Current USACE Research Efforts Related to Seismic Stability of Dams

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ABSTRACT

Earthquake induced liquefaction continues to be a major threat to many engineered structures around the world. This type of damage is particularly problematic when performing an evaluation of a dam/foundation system where potentially liquefiable materials exist in either the dam or the foundation. Predominantly, analyses for such systems are performed utilizing some type of finite element or finite difference procedure. Verification or validation of the analyses relies on limited field performance data with reduced knowledge of the full scope of system conditions or loading conditions.

Research reported in this paper represents a portion of ongoing work to obtain a database of information useful for numerical model calibration. Specifically, a model of an earth dam with clay core founded on a liquefiable foundation subjected to earthquake loading is being studied. Several properties in the liquefiable foundation are varied to determine the related effects to the earth dam. In this paper, results from three centrifuge physical models will be presented. The models are identical, with the exception of the location (depth) of a liquefiable layer in the foundation, and subjected to the same shaking excitation. Results and discussion related to the significance of the depth of a liquefiable layer in the foundation and resulting damage to the earth dam will be presented.

KEYWORDS: Centrifuge, physical modeling, numerical modeling, liquefaction.

1.0 INTRODUCTION

It can be conservatively estimated that there are many seismically inadequate embankment dams around the world. Some of these embankments are founded on liquefiable soils, in many cases, necessitating the development of appropriate analyses options and remediation countermeasures [1]. Predominantly, these types of systems are analyzed and evaluated for safe performance during earthquake loading from some type of numerical finite element procedure. These procedures have been validated through laboratory testing and through limited confirmation from field case histories. Fortunately, there has been very few failures or near failures of earth dams. Therefore, the profession relies on results from the numerical procedures with an appropriate amount of conservatism on which to base decisions regarding safety or remediation. The analyses and consequently any remediation that may be required is an increasingly cost prohibitive process. There is a real need to establish some method of validating and verifying the results from the numerical analyses. Presently, the most attractive methodology is through scaled physical modeling where key aspects of the analysis or design can be verified. The only other alternative is to perform repeated numerical simulations with different algorithms, or perform some type of full scale testing. Neither one of these latter alternatives are very attractive.

The Corps of Engineers is currently in the process of updating their guidance on the dynamic evaluation of earth and rockfill dams [2]. 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 COE 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 centrifuge physical model tests have been completed as part of this research program. This paper will present the results from three of the physical models specifically concentrating on a liquefiable layer in the foundation that varies with depth.

2.0 TESTING PROGRAM

A total of 12 dynamic tests were performed on 12 different soil models. At a 100g gravitational acceleration field, the models depicted in Fig. 1 (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 1 gives the summary of the conducted centrifuge tests. As shown in Table 1, three key aspects of a liquefiable foundation layer were studied; I) thickness, II) depth, and III) location on the dynamic performance of the dam-foundation system. 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 FE codes or numerical schemes. Reported in this paper are the results of one of the Series II tests with preliminary results discussed. For a discussion of the Series I tests please see [3-5]. All of the test results are presented and discussed in prototype units, unless otherwise stated.

Table 1: CENTRIFUGE TESTING PROGRAM

Centrifuge Test Series	Model Configuration F1, F2, F3 D: 70% D_r; L: 35% D_r
Series I Effect of liquefiable layer *thickness*	L-L-L D-L-L D-D-L D-D-D
Series II Effect of liquefiable layer *depth*	D-D-L D-L-D L-D-D
Series III Effect of liquefiable layer *location*	L-D-D—D-D-D D-L-D—D-D-D D-D-D—L-D-D D-D-D—D-L-D
Series IV Clay Interlayer and Large Earthquake	Clay-D-L-L— Clay-D-L-L D-L-D—D-L-D

D_r: Relative Density

Nevada-120 sand was used in all tests. This is a fine (d_{50} = 0.15mm), uniform, sub-round, clean sand. Extensive data about the monotonic and cyclic response characteristics of this soil has been documented [6]. The embankment core was constructed of kaolin clay compacted at 32% water content (2% wet of optimum). It had a dry unit weight of 13.8 kN/m^3 and unconfined shear strength (Su) of 18-20 kPa. The soil model was constructed by air pluviation with the process interrupted periodically to place instrumentation. Thin bands of colored Nevada sand were placed at the interface of each horizontal foundation layer. Thin spaghetti sticks were also 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. A water/metulose solution having 50 times the viscosity of water was used as the foundation soil pore fluid. Therefore, this model is simulating a deposit in the field with foundation permeability twice that of Nevada Sand (approximately 1x10^{-4} m/s). The embankment shells and reservoir fluid was water rather than a viscous fluid to simulate a larger permeability. This was an attempt to model (strictly from a flow process) a coarser material more typically found in embankment shells as opposed to the foundation.

