Embankment Load Tests on an Active Ash Basin

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ABSTRACT

Two large-scale test fills were constructed on an active coal ash pond to gauge the response (settlement, lateral deformation, and pore pressure generation/dissipation) of the basin deposits to embankment loadings. The field tests were undertaken to support the design of a vertical expansion to the ash basin, wherein the existing containment elevation will be raised 60 ft (18 m). Site constraints will require constructing a 72-ft (22-m) tall embankment over fly ash and bottom ash deposits that are over 100 ft (30 m) deep. The test embankments were constructed in an area of the basin underlain by about 35 ft (11 m) of saturated fly ash. The footprint of one test fill was treated with vibrocompaction, wherein a pipe pile was vibrated to full depth and retracted on spacings of three diameters. The test embankments were constructed of transported bottom ash and reached average heights of 20 and 23 ft (6 and 7 m). Settlements up to 0.64 ft (0.20 m) and lateral deformations up to 0.24 ft (0.07 m) were measured. In situ vibrocompaction was effective in reducing the compressibility of the ash deposits. Significantly, pore water pressures generated by the embankment loading dissipated rapidly (within a few days) due to the permeability of the ash. The embankment tests conclusively demonstrated that the proposed construction is feasible.

INTRODUCTION

The E. W. Brown Generating Station is owned and operated by Kentucky Utilities (KU), a subsidiary of E.ON U.S. The facility is located adjacent to Dix Dam and Herrington Lake near Burgin, Kentucky, approximately 30 miles southwest of Lexington. Since the mid-1960s, coal combustion products (mostly fly ash with some bottom ash) from the plant have been sluiced to an adjacent ash treatment basin (Figure 1). The basin, which has been expanded twice, will be filled to capacity in 2010 at current production rates. A third expansion is currently being designed to store future ash and gypsum production.
The ash pond was raised around 1973, using an engineered fill placed over both the upstream and downstream faces of the original dam. The dam was raised again in 1989 to provide additional storage capacity. Designed by Fuller, Mossbarger, Scott and May Engineers (FMSM), the 1989 expansion involved a downstream overlay of the existing dam, with internal drainage to control seepage. To keep the elevated pond from encroaching on an adjacent railroad line, a containment dike was built along the north end of the basin (Figure 1). The dike, which bears on existing ash deposits, was built on a stone foundation layer reinforced with geogrid. Prefabricated vertical drains were installed to speed drainage from the underlying ash during construction.

A wet scrubber with forced oxidation, currently being added to the E. W. Brown Generating Station, is expected to begin operating in the summer of 2009. Lacking a viable alternative, gypsum produced by the scrubber unit will be stored at the plant site. KU has few options for developing new storage capacity, given numerous site constraints and the plant’s location immediately adjacent to a recreation lake. Therefore, FMSM has developed plans for expanded storage capacity within the footprint of the existing ash basin. Using gypsum as an embankment material, a new containment dike will be built around the ash pond. The gypsum dike will be built upstream of the existing dam and bear on ash deposits over much of its length. The design includes a flexible...
membrane liner that will be placed over the surface of the existing ash deposits; by cutting off the infiltration of water, high piezometric levels in the basin and seepage pressures on the dam will be reduced. A key challenge in this approach is the ability to construct embankments over soft, dewatered fly ash. In the deepest portion of the basin, the new gypsum dike will be 72 ft (22 m) tall and will rest on over 100 ft (30 m) of ash deposits.

FIELD EMBANKMENT TESTS

To support the analysis and design of structures planned for the basin, a field testing program was undertaken to characterize the engineering properties of the ash deposits. In addition to geotechnical explorations and other test activities, two large embankments were constructed on the dewatered surface of the ash basin. Testing was planned for early 2006, but work was accelerated when an unscheduled, partial shutdown of the plant allowed for a temporary drawdown of the ash pond during October and November of 2005. Installation of instruments and construction of the two test fills was completed in less than four weeks.

The aerial photograph in Figure 1 was taken in the summer of 2004, when water levels were higher than during the 2005 field tests. Since completion of the 1989 expansion, the plant has deposited bottom ash along the inside of the dike on the north end of the basin, forming a “bottom ash beach” that extends above the water surface for several hundred feet into the basin (Figure 1). This part of the basin was used as a staging area for the test activities, and as a starting point for two access roads that were built out over the dewatered ash. At the end of the western access road, two test embankments were constructed in the locations shown in Figure 1. Geotechnical explorations and other related tests were conducted at locations along the eastern access road. The coarse bottom ash deposit at the north end of the basin was also excavated for use as fill in building the access roads and test embankments.

