Final Report VTRC 06-R36

Virginia Transportation Research Council

research report

# Performance of a Pile-Supported Embankment

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EDWARD J. HOPPE, Ph.D., P.E.
Senior Research Scientist

STANLEY L. HITE, P.E.
Assistant State Materials Engineer

Virginia Transportation Research Council, 530 Edgemont Road,
Charlottesville, VA 22903-2454, www.vtrc.net, (434) 293-1900

**Standard Title Page - Report on Federally Funded Project**

| 1. Report No. | 2. Government Accession No. | 3. Recipient's Catalog No. |
|---|---|---|
| FHWA/VTRC 06-R36 | | |

| | |
|---|---|
| 4. Title and Subtitle<br>Performance of a Pile-Supported Embankment | 5. Report Date<br>June 2006<br>6. Performing Organization Code |
| 7. Author(s)<br>Edward J. Hoppe, Ph.D., P.E. and Stanley L. Hite, P.E. | 8. Performing Organization Report No.<br>VTRC 06-R36 |
| 9. Performing Organization and Address<br><br>Virginia Transportation Research Council<br>530 Edgemont Road<br>Charlottesville, VA 22903 | 10. Work Unit No. (TRAIS)<br><br>11. Contract or Grant No.<br>75448 |
| 12. Sponsoring Agencies' Name and Address<br><br>Virginia Department of Transportation — FHWA<br>1401 E. Broad Street — P.O. Box 10249<br>Richmond, VA 23219 — Richmond, VA 23240 | 13. Type of Report and Period Covered<br>Final<br><br>14. Sponsoring Agency Code |

15. Supplementary Notes

16. Abstract

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This report contains field monitoring data and analysis. Prestressed concrete piles were driven at 7-ft (2.1 m) spacing and topped with 3 ft by 3 ft (0.9 m by 0.9 m) precast concrete pile caps. Several layers of high-strength geosynthetic fabric were used for base reinforcement. The maximum embankment height was approximately 6 ft (1.8 m).

Earth pressure sensors installed onsite confirmed the formation of soil arching in the embankment fill between columns. Numerical analysis pointed to the large impact of the upper foundation soil layer properties on the magnitude of the final embankment settlement and fabric strain. This shows that accurate material characterization is essential for a cost-effective design.

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| 17 Key Words | 18. Distribution Statement |
|---|---|
| Column-supported embankment, ground improvement, stabilization. | No restrictions. This document is available to the public through NTIS, Springfield, VA 22161. |

| 19. Security Classif. (of this report) | 20. Security Classif. (of this page) | 21. No. of Pages | 22. Price |
|---|---|---|---|
| Unclassified | Unclassified | 30 | |

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**FINAL REPORT**

# PERFORMANCE OF A PILE-SUPPORTED EMBANKMENT


**Edward J. Hoppe, Ph.D., P.E.**
**Senior Research Scientist**

**Stanley L. Hite, P.E.**
**Assistant State Materials Engineer**



Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation and
the University of Virginia since 1948)

In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

June 2006
VTRC 06-R36

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**ABSTRACT**

The purpose of this study was to evaluate the field performance of the first pile-supported highway embankment constructed in Virginia. The project involved construction of an approach to the new bridge over the Mattaponi River, replacing the existing Lord Delaware Bridge at West Point. The scope of work included field instrumentation and data gathering as related to stress transfer and settlement. The objective was to measure actual soil pressures that are exerted at the geotextile fabric bridging pile caps and to measure stresses acting over pile caps. In addition, data analysis was to be carried out to provide information that VDOT engineers could use to optimize future designs of pile-supported embankments.

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## INTRODUCTION

In 2004, the Virginia Department of Transportation (VDOT) initiated corridor improvements along Route 33 in and around the Town of West Point. The project starts at F Street in West Point and ends at Ashby Road in King and Queen County. It comprises three road sections and two bridges, one over the West Point Creek and the other over the Mattaponi River.

Marginal soil conditions presented significant construction challenges. The site is located in a marshy area, underlain by very soft deposits of normally consolidated marine clays. It is technically difficult to construct an embankment over a soil that has a very low bearing capacity and that is prone to relatively large settlements. Failure often occurs when the underlying foundation soil cannot support the weight of a new embankment. To address this problem, various ground stabilization techniques, including wick drains and pile-supported embankment, were specified along some sections of the proposed roadway.

Pile-supported embankments have not been constructed by VDOT in the past, partly because of the additional costs involved. The embankment at West Point is the first one of its kind built in Virginia. Consequently, the Virginia Transportation Research Council (VTRC) was asked to perform a field assessment and provide feedback for similar ground improvement projects that may be planned by VDOT in the future.

## PURPOSE AND SCOPE

The purpose of this study was to evaluate the field performance of a pile-supported embankment constructed over the eastern approach (King and Queen County) to the new bridge over the Mattaponi River (replacing the existing Lord Delaware Bridge). The scope of work included field instrumentation and data gathering as related to stress transfer and settlement. The objective was to measure actual soil pressures that are exerted at the geotextile fabric bridging pile caps and to measure stresses acting over pile caps. In addition, data analysis was to be carried out to provide information that VDOT engineers could use to optimize future designs of pile-supported embankments.

# METHODOLOGY

## Site Description

The project is located in the Coastal Plain Physiographic Province. Ground surface elevations range from about Elevation 0 in the marsh areas to about Elevation +10 ft (3.1 m) in the Town of West Point. Parsons Brinckerhoff Quade and Douglas, Inc. (Parsons Brinckerhoff), was retained by the Virginia Department of Transportation (VDOT) to conduct the geotechnical site investigation. They performed 47 borings and 32 cone penetration tests throughout the construction corridor. Representative subsurface conditions at the location of the pile-supported embankment are depicted in the log of Borehole BP-12, as shown in the Appendix. This borehole was located on the bridge abutment centerline. Project location and layout are shown in Figure 1.

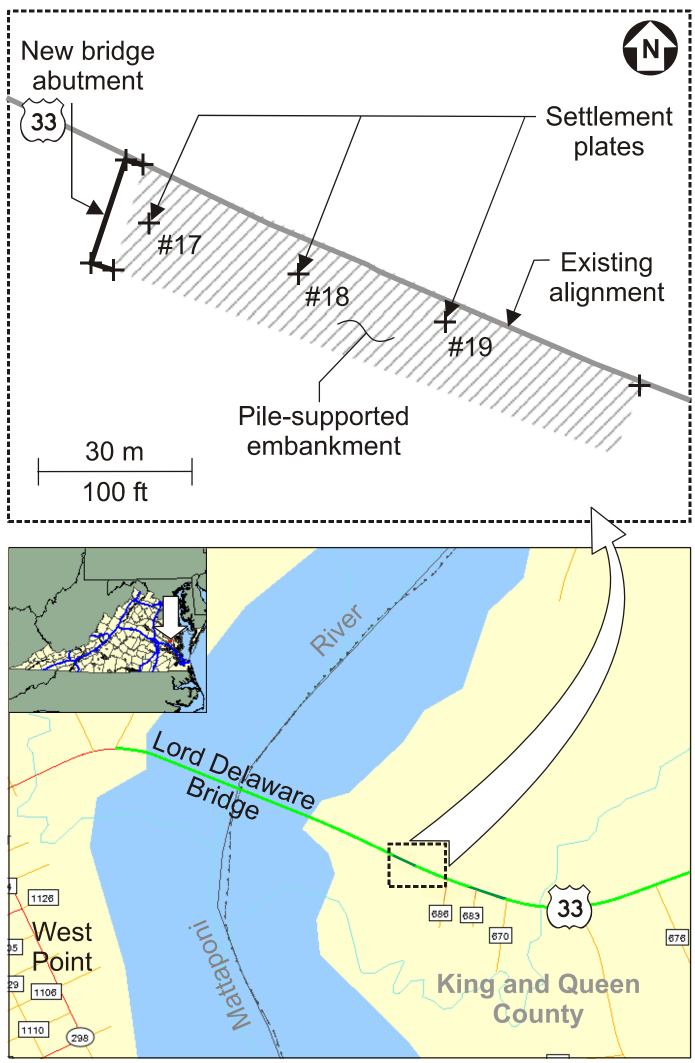



**Figure 1. Site Location and Layout**

The following subsurface conditions reflecting the marshy area spanned by the pile-supported embankment were identified in the Parsons Brinckerhoff report:[1]

*Stratum 1.* Stratum 1 soils, consisting of unconsolidated fills of silty sand (SM), extend to a depth of approximately 18 ft (5.6 m). Standard penetration test (SPT) N values range from 5 to 9.

*Stratum 2A.* Stratum 2A soils consist of normally consolidated marine deposits of dark gray organic clay (OH). Zones of peat are occasionally encountered. This layer extends to a depth of about 33 ft (10.3 m). SPT N values range from 2 to 4.

*Stratum 5.* Stratum 5 marine deposits consist of grayish green clay (CH) with traces of fine sand. This layer was encountered through the end of boring at a depth of 101.5 ft (31.6 m). Appreciable increase in the SPT N values was detected at a depth of approximately 56 ft (17.4 m). SPT N values range from 22 to 37 in the depth interval of 56 ft (17.4 m) to the end of borehole. Stratum 5 soils belong to the Calvert formation.

Groundwater was encountered in Borehole BP-12 at a depth of 4 ft (1.2 m), corresponding to approximately Elevation 0, 1 day following drilling.

## Material Properties

The Parsons Brinckerhoff report recommended the following average material properties, reflecting soil strata encountered beneath the pile-embankment site:

*Stratum 1:* Angle of internal friction of 28 degrees, effective buoyant unit weight of 55 pcf (8.6 $kN/m^3$), and the saturated unit weight of 110 pcf (17.3 $kN/m^3$).

