Earthquake Engineering, Tenth World Conference © 1994 Balkema, Rotterdam. ISBN 90 5410 060 5

Centrifuge modelling of soil liquefaction

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ABSTRACT: Seismically-induced liquefaction phenomena involving uniform and layered sand and gravel deposits are studied with the help of centrifuge modelling. Results of centrifuge tests of purely soil systems and of soil-shallow foundation systems subjected to base shaking are discussed, and they are related to prototype field situations involving liquefaction of horizontal deposits, lateral spreads in gravels, and liquefaction-induced bearing capacity failures of foundations.

INTRODUCTION

Liquefaction and lateral spreading of water–saturated cohesionless soil deposits ranging from silt to sand and gravel is one of the most common causes of damage to constructed facilities during earthquakes. Earth dams and embankments, retaining structures, buried lifelines and foundations of buildings and bridges have suffered extensively due to liquefaction ground failure in many earthquakes. Hamada and O'Rourke (1992) and O'Rourke and Hamada (1992) have documented the effects of liquefaction for ten earthquakes in the United States, Japan, and the Philippines between 1906 and 1990. The most dramatic and well–known case of liquefaction–induced bearing capacity failures of building foundations occurred during the 1964 Niigata, Japan earthquake, with hundreds of multistory buildings sinking up to 3.8 m and tilting as much as 80° (Kishida, 1966; Seed and Idriss, 1967).

Although many investigations have been conducted on the subject in the last three decades, a number of key aspects of the liquefaction phenomenon remain poorly understood, and research continues (NRC, 1985). Centrifuge modelling of seismically–induced liquefaction, started at Cambridge University in England in the late 1970s, has proven to be a particularly useful tool in that respect (Schofield, 1981; Whitman and Arulanandan, 1985; Hushmand, et al., 1988).

After a brief review of the basics of centrifuge earthquake testing, the paper discusses liquefaction results obtained in our centrifuge at Rensselaer Polytechnic Institute relevant to lateral spreading and to the response of shallow foundations on liquefiable soil. Additional details on these tests are provided by Liu (1992).

BASICS OF CENTRIFUGE EARTHQUAKE MODELLING

A centrifuge is any device that spins and generates centrifugal forces to achieve some practical purpose. It produces what is essentially an artificial gravitational field that is higher than the earth's 1g field. In a geotechnical centrifuge like the one sketched in Fig. 1, we subject a small–scale soil or soil–structure model to a centrifugal acceleration typically somewhere between 30 g's and 200 g's. When the arm spins around the axis, the platform and model gradually rotate about 90° around the hinges during the flight. In Fig. 1, the model is supposed to be an earth embankment, but it could be any soil or soil–structure system. If we spin this model for a few minutes or a few hours, we are simulating the action of the earth's gravitational field on a real prototype embankment many times bigger. If we then apply a horizontal accelerogram to the model while everything is still in flight, as also sketched in Fig. 1, we are testing the seismic response of our model and we are providing answers which are applicable to the real prototype embankment.

Why do we need the centrifuge in geotechnical engineering? The best way to answer this question is to divide it in two parts (Whitman and Arulanandan, 1985).

First of all, we need small–scale model tests of our earth structure or soil–structure systems because the very complex and nonlinear stress–strain behavior of the soil makes it very difficult to predict the response of the system. This is

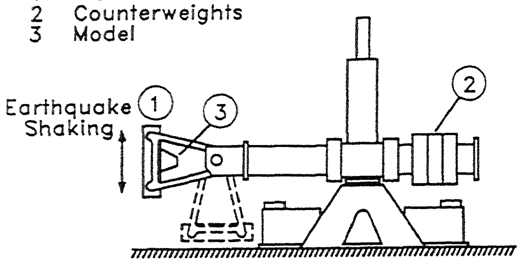

Figure 1. Sketch of RPI geotechnical centrifuge

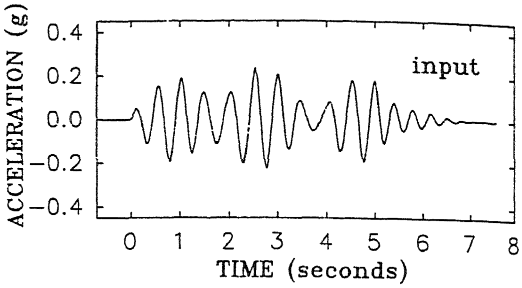

Figure 2. Horizontal acceleration in prototype units measured at base of model during a centrifuge test

especially true at large strains for both static and dynamic loading. Case histories and field observations are very useful, but they generally do not give us all the information we need. Model tests are a very attractive way of learning about basic mechanisms, filling gaps in our knowledge and validating numerical methods.