The two embankment tests were built directly on about 35 ft (11 m) of saturated fly ash. Prior to construction of the second test fill, the ash was treated with vibrocompaction. Settlements, lateral deformations, and pore water pressures were measured under both embankment tests. In addition to providing quantitative data on the mechanical behavior of the ash deposits, these nearly full-scale embankment tests confirmed that the proposed dike will be stable. Of equal significance, the tests demonstrated how geosynthetics can be used to build the proposed structures over the soft ash deposits.

CONSTRUCTION OVER SATURATED ASH

Construction over saturated ash can be successfully executed by proceeding in a well planned sequence and allowing time at each step for pore pressure dissipation and consolidation of the ash. During this test project, the water level in the basin was lowered to expose the ash surface during the two weeks prior to the start of construction. In most areas, the ash was too wet and soft to step on without sinking throughout the test period. However, as the phreatic surface dropped and a capillary
fringe formed near the surface, the shear strength of the ash increased to a point where a person could walk on the exposed surface at higher locations. Accessibility in different areas of the basin was determined by the local depth to the ground water table.

Heavy construction equipment was advanced across the basin on access roads built with low ground-pressure dozers. Starting from the bottom ash beach, a 600-ft (180-m) long access road was constructed to an area between the test fill locations, where a 50 ft by 250 ft (15 m by 75 m) working pad was built. The access embankments were constructed of stockpiled bottom ash (containing about 20% fines) and geosynthetic materials. First, a nonwoven, needle-punched filter fabric was rolled across the exposed ash surface (Figure 2a). Using a small, low ground-pressure dozer (John Deere Model 650J-LGP), 1 to 2 ft (0.3 to 0.6 m) of bottom ash was then pushed out across the fabric (Figure 2b). Biaxial geogrid was rolled over the initial lift, transverse to the road centerline. Adjacent sections were overlapped at least 3 ft (1 m) and tied together with plastic zip ties. Additional fill was then placed (Figure 2c) to raise the embankments to a total height between 3 and 5 ft (0.9 and 1.5 m), depending on location. Filter fabric and geogrid were not used under the base of the two test fills.

![Figure 2. Construction of access road to the embankment tests:](image)

(a) filter fabric rolled over the soft ash, (b) placing 1 to 2 ft (0.3 to 0.6 m) of fill on top of the fabric, and (c) biaxial geogrid and additional fill.

Construction of the embankments induced significant excess pore water pressures in the underlying saturated ash, reducing the effective stresses and weakening the deposits. Localized failures were frequent during construction. Small mud waves developed at the toe of the fill and, at one location, a small bearing capacity failure occurred. In other areas, after the low ground-pressure dozer made several passes pushing the initial lift of material over the filter fabric, the ground surface would pump up and down several centimeters under the weight of the dozer. For perhaps 15 to 30 minutes after the dozer moved away, the ground surface would roll up and down as workmen walked over the area. The filter fabric appeared to contain the underlying,
liquefied fly ash and prevent the rapid venting of excess water pressures to the surface. In another location where no filter fabric was used, a boil of liquefied fly ash pushed up through about 2 ft (0.5 m) of fill (Figure 3). When the boil was excavated with a shovel, a stream of pressurized, fluid ash flowed to the surface.

Attention to safety is advised when working on saturated ash deposits. Here, liquefaction of the ash did not pose a risk to construction personnel because the work was confined to low embankments on level ground, thus limiting the size and extent of potential ground failures. Where pumping or ash boils were observed, construction activity was diverted to another area until the excess pore pressures dissipated. Greater caution is warranted where equipment might roll over, when working on or near a slope, or where a deeper bearing capacity failure could develop.

