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ABSTRACT

Densification (Compaction) of loose saturated soils has been the most popular method of reducing earthquake related liquefaction potential. Compaction of a foundation soil can be economical when limited in extent, leading to a case of an "island" of improved ground (surrounded by unimproved ground). The behavior of the densified sand surrounded by liquefied loose sand during and following earthquakes is of great importance in order to design the compacted area rationally and optimize both safety and economy. This problem is studied herein by means of dynamic centrifuge model tests. The results of two heavily instrumented-dynamic centrifuge tests on glycerin-water saturated models of loose and dense sand, prepared adjacent to each other are discussed. Observed model response provided an improved understanding of dynamic-liquefaction behavior of a densified ground surrounded by a loose liquefiable ground. The test results suggest the following concerns about "Islands" of densified soil: 1) there is a potential strength loss in the densified zone as a result of pore pressure increase due to migration of pore water (or fluid) into the island from the adjacent (loose) liquefied ground; 2) there is a potential for lateral deformation (sliding) within the densified island as the surrounding loose soil liquefies.

KEYWORDS

Liquefaction countermeasure, densification, soil improvement, centrifuge testing, pore fluid migration.

INTRODUCTION

The effect of densification on mitigation of liquefaction hazards in loose saturated liquefiable foundation soils was studied by means of dynamic centrifuge model tests as discussed herein. The work was originally conducted as a part of the first author's Ph.D. thesis at Rensselaer Polytechnic Institute (RPI), Troy, NY (Adalier 1996) and was motivated by a VELACS test (Arulanandan and Scott 1993) that was proposed and conducted by Scott et al. (1993) at CalTech.

Compaction is the most popular soil remediation method against liquefaction. Compaction of a foundation soil can economically be done to a finite area only, leading to a case of "island" of improved ground surrounded by unimproved ground. The behavior of the densified sand surrounded by liquefied loose sand during and following earthquakes is of great importance in order to design the compacted area rationally and optimize both safety and economy. The importance of this topic is well explained in the state-of-the-art paper by Mitchell (1981) as: "In many cases the volume of soil densified by deep compaction lies within a potentially liquefiable deposit of much larger lateral extent. The question arises then concerning whether, if in an earthquake the surrounding soil liquefies, there will be the possibility of loss of stability in the densified zone. Conceivably, the development of high pore pressures in the liquefied zone could generate higher pore pressures in the densified zone with consequent loss of strength. To guard against this possibility it should be sufficient to extend the zone of soil improvement laterally outward from the foundation area a distance equal to the thickness of the layer being densified." On the same topic, Mitchell (1992) further stated that: "There is a need for analysis and design methods for the required magnitude of improvement and size of the treated area to insure its stability within a larger area of unstable or liquefied ground." The compacted region of liquefaction prone ground is covered by design standards only in the vertical direction (based on judging whether liquefaction will occur or not). However, no such standards or design procedures are available for the horizontal direction, i.e., what size of area should be compacted assuming both safety and economy (Taguchi et al. 1992).

In this paper, two dynamic centrifuge tests on saturated specimens of loose and dense sand columns prepared adjacent to each other in a rigid model container are discussed. The main objective of this study was to investigate the mechanics involved in the problem of dynamic-cyclic response of a densified zone surrounded by loose easily liquefiable ground.

PREVIOUS MODEL TESTING STUDIES

Iai et al. (1994) proposed a procedure to determine the size of densified area behind or/and in front of quay walls based on 1-g shake table test results. Taguchi et al. (1992) studied the dynamic behavior of the boundary between compacted and non-compacted ground by means of a series of 1-g shaking table tests. These tests involved sandy model ground which was compacted in the center and remained loose outside of the compacted area. Model ground was only 55 cm deep, 400 cm long (in the shaking direction), and 40 cm wide. A rigid wall container was used. The main conclusions of this study by Taguchi et al. (1992) can be summarized as: i) both dense and loose areas practically reached initial liquefaction and there was some indication of water migration from loose surrounding soil to the dense area, ii) compacted ground never resulted in sliding failure (due to liquefaction) into the non-improved ground, iii) dense ground settled much less than the surrounding loose ground due to shaking. Although these two initial studies provided some valuable insights on the problem, there are concerns regarding the confinement stresses in these 1-g tests (i.e., stresses in the soil model were small and not simulate a typical field case). It was thought that large-scale field or centrifuge testing of the problem might give more realistic results.

