A case study of a translational landslide along Pennsylvania Turnpike, USA

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Abstract: A major landslide in Somerset County, Pennsylvania, USA, known as the New Baltimore landslide, has been problematic for Pennsylvania Turnpike (I-76) since its construction in 1940. The landslide extends approximately 2000 ft (606 m) upslope and 1000 ft (303 m) laterally, and moves toward the east-bound lane of I-76 at a rate of 3.5-5 in/yr (9-13 cm/yr). Overall, the landslide can be classified as a translational failure with localized rotational slides, rock falls, and flows. The geology at the site consists of the Upper Devonian Catskill Formation that includes sandstone, siltstone, mudstone, and shale units.

The New Baltimore landslide was mapped in detail to show the presence of major and secondary scarps, tension cracks, drainage channels, and depressions. A subsurface investigation was conducted by American Geotechnical & Environmental Services, Inc. of Pennsylvania. It consisted of 18 borings, ranging in depth from 39.4 ft (11.9 m) to 118.1 ft (35.8 m), the installation of 15 piezometers, 11 Time Domain Reflectometry (TDR) cables, and 3 slope inclinometers. These instruments were monitored quarterly to investigate the pore water pressures at different depths and to determine the location of the failure plane. Additionally, three transect lines were installed within the slide area to monitor any near surface movement. The core from the boreholes was logged to establish the stratigraphy and to select samples for laboratory testing. The factor of safety against sliding was determined for varying drainage conditions.

Results of the study indicate that the primary failure is located along the bedding plane at an approximate depth of 10 ft (3 m) in the toe area and 75 ft (22.7 m) near the crest of the slope. The material along the failure plane is a nondurable (slake durability index < 37%) soil-like claystone of low shear strength (cohesion = 0 psf or 0 kN/m2; friction angle = 21 degrees). The instrumentation data show that most movement and higher pore pressure occur in the months of February through May. For varying drainage conditions, the factors of safety range from 1.0 (for dry slope) to 0.8 (for maximum pore pressure measured along the failure plane). A number of stabilization alternatives were evaluated as possible remedial measures including rock dowels, drilled piles, and slope re-gradation.

Résumé: Le glissement de terrain à New Baltimore (dans le comté de Somerset, Pennsylvania, Etats-Unis), a été problématique pour le Péage de Pennsylvania (I-76) depuis sa construction en 1940. Ce glissement de terrain se déplace vers le côté de la route allant vers l'est à un taux de 3.5-5 in/an (9-13 cm/an). La géologie de l'emplacement se compose des alternances des lits de roches sédimentaires composées de sable, de limon, et d'argile, y compris des lits de schistes argileux. Les investigations récemment réalisées sont composées de 18 sondages (qui s'étendent de 39.4 ft [11.9 m] à 118.1 ft [35.8 m] de profondeur), et de l'installation de 15 piezometers, de 11 câbles de réflectométrie de domaine de temps (TDR), et de 3 inclinomètres de pente. Ces instruments ont été surveillés par trimestre pour étudier les pressions d'eau interstitielle à différentes profondeurs et pour determiner le plan d'échec (le zone ou surface de detachment) au fond du glissement. Une analyse détaillée de stabilité pour des états variables de drainage a été exécutée. Les résultats de l'étude indiquent que surface de rupture primaire est situé le long d'une couche d'argile à une profondeur approximative de 10 ft (3 m) en secteur terminal et 75 ft (22.7 m) près de la crête de la pente, donc cet éboulement est un glissement plan, sans rotation. Les données reçus des instruments prouvent que la plupart de deplacement et de pression interstitielle élevée se produisent du mois de Février à Mai. Dans les conditions variables de drainage, les facteurs de sûreté peuvent s'étendre de 1.0 (pour les pente sèches) à 0.8 (pour les pressions interstitielles maximaux). Plusieurs solutions possibles ont été évaluées pour stabiliser le glissement de terrain, comprenant des 'rock dowels', des piles forées, et le regularisation de la pente.

Keywords: landslides, shear strength, slope stability, inclinometer, pore pressure.

INTRODUCTION

The New Baltimore Landslide is located at milepost 128 along the east-bound lane of the Pennsylvania Turnpike (I-76) in Somerset County, Pennsylvania (Figure 1). It is an old (pre-1883) landslide that is approximately 1000 ft (305 m) wide and extends 2000 ft (610 m) upslope. Overall, the landslide can be classified as a translational movement with localized rock falls, rotational slides, and flows. The landslide has several distinguishing features that indicate various episodes of movement inside the mobilized mass, the most prominent being the repetitious series of scarps and slumps within the hummocky terrain that extends from the toe to the crest of the slope. The landslide has posed a problem to the Pennsylvania Turnpike ever since the completion of its construction in 1940. It continues to move toward the Interstate 76 at a rate of 3.5 to 5 inches (9 to 13 cm) per year. The active nature of the landslide has led to a relatively large unstable rock mass at the toe of the slope and heaving of the east-bound lane shoulder as shown in Figures 2 and 3, respectively. There have been numerous rock falls along the east-bound lane of I-76, like the one shown in Figure 4, that pose a constant threat to turnpike traffic.