The access roads became more stable with time, as the pressurized pore water drained, the effective stresses increased, and the ash consolidated under the weight of the embankments. In some areas, consolidation water being “squeezed” out of the ash deposits was visible around the periphery of the embankments (Figure 4). The completed access roads were repeatedly traversed by construction equipment, including heavily loaded off-road trucks. A crawler-mounted crane used in constructing the embankment tests was successfully moved into position without incident (Figure 5). The geogrid reinforcement appears to have been key in supporting these heavy loads on the soft ash deposits. The filter fabric, included to separate the fill from the underlying fly ash, was perhaps unnecessary for these temporary construction roads.

The 2005 field tests were conducted shortly after the pond surface was drawn down, with the phreatic surface within a couple of feet of the ground surface. Given longer to
drain, the water table will gradually drop to a significantly lower elevation. Suction pore pressures above the water table, resulting from capillary effects in the pores of the ash, will significantly improve the strength and stiffness of the shallow material. Hence, conditions during construction of the basin expansion project should be better than encountered during the 2005 field tests.

**VIBROCOMPACTION**

The planned basin project will involve constructing large embankments over the existing ash deposits; in-place vibratory densification was considered an option for improving the properties of these sediments. However, it was unclear prior to the test if fly ash could be densified with a vibrating probe. Some thought vibrocompaction would be effective, while others believed the low hydraulic conductivity of the ash would hinder densification. The material under the west test fill was thus vibrocompacted to assess the potential effectiveness of this technology in fly ash.

Vibrocompaction was achieved using a vibratory pile hammer (Hercules Machinery Corp. Model 13H), operating at 1900 vpm (32 Hz), to drive a 2-ft (0.6 m) diameter, 40-ft (12-m) long steel pipe (Figure 6a). The pile was advanced to full depth in the fly ash (about 35 ft, or 11 m) and then withdrawn, working on a grid pattern (Figure 6b) with a 6-ft (1.8 m) center-to-center spacing (three pile diameters). Initial test probes induced surface depressions with a radius of about 3 ft (0.9 m), indicating that the probe spacing...
was appropriate. This conclusion was confirmed by data from a temporary pressure transducer installed in the ash beneath the crane. Excess pore pressures were recorded when the pile was driven 12 ft (3.6 m) away; much higher pressures were observed when the pile was advanced within 6 ft (1.8 m) of the transducer. Some settlement of the treated surface was apparent when the effort was completed, but was difficult to measure given the irregular ground surface (Figure 6b).

Data from the two test embankments, presented later, were compared to assess the effectiveness of the vibrocompaction effort. A direct comparison of the settlement data is complicated by differences in the fill heights and in the elevations of the settlement reference points. A simplified, one-dimensional analysis was used to quantify the compressibility of the untreated and treated ash, respectively, under the east and west test embankments. The ash deposits were assumed to be normally consolidated and the change in stress due to the embankment loads was computed using an elasticity solution. In the upper 30 ft (9 m), the one-dimensional compression index was found to be 0.28 in the untreated ash, and 0.14 in the vibrocompacted ash. Hence, vibrocompaction reduced the expected settlements by about half.

EMBANKMENT LOAD TESTS

Two embankment tests were built: the east fill was constructed directly on undisturbed ash deposits, while the ash under the west fill was vibrocompacted. In both locations, the surface of the dewatered sediments was approximately level, the ash was about 35 ft (11 m) deep, and the deposits were mostly fly ash. The base of each embankment measured, nominally, 80 ft wide by 120 ft long (24 by 36 m); the instrumented cross sections were at the centerline and oriented across the width of the fills (Figure 7). About 3,500 cubic yards of uncompacted bottom ash was used to construct each fill, with side slopes at the angle of repose (measured at 36º or 1.4H:1V).

Photographs of the construction are shown in Figure 8. The east test fill (Figure 8b) was built to a height of about 23 ft (7 m, average height across the top of the completed fill) in two phases. The embankment was first built to a height of 12 ft (4 m) in three lifts, using a low ground-pressure dozer. As a precaution against a potential slope failure, the

![Figure 7. Schematic cross section of test embankments and instrumentation.](image-url)
dozer was not permitted on the fill above 12 ft (4 m). The remaining fill was placed using a crane and clamshell (Figure 8a). The fill was left in place for four days, and then reused in the construction of the west fill. To improve the stability of the crane, 5 ft (1.5 m) of material was placed in the west fill prior to vibrocompaction and the installation of instruments. Based on the proven stability of the east test fill, the west fill was built to a full height of about 20 ft (6 m) using two small dozers (Figure 8c).