Another major study done on this topic was that of VELACS Model No.3 (Arulanandan and Scott 1993). The model was proposed by Scott et al. (1993), and consisted of a loose (D_r= 40%) and a dense sand columns (D_r= 70%) prepared adjacent to each other. The specimen was saturated with water. Nevada 120 sand was used for both loose and dense sand columns. Excited horizontally at the base with an earthquake like dynamic motion, at a centrifugal acceleration field of 50g, the soil model simulated a prototype of about 11 m thick. Tests were performed in laminar box at CalTech (Scott et al. 1993), UC Davis (Farrel and Kutter 1993), and RPI (Taboada and Dobry 1993). Test results from each institution are discussed by Hushmand et al. (1993). Observations of each institution from the conducted tests were essentially similar (i.e., similar trends in behavior) and can be summarized briefly as:

i) There was a great similarity in pore pressure and acceleration traces measured in the loose and dense sand columns at corresponding locations.

ii) In contrast to Taguchi et al.'s (1992) observations, the surface settlement measured in the dense sand column was more than the one in the loose column (at the mid-point of the dense and loose sand columns). On average, the dense sand settled about 25 cm while the loose sand settled only about 17.5 cm. This was attributed to the movement of dense sand towards loose sand as the two layers liquefied.

It is noteworthy that, during these VELACS tests no significant redistribution or migration of excess water from loose to dense sand was observed. While significant redistribution might have occurred, the high prototype permeability precluded this phenomenon from appearing as a distinct phase of response. Liu (1992) in centrifuge tests on foundations on liquefiable soils showed that redistribution of excess pore fluid from high excess pore pressure to low excess pore pressure regions is a function of permeability (and in this case, also a function of pore fluid viscosity). High viscosity pore fluid requires more time to seep through soil, and the rate of inflow depends on: 1) permeability of soil, and 2) excess pore pressure difference between two zones (i.e., dense and loose). Hence, correct simulation of soil permeability is of much relevance to this problem. Thus, the soil behavior observed during these VELACS Model No. 3 tests represents a prototype high permeability soil such as gravel or coarse sand (rather than of a fine sand).

The tests that will be discussed in the following paragraphs were undertaken to augment the above earlier tests and address the response of medium-fine sand. Rather than using water as pore fluid, a water-glycerin solution of 10 times water viscosity was used (to reduce the permeability approximately 9 times with respect to the case of water as pore fluid). Considering the fact that the tests were conducted at a 50g gravitational acceleration field, and in view of the scaling laws applicable to centrifuge experiments, a fine-to-medium sand was simulated (in terms of permeability). Reducing permeability enhances the process of excess pore pressure build-up and slows down the process of pore pressure dissipation and migration.

CENTRIFUGE TESTS

The RPI's 100-g ton Acutronic centrifuge (Elgamal et al., 1992) was used in this study. A rigid model container with inner dimensions of 0.597 m in length, 0.27 m in width, and 0.15 m in height was employed. Teflon-on-teflon arrangement on the long sidewalls was used to reduce side friction and arching. VELACS Nevada #120 fine sand was used as the model soil. Extensive data about the properties and cyclic response characteristics of this soil (under triaxial and simple shear conditions) was reported by Arulmoli et al. (1992). Figure 1 shows the side-view of the soil models with instrumentation. At 50g, the soil model simulated a prototype of 30 m in length, 13.5 m in width, and 5 m in height. In this testing program, two centrifuge tests were performed. In the first model, the loose sand layer was at a relative density (D_r) of 47%, and the dense sand layer at a D_r of 70%. In the second model, the loose layer D_r was 40%, and the dense layer D_r was 90%. All tests were performed at a 50g centrifugal acceleration field using a uniform harmonic input motion of 10 cycles, 0.19g amplitude (prototype), and 2 Hz frequency (prototype). For detailed descriptions of model construction, instrumentation, testing procedures, and soil properties the reader is referred to Adalier (1996).