The physical models were all subjected to the same input motion, as shown in Fig. 2. This is a sinusoidal, 30 cycles, 150 Hertz signal input parallel to the base of the laminar box, with uniform acceleration amplitude of approximately 20 g. For the 100 g-centrifuge acceleration of these tests, this corresponds to a frequency of 1.5 Hertz, and peak horizontal acceleration of 0.2 g in prototype units (Fig. 2).

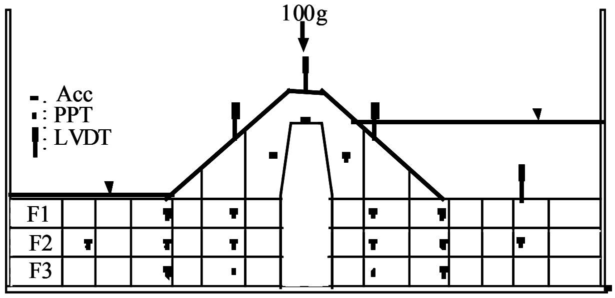

Fig. 1: Setup and instrument locations of centrifuge models.

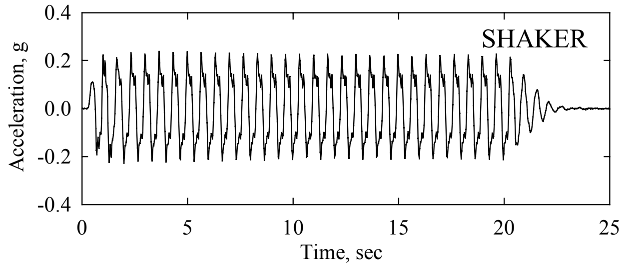

Fig. 2: Recorded input acceleration (g), centrifuge models.

3.0 RESULTS

As shown in Fig. 1 and Table 1, model DDL consists of a zoned earth dam founded on a foundation material having a permeability of approximately $1x10^{-4}$ m/s. The foundation is composed of three layers of material each 3 m thick composing the 9 m thick foundation. The 3 m thick foundation surface layer was constructed at a relative density of D_r = 70%, the middle layer to D_r = 70% and the bottom layer to D_r = 35%. The clay core extends all the way to the bottom of the model to provide a positive seepage cutoff. The sinusoidal motion of Fig. 2 was applied to the model with resulting accelerations, pore pressures, and displacements recorded and analyzed.

The recorded accelerations for model DDL are shown in Fig. 3, the recorded excess pore pressures in Fig. 4, and the measured deformations in Fig. 5. The recorded accelerations and excess pore pressures are used to determine the locations in the foundation where liquefaction occurred. For the purposes of this research, liquefaction is defined as occurring when the recorded excess pore pressures reach 100% of the initial vertical effective stress. There is a consistent pattern of agreement between the recorded accelerations and excess pore pressures related to occurrence of liquefaction. As expected, the loose D_r = 35% layer at the bottom of the foundation, very near the input motion, liquefied very rapidly. This occurred with a very high rate of pore pressure build up and within the first cycle of shaking. It is also apparent from the data that the material beneath the toes, both upstream and downstream, liquefied in the middle and top foundation layers of D_r = 70% material. Liquefaction may have extended in these layers out into the free field for some distance away from the toes; however, data was not recorded in these areas. Closer examination of the recorded excess pore pressures reveal a different rate of pore pressure build up for the data in the top and middle layer as opposed to the bottom layer. The build up is somewhat slower and it requires slightly more cycles of the input motion for liquefaction to occur. This could be attributed to the increased confining pressure from the dam or possibly to the pore pressure redistribution from the underlying liquefying layer. The recorded data beneath the embankment closer to the core indicates that liquefaction did not occur in this area. It is interesting to note that the recorded data from the top foundation layer beneath the dam reveals that initially the pore pressures began to increase and then went negative (most obvious in P7) before returning to a positive value. Apparently, there was a period during shaking where the pore fluid was being suctioned away from the area.

