Centrifuge Model Test of a Sand Embankment Built on Saturated Coal Ash

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KEYWORDS: fly ash, ash pond, slope stability, centrifuge

ABSTRACT

When an embankment of coal combustion residuals or soil is built over a coal ash pond, pore water pressures can accumulate in the underlying saturated ash deposits and trigger a rapid slope failure. A scale model was tested to obtain data on the conditions that may lead to a slope failure. A 6.5-inch tall sand embankment was built on top of a 6-inch thick deposit of saturated fly ash. The strength of the fly ash was characterized using triaxial compression and direct simple shear tests. To create stresses in the laboratory model that would match conditions in a prototype embankment in the field, the model was tested inside a geotechnical centrifuge. The model was spun to accelerations up to 100 g, simulating a 54-foot tall sand embankment. An air bladder was used to apply a surface pressure that simulated another 27 feet of sand fill. Instrumentation recorded displacements in the embankment and pore water pressures in the saturated ash. The embankment did not fail in the centrifuge test, consistent with analytical predictions. The stability of the model slope is attributed to the favorable strength of the ash deposit used in this particular test. The results demonstrate the feasibility of using centrifuge models to study the conditions that develop when an embankment is built over saturated coal combustion residuals.

INTRODUCTION

When a storage cell for coal combustion residuals failed at TVA’s Kingston plant in 2008, several million cubic yards of fly ash were released. Walton and Butler (2009) concluded that the collapse initiated within a thin, soft stratum at the base of the old ash deposits. Creep movements within this layer precipitated an undrained slope failure, and that resulted in static liquefaction of the retained, saturated ash. The Kingston failure led to concerns that similar problems could develop in active or closed ash ponds at other power plants.

In this study, a laboratory scale model was constructed with a sand embankment on top of a saturated fly ash deposit (Figure 1). The model was spun in a geotechnical centrifuge, to generate stresses matching those in a full-scale field embankment. A
surface pressure was also applied, in an attempt to induce an undrained slope failure. Instrumentation in the model monitored the pore pressure response of the ash and slope deformations during the test. Stability of the slope was evaluated using conventional, limit equilibrium analyses. Stress analyses were used to interpret the measured pore water pressures.

WHY TEST IN A CENTRIFUGE?

Natural soils and coal combustion residuals are particulate materials, which exhibit behaviors and properties that vary with confining pressure. Both strength and stiffness increase with depth below the ground surface. To generate realistic field stresses and soil behavior in the laboratory, a scale model of a soil slope can be spun inside a geotechnical centrifuge (Ling et al. 2009).

For example, an embankment that is 50 feet tall behaves differently than a model that is 6 inches tall. The stresses at the base of a 50-foot tall embankment are about 5,000 psf, but the vertical stress is only about 50 psf under a 6-inch embankment (Figure 2). However, if the 6-inch tall model is subjected to a centrifugal acceleration of 100 g, the behavior will mimic that of the 50-foot tall embankment.

The scaling law for seepage is not linear, and pore water in a centrifuge model tends to drain more quickly than in the field. Hence, to induce an undrained soil failure, loads must be applied more quickly in the centrifuge.
Here, the “model” is the ash deposit and embankment constructed for testing. When spun in the centrifuge, the model is a scale model, representing a “prototype” structure in the field. Testing at the prototype or field stress levels is important in understanding the behavior of coal ash under embankment loads.

Saturated coal ash may be subject to static liquefaction, but only under certain conditions where the ash exhibits a strain-softening response. If the material is contractive, shear stresses applied without drainage will cause increasing pore water pressures and a loss of strength. Static liquefaction will not occur if the ash is dilative, where shearing causes decreasing pore pressures and increasing shear resistance. For a saturated soil at a given density, the response will change from dilative at low confining pressures, to contractive and subject to liquefaction under high confinement. Hence, to study the potential for static liquefaction in the laboratory, testing at realistic stress levels is critical.

CENTRIFUGE MODEL

The model tests were conducted in the geotechnical centrifuge at the U.S. Army Corps of Engineers, Engineering Research and Development Center (USACE-ERDC) in Vicksburg, Mississippi. Their centrifuge can carry a maximum test payload of 8.8 tons, on a testing platform that measures 1.3 m by 1.3 m. The radius of the centrifuge arm is 6.5 m, from the center of rotation to the platform base. Models can be accelerated to a maximum of 350 g, with the machine spinning at 230 revolutions per minute.

