Monitoring Stability While Stacking Fly Ash During the Kingston Recovery Project

Alan F. Rauch, PhD, PE, Michael J. Steele, PE, Kirk A. Jenkins, PE, and Jim Andrew, PE

Stantec, 3052 Beaumont Centre Circle, Lexington, Kentucky 40513
Alan.Rauch@Stantec.com, Mike.Steele@Stantec.com,
Kirk.Jenkins@Stantec.com, and Jim.Andrew@Stantec.com

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ABSTRACT

Power companies are closing Coal Combustion Residual (CCR) impoundments at many sites. Operating heavy equipment, placing soil fill for cover, or stacking additional CCRs can initiate failures within the underlying deposits of saturated fly ash. These activities generate excess pore pressures that may trigger static liquefaction, localized bearing capacity failures, and/or much larger embankment slope failures. Subsurface instrumentation and robust monitoring programs can be used to detect worsening conditions and make adjustments to avoid failures. The program used to monitor stability during TVA’s Kingston Recovery Project is described. During the clean-up following the 2008 failure, about two million cubic meters of recovered ash were stacked across the site. At the peak, over 30,000 cubic meters of material were placed per week. The monitoring program included a changing array of piezometers and inclinometers. Alarm limits were set for each instrument, but were not fail-safe. Careful engineering review of the data trends was a critical component of the program. On several occasions when the data indicated deteriorating conditions, stacking operations were restricted and mitigation steps were implemented. In the end, the site was successfully restored and closed without triggering a failure.

INTRODUCTION

Stability is a primary concern when stacking Coal Combustion Residuals (CCRs) over saturated deposits of fly ash. Excess pore pressures can accumulate in the ash during rapid loading and activate undrained foundation failures. In some CCR materials, small deformations can trigger static liquefaction. However, if construction is carefully planned
and controlled, stable embankments can be built over saturated fly ash. Subsurface instrumentation and a strong monitoring program are essential.

This paper reviews the experience gained during the Kingston Recovery Project (KRP). Following the 2008 failure, the Tennessee Valley Authority (TVA) dredged the adjacent river and embayment, processed the recovered ash slurry, and gradually dried the material to manageable moisture contents (TVA 2011). About three million cubic yards (two million cubic meters) of the recovered material was stacked over the failed dredge cell and the adjacent ash pond. The embankment had to be stable without the benefit of the stabilized perimeter, which was being built concurrently in segments (TVA 2015).

Extensive monitoring was used to ensure that slope failures did not occur while ash was rapidly stacked. The monitoring program included a large number of piezometers. Some were read automatically, but most were manually recorded. Ground deformations were monitored with conventional settlement plates and slope inclinometers. Alarms (threshold and action limits) were established for each instrument reading. Regular, ongoing review of the data by a professional engineer was key to avoiding failures while completing the project.

This paper does not address stability of construction equipment working on CCR materials. Heavy equipment can be vulnerable to localized, shallow bearing capacity failures. Careful planning and other precautions are necessary to ensure the safety of equipment and personnel operating on saturated fly ash.

**KINGSTON RECOVERY PROJECT**

On December 22, 2008, a dike failure at TVA’s Kingston Fossil Plant released 5.4 million cubic yards (4.1 million cubic meters) of CCR materials. The displaced material covered roughly 300 acres (120 hectares), directly impacted about 40 area homes, destroyed a highway and railroad line, and blocked the adjacent Emory River. An aerial view of the failed facility is provided in Figure 1. A root cause analysis of the failure was completed by AECOM (2009).

In the aftermath, TVA moved quickly to dredge ash from the river. An aggressive schedule was established for clean-up and site closure. About half of the recovered ash was shipped to an off-site landfill for disposal. The remaining, recovered CCR materials were stacked over the failed cell and adjacent ash pond, in the areas delineated in Figure 2. The former impoundment facilities were then capped and closed.

In the first stage of construction, a test embankment was built within the footprint of the failed facility (Figure 2). Geotechnical instrumentation was installed and a monitoring program was established. This large-scale test program demonstrated how reclaimed ash could be safely re-stacked across the area that had failed. The stability monitoring program was refined during the test embankment phase and later applied to production stacking efforts.
During ash stacking, piezometers were used to track increasing pore pressures due to surcharge loading. Slope inclinometers were used to watch for excessive strains within the ash foundation. Mitigation was required where these readings exceeded design thresholds. The response generally involved increased monitoring, slope stability evaluations, adjusted stacking operations, flattening slopes, filling ditches, and/or construction of stabilizing berms.

