity to prevent rocking. Generally they are pre-stressed concrete shell type structure, consisting of a circular wall and a dome on the top. The typical diameter of the dome is about 40-50 meters and the total height close to 40 meters. Usually the embedded depth varies from 5-10 m. The design of the model containment was arrived after considering the different combinations of materials, which would give the ideal bearing pressure and stiffness. Finally the embedded base plate was selected as steel and the dome was made of dural.

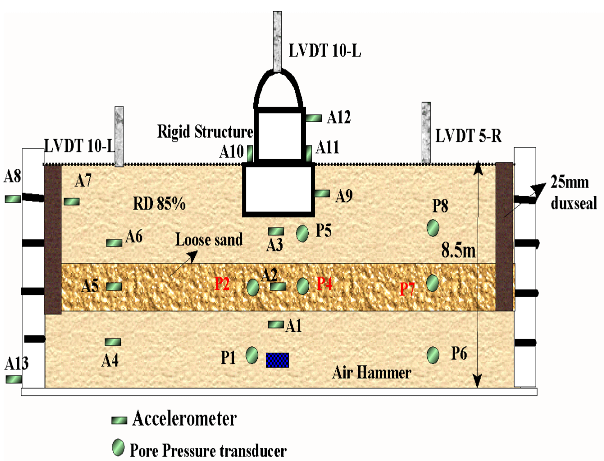

Fig. 4: Instrumentation and test layout for a typical test.

Table 1: Schedule of tests reported in the paper

Test Identity	RD %	Ground Stratification	Embedment
BG-01	52	Uniform loose	1.5m
BG-02	85	Uniform Dense	1.5m
BG-03	Loose-52% Dense-85%	Layered (D-L-D)	1.5m

MODEL PREPARATION

The model was constructed within an ESB (Equivalent Shear Beam) model container whose stiffness is matched with the stiffness of the enclosed soil column. This minimizes the stress wave reflections from the end walls. The design and performance of this box has been discussed by Zeng & Schofield (1996). The internal dimensions of this box are 560mm x 235mm x 220mm. This is equivalent to a soil bed 28m x 11.75m in plan and 11m deep in a 50g test.

The model was prepared by air pluviation of Fraction E silica sand whose properties are shown in Table 2. The sand was poured up to a depth of 30mm and then the air hammer (Ghosh et al. 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 shear wave velocity measured were used to estimate the small strain stiffness of the soil. Instrumentation in the form of accelerometers and pore pressure transducers were placed in the appropriate locations during the model preparation. Different densities were achieved by varying the rate of pouring. The total depth of the prototype was 8.5m. The model was then subjected to vacuum and saturated from the base by using 50cSt silicone oil to correctly model the excess pore pressure generation and dissipation rates. The ground water table was at the soil surface in all the tests.

Table 2: Soil Properties

Property	Value
D_{10} grain size	0.095mm
D_{50} grain size	0.14 mm
D_{90} grain size	0.15mm
Specific gravity Gs	2.65
Minimum void ratio	0.613
Maximum void ratio	1.014
Permeability	0.000098ms^{-1}
Critical angle of friction	32°

After model preparation and saturation the ESB box was loaded onto the SAM (Stored Angular Momentum) earthquake actuator, whose performance has been reported widely by Madabhushi et al. (1998). At 50g the actuator was invoked to excite the model with an earthquake of frequency 50Hz and duration 500ms. In prototype scale this represents an earthquake event of frequency 1Hz and duration 25 seconds. The total bearing pressure was 148 kPa at 50g. The dimensions of the building were somewhat restricted by the size of the available ESB (Equivalent Shear Beam) box. Generally the aim was to instrument the area of the soil participating in the soil structure interaction more densely than the surrounding area.

TEST RESULTS AND DISCUSSIONS

In this section the test results obtained from the various centrifuge tests will be highlighted. In addition the modification of base motion due to layering will be highlighted. All the results are presented in prototype scale.

