

DYNAMIC BEHAVIOR OF EMBANKMENT DAM ON LIQUEFIABLE FOUNDATION SUBJECT TO MODERATE EARTHQUAKE LOADING

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SUMMARY

Liquefaction-induced ground displacements resulting from earthquake shaking are a major threat to the stability of earth dams comprising of, or underlain by, loose saturated granular soils. Many liquefaction induced earth dam failures or near-failures have been reported around the world during various earthquakes. Such embankment damages were particularly destructive when the underlying saturated granular soils liquefied, resulting in cracking, settlement, lateral spreading, and slumping of the embankment.

In a series of twelve separate geotechnical centrifuge physical modeling tests, seismic behavior of a zoned earth dam with saturated sandy soil foundation was studied under moderate earthquake conditions. Soil response during and after shaking was monitored by many miniature accelerometers, pore pressure sensors, and displacement gauges placed throughout the soil model. The effect on the seismic behavior of the dam of different parameters such as the thickness, location, and depth of liquefiable layer is studied. This paper describes some of these tests and briefly presents the preliminary results. Valuable insights into the dynamic behavior of the employed embankment-foundation systems are provided. Currently, such testing results offer a valuable alternative to actual full-scale dynamic response, which is virtually non-existent.

INTRODUCTION

Liquefaction-induced ground displacements resulting from earthquake shaking are a major cause of damage to earth structures comprising of, or underlain by, loose saturated granular soils. Many liquefaction induced failures or near-failures of structures such as river dykes, highway embankments and earth dams have been reported around the world during various earthquakes (Seed [1]; Japanese Geotechnical Society [2]; Adalier [3]). Such embankment damages were particularly destructive when the foundation soils liquefied (Duke [4]; Yokomura [5]; McCulloch [6]; Seed [7]; Tani [8]; Krinitzsky [9]) resulting in cracking, settlement, lateral spreading, and slumping of the embankment.

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It is estimated that there are more than 75,000 dams in the U.S., many of them earth dams. The U.S. Army Corps of Engineers (COE) alone is responsible for the care and maintenance of over 700 medium or large size embankment dams in the U.S. Many of these, over 200, are in highly seismic areas of the country. The vast majority was constructed in the 1940's and 1950's when earthquake engineering was in its infancy and seismic hazards were neither recognized nor understood, leaving some inadequate for a seismic event. It is estimated that there are many other such seismically inadequate dams around U.S. under the responsibility of federal, state and local governments, as well as the private sector.

Earth dams founded on liquefiable soil deposits pose a particularly difficult problem. Such cases if found vulnerable to earthquake shaking often necessitate the development of appropriate remediation countermeasures, which can be very challenging both economically and technically. Hence, it is not feasible to remediate all such structures, even technically doable, due to the considerable costs involved. A determination of how much damage such structures could tolerate and still be able to perform their primary function could alleviate the need to remediate many dams. In cases that require remediation, understanding the deformations and dynamic response mechanisms of such dam/foundation systems would enhance our ability to design remedial procedures in a more effective and economical way. The concern over earthquake resistance of older embankment dams drives the need for remedial construction costing hundreds of millions of dollars of public funds annually. The potential for economic savings through better understanding of the involved mechanisms can be enormous considering the high cost of remedial treatment and the volume of dams that may require treatments in the coming years. Foundation remedial projects of Sardis Dam in Northern Mississippi and Mormon Island Dam north of Sacramento, California both owned by the COE are only two recent cases, each of them costing over \$30 million. Casitas Dam, Ventura-California owned by U.S. Bureau of Reclamation is another earth dam that is currently receiving a remedial treatment against foundation liquefaction with a cost of over \$20 million. A similar \$15 million upgrade is under way at the Bureaus' Bradbury Dam in the Santa Barbara-California area. Many other dams are currently under evaluation for liquefaction-induced hazards mainly by several federal and state governmental agencies.