Unfortunately, if we make a small–scale model of a system using the same soil as in the prototype, the confining stresses will be too small and the stress–strain behavior of the model will be quite different from that of the real system. To get the stresses back up to their correct values, we increase the g–level by putting the model on a centrifuge, and we expect - based on the relevant scaling laws - that the resulting strains and deformations will be close to those in the field. Therefore, a basic fact of centrifuge testing is this expectation that the stresses and strains are the same for corresponding points of the model and prototype.

All results presented in this paper were obtained at our centrifuge at Rensselaer Polytechnic Institute (RPI). This machine, sketched in Fig. 1, has a capacity of 100 g–ton; that is, it can spin 1 ton at 100 g's or 0.5 ton at 200 g's. The arm has a total length to the platform in flight of 3 m.

The problem of applying a simulated earthquake to the base of the centrifuge model while everything is spinning is quite complicated, and a number of techniques have been used by different groups. Two years ago we built an electrohydraulic shaker at RPI for this purpose. This shaker can generate either sinusoidal functions or actual earthquake records. Figure 2 shows a typical input record generated by our system at the base of a model while spinning it at a centrifugal acceleration of 50 g's. This accelerogram is essentially a 5–second duration earthquake, consisting of about ten cycles and with a peak horizontal earthquake acceleration of 0.2 g. Now, it must be realized that the actual record applied to the model is 50 times shorter; that is, it has a total duration of 0.1 seconds instead of 5 seconds. Also, the peak earthquake acceleration is 50 times bigger; that is, 10 g's instead of the 0.2 g's shown in Fig. 2. The accelerogram in Figure 2 is in prototype units, and the scaling laws require this multiplication of the times by 50 and division of the earthquake accelerations by a factor of 50 when going from model to prototype. This factor is always equal to N = the number of g's at which we are spinning the centrifuge.

Correct application of the scaling laws is critical to the conduction and interpretation of centrifuge earthquake model tests. If we assume that we are spinning at $N = 50$ g's, 1 foot of any linear dimension in the model represents 50 feet in the prototype. Similarly, a displacement of 1 centimeter measured in the model corresponds to 50 centimeter in the prototype. On the other hand, a stress of, say, 1 kg/cm^2 in the model corresponds to the same stress, 1 kg/cm^2 in the prototype. The strains are also the same in both model and prototype.

The scaling of the time is especially important for earthquake simulations. From the viewpoint of the dynamic response, in our example of $N = 50$, 1 second in the model corresponds to 50 seconds in the prototype. However, unfortunately the time scaling factor for consolidation phenomena is different, $(50)^2 = 2500$. This discrepancy between dynamic response and consolidation times is relevant to the liquefaction phenomena modelled in our work, where dynamic response and consolidation of the saturated soil often happen simultaneously during the shaking. In many cases we solve that problem by using a pore fluid more viscous than water, as further discussed later herein.

What are the most important applications of centrifuge modelling to geotechnical and earthquake engineering? First and foremost, the centrifuge is invaluable for identifying the actual mechanisms of behavior that are taking place. Sometimes we even find completely new patterns of behavior which we had not anticipated, and which guide us in the modification of the corresponding design methods or analytical techniques. Centrifuge modelling also allows us to quantify the influence of important factors for evaluating alternative designs for new structures and remediation techniques for existing ones, and to validate computer codes to be used directly in design. An important recent example of this last application is the seismic design of retaining structures for the planned expansion of the Port of Los Angeles in the United States, where centrifuge tests have been required for validation of the corresponding computer programs.

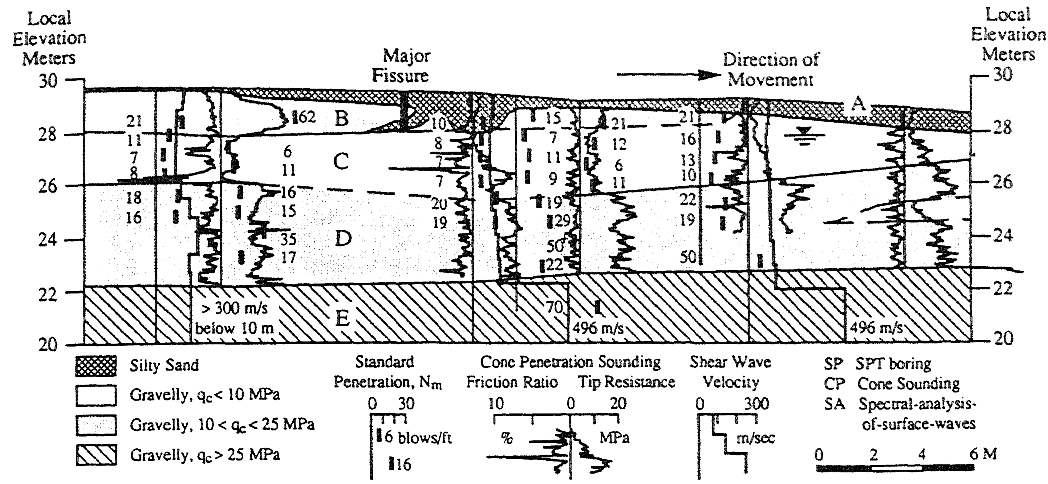

Figure 3. Cross section of liquefaction-induced lateral spread at Pence Ranch site during the 1983 Borah Peak, Idaho earthquake (Andrus, et al., 1991).