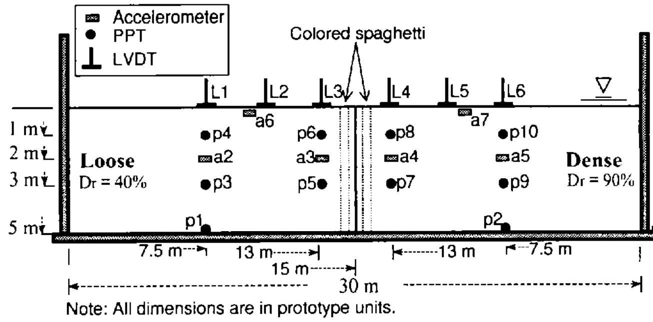

Fig. 1 Side-view of the models tested.

CENTRIFUGE TESTS RESULTS

All of the test results are presented and discussed in prototype units, unless stated otherwise. In Model 1, the difference in relative densities of two adjacent layers was of a moderate degree (i.e., 47% vs 70%). Accordingly, the effect of densification was less pronounced than that in Model 2 (i.e., D_r = 40% vs D_r = 90%). Moreover, Model 2 is believed to be a better representation of a typical field densification countermeasure case. In view of these two points and due to space limitation, emphasis will be given to Model 2 results as described in the following paragraphs. Extensive discussions on both tests results can be found in Adalier (1996).

Model 1 (Loose - D_r = 47%, Dense - D_r= 70%)

Despite the moderate contrast in D_r of two adjacent sand columns, their dynamic behavior was noticeably different (Adalier 1996). The decay of accelerations in the loose soil was much more rapid than in the dense soils. In the dense soil, a tendency for dilation was exhibited in the form of spiky acceleration response. In general, throughout shaking, the dense sand column behaved in a stiffer manner than the loose layer (acceleration response). The pore pressure traces measured in the loose and dense sand columns at corresponding locations showed great similarity (i.e., the rate of excess pore pressure (EPP) build-up was similar in both columns). However, the loose column reached initial liquefaction (i.e., r_u = 1.0, in which r_u is excess pore pressure ratio = EPP/σ_v; where σ_v is initial effective vertical stress), while the dense sand column measured r_u = 0.9 only. In contrast to the VELACS test results, in this test the dense sand EPP dissipated faster than EPP measured at corresponding locations in the loose sand. This might indicate that the dense soil presumably experienced less post-liquefaction volumetric consolidation strains. In addition, in contrast to the VELACS results, the loose soil settled more than the dense soil (10 cm vs 5 cm). Post-test observation of soft spaghetti noodle markers near the interface vaguely revealed some slumping of the dense soil into the loose soil. However, probably due to the involved small magnitudes (relative to the accuracy of the employed displacement detection technique which basically relied on visual observation and measurements by a ruler), no clear deformation pattern could be confirmed.

Model 2 (Loose - D_r = 40%, Dense - D_r = 90%)

Due to limitation of space, only selected-representative transducer measurements will be presented herein. For full sets of data the reader is referred to Adalier (1996).

Acceleration Response. Figure 2 shows the lateral accelerations measured in the loose and the dense sand columns. As seen, accelerations measured in loose and dense sand columns are very different. While the loose sand acceleration response showed severe reduction of base input accelerations due to liquefaction, the dense ground response showed some amplification of base input accelerations. The measured surface accelerations in the loose sand column (A6) virtually disappeared after about 1.5 cycles of input shaking due to liquefaction. Located at 2 m depth and away from the interface, A2 showed a drastic drop of accelerations after about 1.5 to 2 cycles of input motion. During the first cycle of input motion, A3 located at 2 m depth near the interface measured accelerations identical to A2. However, during the second and third cycles of input shaking, A3 measured a relatively large response (i.e., high amplitude dynamic response), similar to that of the denser sand, before the deamplification phase. In addition, after the third cycle A3 response was larger than that away from the interface (A2) in the loose sand column (i.e., there is less decrease of accelerations at the locations far from the interface). This might be due to the reinforcing effect of the adjacent dense sand column at points in the loose ground near the interface.

In the dense sand column, during the first 1.5 cycles of input motion, the measured accelerations were very similar in shape and magnitude to the input motion. After about two cycles of shaking, as the soil softened due to high EPP, the shape of measured accelerations in the soil becomes much different from that of the input. Acceleration data showed a dynamic response that is representative of dense sand in the form of no reduction in acceleration amplitudes due to liquefaction (on the contrary there was some amplification). This is due to the fact that, dilating soil even if it is liquefied has an ability to transmit vertically propagating dilational and shear waves (Adalier 1996). The dense sand acceleration records, especially A4 (near interface at a depth of 2 m), and to much lesser extent of A5 (away from interface at a depth of 2 m), showed a directional bias. In this regard, near the interface (A4), large acceleration spikes occurred exclusively in the negative direction, accompanied by low amplitude acceleration response in the positive direction. This observed bias in acceleration magnitudes at A4 was presumably due to the movement of the dense soil near the interface (i.e., near the loose sand column) towards the loose sand column during shaking.