Technical literature indicates that the centrifuge dynamic model testing technique can play a major role in understanding the dynamic behavior of various earth structure-soil systems including earth dams (Arulanandan [10]; Kimura [11]; Adalier [12]). Centrifuge testing is the best, most practical, most economical, and the only method for properly investigating and verifying earthquake induced equivalentprototype behavior in soil, which behaves non-linearly and is stress-state dependent. Additionally, this centrifuge experimental model response also provides a basis for calibration of design and computational modeling procedures currently being performed by the Corps of Engineers, Engineers Research and Development Center (ERDC). The Corps is currently in the process of updating their guidance on the dynamic evaluation of earth and rockfill dams (COE [13]). A key aspect of the new guidance will be a required validation of any numerical analyses results that are used for remediation of an existing structure. This new guidance is offering scaled physical modeling as the most attractive method of numerical model verification. Prior to release of the new guidance, the Corps of Engineers has been conducting research into the behavior of earth dams on liquefiable foundations with the intention of establishing a database to be used for numerical model verification. Several series of centrifuge physical model tests have been completed as part of this research program. This paper will discuss one of these test series on zoned earth dams.

In a series of twelve separate geotechnical centrifuge physical model tests, seismic behavior of a zoned earth dam with saturated sandy soil foundation was studied under moderate earthquake conditions. The effect on the seismic behavior of the dam of different parameters such as the thickness, width, and depth of liquefiable layer is studied. Currently, such testing results offer a valuable alternative to actual full-scale dynamic response, which is virtually non-existent. Soil response during and after shaking was

monitored by many miniature accelerometers (in the horizontal direction), pore pressure transducers, Linear Variable Differential Transformers (LVDTs), and vertical and horizontal displacement gages, placed throughout the soil model.

This paper describes some of these tests and briefly presents the preliminary results. Valuable insights into the dynamic behavior of the employed embankment-foundation systems are provided. In these test, particular attention was given to accurate and detailed measurement of the induced deformations. In addition to the pore pressure, acceleration, and model surface displacement measurements that are commonly done in most centrifuge tests, very detailed and extensive deformation mapping was performed based on pre- and post-shake meshes of markers throughout the body of the models. Such earthquakeinduced deformations and damage is the key to making well-informed seismic safety and remediation decisions for embankment dams. Practice has demonstrated that remediation measures based on displacement criteria are much more cost-effective than those based on the factor of safety approach (Finn [14]). This study is believed to significantly expand and enhance our earthquake case history database regarding the earth dam on liquefiable foundation problem with more completely known and defined conditions and earthquake responses. Such an earthquake response database will provide: (1) a basis for modification and improvement of current methodology and assumptions, (2) realistic data for validation and improvement of numerical procedures (a current ongoing research), and (3) definitions of the physical processes and mechanisms involved in the liquefaction process and resultant effects on soil-embankment behavior. It will also significantly enhance our capability to determine and define liquefaction occurrence, potential to cause damage, and need for mitigation, thereby resulting in economic savings to U.S. Corps of Engineers.

CENTRIFUGE MODELING EQUIPMENT

Geotechnical Centrifuge

The tests were done using the Rensselaer Polytechnic Institutes' (Troy, NY, USA) 100g-ton, 3-m arm radius geotechnical centrifuge (http://www.rpi.edu/~dobryr/centrifuge/).

The main principle in centrifuge modeling is that a 1/N scale model subject to a gravitational acceleration of Ng (g is acceleration of gravity) will feel the same stress as the prototype. Then, stress-strain relationships at all equivalent points in the model and prototype will be the same if the same soil is employed and the behavior of the model will mimic the behavior of the prototype. Consequently, with the help of scaling laws (Schofield [15]) measurements in centrifuge tests under closely controlled conditions can be related directly to an equivalent full-scale prototype. The centrifuge modeling technique allows soil liquefaction tests to be performed at a conveniently reduced scale, and provides data applicable to full-scale problems.

In-Flight Shaker

An in-flight shaker was employed to impart the model base shaking. Designed and built at Rensselaer, the centrifuge shaker is an electro-hydraulic, servo-controlled device with dual actuators (Van Laak [16]). Total maximum force capability of the simulator is 80-kN. Input motion is imparted (in-flight) in a direction parallel to the centrifuge axis of the Rensselaer shaker.

