Cyclic Triaxial Testing of Water-Pluviated Fly Ash Specimens

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KEYWORDS: sluiced fly ash, cyclic laboratory testing, seismic design, stress-strain behavior

ABSTRACT

Liquefaction resistance of soils during earthquake shaking can be quantified in terms of the cyclic resistance ratio (CRR). The CRR can be estimated based on field and/or laboratory methods. However, field data correlations are typically based on behavior of natural soils (sands, silty sands, etc.) and may not be directly applicable to unique materials such as Class F fly ash. In these cases, cyclic testing in the laboratory may provide more definitive results. As part of the closure design process for the Class F fly ash pond at American Electric Power's (AEP) John E. Amos Plant, cyclic triaxial testing was performed to estimate CRR. Because it is difficult to obtain undisturbed samples of the soft, sluiced fly ash from the pond, specimens were reconstituted via water-pluviation methods in the lab. For the confining stresses that were considered, the behavior of the fly ash was shown to be dilative, which was consistent with the material response during conventional triaxial compression testing. Dilative behavior during an earthquake is associated with "cyclic mobility", as opposed to "flow failure". Cyclic mobility is associated with relatively low deformations, while flow failure is associated with relatively large deformations. The results of the testing were combined with testing by others (using the same material) to evaluate trends in CRR versus number of loading cycles (N_{liq}) that causes liquefaction. These trends were then compared to anticipated earthquake loading from a design event, which showed that liquefaction was unlikely to occur.

BACKGROUND

The John E. Amos Power Plant is located between Old US Route 35 (Winfield Road) and the Kanawha River in Putnam County, West Virginia. It is owned and operated by Appalachian Power Company, a subsidiary of American Electric Power (AEP). Located southwest of the plant, the Fly Ash Pond Complex was constructed and operated as a wet disposal facility for sluiced Class F fly ash. It is no longer in use. The approximate surface area of the impoundment is 160 acres (64.8 hectares) at a normal pool elevation of 858 feet (262 meters). The dam's current height, as measured from the downstream toe, is approximately 220 feet (67 meters). Stantec Consulting Services Inc. (Stantec) has been contracted by AEP to provide permitting and design services for

the closure of the Fly Ash Pond Complex. The closure must be designed to meet certain stability criteria, including seismic loading cases. Therefore, an evaluation of the liquefaction potential of the sluiced fly ash was performed.

PURPOSE AND OBJECTIVES

During the Background Review and Gap Analysis in the initial phase of the project, one of the main data gaps identified was a need for characterization of sluiced ash deposits, soil, and rock within the ash pond footprint. Limited properties and data existed for the ash contained at this facility. The properties of interest include thickness and variation, piezometric levels, and engineering properties.

To support engineering design, estimates of in-situ properties such as strength, unit weight, compressibility, hydraulic conductivity, etc. are required. One common approach is to obtain relatively undisturbed samples from the field and to perform laboratory tests on trimmed specimens. Stantec performed such tests, which were documented in our geotechnical data report[1]. Sluiced fly ash is often deposited in a soft/loose state, such that sample disturbance can make it difficult to obtain representative samples for laboratory testing. Another common approach is to perform in-situ field tests that provide direct measurement of properties, or allow correlation to various properties based on published relationships. An example of a common correlation is to relate standard penetration test (SPT) blowcounts to the angle of internal friction. Again, the soft/loose state of sluiced fly ash renders many common field tests invalid due to low resistance to penetration. Further, common correlations are often derived for natural sands, silts, and/or clays and may be not applicable to special materials such as fly ash.

In an effort to perform laboratory testing on representative samples of fly ash, AEP has supported research and development of new sample preparation and testing methods. In conjunction with researchers at The Ohio State University (OSU), AEP has developed methods to prepare "pluviated samples", meaning the soil (fly ash, in this case) is deposited into a column of water and allowed to settle out of suspension until a sample is formed[2]. "Undisturbed" specimens can be taken from the sample and subjected to various engineering tests. While these methods have yet to be formally standardized, AEP has provided references and experienced personnel to help promote consistency as they train consultants (such as Stantec) to apply these methods for various projects.

WATER-PLUVIATED SAMPLE PREPARATION

In order to prepare pluviated samples and obtain "undisturbed" specimens for testing, three main pieces of equipment are necessary: (1) a pluviation chamber, (2) apparatus for consolidating the sample, and (3) thin-walled tubes to obtain specimens. Stantec developed a standard operating procedure (SOP) for preparation of the samples. AEP reviewed the SOP and offered comments, which were incorporated into the final version.