The response of the ash deposits beneath each test fill was measured with an array of instrumentation installed along a central section (Figure 7). Consolidation of the ash was measured under the center of each fill, using multiple magnetic reference points anchored in two boreholes. The elevation of each reference point was measured using an electronic sensor lowered into a central casing, with the casing extended upward as the fill was placed. Due to difficulties encountered during installation, the settlement array under the east fill did not extend to the bottom of the basin deposits (Figure 7). Horizontal movements at the toe of both fill slopes were monitored using an inclinometer probe lowered down cased boreholes. Six vibrating wire piezometers were installed under each test fill; data were recorded continuously using an electronic data acquisition system. The elevation and profile of the test fills were periodically measured using a survey-grade GPS unit tied to a site benchmark.
The measured compression of the ash under the two embankments is plotted in Figure 9. On the east, settlements occurred rapidly during fill placement in the first 3.4 days (Figure 9a). Additional compression occurred over the next four days, when there was no construction activity. The total vertical compression, measured between sensors that were initially 22.5 ft (6.86 m) apart, was 0.64 ft (0.20 m). When the fill was removed after eight days, measurable rebound occurred.

No data was obtained at the start of the west fill (Figure 9b), because the instruments were installed after vibrocompaction was performed through the first 5 ft (1.5 m) of fill. In addition, the first few sets of readings were erratic, apparently due to collapse of the borehole around the sensors. Starting on day 5, the measurements showed rapid compression as fill was placed, with smaller settlements occurring for about three days after the fill was completed. The total vertical compression was 0.32 ft (0.98 m), measured between sensors that were initially 40.2 ft (12.3 m) apart.

Comparing the measured settlements is made difficult by differences in fill height and measurement interval. As reported above, the measured settlements correspond to a one-dimensional compression index of 0.28 in the untreated ash, and 0.14 in the vibrocompacted ash.

The maximum vertical compression measured between sensor elevations is plotted in Figure 10. Peak vertical strains of 3.9% and 1.1% were measured under the east and west test fills, respectively. For comparison, the lateral deformations measured with the inclinometer probe at the outside toes of both fills are plotted in a similar fashion in Figure 11. The maximum lateral movement was 0.24 ft (0.07 m), measured under the east fill. The data in Figures 10 and 11 exhibit remarkably similar trends: vertical compression and lateral deformation in the ash are maximum at about one-third the
depth of the deposits under the embankment. These trends can be attributed to the three-dimensional stress distribution, shearing-induced volumetric changes, and decreasing compressibility at greater depth in the ash under the embankments.

PORE WATER PRESSURE RESPONSE

Excess pore water pressures result from increased mean stress, as well as the tendency for volumetric compression when the loose deposits are subjected to shear stress. The pore pressure response can be conveniently viewed in terms of the nondimensional, excess pore pressure ratio ($r_u$):

$$r_u = \frac{u - u_0}{\sigma'_{vo}}$$

- $u =$ measured pore water pressure
- $u_0 =$ hydrostatic pore water pressure
- $\sigma'_{vo} =$ initial vertical effective stress

A value of $r_u = 1.0$ would correspond to a quick condition, or zero effective stress.

Pore water pressures were monitored at six locations under each test fill (Figure 7). The measured excess pore pressure ratios ($r_u$) are plotted in Figure 12. In both tests, pore water pressures were observed to rise quickly as fill was placed in the embankments. When construction activities ceased at the end of the work day, the pore pressures at all locations were observed to dissipate overnight.

The east test fill was built over four days, and then allowed to stand undisturbed for four days. The maximum recorded excess pressure corresponded to $r_u = 0.36$ (Figure 12a). However, the excess pore pressures had fully dissipated within two to four days.
following completion of the fill. (Small bumps in the data records at five to seven days may be related to changing barometric pressures at the site). After the eighth day, additional excess pressures were generated as construction equipment worked to excavate the east test fill.

Similar trends were observed in the data from the west test fill (Figure 12b). Five feet (1.5 m) of the west fill was placed prior to installing the piezometers, which could be installed only after the vibrocompaction work was complete. The next 4 ft (1.2 m) was placed just prior to a four-day break in construction; the fill was completed after the fifth day. Because the west fill was built in stages over a longer period of time, the maximum excess pressure that built up \( (\frac{r_u}{\text{Initial Vertical Effective Stress}}) = 0.22 \) was less than observed under the east test fill. This demonstrates that improved stability can be realized from building a fill in lifts, with time allowed between lifts for dissipation of pore pressures.