The centrifuge model (ash test bed and sand embankment) was constructed inside a rigid rectangular box. The interior of the box measured 25 inches by 50 inches in plan, by 14 inches in height. A schematic cross section of the model test embankment is depicted in Figure 3. The saturated ash deposit was 6 inches deep, and filled the length and width of the test box. The sand embankment was 25 inches wide by 6.5 inches high, and was built with a surface slope of 1.5 horizontal to 1 vertical (1.5H:1V).

The test box had one transparent side to allow observations of the slope cross section during the test. The box, with the ash and sand model, was mounted in a hanging position at the hinged end of the centrifuge arm. As the arm was spun, the box swung up to a horizontal position. The applied accelerations acted perpendicular to the base of the model box, along the vertical axis of the prototype embankment (Figure 2).

Fig. 3. Cross sectional geometry of test embankment and ash foundation
PROPERTIES OF THE FLY ASH

The test bed ash was collected from a dry storage silo at a coal-fired power plant. The average fines content was 91% (percent by weight smaller than 0.075 mm). The specific gravity of the ash, measured using the water pycnometer method, was 2.51. Bachus and Santamarina (2012) observe that specific gravities determined in this manner can vary widely in coal ash, due to hollow cenospheres, internal grain porosity, poor wettability, and heavy ferromagnetic grains.

The shear strength of the ash was measured in five isotropically consolidated, undrained triaxial compression (CU triaxial) tests and three, K₀-consolidated direct simple shear (DSS) tests. The test specimens were prepared from a slurry of ash, like the test bed deposit (described later). The triaxial test specimens had an average dry unit weight of 100.7 pcf; the DSS specimens were looser, with an average dry unit weight of 93.5 pcf. Consolidation pressures between approximately 1,000 psf and 13,000 psf were used, representative of vertical effective stresses at depths between roughly 20 and 200 feet in an ash pond.

All five of the ash specimens exhibited a dilative response in triaxial compression. Without drainage, shearing caused decreasing pore pressures, increasing effective stresses and shear resistance, and strain hardening as the triaxial tests progressed. The DSS test at the lowest consolidation pressure (1,500 psf) also showed a dilative response. However, two DSS tests at higher consolidation pressures (7,200 and 13,000 psf) indicated a contractive behavior, with moderate softening past the peak strength. The contractive versus dilative behavior can be attributed to the lower density of the DSS specimens and the different stress paths applied in the two test methods.

SHEAR STRENGTH OF FLY ASH

Shear strength parameters determined from these tests are summarized in Table 1. The drained strength parameters (c' = 0, ϕ' = 30°) were fit to the triaxial and direct simple shear data (Figure 4) using the stresses on the failure plane at failure (σ'ff and τff). These stresses are directly measured in the DSS tests, and computed from the principal stresses measured in the CU triaxial tests.

Using only the CU triaxial data, a conventional “R-envelope” was fit to the measured undrained strengths (Figure 5). The fit was accomplished using p-q points, which

<table>
<thead>
<tr>
<th>Strength Envelope</th>
<th>Intercept</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drained</td>
<td>c' = 0</td>
<td>ϕ' = 30°</td>
</tr>
<tr>
<td>Undrained Conventional</td>
<td>c = 1,700 psf</td>
<td>ϕ = 27°</td>
</tr>
<tr>
<td>Undrained Triaxial Alternative*</td>
<td>d = 2,400 psf</td>
<td>ψ = 36°</td>
</tr>
<tr>
<td>Undrained DSS Alternative*</td>
<td>d = 500 psf</td>
<td>ψ = 17°</td>
</tr>
</tbody>
</table>

*Alternative envelopes defined using normal consolidation stress and shear stress on the failure plane.
Fig. 4. Drained strength envelope for fly ash, fit to CU triaxial compression and direct simple shear test results

represent the top of the stress circle, computed using the effective consolidation stress and the deviator stress at failure. The conventional R-envelope fit to the triaxial data results in $c = 1,700$ psf, $\phi = 27^\circ$. The ash was dilative over the range of test pressures and the pore pressures were negative at failure, resulting in higher shear strength compared to a drained loading. In Figure 5, the undrained strength envelope is thus above the drained strength envelope.