Based on similar equipment used by AEP personnel who had previously done research at OSU[2], Stantec modified a triaxial chamber to serve as the pluviation chamber. The

chamber has an inside diameter of 4.5 inches (11.43 cm) and a height of approximately 12 inches (30.48 cm). A photo of the chamber is shown in Figure 1. A porous plastic plate was inserted into the bottom of the chamber and covered with filter paper. Dry fly ash was mixed with water to form a slurry, which was poured into the chamber in at least three portions (i.e., lifts) to make a sample. The ash particles were allowed to settle out of suspension in between each lift. After the sluicing process was complete, filter paper was placed on top of ash and a porous plastic top cap was installed. The top portion of the triaxial chamber was assembled and the chamber was placed in a GeoJac load frame for the consolidation phase. The sample was consolidated under a vertical stress of approximately 5 pounds per square inch (34.5 kPa), while allowing drainage from both the top and bottom of the specimen. After the consolidation phase, the top portion of the chamber and the top cap were removed. Thin-wall steel tubes (1.4-inch (3.6 cm) or 2.9-inch (7.4 cm) diameter) were pushed by hand into the ash to obtain specimens for testing.

Figure 1. Modified Triaxial Chamber for Preparation of Pluviated Samples

CONVENTIONAL LABORATORY TESTING

Engineering Classification

Three composite fly ash samples were subjected to the following engineering classification tests: particle size analysis (ASTM D422), Atterberg limits (D4318), and specific gravity (D854). Composites 2 and 3 were nearly identical in terms of particle size distribution, Atterberg limits, and specific gravity. Therefore, only one of these

(Composite 3) was advanced for pluviated sample preparation and testing. Composite 1 had a slightly greater sand-sized fraction (roughly 22 percent, compared to 12 to 14 percent for Composites 2 and 3), and was also advanced for pluviated testing. Composite 1 classified as silt with sand (ML), while Composite 2 and 3 classified as silt (ML).

Static Shear Strength

Consolidated undrained (CU) (D4767) and unconsolidated undrained (UU) (D2850) triaxial tests were performed on pluviated specimens from Composites 1 and 3. The range of confining pressures was selected consistent with the testing, which was representative of the range present in the field.

The stress-strain and pore pressure responses during CU triaxial testing was consistent with the behavior of undisturbed Shelby tube specimens tested at similar confining stresses. At lower confining stresses (20 and 40 psi; 138 and 276 kPa) the specimens exhibited dilative behavior, including little to no drop in post-peak strength and generation of negative pore pressures during shearing. At a confining stress of 60 psi (414 kPa), which would represent only the deepest ash deposits within the central valley of the pond footprint, the behavior transitions towards contractive, with a small drop in post-peak strength. However, the pore pressure response was still negative.

Estimates of drained friction angle (assuming c'=0) were 35 and 36.5 degrees for Composites 1 (1 data point) and 3 (2 data points), respectively. These results are consistent with estimates based on undisturbed Shelby tube specimens. Undrained shear strengths based on CU triaxial testing would be greater than the drained strength due to dilation. However, it is common to neglect such additional strength when performing slope stability analyses because it may not be reasonable to rely on negative pore pressures during various loading scenarios.

Estimates of undrained shear strength (based on UU testing and assuming phi=0) range widely from approximately 3,800 pounds per square foot (psf) (182 kPa) for Composite 1 (1 data point) to 8,700 psf (417 kPa) for Composite 3 (2 data points, same result). The UU results for Composite 3 are well above the range observed for undisturbed Shelby tube samples, but are strongly influenced by the component of shear strength that is due to dilation (i.e., negative pore pressures). Again, in many cases it is not appropriate to use higher strengths that are due to negative pore pressures.

Dilative Behavior

The CU triaxial compression testing results would indicate that the sluiced ash has higher undrained strength than drained strength, which can only occur if negative pore pressures are generated and lead to additional strength via suction pressures. Typically, sluiced ash is assumed to be deposited in such a loose state that it will contract upon shearing, even at low confining pressures. However, most of the CU triaxial test results show that the sluiced ash demonstrates classic dilative behavior, illustrated by the shape of the stress path and the pore pressure response (an example is shown in Figure 2).

