EFFECTS OF SAND PERMEABILITY AND WEAK AFTERSHOCKS ON EARTHQUAKE-INDUCED LATERAL SPREADING

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ABSTRACT

Centrifuge research conducted at Rensselaer Polytechnic Institute (RPI) is summarized focusing on the effect of sand permeability and of weak aftershocks on earthquake-induced lateral spreading. One-dimensional shaking tests were conducted in a laminar box at a centrifugal acceleration of 50 g, simulating a 10 m thick homogeneous layer of clean Nevada sand of relative density 40–45% inclined a few degrees to the horizontal. Seven centrifuge experiments were done using either water or a viscous pore fluid fifty times more viscous than water, thus varying the soil permeability by a factor of fifty and simulating either a coarse prototype sand or a fine sand in the field. These seven tests were subjected to the same shape of base input excitation but with peak input accelerations ranging from 0.18 g to 0.46 g between tests. An additional special centrifuge test of a similar model with viscous pore fluid was subjected to a strong base “main shock”, followed after a few seconds by two “weak aftershocks”. Excess pore pressures, accelerations, settlements and lateral ground deformations were measured. The results are discussed in detail and several correlations are presented between testing and measured parameters such as thickness of liquefied soil, ground surface settlement, lateral ground displacement and input peak acceleration. The results of the special test, together with other evidence, provide strong support to the hypothesis that reported continued ground deformations from lateral spreads in the field are often caused by continued weak vibration or aftershocks occurring after the main shock.

Key words: centrifuge model test, lateral spreading, liquefaction, permeability (IGC: E7)

INTRODUCTION

Liquefaction-induced lateral spreading of mildly sloping terrain or of ground near a waterfront is a very common and costly type of ground failure. It affects buildings, bridges, embankments, buried lifelines and waterfront structures. It has occurred in a number of earthquakes in Japan including the 1964 Niigata, 1983 Nihonkai-Chubu and 1995 Hyogoken-Nambu earthquakes; it has also affected areas in California and Alaska in the USA in several earthquakes over the years; and has been reported for many other earthquakes all over the world including, in the last 10–15 years, the 1987 Edgecumbe (New Zealand), 1990 Luzon (Philippines), 1991 Limon (Costa Rica), and 1995 Manzanillo (Mexico), earthquakes (Hamada et al., 1986, 1996; Youd et al., 1992; Hamada and O'Rourke, 1992; O'Rourke and Hamada, 1992; Ishihara, 1993; Bartlett and Youd, 1995; Ishihara et al., 1996; Tokimatsu et al., 1996; Soils and Foundations, 1996; Swan et al., 1996; Berrill et al., 1997).

Recent examples of lateral spreads were reported in the 1999 Kocaeli, Turkey and Chichi, Taiwan, earthquakes (*see* Web site at <http://rccg03.usc.edu/>).

Model testing of lateral spreads in shaking tables at 1 g and in the centrifuge has emerged as a main tool to study the mechanics, calibrate analyses and define parameters for the engineering evaluation of lateral spreading and its effects on constructed facilities. In the last 10–15 years these physical model tests have provided a wealth of useful information and allowed studies that supplement the records and observations from the field (e.g., Towhata et al., 1988, 1989; Sasaki et al., 1991a, b, 1992; Yasuda et al., 1991; Scott et al., 1993; Dobry and Taboada, 1994; Fiegel and Kutter, 1994; Dobry et al., 1995; Taboada et al., 1996; Tsunoda et al., 1997; Taboada and Dobry, 1998; Dobry and Abdoun, 1998; Kokusho, 1999; Sharp, 1999). Some of the model test results previously reported in these publications, have been obtained by the authors at the 100 g-ton geotechnical centrifuge at Rensselaer Polytechnic Institute (RPI) in the US. They have helped

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clarify the basic mechanics of lateral spreading in clean sand, as well as the significance for lateral ground deformation of factors such as soil relative density, thickness of liquefiable layer, sand permeability, ground slope and intensity, duration and frequency content of the input shaking.

This paper focuses on the effect of soil permeability on lateral spreading for a range of peak input ground accelerations between about 0.2 g and 0.5 g. It also discusses a hypothesis, initially advanced by Dobry et al. (1995), to explain the additional ground deformations which have been observed by witnesses in the field after the main shaking (e.g., Kawakami and Asada, 1966; Hamada, 1992). In this hypothesis, this delayed ground deformation after the main shock is due to smaller continued vibrations or aftershocks acting on the already liquefied soil. One-dimensional, infinite slope centrifuge test results on clean sand using water and a viscous pore fluid are systematically discussed and compared to clarify the effect of soil permeability; and a centrifuge experiment is presented which was specifically conducted to verify the proposed hypothesis of the cause for additional ground deformations after the main shock. All these centrifuge models, listed in Table 1, were tested at RPI, either by the main author or by the other co-authors; some of the results have also been reported in other publications (Taboada, 1995; Taboada and Dobry, 1998; Sharp, 1999).

