The Use of Dilatometer and In-Situ Testing to Optimize Slope Design

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ABSTRACT: Finite-element analyses can accurately model soil's response to loading conditions. However, without realistic geotechnical parameters to model the stress-strain and strength characteristics of soils, its accuracy diminishes. This paper discusses use of finite-element analyses with the computer program, PLAXIS, to evaluate long-term performance of cut slopes at the Virginia Route 288 project, near Richmond, Virginia, USA. The 9-meter high cut slopes are located near an area with a history of slope failures.
Limit-equilibrium slope stability analyses based on the conventional subsurface investigation approach using borings and overly-conservative soil parameters derived from Standard Penetration Test results and back-analyses of historical slope failures near this area, indicated that the cut slopes will be stable at a slope ratio of 5-horizontal-to-1-vertical (5H:1V). Using the finite-element analyses with soil parameters developed based on the results of dilatometer tests (DMT) and piezo-cone penetrometer tests (CPTU), the cut slopes were found to be stable at a slope ratio of 3H:1V. The slope has been observed over the past 4 years and found to be stable, with no sign of distress.

1 INTRODUCTION

The Virginia 288 PPTA (Public Private Transportation Act) project was approved for construction in December 2000, and construction started in April 2001. The project includes construction of approximately 17 miles of new highway with 23 bridges and overpasses. The project design team, led by CH2M HILL, was asked to reduce the cost of a cut slope within a segment of the project designated as "Cut C." Cut C is located along the Virginia Route 288 mainline, immediately south of the James River. Documented historical slope failures near this area of the project led to conservative slope design in Cut C. The cut slopes were originally recommended to be at a slope ratio as flat as 5H:1V, including a drainage blanket. A proposal by the contractor initiated the study presented in this paper to re-evaluate the cut slope stability. Results of this study led to a more reasonable and cost-saving design. The general location of this project is shown in Figure 1.

Figure 1. Site Location Map of the Virginia Route 288 Project

2 PROJECT GEOLOGY

The project is located in the Piedmont Physiographic Province of Central Virginia. The region is characterized by complexly folded and faulted igneous and metamorphic rocks of Late Precambrian to Paleozoic age (Wilkes, 1988) below Triassic-aged coal measures, shales, and interbedded sandstones and shales. Geologic literature for the Midlothian Quadrangle of Virginia reports that a Tertiary-aged gravelly terrace deposit is present at the cut slope location, south of the James River flood plain and north of Bernard's Creek (Goodwin, 1970). This material is composed mostly of coarse gravel, with clayey sand beds inter-layered with the gravel. The

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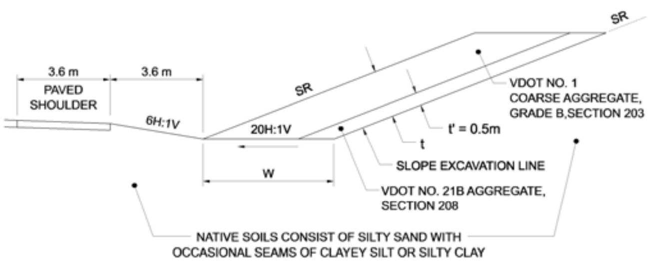
matrix of the formation is predominately sand with varying amounts of clay.

3 PROJECT DESCRIPTION

The cut slope extends approximately between Virginia Route 288 mainline stations 158+20 and 161+00 and is entirely within the limits of Cut C, which extends from station 153+00 to station 163+00. The original designer of this roadway cut slope recommended a slope ratio as flat as 5H:1V at some cuts. The design included a drainage blanket. A schematic design cross-section is presented in Figure 2.



Slope Ratio (SR) (Location Used)	Blanket Width (w) (meters)	Recommended Blanket Thickness (t) (meters)
2H:1V (Bridge)	8.0	3.5
2H:1V (Roadway)	6.0	2.7
3H:1V (Bridge)	6.0	1.9
3H:1V (Roadway)	5.5	1.7
4H:1V (Roadway)	4.0	1.0
5H:1V (Roadway)	2.5	0.5

Notes: Width (w) is horizontal dimension; Thickness (t) is perpendicular to slope.