The pore pressure response in the CU triaxial tests is particularly indicative of dilation during shear, in which negative pore pressures are generated as the sample attempts to expand. Stress-strain behavior is also generally dilative, indicated by a lack of significant post-peak drop in strength. However, as confining pressures increase, the behavior of this same material should change to contractive. In a small number of tests at the highest confining pressures (representative of the ash near the bottom of the deepest portions of the pond), this transition can be seen in terms of slightly positive pore pressures during shearing. It should be noted that a portion of the pore pressure readings for Tests B and C in Figure 2 are probably incorrect as the readings approach roughly -3 to -4 tons per square foot (tsf) (-290 to -380 kPa), as indicated by a constant (i.e., flat) value for the remainder of the test. This may be an indication of air bubbles forming within the testing apparatus, a common problem when soils attempt to dilate during CU triaxial testing. If valid readings could have been obtained for the entire test, the pore pressure would be expected to decrease further, thus causing the deviator stress to increase further (consistent with dilative behavior). The net result is that the test results are still useful, but may underestimate the undrained strength that would be measured by the test.

During design, the increase in undrained shear strength due to negative pore pressures is often neglected because such suction pressures cannot be sustained. This may be particularly true for the relatively pervious sluiced ash, which will dissipate excess pore pressures rather quickly during and after shearing. In such cases, it is common to assume that the lower drained strength will control design during otherwise undrained loading scenarios.

Figure 2. Dilative Response of Sluiced Fly Ash during CU Triaxial Testing
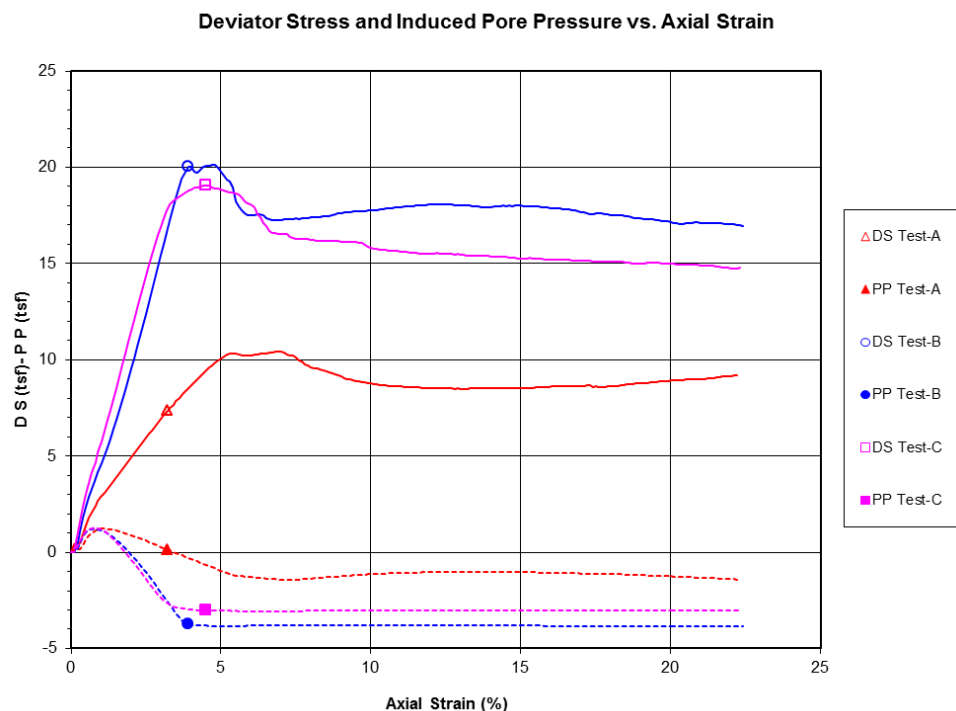
CYCLIC TRIAXIAL TESTING AT UK

Description of Specimens

Specimens were reconstituted using representative samples of fly ash recovered from the John E. Amos Power Plant. Specimens were prepared by pluviating wet fly ash into a cylinder and confining the material in a load frame under a constant vertical confining stress. After confining the specimens for approximately one day, the specimens were extruded and trimmed to a size that was compatible with the cyclic triaxial testing system at the University of Kentucky (UK) with a nominal diameter of around 2.8 in. (7.1 cm). Specimens were then placed into the cyclic triaxial cell, saturated, and consolidated using a mean effective confining stress of 20.0 psi (138 kPa). Details regarding the two trimmed specimens after saturation and consolidation are given in Table 1.

Table 1. Properties of UK Specimens Used for Cyclic Triaxial Testing

Specimen No.-→	1	2
Mass, lbs (g)	2.30 (1044.0)	2.46 (1117.74)
Height after saturation, in (cm)	5.96 (15.15)	6.35 (16.13)
Height after consolidation, in (cm)	5.94 (15.09)	6.33 (16.09)
Void ratio after saturation	0.82	0.89
Void ratio after consolidation	0.81	0.88
Total unit weight after saturation, pcf (kN/m^3)	110.8 (17.4)	107.6 (16.9)
Total unit weight after consolidation, pcf (kN/m^3)	111.5 (17.5)	108.3 (17.0)
Plasticity index (%)	Non-plastic	Non-plastic
Specific gravity	2.31	2.31
Calculated B-value during saturation (%)	100.8	99.0
Moisture Content (%)	39.7	41.2

Test Results

Details regarding the parameters used for cyclic loading are given in Table 2. For each specimen, an initial cyclic deviator stress was selected to achieve a target cyclic stress ratio. During testing, the stresses are measured directly, and are typically found to be slightly different than the target values. As a result, actual cyclic stress ratios are typically slightly different than the target values.