*Stratum 2A:* Undrained shear strength of 300 psf (14 kPa), effective overburden pressure ratio of 0.3, effective buoyant unit weight of 30 pcf (4.7 $kN/m^3$), saturated unit weight of 90 pcf (14.1 $kN/m^3$), compression ratio of 0.30, recompression ratio of 0.05, and coefficient of consolidation of 0.03 $ft^2$/day.

*Stratum 5:* Angle of internal friction of 36 degrees, buoyant unit weight of 60 pcf (9.4 $kN/m^3$), saturated unit weight of 125 pcf (19.6 $kN/m^3$), and undrained shear strength of 4 to 8 ksf (190 to 380 kPa).

## Design and Construction of Pile-Supported Embankment

The embankment design called for square 12 by 12 in (0.31 by 0.31m) precast, prestressed concrete piles, driven at 7 ft (2.1 m) on centers, topped with 36 by 36 by18 in (0.91 by 0.91 by 0.46 m) pile caps. It was recommended that piles be driven to an allowable capacity of 60 tons. No bitumen coating of piles was specified. Precast concrete pile caps were rested on top of the piles (6 in [0.15 m] recess), without any bonding, because no uplift was anticipated.

The reinforcing fabric design was based on the method recommended in British Standard BS 8006. Four layers of geogrid (the upper two placed only in the side slopes) with a minimum strength of 18.5 kips per ft (270 kN/m) were initially specified for placement over pile caps, with 6 in (0.15 m) of fill separating each layer and pile cap. Ultimately, Huesker Comtrac 200.200 polyester woven geotextile, as shown in Figure 2, was used in construction. Its physical properties are listed in Table 1. Geotechnical estimates indicated a strain of 2 percent or less at all locations in high-strength geotextile materials.[2]

Plan and elevation views of the pile-supported embankment at the approach to the Mattaponi River Bridge are shown in Figure 3. The height of embankment at that point is approximately 6 ft (1.8 m). The roadway width is 68 ft (21 m). The total length of the pile-supported embankment is approximately 400 ft (120 m).

**Figure 2. High-Strength Reinforcement Geotextile**

**Table 1. Geotextile Properties**

| Property | Test Method | English Units | SI Units |
|---|---|---|---|
| Mass/Unit Area | ASTM D-5261 | 22 oz/yd$^2$ | 750 g/m$^2$ |
| Tensile Strength | | | |
| Machine Direction | ASTM D-4595 | 1142 lb/in | 200 kN/m |
| Cross Machine Direction | ASTM D-4595 | 1142 lb/in | 200 kN/m |
| Tensile Strength at 5% | ASTM D-4595 | 514 lb/in | 90.0 kN/m |
| Elongation at Break | ASTM D-4595 | 10% | 10% |
| Apparent Opening Size | ASTM D-4751 | No. 40 U.S. sieve | 0.425 mm |
| Long-Term Design Strength | | | |
| Sand | GRI – GT7 | 490 lb/in | 86 kN/m |
| Gravel | GRI – GT7 | 425 lb/in | 75 kN/m |

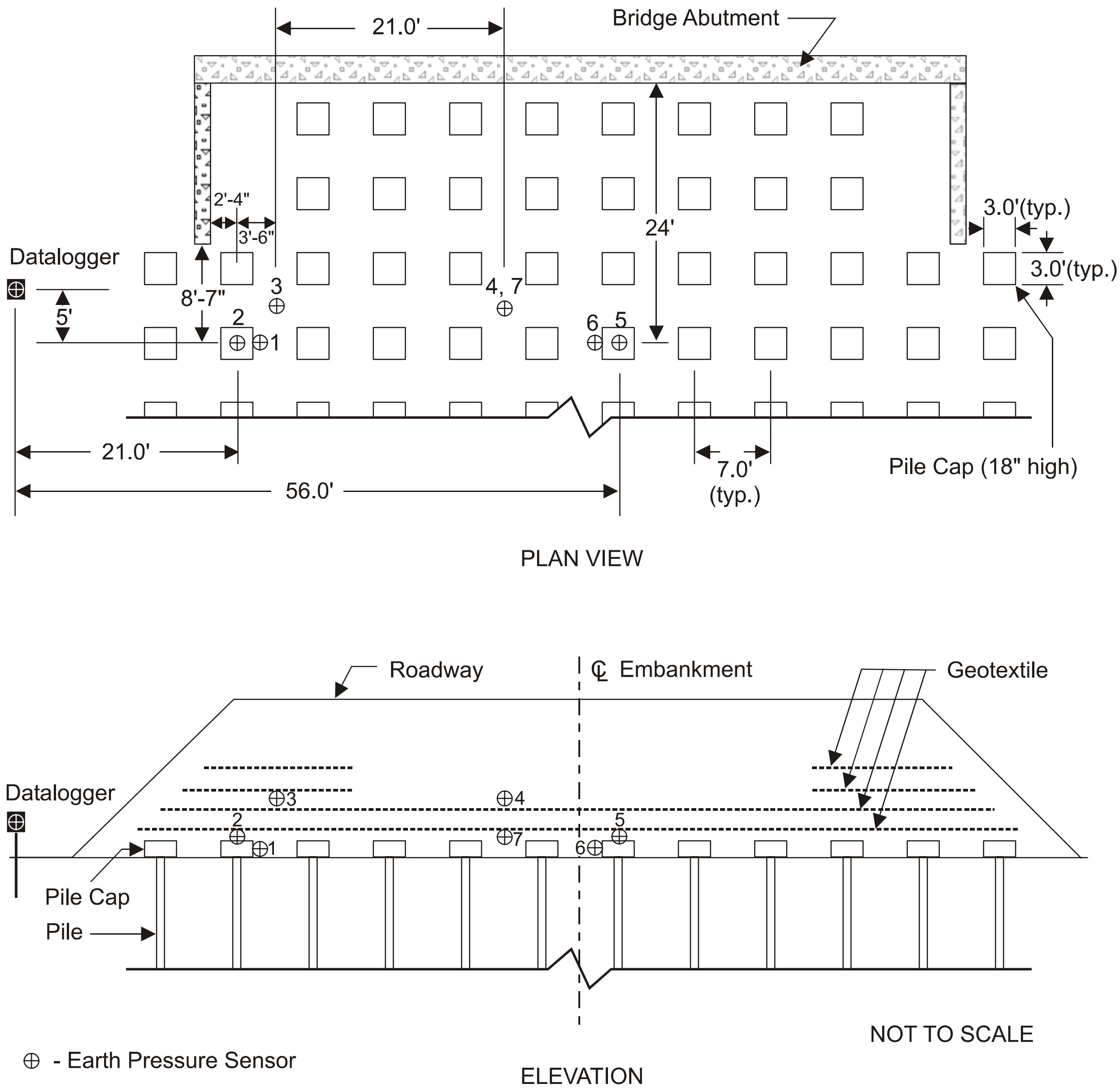



**Figure 3. Pile-Supported Embankment Plan and Elevation Views**

One layer of a lower strength geotextile (Huesker PES 200/100CC) was placed at the initial work platform elevation to provide sufficient stability for the construction equipment. The contractor had to cut the geotextile at all pile installation points prior to driving. The pile driving process begun on January 4, 2005, and ended on March 9, 2005. Piles in the vicinity of the bridge abutment were driven to a depth of 70 ft (22 m). Pile lengths ranged from 56 ft (17 m) to 46 ft (14 m) further away from the bridge abutment. The embankment fill placement began in May 2005. Two layers of high-strength geotextile (Huesker 200.200) extending across the entire embankment were doubled sheets. The contractor did not report any problems with pile installation and subsequent embankment construction.[3]

## Field Instrumentation

Figure 2 also shows the layout of the field instrumentation implemented in this study. VTRC personnel decided to measure earth pressures at various points in the embankment using seven pressure cells interfaced with an electronic datalogger. Vibrating wire earth pressure cells, Model 4800, manufactured by Geokon were installed as shown in Figure 2. They were connected to the Campbell Scientific CR-10X datalogger, sampling once an hour. Geokon pressure cells were constructed from two circular stainless steel plates, 9 in (230 mm) in diameter, welded along the periphery and separated by a narrow gap filled with hydraulic fluid. A pressure transducer was used to convert fluid pressure into an electrical signal that was subsequently recorded by the datalogger.

Lateral pressure sensors 1 and 6 were installed on the sides of pile caps at the edge of the embankment and at the center, respectively. Pressure sensors 2 and 5 were installed at the corresponding tops of pile caps to measure vertical stresses. Pressure sensors 4 and 7 were installed at the midpoint between pile caps to measure vertical stresses on top and below the geotextile layers, respectively. Pressure sensor 3 was installed at the midpoint area close to the edge of the embankment and at approximately the same level as sensor 4.

In addition to earth pressure cells, one elevation sensor was installed over sensor 4. The purpose of this elevation sensor was to measure differential settlement between the pile cap and the geotextile at the midpoint location. The sensor was constructed using a water-filled polyethylene tubing, 0.25 in (6.3 mm) in diameter, connected to an Omega PX820 pressure transducer. A small air hole was provided at the uppermost part of the tubing loop to allow the gage to register the hydrostatic pressure of the water column. The elevation sensor was calibrated in the laboratory to correlate the pressure of the water column with the elevation change. The transducer (settling part) was attached to a circular aluminum plate 10.5 in (0.267 m) in diameter and 0.125 in (3.2 mm) thick. The elevation sensor was not interfaced with the CR-10X datalogger. It was sampled manually during site visits. The sensor required a battery to supply the excitation voltage and a precision voltmeter to read the output.