The geology at the New Baltimore Landslide site consists of the Devonian-aged Catskill Formation that is approximately 1600 to 2000 ft (488 to 610 m) thick and comprised of alternating units of sandstone, siltstone, and claystone (Flint, 1965). The sandstone is very fine-grained to fine-grained, and locally micaceous and conglomeratic. The siltstone is thinly to thickly bedded, micaceous, and argillaceous. The claystone is commonly silty, thinly to medium bedded, and variegated (McElroy, 2001). Alternating sequences of clayey siltstone to silty claystone are very common. The colluvial and residual soils consist of sandy silts with intermixed clay and fragments of claystone, siltstone, and sandstone. The landslide is located on the western limb of the Deer Park anticline, a major structural feature of the area. The axis of the fold strikes N 35 E and dips 15 to 25 degrees towards the northwest (McElroy, 2001).

Based on historical records provided by the Pennsylvania Turnpike Commission (PTC) (PTC, 2003), the New Baltimore Landslide appears to have been active since initial construction of the turnpike in 1940. The sketches and maps of the landslide drawn by the PTC in the 1940s indicate line and grade shifts due to slope movement. Periodically, the PTC established baselines to monitor surface movements, which is indicated on the older sketches. The PTC made an attempt to remediate the slope in the 1950s by cutting an approximately 40-ft (12 m) wide bench and providing a 1H:1V back slope extending to the initial head scarp. In 1972, Geomechanics, Inc., a geotechnical firm from Pittsburgh, completed a subsurface investigation consisting of three borings within the slide mass to locate the failure plane. In 1999, American Geotechnical & Environmental Services (AGES), Inc. were hired by the PTC to determine the location of the failure plane and the rate of the movement, and suggest possible remedial measures to prevent the mass movement from reaching the highway. The study presented herein incorporates the subsurface and instrumental data collected jointly by the authors and the AGES, Inc.

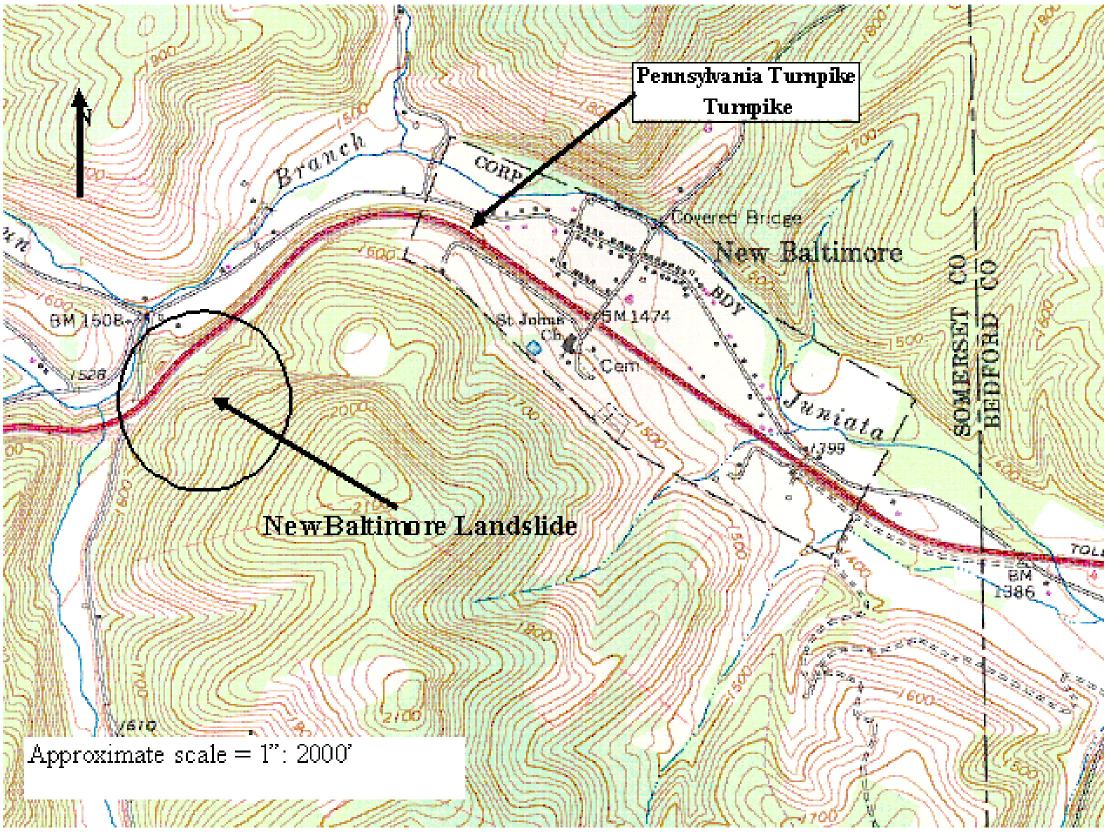


Figure 1. Location Map of the New Baltimore Landslide (USGS, 1972).