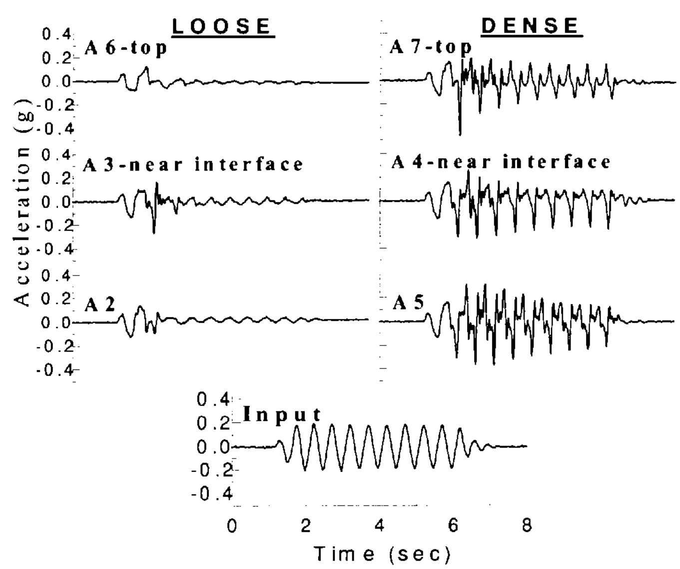

Fig. 2 Lateral accelerations measured in the loose and dense sand columns in Model 2 test.

Excess Pore Pressure (EPP) Response. Figure 3 shows a comparison of EPPs in the loose and dense sand columns at selected locations. As seen, the rate of EPP build-up is higher and the dissipation is slower in the loose sand compared to the dense sand at corresponding locations. The difference in the rate of EPP build-up between dense and loose sand is more pronounced away from the interface. At 3 m depth, near the interface (p5 and p7), EPP measured in the dense sand becomes equal to EPP measured in the loose (after three cycles of shaking); while at the same depth but away from the interface (p3 and p9), EPP in the dense sand becomes equal to EPP in the loose sand only sometime after the end of dynamic excitation. This suggests that at least part of the EPP in the dense soil might be due to migration of water from the adjacent loose ground. At the base away from the interface (p1 and p2), EPP in the loose soil was always higher than that measured in the dense sand. Hence, at every corresponding location other than at the base, EPP in the dense and loose sand eventually reached the same level. However, in terms of r_u, the case was different. Figure 4 shows the comparison of EPP traces in terms of r_u in the loose and dense sand columns. As seen, in the loose ground, the EPPs at every depth near the interface and far from the interface were equivalent to the initial effective overburden pressure, σ_v (i.e., initial liquefaction; r_u= 1.0). In the dense ground, r_u appears to be less than 1.0 throughout

Another interesting observation is that in the dense sand, EPP measured away from the interface (p2 and p9) showed "double cycling" effect (Adalier et al. 1998) suggesting that the soil at these locations was dilating in both directions (i.e., towards and away from the interface). At the locations near the interface (p7 and p8) no such "double cycling" of EPP was evident, presumably because at these locations the soil dilates mainly in one direction (towards the loose sand), as also shown by the acceleration response.