A conventional R-envelope is conservative. At a given effective consolidation stress, the shearing strength computed from the R-envelope is less than the shearing resistance
on the failure plane in a test specimen consolidated to the same pressure. An alternative representation of the measured strength can be written using the shear resistance on the failure plane at failure ($\tau_{ff}$), which is computed as a linear function of the normal effective stress on the failure plane in the consolidated state ($\sigma'_{fc}$). The slope and intercept of this strength envelope is denoted by $d$ and $\psi$ (Duncan and Wright 2005). When defined this way, the envelope matches how stresses are computed along a slip surface in a limit-equilibrium slope stability analysis.

The results from the CU triaxial tests are plotted in terms of the stresses on the failure plane ($\sigma'_{fc}$, $\tau_{ff}$) in Figure 6. The shear stresses on the failure plane were determined using the drained friction angle and the equations shown on the plot. A line through these data points has a slope and intercept of $d = 2,400$ psf, $\psi = 36^\circ$. Both sets of undrained strength parameters in Table 1 (conventional and alternative) represent the same triaxial test results. When used in a stability analysis, the $d$ and $\psi$ parameters result in a higher computed shear resistance compared to the conventional $c$ and $\phi$ envelope. However, Ladd (1991) argues that triaxial compression tests should not be used in design problems to characterize undrained strengths in soft soils, because the resulting strengths tend to be unconservatively high.

The stresses on the failure plane were explicitly measured in the DSS tests and are plotted in Figure 6. An alternative strength envelope fit to these points results in $d = 500$ psf, $\psi = 17^\circ$. The DSS tests indicate significantly lower undrained shear strengths, compared to the triaxial compression results. The lower DSS strengths can be partly attributed to the lower average density of the test specimens and the contractive behavior observed in the tested pressure range. In addition, the DSS test imparts a different stress path to the soil, which better represents the average stress path along a slip surface in a slope stability problem (Ladd and Foott 1974).

Fig. 6. Alternative undrained strength envelopes from consolidated undrained triaxial compression and direct simple shear tests on fly ash
CONSTRUCTION OF ASH DEPOSIT AND SAND EMBANKMENT

The test bed deposit of saturated ash was prepared by slurry consolidation. Using distilled water, the ash was mixed to an average water content of 35%, forming a thick slurry that readily flowed. The ash slurry was mixed in eight batches and poured in lifts over a period of three hours (Figure 7). The ash was not remixed between lifts, and there were no drains in the bottom or sides of the test box. The test bed was allowed to consolidate under self-weight overnight, prior to constructing the sand embankment. The final thickness of the consolidated ash was 6 inches; less than one-half inch of free water accumulated on top (Figure 8).

![Fig. 7. Pouring ash slurry in lifts](image1)
![Fig. 8. Completed 6-inch thick ash deposit](image2)

The test box was placed on a platform scale, so that the weight of ash slurry in each lift could be accurately measured. A total of 526.5 lbs of ash slurry was poured into the test box. The properties of the consolidated ash are listed in Table 2.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content</td>
<td>29.5%</td>
</tr>
<tr>
<td>Saturation</td>
<td>100%</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>89.9 pcf</td>
</tr>
<tr>
<td>Saturated Unit Weight</td>
<td>116.5 pcf</td>
</tr>
</tbody>
</table>

Table 2. Density properties of the ash

After removing the free water, the dry sand embankment was constructed directly on the wet ash surface. The embankment was built in lifts, using an overhead hopper and flexible tube to place the sand (Figure 9). A moderate flow rate was maintained as the sand was spread across the surface. To help prevent shallow surface slides, the sand slope was reinforced with a fine nylon mesh. The mesh sheets were laid with an edge 7.5 inches from the slope face, and wrapped up and over the end of each lift. The slope face was manually shaped to achieve the desired geometry (Figure 10). The completed embankment was 6.5 inches tall, was built in six lifts, and had five levels of mesh reinforcement. No visible compression of the ash occurred during construction of the sand embankment.

A total of 282 lbs of dry sand was used in constructing the embankment. Based on these measurements and an assumed specific gravity of 2.65, the sand embankment
had a void ratio of 0.716 and a dry unit weight of 96.3 pcf. A single, drained triaxial compression test indicated that the sand had a strength of $c' = 0$, $\phi' = 38^\circ$.

AIR BLADDER FOR SURCHARGE PRESSURE

An air bladder on top of the sand embankment (Figure 11) was used to simulate a taller embankment. The bladder was a 24 by 24 inch pneumatic lifting bag, with a rated capacity of 29.9 tons. An aluminum plate and reaction frame on top of the air bladder was bolted into the test box (Figure 12). The air bladder was remotely inflated and the inlet pressure was measured with a transducer.