Pressure cells 1, 2, 5, 6, and 7 were installed on March 22, 2005. The remaining pressure cells and the settlement sensor were placed on May 2, 2005.

Embankment settlement plates 17, 18, and 19, as shown in Figure 1, were installed and maintained by the contractor, as per the construction drawings. These plates were not a part of the VTRC field monitoring plan. Elevation data collected from the settlement plates were supplied to the VTRC personnel by the contractor.

# RESULTS

VTRC personnel conducted numerous site visits to collect sensor data and measure ground elevations at various stages during the embankment construction. Figures 4 through 9 show the progress of work and instrumentation close-ups.

Figure 4.  Pile Installation

Figure 5.  Installed Pile Caps

**Figure 6. Pressure Cells 5 and 7 (Foreground)**

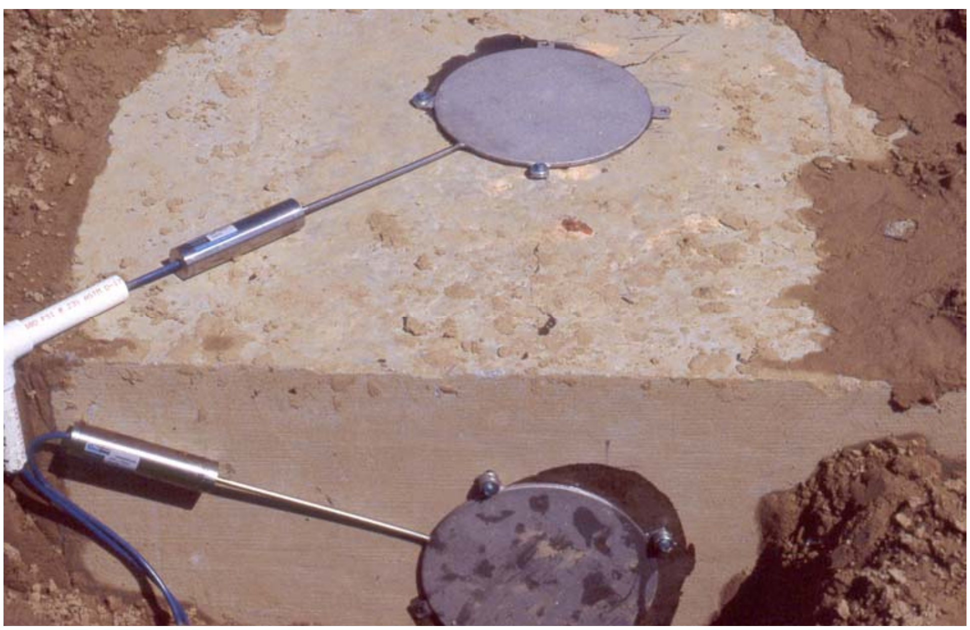


**Figure 7. Pressure Cells 5 (Top of Cap) and 6 (Side of Cap)**

**Figure 8. Reinforcing Fabric and Settlement Pipes**

**Figure 9. Paved Roadway**

Samples of the embankment fill material were collected, and grain size distribution analysis was performed in the laboratory. The results are shown in Tables 2 and 3.

Data collected from earth pressure cells are presented in Figures 10 through 12. The embankment served as an access route for the bridge project, with numerous vehicles and equipment traversing or stationed on top of the embankment. This may explain a number of recorded pressure spikes. Some distinct construction events that correlate with the recorded earth pressure responses were as follows:

- *7/21/2005 to 7/22/2005 and from 7/27/2005 to 7/28/2005*. Large-capacity cranes (120 tons) were stationed in the vicinity of sensors 4 and 7. These cranes were used for the bridge construction.

- *8/2/2005*. Large pile of loose crushed stone (approximately 5 ft [1.5 m] high) was placed in the vicinity of sensors 4, 7, 5, and 6.

- *9/14/2005*. Crushed stone bedding for the approach slab was placed near the bridge backwall.

- *9/26/2005*. Dump trucks were delivering fill for the pile-supported embankment.

- *9/27/2005*. Large, wheeled crane was stationed on the embankment centerline, close to the bridge abutment.

- *10/19/2005*. Sleeper slab was cast (18 in [0.46 m] stem width, 15 in [0.38 m] base height, 5.5 ft [1.68 m] base width). Concrete trucks were traversing above the sensor locations. The sleeper slab is located over sensors 3, 4, and 7.

- *10/21/2005*. Approach slab was cast (20 ft [6 m] long, 15 in [0.38 m] high).

- *3/8/2006*. Cement-treated aggregate (CTA) layer (6 in [0.152 m] thick) was placed on the embankment.

- *3/16/2006*. Base (5 in [0.127 m] and intermediate layers (2.5 in [0.064 m] thick) were placed. The remaining 1.5-in (0.381-m) layer of asphalt surface mix is scheduled for July 2006.

**Table 2. Grain Size Distribution Of Fill Material At The Pile Cap Elevation** (% Finer Than)

| | | |
|---|---|---|
| 50.8 mm | [2 in] | 100% |
| 25.4 | [1 in] | 100 |
| 19.05 | [3/4 in] | 100% |
| 9.525 | [3/8 in] | 99.5 |
| 4.76 | [No. 4] | 98.0 |
| 2.00 | [No. 10] | 94.1 |
| 0.42 | [No. 40] | 64.0 |
| 0.074 | [No. 200] | 5.8 |

**Table 3. Grain Size Distribution of Fill Material Covering First Geotextile Layer (% Finer Than)**

| | | |
|---|---|---|
| 25.4 mm | [1 in] | 100% |
| 19.05 | [3/4 in] | 97.8 |
| 9.525 | [3/8 in] | 96.7 |
| 4.76 | [No. 4] | 94.0 |
| 2.00 | [No. 10] | 87.7 |
| 0.42 | [No. 40] | 37.2 |
| 0.074 | [No. 200] | 2.4 |

