Cement-Bentonite Slurry Walls for Seismic Containment of the Kingston Coal Ash Landfill

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Abstract

TVA recently capped a coal ash landfill at the Kingston power plant. Cement-bentonite slurry walls were built around the two-mile (three-kilometer) circumference of the facility. The landfill contains roughly 18 million cubic yards (14 million cubic meters) of coal fly ash, including material recovered after the 2008 dike failure at the site. Constructed on the footprint of the failed facility, the new landfill must survive a 2,500-year seismic event. The subsurface, perimeter retaining wall system was designed to stabilize the landfill slopes and contain the stored ash in an earthquake that triggers soil liquefaction. The wall layout consists of evenly spaced shear walls, oriented perpendicular to the landfill perimeter, plus circumferential walls in critical segments. Stantec designed the walls using complex, 2D dynamic computer simulations and 3D structural stress analyses. Deep mixing methods were assumed in the bid package, but prospective contractors were encouraged to propose alternate construction technologies. The winning contractor (Geo-Con, now Geo-Solutions) successfully built the walls using cement-bentonite, slurry trench methods. Over 11 miles (18 kilometers) of wall were constructed, requiring over 520,000 cubic yards (400,000 cubic meters) of slurry. Challenges during construction included characterization of achieved wall strength, mitigation of soil inclusions in the slurry walls, treatment of cold construction joints, soft working conditions on top of the old ash deposits, and collapse of trenches in areas with high groundwater levels.

INTRODUCTION

In December 2008, the largest coal ash spill ever in the United States occurred at the Tennessee Valley Authority (TVA) power station in Kingston, Tennessee. When the dikes containing a wet dredge cell failed, over five million cubic yards (four million cubic meters) of coal ash slurry flowed out from the site and filled the adjacent Emory River channel. TVA committed over $1 billion for the response, clean up, and site restoration. The recovery program was managed under EPA’s CERCLA regulatory framework. Much of the released ash was dredged and shipped by
rail for off-site disposal. Approximately 18 million cubic yards (14 million cubic meters) of coal fly ash, including three million cubic yards (two million cubic meters) recovered from the failure, were permanently stored in a capped landfill. The new ash landfill was built over the footprint of the failed cell and the adjacent ash pond.

The most difficult design challenge was how to contain the ash in the new landfill during a large earthquake. Tennessee regulations require new landfill facilities to withstand a 2,500-year seismic event. At Kingston, the design earthquake is a $M_w = 6.0$ local event (peak acceleration = 0.114 g at the site) or a $M_w = 7.6$ event in far western Tennessee (peak acceleration = 0.035 g). Engineering analyses showed that these earthquakes would liquefy both the saturated ash deposits that remain in the closed cell, and the 10 to 20 feet (3 to 6 meters) of alluvial sands below the site.

The solution was to construct a stabilized perimeter around the landfill. Subsurface walls, which are seated into bedrock, were designed to contain the liquefied ash during an earthquake. The walls were built using cement-bentonite slurry trench techniques. The walls were not designed to impede groundwater movement, and were built without a specification for permeability. Key features of the design and construction challenges are presented here. Additional details on these and other aspects of the Kingston project are provided in Bussey et al. (2012), Dotson et al. (2103), Rauch et al. (2013), and Rauch (2014).

**LAYOUT OF STABILIZED PERIMETER**

The completed ash landfill covers approximately 230 acres (93 hectares). The stabilized perimeter was divided into eight design segments, as indicated on the site plan in Figure 1. The distance around the landfill perimeter is 11,270 feet (2.1 miles or 3.4 kilometers).

The cement-bentonite walls (CB walls), depicted in Figure 2, are arranged to stabilize the landfill perimeter and contain the ash in an earthquake. Evenly-spaced shear walls, which are aligned with the perimeter cross section, transfer lateral pressures from the landfill to the underlying bedrock. The 4-foot (1.2-meter) thick walls vary from 50 to 100 feet (15 to 30 meters) in length and are spaced at 14.6 to 20.4 feet (4.45 to 6.22 meters) on-center, depending on the perimeter segment. Inboard perimeter walls distribute the outward pressures to the shear walls. Outboard perimeter walls prevent ash between the shear walls from entering the river. The walls are not exposed above grade, and the maximum wall depth is about 75 feet (23 meters).