The air bladder was relatively stiff, particularly around the edges. When the centrifuge model was disassembled after the test, the surface of the sand had a uniform imprint of the bladder surface. This suggests that the bladder, while restrained against significant expansion, had imparted a relatively uniform pressure on the sand. However, due to the stiffness of the bladder, the pressure applied to the top of the sand was less than the measured inlet air pressure. The system was calibrated to correct for this effect.
INSTRUMENTATION

Deformation markers were installed on the transparent front panel of the model box, to allow tracing the path of subsurface displacements. Using a dab of vacuum grease, multi-colored plastic tabs were temporarily stuck to the face of the clear panel, on a one-inch grid spacing. In the sand embankment, a line of glitter was placed on top of each lift, against the clear panel. The glitter lines and plastic markers can be seen in Figure 13. Six video cameras were positioned to allow remote observation of the model during the test.

Electronic instrumentation (Table 3 and Figure 14) was used to monitor conditions in the model. During each test run, the instrumentation was sampled and recorded at a rate of 10 times per second.

![Fig. 13. View of deformation markers through the transparent side of the test box](image)

**Table 3. Summary of electronic instrumentation**

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Number</th>
<th>Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laser Range Finders</td>
<td>2</td>
<td>Crest of Slope</td>
</tr>
<tr>
<td>LVDTs</td>
<td>2</td>
<td>Toe of Slope</td>
</tr>
<tr>
<td>Pore Pressure Transducers</td>
<td>8</td>
<td>2 Cross Sections</td>
</tr>
<tr>
<td>Bladder Air Pressure</td>
<td>1</td>
<td>Top of Sand</td>
</tr>
</tbody>
</table>

![Fig. 14. Instrument layout, with identification numbers](image)

Two linear variable differential transformers (LVDTs) were mounted at the toe of the slope, to measure vertical heave. Plastic discs were placed under the tips of the probes, to act as bearing surfaces to prevent penetration of the tips into the ash deposit. Two non-contact, laser range finders were installed to measure vertical settlements at the slope crest. These devices were installed side-by-side, to provide redundancy.

Miniature pressure transducers (PPTs) were installed in the ash to monitor pore water...
pressures. Eight transducers were used: two were mounted on posts standing on the bottom of the box (Figure 7), and six were inserted into the ash from the surface. The pressure transducers were placed in four locations on two cross sections, spaced 6.5 inches apart in the model. Four of the transducers stopped providing useful data during the centrifuge test runs. However, the transducer arrangement provided for redundancy, and measurements were obtained at all four of the instrument locations.

CENTRIFUGE TEST RUNS

The model was tested in the centrifuge on August 15, 2013. Three test runs (Table 4) were completed to increasing accelerations and surcharge pressure. Test Run 1 was stopped due to a malfunction in the data acquisition system, and Test Run 2 was stopped to secure a loose fitting. Each time a new acceleration level was reached, the speed was held constant for about 30 minutes to allow the induced pore water pressures to stabilize.

Figure 15 shows the slope toe at the end of Test Run 2, after the model had been subjected to an acceleration of 40 g. The markers indicated small vertical settlements under the sand embankment (the colored markers were initially at the crossings of the black grid lines). Less settlement was observed at markers that were not under the embankment. The markers also showed a small movement to the right. These displacements are consistent with vertical settlement and lateral bulging due to constant volume deformation and consolidation under an embankment. These observations do not indicate an incipient slope failure.

An effort was made to induce an undrained failure in Test Run 3. The centrifuge was ramped from 40 g to 100 g, which took 380 sec (6.3 min). The machine was run at 100 g for about 2 min, long enough to confirm that the instruments were reading, then the air bladder was inflated. Pressure in the air bladder increased rapidly for about 20 sec, when bolts on the reaction frame broke. The results are discussed later, but a slope failure did not occur.