At Kingston, the design earthquake is expected to liquefy the deeper, saturated ash and alluvial soils. The site perimeter was stabilized to contain the CCR material in the event of an earthquake. Using cement bentonite slurry methods, subsurface shear and perimeter walls were built around the facility circumference and keyed into bedrock. Over 11 miles (18 kilometers) of wall were constructed, requiring over 520,000 cubic yards (400,000 cubic meters) of slurry.

Construction of the stabilized perimeter was completed in late 2014. The ash stack (CCR landfill) was topped out and the final cover was completed in 2015. Containing about 18 million cubic yards (14 million cubic meters) of CCR material, the Kingston facility is now capped and safely closed. Overviews of the Kingston Recovery Project, including details of the design and construction, are provided by TVA (2011; 2015), Bussey et al. (2012), Dotson et al. (2013), Rauch (2014), and Rauch et al. (2013; 2017).

Figure 1. Failed CCR dredge cell at the Kingston power plant (from TVA 2011).
TEST EMBANKMENT

With the aggressive recovery schedule, there was a shortage of space to process and temporarily store the dredged material. Recovered ash had to be stacked within the footprint of the failed North Dredge Cell, prior to completing the stabilized perimeter. Below the existing surface, there was approximately 40 feet (12 meters) of old, saturated fly ash deposits. These materials are below the elevation of the adjacent Emory River and dewatering was impractical. There was a significant risk that ash stacking could trigger another failure before the perimeter containment was complete.

A test embankment was thus planned and constructed, to demonstrate and refine the procedures that would be used to safely build the CCR landfill and close the site. The embankment was built of recovered ash that was moisture conditioned, spread in lifts, and compacted per the specifications. The completed test embankment (Figure 3) was 45 feet (14 meters) tall and contained over 250,000 cubic yards (190,000 cubic meters) of CCR materials. This configuration represented the fill height and loading conditions that would be realized across the site later in the recovery project.

Figure 2. Defined areas for stacking ash, Kingston Recovery Project.
The monitoring program and quality control procedures (described later) were refined during this program. On two occasions, test embankment operations were modified when the instruments indicated developing problems. In the first case, the piezometers detected rising groundwater that resulted from dewatering stockpiles of ash in an adjacent area. These activities were then restricted in the vicinity. In the second case, small slope deformations were attributed to erosion at the base of the slope. A buttress of ash was then built to stabilize this portion of the test embankment.

The test embankment was successfully completed in April 2010. The success of the test embankment program helped to allay concerns that new stacking would trigger another failure. The project team could then move to production ash stacking, as the project moved toward the final closure geometry.

PRODUCTION ASH STACKING

As the Recovery Project advanced, four design packages were developed for stacking the CCR materials within sectors of the site. These areas, outlined in Figure 2, were identified as the North Dredge Cell, Relic Area, Lateral Expansion, and Ash Pond. The design and construction challenges included:

- Lack of surface drainage and stormwater control in the area of the failure.
- Soft bearing conditions on the initial surface, at the start of stacking. Operating equipment and compacting the first few lifts of fill was difficult.
• Rapid loading of the foundation materials. Below the existing surface grade were 30 to 50 feet (10 to 15 meters) of older, saturated, sluiced ash deposits.
• In most areas, ash stacking proceeded ahead of constructing the stabilized perimeter. The embankment slopes had to be stable without the benefit of the subsurface walls.

In places, up to about 35 feet (11 meters) of ash fill was specified. Interim geometry was analyzed for slope stability, with particular attention to the potential for deep seated failures like the 2008 event. Maximum 6:1 (horizontal to vertical) slopes were specified for interior embankments.

The recovered ash was placed in lifts and compacted with engineering controls (Figure 4). Lifts were limited to a maximum uncompacted thickness of 24 inches (0.6 meters). The material was compacted to a minimum dry unit weight equal to 90 percent of the standard Proctor maximum dry density. The specifications required a moisture content in the ash fill no less than 2 percentage points below and no greater than 6 percentage points above the standard Proctor optimum.