The CB wall layout selected for each perimeter segment is indicated in Table 1. The design was tailored to provide additional stability in critical areas, especially in segments that border the Emory River and the adjacent public road. This was accomplished by varying the length and spacing of the shear walls, and by the inclusion or exclusion of inboard and outboard perimeter walls connecting the shear walls.

About 60,000 linear feet (18,000 meters) of wall were built, requiring roughly 520,000 cubic yards (400,000 cubic meters) of CB slurry. The project was completed in six work packages, which were sequenced so that construction could proceed on one segment while the design was finalized on the next.
Table 1. Summary of CB wall layout by perimeter segment (wall thickness = 4 feet).

<table>
<thead>
<tr>
<th>Segment</th>
<th>Perimeter Length (feet)</th>
<th>Inboard Wall?</th>
<th>Outboard Wall?</th>
<th>Stabilized Width (feet)</th>
<th>Shear Wall Spacing (feet)</th>
<th>Embedment in Rock* (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1,800</td>
<td>Yes</td>
<td>Yes</td>
<td>100</td>
<td>20.4</td>
<td>1.7 or 3.1</td>
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<tr>
<td>2</td>
<td>2,230</td>
<td>Yes</td>
<td>Yes</td>
<td>75</td>
<td>14.6</td>
<td>1.8</td>
</tr>
<tr>
<td>3</td>
<td>1,670</td>
<td>No</td>
<td>No</td>
<td>60</td>
<td>19.0</td>
<td>4.0</td>
</tr>
<tr>
<td>4</td>
<td>1,000</td>
<td>No</td>
<td>No</td>
<td>60</td>
<td>16.6</td>
<td>2.7</td>
</tr>
<tr>
<td>5</td>
<td>2,070</td>
<td>No</td>
<td>No</td>
<td>70</td>
<td>19.0</td>
<td>6.9</td>
</tr>
<tr>
<td>6</td>
<td>690</td>
<td>Yes</td>
<td>No</td>
<td>90</td>
<td>18.5</td>
<td>2.7 or 4.6</td>
</tr>
<tr>
<td>7</td>
<td>1,810</td>
<td>Yes</td>
<td>No</td>
<td>70</td>
<td>17.7</td>
<td>2.1</td>
</tr>
<tr>
<td>8</td>
<td>710</td>
<td>Yes</td>
<td>Yes</td>
<td>50</td>
<td>19.7</td>
<td>2.0</td>
</tr>
</tbody>
</table>

* In two segments, the required embedment varied with the encountered rock formation

**DESIGN ANALYSES**

A typical cross section through the landfill perimeter is shown in Figure 3. Limit-equilibrium, slope stability analyses were used in the initial design stages. Beneath the perimeter berm, the stabilized zone was modeled using strengths that represent a weighted average of the CB walls and the untreated soil. These 2D analyses were used to find the critical cross section in each design segment and to evaluate the post-liquefaction stability of the landfill slopes. However, slope stability analyses cannot model structural failure modes within the walls.

Dynamic, two-dimensional analyses (Rauch et al. 2013) were completed on the critical cross sections using FLAC. Six acceleration time histories, representing the two design earthquake scenarios, were applied to the base of the solution grid. Progressive liquefaction in various soil
zones was approximated using an empirical model available in FLAC. Predicted post-earthquake deformations of the perimeter berm were generally less than 0.5 feet (0.2 meters), with a maximum of 1.35 feet (0.41 meters). These displacements were considered acceptable. The appropriate stiffness for the stabilized zone in the dynamic 2D FLAC model was derived from a static FLAC3D analysis (Rauch et al. 2013). The lateral loads on the stabilized zone were recorded during the 2D dynamic simulations, and the maximum values were subsequently applied to the 3D model to check the structural integrity of the cement-bentonite walls.

Stresses within the walls were analyzed with the FLAC3D model (Figure 4). Where the shear walls intersect a perimeter wall, weaker zones were modeled in the analysis to represent the effect of cold joints (discussed later). The factor of safety (FOS) for each wall element was defined as the shear strength of the cement bentonite divided by the maximum shear stress computed in the 3D analysis. The performance was judged acceptable if 100% of the wall elements had a FOS > 1.0, and furthermore:

- for segments that border the river, 90% of the wall elements had a FOS > 1.5,
- for segments that border the public road, 90% of the wall elements had a FOS > 1.2, and
- for all other segments, 90% of the wall elements had a FOS > 1.1.