Figure 2. Original Schematic Design Cross-Section of the Cut Slope (after HDR Engineering, Inc., 1999)

Groundwater levels in the Cut C area along Route 288, indicated by borings and monitoring wells, are summarized in Table 1. Generally, groundwater between stations 154+00 and 163+00 is observed to be near or above the finished grade. At maximum, groundwater is approximately 4 to 5 meters above the finished grade between station 155+00 and 160+00.

Table 1. Summary of Measured Groundwater Levels in Cut C Area (after HDR Engineering, Inc., 1999)

Station	Cut Depth (m)	Ground-water Elevation (m)	Ground-water Depth from Surface (m)	*Groundwater Height above Finished Cut (m)
153	2	Dry	3	-1
154	5	58	6	-1
155	8	61	3	5
156	10	62	5	5
157	8	60	4	4
158	9	60	5	4
159	8	60	3	5
160	6	59	2	4
161	4	56	1	3
162	5	54	3	2
163	2	52	3	-1

* Note that negative values indicate groundwater table below the finished cut.

Because geotechnical properties of soils are generally site-specific even within the same geological formation, in-situ testing was performed and slope stability re-evaluated upon the contractor's proposal to increase the slope ratio and avoid using a drainage blanket, to save valuable construction dollars. Based on the study presented hereafter, the cut slope is found to be stable at a slope ratio of 3H:1V.

4 IN-SITU TESTING

The in-situ testing program consisted of both dilatometer tests (DMT) and piezo-cone penetrometer tests (CPTU), which are near-continuous soil profiling techniques, to delineate subsurface stratigraphy and soil properties. The CPTU data require a good estimate of correlation coefficients to determine strength and deformation parameters. These coefficients depend on the geologic formation and can be site-specific.

The Marchetti dilatometer test is a calibrated static deformation test. The thrust to push the DMT blade, the lift-off pressure, p_0, and the pressure at full expansion, p_1, are measured. These measurements are used to compute the Marchetti indices: I_D, K_D, and E_D. These independent indices are used to compute other soil parameters through triangulation (two variables to get a third variable). Vertical constrained deformation modulus, M, was calculated using Marchetti's (1980) correlation. This modulus is obtained after combining the dilatometer modulus, E_D, with the horizontal stress index, K_D, which is an indicator of stress history, and I_D, which is a soil classification index based on its mechanical behavior. Schmertmann's (1982) method, which used the thrust measurement, for determining the drained friction angle in the cohesionless soils was used.

In this study, in-situ testing including three CPTUs, designated as PZ-1, PZ-2, and PZ-3, and four DMTs, designated as DT-1, DT-2, DT-3, and DT-4, were performed at selected locations shown in Figure 3. DT-1, DT-2, and PZ-1 are located at the top of the cut slope on the south-bound-lane (SBL) side of the highway and DT-3, DT-4, and PZ-2 are located at the bottom of the cut slope on the SBL side. PZ-3 is an additional CPTU located at the top of the cut slope on the north-bound-lane (NBL) side of the highway. At the time of testing, the slope had already been cut close to the planned finished elevation, at a slope ratio of 3H:1V, without obvious distress.

Typical CPTU and DMT results from this study are presented in Figures 4 and 5, respectively. These results were obtained at testing locations PZ-1 and DT-1, shown in Figure 3. Interpreted DMT strength and deformation parameters from testing at DT-1 are

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presented in Figures 6 and 7, respectively. Testing results consistently show that soils within the cut slope are primarily sandy soils with occasional seams of clayey silt or silty clay, which correlates well with geological literature (e.g., Goodwin, 1970).