Liquefaction is defined herein as the state where the effective stress in the specimen reaches zero. It is typically expressed in terms of pore pressure ratio r_u, where r_u is defined as excess pore pressure in the specimen divided by the initial mean effective confining stress in the specimen prior to cyclic loading. When the pore pressure ratio reaches unity, liquefaction is said to have occurred. Note that the criteria for defining liquefaction may vary between testing laboratories.

**Table 2. Parameters Used for Cyclic Triaxial Testing of UK Specimens**

| Specimen No. →	1	2
Loading frequency (Hz)	1.0	1.0
Cyclic deviator stress, psi (kPa) (peak to peak)	12.0 (82.7)	22.0 (152)
Target cyclic stress ratio	0.150	0.275
Actual cyclic stress ratio	0.135	0.269
Number of cycles to liquefaction (i.e., r_u = 1.0), N_{liq}	101	8

Behavior of UK Specimen 1

As seen in Figures 3 and 4 (tested with a cyclic stress ratio of 0.135), axial strain starts to increase at a pore pressure ratio of around 0.9 (equivalent to 90 cycles). At this point, the specimen has been softened by cyclic loading. However, axial strains remain relatively low (less than 2%) throughout the test.

Plots of hysteresis stress-strain loops measured over the duration of the test are shown in Figure 5. The pore pressure ratio plot (Figure 4) indicates that for this lower cyclic stress ratio, the rate of accumulation of excess pore pressure is relatively constant. The hysteresis