RESEARCH SCOPE, MODEL PREPARATION AND TESTING PROCEDURES

Soil and Pore Fluid

Nevada 120 sand deposited at a relative density $D_r=40-45\%$ was used in all eight centrifuge tests reported in this paper (Table 1). This sand is composed predominantly of quartz particles with small amounts of feldspar, with the grain shapes ranging from rounded to subrounded. It is a fine sand of $D_{50}=0.15$ mm and a permeability (for the sand at $D_r=40\%$, tested at 1 g using water as pore fluid) of $k=0.0021$ cm/s. The specific gravity of the sand is 2.68 and its minimum and maximum void ratios are $e_{\min}=0.516$ and $e_{\max}=0.894$, respectively. Additional information on this soil is provided by Arulmoli et al. (1992).

The model sand deposits utilized in this centrifuge study were saturated, either with water (Tests W1 to W4 in Table 1) or with a viscous fluid (Tests V1 to VA in Table 1). The viscous fluid was a mixture of water and hydroxypropyl methylcellulose (type 65SH-50) termed Metolose from Shin-Etsu Chemical Company (Shin-etsu Chemical, 1997). This Metolose solution pore fluid was prepared by dissolving 2% Metolose by weight in water, so as to achieve a viscosity of about 50 times the viscosity of water (50 cst kinematic viscosity).

The use of a Metolose solution pore fluid of 50 cst in Tests V1 to VA was prompted by the use of a 50 g centrifugal acceleration—and hence a prototype/model scaling factor of 50—in all experiments reported in this

Table 1. Summary of test conditions and results in prototype units (all tests were conducted at 50 g centrifugal acceleration)

Test	Peak acceleration of input motion, $a_{\max}$ (g)	Frequency of input motion, f (Hz)	Inclination of model, α (degree)	Inclination of prototype, α_{field} (degree)	Relative density, D_r (%)	Thickness of liquefied layer, H_l (m)	Lateral ground displacement, D_H (m)	Final settlement of ground surface, S (m)	Permanent vertical strain, S/H_l (%)	References
V1	0.23	2	2.0	5.0	45	7.8 (10*)	0.66	0.12	1.5	Sharp (1999)
V2	0.38	2	2.0	5.0	43	9.0 (10*)	0.85	0.09	1.0	
V3	0.41	2	2.0	5.0	45	9.3 (10*)	0.85	0.16	1.7	Sharp (1999)
VA	0.48	2	2.0	5.0	44	7.9 (10*)	0.82	0.51	6.4	
W1	0.18	2	2.0	5.0	45	3.7 (3.5)	0.44	0.09	2.4	Taboada (1995), Taboada and Dobry (1998)
W2	0.23	2	1.9	4.8	45	4.0 (3.5)	0.47	0.12	3.0	
W3	0.38	2	2.0	5.0	42	7.5 (6.7)	0.71	0.19	2.5	
W4	0.46	2	2.2	5.4	40	8.9 (10*)	0.97	0.20	2.2	Taboada (1995), Taboada and Dobry (1998)

1) The letter "V" in the test code indicates viscous fluid saturated models, while "W" represents the water-saturated model.

2) Numbers in parentheses in the column of H_l indicates thickness of liquefied layer determined from piezometric isochrones.

* H_l from isochrones for a model without pore pressure transducers installed beneath the liquefied/nonliquefied boundary.

paper. The purpose of centrifuge testing with a viscous pore fluid is to simulate more closely the prototype permeability; comparison with similar centrifuge tests using water allows investigating the effect of permeability on liquefaction and lateral spreading.

One consequence of using water in a centrifuge test at 50 g to model liquefaction of water-saturated prototype soil in the field is that the actual prototype permeability being simulated is 50 times larger than that of the model (Tan and Scott, 1985). In this case, $k_{\text{prototype}} = 0.0021 \times 50 = 0.105 \text{ cm/s}$, which corresponds to the permeability of a coarse sand in the field rather than that of the fine sand used in the centrifuge experiment. This is the case of the four centrifuge tests using water as pore fluid reported here (Tests W1 to W4). On the other hand, if the model is saturated with a viscous fluid 50 times the viscosity of water, the centrifuge test simulates a water-saturated prototype soil permeability of the same sand tested, that is $k_{\text{prototype}} = 0.0021 \text{ cm/s}$, corresponding to fine Nevada sand in the field (tests V1 to VA). Therefore, the eight centrifuge models listed in Table 1 simulate induced liquefaction and lateral spreading in the field of the same prototype layer (homogeneous deposit of a uniform clean sand of $D_r = 40\text{--}45\%$ on a firm impermeable base), but having two different permeabilities. That is, four of the tests simulate the liquefaction and lateral spreading response of a coarse sand while the other four simulate a fine sand in the field. Comparison between results of corresponding tests can then be used to examine the influence of sand permeability on liquefaction and lateral spreading response.

A main question that arises when a viscous pore fluid like the Metolose solution is used is whether the fluid affects the mechanical properties of the soil other than permeability. Results of laboratory and centrifuge tests have shown that the constitutive properties of Nevada sand are not altered significantly when a Metolose solution is used (Dewoolkar et al., 1999; Stewart et al., 1998). Also, the density of the viscous fluid, 1.01 g/cm^3 , is essentially identical to that of water, because the amount of Metolose mixed with water is small; thus no correction is needed because of different pore fluid densities between model and prototype. Therefore, it will be assumed here that all eight tests listed in Table 1 simulate the same field situation, differing only in their soil permeability and in the base shaking used in the test.