Per the scaling law, the prototype embankment height is equal to the model height times the acceleration. At the maximum acceleration of \( N = 100 \) g, the 6.5-inch tall

<table>
<thead>
<tr>
<th>Test Run</th>
<th>Applied Accelerations (g)</th>
<th>Maximum Bladder Inlet Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2, 10, 20, 30</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2, 10, 20, 30, 40</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>2, 10, 20, 30, 40, 100</td>
<td>30</td>
</tr>
</tbody>
</table>

Table 4. Summary of centrifuge test runs

Fig. 15. Slope toe after Test Run 2
model represented a 54-foot tall prototype embankment. The maximum surcharge pressure of 18 psi (at a bladder inlet pressure of 30 psi) represented another 27 feet of dry sand on top of the embankment.

MEASURED DISPLACEMENTS

Vertical displacements measured in Test Runs 2 and 3 are plotted in Figures 16 and 17. In general, the data indicate small settlements that can be attributed to consolidation. There was no measured heave at the toe of the slope, and no significant movements that would indicate slope instability.

![Fig. 16. Vertical displacements measured in Test Run 2](image-url)
The laser devices (A2 and A4) were positioned to measure settlements at the slope crest, while the LVDTs (D1 and D2) monitored displacements of the ash surface at the slope toe (Figure 14). All four instruments were supported on the reaction frame on top of the test box. The centrifugal accelerations during the test increased the weight of the frame, causing it to sag. The resulting movement of the instrument mounts appears as a fictitious, upward heave of the ground surface in the data (Figures 16 and 17).

One way to separate out these effects is to consider only the measurements at the beginning and end of a test run, when the applied accelerations are zero. For Test Run 2 (Figure 16), displacements measured at the end of the test provide a clear indication of small vertical settlements. The surface of the exposed ash settled about 0.01 to 0.02 inch, due to consolidation of the ash under the applied acceleration. At the slope crest,
the surface settled about 0.04 to 0.05 inch. More settlement occurred here, due to compression of the sand and consolidation of the ash under the weight of the embankment. At prototype scale, the settlements would be 0.4 to 0.8 inch at the toe, and 1.6 to 2.0 inches at the slope crest.

In Test Run 3, inflation of the air bladder caused an upward movement of the reaction frame; this should appear as a vertical settlement in the data records. However, only a few data points were recorded during inflation of the air bladder, making it difficult to discern this effect in Figure 17. Lacking data at the end of Test Run 3, the measured displacements were not corrected for sag of the support frame.

PREDICTED STABILITY

Stability of the model embankment was evaluated using conventional limit-equilibrium methods. Post-test analyses were completed for the maximum applied acceleration ($N = 100$ g) with 0 or 35 psi surcharge pressure.

The model dimensions (the ash was 6 inches deep) and measured unit weights were used in the analyses. A vertical, pseudostatic coefficient was specified to represent the applied acceleration (equivalent to scaling the unit weights). The reinforcing mesh was modeled by adding horizontal elements that impart tensile forces within the sand, assuming deformations sufficient to fully engage the reinforcement. Hydrostatic pore pressures were assumed at the start of each load step; suction pressures in the sand embankment were ignored.

The stability was analyzed for drained and undrained conditions in the ash, using the four sets of strength parameters in Table 1. Single-stage analyses were used, where the soil strength and factor of safety were both computed with the applied load. None of the analytical results (Table 5) predict a slope failure, consistent with the behavior observed during the centrifuge tests.

<table>
<thead>
<tr>
<th>Strength Envelope for the Ash</th>
<th>$N = 100$ g Surcharge = 0 psi</th>
<th>$N = 100$ g Surcharge = 35 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drained</td>
<td>1.86</td>
<td>1.43</td>
</tr>
<tr>
<td>Undrained Conventional</td>
<td>2.17</td>
<td>1.57</td>
</tr>
<tr>
<td>Undrained Triaxial Alternative</td>
<td>2.16</td>
<td>1.57</td>
</tr>
<tr>
<td>Undrained DSS Alternative</td>
<td>1.71</td>
<td>1.29</td>
</tr>
</tbody>
</table>

MEASURED PORE WATER PRESSURES

Eight pore pressure transducers (PPTs) were buried in the ash, at four locations on two parallel cross sections (Figure 14). Some of these instruments malfunctioned during the test runs. The transducer readings were zeroed at the start of each test run; the data
plotted here thus represent the change in pressure during the test.

Pore pressure data from Test Run 2 are plotted in Figure 18. Each of the pressure records show a rapid climb as the centrifuge was ramped up to 40 g at about 1500 sec. The peak pore pressure was followed by a gradual dissipation to the static pressures noted on Figure 18. The static pressures changed with the applied acceleration, which multiplied the unit weight of water. The largest peaks and declines were measured at PPT3 and PPT6, which are similarly located beneath the full height of embankment. The records from PPT2 and PPT4 show a less pronounced peak and decline; these transducers were located at different depths under the middle of the embankment slope.

