ABSTRACT: The Piedmont Triad International Airport (PTIA), Greensboro, NC has been undergoing an expansion including a new runway (Runway 5L-23R) and taxiway (Taxiway E) as a result of the construction of a Mid-Atlantic regional hub for Federal Express. The presence of soft ground in the wetlands area at Taxiway E crossing required careful consideration of the effects of construction, and wetland permitting requirements. The design of the embankments had to include minimizing the total area impact as well as prevent slope stability and bearing capacity issues during construction. The use of ground stabilization to provide a working platform in conjunction with toe berms to maintain slope stability and wick drains to accelerate drainage, strength gain, and settlement offered benefits to all members of the project team. An instrumentation program was implemented to determine the fill placement rates or appropriate waiting periods during fill placement, and to prevent slope failures.
1 BACKGROUND INFORMATION

The Piedmont Triad International Airport (PTIA) located in Greensboro, North Carolina is undergoing an approximate $550 million expansion as a result of Federal Express selecting this site for their Mid-Atlantic regional hub. A photograph of the overall site is provided in Figure 1.

Figure 1. Piedmont Triad International Airport (PTIA)

Part of the construction of the associated Taxiway E includes the construction of the embankment over a wetlands area in an alluvial floodplain. The wetland crossing is from STA 11+00 and STA 17+00, for an approximate distance of 183 meters (600 feet). The final grades in this area require the construction of an earth embankment up to 18 meters (60 feet) in height over the existing soft ground, adjacent to Brush Creek. The footprint of the taxiway alignment is restricted by a Corps of Engineers agreement and other water quality standards, including wildlife habitat requirements imposed by state agencies. The wetland water quality and flow could not be impacted by the crossing, requiring that the potential for slope failures be minimized. The flow of Brush Creek was diverted into a box culvert approximately 160 meters (525 feet) in length.

2 SUBSURFACE CONDITIONS

The subsurface exploration indicated the ground profile to consist of an upper zone of very soft to soft alluvial soil underlain by a residual soil profile, with groundwater at or near the ground surface. See Figure 2 for a typical boring record in the wetland area. The alluvial soils present in the borings within the wetland area extend from 1 meter (3 feet) to 8 meters (27 feet) below the existing ground surface. In general, the alluvial soils consist of either sandy silts or silty sands with varying amounts of mica and clay.
Figure 2. Boring Record

<table>
<thead>
<tr>
<th>DEPTH, METERS</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>Alluvial - Very Moist Very Loose Brown Silty Coarse to Fine SAND</td>
</tr>
<tr>
<td>0.9</td>
<td>Alluvial - Saturated Very Loose Tan-Grey Slightly Micaeous Medium to Fine SAND</td>
</tr>
<tr>
<td>1.4</td>
<td>Alluvial - Saturated Very Soft Tan-Grey Medium to Fine Sandy Clayey SILT</td>
</tr>
<tr>
<td>2.6</td>
<td>Alluvial - Wet Firm Tan-Grey Medium to Fine Sandy Clayey SILT</td>
</tr>
<tr>
<td>3.4</td>
<td>Alluvial - Wet Very Soft Grey-Black Slightly Clayey Medium to Fine Sandy SILT</td>
</tr>
<tr>
<td>5.2</td>
<td>Alluvial - Saturated Soft Grey Slightly Micaeous Medium to Fine Sandy SILT</td>
</tr>
<tr>
<td>6.7</td>
<td>Alluvial - Very Moist Soft Black-Grey Slightly Micaeous Medium to Fine Sandy SILT</td>
</tr>
<tr>
<td>8.2</td>
<td>Residual - Moist Very Firm Brown Micaeous Silty Medium to Fine SAND</td>
</tr>
<tr>
<td>9.1</td>
<td>Boring Terminated</td>
</tr>
</tbody>
</table>

Note: For angles <32°, Triaxial - Plane.
The soil test borings and dilatometer soundings indicated a very heterogeneous alluvial profile, with the areal extent and thickness of the different soil types varying significantly. Standard Penetration Resistance values obtained in the alluvial soils ranged from Weight of Rod (WOR) to 10 blows per foot (bpf), with the majority of values less than 5 bpf. Undrained shear strengths in the fine grained soils measured between 0.96 kPa (20 psf) to 48.9 kPa (1000 psf), with the majority of values less than 24.5 kPa (500 psf) (Hayes, September 1990). The surface of the alluvial soils, after stripping of the rootmat, would not support the weight of a person.

Below the alluvial materials is a residual soil profile consisting of sandy silts and silty sands overlying weathered rock. Standard Penetration Resistance values obtained in the residual soils range from 6 to 90 blows per foot (bpf), with the majority of values between 15 and 55 bpf. Weathered rock consists of residual materials that exhibit SPT N-values of 100 bpf or greater. Groundwater is generally within 0.6 meter (2 feet) of the ground surface. A photo of the site after clearing, but prior to construction, is provided in Figure 3.