Laminar Box

A state-of-the-art, rectangular laminar container built at Rensselaer Polytechnic Institute (RPI) was used in the eight centrifuge tests. The box is designed to shake a plane strain geotechnical model in the long direction of the box. This box is optimized to accommodate and measure accurately a wide range of cyclic and permanent lateral displacements occurring in the soil model. A side view of the instrumented laminar box is presented in Fig. 1(a). Most of the box is made of a high strength aluminum alloy. This box is comprised of a stack of up to 39 rectangular rings separated by linear roller bearings, and has inter-

Fig. 1. Modeling of lateral spreading using laminar box container

nal dimensions of 25.4 cm wide, 45.7 cm long and 25.4 cm deep.

Two aspects of the design of the laminar box are worth mentioning. Thin rings are used to increase flexibility. This is done to approximate closely a continuous shear strain field in the soil during shaking and accommodate possible shear strain concentrations. Also, bearings have been inserted between rings at the end of the container. This complicated the design considerably but was necessitated by the large complementary shear forces developed at the ends of the container and by the need for additional vertical support due to the increased flexibility of the thin rings. Also, because the main anticipated use of the box was in the modeling of liquefaction and lateral spreading of saturated sand, the device incorporates provisions for large lateral displacement. More detailed information on the box is presented by Van Laak et al. (1994a) and Van Laak (1996).

Model Preparation

A latex rubber membrane, 0.2 mm in thickness, was used to line the inside of the laminar box and prevent leakage of the contents. During model construction, the external side of the box was sealed and a vacuum of several kPa was applied, which forced the inner latex membrane flush against the rings. The dry sand was rained into the laminar box to a depth of 20 cm through a funnel with a row of holes 2 mm in diameter and spaced 5 mm apart. The width of the funnel equals the width of the laminar box. The box was manually moved back and forth along the longest dimension of the box and the free falling height was kept constant to provide uniform sand

deposits with the desired relative density. During sample preparation, accelerometers and pore pressure transducers were installed at the proper orientations and locations. The sand surface was prepared flat, the sample height was measured at several points and averaged to give the initial model height.

De-aired pore fluid (water or Metalose solution depending on the test) was introduced through the top of the model under a vacuum of 95 kPa for saturating the soil model until the fluid level in the laminar box rose above the surface of the soil. Then the vacuum was released and LVDTs were placed; one for measurement of surface settlement at the centerline, and six for lateral displacements of the rectangular rings—and thus of the soil—at different depths in Tests V2, W3 and VA as shown in Fig. 1(a). The specific arrangement of the LVDTs in the other tests was similar and can be found in Taboada (1995) and Sharp (1999). The laminar box was moved onto the centrifuge platform and the centrifuge was brought to 50 g gradually in about 30 minutes in order to ensure a smooth transition of the soil model from 1 g to 50 g. Then one dimensional lateral shaking was imparted along the model long axis using an electro-hydraulic shaker (Van Laak et al., 1994b) while in flight at 50 g. During all tests, data was recorded at a sampling rate of 3000 samples/sec. with a filter cut-off frequency of 600 Hz.

Test Conditions

A total of eight centrifuge model tests were carried out as listed in Table 1. Seven of these experiments were similar in all respects, except that two main testing parameters were varied between tests: the permeability of the soil and the peak input acceleration, $a_{\max}$, of the base shaking. These seven models consisted of a depth of approximately 20 cm of Nevada sand having a relative density of 40–45%. Three of these models (V1–V3) were saturated with the viscous fluid (Metalose solution) having a viscosity 50 times that of water, while the other four (W1–W4) were saturated with water. All time histories of the lateral input shaking applied in-flight to the base of these seven models have the basic shape indicated in Fig. 1(b), which is in prototype units. That is, the input time history to the model consisted of 22 cycles of a sinusoidal wave with a model frequency of 100 Hz and variable acceleration amplitude. The peak acceleration of this input motion changed between tests, ranging from 9 g to 23 g in model units. For the 50 g centrifugal acceleration, this corresponds to a prototype frequency of 2 Hz and a prototype peak acceleration, $a_{\max}$, ranging between 0.18 g and 0.46 g (Table 1), applied to the base of a homogeneous sand layer with a prototype thickness of 10 m and a relative density of 40–45%. As described later, in addition to these seven experiments, Test VA was conducted on a similar Metalose solution-saturated model excited laterally with the base prototype acceleration time history shown in Fig. 18(c).