![Fig. 18. Pore pressures measured under 40 g in Test Run 2](image-url)
PPT8 was located at the slope toe. This transducer recorded a small peak, followed by gradually increasing pressure. At the start of the test, capillarity had saturated the lower 1.5 inches of the sand embankment. As the model was accelerated, some of this water was pushed out of the sand and accumulated on top of the ash, gradually increasing the static pore pressures. The change in pressure at PPT8 was about 0.15 psi, corresponding to a water depth of 0.1 inch at 40 g. This is consistent with the accumulated water observed by video during the test. In subsequent analyses of the excess pressures at the other instruments, the static pressures were corrected by 0.15 psi to account for this effect.

Data from Test Run 3, for the portion where the applied acceleration was 100 g, are shown in Figure 19. As in Test Run 2, the data shows peak pore pressures when the centrifuge reaches 100 g, followed by dissipation. The centrifuge was not run at 100 g long enough to measure a steady, static pore pressure. To quantify the excess pressure for further analysis, the static pressures at 100 g were computed using sensor depths inferred from the static pressures measured at 40 g. In contrast to the results from Test Run 2, the largest excess pressures in Test Run 3 were measured below the mid-slope at PPT2. In addition, the excess pressures at PPT6 were significantly higher than recorded at PPT3, even though both transducers were in similar locations.

The air bladder was inflated during Test Run 3, with a constant acceleration of 100 g. The data are plotted in Figure 20. The pressure at the inlet to the air bladder reached a maximum of 59.08 psi at 3,203 sec. At that point, the mounting bolts on the reaction frame broke, the bladder expanded, and the inlet pressure dropped rapidly. The pressure started to build up again, before the operator vented the bladder. At the time of the peak bladder pressure (3,203 sec), the pore pressures in the ash are declining (Figure 20). The bladder inlet pressure and the ash pore pressures all rose sharply as the bladder was initially inflated. The slope of the pressure-time curves changed at 3,191 sec, beyond which the pore pressures did not rise as quickly as the bladder inlet pressure. The change at 3,191 sec appears to mark when the first bolt on the reaction frame broke. Past that point, it is unclear how much of the bladder pressure was being applied to the embankment surface. Hence, the excess pore pressures were evaluated for the instant at 3,191 sec. The inlet pressure to the bladder was 30.10 psi; based on the bladder calibration, the pressure on the sand surface would have been 18 psi.

The measured excess pressures were much less than the applied surcharge pressure. The largest value (about 12% of the 18 psi surcharge) was at PPT6, which was located under the sand, but just beyond the horizontal limits of the air bladder. The excess pressures at PPT2, located under the mid-slope, were about the same magnitude.

STRESS AND DEFORMATION ANALYSIS

A finite element analysis was used to compute the stresses induced in the centrifuge model. The results were used to interpret the measured pore pressures, as discussed later.
The finite element analysis assumed no drainage, used total stresses, and assumed isotropic, linear-elastic soil behavior. Young's moduli were determined from the triaxial test results: $E = 600$ ksf for the sand, $E = 1,100$ ksf for the ash (for axial strains up to 0.5%). Poisson’s ratios were estimated: $\nu = 0.28$ for the sand, $\nu = 0.49$ for the ash.

The pattern of displacements predicted with the finite element analysis generally matched the pattern visible in Figure 15. However, for Test Run 2 at 40 g, the predicted lateral displacement at the slope toe (0.003 inch) was much less than observed (about 0.1 inch in Figure 15). The finite element analysis did not model vertical consolidation settlements, which are visible in Figures 15 and 16.
ANALYSIS OF INDUCED PORE PRESSURES

The excess pore pressure ($\Delta u$) resulting from a load increment is the difference between the peak measured pressure ($u_{\text{peak}}$) and the static pressure ($u_0$):

$$\Delta u = u_{\text{peak}} - u_0$$  \hspace{1cm} (1)