MODELLING OF LIQUEFACTION IN LAYERED DEPOSITS

Many cohesionless soil deposits susceptible to liquefaction are layered to various degrees, and this layering has a profound influence on the response. This is true for lateral spreading of mildly sloping ground - one of the most common and destructive phenomena associated with liquefaction during earthquakes - as well as for the bearing capacity response of foundations on liquefiable ground (for examples, see Seed and Idriss, 1967; Youd and Perkins, 1987; Dobry and Baziar, 1992).

Until about ten years ago, most geotechnical engineers thought that liquefaction was only a problem in saturated sands and nonplastic silts, but never in clean gravels due to their high permeability. After all, gravel drains are often placed in liquefiable sand deposits and earth dams as a remedial measure to prevent liquefaction. This confidence was shattered after a magnitude 7.3 earthquake that happened in 1983 in the State of Idaho in the United States, which caused extensive liquefaction and lateral spreading of gravels, including observations of gravelly sand boils at the ground surface (Youd, et al., 1985). Therefore, we need to understand better the circumstances and consequences of gravel liquefaction, and centrifuge modelling is one of the tools we can use to this end.

Figure 3 shows the cross section of the soil at Pence Ranch, one of the sites that liquefied in Idaho in 1983 (Youd, et al., 1985; Andrus, et al., 1991). This cross section is not distorted, and thus the section shows the actual ground surface slope, which is of the order of 6%. The direction of the lateral movement was downslope, as is usually the case in lateral spreads, and gravelly sand boils and large fissures were observed in the back of the slide as indicated in the figure. Most of the soils at the site are clean sandy gravels and gravelly sands with less than 5% fines. Unit C, which is the loose sandy gravel that liquefied, is covered by a very silty sand cap, Unit A, which has a much lower permeability. Obviously the presence of this low permeability fine-grained layer A had a lot to do with the liquefaction and failure of the site (Andrus, et al., 1991).

Figure 4 shows the setup of the centrifuge model we tested at RPI to help us understand this liquefaction of a gravel deposit with an impervious layer on top. We have a fine Nevada sand layer of 40% relative density and 6 cm thick, which, at N = 50 g's centrifugal acceleration, corresponds to a cohesionless layer 3 m thick in the field. As we used water as pore fluid in these tests, the consolidation times in the field correspond to a soil much more pervious, like a coarse sand or a gravel. On top of this sand or gravel is a saturated silt, also about 3 m in the field, which is much less pervious. All of this was placed inside a box, instrumented with 6 accelerometers, four pressure transducers, and a couple of LVDTs to measure the settlements of the top of the sand and the ground surface. Then we spun the model at 50 g's and we subjected it to the base input accelerogram of