RESEARCH METHODS

Field Mapping, Subsurface Investigations, and Monitoring

The main and lateral scarps of the landslide were mapped to delineate the limits of the landslide. Also, minor scarps, tension cracks, depressions, and groundwater seeps were mapped to evaluate surface conditions. The site was visited periodically for more than one year to revise the map for the presence of groundwater seeps or newly developed tension cracks.

Detailed subsurface investigations were conducted in collaboration with the American Geotechnical and Environmental Services (AGES), Inc. as a part of investigation of the New Baltimore Landslide for the Pennsylvania Turnpike Commission (PTC). The main purpose of subsurface investigations was to sample the strata above, close to, and below the primary failure plane for laboratory testing, and to install monitoring instrumentation to determine the location of the failure plane and evaluate the direction and rate of movement. Also, there was a need to monitor the fluctuations in the ground water conditions of the slide area. Eighteen vertical test borings were drilled to depths ranging from 39 ft (12 m) at the toe and 118 ft (36 m) near the crest of the lower, more recent landslide. The test borings were aligned in two lines from toe to crest to get a full representation of stratigraphy of the lower slide area. Ten of the 18 boreholes were sampled continuously using an NX size (54 mm) core barrel.

The methods used to monitor the slope movement included the use of 3 transect lines, 3 slope inclinometers, and 11 time domain reflectometry (TDR) cables (Henderson, 2000). Fifteen vibrating wire piezometers were used to monitor the water table functions leading to the pore pressure variations (Henderson, 2000). Figure 5 shows the locations of the slope instrumentation within the slide mass. The slope instrumentation was monitored quarterly from August 1999 to May 2001. The data from slope inclinometers and the TDR cables indicated similar depths of the primary failure plane within the slide. Figure 6 shows the graphical outputs of both the TDR and slope inclinometer results at test boring 2. Tables 1 and 2 summarize the depths of the failure plane at various locations as indicated by the slope inclinometers and TDR cables, respectively. These results indicate that the primary failure plane is located within a weak, soil-like ($Id_2 \leq 37\%$) claystone unit at an approximate depth of 10 ft (3.3 m) in the toe area and 75.5 ft (23 m) near the crest of the slope. Piezometric data suggest that the higher pore water pressures occur in the late winter and early spring months. Table 3 summarizes the pore pressure readings near the failure plane.

Figure 2. Overview of unstable rock mass at the toe of the landslide along I-76.

Figure 3. An overview of pavement heaving of eastbound land of I-76.

Figure 4. A major rockfall within the toe area of New Baltimore Landslide along I-76 in July of 2001.

Laboratory Investigations

Laboratory tests were conducted on the core samples to determine dry density, percent absorption, Atterberg limits, slake durability index, unconfined compressive strength, tensile strength, and shear strength parameters. All tests were conducted in accordance with American Society for Testing and Materials (ASTM, 1996) Standards, where applicable. The purpose of laboratory tests was to characterize the materials involved in sliding and to obtain engineering property data needed for stability analysis. Two types of direct shear tests were conducted to obtain shear strength data, one involving shearing along the discontinuity and the other involving shearing through the intact rock. Competent sandstones and siltstones were failed along two saw-cut surfaces to simulate shearing along the discontinuity. The highly weathered, soil-like claystone, collected in the vicinity of the failure plane, was tested using intact cores.

Tables 4 and 5 summarize the engineering properties of various rock units. The sandstone samples, failed along the saw-cut surfaces, exhibit peak friction angle values ranging from 12 to 25 degrees and residual friction angle values ranging from 8 to 25 degrees. The peak and residual cohesion values for the sandstones range from 166 to 286 psi

(1144 to 1282 kPa) and 109 to 215 psi (751 to 1482 kPa), respectively. The siltstone samples that were failed along the saw-cut surfaces have peak and residual friction angle values of 21 to 31 degrees and 21 to 27 degrees, respectively. The peak cohesion values for the siltstone samples range from 0 to 171 psi (0 to 1179 kPa) whereas the residual cohesion is approximately zero. The siltstones with interbedded claystones have a peak and residual fiction angle value of 11 degrees, with peak and residual cohesion values of 240 and 136 psi (1655 and 938 kPa), respectively. The weaker claystone samples that were failed in the intact state exhibit peak friction angle values of 21 to 22 degrees and a residual friction angle of 11 to 21 degrees. The peak and residual values of cohesion for the intact claystones range from 86 to 186 psi (593 to 1282 kPa) and 0 to 63 psi (0 to 434 kPa), respectively. The peak values of strength parameters found in this study are in agreement with those reported by Geyer (1982).