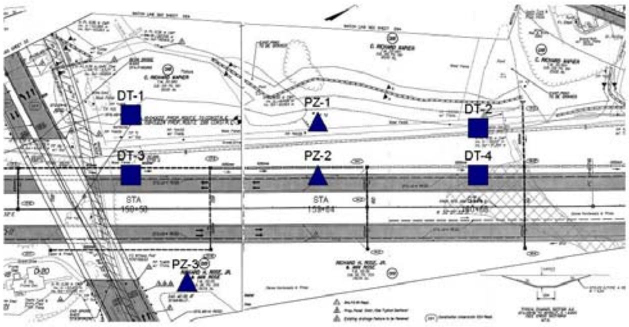

Figure 3. In-Situ Testing Locations

From the DMT results obtained at DT-1, a stiffer sandy soil layer is observed at a depth between 0 and 2 meters below the top of slope, as indicated by the higher thrust required to push the dilatometer blade and the higher M. Below a depth of 4 meters from the top of slope, the stiffness of sandy soils generally increases with increasing depth. For example, between a depth of 4 and 9 m in DT-1, constrained modulus (M) increases from 200 to 900 bars. The drained friction angle (ϕ′) of the sandy soils is generally greater than 37 degrees (ranging between 37 and 47 degrees) under the plane-strain condition. The drained friction angle under triaxial compression (ϕ'_{TC}) is averaging 38 degrees. Also, sandy soil deposits within the slope are generally overconsolidated, with an overconsolidation ratio (OCR) decreasing with increasing depth.