Seismic response of soil bed

The homogeneous loose soil model was subjected to a medium strength earthquake having a peak acceleration of 0.175g. This magnitude is similar to the SSE earthquake magnitude reported in Figure 3. The input motion was a sinusoidal motion of 25 cycles having an input frequency of 1 Hz. Figure 5 shows that attenuation of the input signal occurs with increasing distance from the base. This effect is more noticed in the later cycles of the shaking. The attenuation is mostly due to the softening of the soil with the excess pore pressure generation due to contractile nature of loose soil. But the attenuation is greater in the free field than below the building. This is essentially due to the presence of initial higher confining stress and static shear, and the free field softens more than the soil under the structure. Similar observations have been made by Kyle et al. (1990). FFT transformations for the signals also show that this effect is more pronounced for the first harmonics and the total attenuation is about 75%. Comparison of accelerations in the building and the soil show that during the cycling there is a phase shift between them. The soil layers act as a filter for the high frequency components in the input motion.

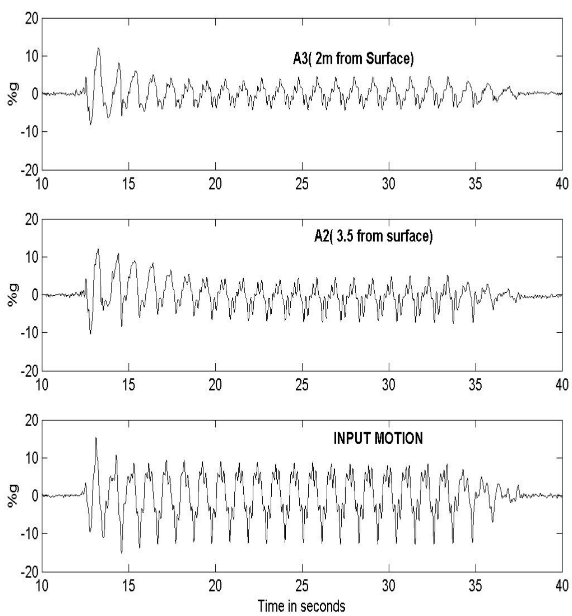

Fig. 5: Transfer of input acceleration from the base in test BG-01.

Soil in the vicinity of the containment undergoes both static and cyclic loads due to gravity and earthquake shaking. The soil around A3 (2m from surface) maybe put in a stress state of compression shear due to dead weight from the structure before an earthquake. Horizontal shaking will then induce cyclic simple shear in the soil element. The soil will then deform under the action of both static load due to gravity and cyclic load due to earthquake shaking.

Test BG-02 consisted of dense sand, which was poured at a relative density of 85% and the entire liquefiable layer was densified. This is a common remediation scheme as suggested by Mitchell et al. (1998) and routinely employed in the field. Figure 7 presents the accelerations recorded underneath the containment for an earthquake having a peak magnitude of 0.2g. The actual duration of the earthquake motion was 25 seconds but the actuator failed to release the clutch completely and thus there is some amount of residual input motion. But this does not affect the transmission of the waves. The motion that is transferred is amplified initially and then significantly same amount of motion is transferred throughout the shaking period. This suggests that there has not been significant degradation of soil stiffness due to shaking but higher accelerations are transferred through the soil. This questions the validity of using densification as a remediation measure as the structure receives strong motion for longer duration.