Instrumentation

A PC - based data acquisition system with the capability of digitizing 64 channels of data at a 15 kHz sampling rate (per channel), or a smaller number of channels at correspondingly higher sampling rate was employed. Transducer excitation and signal conditioning is performed on the centrifuge arm and the signals are then transmitted through slip rings to the recording system, located in the control room. These analog signals are amplified, filtered to avoid aliasing, and finally digitized in real time using array of PC-

type digitizing boards. During flight, three closed-circuit television cameras monitor the centrifuge, the soil model plan, and the soil model side through a window in the model container's wall.

For each test, during and after shaking, a complete set of response time histories was recorded in the form of accelerations at 12 locations, pore pressures at 13 locations and vertical displacements (LVDTs) at five surface locations. In addition, the models were dissected after testing and the deformed configurations were carefully mapped with the aid of soft vertical marker lines (embedded in the embankment and foundation layer before testing) and horizontal lines of colored sand (placed within the soil during model construction). Detailed information about the geotechnical centrifuge modeling instrumentation and sensors used in Rensselaer is given by Adalier [3].

Model Container

A rigid wall model container with inner dimensions of 0.88 m length, 0.37 m width, and 0.36 m height was used. A transparent side window allowed for observation of the side of the model. Rough sand paper was glued to the inner base surface of the container to provide good interface friction between the box and the soil base.

MODEL CONSTRUCTION AND TESTING PROCEDURE

Model Soil

Nevada No. 120 sand with the properties listed in Table 1 was used in all tests. This is fine grained, uniform, sub-round, clean sand. Extensive data about the monotonic and cyclic response characteristics of this soil has been documented by Arulmoli [17] as part of VELACS project and also can be found on the WEB at http://geoinfo.usc.edu/gees/velacs/ and http://rccg03.usc.edu/velacs/Centrifuge/cntdata.

The embankment core was made out of kaolin clay compacted at around 33% water content (3% to wet side of optimum). It had a dry unit weight of 13.4 kN/m³ and unconfined shear strength (Su) of 18-20 kPa.

Property	Value
d ₁₀	0.08 mm
d ₅₀	0.15 mm
Specific gravity, G _s	2.67
Maximum void ratio, e _{max}	0.887
Minimum void ratio, e _{min}	0.511
Maximum dry unit weight	17.33 kN/m ³
Minimum dry unit weight	13.87 kN/m ³
Water Permeability at e = 0.736	6.6 x 10⁻⁵ m/s
Water Permeability at e = 0.661	5.6 x 10 ⁻⁵ m/s
Internal friction angle at D _r = 40%	33 ⁰
Internal friction angle at $D_r = 60\%$	36 ⁰

Table 1. Properties of Nevada No. 120 sand (after Arulmoli [17]).

Model Construction

Oven dried Nevada sand was pluviated through a V-shaped funnel with a row of holes along the funnel tip. The sand particles drop height was adjusted to give different relative densities (e.g., 35% and 70%). Pluviation was interrupted periodically to place instrumentation in the soil model. A very thin band (about 2 cm wide in model scale) of colored Nevada sand was placed at the interface of each horizontal foundation layer (3 cm apart in model scale). Pluviation was stopped 3 to 5 mm above the desired model

height and the model surface was then trimmed to the final shape by removing excess sand particles with a vacuum line. Thin spaghetti sticks were then inserted vertically (driven in a steel tube casing), at predetermined positions. When softened by the pore fluid these sticks acted as inclinometers and made it possible to measure the internal deformations during model dissection.