Segments 3 through 5 do not have perimeter walls, only shear walls (Table 1). The FLAC3D analyses showed that significant soil arching would develop at the inboard ends of the shear walls and, for the design wall spacing, the soils would not extrude between the shear walls. An inboard perimeter wall was not built in these non-critical segments. The outboard perimeter wall was required only in segments along the river, to fully contain the ash between the shear walls.

**CONSTRUCTION METHODOLOGY**

TVA used a best-value selection process to choose a contractor for the perimeter stabilization work. Bids were based on a preliminary design for Segment 1. The original design concept assumed the walls would be built using deep mixing, where overlapping columns of cement-stabilized soil are formed with paddles on a rotating, vertical bar. However, the bid documents were written to encourage proposals for alternative technologies.
TVA selected Geo-Con (now part of Geo-Solutions Inc.) as the winning bidder. Geo-Con proposed and successfully completed the project using cement-bentonite slurry walls. The walls were excavated with two Komatsu 1250, long-reach hydraulic excavators (Figure 5), each capable of digging to the maximum required depth of 75 feet (23 meters). The design specifications allowed for different wall thicknesses, and Geo-Con chose to build 4-foot (1.2-meter) thick walls using 48-inch (120-cm) wide excavator buckets. Ripper teeth were attached to the back of each bucket (Figure 5) to allow digging the required embedment into bedrock. Trench spoils were removed from the working platform, allowed to dry, and then placed in the ash landfill.

The cement-bentonite slurry was comprised of 20% to 25% ground blast furnace slag, 0.5% Portland cement, and 3% to 4% bentonite (each expressed as a percentage of total slurry weight). The slurry was mixed in an on-site grout plant and pumped via 6-inch (15-cm) lines to the trench excavation. The slurry supported the open trench during the excavation and then hardened in place to become the permanent backfill, forming the completed walls (Figure 6). The walls do not contain steel or other reinforcement.

Figure 4. FLAC3D model and predicted shear stresses within the walls.

Figure 5. Excavation equipment used to construct the cement-bentonite walls.
ROCK EMBEDMENT

If the design earthquake occurs, the landfill will exert an outward lateral pressure on the stabilized perimeter. These forces will be carried through the shear walls into the underlying bedrock. The minimum required rock embedment was calculated for each design segment (Table 1) based on the predicted lateral forces. Bedrock strengths were estimated for the rock type and rock mass quality. Completed excavations into rock were verified by depth soundings at intervals of 15 feet (4.6 meters) or less along each trench.

Most of the project site is underlain by the thinly-bedded, Conasauga Shale formation, which could be efficiently dug with the ripper teeth on the excavator buckets. The Rome Formation, with thicker shale and siltstone beds, was encountered under the western side of the site (all of Segments 7 and 8, and parts of Segments 1 and 6). This harder rock required much more effort to excavate, but provided adequate capacity with shallower embedment. Differing embedment depths (Table 1) were thus specified for the Conasauga and Rome formations. Field observations were used to verify which formation was encountered during the excavation of each trench.

SPECIFICATIONS FOR WALL STRENGTH

Soil-cement materials are typically brittle in unconfined compression, but exhibit strain softening and a residual strength when loaded under confinement. To avoid progressive failures, Filz et al. (2012) recommend using 80% of the peak strength for design. The maximum shear stress in a test specimen is half of the compressive strength, so the design assumed the shear strength of the CB walls was 0.5 x 80% = 40% of the unconfined compressive strength.

The design also recognized that significant variability would be realized in the strength of the CB walls. Appropriate strength specifications were developed from reliability analyses, using limit equilibrium slope stability and the Taylor series method (Duncan and Wright 2005). These analyses showed that acceptable performance would be achieved if the coefficient of variation (COV) for the wall strengths did not exceed about 30%. The strength of the cement-bentonite was assumed to follow a lognormal distribution.
The specifications for the wall strength (Table 2) evolved during the project, based on the quality control data collected during construction. Initially, the walls were required to achieve an average unconfined compressive strength of 200 psi (1,380 kPa) or greater in 56 days. At least 90% of the tested specimens had to achieve a compressive strength of 135 psi (930 kPa), corresponding to a COV of 30%. Test data acquired during construction of the first design segments showed that the CB mix was consistently achieving higher strengths to at least 12 weeks. Later segments were then designed for a higher wall strength. The specifications were changed to require an average compressive strength of 265 psi (1,830 kPa) or greater at 84 days, with at least 90% of the specimens required to exceed a strength of 175 psi (1,210 kPa).