![Figure 3: View of wetlands area from the Southwest](image)

3 PREDICTED PERFORMANCE

Settlement predictions for the alluvial soils below the embankment were based on the results of the soil test borings and dilatometer testing. Empirical analyses indicated settlement magnitudes ranging from 0.8 meters (2.5 feet) to 1.2 meters (4 feet) within the central portion of the embankment area. Settlements of 15 cm (6 inches) or less were predicted at the toe of the rock berm. The settlements were predicted using FHWA and Schmertmann methodologies.

A stability analysis was performed in which the in-situ material properties were varied due to the heterogeneity of the alluvial soils below the embankment. The stability analysis was performed at various fill heights with parameters varied from preconstruction values to the anticipated values due to strength gains after consolidation.
Various slope configurations, including one that incorporated a vertical retaining wall, were analyzed using steady state conditions in a value engineering study. This study included settlement potential and slope stability analyses. Also, various excavation and ground improvement alternatives were investigated to improve the safety factors for slope stability since probable failures were predicted due to the soft alluvial soils beneath the embankment (Han, et al., Jan. 29-31, 2004). A brief description of the initially considered scenarios, the estimated cost differential for preparation below the embankment and construction time is provided below:

- Complete removal of all soft alluvial materials within the zone of the taxiway crossing of the wetlands. Cost - $5,550,000.00, Duration – 1 year
- Partial removal of an upper zone of the soft alluvial materials to a depth of 2.4 meters (8 feet) and replacement with a granular material placed below the groundwater surface. Cost – $3,850,000.00, Duration – 1 year
- The use of stone columns under the toe of the slopes to provide for adequate slope stability. Cost – $2,600,000.00, Duration – 1 year
- Sequencing the fill placement and construction over a long period of time (Ladd, 1991). Cost – $780,000.00, Duration – 3 years

The four considered scenarios did not fit within the desired construction budget or schedule, as dictated by the construction schedule for the Federal Express facility. The chosen option included the use of vertical wick drains in the alluvial materials to improve drainage for faster consolidation to allow shear strength improvements within the alluvial soils. Temporary rip rap berms were used beyond the toe of the final slopes in the wetlands before embankment placement to obtain the needed resisting forces from the shear strength improvement in the alluvial materials for embankment stability purposes. The berm materials outside the slope toe were removed in later stages and used for protection of the embankment faces. The timeframe required for this scenario was estimated to be up to 2 years, as the construction contract included a minimum waiting period of 90 days between 4 stages of fill placement. A sketch of the embankment cross section for this scenario is provided in Figure 4 below.

An instrumentation program was considered to be a necessity, to monitor the settlement magnitude, pore pressure buildup and dissipation in the soft alluvial materials, and lateral movement of the soils beyond the embankment toe to safely control the filling rates. The required waiting periods between fill placement stages could only be reduced based on the results of the instrumentation monitoring program which added on additional $225,000.00 to the below grade construction cost.
5 INSTRUMENTATION PROGRAM CONCEPT

The instrumentation program consisted of pore pressure and settlement monitoring, slope indicator measurements of the horizontal deformations, and measurements of the vertical deformations (heave) beyond the toe of the slope. This data was used to control the fill placement rate and appropriate waiting periods during fill placement to accommodate consolidation and shear strength increases in the alluvial materials. The horizontal and vertical deformations of the materials were also monitored and if found to be excessive filling could be suspended to prevent slope failures from occurring.

6 CONSTRUCTION

The ground improvements within the floodplain area of Taxiway E included clearing the area of above-ground vegetation, and selective grubbing; the rootmat was allowed to remain since it would be very difficult to remove due to the very low shear strength of the underlying alluvium. Prior to the beginning of construction for Taxiway E, a temporary haul road had been constructed through a portion of the left side of the embankment footprint in the alluvial floodplain. The haul road was used initially and then later removed. The initial presence of the haul road restricted the location of some of the instrumentation components.

Over the cleared ground, a stabilized working platform was constructed. This working platform design section consisted of a combination of biaxial geogrids, geotextile filter fabric, and 0.6 meter (2 feet) of clean, uniformly graded crushed stone (Kamal, Lane, and Heshmati, March 21-22, 2005). After the working platform was constructed, wick drains were installed to facilitate drainage of the alluvial soils. The wick drains were installed utilizing a five feet triangular grid pattern through the alluvial soils, into the underlying residual material. Once the wick drain installation was completed, a layer of uniaxial geogrid was placed on the clean stone followed by 30 cm (1 foot) of well graded crushed stone, to provide a stable working platform for filling operations. A photograph of the construction of the working platform is provided in Figure 5 below.

Figure 5. Construction of working platform
Once filling operations began, approximately 0.6 meters (2 feet) of new fill was placed each 3 working days. Some delays were experienced due to weather and seasonal conditions. Filling was suspended during the winter months due to difficulties with moisture in achieving adequate completion. At other times, filling was suspended due to rainfall. Typically, up to 4 days were lost per significant rainfall event from the rain time and drying time to resume fill placement.