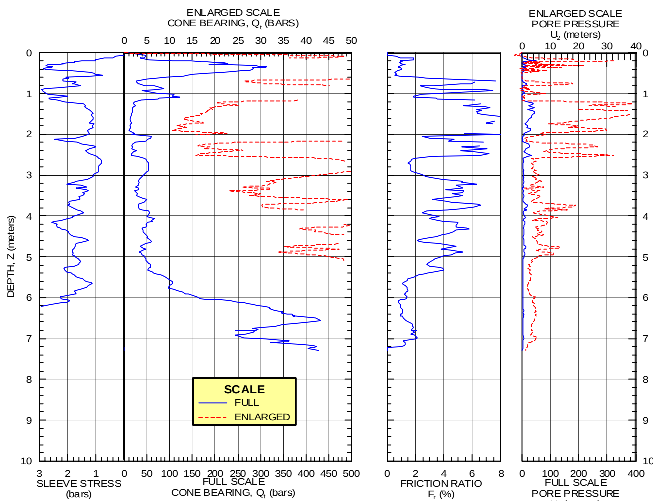

Figure 4. CPTU Results Obtained at Testing Location PZ-1

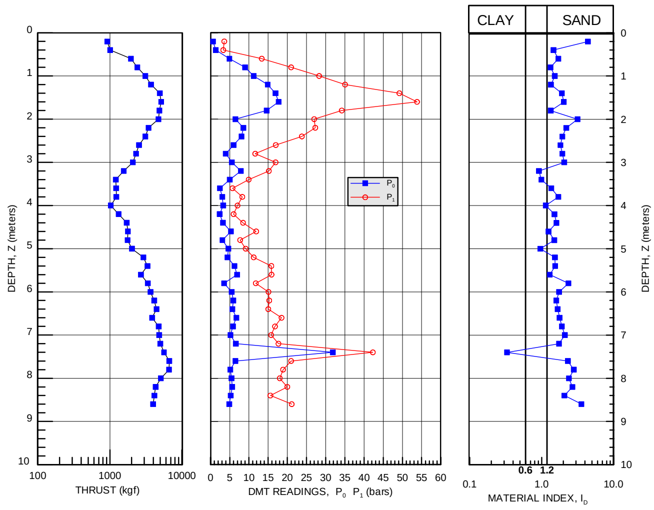

Figure 5. DMT Results Obtained at Testing Location DT-1

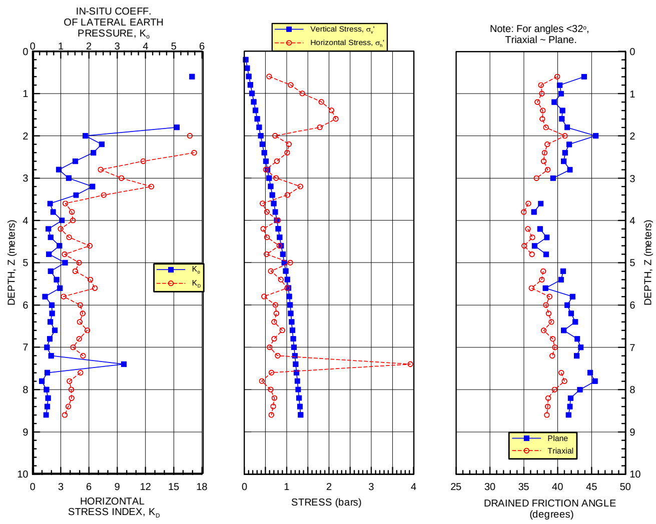

Figure 6. Interpreted DMT Strength Parameters from Testing Results Obtained at DT-1

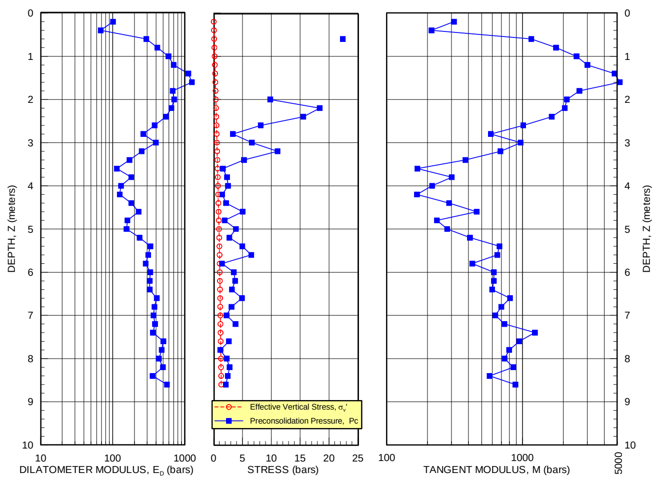

Figure 7. Interpreted DMT Deformation Parameters from Testing Results Obtained at DT-1

5 STABILITY ANALYSES

Slope stability analyses using a finite-element based computer program, PLAXIS (Brinkgreve and Vermeer, editors, 1998), were executed to evaluate the cut slope performance. A cross-section at the SBL side of Virginia Route 288 mainline station 158+20 was analyzed. This cross-section represents one of the deepest cut sections along this slope. The cut depth is approximately 9 m, with a revised slope ratio of 3H:1V. The top of the slope is at an elevation of 65 m above mean sea level (MSL) and the bottom of the slope is at an elevation of 56 m above MSL. The top of bedrock is at an approximate elevation of 50 m above MSL (4 m below the bottom of cut). A single soil type was used for soils above the rock, which is assumed as fixity in the model. This cut section was analyzed under the following groundwater conditions:

1) Normal groundwater condition, with the groundwater level at an elevation of 60 m above MSL (4 m above the bottom of cut).
2) The worst-case groundwater condition with the groundwater level at an elevation of 65 m above MSL (corresponding to a fully-saturated cut slope).

In the model, the cut was excavated in three steps. Each cut step involved removal of soil of 3-m vertical thickness in accordance with the 3H:1V slope ratio, during a 2-month period. Groundwater drawdown characteristics were modeled with the groundwater flow module in PLAXIS during each cut step, such that effective stress within the cut slope can be estimated more accurately.

Soil behavior was modeled using the hardening soil model presented in Table 2, with various strength, deformation, and groundwater flow parameters. Strength and deformation parameters were considered the most critical ones for this particular cut slope with regard to its stability, and the DMT results were used to develop these parameters. CPTU results were used to confirm that variation of soil properties within the slope profile was small and a single soil type can reasonably represent the slope behavior. Sources or correlations where these parameters were developed are presented in Table 2 and discussed hereafter.