The dry model was carefully carried to the centrifuge with a forklift and placed on the shaker platform. Surface measurements were taken before and after this stage to make sure that no densification had been induced in the model during this transportation and installation process. Following the conventional saturation procedure (Adalier [3]), a lid was used to seal the model box, and air inside the box was evacuated under vacuum pressure of 100 kPa applied for an hour. The vacuum was then shut off and carbon dioxide was introduced into the box to help dissolve possible remaining oxygen in the soil voids. After half an hour, vacuum of 100 kPa was applied for one hour. This negative pressure was reduced to 90 kPa and de-ionized/de-aired water/metulose solution of 50 times water viscosity was introduced very slowly to the model surface at both downstream and upstream sides of foundation layer. This highly viscous fluid slows down the rate of diffusion and dissipation and unifies time scaling factors for dynamic and consolidation events, hence gives a better simulation of the field case. Considering the fact that the tests were conducted at a 100g gravitational acceleration field, and in view of the scaling laws applicable to centrifuge experiments, about a two times more permeable foundation sand (relative to 1-g water permeability) was simulated (still a fine sand permeability). This saturation process was performed slowly over an approximate 24 to 36 hour period, to ensure good saturation and prevent local liquefaction or piping in the soil. Foundation layers on both sides were saturated with this viscous fluid to about 0.5 m (prototype) above the ground surface. Above this level, the upstream side of the embankment was saturated with de-ionized and de-aired water, rather than a viscous fluid to simulate a coarser embankment material (as mainly found in field cases). It should be stressed that the saturation was done while the model container rested on the centrifuge swing platform to avoid any possible disturbance to the models during any transportation process. This assured high quality uniform models. After saturation, the model was opened to atmosphere, and the model geometry was measured. Vacuum suction or saturation process was found to cause no detectable settlements or deformations in any of the soil models. The LVDTs to measure vertical displacements along the model surface at various locations were placed. Figure 1 shows a typical test setup on the centrifuge platform.



Fig. 1: An appearance from a typical test setup on the centrifuge platform.

Testing Procedure

The tests were conducted in a 100g gravitational field. The centrifuge was brought to 100g very gradually in about 30 minutes in order to ensure a smooth transition of the soil model from 1g to 100g. An additional 10 minutes of spinning time was allowed at 100g before imparting the dynamic base excitation. This way the pore pressures were allowed to buildup almost statically without causing any potential instability problems. Self-weight compression of the model was monitored in each test and found to be insignificant compared to those dynamically-induced during the subsequent shaking events. All of the models were subjected to the same sinusoidal base horizontal acceleration (Fig. 2) of 30-cycles, 0.2g magnitude, and 1.5 Hz dominant frequency (prototype). In all tests, the vertical base motion was minimal (less than 10% of the horizontal input). Soil response was measured by accelerometers, LVDT's, pore pressure transducers (PPT), and deformation markers (i.e., colored sand and spaghetti noodles). During all tests, a model sampling rate of 4000 samples/second/channel with a filter cutoff frequency of 1200 Hz was used.



Fig. 2: Recorded input acceleration (g), centrifuge models.

After the test was complete, the specimen was dissected carefully to measure final locations of the spaghetti noodles, colored sand markers, accelerometers, and pore pressure transducers. Digital photographs were taken before and after each test, and a detailed visual observation was conducted of the deformation of each model. The profiles of the models at dissected sections were mapped very carefully (transformed on a thin plexiglass plate) and then digitized into computer graphics. By this method, very detailed pre- and post-test mid-section profile meshes of each model was obtained.

TESTING PROGRAM

A total of twelve dynamic tests were performed on twelve different soil models. At a 100g gravitational acceleration field, the models depicted in Fig. 3 (a setup of a typical model) simulated a prototype earth dam of 10 m in height, 39.5 m in width, sitting on 9 m thick fine sand foundation deposit. Table 2 gives the summary of the conducted centrifuge tests. As seen in Table 2, mainly the effects of liquefiable layer I) thickness, II) depth, and III) location on the dynamic performance of the dam-foundation system were studied. In Series IV, the effects of longer earthquake shaking (40 cycles) and of a clay interlayer at 2.5-3.5 m depth on both downstream and upstream sides of the dam was evaluated. Full sets of horizontal accelerations, pore pressures, and deformations at different locations throughout the foundation-dam model were obtained for further analysis and interpretation. Additionally, these data are being studied numerically and used as a database for calibration and verification of several different finite element and finite difference codes or numerical schemes.