7 INSTRUMENTATION INSTALLATION

The instrumentation for Taxiway E consisted of the installation of piezometers, slope indicators, horizontal conduits for settlement monitoring, and benchmarks to measure heave. Instrumentation was installed at several locations within the embankment, perpendicular to the slope at four selected stations.

Piezometers were installed at five locations along each station; one below the rip rap berm on both sides, two at the top edge of the slope at varying depths on both sides, and one under the centerline of the embankment. Soil test borings were performed at each piezometer location to delineate the zones of fine grained soils in which to install the piezometers. The piezometer tips were pushed into the fine grained soils in order to monitor the soil possessing the potential for the greatest increases in pore pressure.

Slope indicator pipes were installed approximately 9 meters (30 feet) outside the toe of the rip rap berm utilizing an ATV drill rig. The slope indicator pipes used were 8.5 cm (3.34 inches) in diameter installed within 16 cm (6.25) inch boreholes. The annular spaces were filled with sand. The pipes were placed to a depth of approximately 1.5 meters (5 feet) into the underlying residual soil.

A total of eight (8) horizontal settlement monitoring conduits were installed approximately 20 cm (8 inches) below the surface of the working platform. Four conduits were installed on each side of the taxiway centerline, at the four instrumentation stations. The conduits on the west side extended to the centerline of the embankment while the conduits on the east side extended upward from the toe approximately 75 feet. By using these conduits, the settlement of the embankment is determined along a continuous profile.

Benchmarks were installed approximately 6 meters (20 feet) from the toe of the slope on both sides. The benchmarks were monitored to determine the magnitude of vertical ground movements (heave) beyond the toe of the embankment at the four stations.

8 INSTRUMENTATION RESULTS

The instrumentation was monitored from August, 2006 to February, 2009. At the time this paper was prepared (February, 2009), approximately 60 cm (2 feet) of embankment fill remained to be placed. The use of the instrumentation program has proved to be an excellent tool to confirm the performance of the engineering design in the field. The results were used to control the rate of filling based on actual measurements instead of relying on estimated waiting periods, thus accelerating the construction schedule. The initial construction sequencing included a waiting period of 90 days at selected fill heights which would have resulted in an increase in the construction schedule of one year beyond the time required to place the fill. For this project, the end result was the elimination of the waiting periods altogether. The only delays experienced were weather-induced. The settlement rates were also used to determine a safe point at which to install the permanent retaining wall facia so that damage due to differential settlement along the wall alignment would not occur. The completed wall is shown in Figure 6.
The horizontal settlement monitoring conduit measurements indicated settlement magnitudes in general agreement with the predicted values, based on the height of fill placed. Measured settlements under the central portion of the embankment ranged from 0.33 meters (1.1 feet) to 0.81 meters (2.7 feet). Measured settlements at the toe of the rock berm ranged from 8 cm (3 inches) to 24 cm (9.5 inches). The ratio of measured settlements to predicted settlements ranges from approximately 0.38 to 0.67. This is attributed to the heterogeneity of the subsurface profile, the conservatism of the preliminary analyses, and likely, the presence of the haul road that was installed some time before the instrumentation; however, areas not monitored particularly close to the creek could have experienced larger settlement magnitudes.

An important issue of soft ground construction is the excess pore pressures generated by the filling process. To evaluate the effect of the increase in pore pressures, the ratio of the change in excess pore pressures ($\Delta\mu$) to the change in overburden pressure ($\Delta\sigma_1$), termed the "B" value, was calculated each time the field measurements were obtained. This “B” value was used to control the rate of fill placement (Fell and Hunter, 2003). For this project, the critical value of “B” was set to 1.0, based on the soil types. The piezometers indicated that the pore pressure “B” values increased to no more than 0.5, as compared to the critical value for the soil types at the project site. Measurements obtained during weather delays indicated that the excess pore pressure generally dissipated within a time period of about 10 days after the cessation of filling operations.

The slope indicator measurements did not indicate the occurrence of significant lateral movement. Lateral movements of less than 1.5 cm (0.6 inch) were measured for the timeframe reported in this paper. Measurements on the heave points did not indicate the occurrence of significant vertical movement. Vertical movements of less than 1.5 cm (0.6 inch) were measured for the timeframe reported in this paper.
9 PRACTICAL ISSUES

During the design phase, the issues most important to the project team were cost and construction schedule. The least cost alternative could be used and the construction schedule potentially shortened by using the instrumentation results to control the rate of fill placement. The concerns for a slow construction schedule and the risk of slope failures into the wetlands was eliminated.

The embankment was constructed within the estimated time frame at a cost differential of $1,500,000.00 (estimated) below the next least expensive option. The instrumentation devices were also monitored for settlement to allow placement of the retaining wall facia and final paving operations without concern for additional settlement that would _____ their performance.

An unexpected benefit of the instrumentation program was that while monitoring the construction of the working platform, it was noted that the surface of the first phase of the working platform exhibited better stability than expected. As a consequence, the depth of the upper layer of well graded crushed stone was decreased by 30 cm (1 foot), saving the owner approximately 20,000 tons of crushed stone and 10 days off the construction schedule.

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11 REFERENCES