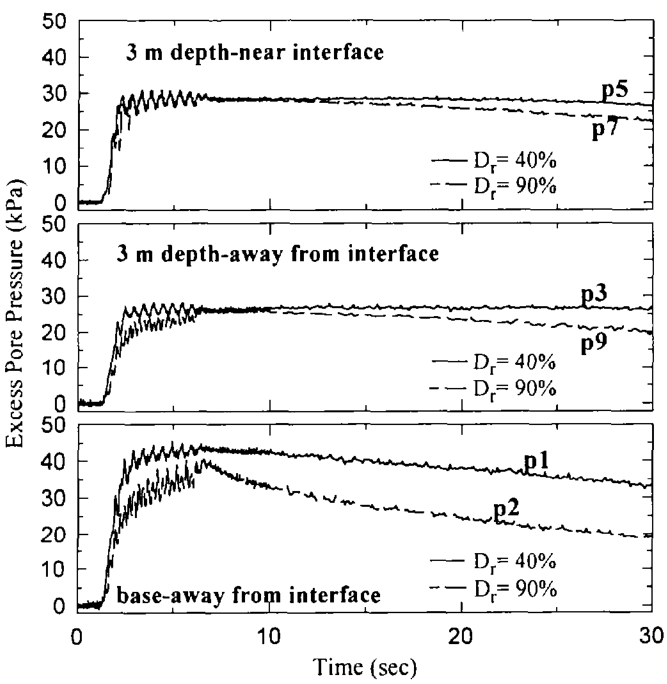

Fig. 3 Excess pore pressure time histories at selected positions for Model 2 test.

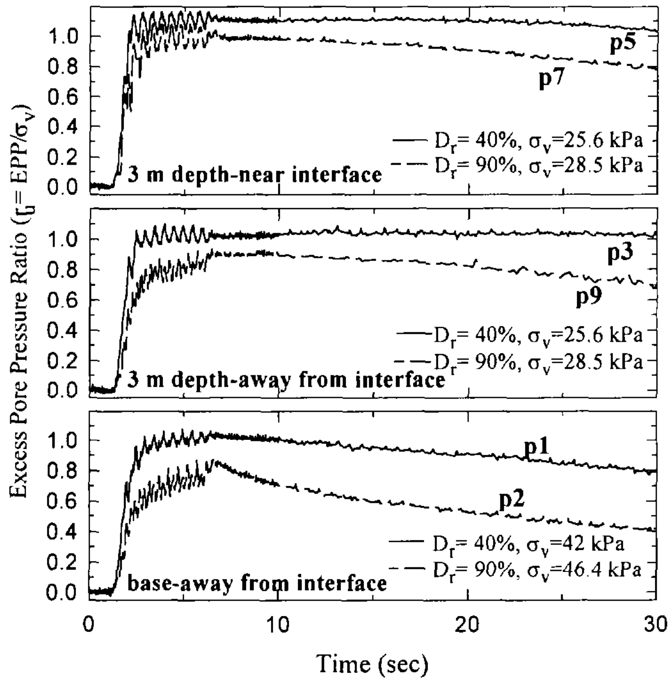

Fig. 4 Excess pore pressure ratio (r_u) time histories at selected positions for Model 2.

Figures 5 and 6 show the evolution of EPP at different distances from the interface at a depth of 3 and 4.85 meters respectively in the loose and dense sand layers during and after shaking. It is noted that similar trends were observed for depth of 1 m. As seen in Fig. 5a and Fig. 6a, the loose sand built-up EPP faster than the dense sand. EPP build-up rate in the dense sand increased toward the interface (Fig. 5a) indicating the weakening influence of the adjacent loose sand column. No such clear trend was observed for loose sand column. As seen, throughout shaking, EPP in the dense soil (especially at the region away from the interface) was lower than that in the loose soil. However, considerable pore pressure increase immediately after shaking (compare EPP at the end of shaking-6.2 sec to 7.2 sec) was observed in p2 and p9 (away from interface), suggesting that this increase of EPP was not due to cyclic shearing but due to migration of EPP from the adjacent loose sand. Presumably, due to difference in dynamically generated EPP in the dense and loose layers (higher in the loose layer) a hydraulic gradient pointing towards the dense layer was established during shaking which led to migration of excess pore fluid toward the dense sand column from loose sand column. Actually, in general, EPP at any point may not be just a consequence of undrained cyclic strains on that soil element but a combination of this and pore fluid migration to, or from, the surrounding soil.

As seen in Figs. 5 and 6, dissipation of EPP started first in the dense layer and progressed faster than in the loose layer. Consolidation of once liquefied soil is slowest in the loose sand at locations away from the interface, and fastest in dense sand at locations away from the interface (see EPP at 20 sec, 60 sec, and 100 sec). This is very reasonable since the loose sand away from the interface most probably experienced the largest liquefaction induced volumetric strains (thus, it would take more time to settle down). In the loose sand, regions near the interface probably experienced less volumetric strain than the remote regions due to the reinforcing effect of the adjacent dense sand column. In the dense sand column, the interface region (i.e., near the loose, less stable sand column) most probably experienced more volumetric strains than the regions away from the interface due to the weakening effect of the adjacent loose ground, and therefore might have needed more time to settle down. These speculations are supported by the LVDT data, as will be discussed in the following paragraphs. However, it is recognized that, other considerations may have also been a factor in this complicated response phenomenon.