A comprehensive quality control program was implemented. A resident team of engineers and technicians conducted routine field observations and testing to verify that the CCR materials were placed and compacted within specification. Specific criteria included subgrade verification (prior to filling), material selection, moisture condition, lift thickness, rate of placement, and compaction.

The dredged CCR materials were variable. Fill handling and compaction were particularly sensitive to moisture content. The ash would approach saturation at water contents 6 percent above optimum. This would result in a wet surface subject to pumping as the fill was rolled and worked. The observed response of the ash to

![Image of ash stacking operations]

Figure 4. Production ash stacking operations.
equipment loading was a critical quality control tool. As needed, the allowable moisture content was adjusted in the field based on observations of fill placement. Areas with unacceptable subgrade were excavated and replaced with drier material. In particularly wet and soft areas, stone aggregate was used to improve subgrade drainage.

At the peak of construction, ash was stacked at rates up to 40,000 cubic yards (30,000 cubic meters) per week. The chart in Figure 5 shows the approximate volume of recovered ash that was stacked each week during the project.

![Figure 5. Approximate volume of recovered material placed in the ash stack during the Kingston Recovery Project.](image)

**INSTRUMENTATION**

Slope inclinometers, settlement plates, and piezometers were used to measure movements and pressures within the saturated foundation. Data from these instruments were monitored for signs of potential instability.

Piezometer instrumentation varied across the site during the project. In a few locations, Casagrande-type piezometers (open standpipes) were installed and water levels were measured manually. Automated sensors were installed inside some standpipes. Most piezometer installations had vibrating wire transducers backfilled with sand over discrete intervals, and a few were fully grouted. A small number of the vibrating wire sensors were automated, but most were manually recorded with a hand-held readout. In a few locations, pneumatic pressure transducers installed by earlier teams were used. Depending on the installation, the piezometers read pressures in the saturated ash or the deeper alluvial soil deposits. Readings were generally acquired twice per day in areas of active embankment construction, and daily elsewhere.

Settlement plates were conventional installations, with one-meter square steel plates placed at the base of the embankment and within the fill. Telescoping pipe sections (telltales) extending to the ground surface indicated vertical movements of the plates.
Elevations were determined by surveying. Slope inclinometers were not automated, and measurements were individually collected by technicians using a downhole inclinometer probe (Figure 6). Settlement and slope inclinometer measurements were typically recorded each day in areas where the embankment was being raised.

The number of active instruments varied throughout the project, as instruments were added, abandoned, damaged, and/or replaced. At the peak, the site team was monitoring a network of over 200 subsurface instruments. Table 1 provides an indication of the number of measurements recorded in different areas of the site. As an example, instruments installed within the Lateral Expansion area are shown in Figure 7, along with section lines used to evaluate interim slope stability.

![Figure 6. Field technicians recording slope inclinometer data.](image)

**Table 1. Approximate number of instruments monitored during ash stacking.**

<table>
<thead>
<tr>
<th>Project Phase</th>
<th>Area of Site</th>
<th>Number of Piezometers*</th>
<th>Number of Inclinometers</th>
<th>Number of Settlement Plates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>acres</td>
<td>hectares</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test Embankment</td>
<td>11</td>
<td>4</td>
<td>48</td>
<td>11</td>
</tr>
<tr>
<td>North Dredge Cell</td>
<td>64</td>
<td>26</td>
<td>85</td>
<td>11</td>
</tr>
<tr>
<td>Relic Area</td>
<td>43</td>
<td>18</td>
<td>21</td>
<td>5</td>
</tr>
<tr>
<td>Lateral Expansion</td>
<td>35</td>
<td>14</td>
<td>61</td>
<td>26</td>
</tr>
<tr>
<td>Ash Pond</td>
<td>47</td>
<td>19</td>
<td>21</td>
<td>6</td>
</tr>
</tbody>
</table>

* Included up to 4 sensors at different depths at each piezometer location
Figure 7. Instrumentation monitored in the Lateral Expansion area.