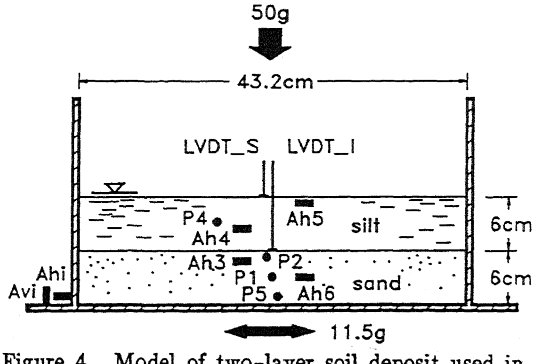

Figure 4. Model of two-layer soil deposit used in centrifuge test

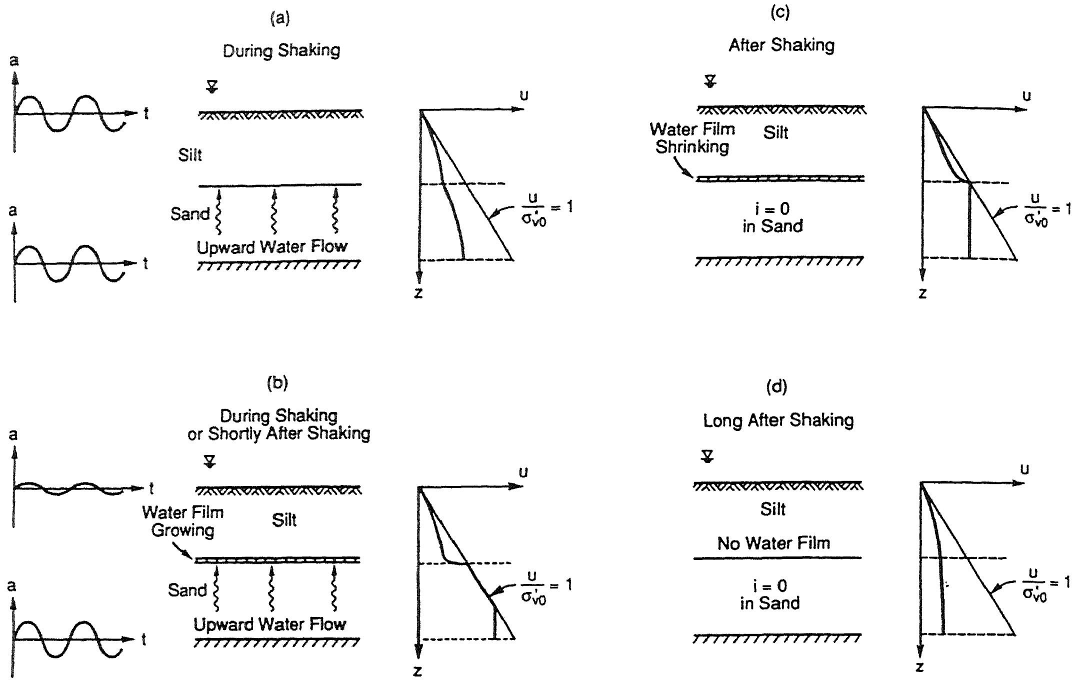

Figure 5. Four stages observed in centifuge test of two-layer deposit.

Fig. 2, having a duration of 5 seconds and a peak acceleration of 0.23 g in prototype units (11.5/50 = 0.23 g).

Figure 5 shows in a schematic way the stages of behavior we inferred based on the pore pressures and accelerations measured in the soil in this test, during and after the shaking. Figure 5a illustrates the first stage, corresponding to the first two seconds of shaking. The excess pore pressures are increasing rapidly in the sand and in the silt due to the shaking, but they have still not reached initial liquefaction, which is defined by the line labelled "$u/\sigma'_{vo} = 1$." (Throughout this paper, a = acceleration in prototype units, u = excess pore pressure, and σ'_{vo} = initial effective overburden pressure before the shaking.) As a result, the accelerations on top of the soil in Fig. 5a are similar to the input, of the order of 0.2 g in both cases. However, the upward hydraulic gradient in the sand, combined with its high permeability at 50 g's is already producing the upward water flow sketched in the figure, and water is starting to accumulate at the base of the fine-grained layer.

Figure 5b presents the second stage we observed, which lasted from about 2 seconds to the end of the shaking at 5 seconds, and extended a little bit beyond shaking. Initial liquefaction had been reached in the upper part of the sand layer, and a pure water or almost pure water film or water interlayer has already formed at this stage at the interface between the two layers. As this water film cannot transmit shear, suddenly the two layers become dynamically uncoupled starting at about two seconds, and we observe a sharp decrease in the soil accelerations recorded at the ground surface. While the sand continues to sediment and consolidate in this stage as it dissipates the excess pore pressures induced by the shaking, the upward water flow continues and the thickness of the water film keeps growing until 11 seconds, that is, until about 6 seconds after the shaking had ended.

The third stage is sketched in Fig. 5c and is extremely interesting. It occurs after shaking and it is very long, about 4 minutes in prototype time. During all this time, the only point of the whole profile which has a pore pressure ratio of 100% is at the boundary between the two layers, indicating that the water film is still there for these 4 minutes, most probably shrinking as the water makes its way slowly through the silt to the ground surface. However, during these 4 minutes, the rest of the sand layer is not liquefied at all, but rather it has fully resedimented and has dissipated the excess pore pressures generated by the shaking. That is, the layer of silt is floating on this water film for about 4 minutes, and the constant excess pore pressure in the sand, with zero hydraulic gradient, simply corresponds to the buoyant weight of the silt layer. Therefore, the sand is not really liquefied in the traditional sense, and any engineering effect, like for example a lateral spread of the system if it had a mild slope instead of being horizontal, would be caused by the existence of this water film under the silt for an extended period of time during and after earthquake shaking. This is clearly what

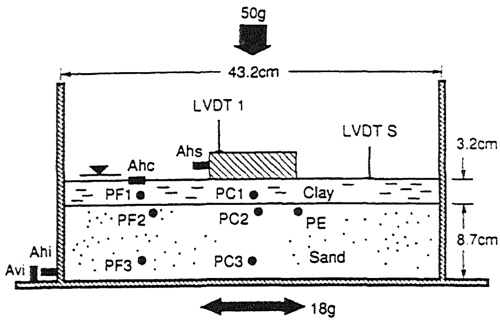

Figure 6. Model of shallow foundation on two-layer soil deposit used in centrifuge test

happened at the gravel site in Idaho previously mentioned. Kutter and Fiegel (1992) obtained additional evidence generally consistent with this suggested mechanism of lateral spreading, by shaking a centrifuge model using the same sand and silt used at RPI but with a 5% ground surface slope. Therefore, engineers should watch for possible earthquake-induced liquefaction of saturated loose gravel deposits covered by a clay or silt layer.