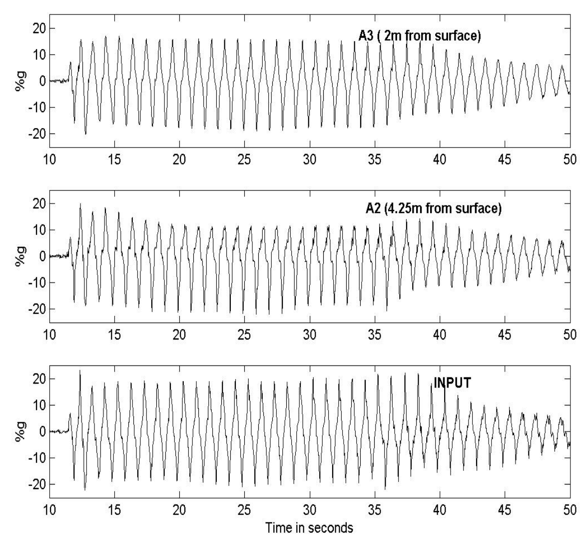

Fig. 7: Transfer of input accelerations from the base in test BG-02 in uniform dense soil.

Test BG-03 consisted of a loose layer having a thickness of 2.5m deposited (RD 45%) uniformly between dense layers having a RD of 85%. Dense sand is generally considered non susceptible to liquefaction. It is however necessary to evaluate the response and the deformation of the ground against extremely strong motion for vital structures such as nuclear power plants. Natural period of the ground becomes longer by softening during excitation due to the soil structure interaction. Figure 8 presents the accelerations recorded underneath the structure for an earthquake having an average peak magnitude of 0.1735g. It is seen that while propagating upward through a layered soil medium the frequency content and amplitude of the earthquake motion may be greatly modified. 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 flatter tops in the acceleration traces indicate the change in the frequency content. The softening of the sandwiched loose layer has significantly reduced the acceleration transmitted to the upper soil layers.

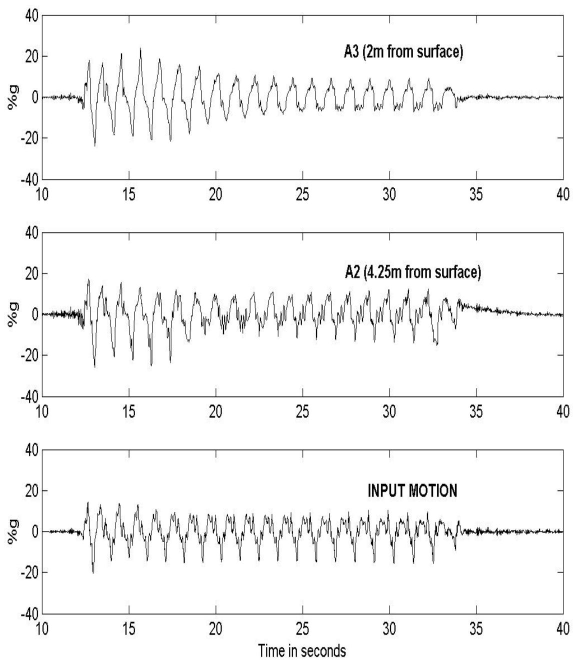

Fig. 7: Transmission of accelerations through layered soil in BG-03

Structural acceleration

In all the three types of tests described above the acceleration was measured at the structure base and would give an indication of the modifications of the base motion due to the presence of the structure. Figure 8 compares the accelerations measured at the structure base for the three type of test. In the dense soil there is not much softening of the soil and the entire shear is transmitted to the structure. The base shear experienced by the structure will be very high in this case and inertial interaction will be maximum. The shaking induced damage due to inertia will be maximum in this case. In the layered soil the acceleration traces show spikes in the recordings in each half cycle. These spikes are coincident with the pore pressure decrease during each cycle corresponding to dilatant behaviour, and has been termed as de-liquefaction shock waves by Kutter et. al. (1999). In the homogeneous loose soil considerable softening of the loose soil has reduced the base accelerations in the beginning of the cycle, but later the interactions of the structure and the soil increase the base response as such interactions depend on the relative stiffness of the structure and the soil. In this case as the stiff structure is resting on soft soil there is maximum interaction.