Due to space limitation, only selected response records of Series I tests will be presented in the following sections and preliminary results will be briefly discussed. Discussions on the Series II tests are provided by Sharp [18]. In the following paragraphs, all of the test results are presented and discussed in prototype units, unless otherwise stated.



Fig. 3: Schematic of a typical dam-foundation model.

Centrifuge Test Series	Model Configuration 1DS-2DS-3DS—1US-2US-3US D: 70% D _r ; L: 35% D _r
Series I-	L-L-L-L-L
Effect of liquefiable	L-L-D—L-L-D
layer <i>thickness</i>	L-D-D—L-D-D
-	D-D-D-D-D
Series II-	
Effect of liquefiable	D-D-L—D-D-L
layer depth	D-L-DD-L-D
	L-D-D—L-D-D
Series III-	D-D-L—D-D-D
Effect of liquefiable	D-L-D-D-D-D
layer <i>location</i>	D-D-D-D-L
-	D-D-D—D-L-D
Series IV-	
Clay Interlayer and	L-L-Clay-D—L-L-Clay-D
Large Earthquake	D-L-D-L-D
D _r : Relative Density; DS: Downstream; US: Upstream	

 Table 2: Centrifuge testing program.

SERIES-I TESTS RESULTS - EFFECT OF LIQUEFIABLE LAYER THICKNESS

Model 1: LLL-LLL: Entire Foundation at $D_r = 35\%$

Figure 4 depicts model response at selected transducer locations and post-test deformed shape of the model cross-section (after mid-section dissection). As seen, the induced base excitation caused very large deformations both in embankment and in the foundation. An excessive embankment crest settlement of 2.4 m was measured. Essentially, most of this settlement took place during the course of the base excitation, at a nearly uniform rate with time. Both sides of the embankment slumped and moved laterally

away from the centerline on the excess pore pressure softened foundation. The migration (mostly lateral) of underlying foundation soil towards the free field, as indicated by the post-test deformed mesh (Fig. 4), was largely responsible for the observed embankment slump. Movements as large as 4 m were observed near the toe areas. In general, the deformations on the downstream side were slightly bigger than those of the upstream side. This is mainly attributed to the higher initial static shear stresses in the downstream foundation. Deep failure planes in the foundation zone are obvious. The lateral deformation in the foundation soil, both at upstream and downstream sides, was found to attain its maximum near ground surface, and to decrease with depth. This deformation may be associated with an average accumulated normal lateral tensile strain of about 20% along the embankment base. Indeed, this tensile strain was clearly manifested in the form of stretching and slumping of the embankment body.



Fig. 4: Selected transducer data and post-shake deformed model (mid-cross-section)-Model 1.

At the upstream toe-foundation, the acceleration response (i.e., a3) gradually decreased within 3-4 cycles of base excitation, reflecting the associated loss of soil stiffness and strength due to induced high excess pore pressures. However, after a few initial cycles, a very peculiar behavior of large asymmetric acceleration spikes started to appear. Such asymmetric spiky acceleration response has been thoroughly investigated before (Dobry [19]; Elgamal [20]) and has been attributed to the occurrence of significant post-liquefaction cyclic down-slope deformations and the dilation of soil as the stress state reaches the phase transformation line. Below the downstream toe, a considerably stiffer response prevailed, reflected as stronger acceleration records. At this location it took about 15 cycles of excitation for the dilative acceleration spikes to appear. This is related to the distance of the initial stress state to the phase transformation line (i.e., the initial stress state of upstream toe foundation area is closer to the phase transformation line than that of downstream). Likewise, a faster buildup of excess pore water pressures at the upstream side helped bring the stress state to phase transformation quicker. Notice that the direction of these spikes are in opposite directions in upstream (a3) and downstream (a10) sides, as the lateral movement, and the initial static lateral shear stresses, are actually in opposite directions. It is noteworthy that the core/crest accelerations were significantly attenuated relative to the base input. As will be discussed later, the crest motions were largely affected by the embankment and foundation sandy soil state of strength during shaking event.