The acquired test data were also used to assess strength gain in the CB walls over time. Between 28 and 84 days, the cement bentonite was found to exhibit (conservatively) a curing rate of 90 psi (620 kPa) per log cycle of time (in days). The strength specifications were then modified to the criteria depicted graphically in Figure 7. If the average strength plotted in the green zone above the line prior to 84 days, the constructed wall was accepted assuming the strength would reach 265 psi (1,830 kPa). This modification to the specifications, which was substantiated with project-specific test data, allowed the adoption of a higher design strength without changing the CB slurry mix or delaying the acceptance of completed walls.

<table>
<thead>
<tr>
<th>Design Segments</th>
<th>Curing Period for Strength Specification</th>
<th>Average Unconfined Compressive Strength</th>
<th>90% of Unconfined Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, 7, 8</td>
<td>56 days</td>
<td>≥ 200 psi</td>
<td>≥ 135 psi</td>
</tr>
<tr>
<td>3, 4, 5, 6</td>
<td>84 days</td>
<td>≥ 265 psi</td>
<td>≥ 175 psi</td>
</tr>
</tbody>
</table>

Figure 7. Graphical representation of specifications for cement-bentonite strength.
QUALITY CONTROL TESTS

Unconfined compressive strengths were measured per ASTM D2166. At the beginning of the project, the strength of the completed CB walls was verified by testing only core samples. Low strengths were suspect, however, considering the likely disturbance of the material during coring and handling. The specifications were then modified to allow testing of “wet-grab” specimens, formed from slurry samples acquired at depth in the trenches. While not weakened by coring operations, wet-grab samples could be stronger due to the screening of unmixed inclusions and the controlled curing environment.

In the specifications, the laboratory strength of core specimens was assumed to equal the strength of the cured walls. Limited test data from a demonstration section indicated that wet-grab samples might be 40% stronger than core samples. A more extensive data set accumulated during construction of the first perimeter segments showed that the wet-grab samples were, on average, only 10% stronger than the cores. Hence, strengths determined from wet-grab samples were reduced by a factor of 1.1 before comparing to the design requirements.

Test parcels were identified for the quality control evaluations. Each test parcel was 100 feet (30 meters) of wall, selected from 500 feet (150 meters) of completed length. In five locations in each test parcel, wet-grab samples were acquired from various depths using a “mailbox” type sampler (a lid was opened to acquire samples at discrete depths). Test specimens were formed in 3x6-inch (7.6x15-cm) cylinders. Core samples were acquired with a PQ-size, triple-barrel core sampler. At least 25 test specimens, from either wet-grab or core samples, were required for each test parcel. Samples from the upper 5 feet (1.5 meters) of the walls were not used in the strength verification tests.

WALL DEFECTS AND INCLUSIONS

At least three boreholes were advanced in each test parcel, to check for potential inclusions and defects in the walls. Depth intervals where soil inclusions or unfixated materials were encountered, including intervals of no core recovery, were identified. In select holes, down-hole cameras were used to verify that the core logs accurately represented conditions in the wall.

Soil or unfixated materials in continuous lengths of 6 to 24 inches (15 to 60 cm) were defined as “inclusions” in the specifications. These intervals were assigned an unconfined compressive strength of 10 psi (70 kPa, representative of soil) and then used in the statistical evaluation of wall strength. In this manner, the impact of soft inclusions could be rationally assessed. Most of the wall panels that were found to contain soil or unfixated inclusions were accepted.

Intervals with soil inclusions or unfixated materials over 24 inches (60 cm) in length (half the wall thickness) were identified as “defects”. Each wall defect required mitigation. Depending on the location and size, defective areas were addressed by constructing a parallel, overlapping wall panel or by jet grouting within the wall.
SOFT GROUND CONDITIONS AND TRENCH COLLAPSE

A significant challenge during excavation of the CB trenches was the stability of the working platform. Around most of the site, the construction footprint was on top of thick deposits of wet fly ash. The excavators weighed over 150 tons when outfitted with the long-reach attachments. In digging the CB trenches, the excavators would repeatedly rock back and forth on the same location for several hours. In areas with saturated ash in the subgrade, water would pump to the surface causing rapid deterioration of the working platform. Timber crane mats were used under the excavators, but in some places, two to three layers of mats had to be added to stabilize the equipment as the trenches were dug.