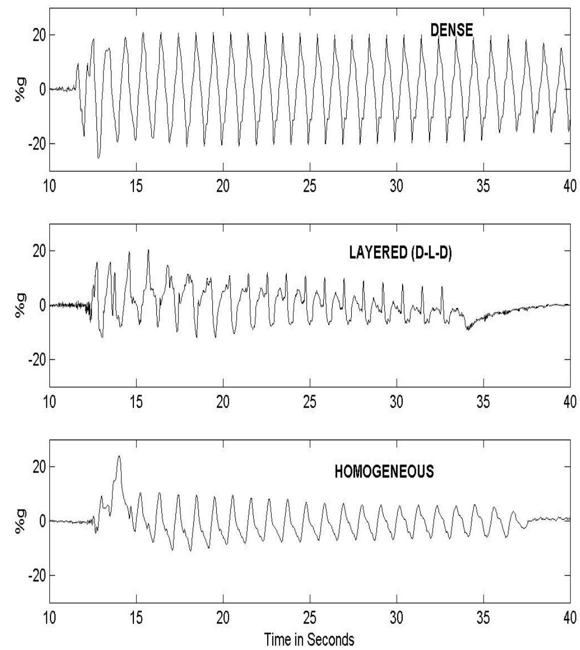

Fig. 8: Comparison of structure base accelerations for different types of soil layering

SETTLEMENT UNDER DIFFERENT TEST GEOMETRY

Usually bearing capacity, settlement and uplift pressure are the factors that have to be considered for foundation design under ordinary conditions. However, when the ground is subjected to cyclic motion due to earthquake loading saturated sands lose their shear strength and behave like a liquid for a short period of time. This is termed as liquefaction and upon liquefaction the bearing capacity of the soil is sharply reduced and the building foundation may suffer excessive settlement and rotation. In shallow foundations superstructure resting on liquefied soil tend to settle relative to surrounding soil surface often unevenly. In free surface settlement the soil deforms in simple shear mode but settlement under the presence of the foundation is largely dependent on the local shearing of the liquefied soil by structure and seismic loads.

Figure 9 presents the settlement of the building top measured by the LVDT during a medium strength earthquake in tests BG- 01, BG-02, and BG-03. The core of the LVDT was rested on a small plate so as to prevent the core from penetrating into the soil. It is seen that as shaking proceeds the initial rate of settlement is same for three types of soil configuration but as the shaking proceeds with the generation of excess pore pressure the dilative stress strain response dominates after some strain has accumulated in dense soil as seen in Fig.11. This dilative response will temporarily restore the effective stresses and increase the shear resistance in sand. This will limit the magnitude of ultimate settlement in dense soil. It is clearly seen that the first few cycles cause the main part of the settlement and after that the rate of the vertical settlement is decreasing. A small amount of layering induced in the otherwise dense soil changes the ultimate settlement values by a significant value. The ultimate settlement in loose soil is 280mm at the end of the shaking period but some amount of settlement continues after the shaking has ceased. The total volumetric strain at the end of the shaking period is approximately 3.5% for loose soil, 2.47 % for layered soil, and 1.56% for dense soil. As most of the settlement is occurring during the shaking period it is clear that the process is not undrained as commonly assumed. The permeability of the soil is changing due to constant rearrangements of the soil particles during densification induced by the seismic shaking.

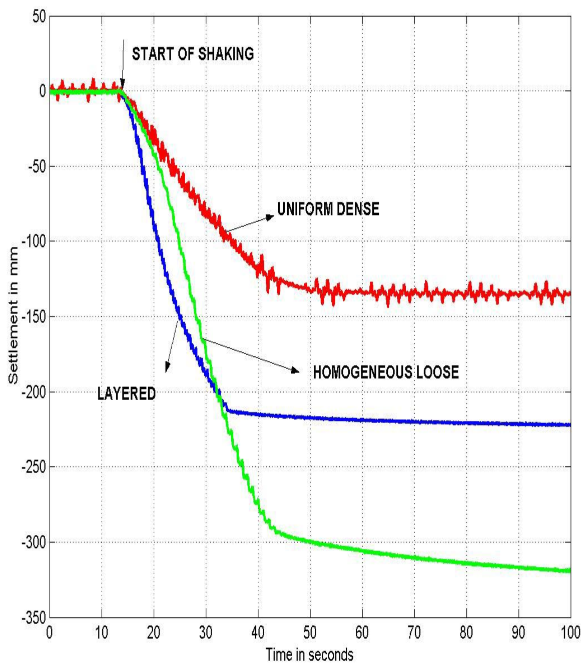

Fig.9: Settlement profile of the containment for different soil stratifications.