Pore pressure dissipation rates were much faster than anticipated. Cone penetration profiles showed that the sediments are not uniform, but exhibit significant horizontal bedding. The lateral permeability of the basin deposits is probably significantly higher than the permeability for vertical flow, given the tendency to deposit thin layers of ash with differing gradations. Three-dimensional drainage effects were thus significant in the field tests, although the much larger footprint of the planned embankments will slow
lateral drainage. However, given the much greater volume of fill required, construction of the gypsum dikes will be accomplished more slowly than occurred in the field tests. The expected rates of construction should be sufficiently gradual to limit the hazardous build-up of excess pore pressures in the ash sediments, and thus assure the stability of the dike during construction.

CONCLUSIONS

Extensive field testing activities were undertaken to support the expansion of an ash treatment basin at the E. W. Brown Generating Station. The planned project will involve the construction of a 72-ft (22-m) tall gypsum embankment over existing coal ash sediment, which exceeds 100 ft (30 m) in thickness at some locations. The objective of the field tests, which included geotechnical explorations in addition to two large-scale test embankments, was to characterize the mechanical response of the existing ash sediments. The field testing activities were accomplished on the dewatered basin during a temporary drawdown of the pond, made possible by an unplanned, partial shutdown of the plant in the fall of 2005.

Construction of the test embankments provided an opportunity to evaluate the planned approach to constructing across the very soft, dewatered ash surface. With a small, low ground-pressure dozer, a few feet of fill could be advanced over a surface that was difficult to walk across. Temporary haul roads, which successfully supported loaded off-road trucks and a large crane, were built over the basin using a single layer of biaxial geogrid. The saturated ash sometimes liquefied when the small dozer was working on the first 2-ft (0.6-m) lift of material; ash boils pushed through to the surface where filter fabric was not used under the fill. Given sufficient time for the localized excess pore pressures to dissipate, construction could safely progress across the dewatered basin. Moreover, the phreatic surface was near the ground surface during the field tests. Lowering the ground water table will set up increased, beneficial suction pore pressures that will improve mobility across the basin during the planned construction project.

During the field tests, instrumentation measured vertical compression up to 0.64 ft (0.20 m) and lateral deformations of 0.24 ft (0.07 m) in the ash deposits beneath the test embankments. The measured vertical compression under the center of the fills and the outward lateral displacement in the ash at the toe of the slopes showed similar patterns, with maximum deformations occurring at about one-third the depth of the ash below the ground surface.

Significantly, the piezometer data indicated rapid dissipation of excess pore water pressures. Both test embankments were built in areas of the basin containing mostly fly ash, where excess pressures dissipated within four days or less. This time period may be longer for different loading conditions or locations within the basin, as the rate of dissipation is related to the magnitude of the excess pressure, hydrostatic pressures, drainage lengths, layering within the deposit, localized ash gradation, etc. However, the overall conclusion is that the ash basin deposits can be expected to dissipate excess pore water pressures within days.
Under one of the test fills, the ash was improved using a vibratory pile hammer to drive and retract a pipe pile on a grid spacing of three pile diameters. By comparing the one-dimensional compressibility for the treated and untreated ash, vibrocompaction was shown to have been effective in reducing the measured settlements. However, because the pore pressure response was found to be rapid, the proposed embankments can be built without the need for in situ densification of the existing sediments.

The completed field tests represent a significant investment by KU and E.ON U.S. Savings in the final project will result from less conservatism in the design, stemming from better confidence in the material properties and an improved understanding of the behavior of the ash deposits. This has, in turn, resulted in the specification of less geosynthetics and less select fill for the subgrade foundation of the new lined basin. The reduced cost of construction is expected to generate savings approaching 50 times the cost of the field test. The documented test experience should also reduce the uncertainties for potential contractors and, hopefully, result in lower construction bids.

In recognition of these accomplishments, the field performance tests in the ash basin at the E. W. Brown Generating Station received the 2007 Grand Conceptor Award for Engineering Excellence, awarded by the American Council of Engineering Companies of Kentucky.