A variety of mitigation measures were tried to stabilize the working platform. These efforts included stabilizing the surface with Portland cement, using geosynthetics, and placing layers of coarse bottom ash or stone aggregate. These were minimally successful. The excavation sequence was altered to provide more time for the working platform to drain between digging adjacent trenches. A key component was good management of the trench spoils, providing for drainage and timely removal of wet spoils from the working platform.

Significant problems with sloughing and trench collapse (Figure 8) occurred in areas with high groundwater, which overwhelmed the stabilizing effects of the slurry in the excavation. These issues were most pronounced along the western side of the site, where the groundwater was within about 3 to 4 feet (about one meter) of the ground surface. In addition, groundwater pressures in the deeper alluvial sands were found to be slightly artesian in this area. In some trenches, water was observed percolating up through the CB slurry during excavation (Figure 9).

Ultimately, to stabilize the working platform along Segments 4 through 8, TVA engaged Griffin Dewatering to lower the groundwater levels ahead of CB wall construction. Vertical drains were driven 40 feet (12 meters) into the ash and attached to a vacuum extraction system. Along perimeter Segments 6 and 7, 60-foot (18-meter) dewatering wells were installed into the deeper sands. This approach was successful, and allowed for timely completion of the CB walls in these problematic areas.

![Figure 8. A collapsed slurry trench.](image1)

![Figure 9. Artesian flow in a trench.](image2)
COLD JOINTS

The wall grid pattern (Figure 2) required constructing the intersecting panels in sequence over several days. Where individual panels connected, a cold construction joint was expected to leave a structural weakness. The 3D design analyses (Figure 4) demonstrated that weaker joints were tolerable, as long as the joint strength was not less than about half the shear strength of the walls.

To combat the soft ground conditions on the working platform, Geo-Con installed half of each shear wall, skipped one or more walls, and then returned to complete the other half. This resulted in a cold joint in nearly every shear wall, plus additional cold joints at the inboard and outboard perimeter walls. Mitigation (reinforcement) was required for all cold joints formed with a CB panel that had cured for more than 7 days without additional time limit.

Initially, the specifications required mitigation of any “cold joint” excavated into a wall panel that was more than 7 days old. This limitation was difficult to avoid in the construction sequence, and data was sought to better substantiate the specified limit. Laboratory direct shear tests showed that adequate strength developed across an interface with cured cement bentonite. There was a secondary concern that excavating into brittle, cured CB would leave a fractured interface. Trial excavations of a field test section (Figure 10) were made to evaluate the plastic behavior of the CB material as it was cured. Based on these test results, the specifications were changed to allow constructing joints up to 20 days without additional mitigation.

High pressure jet grouting was used to reinforce cold joints where the 20-day limit could not be met. The grouting process was planned to create two variable-diameter grout columns in the joint. The grout columns were designed to transfer vertical shear stresses across the joint interface. The initial step was to bore two, 12-inch (30-cm) diameter holes along the joint using an air rotary drill rig. Verticality within 1% of plumb was verified with downhole measurements in each bore hole. Grout was then injected at 5,000 psi (34 MPa) while slowly rotating and lifting the drill pipe. Jet grouting was sequenced to form 14-inch (36-cm) grout bulbs in 1-foot (30 cm) vertical intervals, in every other foot of height. A neat cement-water grout, with a targeted unconfined compressive strength of 315 psi (2,170 kPa), was used. Test columns were installed to prove the method. A total of 188 cold joint locations were mitigated with jet grouting over the course of the project.

Figure 10. Test excavations used to establish the time limit for cold joints.
CONCLUSIONS

TVA faced significant challenges in responding to the 2008 coal ash failure at the Kingston power plant. To close the site and cap the remaining ash in place, the site perimeter had to be stabilized to contain the stored material during a large earthquake. The design considers 2,500-year seismic events, which are strong enough to liquefy the saturated ash and deeper alluvial sands. The solution involved constructing over eleven miles (18 kilometers) of cement-bentonite walls. With excellent cooperation, the project team overcame numerous challenges and finished the CB walls in late 2014. The coal ash landfill has since been capped, completing the successful recovery and closure of the Kingston site.

REFERENCES