The laminar box as well as the electro-hydraulic shaker were inclined in all tests at an angle $\alpha = 2^\circ$ to the horizontal by placing a wedge between the centrifuge platform

Fig. 2. Sand layer submerged below water and confined by laminar box rings

and the shaker. There are certain considerations and corrections to this slope angle that should be taken into account when interpreting the test results in terms of the prototype. Figure 2 sketches the submerged soil model in the rectangular rings tilted by an angle $\alpha = 2^\circ$. For an infinite slope in the field, the effective static stress ratio prior to the shaking, acting on a plane parallel to the slope, τ/σ' , is equal to $\tan \alpha$, where τ is the shear stress and σ' is the effective normal stress on the plane. However, after considering in the model the combined effect of the weight of the rings parallel to the slope, the different hydrostatic pressures at both ends of the rectangular rings, and the friction between rings, the average effective stress ratio acting on a soil plane parallel to the slope in the model is equal to $\tan \alpha_{\text{field}} > \tan \alpha$, where $\tan \alpha_{\text{field}}$ is given by Eq. (1) (Taboada, 1995):

$$\tan \alpha_{\text{field}} = \frac{\tau}{\sigma'} = \left(1 + \frac{W_r}{A\gamma'} + \frac{\gamma_w}{\gamma'} \right) \tan \alpha - K_f \frac{W_r}{A\gamma'} \quad (1)$$

In Eq. (1), W_r ($=0.660$ kN/m) is the weight of the rings including bearings per unit height, A ($=0.116$ m²) is the plane area of the model, γ_w ($=9.81$ kN/m³) and γ' (≈ 9.50 kN/m³) are the unit weight of water and the submerged unit weight of the sand, respectively, and K_f ($=0.01$) is the coefficient of friction between rings. Thus the initial shear stress of the models listed in Table 1 is equivalent to a prototype having a slope angle $\alpha_{\text{field}} = 4.9^\circ$, rather than $\alpha = 2^\circ$ used in the modes. Therefore, the models simulate a fully submerged, infinite, approximately 5° sand slope, subjected to shaking parallel to the slope. Also, there is an additional dynamic shear stress imposed on the soil during shaking due to the inertia of rings and bearings, which is equivalent to an increase in lateral accelerations in the prototype in the free field by a factor of 22%, as compared with the lateral accelerations measured in the model. Further information on these testing scheme and corrections can be found elsewhere (Taboada and Dobry, 1993; Dobry and Taboada, 1994). It should be noted that the word “lateral” instead of horizontal and “normal” instead of vertical are used throughout the paper to remind readers that the whole model, including instruments, as well as the shaker are tilted with respect to the horizontal direction. In the following discussions all results and comparisons are presented in prototype units unless mentioned otherwise.

Fig. 3. Excess pore pressure records

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Effect of Soil Permeability on Measured Pore Pressures

Figure 3 compares the excess pore pressure ratios, r_u , measured in Tests W3 and V2, where water and the Metolose solution were used as pore fluid, respectively, and with both tests having an input peak acceleration of 0.38 g. The broken horizontal lines in the figure indicate $r_u=1.0$, that is the condition of sand liquefaction. The figure shows clearly the effect of soil permeability on pore pressure dissipation after shaking. For Test W3, r_u starts to decrease at all elevations at most a few seconds after the end of shaking, while it stays close to unity for a much longer time in Test V2.

The excess pore pressure isochrones of the same two experiments are included in Fig. 4. The broken lines in the figure represent the initial effective vertical stress, and thus the same liquefaction condition associated with $r_u=1.0$. It can be seen that at the beginning of shaking ($t=6$ and 7 sec), the excess pore pressures (and thus also r_u) were generally higher in Test V2 than in Test W3 (except for the depth, $z=7.5$ m at $t=6$ sec). It can also be observed that in Test V2 liquefaction progressed down to at least $z=7.4$ m or deeper toward the end of shaking, while it stayed at a depth somewhere between 5.0 and 7.5 m in Test W3. This indicates that when the permeability decreases (viscous fluid saturated model), liquefaction reaches a greater depth.

Effect of Soil Permeability on Measured Soil Accelerations

The acceleration records obtained in the soil from Tests W3 and V2 are depicted in Fig. 5. In both experi-

Fig. 4. Excess pore pressure isochrones

ments, the acceleration time history at the shallow depth, $z=0.5$ m, follows the imparted base motion during the first one or two cycles of shaking, and then the amplitude of the acceleration attenuates significantly, indicating that the mass of soil beneath has liquefied, effectively isolating the shallow soil from the base motion. The most

Fig. 5. Accelerations in soil

important feature in this figure is the strong asymmetry of the records in much of the soil mass, characterized by negative (upslope) large amplitude spikes, where the downslope direction is taken as positive for both lateral displacement and acceleration. In Test W3, this involved the soil between about $z=2.5$ m and $z=7.5$ m. It was reported by Taboada and Dobry (1998) that this asymmetry was not observed in the measured accelerations of horizontal models, and thus is clearly restricted to the case of sloping sand layers. These spikes are explained by a strength increase in the saturated sand because of its dilative stress-strain response in part of the cycle. This is also confirmed in Figs. 3 and 5 by the coincidence in timing between the negative acceleration spikes and the dips in the corresponding pore pressure record at the same depth, as also shown by Dobry et al. (1995) and Zeghal et al. (2001). This asymmetric behavior is restricted to accelerometers located above the liquefied/nonliquefied boundary, and the asymmetry and amplitude of the spikes is typically maximum for the liquefied soil closer to that boundary.