Table 1. Depth of failure plane below the ground surface at various slope inclinometer locations.

Inclinometer	**Depth of Failure Plane Below the Ground Surface**
I-2	25 ft / 7.62 m
I-4	73 ft / 22.25 m
I-8	17 ft / 5.18 m

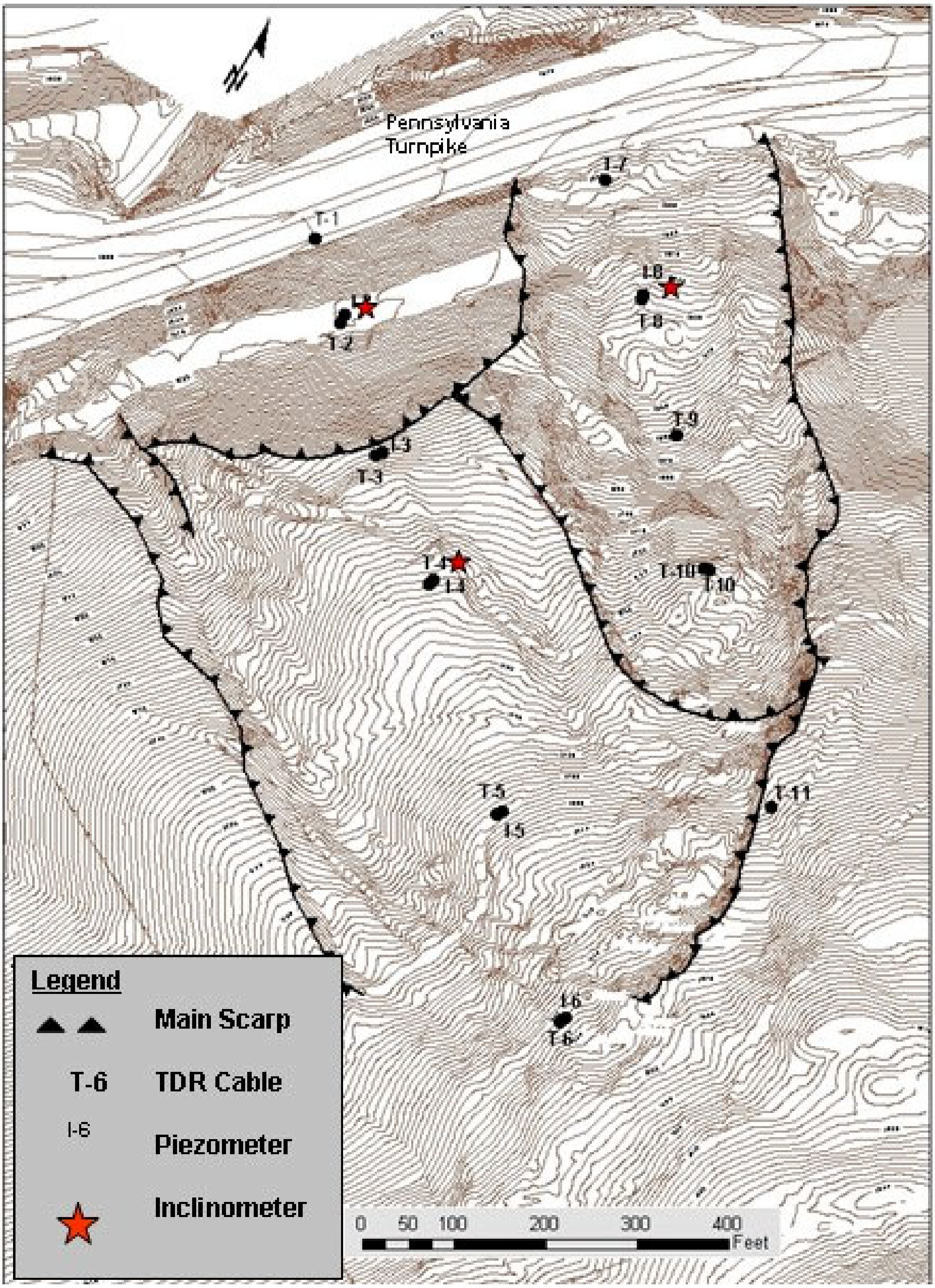
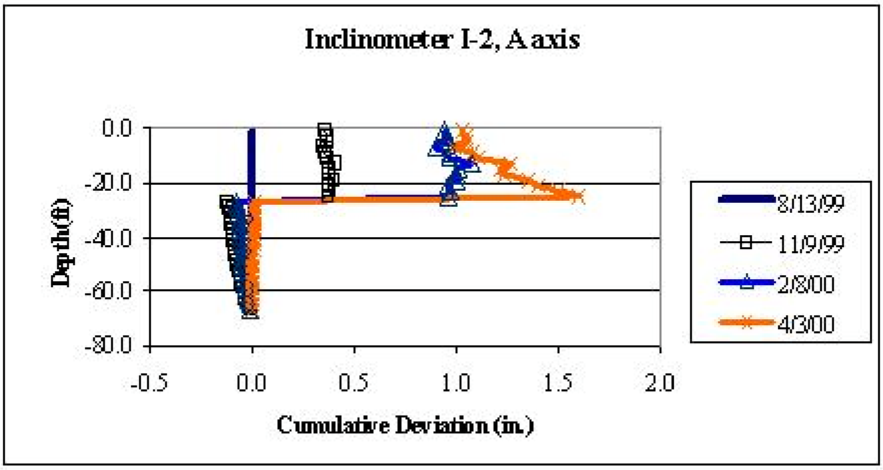