loops shown in Figure 5 reveal that the loops are relatively steep during the early cycles of the test, so the specimen is behaving in a relatively stiff manner. As the test progresses, the loops flatten as the specimen softens. However, at larger strains near the end of the test, the loops begin to exhibit a banana-like shape, which is indicative of strain hardening. Within a given cycle of loading, the specimen is loaded with a stress, so it strains. At smaller strains within the cycle, the specimen acts like a contractive material, so the slope of the stress-strain curve is relatively flat. But as strain increases within the cycle, the behavior transitions from contractive to dilative. Once in the dilative portion of the cycle, the stiffness of the specimen begins to increase, which gives the banana-like shape to the hysteresis loop. This type of behavior tends to limit the amount of axial strain in the specimen to a relatively low level, and is often referred to as "limited liquefaction" or "cyclic mobility."[3]
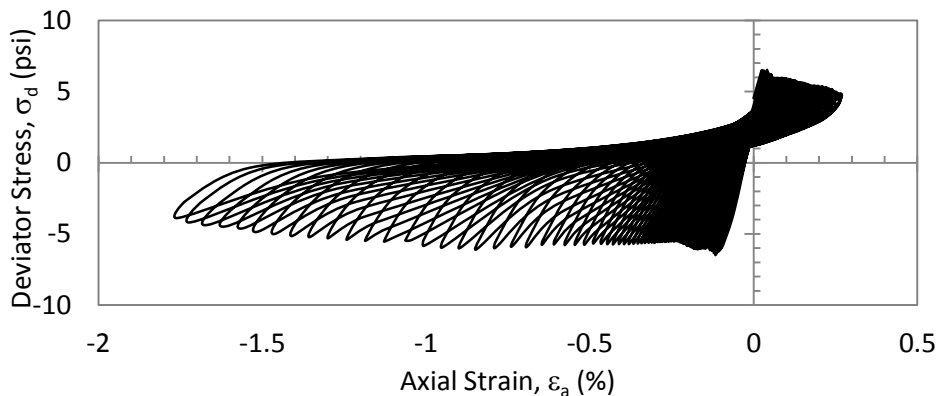
Figure 3. Axial Strain vs. No. of Cycles - UK Specimen 1 (CSR = 0.135, N_{liq} = 101)
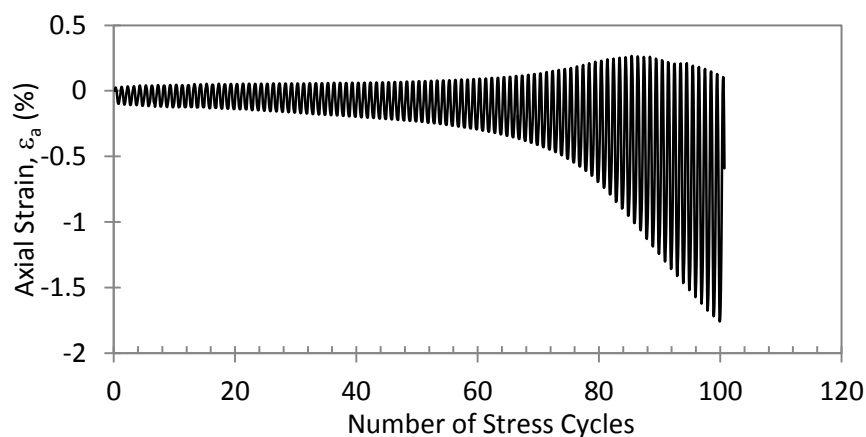
Figure 4. Pore Pressure Ratio vs. Stress Cycle Ratio - UK Specimen 1 (CSR = 0.135, N_{liq} = 101)
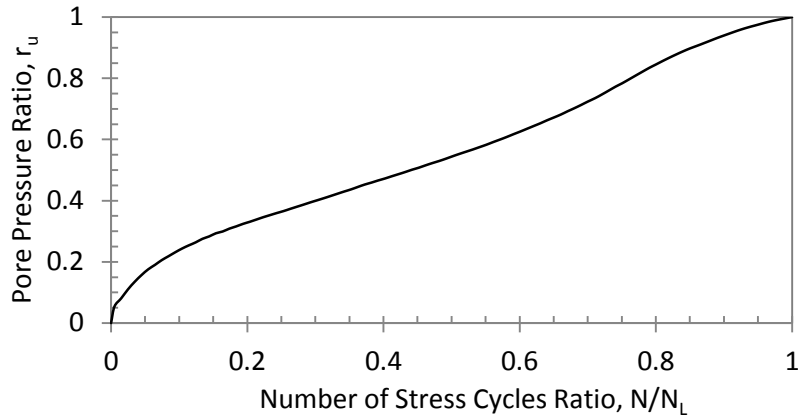
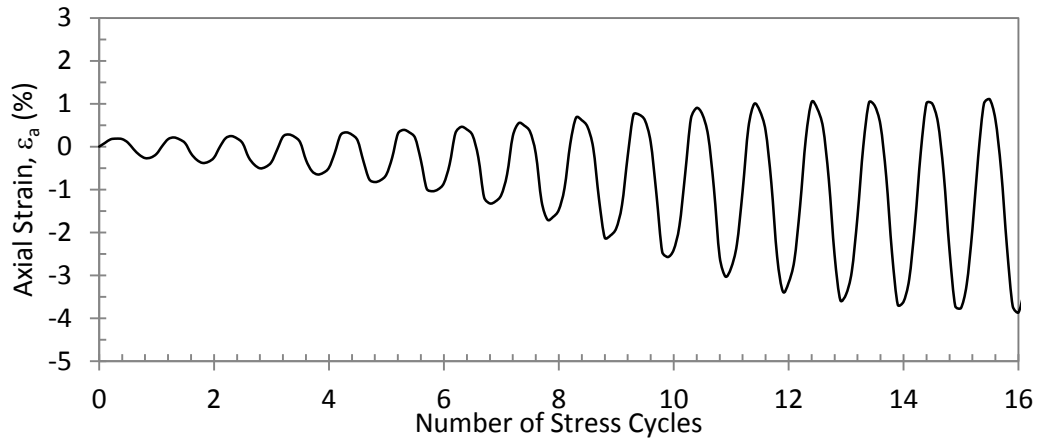**Figure 5. Stress-Strain History - UK Specimen 1 (CSR = 0.135, N_{liq} = 101)**