At P3 and P12, the soil built up excess pore pressures corresponding to initial liquefaction. The buildup at the upstream side was somewhat faster. Owing to very large shear strains induced in the foundation

and associated dilation effect the foundation excess pore pressures (P6) did not reach initial liquefaction values. The estimated excess peak pore pressure ratio at P6 is around 0.7.

Model 2: DLL-DLL: Top 3 m Densified Foundation

In this model the top 3 m of the entire foundation layer was densified to $D_r = 70\%$. In practice, various ground treatment strategies can be used to mitigate the liquefaction hazards. The most common improvement techniques have been densification, soil replacement, or cementation. Among these, compaction or densification has been the most popular method for embankment foundation remediation projects (Adalier [12]). This model may be considered simulating a case where top 3 m of the foundation soil densified as a countermeasure or this kind of layering exist in nature.

Figure 5 depicts model response at selected transducer locations and post-test deformed shape of the model cross-section (after mid-section dissection). As seen, both the embankment and the foundation deformations were much reduced when compared with the previous model (all foundation layer is loose, i.e., LLL-LLL). Embankment crest settlement was 1.3 m (i.e., about half of that of Model 1). As in the first model, essentially all of the measured crest settlement took place during input excitation. The pattern of deformations were shifted downwards to the 6-m loose layer. From the deformed mesh (Fig. 5) one may deduce that the 3-m top dense soil plus the embankment basically translated sideways (without internally experiencing much shear deformations) on the excess pore pressure softened 6-m loose underlying layer. As in the first model, the deformations on the downstream side were slightly larger than those on the upstream side. The average accumulated normal lateral tensile strain along the embankment base was about 6.5% (compared to 20% in the Model 1).



Fig. 5: Selected transducer data and post-shake deformed model (mid-cross-section)-Model 2.

The accelerations at foundation mid-depth were somewhat attenuated relative to base input. Also the asymmetric spiky response phenomenon was much less significant (compared to Model 1 case) as the lateral deformations were reduced. Clay core crest accelerations, while attenuated relative to the base input motion, were higher than those of Model 1. This is attributed to overall stiffer and stronger foundation material surrounding the core. Both P3 and P12 at the mid-depth (in loose layer) measured excess pore pressures corresponding to initial liquefaction. P6 under the embankment also measured excess pore pressure values corresponding to initial liquefaction. Contrary to the Model 1 case, reduced

overall lateral spreading and dilation effects helped the soil to buildup higher excess pore pressures (Adalier [21]).

Model 3: DDL-DDL: Top 6 m Densified Foundation

In this model the top 6 m of the entire foundation layer had a D_r of 70%. Figure 6 depicts the Model 3 response at selected transducer locations and post-test deformed shape of the model cross-section (after mid-section dissection). As seen, embankment crest settlement was a little less than the one measured in Model 2 at about 0.8 m. Likewise the embankment and foundation internal deformations were further reduced by the increase in densified layer thickness from 3 to 6 meters. The maximum shear deformations were observed in the base 3-m loose layer. As in the previous two cases, the downstream deformations were somewhat larger. The average accumulated normal lateral tensile strain along the embankment base was about 4.5% (compared to 20% in Model 1).

The acceleration response of a3 and a10 were significantly stronger than those observed in Model 1 and Model 2, as these transducers were in dense soil. Asymmetric acceleration behavior was less significant. Clay core crest accelerations were significantly larger than the ones measured during Model 1 and 2 tests. Despite the fact that P3 and P12 were located in the dense zone, they measured excess pore pressures corresponding to initial liquefaction. Their position being close to the underlying loose layer probably helped the pore pressure buildup at these zones.



Fig. 6: Selected transducer data and post-shake deformed model (mid-cross-section)-Model 3.