The excess pore pressures measured in the centrifuge tests (Table 6) can be related to changes in the mean and shear stresses caused by the imposed loads. Using the finite element solution, principal stresses ($\sigma_1$, $\sigma_2$, $\sigma_3$) were calculated at each piezometer location. The mean stress and shear stress were represented using the normal and shear stresses on the octahedral plane ($\sigma_{\text{oct}}$, $\tau_{\text{oct}}$):

$$\sigma_{\text{oct}} = \frac{1}{2} (\sigma_1 + \sigma_2 + \sigma_3)$$  \hspace{1cm} (2)

$$\tau_{\text{oct}} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2}$$  \hspace{1cm} (3)

The finite element model assumed linear elastic behavior, so the superposition principle can be used to determine the change in stress (induced stress) between subsequent load steps. The induced stresses ($\Delta \sigma_{\text{oct}}$, $\Delta \tau_{\text{oct}}$) in Table 6 were thus determined by subtracting results from two finite element solutions.

Henkel proposed a pore pressure equation that accounts for the effects of the intermediate principal stress (Holtz and Kovacs 1981):

$$\Delta u = B (\Delta \sigma_{\text{oct}} + a \Delta \tau_{\text{oct}})$$  \hspace{1cm} (4)

where $a$ and $B$ are empirical coefficients. Considering only stress paths where $\sigma_2 = \sigma_3$, Skempton proposed this pore pressure equation (Holtz and Kovacs 1981):

$$\Delta u = B [\Delta \sigma_3 + A (\Delta \sigma_1 - \Delta \sigma_3)]$$  \hspace{1cm} (5)

where $A$ and $B$ are empirical coefficients.

In soft, saturated soils (such as the test bed fly ash), $B$ will be close to 1 in Equations (4) and (5). The values of $a$ and $A$ depend on the stress path, strain level, and behavior of the soil when sheared. Values of $a > 0$ or $A > \frac{1}{2}$ indicate a contractive soil that generates increasing pore pressures when sheared, and $a < 0$ or $A < \frac{1}{2}$ indicates a dilative soil that generates decreasing pore pressures when sheared.

Assuming $B = 1.0$, pore pressure parameters were computed using the excess pressures measured in the centrifuge tests and the induced stresses computed with the finite element analysis. The results are tabulated in Table 6. This data is from relatively small strains, and not failure conditions as more typically represented with pore pressure parameters.

The pore pressure parameters at PPT3 and PPT6 are similar for each load increment, because these instruments are at the same relative location under the slope and experience similar stress paths. In general, the computed stresses and pore pressure
parameters are affected by the uncertainty in the locations of the pressure transducers. An error of 0.5 inch in a location within the model represents an error of 50 inches (4 feet) at the prototype scale, when the model is subjected to 100 g.

Table 6. Pore pressure parameters (a and A) determined for the test bed fly ash

<table>
<thead>
<tr>
<th>Measured at</th>
<th>Change in Load Condition</th>
<th>Excess Pore Press. Δu (psf)</th>
<th>Induced Stresses</th>
<th>Assuming B = 1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Δσoct (psf)</td>
<td>Δτoct (psf)</td>
<td>a</td>
</tr>
<tr>
<td>PPT2</td>
<td>30 g to 40 g</td>
<td>31</td>
<td>496</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>40 g to 100 g</td>
<td>160</td>
<td>2976</td>
<td>392</td>
</tr>
<tr>
<td></td>
<td>100 g + 18 psi</td>
<td>316</td>
<td>514</td>
<td>247</td>
</tr>
<tr>
<td>PPT3</td>
<td>30 g to 40 g</td>
<td>51</td>
<td>516</td>
<td>57</td>
</tr>
<tr>
<td></td>
<td>40 g to 100 g</td>
<td>28</td>
<td>3094</td>
<td>340</td>
</tr>
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<td></td>
<td>100 g + 18 psi</td>
<td>206</td>
<td>1022</td>
<td>199</td>
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<td>PPT6</td>
<td>30 g to 40 g</td>
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<td>516</td>
<td>57</td>
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<tr>
<td></td>
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<td>71</td>
<td>3094</td>
<td>340</td>
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<tr>
<td>PPT8</td>
<td>30 g to 40 g</td>
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<td>179</td>
<td>66</td>
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<tr>
<td></td>
<td>40 g to 100 g</td>
<td>3</td>
<td>1075</td>
<td>396</td>
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<tr>
<td></td>
<td>100 g + 18 psi</td>
<td>52</td>
<td>302</td>
<td>245</td>
</tr>
</tbody>
</table>

DILATIVE VERSUS CONTRACTIVE BEHAVIOR

Static liquefaction is associated with a substantial loss in shearing resistance past the peak strength, resulting from increasing pore pressures when contractive soils are sheared without drainage. For a given density, a soil will be dilative at low confining pressures and contractive under high confinement. Bachus and Santamarina (2012) concluded that even loosely placed fly ash tends to be dilative when sheared, and this tends to prevent static liquefaction in ash ponds that are less than about 20 to 25 m (65 to 80 feet) deep.