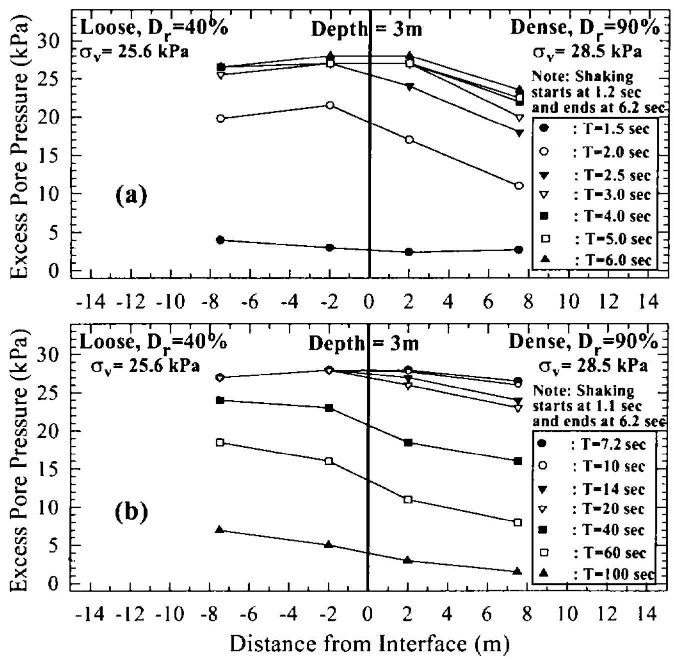

Fig. 5 Excess pore pressure isochrones at depth of 3 m in Model 2 test (a) during shaking; (b) after shaking.

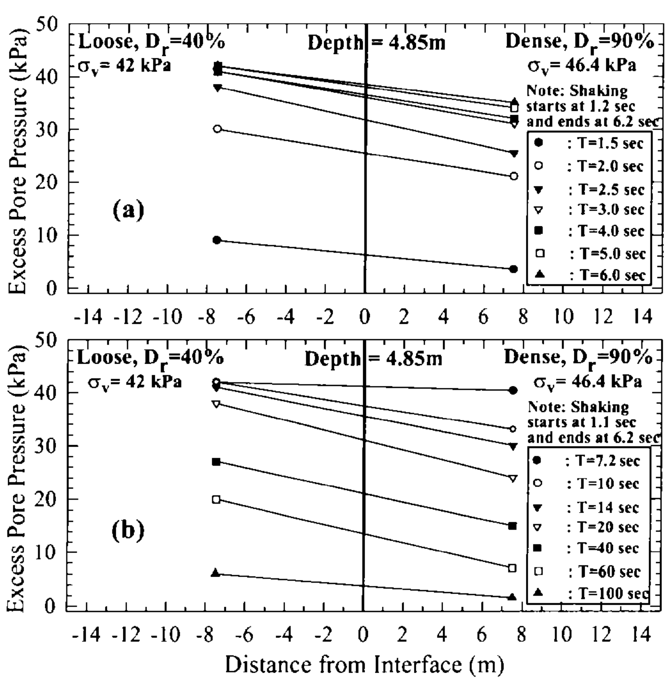

Fig. 6 Excess pore pressure isochrones at depth of 4.85 m (base) in Model 2 test (a) during shaking; (b) after shaking.