Figure 5d shows the fourth and last stage observed in the RPI test after 4 minutes; that is, after the disappearance of the water film. This last stage is characterized by a regular 2-layer coupled consolidation, with the corresponding dissipation of excess pore pressures.

It must be noted that the existence of water interlayers such as those inferred from the results of the RPI centrifuge test in Figs. 4 and 5, has also been visually observed or inferred in 1g and centrifuge tests of layered soil models by several researchers, at RPI (Elgamal, et al., 1989; Liu, 1992) as well as elsewhere (Liu and Qiao, 1984; Arulanandan, 1988; Kutter and Fiegel, 1992).

Figure 6 presents a very similar two-layer horizontal soil deposit, but now with a shallow foundation placed on top of the ground. Instrumentation includes accelerometers and LVDTs on the foundation and on the soil away from the foundation, as well as pore pressure transducers under the foundation and away from it. The foundation model is circular and has a diameter of 10 cm, corresponding to 5 m in the prototype. During spinning the foundation applies a bearing stress of 125 kPa. The sand is loose with a relative density of 45%, and again water is used as pore fluid.

We applied at the base of the sand again a 5 second duration accelerogram with a peak acceleration of 0.36 g in prototype units. The upper two traces of Fig. 7 present the excess pore pressures recorded in the sand by transducers PF2 and PF3. They show a typical free field behavior during shaking, with initial liquefaction reached at 1.5 or 2 seconds at both depths. At the end of the shaking the pore pressure at PF3 starts dissipating, but PF2 stays up there due to the presence of the upward water flow toward the water interlayer.

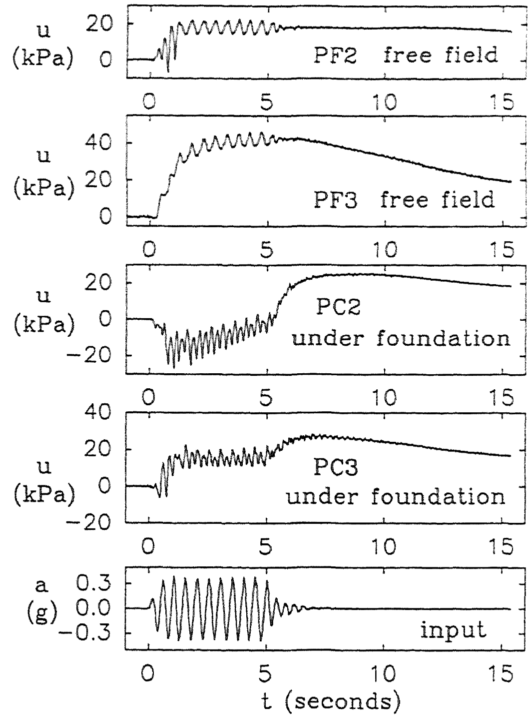

Figure 7. Acceleration and pore pressure records obtained during centrifuge model test of Fig. 6.

The pore pressure behavior under the foundation, also included in Fig. 7, is quite different and very interesting. Immediately under the foundation, at PC2, the excess pore pressures during shaking are negative due to soil dilatancy in the presence of the large static shear stresses caused by the foundation. At a larger depth, transducer PC3 shows positive pore pressures but smaller than in the free field (compare PC3 and PF3). After the shaking there is a horizontal flow of water, first from the free field toward the foundation, and then in the opposite direction.

Figure 8 shows the pore pressure pattern more clearly, while the sketch in Fig. 9 has our interpretation of what happened during this test.

Figure 8 presents the recorded excess pore pressures plotted versus distance from the axis of the foundation, near the top of the sand layer. For example, the line labelled "5 sec" shows that at the end of the shaking the pore pressure is slightly negative under the center of the foundation, while at the same depth it has already reached initial liquefaction in the free field, corresponding to $u \approx 20$ kPa. The inclination of this line gives the hydraulic gradient in a horizontal direction, which points toward the foundation axis and consequently there is water flow from the free field toward the center. However, after about 7 seconds the pore pressure under the foundation has grown larger

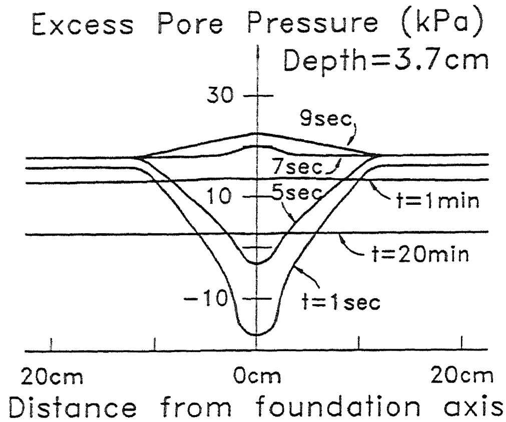

Figure 8. Observed pore pressure isochrones at a depth of 3.7 cm during centrifuge model test of Fig. 6.