In the centrifuge test at 100 g, the vertical effective stresses in the ash were less than 8,000 psf. Test specimens consolidated in this pressure range showed a dilative response in triaxial compression. The direct simple shear tests, which are thought to better represent the average stress path on a slip surface, showed a transition from a dilative to a contractive response in this pressure range.

All of the a parameters in Table 6 are negative, indicating a dilative response in the ash in each load increment. While positive excess pore pressures were measured, the observed increases were less than the increases in mean stress. This implies that there were concurrent decreases in pore pressure, due to a dilative response in the ash under the imposed shear stresses.
DRAINAGE DURING THE TEST

Comparing the values in Table 6 at any of the PPT locations, $a$ and $A$ are similar in the test increments to 40 g and 100 g. The values are then much less negative under the surcharge pressure. The pore pressure parameters determined under the 18 psi surcharge are reasonable for a saturated, dilative soil sheared without drainage.

The more negative pore pressure parameters for the acceleration increments were computed from the small excess pressures (relative to the magnitude of the induced stresses) that were measured. This may be explained, however, by partial drainage of the excess pressures as the centrifuge accelerations were increasing. The surcharge pressure was applied rapidly (about 10 sec to go from 0 to 18 psi), much more rapidly than the centrifuge accelerations could be increased (over 6 minutes to increase from 40 g to 100 g). Dissipation of excess pressures occurred more rapidly in the model than would occur in a field prototype, because seepage rates in a centrifuge model do not scale linearly.

Hence, undrained load conditions were achieved only during application of the surcharge pressure. The pore pressure parameters derived from the surcharge test thus provide a better indication of ash response.

CONCLUSIONS

A 6.5-inch tall sand embankment was built on top of 6 inches of saturated fly ash. The model was spun in the centrifuge to an acceleration of 100 g, where the embankment had a prototype height of 54 feet. A surcharge pressure of 18 psi was then applied, simulating the rapid addition of an additional 27 feet of sand.

The slope did not fail in the test, and measured displacements were small. Slope stability analyses indicated that the slope would not fail. These analyses were completed using ash strengths measured in the laboratory. The laboratory data showed that the test bed ash, at the tested density and stress levels, had a tendency to dilate when sheared.

Excess pore water pressures were measured in the ash under the slope in each increment of the test. These measurements were combined with numerical computations of the induced stresses to quantify pore pressure parameters. The parameters computed for the different load increments indicated a dilative response in the ash deposit. Partial drainage was apparently occurring in the ash as the centrifuge was spun to higher accelerations. Undrained conditions were created only with rapid application of the surcharge pressure.

The completed test demonstrates the feasibility of using centrifuge models to study the stability of embankments built over ash. However, because of inherent experimental limitations, centrifuge test results should not be assumed to directly indicate field behavior. The rate of pore pressure dissipation is faster in the centrifuge model than will
occur in the field, and this effect must be considered in planning and interpreting centrifuge model tests.

Centrifuge testing is most valuable when the results are used to validate methods of engineering analysis. The centrifuge model results can be used to test the appropriateness of design procedures, including limit-equilibrium slope stability analyses and the interpretation of laboratory tests to characterize ash strength. If validated against centrifuge results, these methods can be used with greater confidence to predict behavior in the field.

The completed laboratory tests, engineering analyses, and results from the centrifuge model demonstrate that this embankment geometry would be unlikely to fail if built over a saturated deposit of this particular ash. However, the results from this one test should not be interpreted as an indication that slope failures will not occur when other embankments are built over other ash deposits.

The stability of embankments built in the field will depend on a number of factors that require appropriate engineering evaluation. Stability depends on embankment height, steepness of the slope, thickness of the ash deposit, groundwater levels, unit weights, and soil and ash strength. Key factors are:

- The tendency of the ash to generate increasing or decreasing pore pressures when sheared (contractive versus dilative response).
- The rate of load application relative to the rate of pore pressure dissipation, leading to partial drainage conditions.

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REFERENCES