MONITORING PROGRAM

A detailed instrument monitoring plan defined protocols for collecting, processing, and interpreting the field data during construction. Instruments were read daily in the areas of active fill placement, with weekly readings collected in adjacent areas of the site. An on-site professional engineer was tasked with reviewing the field data.
Data collected by the field technicians was processed and plotted by the office support team, and reviewed by the on-site professional engineer. Each week, a summary report of the data was distributed to key engineering and construction personnel. Typical plots of this data are provided in Figure 8 and Figure 9. Two cases of unexpected data trends are illustrated on the plots.

For each instrument, alarm limits were established to call attention to high readings and initiate further engineering evaluation. As described below, the alarms were established based on the expected ash behavior and engineering stability analyses. However, data recorded by piezometers and inclinometers do not provide a direct measure of slope stability. The established alarm limits were only a guideline. Unsatisfactory conditions were sometimes identified even when the instruments were below the alarm levels, and failures did not occur when readings were sometimes over the alarm limits. On-going observation and evaluation by the engineering team was required.

Notifications of instrument alarms, disturbing trends, increased monitoring, and/or stacking restrictions were distributed in accordance with the monitoring plan. When alarm values were exceeded, or if other trends in the data suggested potential instabilities, the project team conducted further evaluations. The current embankment configuration, measured pore pressures, prior analyses, etc. were assessed. In some cases, stability calculations were updated for the current field conditions.

Figure 8. Typical plots of piezometer readings and fill height.
Results were then used to define corrective actions and adjust operations, including stacking restrictions within defined areas. These restrictions affected the construction schedule. The engineering team was then challenged to provide alternatives such as revised embankment configurations, reduced loading rates, or relocation of construction operations to allow continued safe stacking of the recovered ash.

**INSTRUMENT ALARM LIMITS**

Alarm limits were pre-determined for each instrument. Limits were set to highlight elevated readings that could signal an incipient failure. Setting appropriate instrument alarm limits is challenging and must consider several factors:

- Failures may result if the alarm limits are set too high. The alarm must be low enough to allow time for a response, from initiating a data review to modifying construction operations. Several days may be required to fully react to an alarm.

- If alarm limits are set too low, the alarms may sound too frequently. This leads to complacency and ignored alarms, and developing failures may be overlooked.

![Typical plots of horizontal and vertical displacements.](image)
More conservative alarm limits are warranted where subsurface conditions are not well understood, where there is greater uncertainty in the mechanics of a potential failure, or where the consequences of a failure are higher.

Instruments provide measurements at a single location, which may not represent the conditions throughout the much larger embankment and foundation. No single instrument reading can be directly correlated to embankment stability.

The instrument alarm limits selected for the Kingston project are defined below. These limits may not be applicable to other sites or similar projects. In all cases, alarms for monitoring instruments should be set by a professional engineer after evaluating the site-specific foundation conditions, embankment loading, consequences of failure, and other relevant considerations.

**Definitions.** For test embankment construction and production ash stacking at Kingston, two alarm limits were set for each instrument:

- **Threshold Limit.** Embankment construction could continue with routine monitoring when the instrument readings were below the threshold limit. When a reading exceeded the threshold, the engineering team was notified and, at a minimum, more frequent monitoring was implemented.

- **Action Limit.** When an instrument reading exceeded an action limit, fill placement in that area was automatically stopped. Stability of the embankment was analytically evaluated by the engineering team and appropriate restrictions or changes in construction were implemented.

**Pore Pressure Limits.** Alarm limits for the piezometers were determined on the basis of drained slope stability analyses, using assumed excess pore pressure ratios. The alarm limits were initially determined for target factors of safety for stability. However, considering the high consequences of another failure at Kingston, the alarm limits were conservatively set at much lower pressure ratios.

The alarm limits were defined in terms of the excess pore pressure ratio (PR):

- **PR = \( r_u = \frac{\text{Excess Pore Water Pressure}}{\text{Added Vertical Stress}} \)**
- **Threshold Limit:** \( PR = 10\% \) in alluvial foundation soils
- **Action Limit:** \( PR = 15\% \) in alluvial foundation soils
- **Action Limit:** \( PR = 10\% \) in saturated ash deposits

Pore pressure ratios from one piezometer in the foundation soils are plotted in Figure 10. Relatively high values occurred early in construction, during a period of heavy rain in the spring of 2011. The embankment slopes were still nearly flat, so ash stacking was not stopped. As the embankment was raised, elevated pressure ratios were recorded in early 2012, but these readings did not exceed the threshold limit.
Using a pressure ratio for the alarm limits, instead of a pressure reading or water level, complicated the monitoring program. The surcharge pressure, resulting from the weight the embankment fill, was required to compute PR for each piezometer reading. This necessitated daily surveys of the embankment surface elevation in the vicinity of each piezometer, to compute the accumulated change in vertical stress. The pressure readings (or hydraulic head) that would correspond to an alarm thus varied as the embankment was raised. In the early stages of construction, the change in vertical stress was small. The denominator in PR was small, which set off alarms for minor pressure increases. This caused unnecessary concern at the start of construction.