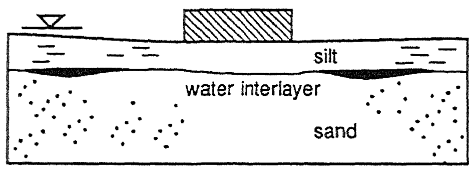

Figure 9. Inferred pattern of water interlayer formation due to shaking of surface foundation on two-layer soil deposit.

than in the free field, and the sign of the hydraulic gradient is now towards the free field. After a very long time, like 1 minute or more, there is no initial liquefaction in the free field any more, and also the horizontal water flow has stopped.

Figure 9 summarizes our interpretation of what happened after the end of the shaking. A water film had formed in the free field, similar to what we found in our tests of layered soil without a foundation (Fig. 5), and the fine-grained layer is floating on water. However, under the foundation the weight of the structure is forcing the water out toward the free field, probably contributing to a larger thickness of the water film just outside the foundation. These outward gradients at 7 and 9 seconds would correspond to the weight of the foundation sending the water out to the free field. This also means that during the time when there is a water film just outside the foundation, its existence is causing a redistribution of the foundation static stresses within the soil, which are now taken mostly or exclusively by the column of soil directly under the structure. This picture of water expelled from under the foundation probably explains why we typically observe so many sand boils near the structures after earthquakes (see also Liu and Qiao, 1984). But the most important question is: What would have happened if the foundation had been on sloping ground and the soil

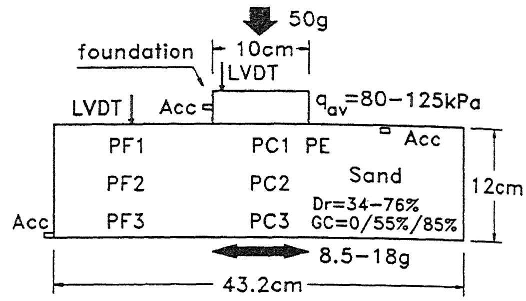

Figure 10. Centrifuge model of shallow foundation on uniform saturated cohesionless deposit.

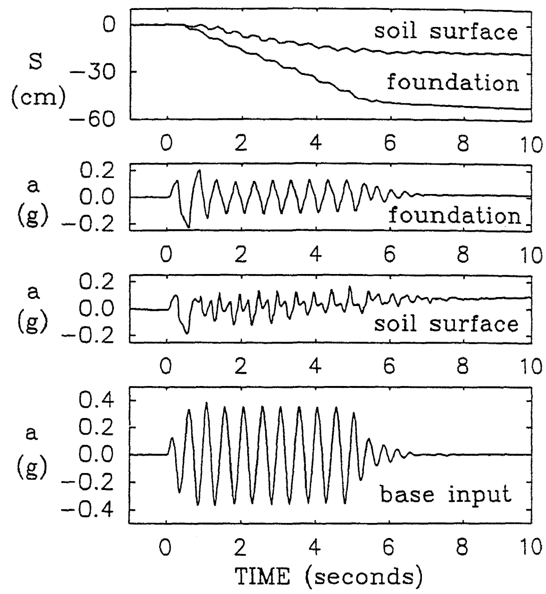

Figure 11. Acceleration and settlement records obtained during one of the centrifuge model tests sketched in Fig. 10.

layering had extended for a long distance? Then most probably a lateral spread would have taken place, with the top layer sliding along the water film and with the structure riding on this top layer and experiencing a large permanent displacement. We clearly need some centrifuge experiments with model foundations on sloping ground to verify this.

MODELLING OF SHALLOW FOUNDATION ON SATURATED UNIFORM SOIL

We also conducted a series of centrifuge tests at RPI with a similar circular model foundation, but now on a uniform saturated sand layer ranging from very loose to dense (Fig. 10). The same fine Nevada sand previously mentioned was used in