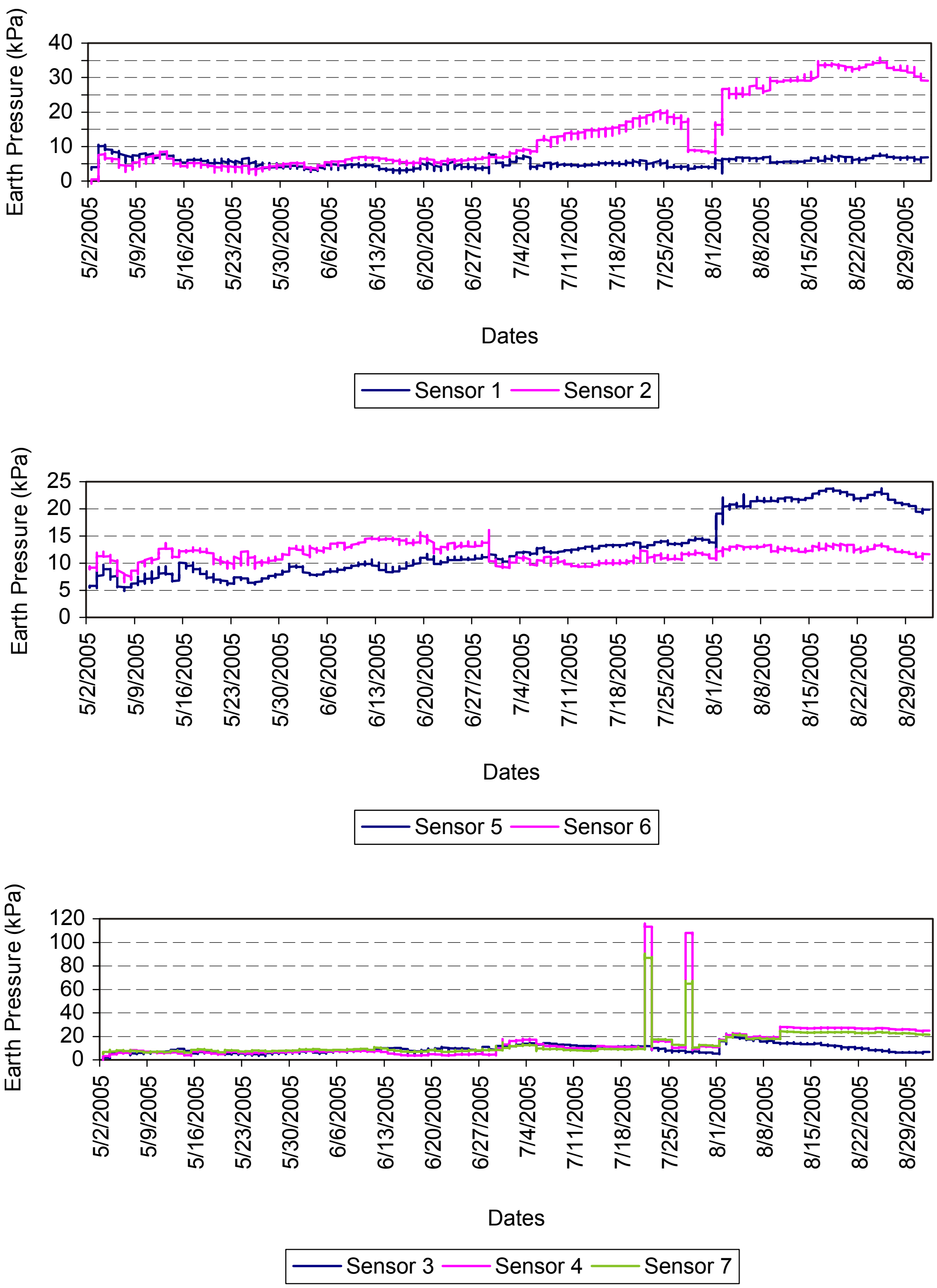



**Figure 10. Earth Pressures Recorded from 5/2/05 to 8/31/05**

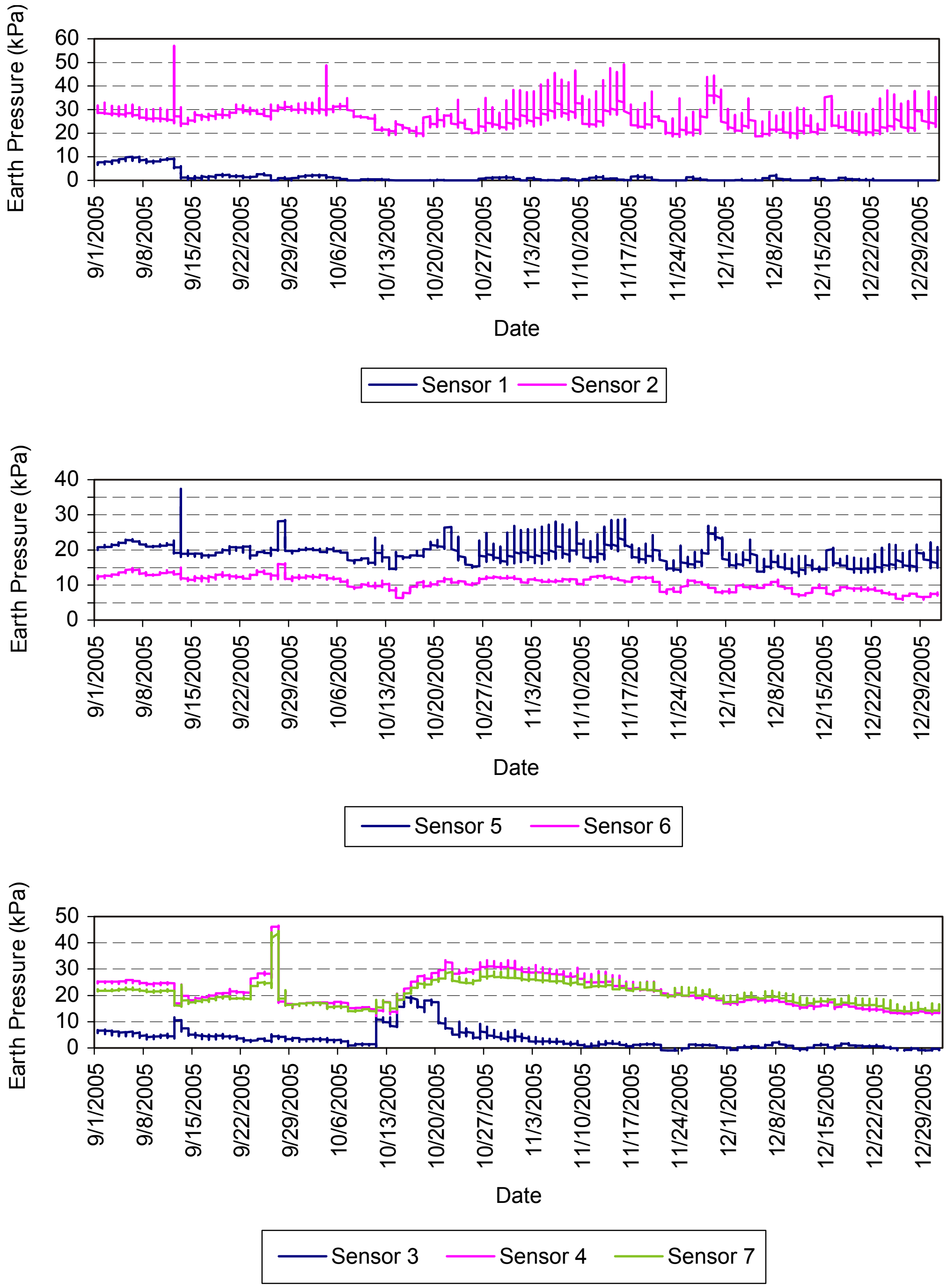



**Figure 11. Earth Pressures Recorded from 9/1/05 to 12/31/05**

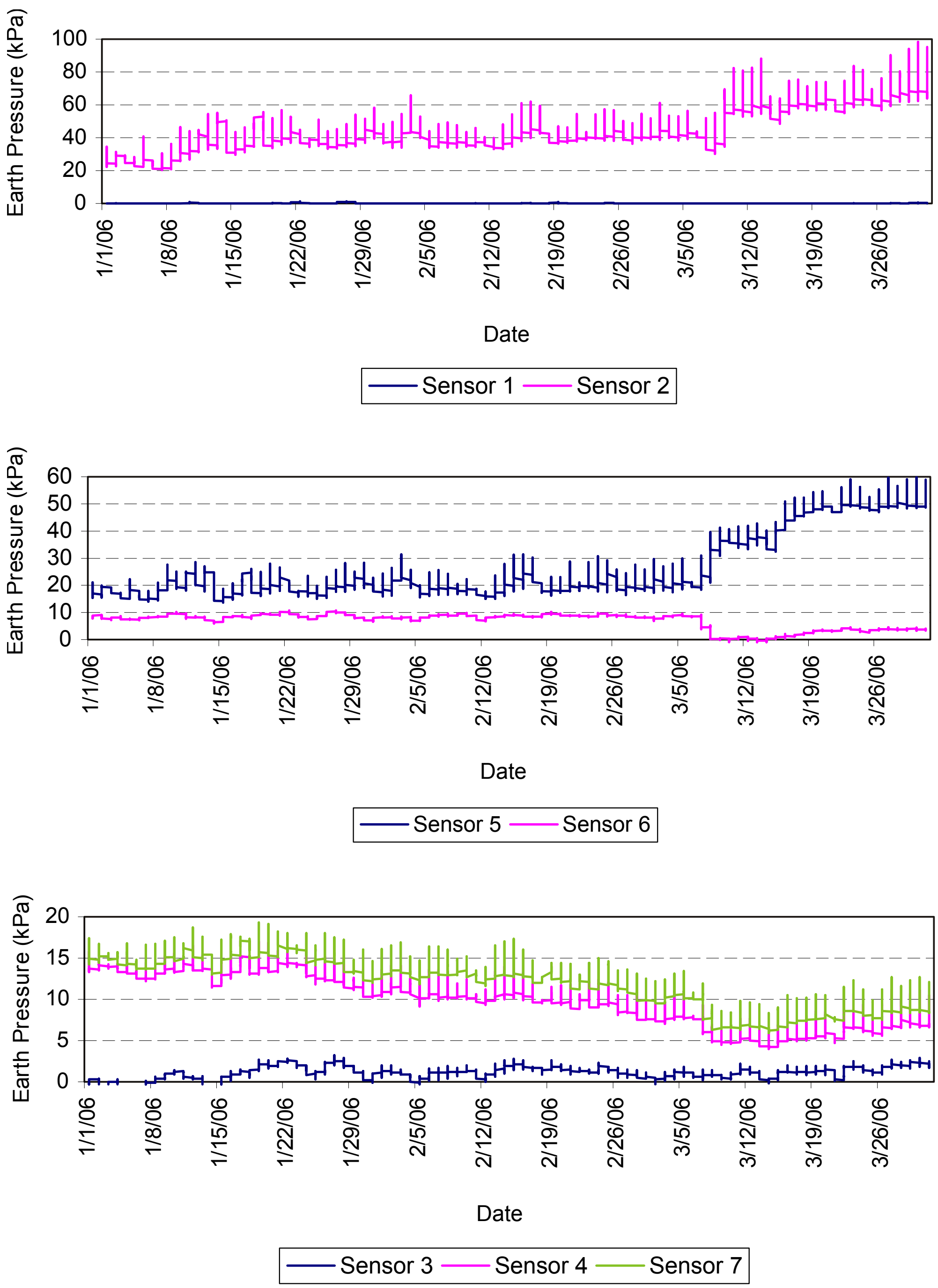



**Figure 12. Earth Pressures Recorded from 1/1/06 to 3/31/06**

Table 4 shows the heights of embankment fill and pavement layers over various sensors as of 3/21/2006, approximately 324 days after the start of embankment construction.

**Table 4. Fill Heights as of 3/21/2006**

| Sensor | Height (ft) | Height (m) | Comments |
|---|---|---|---|
| 1 | 5.8 | 1.77 | Side of cap, embankment edge |
| 2 | 5.0 | 1.52 | Top of cap, embankment edge |
| 3 | 4.3 | 1.31 | Between caps, under sleeper slab |
| 4 | 4.6 | 1.40 | Between caps, under sleeper slab |
| 5 | 5.5 | 1.68 | Top of cap, embankment center |
| 6 | 6.3 | 1.92 | Side of cap, embankment edge |
| 7 | 5.4 | 1.65 | Between caps, under sleeper slab |

Table 5 shows settlement sensor readings recorded during embankment construction.

**Table 5. Observed Settlements over Sensor 4**

| Date | Settlement (in) | Settlement (mm) | Comments |
|---|---|---|---|
| 5/2/2005 | 0 | 0 | Installation |
| 5/25/2005 | 0.67 | 17 | |
| 6/23/2005 | 1.50 | 38 | |
| 8/18/2005 | 2.45 | 62 | Pile of stone |
| 9/27/2005 | 1.78 | 45 | |
| 11/30/2005 | 1.78 | 45 | |
| 1/10/2006 | 1.78 | 45 | |
| 3/21/2006 | 1.78 | 45 | |

Table 6 shows settlement plate readings collected by the contractor 259 days following the installation (installed on 5/9/2005, last reading taken on 1/23/2006, removed on 1/30/2006).