Figure 5. Location of test borings and slope instrumentation within the New Baltimore Landslide.



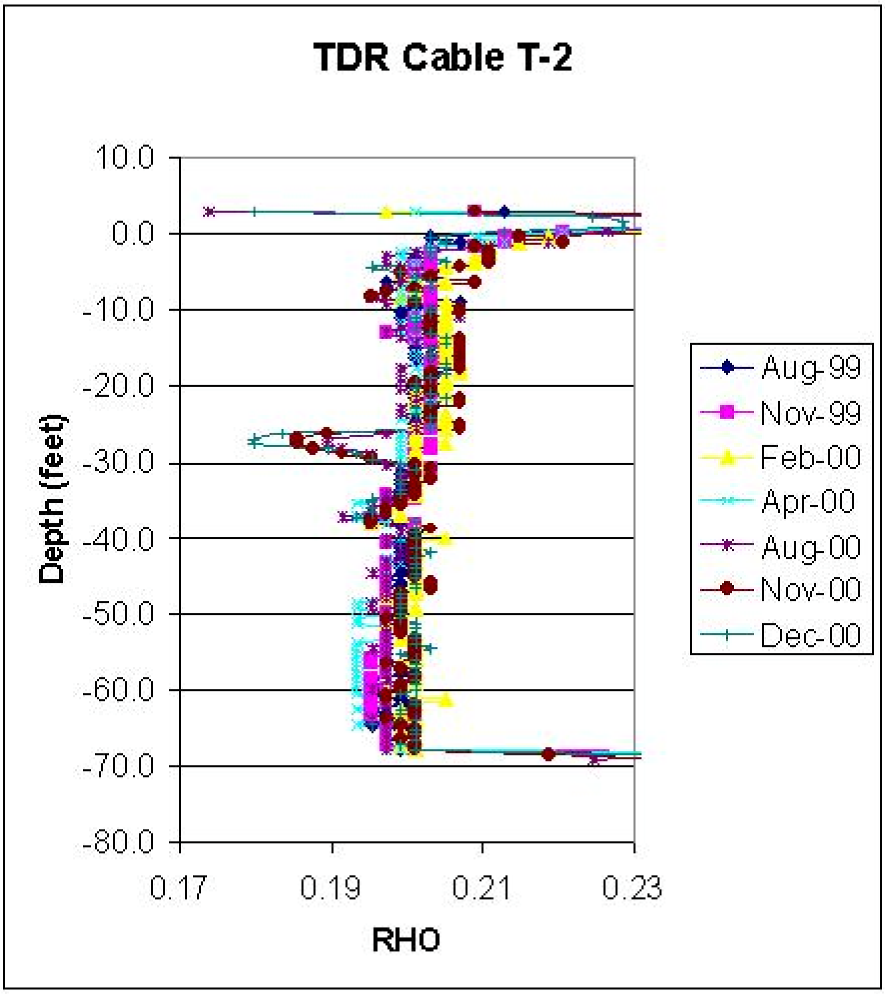


Figure 6. Slope inclinometer (top) and TDR Cable (bottom) graphs indicating similar depth of the primary failure plane in test boring T-2. Note: 1 foot = 0.303m., 1 inch = 25.4mm

Table 2. Depth of failure plane below the ground surface at various TDR cable locations.

TDR Cable	Depth of Failure Plane Below Ground Surface
T-2	26-29 ft / 7.92-8.84 m
T-3	72-75 ft / 21.95-22.86 m
T-6	43-46 ft / 13.10-14.02 m
T-8	19-22 ft / 5.79-6.71 m
T-9	9-13 ft / 2.74-3.96 m
T-10	34-38 ft / 10.36-11.58 m

Table 3. Pore water pressure data from November 1999 through May 2001.