Maximum Depth of Liquefied Layer, H_l

This section discusses the effects on the maximum thickness of liquefied soil, H_l , for a given centrifuge experiment, of the amplitude of peak input base acceleration, $a_{\max}$, and soil permeability, that is of the two testing parameters varied between the tests listed in Table 1. One possible procedure to obtain the value of H_l is to make direct use of the piezometric isochrones such as shown in Fig. 4 (Taboada, 1995). In this procedure, H_l is the maximum depth of the boundary between liquefied soil (where $r_u=1.0$) and nonliquefied soil. However, since the number of pore pressure cells arrayed in the model in this study was limited, the value of H_l determined this way may not be accurate.

An alternative procedure for H_l is to extrapolate back to the end of shaking the depth of the liquefied/nonliquefied boundary. This procedure assumes that the depth of liquefaction is maximum at the end of shaking, which is the case for all eight tests of Table 1. As illustrated in Fig. 3 for Test W3, dissipation of excess pore pressures starts at depth, with this boundary or solidification front

Fig. 6. Relationship between depth and time when solidification front passed

moving up with time until it reaches the ground surface. Above this liquefied/nonliquefied boundary the sand particles are in suspension in the pore fluid, and they settle and regain contact typically some time after the base shaking stops (Florin and Ivanov, 1961). As the settling particles reach solid material of nonliquefied underlying soil, they accumulate to form a solidified zone which increases in thickness with time. In the liquefied region above the boundary, the pore pressure at any depth remains at a constant level corresponding to $r_u = 1.0$ until the solidification front reaches that depth. After the shaking, the solidification front moves upward at essentially a constant speed for a uniform deposit until it reaches the surface (Scott, 1984). Figure 6 shows plots of the depth of the solidification front versus elapsed time from the end of shaking based on the piezometric readings from Tests V2 and W3. As some of the measurements of r_u in the liquefied soil were slightly smaller than the condition $r_u = 1.0$, Fig. 6 was constructed by taking as the time of passage of the solidification front at a given depth the time at which the pore pressure begins to decrease at this depth. It can be seen that the elapsed time in Fig. 6 increases approximately linearly with decreasing depth, indicating upward propagation of the solidification front at a constant speed. The intersection of the least-square-fit straight line in the figure provides the depth of solidification front at the end of shaking, which is also H_l for that particular test. From Fig. 6, $H_l = 9$ m and 7.5 m for Tests V2 and W3, respectively. Note that the lines for Tests V2 and W3 in Fig. 6 correspond to different time scales along the abscissa axis.

The values of H_l obtained by this procedure are summarized in Table 1 for all eight tests. It is of interest to note that the speed of propagation of the solidification front, obtained from the slopes of the lines in Fig. 6 is approximately 50 times higher for Test W3 than for Test V2. According to theory, this speed of propagation should be proportional to the coefficient of consolidation of the liquefied soil (Scott, 1984), and thus should be proportional to the soil permeability for these tests. Therefore, the

Fig. 7. Variation of thickness of liquefied soil with input peak acceleration

measured factor of 50 in the slopes of the lines in Fig. 6 is completely consistent with the fact that prototype soil permeability is 50 times greater in Test W3 compared to Test V2. It should also be pointed out that H_l determined from the solidification front is also quite consistent with H_l determined from the piezometric isochrones and published before for some of the same tests by Taboada (1995) and Sharp (1999). The difference between H_l derived from the two methods was found to be less than 22% as listed in Table 1. However, measurement from pore pressure transducers just above and beneath the liquefied/nonliquefied boundary should play an important role in determining H_l from isochrones; therefore, H_l from isochrones for a model without pore pressure transducers installed just above and beneath the boundary could not be very accurate.

Figure 7 depicts the influence of input peak acceleration, a_{max} , on H_l . As a_{max} increases, H_l also increases from about 3 to 10 m, with H_l obtained from the tests with viscous pore fluid (Metolose solution) being greater than that from the tests using water. This strongly suggests that an undrained condition is not satisfied during shaking in the tests using water because of the high soil permeability. The difference in H_l due to soil permeability is apparent when a_{max} is relatively small, at around 0.2 g, and becomes less marked as a_{max} increases and H_l approaches 10 m. Of course, this conclusion may be restricted to cases in which the depth of liquefiable sand is 10 m, as is the case for all tests discussed here, and may not be valid for other test or field situations where a greater thickness of liquefiable sand is involved. For the highest values of a_{max} adopted in the tests presented here, of the order of 0.4 g, and for the duration of the shaking used, even in the water saturated models the shaking was strong enough to liquefy close to the whole 10 m total thickness of the layer.

Settlements and Lateral Ground Deformations

The height of the sand surface was measured at several

points at 1 g, just before and immediately after the centrifuge test to obtain the corresponding settlements. Typical settlement profiles are displayed in Fig. 8 for two of the tests. Most of settlements shown in the figure occurred due to the shaking and subsequent liquefaction of the sand. The static settlement due to the centrifugal acceleration increase from 1 g to 50 g, measured by LVDT1 (Fig. 1) at the center of the model was about 1–3 cm, significantly less than the 6–11 cm total settlement revealed by Fig. 8. The figure shows that the settlement was relatively uniform along the long axis of the box, except in the vicinity of the downslope wall of the model container, where there was less settlement. The net result was that the slightly inclined ground surface became more horizontal after shaking, as expected. In addition, the pore pressure records along the centerline (P1–P4 in Fig. 1) and the quarterline (P5–P8 in Fig. 1) are compared in Fig. 9, confirming that they are quite similar at comparable depths, as previously reported by Dobry et al. (1995). These observations indicate that the desired unidimensional infinite slope condition was achieved in most of the model.