PORE PRESSURE RESPONSES

One particular feature observed in the centrifuge model was that regardless of the initial relative density, high pore pressures develop. Figure 10 compares the excess pore pressures measured under the containment and the free field for test BG-01. The strength of the earthquake was 0.1715g in prototype scale and the durations were 25 seconds. The excess pore pressures have been normalized with the estimated effective vertical stress at those locations. It is seen that under the structure the E.P.P.R (Excess Pore Pressure Ratio) never reach 1 but still there is a failure in terms of tilting and rotation. 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. In the free field liquefaction conditions exist after a few cycles of shaking and there also exists a transient hydraulic gradient at all times between the free field and the zone of influence under the structure. This result in the hump seen in the pore pressure trace under the containment after the shaking has stopped. This continues till the transverse pore pressure gradients have been equalized. It is also seen that it is the free field at shallow depth, which liquefies first and remains liquefied for the longest period of time. Similar observations have been made by Liu et al. (1997).

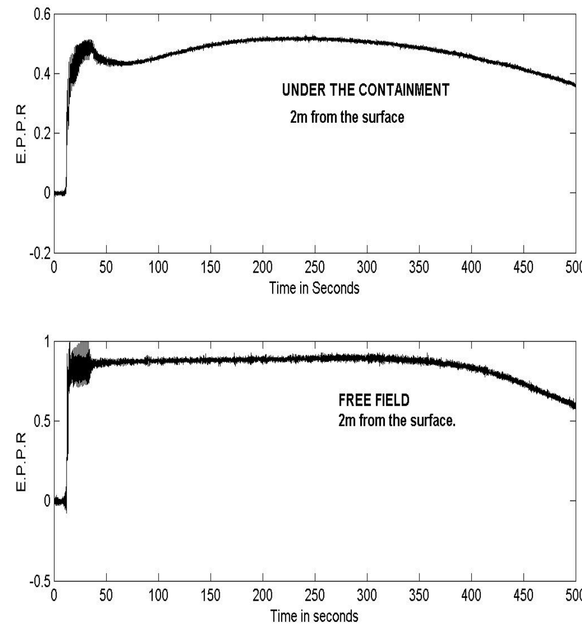

Fig. 10: Excess pore pressure traces under the containment and the free field for homogeneous soil.

Figure 11 compares the excess pore pressure responses measured under the structure for the different tests at the same location. The pattern of pore pressure generation is completely different for different types of layering. The excess pore pressure measured has been normalized with respect to the estimated initial effective stress at that location. In terms of magnitude the steady pore pressure values are similar in the three tests. Large dilations are seen for the dense soil then the loose soil. Cyclic shear loading action shifts the deviator stress path progressively to the left towards the origin due to the pore pressure build up. If the cyclic deviator stress does not cross the abscissa, effective stress zero condition is never reached. Because once the stress path reaches the strength envelope it tends to stabilize and the pore pressure does not build up further. This behavior is seen in the layered and the dense soil stratifications. P2, which is under the structure, shows significant transient decrease in pore pressure and corresponding soil strength gain due to suppressed dilatancy. However, permanent shear strains accumulate in the dense soil even after the stress path has stabilized at the strength envelope.

In the homogeneous loose soil there is steady build up of pore pressures till the shaking continues and though complete liquefaction conditions is never reached but there is considerable softening of the subsoil. This softening is enough to cause rotation and tilting of the structure.