Date	Piezometer at I-2 (28.5 feet)	Piezometer at I-4 (75.5 feet)	Piezometer at I-5 (50 feet)	Piezometer at I-6 (45 feet)
Nov 9, 1999	0.37 psi	1.60 psi	0.40 psi	1.57 psi
Feb 8, 2000	0.52 psi	2.20 psi	0.61 psi	1.62 psi
Apr 3, 2000	0.25 psi	2.70 psi	1.03 psi	1.39 psi
Aug 7, 2000	0.34 psi	2.97 psi	0.54 psi	1.47 psi
Nov 16, 2000	0.26 psi	2.59 psi	0.36 psi	1.37 psi
Feb 28, 2001	0.36 psi	3.42 psi	1.13 psi	1.45 psi
May 3, 2001	0.44 psi	2.80 psi	Possible Shear of Cable	1.53 psi

(1 foot = 0.3048m, 1psi = 6.895kPa)

STABILITY ANALYSIS

Because of the planar nature of the failure surface indicated by the borehole data, a plane failure (sliding block) analysis was conducted to analyze the stability of the slope according to the procedure outlined in Hoek and Bray (1981). The following equation was used to compute the factor of safety (F.S.) against sliding:

$$F.S. = [cA + (W\cos\psi_p - U - V\sin\psi_p)\tan\varphi)/(W\sin\psi_p + V\cos\psi_p)]$$

where:

c = cohesion
A = area of sliding
W = weight of block
ψ_p = angle of failure plane
U = uplift pressure
V = lateral pressure due to water
φ = friction angle

Table 4. Strength properties of various rock units comprising the landslide mass.

Lithology	Unconfined Compressive Strength (psi)		Tensile Strength (psi)		Shear Strength Parameters			
	Range	Average	Range	Average	$c_{Peak\ (psi)}$	φ_{Peak} (°)	$c_{Residual\ (psi)}$	$\varphi_{Residual}$ (°)
Sandstone	15,897.5 - 29,396.2	22,646.0	1,449.0 – 3,368.0	2,408.0	166 – 286	12 – 25	109 – 215	8 - 25
Siltstone	14,493.5 – 21,576.9	18,034.0	650 – 1,521.0	1,085.0	0 - 171	21 – 31	0	21 - 27
Siltstone w/ interbedded claystone	4,582.8 – 8,316.9	6,451.4	500 – 1,394	947.0	240	11	136	11
Claystone	NT	NT	NT	NT	86 - 240	21 – 22	0 – 136	11 - 21

(1psi = 6.895kPa)

Table 5. Index properties of rock units comprising the landslide mass.

Lithology	**Dry Density (pcf)**		**Absorption (%)**	**Slake Durability Index Id^2 (%)**
	Range	Average		
sandstone	166.6-174.6	170.0	0.9-1.5	63.5-92.1
Siltstone	171.7-174.8	173.2	1.9-2.2	92.7-95.9
Siltstone w/ interbedded claystone	174.4-176.6	175.5	1.3-1.9	36.9-95.7
claystone	NT	NT	NT	25.0-92.8

(1pcf = .01602 Mg/m^3)

For the purpose of analysis, the landslide was treated as one continuous block from the crest to the toe. Figure 7 shows the slope geometry for the cross-section used in the global stability analyses. The primary failure plane angle (ψ_p) was determined to be 16 degrees. The tension crack at the crest of the slope was assumed to be vertical. The cross-section shown in Figure 7 was analyzed for stability under varying drainage conditions including a dry slope and maximum pore pressure along the failure plane. The maximum pore water pressure along the failure surface was simulated by incorporating the piezometric data.

The stability analysis was conducted using different shear strength parameters. The analysis was first conducted using the residual strength parameters ($c = 0$ and $\varphi = 21$ degrees) of the weathered claystone as determined by the direct shear test. The results showed that the slope was stable (F.S.>1.0) under both dry and maximum pore pressure conditions. Since, the slope at the New Baltimore Landslide site has already failed and the slide continues to move throughout the year (under all drainage conditions), it was concluded that the friction angle for the weathered claystone used for stability analysis was high and was not representative of the field conditions. Therefore, a back analysis was conducted to determine the friction angle value for $c = 0$ and a safety factor equal to 1.0 under dry slope conditions. The back-calculated friction angle was determined to be 16 degrees. A second stability analysis was then conducted using $c = 0$ and the back-calculated value of friction angle of 16. Table 6 shows the results of stability analyses under varying drainage conditions with residual and back-calculated shear strength parameters. The back-calculated value of friction angle is in agreement with the commonly used friction angle values 12 to 16 degrees for similar claystones in southwestern Pennsylvania (Hamel, 1969).