Fig. 8. Normal settlement of ground surface

Fig. 9. Excess pore pressure ratios measured on centerline and quarterline (Test V2)

Figure 10 shows the accumulation of lateral displacement at several depths for Tests V2 and W3. In Test W3, the lateral movement stopped as soon as the shaking ended at all elevations (Fig. 10(b)). This was also true for all tests using water in Table 1 (W1–W4), and this has always been true with other similar tests on water-saturated sand and inclined surface done at RPI (Taboada, 1995; Sharp, 1999) and Caltech (Scott et al., 1993), independently of the values of α_{field} , a_{max} , frequency of input motion and relative density of the sand. In these tests using water and thus having a high soil permeability, the excess pore water pressures also started dissipating when the shaking ended, as illustrated by Fig. 3. On the other hand, in low permeability Test V2 shown in Fig. 10(a), the lateral displacement also stopped as soon as the shaking ended, despite the fact that the excess pore pressure of the liquefied sand did not dissipate until much later, and thus the sand remained in an extremely low effective stress condition during a much longer time than in Tests W1–W4. Lateral displacement for the other low permeability tests are shown in Fig. 11, which confirms that lateral displacement stopped when shaking ended for all viscous fluid saturated model tests conducted in this paper.

Figures 12 and 13 show the evolution of the lateral displacement and shear strain profiles, respectively, with time of shaking for Tests V2 and W3. The shear strain is the average strain between two LVDT's at adjacent elevations, obtained as the difference between the LVDT displacement measurements divided by the normal distance between the two LVDT's. In Fig. 12, a very small displacement can be seen in both tests at a depth of about 9.2 m, which is greater than H_1 and therefore corresponds to sand that did not liquefy. It can be observed in Fig. 13 that the maximum shear strain was attained for Test V2 at a depth somewhere between 7.6 m and 9.2 m, and for Test W3 between 5.0 m and 9.1 m. Both ranges of depths enclose the corresponding boundary separating liquefied and nonliquefied soil, as shown by the horizon-

Fig. 10. Lateral displacement measured at several depths

Fig. 11. Lateral displacements measured near surface

*: from extrapolation of the record shown in Fig. 10

Fig. 12. Evolution of lateral displacement with time

tal broken lines in Figs. 13(a) and (b). At the end of shaking, the relative displacement contributed by these ranges of depths is of the order of one half to two thirds of the total lateral displacement measured at the ground surface, D_H . This suggests that the boundary between liquefied and nonliquefied soil tends to act as a failure surface or zone of high straining during the development of

downslope permanent lateral displacement during shaking, consistent with the observations of Dobry and Taboada (1994) and Sharp (1999).

Fig. 13. Evolution of shear strain with time

Fig. 15. Variation of D_H with H_l Fig. 14. Relationship between D_H and a_{max}

Fig. 16. Normal settlement due to shaking

Figure 14 is a plot of lateral displacement measured after shaking at the ground surface, D_H , versus a_{max} for the seven tests. The value of D_H increases with increasing a_{max} regardless of whether the models were saturated with water or with a viscous fluid. It is interesting to correlate D_H with the maximum thickness of liquefied soil H_l discussed earlier. Figure 15 gives the variation of D_H with H_l for the same seven tests. It is observed in this plot that D_H increases as the thickness of the liquefied layer increases, as expected based on previous observations in the field (Hamada et al., 1986 and Bartlett and Youd, 1995) and centrifuge (Taboada and Dobry, 1998). Furthermore, D_H appears to be related more or less uniquely to H_l in Fig. 15, irrespective of the input peak acceleration and prototype soil permeability. Taboada and Dobry (1998) established the independence of this correlation between D_H and H_l from the input shaking for a given α_{field} . Figure 15 suggests that the correlation may also be independent of soil permeability.

The variation with a_{max} of the ratio of final settlement of the ground surface to the maximum thickness of liq-

Fig. 17. Settlement time histories

ued layer, S/H_l , is depicted in Fig. 16 for the same seven tests. The ratio S/H_l is larger for the higher permeability tests than for the viscous fluid saturated

tests, without significant influence of the peak acceleration. It also appears from settlement time histories recorded by LVDT1 shown in Fig. 17 that the settlement during shaking is generally larger in the water saturated tests, while the settlement after shaking is larger in the viscous fluid saturated tests. These facts may suggest that the expulsion of water from the soil during shaking is a more efficient way of compaction than post-liquefaction consolidation after shaking. A possible explanation for this may be that the settlement of the grains during shaking occurs in a high energy environment while the post-shaking consolidation is a purely static phenomenon.