The settlement plates were 4 ft square (1.25 m square). They were positioned between pile caps at the locations shown in Figure 1. It is possible that the relatively large dimensions of the settlement plates as compared to the clear spacing between the pile caps might have resulted in attenuated settlement readings.

**Table 6. Observed Embankment Settlements: Contractor Data**

| Plate | Settlement (in) | Settlement (mm) |
|---|---|---|
| 17 | 0.60 | 15 |
| 18 | 0.72 | 18 |
| 19 | 0.36 | 9 |

## DISCUSSION

In many parts of the world, construction of highway embankments is challenging because of marginal subsurface soils. One potentially available ground improvement solution is to transfer the embankment load deeper, into the more competent soils, through the use of supporting piles. Major impediments of a wider adoption of a pile-supported embankment methodology include incomplete understanding of the magnitude and the interrelationship of forces and the resulting stresses acting on the structure in service. This incomplete understanding has been traditionally compensated for by fairly conservative design assumptions. A feedback from the field can be useful for optimizing design procedures, resulting in a wider use of this technique.

Pile-supported embankment technology is not new. It was first implemented in Europe in the 1960s.[4] Specifications for designing modern pile-supported embankments have been developed in the United Kingdom, Sweden, and Germany. This technology allows for very rapid construction, making it particularly attractive for widening of existing roadways. It often provides the best combination of economy, speed of construction, settlement performance, reliability, and simplicity.

Pile-supported embankments have been used with or without geosynthetic reinforcement.[5] The load imposed on pile caps may be increased by the vertical component of the tension force carried by the reinforcement. A single geosynthetic layer behaves like a tensioned membrane, whereas a multilayer system acts like a stiffened platform because of the interlock of reinforcement and the surrounding soil. Under the influence of weight, the embankment mass between pile caps tends to move downward because of the presence of the soft foundation soil. The embankment mass movement between pile caps is partially restrained by shear resistance provided by fill located over pile caps. The shear resistance reduces the pressure acting on the geosynthetic but increases the load applied to pile caps. This load transfer mechanism was termed the *soil arching effect* by Terzaghi.[6]

The degree of soil arching that develops in the field depends on the height of fill above pile caps and the magnitude of soil displacement between caps. As geosynthetic reinforcement stiffness increases, less soil arching develops and consequently less differential settlement takes place. In addition, as pile stiffness increases, more soil arching develops, thus increasing differential settlement. The maximum embankment settlement at the elevation of the pile cap is greater than that at the ground surface. At the ground surface, differential settlement is mitigated by the embankment compression because of its own weight and surcharge.

Various analytical methods have been proposed to model the soil arching behavior. Existing design approaches, including British Standard BS 8006, assume that a cavity or no support resistance exists below the geosynthetic reinforcement layer. These methods typically ignore the influence of geosynthetic stiffness and pile material elastic modulus on the degree of soil arching. Such assumptions can lead to a very conservative design when the foundation soil is strong enough to provide some embankment support.

A new analytical method developed by Filz and Smith through a VTRC-sponsored study was used to correlate field observations from this study.[7] This method uses the same approach as British Standard BS 8006 to estimate strain and tension in the geotextile, but it calculates the load acting on the geosynthetic differently. It employs one of the existing mechanistically based solutions and combines it with the consideration of the stiffness of the embankment, geosynthetic reinforcement, and existing foundation soil. All necessary calculations are programmed into an

Excel worksheet, GeogridBridge, that can be used to analyze any geosynthetic-reinforced column-supported embankments, including various types of columns. The spreadsheet setup allows a designer to modify critical variables and quickly see the resulting outputs. The spreadsheet also includes a data file for estimating material properties.

Important design variables include pile spacing, area of the cap (or area of the pile if no cap is used), and the area replacement ratio. The area replacement ratio is defined as the area of the cap divided by the area of the unit cell. The area of the unit cell is typically equal to the column spacing squared (in a square layout). Commonly accepted design guidelines are as follows:

1. Embankment height should not be smaller than the clear distance between caps.

2. The clear distance between caps should not be greater than 8 ft (2.44 m).

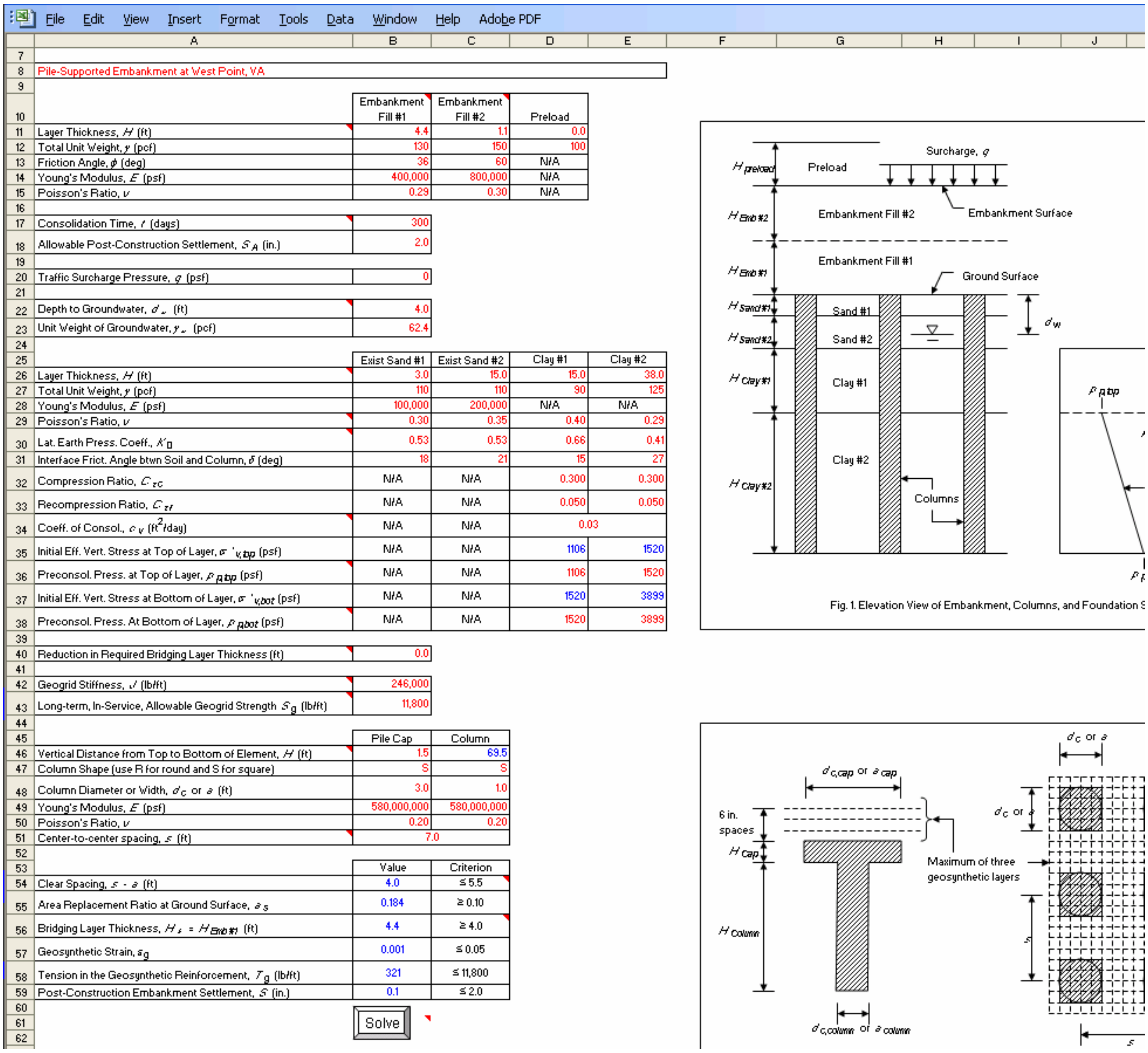
3. The area replacement ratio should not be smaller than 0.10.

Figure 13 shows a screen of input data for the West Point embankment. Estimated material properties, including strength and stiffness, were entered for the embankment fill, geosynthetic fabric, and support piles. The resulting program output is shown in Figure 14.

The GeogridBridge spreadsheet provides an estimated differential settlement of 1.10 in (28 mm) at the base of the embankment. The actual recorded settlement was 1.78 in (45 mm) at the end of construction. These values are not very far apart, considering various loads exerted on the embankment during construction, as shown on the earth pressure graphs and material property approximations. Perhaps more significant, the output indicates that the estimated resulting strain in the geosynthetic layer is very small, less than 1 percent, which is well below the 5 percent allowable limit. This condition may reflect the fact that the geosynthetic reinforcement is subjected to much less stress than anticipated during the design phase. In addition, relatively low mobilized strains may be the reason for essentially the same vertical earth pressures recorded by sensors 4 and 7, as shown in Figure 11.

The GeogridBridge spreadsheet trial calculations indicate that it may be possible to design this embankment using piles spaced at 7 ft (2.1 m) without using any reinforcing geotextile. Alternatively, it may be possible to design it with a pile spacing increased to 8.5 ft (2.6 m) and only one geotextile layer.