Behavior of UK Specimen 2

As seen in Figures 6 and 7 for Specimen 2 (tested with a cyclic stress ratio of 0.269), the deviator stress and axial strain remain relatively constant until a pore pressure ratio of 1.0 is achieved at around 8 cycles. Once the pore pressure ratio reaches unity, axial strain begins to increase at an accelerated rate. For this test with the larger cyclic stress ratio, axial strains after liquefaction are higher than with the lower cyclic stress ratio. When inspecting the plot of pore pressure ratio versus normalized cycles to liquefaction (Figure 7), it is also apparent that at the higher cyclic stress ratio, pore pressure accumulates faster for the first few cycles. The hysteresis loops derived from testing of Specimen 2 (Figure 8) resemble the hysteresis loops derived from testing of Specimen 1. The banana-like shape of the loops also indicates that at larger strains, the specimen behavior transitions from contractive to dilative.
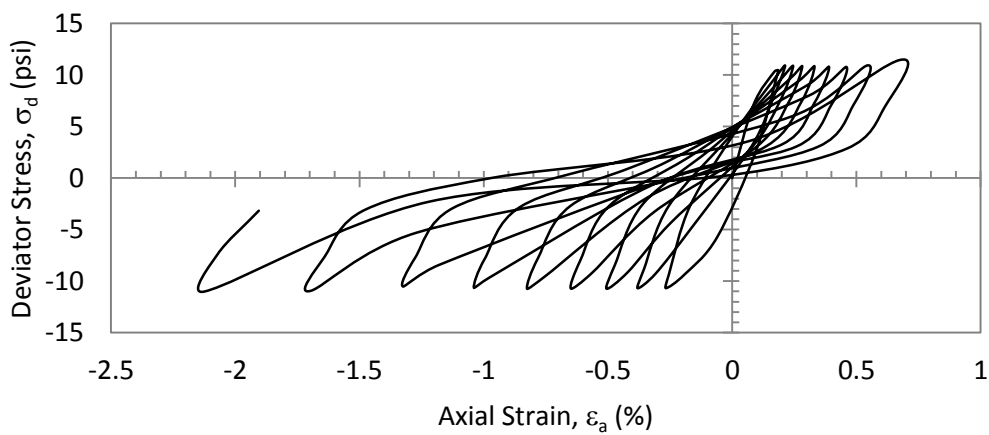
Figure 6. Axial Strain vs. No. of Cycles - UK Specimen 2 (CSR = 0.269, N_{liq} = 8)

Figure 7. Pore Pressure Ratio vs. Stress Cycle Ratio - UK Specimen 2 (CSR = 0.269, N_{liq} = 8)
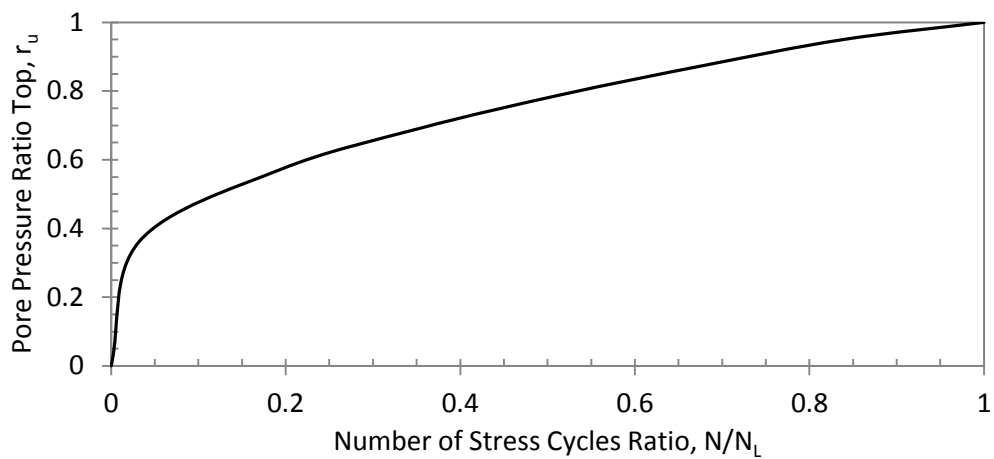
Figure 8. Stress-Strain History - UK Specimen 2 (CSR = 0.269, N_{liq} = 8)

Comparison of UK and OSU Data

AEP provided Stantec with an existing report prepared by Ohio State University (OSU) for the fly ash liquefaction potential at Amos Fossil Plant[4]. OSU prepared ten specimens from the Amos fly ash by pluviating the material in water to a range of initial densities. The specimens were then tested at confining stresses ranging from 10 to 40 psi (69 to 276 kPa) and cyclic stress ratios (CSR) ranging from 0.1 to 0.4 as shown in Table 3. After testing, OSU reviewed the data and number of cycles to liquefaction (N_{liq}) was defined as the first occurrence of either: (1) axial stress less than 95% of the programmed loading, or (2) a pore pressure ratio of unity[4].

Figure 9 presents the number of cycles required for each sample to liquefy using the OSU liquefaction criteria. Typically, it is expected that the number of cycles of liquefaction increases as the confining pressure increases (all else equal). However, data would indicate that the samples tested at 10 psi (69 kPa) are more resistant to liquefaction than those tested at 20 psi (138 kPa) and 40 psi (276 kPa). This could be correct if samples at 10 psi are dilative while samples at 20 psi and 40 psi are more contractive. The results for the 20 psi generally provide a lower bound for the available data.