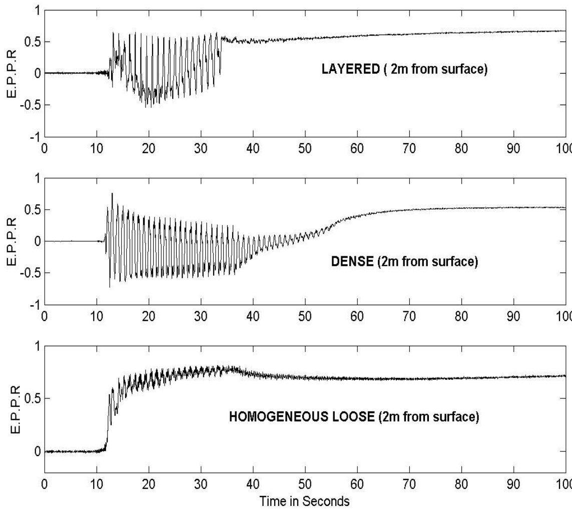

Fig. 11: Comparison of pore pressure responses underneath the structure for different test configurations.

POST TEST CONFIGURATION

Generally it is expected that the reduction in the bearing capacity of the foundation soil due to excess pore water pressure increase will cause the structure to tilt and rotate. The movement of the foundation in this case is associated with significant deformation in the foundation soil resulting in tilt and settlement. Figure 12 presents the post-test configuration in the centrifuge tests. In case of homogeneous soil although the excess pore pressure ratios never reached 1 implying full liquefaction conditions there is considerable tilt and rotation. Post test measurements indicated that the structure has tilted by about 15 degree. This would render the structure useless from serviceability point of view and would seriously endanger its functions. Thus partial liquefaction conditions can also be equally dangerous and the structure can tilt and rotate. In such sites the ground will be improved before constructing an important structure like the containment and natural inhomogenous grounds may also tilt and rotate the structure. Centrifuge test results indicate that the tilt may be upto 4 degrees. In case of dense soil there is no visible tilt or rotation and the ultimate settlement is also less.

As the structure tilts and rotates it starts shearing the soil underneath which offers temporary resistance. As it continues to tilt in one direction the soil underneath experiences shearing due to the seismic shaking as well as structure induced cyclic shearing. The resistance offered by the soil is due to this undrained strength of soil, which is the strength when the soil is sheared at constant volume. Imposition of undrained shear strain will suppress the potential diltancy. The increment of negative pore pressure in the locally sheared soil creates an increase in effective stress, which temporarily provides support. This reduction in pore pressure would induce transient flow in the sheared soil from the neighbouring liquefied soil but not sheared additionally due to the structure tilting. This coupled with progressive softening would lead to failure eventually.