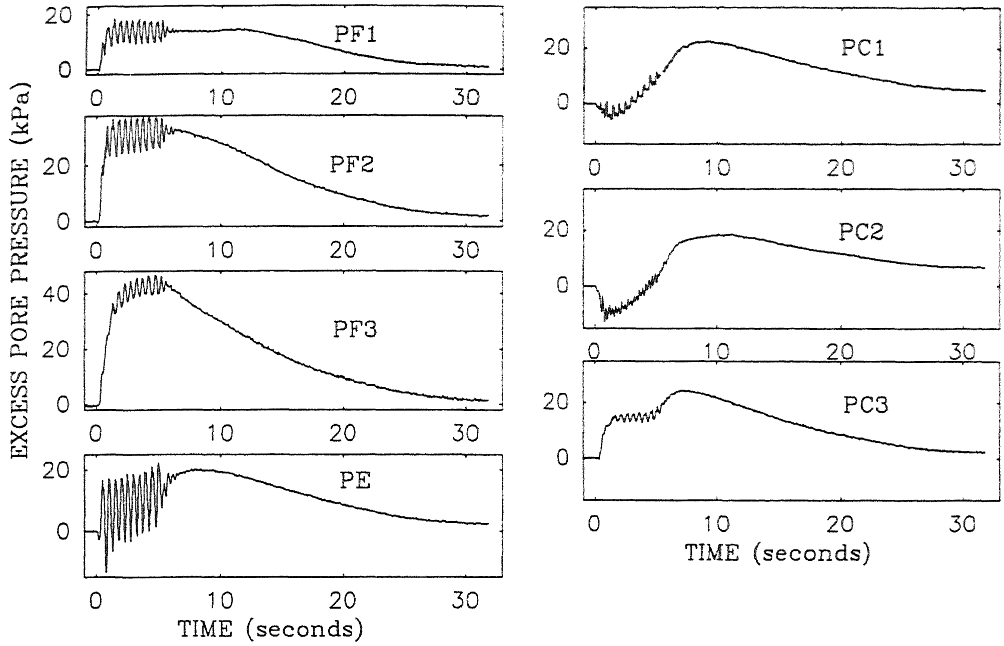

Figure 12. Pore pressure records obtained in centrifuge model test of Figs. 10 and 11.

these experiments. Some of the tests used water as pore fluid, while others used a mixture of glycerine and water. The values of GC in the figure indicate the percentage of glycerine used: GC = 0% mean pure water, and GC = 55% and GC = 85% glycerine contents were also used. For the 50 g's centrifuge acceleration applied in all tests, the correct mixture which will produce the same scaling factors for dynamic response and consolidation times is between GC = 55% and GC = 85%. Figure 10 shows the usual array of accelerometers, LVDT's and pore pressure transducers on the foundation, under the foundation and outside it. We excited the base by peak accelerations ranging between $8.5/50 = 0.17$ g and $18/50 = 0.36$ g in prototype units.

Figure 11 displays typical acceleration (a) and settlement (S) results, for a test where water was used as pore fluid and the base peak acceleration was 0.36 g. As previously discussed, due to the use of water, this test simulates approximately the liquefaction of a 5 m diameter prototype foundation on a 6 m layer of saturated coarse sand or gravel. As the soil liquefied very fast, in the first second of shaking or so, the recorded accelerations on the foundation and on the soil surface presented in the figure are much lower than the input. The soil surface settled about 20 cm in prototype units, while the foundation settled much more, about 50 cm, with all settlements stopping at the end of the shaking.

Figure 12 presents the pore pressure records for the same test. PF1, PF2, and PF3 are typical of the free field response in a horizontal uniform saturated sand or gravel deposit. After 2 seconds, the whole deposit in the free field is in a state of initial liquefaction. The deeper piezometer PF3 shows dissipation of pore pressures as soon as the shaking ends, while PF1, closer to the surface, stays liquefied a longer time due to the upward water flow. The same as we saw before for the two-layer case, the pore pressures under the foundation are smaller, and in fact, they are negative at PC1 due to dilatancy. Again, the same as before, the pore pressures under the foundation continue increasing after the shaking due to flow of water from the free field. That is, the lowest factor of safety against bearing capacity failure may occur sometime after shaking, rather than during shaking. This is consistent with observations after the 1964 Niigata earthquake in Japan, where multistory buildings were observed to tilt and fail several minutes after the earthquake had ended.

However, in some of the tests we observed something else. Figure 13 presents the pore pressure record from transducer PC3, at some depth under the foundation. This is a test where the pore fluid had an 85% glycerine content, more or less simulating correctly both the dynamic response and consolidation response of a fine sand layer saturated with water in the prototype, full-scale system. After the end of shaking we have the interesting situation that the pore pressure at PC3 continues to increase for a few seconds, despite the fact that the hydraulic gradient is pointing away from it! That is, this pore pressure increase cannot be due to water flow coming from

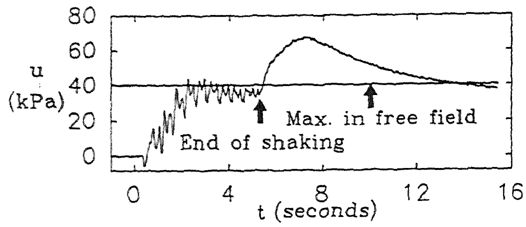

Figure 13. Pore pressure record under the foundation obtained in a uniform sand model with high pore fluid viscosity (GC = 85%).