EFFECTS OF AFTERSHOCKS ON LATERAL SPREADING

An important conclusion from the centrifuge tests described above—using either water or the Metalose solution—is that lateral ground deformation in lateral spreads in mildly sloping ground stops as soon as the shaking ends. That is, even if the soil continues to be liquefied for some time after the end of shaking, lateral deformation stops accumulating as soon as the shaking ends, indicating that there is no further lateral displacement accumulation without shaking. This had been noticed before for shaking table tests at 1 g of models of sloping clean sand layers conducted mainly by Japanese researchers (Towhata et al., 1988; Sasaki et al., 1991a, b, 1992; Yasuda et al., 1991), as well as for centrifuge tests done at RPI and Caltech (Taboada and Dobry, 1993; Scott et al., 1993; Dobry and Taboada, 1994). This evidence from centrifuge and 1 g shaking tests on clean sands—having relative densities of the models ranging between 0% and 100%, always using water as pore fluid—was reviewed in detail by Dobry et al. (1995), who suggested additional centrifuge tests using a viscous pore fluid to verify that the conclusion is truly general and can be extrapolated to field conditions for mildly sloping terrain and clean sand. Centrifuge Tests V1–V3 presented here confirm that this is the case, and that in clean uniform sand with water or viscous pore fluid, in both 1 g and centrifuge experiments, lateral deformation of slightly sloping grounds invariably ends when the shaking stops. However, this rule does not necessarily apply to a liquefied layer under an impervious fine-grained layer due to potential for accumulation at the top of the sand of a water film of essentially zero shear strength (Fiegel and Kutler, 1994; Kokusho, 1999), or to points in a lateral spread close to a steep free face having large static driving shear stresses (e.g. Tsunoda et al., 1997), where other mechanisms are present which may allow lateral deformation to continue accumulation after shaking.

This would appear at first sight to be in contradiction to observations in Japan that permanent lateral ground deformations have been reported by witnesses to continue after shaking (e.g. 1964 Niigata earthquake, as reported by Kawakami and Asada (1966) and Hamada (1992)). In the opinion of the authors, previously advanced by Dobry et al. (1995), this continued ground

deformation was probably caused by shaking due to continued small vibration after the main shaking, or to aftershocks, with this vibration or aftershocks being weak enough not to be felt by the witnesses on the ground surface effectively isolated from the earthquake motion by the underlying liquefied layer, but strong enough to cause significant cyclic and permanent strains and displacements in the already liquefied soil. This capacity of liquefied soil to develop large cyclic shear strains under small accelerations after the end of strong shaking, was demonstrated by Zeghal and Elgamal (1994) through System Identification analysis of accelerograms above and below the liquefied layer, at a site in Southern California in the USA that liquefied in 1987. In fact, more than ten aftershocks, one with a Magnitude as high as 6.1, occurred in the same general area within twenty minutes after the main shock in Niigata in 1964 (The Japan Meteorological Agency, 1964). Weak vibration or aftershocks seconds or minutes after the main shock seem to be a common occurrence after earthquakes, including the 1995 Kobe earthquake (The Japan Meteorological Agency, 1995; Ishihara and Cubrinovsky, 1998; Meneses-Loja et al., 2000), and, more recently, the 1999 Kocaeli earthquake in Turkey (see Web site at <http://kandilli.koc.net/earthquake.htm>).

In order to support the authors' hypothesis that large lateral spreading continuing to occur after the strong shaking, especially at a significant distance from the waterfront where the static driving shear stress was small, was due to the action of subsequent smaller shaking on the already liquefied soil, an additional centrifuge test (Test VA in Table 1) was carried out. A model saturated with the Metalose solution was prepared in exactly the same way as done for Tests V1–V3, and tested also at 50 g centrifugal acceleration. That is, this model in Test VA simulates a fully submerged, 10 meter thick infinite sloping sand layer in the field having the material properties of fine Nevada sand of $D_r=45\%$, but now subjected to the shaking shown in Fig. 18(c). This prototype input acceleration time history at the base of the box consisted of a "main shock" of prototype peak acceleration of about 0.5 g, followed by two "aftershocks" of 0.09 g and 0.04 g, respectively, and with the three shocks separated by 4-second intervals. The exact duration of the mainshock of 3.5 seconds in this test is probably not very important provided that it liquefies the sand. For each of the aftershocks, a duration of 10 seconds was selected, considered to be long enough to verify whether the liquefied soil continued to move with these small accelerations.

The soil accelerations, excess pore pressures and lateral displacements recorded in Test VA are shown in Figs. 18–21. As revealed by Fig. 20, the pore pressure ratios reached the value $r_u \cong 1.0$ indicating liquefaction during the main shock, indicating that, similarly to the previously discussed Test V2, the depth of liquefaction was greater than 7.5 m. Figures 18(b) and 20 clearly show that the pore pressures continued to stay high and close to the condition $r_u=1.0$ throughout the aftershocks, with Fig. 19 showing very small accelerations near the ground sur-

Fig. 18. Lateral ground surface displacement contributed by main shock and successive aftershocks acting on already liquefied soil

Fig. 20. Excess pore pressure time histories

Fig. 19. Acceleration time histories

face. The time history of the recorded lateral displacement at the ground surface is shown in Fig. 18(a), while Fig. 21 depicts the lateral displacement profile at the end of the main shock and of each aftershock. It can be ob-