1) Moist and Saturated Unit Weights: The moist unit weight was estimated from the DMT results, and matched up well with the data in HDR Engineering, Inc. (1999). Therefore, both moist and saturated unit weights are the same as those in HDR Engineering, Inc. (1999).
2) Strength Parameters: Drained cohesion was assumed to be zero for a sandy soil. The drained friction angle was the minimum friction angle (37 degrees) under the plane-strain condition, indicated by DMT results. The correlation between friction angle and dilatancy angle was presented by Bolton (1986). As an order of magnitude estimate, the dilatancy angle was estimated to be: $\varphi = \phi' - 30$ degrees.
3) Deformation Parameters: The oedometer modulus was assumed to be the constrained modulus at a depth of 6 m. As a result, the reference pressure is the effective horizontal stress at a depth of 6 m. An at-rest earth pressure coefficient of 0.9, indicated by the DMT results, was used to estimate the effective horizontal stress. The Young's modulus (E) can be estimated from constrained modulus (M) and Poisson's ratio (υ) by: $E = M(1+\upsilon)(1-2\upsilon)/(1-\upsilon)$. The Poisson's ratio was determined to be 0.29 from the drained friction angle under triaxial compression (ϕ'_{TC}), using the relationship presented in Kulhawy and Mayne (1990): $\upsilon = 0.1 + 0.3\ (\phi'_{TC} - 25\ \text{degrees})/(20\ \text{degrees})$. The power (m) for stress-dependent stiffness was assumed to be 0.5 for dense sand, according to Janbu (1963).
4) Hydraulic Conductivity and Void Ratio: The hydraulic conductivity for dense sand with occasional seams of clayey silt or silty clay was interpreted from the guidelines in Terzaghi et al. (1996). Anisotropy was assumed in hydraulic conductivity such that the ratio between horizontal and vertical hydraulic conductivity is 1.5. The initial void ratio was assumed to be 0.5 for a typical dense sand matrix presented in Terzaghi et al. (1996).

The ϕ-c reduction procedure in PLAXIS was performed to evaluate the stability of this cut slope. The factors of safety calculated from the ϕ-c reduction procedure under the normal and worst-case groundwater conditions are 2.2 and 1.2, respectively. Limit-equilibrium slope stability analyses were also performed to check the cut slope stability. The factors of safety calculated from limit-equilibrium analyses under normal and worst-case groundwater conditions are 1.3 and 1.1, respectively. These factors of safety are lower than the ones obtained from finite-element analyses because a horizontal straight-line phreatic surface broken by the slope was assumed in the limit-equilibrium analyses, while groundwater drawdown was modeled with assigned groundwater heads (as the boundary conditions) and hydraulic conductivity of soils in the finite-element analyses. As shown in Figure 8, groundwater drawdown in sandy soils increases the mean effective stress, and thus increases the shear strength of soils and factors of safety of the slope.

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Table 2. Soil Parameters Developed from In-Situ Testing and Used in the Finite-Element Analyses

Soil Properties	Value	Unit	Source
Moist Unit Weight, γ	18.9	kN/m^3	Estimated from DMT results.
Saturated Unit Weight, γ_{sat}	20.2	kN/m^3	HDR Engineering, Inc. (1999).
Cohesion, c'	0	kPa	Assumed for the drained condition.
Drained Friction Angle, ϕ'	37	degrees	Estimated from DMT results.
Dilatancy Angle, φ	7	degrees	Bolton (1986).
Oedometer Modulus, E_{oed}	57000	kPa	Estimated from DMT results.
Secant Young's Modulus, E_{50}	45000	kPa	Estimated based on E_{oed} and Poisson's ratio.
Power, m	0.5	-	Janbu (1963).
Reference Pressure, p^{ref}	100	kPa	Estimated from DMT results.
Horizontal Permeability, k_x	1.5E-04	cm/sec	Terzaghi, Peck, and Mesri (1996).
Vertical Permeability, k_y	1.0E-04	cm/sec	Terzaghi, Peck, and Mesri (1996).
Initial Void Ratio, e_{init}	0.5	-	Terzaghi, Peck, and Mesri (1996).