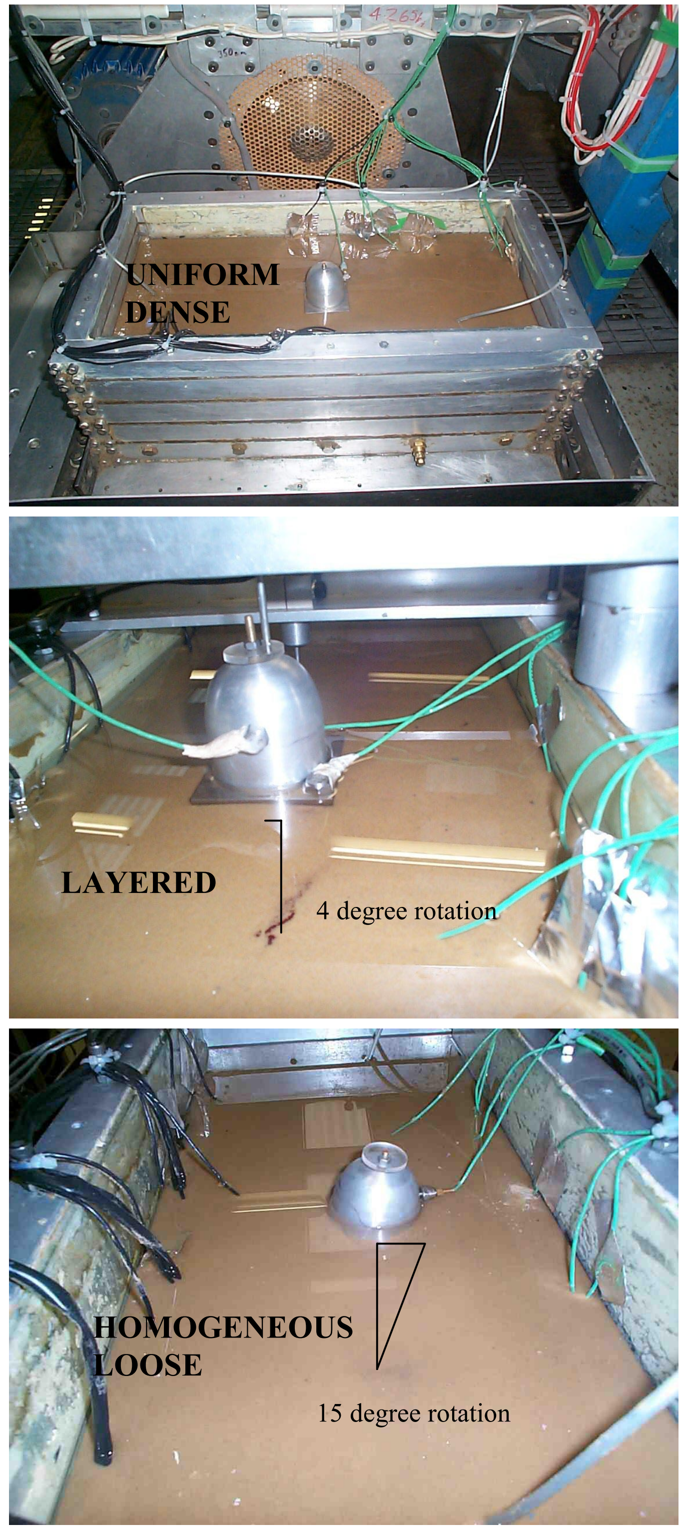

Fig.11: Post test configurations for different tests

CONCLUSIONS

The present paper investigates the wave propagation characteristics and soil structure interaction effects for power plants founded on layered soils. In this paper the results of a series of dynamic centrifuge test is reported. The tests were performed on different types of soil stratifications supporting a nuclear containment structure. Test results indicate that accelerations transmitted to the structure base are dependent on the stiffness degradation of the supporting soil. In all the tests it is seen that the initial deviatoric stress induced in the soil due to gravity significantly affects the excess pore pressure increase. In all the cases the excess pore water pressures which affect the stability and deformation of structures never reach the initial effective vertical stress to cause full liquefaction. This can be attributed to the fact that the shear soil deformation and failure precedes the onset of liquefaction in these areas.

In homogeneous soil it is seen that the accelerations transmitted to the structure base are attenuated to a large extent, depending on the strength of the earthquake. Although full liquefaction conditions never develop under the structure, but the softening of the soil due to pore pressure generation and the local shearing of the soil under the cyclic loads ultimately bring the structure to collapse or violating serviceability criteria.. Thus partial liquefaction is as damaging and should be prevented by suitable remediation schemes.

On the other hand when densification is performed under the structure there is significant amplification of the input motion and there is little degradation of the soil. A loose patch introduced between dense layers changes the frequency contents of the input motion measured in the base of the structure. Under strong shaking these loose patches may liquefy and act as isolators. The structure settlement due to the loose patch is somewhat smooth than unimproved loose soil.

The rate of settlement is also different for different types of test stratifications. When improvements are made to the soil such as densification; the ultimate settlement values are reduced drastically. A small amount of layering introduced in a dense soil changes the ultimate settlement by a substantial amount.

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