To supplement the OSU data, Stantec and the University of Kentucky (UK) performed two additional cyclic triaxial tests on the Amos fly ash as described previously. UK tested the samples with CSR values of 0.135 and 0.269 and both samples had a 20 psi (138 kPa) confining pressure applied. Using the same definition of liquefaction as OSU, Stantec defined the number of cycles to liquefaction for the UK data, and the results are included in Figure 9 along with the OSU results. In order to compare the OSU and UK data, the number of cycles to liquefaction for UK Specimen 1 was reduced from 101 cycles (based on a pore pressure ratio of 1.0) to 72 (based on the axial stress criterion defined by OSU), while number of cycles to liquefaction for UK Specimen 2 was left unchanged at 8 based on the pore pressure ratio criterion of 1.0.

Figure 9 presents the number of cycles required for each sample to liquefy using the OSU liquefaction criteria applied to both the OSU and UK data. The UK data provided additional information in the 0.1-0.2 CSR range which was previously undefined by the OSU data. The UK data reaffirms the trend shown by the OSU data for a confining pressure of 20 psi (138 kPa). Therefore, the 20 psi (138 kPa) cyclic triaxial data was used to estimate the cyclic resistance ratio of the Amos ash.

Table 3. OSU Cyclic Triaxial Laboratory Data for Amos Fly Ash[4]

Test No.	CSR	Confining Stress, psi (kPa)	Dry Unit Weight, pcf (kN/m^3)	Cycles to Liquefaction (N$_{liq}$)
1	0.1	20 (138)	101.1 (15.9)	500[a]
2	0.2	20 (138)	97.6 (15.3)	9
3	0.3	20 (138)	94.9 (14.9)	3
4	0.4	20 (138)	94.1 (14.8)	2
5	0.1	20 (138)	96.9 (15.2)	500[a]
6	0.2	20 (138)	100.4 (15.8)	8
7	0.3	20 (138)	97.0 (15.2)	3
8	0.3	10 (69)	101.0 (15.9)	7
9	0.2	10 (69)	111.0 (17.5)	76
10	0.3	40 (276)	102.1 (16.1)	6

[a] Samples that did not liquefy were tested for 500 cycles before ending the test.

Figure 9. Combined Results from the OSU and UK Cyclic Triaxial Testing of the Amos Fly Ash Specimens
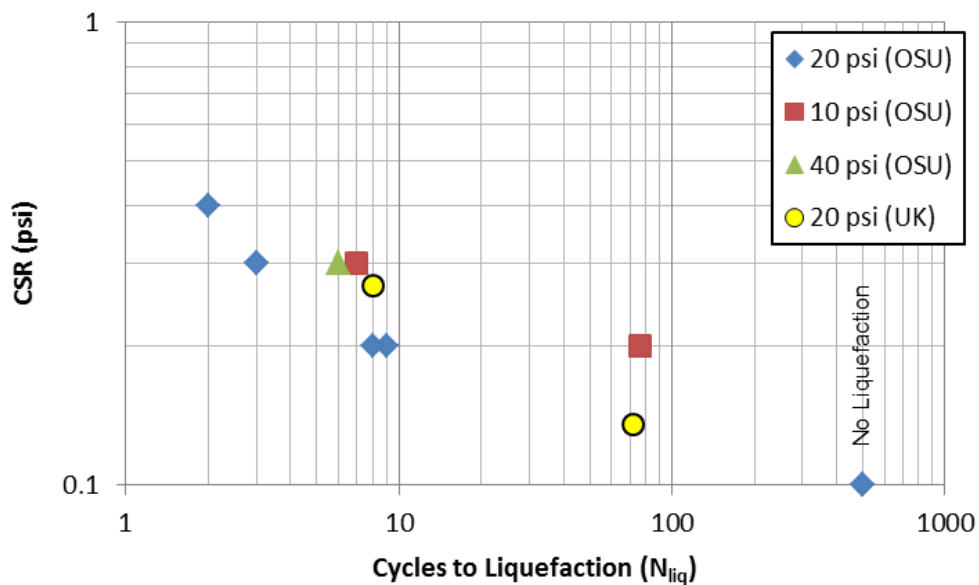
LIQUEFACTION POTENTIAL

For samples of similar initial density, relationships between the imposed cyclic stress and the number of cycles to liquefaction failure are observed. These relationships define the cyclic resistance ratio (CRR) for the material, which can then be compared against the cyclic stress ratio (CSR) imposed by any design earthquake of interest.