The incremental shear strain calculated from the ϕ-c reduction procedure is a good indication of the most-critical failure surface of the slope. Under the normal groundwater condition, the incremental shear strain contours are presented in Figure 9. As shown in Figure 9, the most critical failure surface is influenced by groundwater drawdown and presence of the bedrock (assumed as fixity in the model). These two factors contribute to the overall stability of this cut slope.

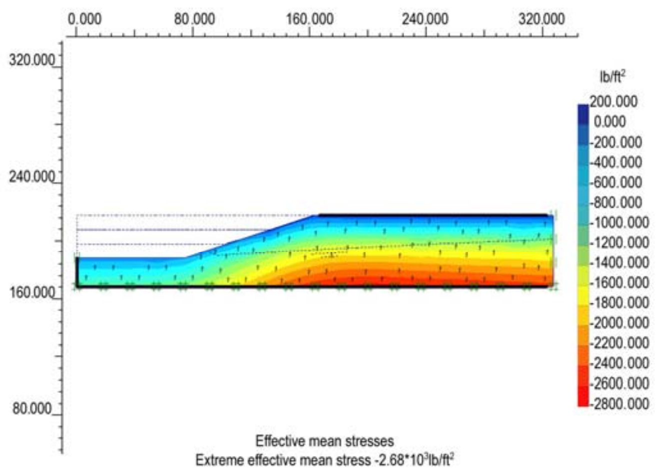

Figure 8. Influence of Groundwater Drawdown on the Mean Effective Stress within the Slope [X-axis and y-axis show PLAXIS coordinates in feet.]

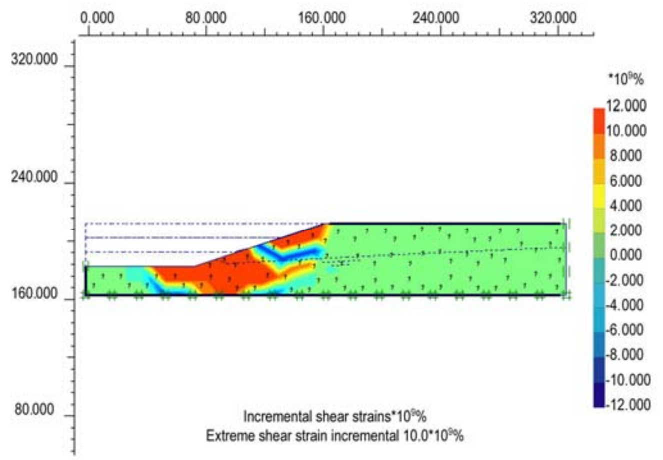

Figure 9. Incremental Shear Strain Contours Showing the Most-Critical Failure Surface of the Slope [X-axis and y-axis show PLAXIS coordinates in feet.]

As a result of the in-situ testing program and analyses using more realistic soil parameters from such testing, this cut slope was determined to be stable at a slope ratio of 3H:1V, without a drainage blanket. The saving of construction spending compared with an original 5H:1V slope with a drainage blanket, along both the NBL and SBL sides of the roadway, was approximately half a million dollars, which was significantly more than the cost of the in-situ testing program and more refined analyses.

6 CONCLUSIONS

The following conclusions can be drawn from the project described herein.

1) Geotechnical properties of soils are site-specific and, under certain circumstances, in-situ testing offers the best measure to characterize various strength and deformation parameters of soils in place. The proper selection of geotechnical properties of soils can reduce overall project cost.
2) In-situ testing is best performed by a specialist who has knowledge of the geology and soil behavior of the site, such that soil parameters can be more accurately estimated.
3) The finite-element analysis can more accurately model the state of stress, stress-dependent deformability and strength, and groundwater characteristic within an earth structure. However, such analysis requires more soil parameters than a conventional limit-equilibrium slope stability analysis. In-situ testing is considered the best way to obtain these soil parameters, especially within a sandy soil deposit where sampling and laboratory testing are more difficult and costly.

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