Deformations. Figure 7 shows the final surface settlements after dissipation of EPP, versus distance from the loose and dense sand column interface. As seen, at every corresponding location, the loose sand settled more than the dense sand. The majority of settlements occurred during shaking, in both layers. It is noted that observed settlements may be due to: 1) compaction settlement, 2) settlement caused by lateral spread of soil layer, 3) both 1) and 2). As seen in Fig. 7, surface settlements in the dense sand were almost identical at 3.75 m and 7.5 m away from the interface. However, settlement increased significantly at 2 m away from the interface. Similarly, in the loose sand at locations 3.75 m and 7.5 m away from the interface, the surface settlements were almost identical. However, at 2 m away from interface these settlements decreased. Hence, in the dense sand, surface settlements increased near the interface due to the less stable adjacent loose ground, whereas in the loose sand column it was just the opposite (i.e., settlements were less near the interface) due to more stable adjacent dense sand column. It appears that the zone of interaction between loose and dense sand ended somewhere between 2 m to 3.75 m away from the interface (since settlements at 3.75 m and 7.5 m were almost identical both in loose and dense sand columns). This data basically shows that part of the dense soil near the interface moved towards the loose sand, as was also suggested by the acceleration data (Fig. 2). Actually, post-test inspection of the installed soft spaghetti noodles during dissection of the model also confirmed this finding. Displacement of spaghetti noodles showed that the upper 2 to 2.5 meters of the dense sand moved towards the loose sand by about 0.1 to 0.2 m, as the two layers liquefied.

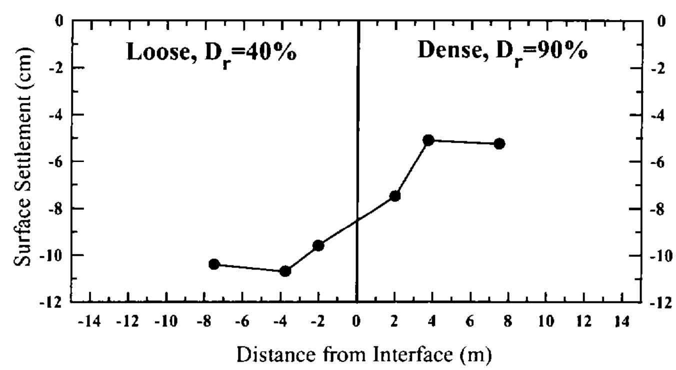

Fig. 7 Soil surface settlement profile in Model 2 test.

DISCUSSIONS AND CONCLUSIONS

In a soil configuration as in these tests, there is a difference in the coefficient of earth pressure at rest (K_0 of the dense ground is greater than that of the loose ground). That is to say, the dense sand column exerts a larger lateral pressure than the loose ground. However, during shaking, the lateral effective pressure and shear resistance in the loose soil column might be lost due to liquefaction, while some effective stress may remain (assuming no liquefaction) in the dense sand column. This would lead to an imbalance of forces near the interface and the dense soil would eventually move towards the loose soil (as also mentioned by Taguchi et al. 1992). This has been inferred from the recorded accelerations, and observed in the LVDT surface settlements, and spaghetti noodles lateral deformation data (especially during the Model 2 test).

Loose sand has a higher liquefaction potential than dense sand. Therefore, during shaking, the loose sand builds-up higher EPP due to local cyclic shearing (than the adjacent dense sand). When this difference in EPP in the adjacent sand columns becomes significant, fluid migration from the loose to the dense sand can occur. Consequently, another potential danger arises from the loss of strength in the dense layer as a result of pore pressure increase (due to seepage flow or migration of pore pressure from the adjacent loose liquefied layer). Hence, it is likely that the liquefied loose soil causes the dense soil at the boundary to liquefy due to: 1) reduction of effective confining stresses as the loose sand liquefies, 2) seepage flow from the loose sand to the dense sand.

The main conclusions of this study can be summarized as:

- The loose sand settled more than the dense sand column. In the dense sand column surface settlements increased towards the interface, whereas in the loose sand surface settlements decreased towards the interface.
- Acceleration, and LVDT data, and post-test observations of the installed soft spaghetti noodles showed that the dense sand moved towards the loose sand as the two sand columns built-up high excess pore pressures.
- EPP built-up faster and then dissipated slower in the loose sand column compared to the dense sand column.
- The test results suggested that there are two major concerns regarding "Islands" of densified soil: 1) there is a potential strength loss in the densified zone as a result of pore pressure increases due to migration of pore water into the island from the adjacent (loose) liquefied ground; 2) there is a possibility of sliding of the island (the upper portion) as the surrounding liquefied soil flows. The first concern can be addressed by placing an impermeable barrier between the loose and dense soil columns. The second concern can be overcome by extending the densified zone far enough away from a supported super-structure.
- A design procedure for an optimum area of compaction to minimize liquefaction effects is yet to be developed. One way of achieving this would be by performing a series of dynamic centrifuge experiments with different sizes of compacted areas, establishing relationships between the area of compaction and the residual deformation in the compacted area or any part thereof.

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