The New Baltimore Landslide has been an active slide since the 1940's. Due to the repeated shearing that has occurred along the primary failure plane, the lowest shear strength parameters are the most representative of the field conditions. With a $c = 0$ and $\varphi = 16$ degrees, the analysis produces an unstable slope under dry and maximum pore pressure conditions, which suggests slope movement throughout the year. The small difference in the factor of safety values for the dry and maximum pore pressure conditions is attributed to the relatively small value of maximum pore pressure compared to the dry weight of the large landslide mass. This is consistent with the slope monitoring data where periodic movement was recorded during a 1.5 year period.

Table 6. Results of plane failure analyses using residual and back-calculated strength parameters.

Drainage Conditions	**F.S.**		**F.S.**	
	$\varphi = 21°$	C = 0	$\varphi = 16°$	C = 0
Dry Slope	1.3		1.0	
Max. Pore Pressure along Failure Plane	1.2		0.9	

REMEDIATION ALTERNATIVES

The New Baltimore Landslide is a very large failure that occupies more than 30 acres and poses a threat to the Pennsylvania Turnpike. Due to the nature and size of this slide, a variety of reinforcement, stabilization, and protective measures were evaluated as possible remediation alternatives. The remediation measures were analyzed to evaluate their effectiveness in stabilizing the driving forces behind the lower section of the New Baltimore Landslide. A stability analysis, considering various remediation alternatives, was conducted to obtain a factor of safety of 1.5, which is generally considered adequate for permanent slopes (Pennsylvania Department of Transportation, 1993). A cost analysis was also performed for the remediation measures considered above. Table 7 provides the results of cost analysis along with a brief description and the advantages and disadvantages of each alternative as well as the results of this analysis. As can be seen from the table, permanent remediation of the landslide will be quite expensive. The New Baltimore Landslide has been a slow moving landslide for the last 65 years. The slide has not caused any major accidents or deaths although the potential for such a hazard does exist. The slope is presently being maintained and monitored at minimal cost compared to the costs of remediation alternatives presented in Table 7. However, a well-engineered remediation is required for long term stabilization of the slide.

CONCLUSIONS

Based upon the results of this investigation, the following conclusions are drawn:

- The New Baltimore Landslide is an old, large, translational, slow moving slide that covers more than 30 acres and extends on both sides of the Pennsylvania Turnpike. Localized rotational slides, rockfalls, and flow type movements are present throughout the slide mass, indicating its complex nature
- Subsurface investigations, slope inclinometers, and time domain reflectometry (TDR) cables indicate that the primary failure plane is located along the bedding plane within a weak claystone unit at an approximate depth of 10 ft (3 m) in the toe area (along east-bound lane of I-76) and 75.5 ft (23.0 meters) near the crest of the slope. The material along the failure plane is a highly weathered ($Id_2 \leq 37\%$), soil like silty claystone (ML-CL) of low shear strength.
- The rate of movement of the slide ranges from 3.5to 5 inches (9 to 13 cm) per year as indicated by the slope inclinometers. The maximum movement and the associated higher pore pressures occur in the late winter and early spring months.
- The stability analysis, using a plane failure scenario, indicates a factor of safety of 1.0 or less for both dry and maximum pore pressure conditions.
- The New Baltimore Landslide poses a continual threat to the Pennsylvania Turnpike. The common occurrences of rock falls, pavement heaving, and potential for a catastrophic failure make this landslide a serious hazard. The Pennsylvania Turnpike Commission continues to monitor the slide and maintain the roadway. However, remedial measures would be needed to stabilize the slope and minimize the risk of a large failure.