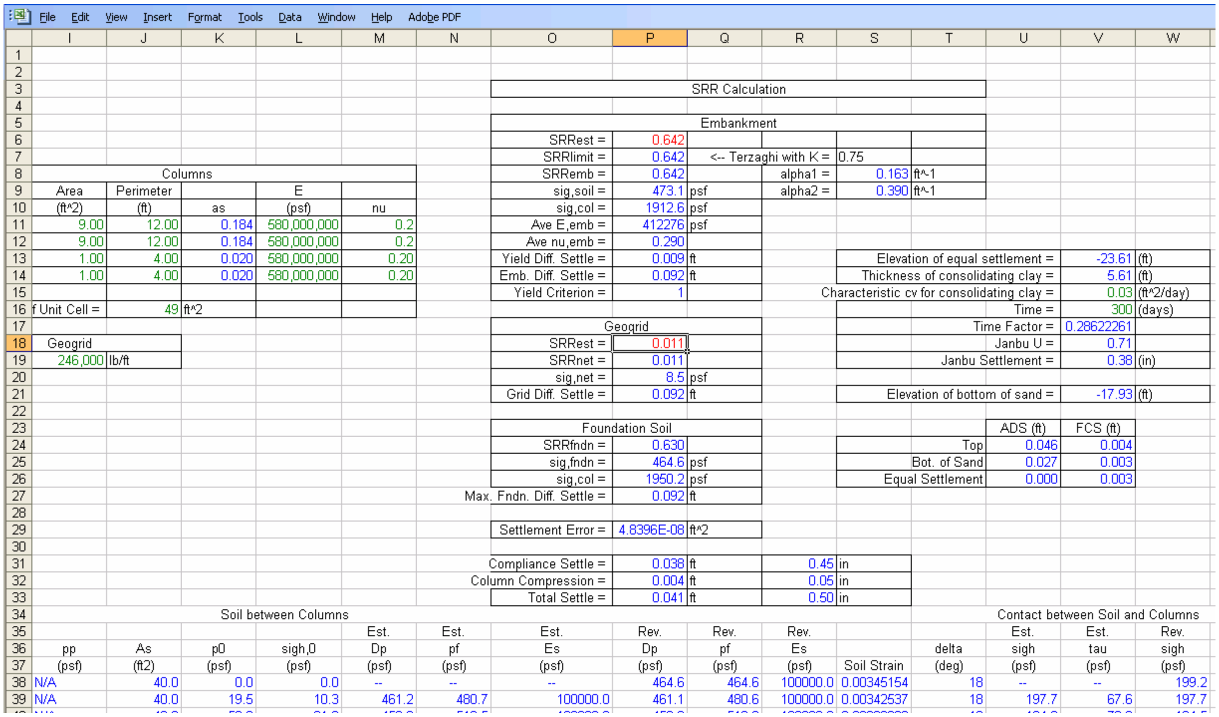
The GeogridBridge output also points out that the results (geosynthetic strain and embankment settlement) are extremely sensitive to the properties of the upper foundation soil layers. The strength and stiffness of the subgrade sand layer has a dominant effect, far exceeding the influence of the geosynthetic and embankment fill layers. It is evident that a detailed in-situ subsurface characterization is essential for a cost-effective design. One possible solution is to employ a flat plate dilatometer. The use of a dilatometer can lead to improved settlement predictions, as the Young's modulus can be derived from the dilatometer modulus.[8] More accurate settlement predictions will also result in a better estimate of the geosynthetic strain, which is mobilized as soil deformation between pile caps takes place.




| | A | B | C | D | E |
|---|---|---|---|---|---|
| 7 | | | | | |
| 8 | Pile-Supported Embankment at West Point, VA | | | | |
| 9 | | | | | |
| 10 | | Embankment Fill #1 | Embankment Fill #2 | Preload | |
| 11 | Layer Thickness, $H$ (ft) | 4.4 | 1.1 | 0.0 | |
| 12 | Total Unit Weight, $\gamma$ (pcf) | 130 | 150 | 100 | |
| 13 | Friction Angle, $\phi$ (deg) | 36 | 60 | N/A | |
| 14 | Young's Modulus, $E$ (psf) | 400,000 | 800,000 | N/A | |
| 15 | Poisson's Ratio, $\nu$ | 0.29 | 0.30 | N/A | |
| 16 | | | | | |
| 17 | Consolidation Time, $t$ (days) | 300 | | | |
| 18 | Allowable Post-Construction Settlement, $S_A$ (in.) | 2.0 | | | |
| 19 | | | | | |
| 20 | Traffic Surcharge Pressure, $q$ (psf) | 0 | | | |
| 21 | | | | | |
| 22 | Depth to Groundwater, $d_w$ (ft) | 4.0 | | | |
| 23 | Unit Weight of Groundwater, $\gamma_w$ (pcf) | 62.4 | | | |
| 24 | | | | | |
| 25 | | Exist Sand #1 | Exist Sand #2 | Clay #1 | Clay #2 |
| 26 | Layer Thickness, $H$ (ft) | 3.0 | 15.0 | 15.0 | 38.0 |
| 27 | Total Unit Weight, $\gamma$ (pcf) | 110 | 110 | 90 | 125 |
| 28 | Young's Modulus, $E$ (psf) | 100,000 | 200,000 | N/A | N/A |
| 29 | Poisson's Ratio, $\nu$ | 0.30 | 0.35 | 0.40 | 0.29 |
| 30 | Lat. Earth Press. Coeff., $K_0$ | 0.53 | 0.53 | 0.66 | 0.41 |
| 31 | Interface Frict. Angle btwn Soil and Column, $\delta$ (deg) | 18 | 21 | 15 | 27 |
| 32 | Compression Ratio, $C_{rc}$ | N/A | N/A | 0.300 | 0.300 |
| 33 | Recompression Ratio, $C_{rr}$ | N/A | N/A | 0.050 | 0.050 |
| 34 | Coeff. of Consol., $c_v$ (ft$^2$/day) | N/A | N/A | 0.03 | |
| 35 | Initial Eff. Vert. Stress at Top of Layer, $\sigma'_{v,top}$ (psf) | N/A | N/A | 1106 | 1520 |
| 36 | Preconsol. Press. at Top of Layer, $p_{p,top}$ (psf) | N/A | N/A | 1106 | 1520 |
| 37 | Initial Eff. Vert. Stress at Bottom of Layer, $\sigma'_{v,bot}$ (psf) | N/A | N/A | 1520 | 3899 |
| 38 | Preconsol. Press. At Bottom of Layer, $p_{p,bot}$ (psf) | N/A | N/A | 1520 | 3899 |
| 39 | | | | | |
| 40 | Reduction in Required Bridging Layer Thickness (ft) | 0.0 | | | |
| 41 | | | | | |
| 42 | Geogrid Stiffness, $J$ (lb/ft) | 246,000 | | | |
| 43 | Long-term, In-Service, Allowable Geogrid Strength $S_g$ (lb/ft) | 11,800 | | | |
| 44 | | | | | |
| 45 | | Pile Cap | Column | | |
| 46 | Vertical Distance from Top to Bottom of Element, $H$ (ft) | 1.5 | 69.5 | | |
| 47 | Column Shape (use R for round and S for square) | S | S | | |
| 48 | Column Diameter or Width, $d_c$ or $a$ (ft) | 3.0 | 1.0 | | |
| 49 | Young's Modulus, $E$ (psf) | 580,000,000 | 580,000,000 | | |
| 50 | Poisson's Ratio, $\nu$ | 0.20 | 0.20 | | |
| 51 | Center-to-center spacing, $s$ (ft) | 7.0 | | | |
| 52 | | | | | |
| 53 | | Value | Criterion | | |
| 54 | Clear Spacing, $s - a$ (ft) | 4.0 | ≤ 5.5 | | |
| 55 | Area Replacement Ratio at Ground Surface, $a_s$ | 0.184 | ≥ 0.10 | | |
| 56 | Bridging Layer Thickness, $H_s = H_{Emb\#1}$ (ft) | 4.4 | ≥ 4.0 | | |
| 57 | Geosynthetic Strain, $\varepsilon_g$ | 0.001 | ≤ 0.05 | | |
| 58 | Tension in the Geosynthetic Reinforcement, $T_g$ (lb/ft) | 321 | ≤ 11,800 | | |
| 59 | Post-Construction Embankment Settlement, $S$ (in.) | 0.1 | ≤ 2.0 | | |
| 60 | | | | | |
| 61 | | Solve | | | |
| 62 | | | | | |

**Figure 13. GeogridBridge Input Data**

Earth pressure data indicate a significant increase in the load transfer to pile caps at the onset of pavement layer construction, which begun on March 8, 2006. At the same time, sensors 4 and 7 (midpoint between caps) registered lower stresses. Figure 15 shows the stress concentration ratio, expressed as the stress applied on top of a pile cap divided by the stress exerted between caps. It can be seen that the stress concentration ratio increases from approximately 1.3 to 9 and then drops off to approximately 7. The stress concentration ratio is an indicator of the effectiveness of the ground improvement scheme. A high ratio means that more load is transferred to the columnar reinforcement, thus reducing the load that needs to be carried by the weak (untreated or unsupported) soil.