**Lateral Displacement Limits.** For the slope inclinometer data, the alarm limits were defined in terms of a displacement ratio (DR):

- \[ DR = \frac{\text{Maximum Horizontal Displacement at any Depth}}{\text{Total Vertical Settlement}} \]
- **Threshold Limit:** \( DR = 20\% \)
- **Action Limit:** \( DR = 30\% \)

An example plot of the displacement ratios for one inclinometer installation is provided in Figure 11. At this location, the data approached, but did not exceed, the action limit in the summer of 2013.

Using the slope inclinometer data, the maximum displacement was obtained from the profile of lateral deformations (see Figure 12). Measured settlements were required to compute the DR for each profile. However, a settlement plate was not installed adjacent to every slope inclinometer casing. The measurement from the nearest settlement plate, or an average value for an area of the fill, was used for these calculations. Also, the induced settlement was essentially zero in the early stages of construction. This set off the DR alarms unnecessarily when insignificant lateral displacements were measured.
Figure 11. Example plot of displacement ratios at one inclinometer.

Shear Strain Limit. Plots of the lateral displacements determined with the slope inclinometer data were plotted versus elevation (Figure 12). These profiles were examined by the engineering team to identify sharp offsets that might indicate shearing along a developing failure plane.

Where offsets were noted, the slope of the profile was used to compute the average shear strain over that depth increment. The interval shear strain (SS) between two adjacent points (A and B) is simply:

\[
SS = \frac{(\text{Horizontal Displacement at Elevation A}) - (\text{Horizontal Displacement at Elevation B})}{(\text{Elevation A}) - (\text{Elevation B})}
\]

- Action Limit: \( SS = 1\% \)

The limiting value of 1% was chosen based on the results from laboratory tests on the site-specific materials. In the example profile in Figure 12, the measured displacements indicated a shear strain of 1% at the base of the ash.

AECOM (2009) concluded that the 2008 failure at Kingston was triggered by creep in a thin sensitive layer at the base of the ash. Direct simple shear tests on undisturbed samples of the sensitive material indicated a peak undrained strength at shear strains of 4% to 13%. However, in one test, the sensitive soil failed due to creep when placed under a constant shear load equal to 85% of the peak strength. Based on the laboratory data, shear stresses up to about 75% of the peak strength caused shear strains up to about 1%. In other words, shear strains that exceed 1% indicate stresses that would be high enough to induce significant creep in the sensitive soil layer.

AECOM (2009) also completed consolidated undrained triaxial compression tests on remolded samples of the Kingston ash. Specimens prepared by moist tamping, with void ratios representative of the field deposits, indicated significant strain softening past a peak strength at 0.3% to 1.3% axial strain (0.5% to 2% shear strain). Hence, shear
strains of 1% would indicate that conditions are approaching the peak undrained strength of the saturated ash.

**Data Trend Limits.** As construction progressed, data from the piezometers and inclinometers were carefully watched and evaluated. The engineering team considered the longer-term data trends, and not just the pre-determined alarm limits. Sharp changes in the measured pore pressures, or accelerating increases in pressure, were viewed as potential indications of developing failures. Likewise, accelerations in the maximum horizontal displacement from the slope inclinometer measurements were examined for indications of possible instability. An example is shown in Figure 13. Numerical alarm limits were not established for these data trends.

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**Figure 12.** Example plot of slope inclinometer data indicating shear strains of 1% at the base of the ash.