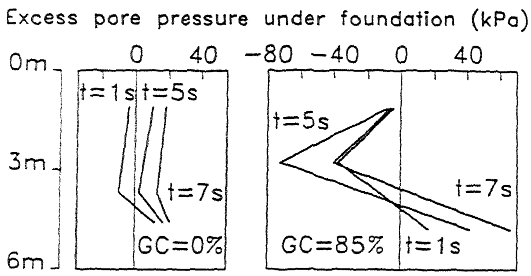

Figure 14. Comparison of pore pressure isochrones under the foundation measured in two tests having different soil permeabilities.

the free field. In fact, after shaking piezometer PC3 had the highest excess pore pressure anywhere in the soil. Our explanation is that as the soil in the free field liquefied, it could not take the static shear stresses applied by the foundation anymore, and the record of Fig. 13 reflects a simultaneous redistribution in total stresses, with the weight of the foundation being transferred to the column of soil directly under the foundation, which thus becomes an island of solid soil in a sea of liquefied material. This is a most interesting phenomenon, and we are now repeating some of these tests but now installing total stress cells both under the foundation and in the free field to verify this stress redistribution hypothesis. A more detailed study of this phenomenon could be very useful to engineers. One of the questions we ask ourselves very often is how much soil should we compact under a foundation or around it for the structure to be safe against liquefaction during the design earthquake. And what we are observing in these centrifuge tests is directly relevant to this question.

Figure 14 shows the effect of permeability on the results of these centrifuge experiments. The graph presents the excess pore pressures versus depth measured under the foundation in two tests: one with pure water (GC = 0) and the other with 85% glycerine and 15% water (GC = 85%). Otherwise the two tests were the same. For the centrifugal acceleration N = 50 g's used in both tests, the test with GC = 85% is close to the correct one which satisfies the time scaling relations for both consolidation and dynamic response. That is, the results for GC = 85% in the figure can be taken as giving more or less the correct response of a prototype fine Nevada sand layer saturated with water. Therefore, we predict that if we put a 5 m foundation diameter (0.10 m × 50), applying a bearing stress of about 120 kPa to the soil, on this fine sand layer 6 m thick overlying rigid impervious rock, with the sand having a 50% relative density, and we shake it at the base with an earthquake acceleration of 0.36 g and a duration of 5 seconds, some of the soil under the foundation will experience strongly negative pore pressures, both during the shaking and some time after it, with very high hydraulic gradients and horizontal water flow from the free field toward the foundation.

This is not what the centrifuge test with GC = 0% shows in Fig. 14. There are practically no negative pore pressures in this case, and the pore pressures equalize too fast with those in the free field. The obvious conclusion is that one has to be very careful in the planning and interpretation of liquefaction earthquake centrifuge model tests, and that a centrifuge test such as that for GC = 0% in Fig. 14 can produce predictions which are quite misleading if applied to a water-saturated deposit in the prototype. In fact, a reasonable prototype interpretation of Fig. 14 is that the results for GC = 85% correspond approximately to a 6 m layer of fine sand, while those for GC = 0% correspond to a 6 m layer of coarse sand or gravel.

The results presented in this section are generally consistent with shallow foundation-on-saturated sand shaking experiments reported by Yoshimi and Tokimatsu (1978) and Whitman and Lambe (1982, 1988), conducted at 1 g and in a centrifuge, respectively. Both groups found that the excess pore pressures were smaller under the foundation than in the free field due to dilatancy, and Yoshimi and Tokimatsu discussed in general terms the possible redistribution of the foundation static stresses due to local softening of the liquefied soil.

CONCLUDING REMARKS

Some general conclusions from this centrifuge modelling work for the seismic response of soil and soil-structure systems and especially for liquefaction phenomena are quite clear.

First, centrifuge modelling can be an effective tool for the study of these phenomena. It is especially suited for the clarification of the actual mechanisms of behavior in the field, and it provides excellent information on the interaction between dynamic response, on the one hand, and consolidation and water flow on the other.

Second, centrifuge modelling is a very versatile tool. It can be applied to purely soil systems like the study of lateral spreading in the free field, as well as to earth dams and embankments. It can also be helpful to study foundations and retaining structures, as illustrated in the paper.

Finally, in addition to its role in clarifying the mechanics of liquefaction phenomena, centrifuge modelling can be used to quantify important factors, to evaluate engineering solutions, and to validate numerical techniques.

ACKNOWLEDGMENTS

The equipment development and the research reported here were supported, respectively, by grant NCEER 90-1507 from the National Center for Earthquake Engineering Research, and by grant BCS-9016880 (VELACS) from the National Science Foundation, U.S.A. The authors also want to thank Mr. Paul Van Laak for his help in performing the centrifuge model tests.

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