Shown in Fig. 5 are the recorded displacements from the crest, mid-slope and free field. The recorded free field values are only slightly positive reaching a maximum value of 0.15 m (positive indicating that the material heaved slightly). The mid-slope recording shows a settlement of 0.45 m and the crest a settlement of 0.8 m. This information is consistent with the internal deformations observed in the dam, also shown in Fig. 5.

From Fig. 5, a section through the dam immediately after shaking, observations related to the deformations of the vertical and horizontal markers placed inside the model during construction can be made. It is fairly obvious that the dam experienced a global settlement into the foundation as a result of the weakened layer with some slight bulging of material in the free field. However, there were no localized failures in the dam or the core. Neither were there excessive movements of the toes, only slight bulging and movements away from the core. The vertical markers provide a clear picture of the areas in the dam and foundation that were experiencing shear straining as a consequence of the loading and resulting liquefaction. The largest amount of shear straining occurred around the toe areas both upstream and downstream.

4.0 CONCLUSIONS

This paper presents the results from three centrifuge physical models simulating a zoned earth embankment sitting on a foundation with a loose liquefiable layer located at varying depths in the foundation. All three models were subjected to the same input motion with subsequent recorded data revealing information particularly related to the resulting deformations in the dam. These tests are part of a series of testing that is being performed to establish a database of physical models useful for verification and validation of numerical analyses routines. Several conclusions can be drawn from the results of the three models discussed in this paper. In all three models regardless of the location in the foundation of the loose layer, there was liquefaction occurring at all depths for the areas of the toes (upstream and downstream) out into the free fields for some undetermined distance. With the exception of the case where the loose layer was at the very bottom of the foundation, there was no liquefaction for any area beneath the dam moving from the toes towards the core. The two possible explanations for this fact are that the increase confining pressure from the dam and/or the static shear strength prevented the excess pore pressures from increasing to 100% of initial vertical effective stress, or that the redistribution of pore fluid away from this area out towards the toes and free fields contributed to this observation. The fact that the D_r = 70% material in the foundation readily liquefied is not surprising (this corresponds approximately to $(N_1)_{60}$ values of 20 bpf) but there are indications from the data that pore pressure redistribution aided in this occurrence. For models DDL and DLD there was clear redistribution of pore fluid from the foundation surface layer towards the toes, this was not observed in the LDD case. Discussions related to the observed deformations in the dam and foundation has already been presented. Suffice it to say that clear shear straining was observed in the foundation particularly the loose layers, the dam in all cases appears to have settled into the foundation as a consequence of strength loss in the foundation, there were no localized failures in the dam or core, the largest amount of deformation occurred at the toes, and deformations were as expected for all cases with the exception of case LDD. In this case smaller crest settlements were observed even though the loose layer was at the ground surface. An explanation was offered that the dam experienced less vertical movement and more horizontal movement as a consequence of the loss of strength in the near surface soil. 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.

5.0 REFERENCES

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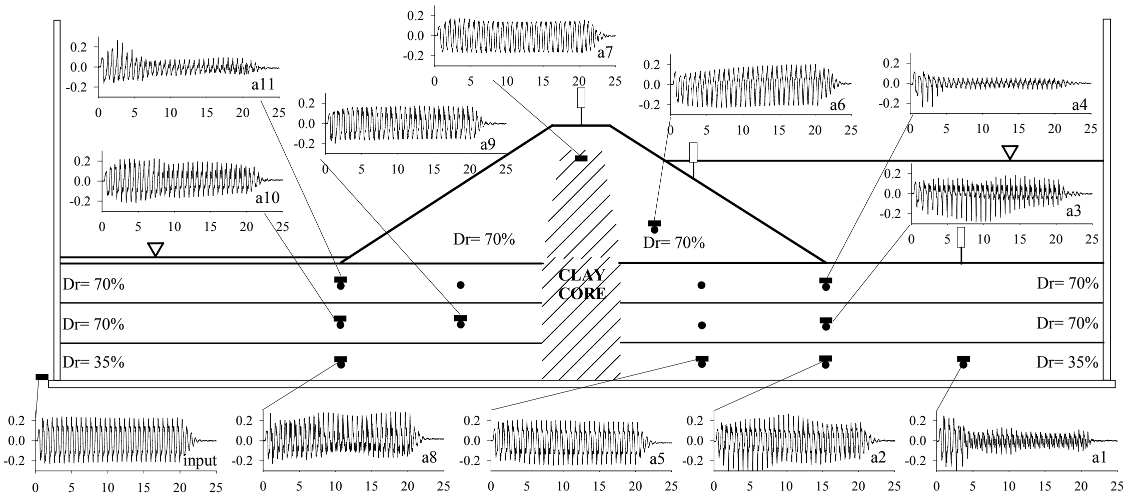

Fig. 3: Plots of time (sec) versus acceleration (g) recorded for model DDL.