File Edit View Insert Format Tools Data Window Help Adobe PDF

| | I | J | K | L | M | N | O | P | Q | R | S | T | U | V | W |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | | | | |
| 3 | | | | | | | SRR Calculation | | | | | | | | |
| 4 | | | | | | | | | | | | | | | |
| 5 | | | | | | | Embankment | | | | | | | | |
| 6 | | | | | | | SRRest = | 0.642 | | | | | | | |
| 7 | | | | | | | SRRlimit = | 0.642 | <-- Terzaghi with K = | | 0.75 | | | | |
| 8 | Columns | | | | | | SRRemb = | 0.642 | | alpha1 = | 0.163 | ft^-1 | | | |
| 9 | Area | Perimeter | | E | | | sig,soil = | 473.1 | psf | alpha2 = | 0.390 | ft^-1 | | | |
| 10 | (ft^2) | (ft) | as | (psf) | nu | | sig,col = | 1912.6 | psf | | | | | | |
| 11 | 9.00 | 12.00 | 0.184 | 580,000,000 | 0.2 | | Ave E,emb = | 412276 | psf | | | | | | |
| 12 | 9.00 | 12.00 | 0.184 | 580,000,000 | 0.2 | | Ave nu,emb = | 0.290 | | | | | | | |
| 13 | 1.00 | 4.00 | 0.020 | 580,000,000 | 0.20 | | Yield Diff. Settle = | 0.009 | ft | | | | Elevation of equal settlement = | -23.61 | (ft) |
| 14 | 1.00 | 4.00 | 0.020 | 580,000,000 | 0.20 | | Emb. Diff. Settle = | 0.092 | ft | | | | Thickness of consolidating clay = | 5.61 | (ft) |
| 15 | | | | | | | Yield Criterion = | 1 | | | | | Characteristic cv for consolidating clay = | 0.03 | (ft^2/day) |
| 16 | f Unit Cell = | 49 | ft^2 | | | | | | | | | | Time = | 300 | (days) |
| 17 | | | | | | | Geogrid | | | | | | Time Factor = | 0.28622261 | |
| 18 | Geogrid | | | | | | SRRest = | 0.011 | | | | | Janbu U = | 0.71 | |
| 19 | 246,000 | lb/ft | | | | | SRRnet = | 0.011 | | | | | Janbu Settlement = | 0.38 | (in) |
| 20 | | | | | | | sig,net = | 8.5 | psf | | | | | | |
| 21 | | | | | | | Grid Diff. Settle = | 0.092 | ft | | | | Elevation of bottom of sand = | -17.93 | (ft) |
| 22 | | | | | | | | | | | | | | | |
| 23 | | | | | | | Foundation Soil | | | | | | ADS (ft) | FCS (ft) | |
| 24 | | | | | | | SRRfndn = | 0.630 | | | | Top | 0.046 | 0.004 | |
| 25 | | | | | | | sig,fndn = | 464.6 | psf | | | Bot. of Sand | 0.027 | 0.003 | |
| 26 | | | | | | | sig,col = | 1950.2 | psf | | | Equal Settlement | 0.000 | 0.003 | |
| 27 | | | | | | | Max. Fndn. Diff. Settle = | 0.092 | ft | | | | | | |
| 28 | | | | | | | | | | | | | | | |
| 29 | | | | | | | Settlement Error = | 4.8396E-08 | ft^2 | | | | | | |
| 30 | | | | | | | | | | | | | | | |
| 31 | | | | | | | Compliance Settle = | 0.038 | ft | 0.45 | in | | | | |
| 32 | | | | | | | Column Compression = | 0.004 | ft | 0.05 | in | | | | |
| 33 | | | | | | | Total Settle = | 0.041 | ft | 0.50 | in | | | | |
| 34 | | | Soil between Columns | | | | | | | | | | Contact between Soil and Columns | | |
| 35 | | | | | Est. | Est. | Est. | Rev. | Rev. | Rev. | | | Est. | Est. | Rev. |
| 36 | pp | As | p0 | sigh,0 | Dp | pf | Es | Dp | pf | Es | | delta | sigh | tau | sigh |
| 37 | (psf) | (ft2) | (psf) | (psf) | (psf) | (psf) | (psf) | (psf) | (psf) | (psf) | Soil Strain | (deg) | (psf) | (psf) | (psf) |
| 38 | N/A | 40.0 | 0.0 | 0.0 | -- | -- | -- | 464.6 | 464.6 | 100000.0 | 0.00345154 | 18 | -- | -- | 199.2 |
| 39 | N/A | 40.0 | 19.5 | 10.3 | 461.2 | 480.7 | 100000.0 | 461.1 | 480.6 | 100000.0 | 0.00342537 | 18 | 197.7 | 67.6 | 197.7 |

**Figure 14. GeogridBridge Output Data**

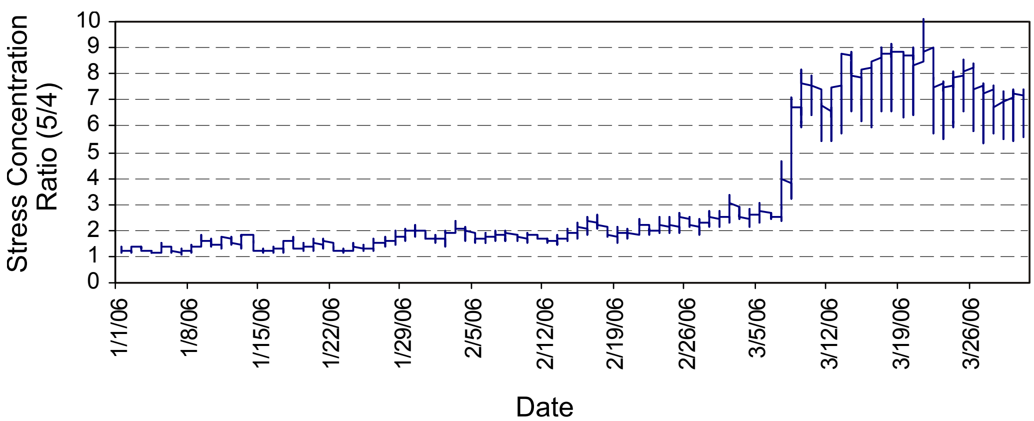



**Figure 15. Stress Concentration Ratio from 1/1/06 to 3/31/06**

Figure 16 shows the ratio of lateral to vertical earth pressures acting on the center cap. One would expect the earth pressure coefficient to reflect the at-rest state of stress on an unyielding structure with no soil arching present. The coefficient of earth pressure at rest can be estimated from the empirical relationship provided by Jaky[9]

$$K_o = 1 - \sin \phi \qquad \text{(Eq.1)}$$

where ø = angle of internal friction.

The initial earth pressure ratio oscillates around 0.4 to 0.5, reflecting essentially the at-rest state of stress, assuming ø equal to approximately 35 degrees. This ratio drops to

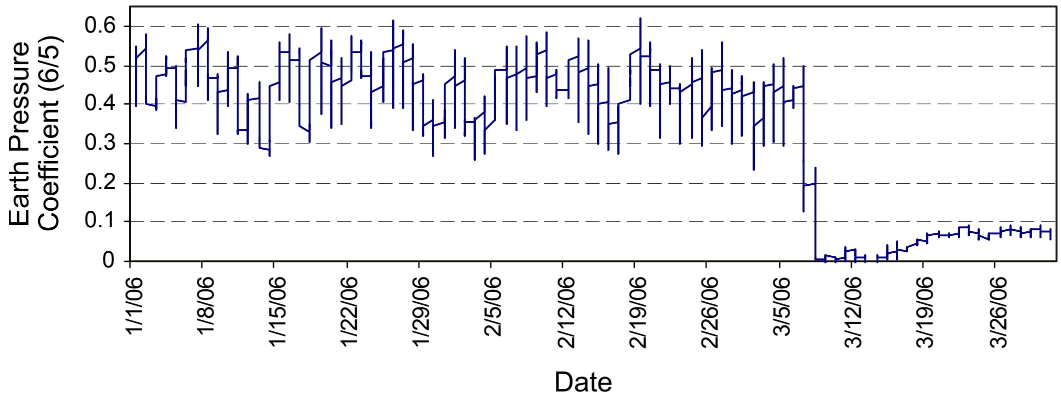


**Figure 16. Earth Pressure Coefficient from 1/1/06 to 3/31/06**

approximately 0.08 with the onset of CTA placement, indicating stress transfer to the tops of pile caps. The stress concentration and earth pressure ratios illustrate the mechanism of soil arching forming as the embankment is constructed. This arching mechanism is responsible for load transfer to the columnar reinforcement, effectively bridging the soil between columns.

Maximum stresses recorded on the top of pile caps were 70 kPa (1462 psf) and 50 kPa (1044 psf) at sensors 2 and 5, respectively. These stresses resulted in pile axial loads of 60 kN (13 kips) and 40 kN (9 kips) at the respective locations. Larger column loads observed at the edge of the embankment may be caused by a higher embankment stiffness provided by four layers of geotextile fabric. A higher material stiffness promotes greater stress transfer to the caps. This may also explain the relatively low readings observed at sensor 1 (lateral earth pressure on the edge cap) and sensor 3 (vertical stress between caps at the edge of the embankment).

If the entire embankment was carried fully by piles, the resulting pile axial load at the center of the embankment (in the vicinity of sensor 5) would be approximately 175 kN (39 kips), neglecting the weight of the pile cap. The actual load of 40 kN (9 kips) exerted on top of the pile cap indicates that the foundation soil between piles provides a substantial support. This may explain the very low computed strain mobilized in the geotextile layer.

The maximum recorded construction-induced stress was approximately 120 kPa (2506 psf) at sensor 4. If this stress had been transferred to a pile cap, the resulting column load would have been 100 kN (23 kips). Accounting for the customary safety factor of 2.25, it appears that the actual pile capacity of 60 tons is more than adequate to support the embankment loading.

## CONCLUSIONS

- *Construction of pile-supported embankments typically does not require specialty equipment or methods. No significant construction problems were reported by the general contractor working on the West Point project.*

- *A rapid increase in the bearing capacity was observed during construction.*

- *Field monitoring confirmed the mechanism of soil arching, which develops in the embankment material, between columns.*

- *Numerical analysis indicated that stiffness and strength of the upper foundation soil layers have a dominant effect on the stress transfer and embankment settlement.*

- *The cost-effectiveness of pile-supported embankments depends on accurate material characterization and rational analytical procedures.*

## RECOMMENDATIONS

1. *VDOT's Materials Division should evaluate the feasibility of pile-supported embankments at locations where the speed of construction is critical.*

2. *VDOT's Materials Division should carry out detailed subsurface investigations, including in-situ strength and deformation testing, at all sites where pile-supported embankments are considered.*

3. *VDOT's Materials Division should develop in-house geotechnical expertise in the analysis and design of pile and other columnar reinforcement for highway embankments.*

4. *VTRC should monitor the field performance of columnar reinforcement projects to provide feedback for future designs.*

## BENEFITS AND COSTS ASSESSMENT

With the use of the recently developed GeogridBridge analysis worksheet, the results indicate that it may be possible to realize substantial cost savings on projects similar in scope to the one constructed at West Point.