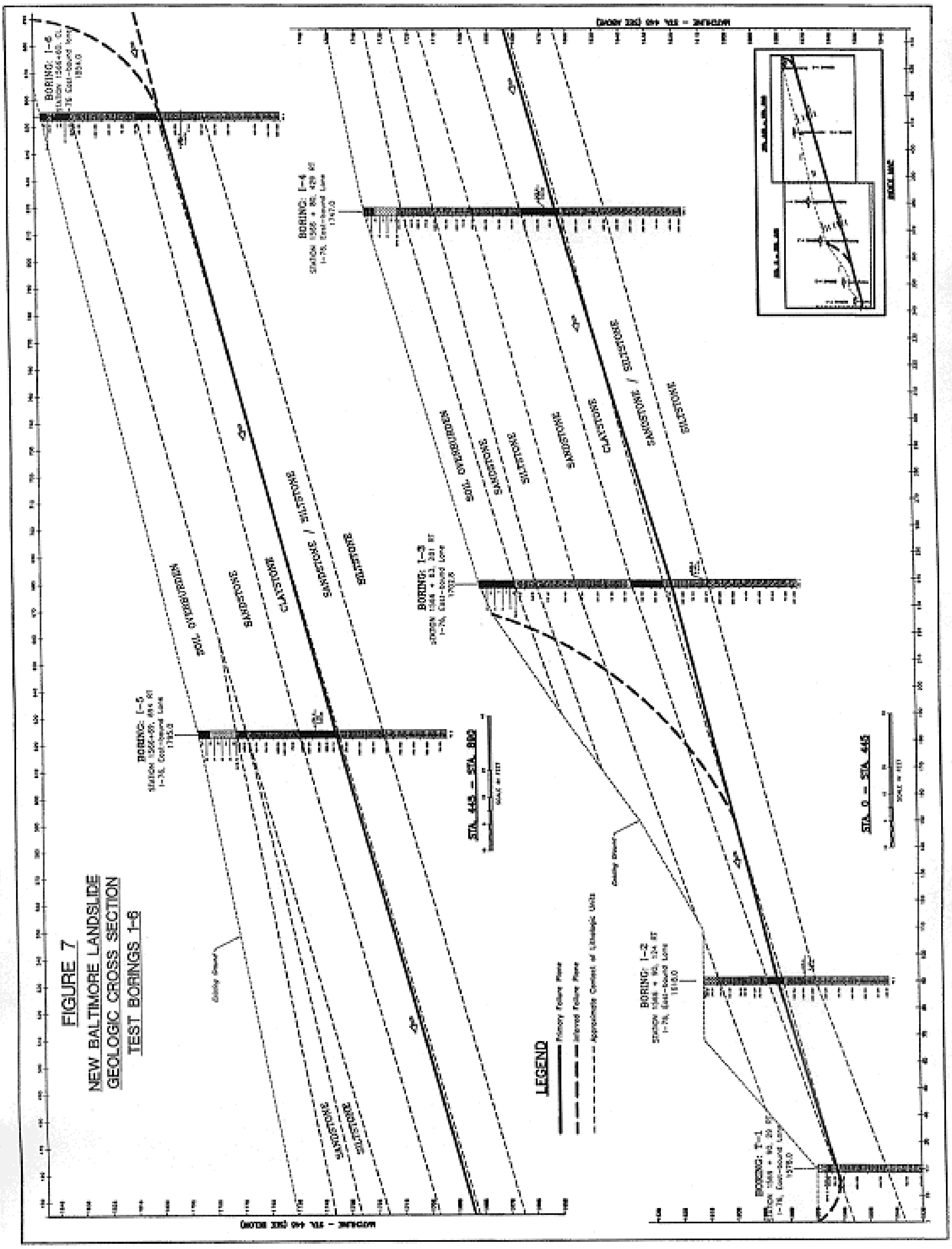


Figure 7. Cross-section of New Baltimore landslide used for stability analysis.

Table 7. Cost analysis of potential remedial measures for the New Baltimore Landslide.

Remediation Alternative	Description	Advantages	Disadvantages	Estimated Cost
Rock Dowels	2.5 in diameter Ultimate stress=150 ksi Number of dowels =2102 Spacing = 7.3 ft	Requires no additional right of way (ROW) Minimal delay of traffic Achieved design factor of safety	Requires shoulder closure	$6,077,237.24
Drilled Piles	HP 12X84 Ultimate stress= 106 ksi Number of piles = 3000 Spacing = 5.6 ft	Requires no additional ROW Minimal delay of traffic Achieved design factor of safety	Requires shoulder closure Requires a large number of piles	$11,251,600.00
Regradation of Slope to A. 3H:1V B. 3.5H:1V C. 4H:1V	A. Removal of 514,007 yd^3 of slide mass B. Removal of 855,349 yd^3 of slide mass C. Removal of 1,324,046 yd^3 of slide mass	Removes slide mass Design factor of safety not achieved Complete removal of slide including failure plane	Requires additional ROW Requires lane closure or lane shift for blasting - delay of traffic Requires a waste area	A. $5,016,177.50 B. $7,491,149.60 C. $14,288,841.70
Provision of Catchment Ditch and Protective Barrier	Cutting existing slope back to a 1H:1V Removal of 7,708 yd^3 of slide mass Installation of concrete barrier for rockfall protection	Requires no additional ROW Provides rockfall protection Provides access for rockfall cleanup Less expensive compared to other alternatives	Requires lane closure/delay of traffic Requires a waste area Requires slope Instrumentation monitoring Does not improve global stability Requires slope Instrumentation warning system	$ 114,747.50
Maintenance and Monitoring	Annual rockfall clean up and shoulder repair Install additional slope instrumentation	Allows time to address needs for further investigation Least expensive in both short and long terms	Global stability is not addressed	$ 4,268.04/ year

1 ft = 0.3048m, $1yd^3 = 0.7646m^3$, 1ksi = 6.8948Mpa.

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