Fig. 21. Evaluation of lateral displacement during mainshock and aftershocks for Test VA

served that the lateral displacement at ground surface, D_H , was 40 cm at the end of the main shock, and that the lateral deformation accumulation stopped as soon as the main shock ended. The accumulation of lateral ground deformation then resumed when the weak aftershocks of 0.09 g and 0.04 g excited the already liquefied soil. These two aftershocks doubled the value of D_H from 40 cm to

80 cm. These centrifuge results strongly support the hypothesis by the authors, that significant lateral spreading of mildly sloping clean sand such as observed in Niigata in 1964 may be due to the combined effect of driving static shear stresses and low amplitude shaking after the main shock, acting on the already liquefied soil. It should be noted here again that static driving shear stresses imposed in the centrifuge tests were much smaller than those at points near the waterfront in Niigata where very large movements of the order of 10 m took place, but more representative of points at a significant distance from the waterfront, perhaps of the order of 100 m from the water. And at about 100 m from water, lateral spreading in Niigata 1964 was not 10 m, but mostly less than several meters, much more similar to the displacement of about 1 m observed in the tests.

A similar possibility of continued lateral deformation accumulation due to low-amplitude shaking acting for several minutes on the already liquefied soil has been discussed by Ishihara and Cubrinovski (1998) and Meneses-Loja et al. (1998, 2000) to explain large ground deformations produced by the 1995 Kobe earthquake. On the basis of results from torsional monotonic simple shear tests with a superimposed simultaneous cyclic axial stress, these authors explain the phenomenon in the field by low-amplitude shaking of the already liquefied soil causing a change in character of the soil from dilative to contractive and reduction of its residual shear strength. On the other hand, Dobry et al. (1995) assumes that the liquefied soil continues to be dilative and that the continuation of lateral deformation by low-amplitude shaking is due to the combination of inertial and static shear stresses that act only during each cycle of shaking, with the actual value of residual shear strength playing no role or a minor role. In principle, the results of centrifuge Test VA shown in Figs. 18–21 is consistent with either explanation. While further research is clearly needed in order to define the exact mechanism and to provide analytical techniques capable of reliable engineering evaluation of this lateral spreading accumulation, the results of centrifuge Test VA, of other centrifuge and 1 g shaking table tests as well as of laboratory tests and field observations in earthquakes such as Niigata 1964 (e.g. Kawakami and Asada, 1966) and Kobe 1995 (Meneses-Loja et al., 2000), strongly suggest that weak vibration or aftershocks acting on the already liquefied soil may contribute significantly to the accumulation of lateral ground deformation in lateral spreads.

CONCLUSIONS

The paper summarizes research using the RPI centrifuge on the effect of sand permeability and of weak aftershocks on earthquake-induced lateral spreading. One-dimensional shaking tests were conducted in a laminar box at a centrifugal acceleration of 50 g, simulating a 10 m thick homogeneous layer of clean Nevada sand of relative density 40–45% inclined a few degrees to the horizontal. Seven centrifuge experiments were done using either

water or a viscous pore fluid fifty times more viscous than water, thus varying the soil permeability by a factor of fifty and simulating either a coarse prototype sand or a fine sand in the field. These seven tests were subjected to the same shape of base input excitation but with peak input accelerations ranging from 0.18 g to 0.46 g between tests. An additional special centrifuge test of a similar model with viscous pore fluid was subjected to a strong base “main shock”, followed after a few seconds by two “weak aftershocks”.

The following main conclusions are obtained from these results:

1. As the sand permeability decreases, liquefaction reaches a greater depth. In both low and high permeability sands the thickness of liquefied soil increases with increasing input peak acceleration. The dissipation of pore pressures after shaking is also much slower in the lower permeability sand.
2. In both high and low permeability sand, after liquefaction triggering, the acceleration records in the liquefied soil mass become asymmetric, with high-frequency upslope spikes in every cycle occurring simultaneously with pore pressure dips. This indicates the soil dilative response consistent with that observed in other reported centrifuge and 1 g shaking table tests.
3. Consistently with previous studies in 1 g and centrifuge shaking tests in clean sand, lateral ground deformations only developed during shaking and stopped as soon as the shaking ended. This was true in both high and low permeability sand, and it is most probably related to the dilative response of the liquefied soil. The ground surface lateral displacement at the end of shaking, D_H , increased with input peak acceleration in both high and low permeability sand, with D_H being greater in the low permeability soil. However, the relation between increasing D_H and increasing thickness of liquefied soil was unique and independent of both peak acceleration and soil permeability; this is consistent with previous studies.
4. In the special centrifuge test with viscous pore fluid, where the inclined sand layer was subjected to strong input shaking followed after quiet intervals by two weaker aftershocks, the value of D_H measured after the main shock doubled due to the two aftershocks (from 0.4 to 0.8 m) in this particular test. This provides strong support to the hypothesis that weak aftershocks or continued vibration after the main shock, acting on the already liquefied sand, is a main cause for the continued ground deformations reported by witnesses after the main shaking in some earthquakes.

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