Potential cost savings are illustrated as follows:

*Case 1: Retain the 7-ft column spacing, but do not use any high-strength geosynthetic fabric.* The West Point project used 20,893 square yards of fabric at a cost of $97,780.
It appears that it may have been possible to construct the embankment without any geosynthetic reinforcement.

*Case 2: Increase the column spacing to 8.5 ft, and use a single layer of high-strength geosynthetic fabric.* The West Point project used 508 piles and pile caps, at a combined cost of $702,910. Increasing the pile spacing from 7 to 8.5 ft would require approximately 30 percent less piles and pile caps. Combined with the use of a single layer of high-strength geosynthetic fabric, the resulting cost savings would be about $250,000.

Significant savings can be achieved on pile-supported embankment projects because of the relatively high material costs involved in this type of construction.

## ACKNOWLEDGMENTS

The authors express their appreciation to Dr. George Filz and Mr. Michael McGuire for their guidance and helpful assistance with the GeogridBridge analysis spreadsheet.

Dr. Mohamed Elfino provided assistance with the pavement material properties. Mr. Arthur Wagner of VTRC assisted with the sensor installation, field data collection, and laboratory testing. Ms. Linda DeGrasse of VTRC processed the field data and plotted the report graphs. The authors also acknowledge Mr. Randy Combs, Mr. Ed Deasy, and Ms. Linda Evans of VTRC for their assistance with the graphics and the editorial process.

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# APPENDIX

# BOREHOLE LOG

Virginia Department of Transportation

VDOT

**PROJECT #: 24644C**
**LOCATION: West Point, Virginia**
**STRUCTURE: Route 33, Bridge**



STATION: 222+50.1 OFFSET: 8.8 ft Right
NORTHING: 442213.3 EASTING: 3778973.1
SURFACE ELEVATION: 4.7 ft

Date(s) Drilled: 8/1/02 - 8/2/02
Drilling Method(s): Mud Rotary
SPT Method: Donut Hammer
Other Test(s):
Driller: P. Clancey, Warren George, Inc.
Logger: M. Abuan

**GROUND WATER**

STABILIZED AT: 4.0 ft (0.7ft ELEV.) AFTER 24 HRS

## FIELD DATA / LAB DATA

| SAMPLE INTERVAL (ft) | STANDARD PENETRATION TEST HAMMER BLOWS | % SOIL RECOVERY | LIQUID LIMIT (LL) | PLASTICITY INDEX (PI) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|
| 0 – 1.5 | 2, 2, 4 | 46 | | | |
| 5 – 6.5 | 3, 4, 5 | 42 | | | |
| 10 – 11.5 | WOH, 2, 6 | 75 | | | |
| 15 – 16.5 | 4, 3, 2 | 82 | | | 15.8 |
| 20 – 21.5 | 2, 1, 1 | 100 | 150 | 100 | 121.2 |
| 22 – 23.5 | | 100 | 127 | 92 | 97.0 |
| 24 – 25.5 | 3, 2, 2 | 100 | | | |
| 30 – 31.5 | 2, 2, 1 | 100 | 120 | 76 | 90.2 |
| 32 – 33.5 | | 75 | 118 | 81 | 93.2 |
| 34 – 35.5 | 2, 3, 2 | 100 | 110 | 79 | 86.9 |

Depth (ft): 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40
Elevation (ft): 0, -5, -10, -15, -20, -25, -30, -35

## DESCRIPTION OF STRATA

0.0 / 4.7
Brown medium to fine SAND, some Silt **SM**

3.0 / 1.7
Orange brown coarse to fine SAND, little Silt, trace fine Gravel (FILL) **SM**

18.0 / -13.3
Dark gray organic CLAY, seams of peat **OH**

33.0 / -28.3
Greyish green CLAY, trace fine SAND, trace vegetation **CH**

**REMARKS:** RIG TYPE: CME 75 - Rubber-Tired ATV.
Stationing and Offset shown above are approximate, Eastings and Northings are from as-drilled survey.

**PAGE 1 OF 3**
**BP-12**

SPT_WESTPOINT.WESTPOINT.GPJ 7.1.039.081505 4/8/06

Virginia Department of Transportation

**PROJECT #:** 24644C
**LOCATION:** West Point, Virginia
**STRUCTURE:** Route 33, Bridge

# BP-12

**PAGE 2 OF 3**

STATION: 222+50.1 | OFFSET: 8.8 ft Right
NORTHING: 442213.3 | EASTING: 3778973.1
SURFACE ELEVATION: 4.7 ft

Date(s) Drilled: 8/1/02 - 8/2/02
Drilling Method(s): Mud Rotary
SPT Method: Donut Hammer
Other Test(s):
Driller: P. Clancey, Warren George, Inc.
Logger: M. Abuan

**GROUND WATER**

STABILIZED AT: 4.0 ft (0.7ft ELEV.) AFTER 24 HRS

FIELD DATA (SOIL: Standard Penetration Test Hammer Blows, % Soil Recovery, Sample Legend, Sample Interval; ROCK: % Core Recovery, Rock Quality Designation, DIP: Strata, Joints; Strata Legend) — LAB DATA (Liquid Limit LL, Plasticity Index PI, Moisture Content (%))

| Depth (ft) | Elevation (ft) | Standard Penetration Test Hammer Blows | % Soil Recovery | Sample Interval | Description of Strata | LL | PI | Moisture Content (%) |
|---|---|---|---|---|---|---|---|---|
| 40 | | WOH 4 1 | 29 | 40 – 41.5 | | | | |
| 42 | | | 75 | 42 – 43.5 | | 81 | 55 | 68.6 |
| 44 | -40 | 3 4 15 | 100 | 44 – 45.5 | grading to | | | |
| 50 | -45 | 3 4 5 | 100 | 50 – 51.5 | Greyish green CLAY **CH** | | | |
| 55 | -50 | 10 10 12 | 100 | 55 – 56.5 | | 102 | 72 | 64.6 |
| 58 | | | | | grading to | | | |
| 60 | -55 | 8 11 15 | 100 | 60 – 61.5 | | | | |
| 65 | -60 | 9 18 17 | 100 | 65 – 66.5 | Greyish green CLAY, trace fine Sand **CH** | | | |
| 70 | -65 | 13 15 22 | 100 | 70 – 71.5 | | 85 | 58 | 44.6 |
| 75 | -70 | 9 9 12 | 100 | 75 – 76.5 | | | | |
| 80 | -75 | | | | | | | |

**REMARKS:** RIG TYPE: CME 75 - Rubber-Tired ATV.
Stationing and Offset shown above are approximate, Eastings and Northings are from as-drilled survey.

**PAGE 2 OF 3**

**BP-12**

SPT_WESTPOINT.WESTPOINT.GPJ 7.1.039 081505 4/8/06

Virginia Department of Transportation

**PROJECT #:** 24644C
**LOCATION:** West Point, Virginia
**STRUCTURE:** Route 33, Bridge

**BP-12**
**PAGE 3 OF 3**

STATION: 222+50.1 OFFSET: 8.8 ft Right
NORTHING: 442213.3 EASTING: 3778973.1
SURFACE ELEVATION: 4.7 ft

Date(s) Drilled: 8/1/02 - 8/2/02
Drilling Method(s): Mud Rotary
SPT Method: Donut Hammer
Other Test(s):
Driller: P. Clancey, Warren George, Inc.
Logger: M. Abuan

**GROUND WATER**
STABILIZED AT: 4.0 ft (0.7ft ELEV.) AFTER 24 HRS

| DEPTH (ft) | ELEVATION (ft) | STANDARD PENETRATION TEST HAMMER BLOWS | % SOIL RECOVERY | SAMPLE INTERVAL | % CORE RECOVERY | ROCK QUALITY DESIGNATION | DIP STRATA | DIP JOINTS | DESCRIPTION OF STRATA | LIQUID LIMIT (LL) | PLASTICITY INDEX (PI) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 82 | | 12 14 22 | 100 | 80 – 81.5 | | | | | Greyish green CLAY, trace fine Sand **CH** | | | |
| 84 | -80 | | | | | | | | | | | |
| 86 | | 12 15 18 | 100 | 85 – 86.5 | | | | | | | | |
| 88 | | | | | | | | | | | | |
| 90 | -85 | 10 12 14 | 100 | 90 – 91.5 | | | | | | | | |
| 92 | | | | | | | | | | | | |
| 94 | -90 | | | | | | | | | | | |
| 96 | | 11 12 16 | 100 | 95 – 96.5 | | | | | | 148 | 127 | 91.6 |
| 98 | | | | | | | | | | | | |
| 100 | -95 | 9 10 10 | 100 | 100 – 101.5 | | | | | | | | |
| | | | | | | | | | 101.5 / -96.8 Bottom of Hole | | | |

**REMARKS:** RIG TYPE: CME 75 - Rubber-Tired ATV.
Stationing and Offset shown above are approximate, Eastings and Northings are from as-drilled survey.

**PAGE 3 OF 3**
**BP-12**

SPT_WESTPOINT.WESTPOINT.GPJ 7.1.039.081505 4/8/06