Methodology outlined by Idriss and Boulanger[3] was used to adjust the laboratory CRR to an appropriate field CRR for the site conditions and design earthquake (geometry, in situ stresses, earthquake magnitude, etc.). The primary benefit of this approach is the use of site-specific materials and development of a CRR relationship for the specific material. This is particularly relevant for fly ash, as many of the published relationships and correlations are based on testing of sands, and may not be as appropriate for fly ash. The primary drawback of such a method is that fly ash samples must be reconstituted in the lab and may not be representative of the undisturbed, in situ materials.

The approach used site-specific ground response analysis to estimate the load (CSR) imposed by the design earthquake. Because the cyclic triaxial results are not specific to a particular location within the fly ash pond, three generic soil columns were derived for the site and were used in ground response analysis and to estimate the CSR. Shear wave velocities for the sluiced ash was based on seismic CPT testing performed during the geotechnical exploration while shear wave velocities for other soils were estimated based on published correlations that are typically related to effective stress and soil type.

The methodologies outlined above were applied for both the M7.5 and M5.5 design earthquakes, with peak horizontal accelerations at the base of soil (top of soft rock) of 0.034 g and 0.043 g, respectively. Considering the CRR derived from laboratory testing of Amos fly ash, the M7.5 event controlled for all three generic profiles. The larger magnitude event controlled because its longer duration (more cycles of shaking) results in a lower CRR compared to the shorter duration, smaller magnitude event. The CSR for the M7.5 event was higher than that of the M5.5 event for two of the three profiles, but the greater difference in CRR was such that the resulting factor of safety (FS_{liq}) was lower for the M7.5 event for all three profiles. For both events and all three profiles, FS_{liq} was shown to be above the typical threshold value of 1.4 (values less than 1.1 indicate fully liquefied conditions, while values of 1.1 to 1.4 are partially liquefied), meaning that no liquefaction would be anticipated.

SUMMARY AND CONCLUSIONS

Stantec was contracted by American Electric Power (AEP) to provide permitting and design services for the closure of the Amos Plant Fly Ash Pond Complex. The closure must be designed to meet certain stability criteria, including seismic loading cases. Therefore, an evaluation of the liquefaction potential of the sluiced fly ash was performed. Primary aspects of the evaluation and significant findings are as follows:

- Sluiced fly ash is often deposited in a soft/loose state, such that sample disturbance can make it difficult to obtain representative samples for laboratory testing. Therefore, to supplement the data derived from laboratory testing on Shelby tube specimens, Stantec tested reconstituted specimens that were prepared using water-pluviation. Preparation was consistent with methods developed by AEP and Ohio State University (OSU) on previous fly ash projects.
- The stress-strain and pore pressure responses during CU triaxial testing of the water-pluviated specimens was consistent with the behavior of undisturbed

Shelby tube specimens tested at similar confining stresses. At lower confining stresses the specimens exhibited dilative behavior, including little to no drop in post-peak strength and generation of negative pore pressures during shearing. At a higher confining stress which represented only the deepest ash deposits within the central valley of the pond footprint, the behavior transitioned towards contractive, with a small drop in post-peak strength. However, the pore pressure response was still negative.

- Cyclic triaxial test results indicate that the fly ash exhibits cyclic mobility, which is accompanied by dilative behavior and relatively small strains. This is in contrast to flow failure, and indicates that in the event of liquefaction, the material may lose significant strength, but strains may be limited.

- Using the same definition of liquefaction as OSU, Stantec estimated the number of cycles to liquefaction for the UK data. The UK data provided additional information in a targeted range of CSR which was previously undefined by the OSU data. The UK data reaffirms the CRR trend shown by the OSU data for tests performed at the same confining pressure.

- Site-specific ground response analysis was performed to estimate the load (CSR) imposed by M7.5 and M5.5 design earthquakes. For both events, the factor of safety against liquefaction was shown to be above the typical threshold value of 1.4, meaning that no liquefaction would be anticipated.

REFERENCES

[1] Stantec Consulting Services Inc. (2012). "Phase 2 Geotechnical Data Report, Fly Ash Dam Complex (Revision 2)" Prepared for American Electric Power, October.

[2] The Ohio State University (OSU) (2006). "Consolidation Characteristics of Class F Fly Ash." Prepared for American Electric Power, September.

[3] Idriss, I.M. and Boulanger, R.W. (2008). "Soil Liquefaction During Earthquakes." Monograph, Earthquake Engineering Research Institute, Oakland, California.

[4] The Ohio State University (OSU) (2012). "Evaluation of Liquefaction Potential of Impounded Fly Ash: John E. Amos Power Plant. Prepared for American Electric Power, June.