Model 4: DDD-DDD: All Foundation Densified

In this model, the entire 9 m thick foundation layer had a D_r of 70%. Figure 7 depicts the model response at selected transducer locations and post-test deformed shape of the model cross-section. It is noted that, in this case, a calibration error resulted in an increase of about 20% in the input shaking accelerations. Nevertheless, the embankment and foundation deformations were reduced drastically. The pattern of internal deformations were somewhat different than those observed in Model 2 and 3 tests, as they were more uniformly distributed through the model height in this case. The crest settled about 0.8 m uniformly during shaking. The average accumulated normal lateral tensile strain along the embankment base was about 4.0%. The foundation deformations were primarily lateral with minimal vertical component, as the dense soil did not contract in volume. The excess pore pressure buildup was overall somewhat slower; however the dissipations were significantly faster than those observed in the other model tests. Significantly faster post-shake excess pore pressure dissipations occurred, related to the smaller bulk modulus of the dense material. Despite the high excess pore pressures, due to dilative characteristics of the initially dense soil, stiff response prevailed throughout the foundation (as suggested by strong accelerations). It should be noted that dense sand, even liquefied, does not deform excessively like loose sand because of its high residual shear strength and dilative behavior arresting large strain increments. Dilative acceleration spikes were apparent at records a3 and a10, helped by the higher base input motion and higher foundation soil stiffness. The relatively high overall foundation sandy soil stiffness increased the effective confinement effect on the clay core resulting in relatively strong core crest accelerations. It is interesting to note that, the densified foundation much reduced the earthquake induced deformations but at the same time increased the embankment accelerations (due to the ability of dense sand to be able to transmit shear stress due to dilative behavior). In this respect, for the dams with sensitive crest structures one may consider Model 3 case as a viable option.



Fig. 7: Selected transducer data and post-shake deformed model (mid-cross-section)-Model 4.

Overall Response

The most important factor affecting the performance of an earth dam subjected to an earthquake is the movement of the dam and the ground supporting it during and after the event. As such Fig. 8 depicts the normalized (relative to Model 1) crest settlements, average dam base tensile strains, maximum foundation settlements, and normalized (relative to base input motion) crest acceleration arias intensities for the four model tests. Densifications to all depths are all found to reduce the embankment and foundation deformations by a range of 50% to 90%. However, the gain in deformation parameters has diminished after 6 m deep densification. The biggest gain was achieved form 3 m densification. Deeper densifications (i.e., 6 m and 9 m) further reduced the deformations though the improvement was at a diminishing rate. However, stronger foundations (denser material) have resulted in stronger embankment accelerations. The variation of dam settlement and acceleration (for the cases that the crest accelerations may be important, e.g., dams with various superstructures or auxiliary systems) with densification depth (Fig. 8) suggest it may be necessary to optimize the treatment depth to reduce the dam settlement and lateral spread to an acceptable level while at the same time ensuring that the dam accelerations are tolerable.



Fig. 8: Some dynamic performance parameters for the Models 1-4.

SUMMARY AND CONCLUSIONS

A model of an earth dam with clay core founded on a liquefiable sandy deposit subjected to a moderate earthquake loading has been studied. A series of highly instrumented dynamic centrifuge tests were performed to investigate the effects of loose foundation layer I) thickness, II) location, and III) depth on the dynamic behavior of the dam. Results of some of these tests were briefly presented herein. Centrifuge testing has proven itself as a very valuable tool to economically studying such complex geotechnical problems. The study provided many valuable insights into the dynamic behavior of earth dams sitting on alluvial soils subjected to moderate earthquake shaking.

The results of the specific group (or sub-group) of tests discussed herein suggest that there may be an optimum depth of densification treatment beneath an earth dam beyond which the reduction of the earthquake-induced deformations is relatively minor. The tests results also indicate that relatively small and isolated zones (e.g., at depth) of loose material within a dense soil volume may not impair the overall effectiveness of treatment and do not necessarily result in damaging displacements. This suggests that the remedial designs should be based on displacement criteria rather than on the factor of safety against liquefaction. It is the author's opinion that the reported tests offer an excellent database for validating and verifying current numerical models. The tests have captured many of the complexities that exist in the field to challenge a numerical predictive algorithm.

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