**Figure 13.** Example case where accelerating displacements and shear strains indicated instability.
STACKING RESTRICTIONS

On 11 separate occasions (Table 2), ash stacking was halted or restricted when the instruments indicated incipient movements or elevated pore pressures, or when interim stability analyses returned low factors of safety. Construction operations were temporarily relocated to other areas of the site when restrictions were implemented. In some instances, interim slopes were flattened or buttressed to stabilize observed movements. In total, the construction restrictions within certain portions of the site lasted for 204 days.

Figure 14 shows three areas of the Lateral Expansion where stacking restrictions were imposed in the fall of 2011. Managing construction in this area was particularly challenging, as the recovered materials were being stacked over the prior ash pond in an area that had not been previously loaded. That is, the saturated ash foundation had not consolidated under a higher overburden pressure. After a few weeks, when excess pore pressures had dissipated and movements had slowed, ash stacking was allowed to continue.

Pore pressures that exceeded an action limit would dissipate over time. Stacking restrictions could be lifted when the readings fell below the action limits. Displacements and strains do not reverse over time, but further movements would slow to a stop if no additional load was applied. The rate of movement was watched to determine when stacking restrictions could be lifted.

<table>
<thead>
<tr>
<th>Date</th>
<th>Trigger</th>
<th>Area of Site</th>
<th>Length (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/30/2009</td>
<td>Porewater Pressure</td>
<td>Test Embankment</td>
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<tr>
<td>2/2/2010</td>
<td>Low FS for Stability</td>
<td>Test Embankment</td>
<td>50</td>
</tr>
<tr>
<td>10/2/2010</td>
<td>Porewater Pressure</td>
<td>North Dredge Cell</td>
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</tr>
<tr>
<td>1/18/2011</td>
<td>Porewater Pressure</td>
<td>North Dredge Cell</td>
<td>1</td>
</tr>
<tr>
<td>2/16/2011</td>
<td>Porewater Pressure</td>
<td>North Dredge Cell</td>
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<tr>
<td>10/25/2011</td>
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<td>Lateral Expansion</td>
<td>73</td>
</tr>
<tr>
<td>11/8/2011</td>
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<td>6/11/2012</td>
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<td>Lateral Expansion</td>
<td>3</td>
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<tr>
<td>6/25/2012</td>
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<tr>
<td>8/26/2013</td>
<td>Horizontal Displacement</td>
<td>Lateral Expansion</td>
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</table>
LESSONS LEARNED

Construction over saturated fly ash requires planning, monitoring, and diligence to detect initial signs of instability and mitigate the potential for failures. Important lessons learned in the Kingston Recovery Project are applicable to similar work at other CCR sites. These include:

Figure 14. Stacking restrictions in the Lateral Expansion area.
• Monitoring programs must be well-planned and diligently executed by responsible professionals.

• An array of instrumentation (including piezometers and inclinometers) should be installed prior to construction. Instrument type, spacing, etc. should be selected for the site conditions and anticipated failure modes (e.g., slope stability and bearing capacity).

• Even with a fairly dense array of instruments, the sensors may not be in the right horizontal or vertical location to detect failure initiation.

• Subtle indications of deteriorating stability can be easily missed or overlooked. Ongoing, careful review of the data by a team of engineers must be part of the monitoring program.

• A written Monitoring Plan should be prepared. The Plan should address the frequency of instrument readings; identify who will collect, process, and plot the data; and name the engineer responsible for interpreting the results.

• The recorded data should be compared to pre-determined alarm limits. The alarms may require adjustment as construction proceeds.

• Selecting appropriate instrument alarm limits can be difficult. When alarms are set too high, elevated instrument readings may be overlooked. Instrument alarms that are set too low can result in false alarms and, eventually, complacency.

• Where feasible, instrument alarm limits should be determined on the basis of stability analyses. Piezometric alarms can be selected through parametric slope stability calculations.

• Rational alarm levels can be defined in terms of limiting pore pressure ratios and displacement ratios. However, these ratios are not well defined in the early stages of construction. The ratios are difficult to track because they require additional information to compute the overburden pressure and settlements.

• A better approach is to set piezometer alarms in terms of hydraulic head (water level elevations), displacement alarms in terms of the rate of horizontal movement, and interval shear strain alarms based on the slope of the inclinometer profile.

• A professional engineer should be designated to evaluate each alarm and review the instrument data for indications of instability. The ongoing data review can be more critical than the pre-set alarms in detecting an incipient failure.

REFERENCES