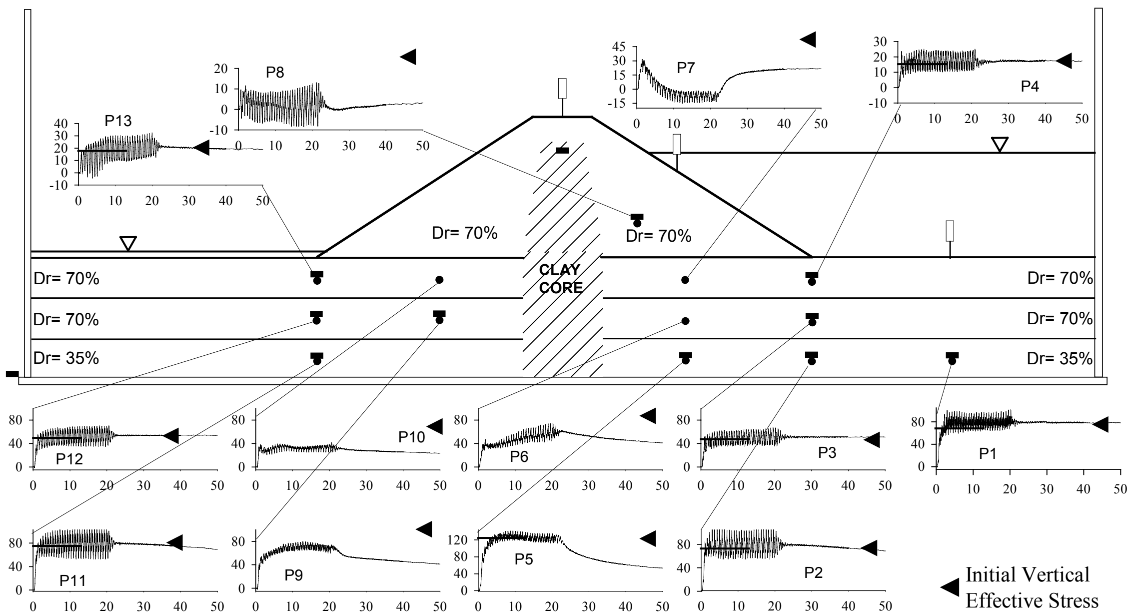

Fig. 4: Plots of time (sec) versus excess pore pressure (kPa) recorded for model DDL.

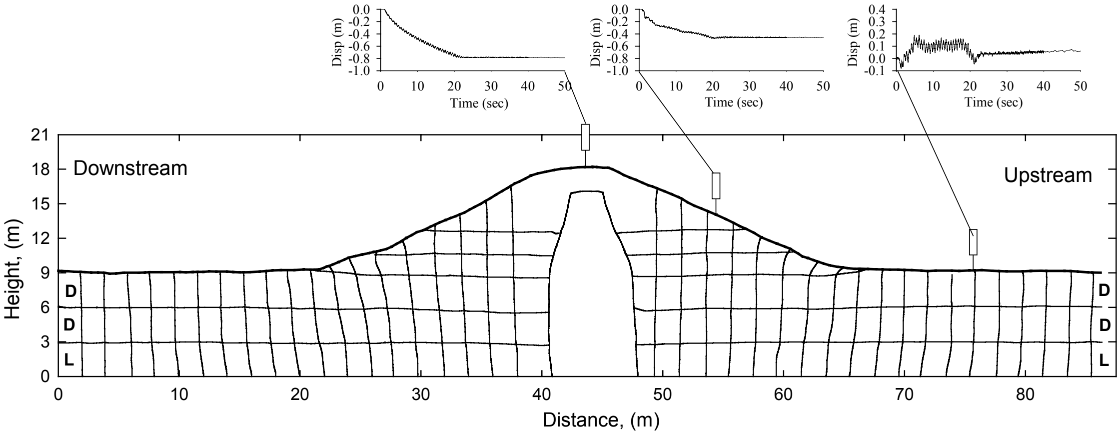

Fig. 5: Measured internal deformations and plots of time (sec) versus displacement (m) for model DDL.